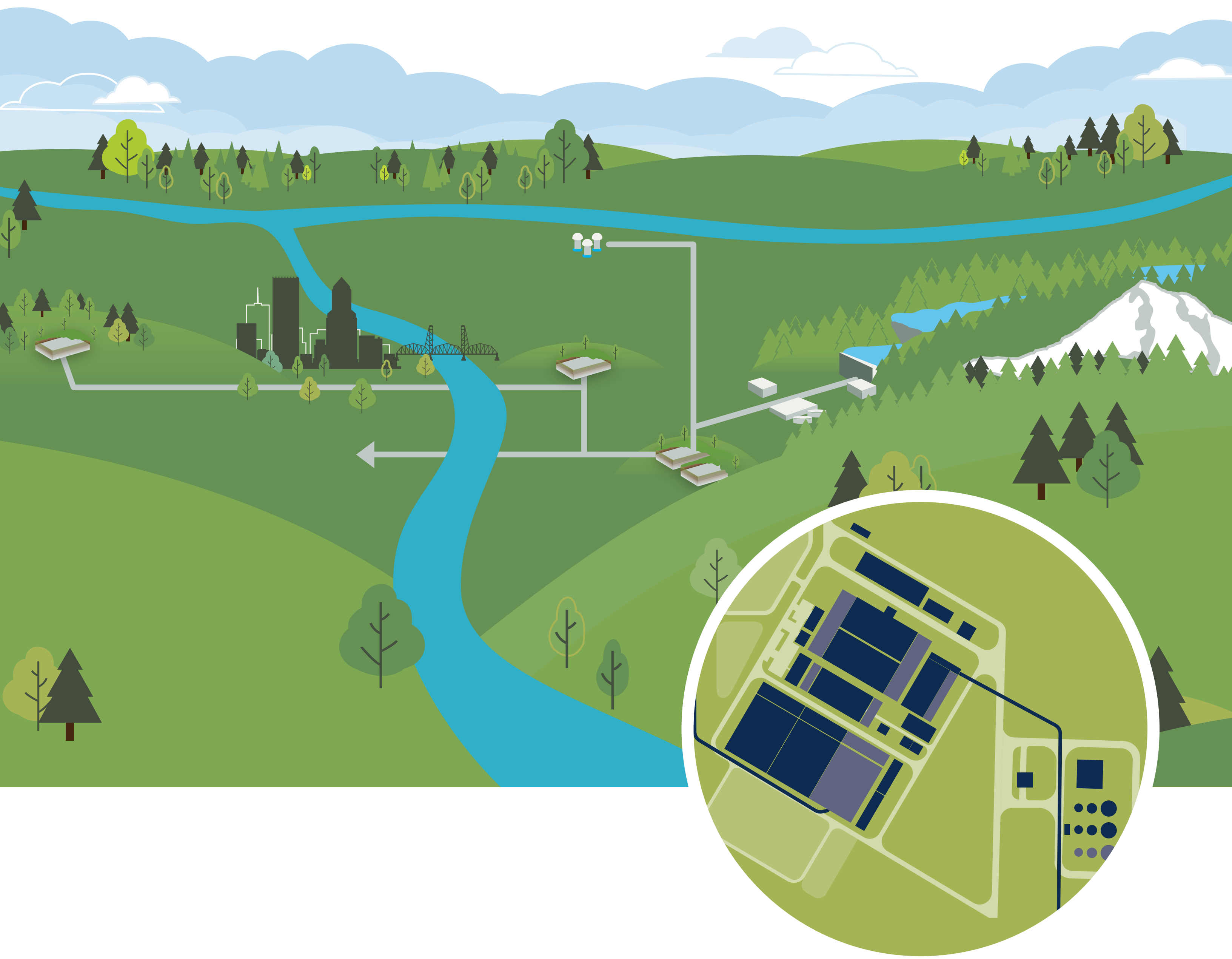
Bull Run
TREATMENT
PROJECTS

Filtration



Project Definition Report

October 2020

Prepared with
assistance from

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Abbreviations and Initialisms

AWOP	Area-Wide Optimization Program
BAF	biologically active filtration
BGS	below ground surface
BTS	Bureau of Technology Services
$CaCO_3$	calcium carbonate
CECs	contaminants of emerging concern
CCC	Clackamas County Code
CCTV	closed-circuit television
CFE	combined filter effluent
CSSWF	Columbia South Shore Well field
CSZ	Cascadia Subduction Zone
CT	concentration time
DAF	dissolved air flotation
DBP	disinfection byproduct
DEQ	Department of Environmental Quality
DIC	dissolved inorganic carbon
DOC	dissolved organic carbon
EPA	US Environmental Protection Agency
ESA	Environmental Site Assessment
FE	filter effluent
FTE	full time equivalent
FY	fiscal year
GAC	granular activated carbon
HAA	haloacetic acid
HAA5	sum of 5 haloacetic acids
HAA9	sum of 9 haloacetic acids
HAB	harmful algal bloom
HGL	hydraulic gradeline
HMI	human-machine interface
LCR	Lead and Copper Rule
LRAA	locational running annual average
LOX	liquid oxygen
MCC	Multnomah County Code
MCL	maximum contaminant level
MCLG	maximum contaminant level goal
MRDL	maximum residual disinfectant levels
MRL	maximum reporting limit
MUA	Multiple Use Agricultural
NOM	natural organic matter
NTU	nephelometric turbidity unit
OAR	Oregon Administrative Rules
OCCT	optimized corrosion control treatment
OHA	Oregon Healthy Authority
O&M	operations and maintenance
PAC	powdered activated carbon
PFAS	per- and polyfluoroalkyl substances
PGE	Portland General Electric
PIS	primary intake structure
PLC	programmable logic controllers
PSW	Partnership for Safe Water
PWB	Portland Water Bureau
RTU	remote terminal unit
SCA	Stream Conservation Area
SDWA	Safe Drinking Water Act
SEC	Significant Environmental Concern
SDS	simulated distribution system
SHPO	State Historic Preservation Office
SLR	surface loading rate
SMCL	Secondary Maximum Contaminant Level
SOR	surface overflow rate
SWTR	Surface Water Treatment Rule
TAC	Technical Advisory Committee
TCR	Total Coliform Rule
TM	technical memorandum
T&O	taste and odor
TOC	total organic carbon
TSS	total suspended solids
TTHM	total trihalomethanes
UCMR	Unregulated Contaminant Monitoring Rule
UFRV	unit filter run volume
USGS	United States Geologic Survey
VFD	variable frequency drive
WAP	wireless access point
WCC	Water Control Center

Units of Measure

C	degrees Celsius
cfs	cubic feet per second
cfu	colony-forming unit
cu	color unit
F	degrees Fahrenheit
ft.	foot/feet
gpd	gallons per day
gpm	gallon(s) per minute
in.	inch
kW	kilowatt
L	liter
L/d	length to diameter ratio
MG	million gallons
mgd	million gallons per day
mL	milliliter(s)
mm	millimeter(s)
MW	megawatt
MwH	megawatt-hours
ppm	part(s) per million
sf	square foot/feet
µg	microgram(s)
µm	micron(s) or micrometer(s)

Executive Summary

The Portland Water Bureau (PWB) is making important improvements that will remove *Cryptosporidium* and other potential contaminants from the Bull Run water supply. These improvements include planning, design, and construction of a new filtration facility and large-diameter pipelines in eastern Multnomah County near the Bull Run Watershed. The filtration facility is required by the federal Safe Drinking Water Act and must be substantially completed by September 30, 2027, under a compliance agreement with the Oregon Health Authority (OHA). The filtration facility and pipelines, collectively referred to as the Filtration project, will be a key component of building and maintaining a resilient water system that serves clean and safe water to nearly 1 million people.

This report documents key outcomes from the planning phase of the Filtration project, including a full year of engineering studies, investigations, analyses, workshops, and tours of water treatment facilities. These outcomes are the foundation of project definition and have been used to make important scope decisions for the filtration facility and pipelines, consistent with PWB and customer values, and to serve as a basis for the first substantive cost estimate and for design.

This executive summary includes:

- Background
- Report Purpose
- Project Definition Process
- Report Organization
- Outcomes

Background

From 2012 to 2017, PWB operated under a unique variance for *Cryptosporidium* regulations. However, after multiple *Cryptosporidium* detections between January and May 2017, OHA notified PWB they would be revoking the variance.

In August 2017, PWB presented treatment options with comparative costs to the Portland City Council. The Council selected filtration to comply with OHA's order to treat the Bull Run supply for *Cryptosporidium*, and to provide other significant water quality benefits (Resolution 37209). A few months later in December 2017, PWB signed a Bilateral Compliance Agreement with OHA that included a schedule for the filtration facility to be substantially complete by September 30, 2027 (Figure ES-1). This gave PWB just under 10 years to plan, design, and construct the Filtration project.

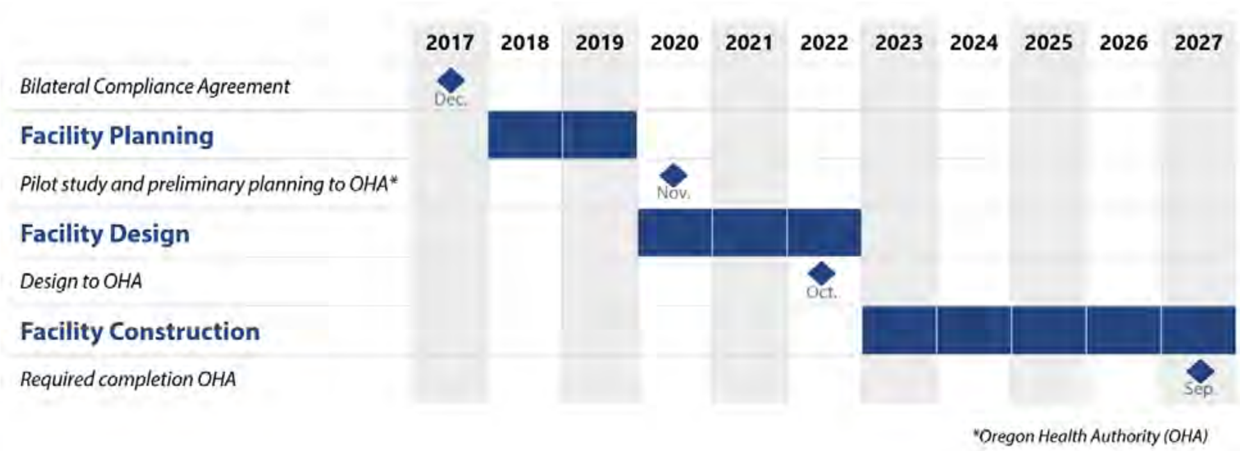

Figure ES- 1. Schedule for Filtration project showing key compliance milestones

Key dates in the Filtration project compliance schedule include:

- **November 30, 2020:** submit pilot study results and preliminary planning
- **October 31, 2022:** submit final construction plans and construction schedule
- **September 30, 2027:** water served meets surface water and *Cryptosporidium* treatment requirements

Report Purpose

The project definition efforts described in this report focused on providing guidance for high-level project decisions. This report documents work to plan for the required filtration facility and pipelines and highlights outcomes that informed the initial cost estimate and will be carried forward to design.

The purpose of the project definition phase was to:

- Conduct field work and engineering evaluations to understand conditions and technical constraints of the filtration facility site and potential pipeline routes.

- Conduct engineering analysis of key considerations that would have a significant impact on project performance, footprint, and cost. That analysis included conducting a pilot study that will inform design decisions.
- Develop an initial cost estimate for the Filtration project, including developing project components to a conceptual design level.

Project Definition Process

This report documents the approach used during project definition to gather information on constraints and opportunities for key project considerations. Using a series of technical workshops and input from nationally recognized treatment and operations experts, PWB completed screening-level analyses to identify feasible alternatives for these key considerations. The screened alternatives were then packaged into project options with initial comparative costs for consideration.

Throughout the planning process, PWB used project values developed with community and PWB staff input to help guide decisions (Figure ES-2). These values were used to characterize, understand, and communicate trade-offs associated with the alternatives for each decision and to evaluate the benefits of feasible options.

Figure ES-2. Public outreach efforts identified community values

The priority values that informed the decision process included:

- *Water Quality*: provide safe, reliable drinking water to a large population in the metropolitan area
- *Resilience*: advance PWB's ongoing efforts to provide a more resilient system
- *Cost and Value*: provide the best value for customers
- *Community and Environment*: implement the projects in a manner that is sensitive to community and environmental impacts

Report Organization

This report is organized into 10 chapters that describe the existing water system; present considerations related to water quality, planning, and design; identify filtration facility support systems and alternatives related to treatment processes and pipelines; evaluate project alternatives; and describe the selected option for the Filtration project.

The description of alternatives in the early chapters of this report include assumptions that form the basis of the draft cost estimate. The draft cost estimate was in turn used for development, screening, and selection of options.

The chapters include:

- **Chapter 1: Introduction** briefly describes the regulatory driver and pre-planning work that informed the project and outlines the process for project definition and the report organization.
- **Chapter 2: Existing Water System** briefly describes PWB's existing water system, including discussion of the Bull Run Watershed and the current water storage and distribution infrastructure. The new filtration facility and pipelines will need to integrate effectively with the vision and infrastructure of the existing system summarized in this chapter.
- **Chapter 3: Water Quality Considerations** focuses on water quality considerations pertinent to design and operation of the filtration facility. These considerations inform the evaluation of treatment approaches presented in subsequent chapters.
- **Chapter 4: Planning Considerations** discusses planning considerations associated with the physical characteristics of the filtration facility site.
- **Chapter 5: Design Considerations** identifies and summarizes engineering principles and industry practices that may serve as guidance or resources for design decisions.
- **Chapter 6: Treatment Process Alternatives** summarizes the screening evaluation to identify feasible treatment process alternatives and inform selection of the treatment train.
- **Chapter 7: Filtration Facility Support Systems** provides an overview of the major support systems assumed for facility operations, including site civil, architectural programming, electrical instrumentation and controls.
- **Chapter 8: Pipeline Alternatives** describes the screening evaluation of feasible pipeline alternatives and the identification of preferred alternatives for the raw water and finished water pipelines.
- **Chapter 9: Alternatives Evaluation** summarizes the treatment process and pipeline alternatives that were packaged into project options and presented to the City Council for input.
- **Chapter 10: Selected Option** describes the selected project option to a level of detail to support the planning level cost estimate and outlines the implementation plan to have an operating facility by the compliance deadline.

This report incorporates content from the March 2020 *Draft Filtration Facility Overview* (chapters 1 through 7) that informed significant scope decisions and served as the basis for filtration facility design beginning in early 2020. This report also includes content (chapters 8 through 10) that documents planning efforts for new pipelines and describes the evaluation of project options.

Outcomes

The planning work described in this report documents the process to select a project option that provides the greatest water quality protections and benefits to Portland customers.

After considering a range of potential investments, the Portland City Council adopted Resolution 37460 on November 27, 2019, which established priority values and expectations to guide design and implementation of the filtration facility and pipelines and identified a project option, illustrated on Figure ES-3, that includes:

- Target capacity of 145 million gallons per day (mgd) to meet most projected peak day demands through 2045
- Conventional treatment, including flocculation, sedimentation, and filtration to best handle turbidity events
- Ozone for enhanced water quality and protection from a range of current and future risks
- Clearwell (onsite water storage) sized for operational flexibility
- Two pipelines to and from the filtration facility to maximize gravity flow, reduce future impacts to the filtration facility neighborhood, and provide a more resilient, easier-to-maintain system

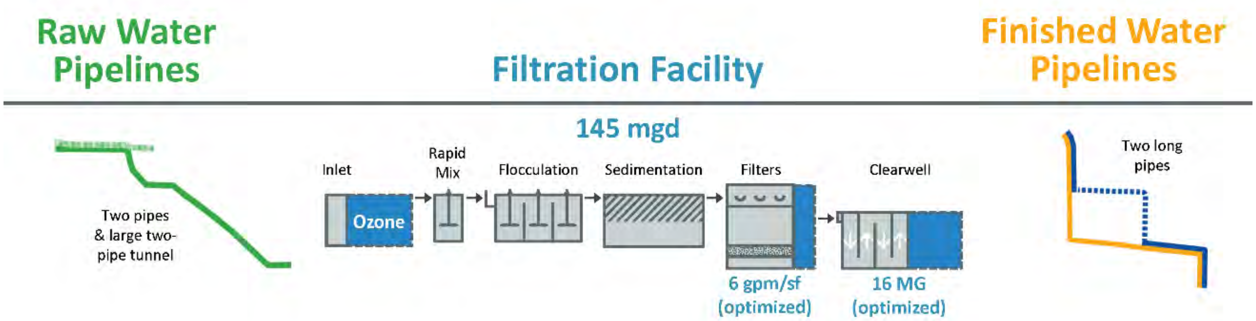

Figure ES-3. Project option adopted by City Council in Resolution 37460

The selected project option provides the greatest water quality protection and benefits to Portland customers, while reducing overall project cost by optimizing the facility capacity, filtration rate, and storage capacity. The figures on the following pages further describe the selected project option identified through project definition.

The conceptual treatment process schematic developed for project definition is illustrated in Figure ES-4. The facility will consist of a series of unit processes designed to treat Bull Run water to meet regulatory standards and consistently provide high-quality clean, safe drinking water.

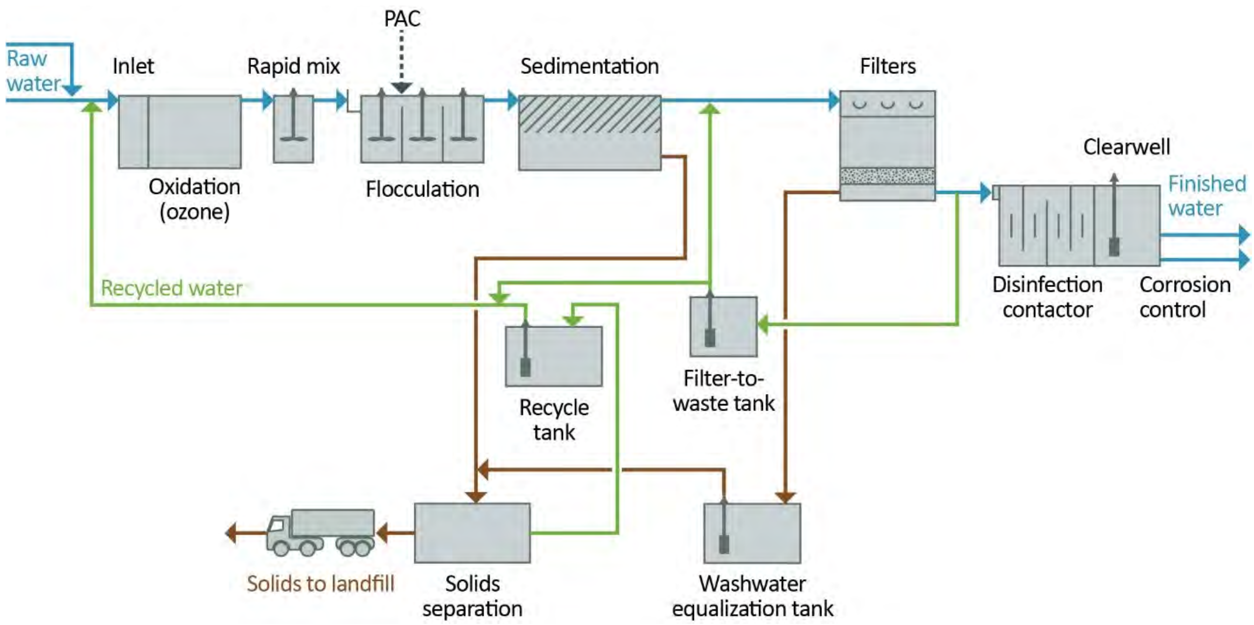

Figure ES-4. Process flow diagram for filtration unit processes

Note that PAC is not used in conjunction with ozone and that pre-ozonation is shown for illustration purposes.

The conceptual hydraulic profile developed for project definition is illustrated in Figure ES-5. Flows entering and leaving the filtration facility will be by gravity—no pumping is anticipated.

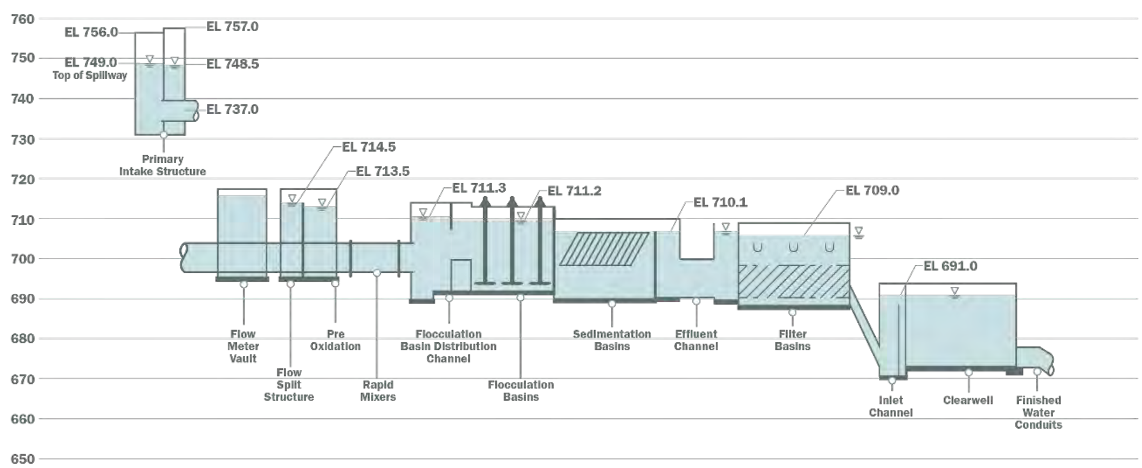

Figure ES-5. Conceptual hydraulic profile for the selected project option

The conceptual site layout developed for project definition is illustrated in Figure ES-6 below. The layout shows the initial design capacity with the ability to be expanded by 50 percent in the future if needed to an ultimate design capacity. This layout is subject to change during design.

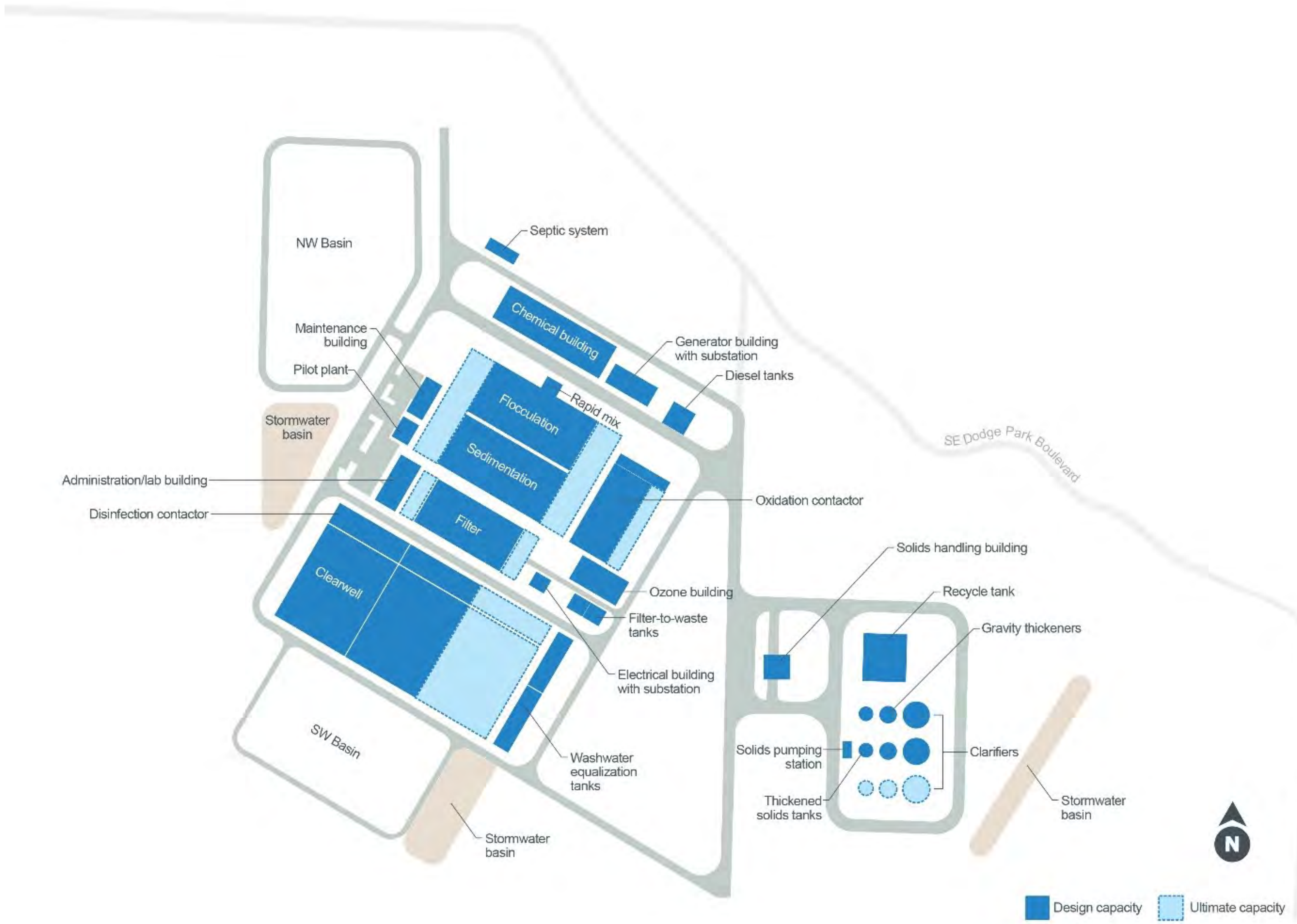

Figure ES-6. Conceptual filtration facility site layout for the selected project option

The preferred pipeline route alternatives identified for project definition are illustrated in Figure ES-7. The selected project option includes two new pipelines to and from the filtration facility to improve overall system reliability and resilience. These pipelines will be built to modern seismic standards and will allow segments of nearby aging conduits to be repurposed or retired.

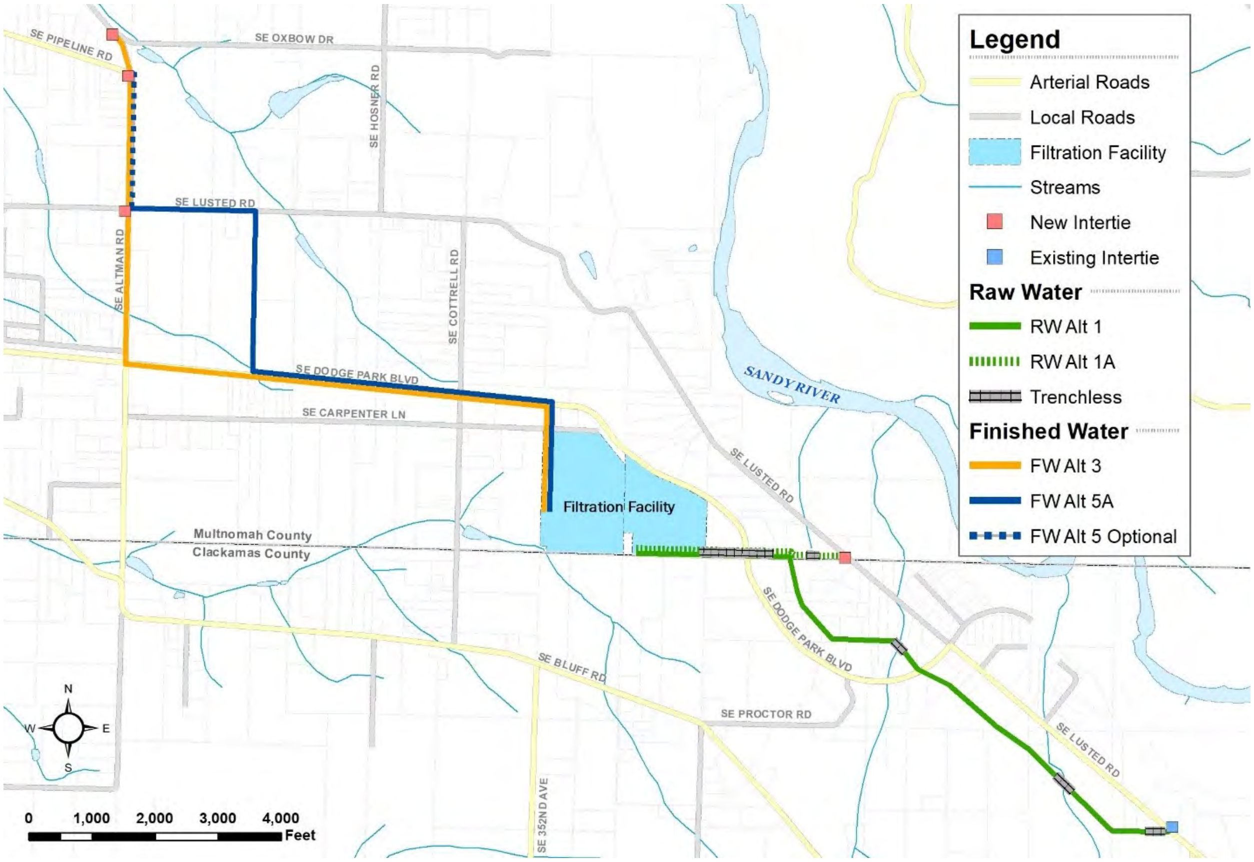

Figure ES-7. Preferred route alternatives for new pipelines to and from the filtration facility

The selected project option produces the highest-quality water and provides multiple barriers for potential contaminants. In addition to removing *Cryptosporidium*, the selected project option provides the best protection from pathogens, reduces disinfection byproducts, reduces the impact of high-turbidity events from fire or storms, helps address algae concerns, keeps sediment out of the distribution system, and better prepares the Portland region for addressing future regulations and emerging contaminants.

The filtration facility and pipelines will be key components of building and maintaining a more reliable, resilient water system that delivers clean, safe water to nearly 1 million people. Once complete, the Filtration project will provide consistent, high-quality drinking water that meets today's water quality standards, helps address future risks and regulations, and improves resilience to help keep our water safe and abundant for generations to come.

Chapter 1
Introduction

The Portland Water Bureau (PWB) is making important improvements to the Bull Run water supply to provide consistent, high-quality drinking water that meets today's water quality standards and helps address future risks and regulations. These improvements include planning, design, and construction of a new filtration facility and large-diameter pipelines in eastern Multnomah County near the Bull Run Watershed. Collectively, the filtration facility and pipelines are referred to as the Filtration project, and their delivery is being managed by a program team that includes both PWB and consultant staff.

This report documents outcomes from a full year of planning work completed by the program team. The purpose of this work was to inform significant scope decisions for the filtration facility, consistent with PWB and customer values, and to serve as a basis for the first substantive cost estimate as well as the beginning of the design phase.

This chapter introduces the content and organization of this report and includes the following sections:

- 1.1 Filtration Overview
- 1.2 Goals and Values
- 1.3 Filtration Background
- 1.4 Report Overview
- 1.5 Introduction Summary

1.1 Filtration Overview

The new filtration facility is required to meet federal drinking water regulations, as described in this section, and will remove *Cryptosporidium* and other potential contaminants from PWB's Bull Run supply, producing cleaner, safer water.

This section includes the following content:

- Project Driver
- Compliance Schedule
- Filtration Benefits
- Program Team
- Filtration Overview Summary

1.1.1 Project Driver

In 2006, the United States Environmental Protection Agency (EPA) issued a federal drinking water rule called the Long-Term 2 Enhanced Surface Water Treatment Rule (LT2 Rule). The purpose of the LT2 Rule is to reduce disease incidence associated with microorganisms in drinking water and specifically to treat for the pathogen *Cryptosporidium*. In 2009, PWB presented LT2 Rule compliance options to the Portland City Council, and Council directed PWB to pursue a variance to the rule, while proceeding with design of ultraviolet (UV) treatment as a backup if the variance was denied (Resolution 36720). In March 2012, PWB received a variance from the requirements to treat for *Cryptosporidium* from the Oregon Health Authority (OHA), the primacy agency in Oregon, based on the results of a year-long intensive sampling program and the limited sources and low occurrence of *Cryptosporidium* in the Bull Run supply.

From 2012 to 2017, PWB operated under this unique variance. However, after a series of low-level *Cryptosporidium* detections in the Bull Run supply from January to May 2017, OHA notified PWB that the variance would be revoked. The number of detections meant that PWB could no longer demonstrate an equivalent level of *Cryptosporidium* in untreated Bull Run water that would be expected with treatment. This forced PWB to re-evaluate its treatment decision and lay out a roadmap to meet federal drinking water regulations.

In August 2017, PWB reviewed treatment options, including UV and filtration, with City Council and community partners. The Council selected filtration to comply with OHA's order to treat for *Cryptosporidium*, and to provide other significant water quality benefits to the nearly 1 million people who use Bull Run water every day and for future generations (Resolution 37309).

1.1.2 Compliance Schedule

On December 18, 2017, PWB signed a bilateral compliance agreement with OHA that included a schedule for the filtration facility to be substantially complete by September 30, 2027. This gave PWB just under 10 years to plan, design, and construct the Filtration project. The approved filtration schedule includes three primary phases: planning, design, and construction. OHA considers this schedule a compliance schedule and includes other key dates for compliance tracking and reporting (Figure 1-1).

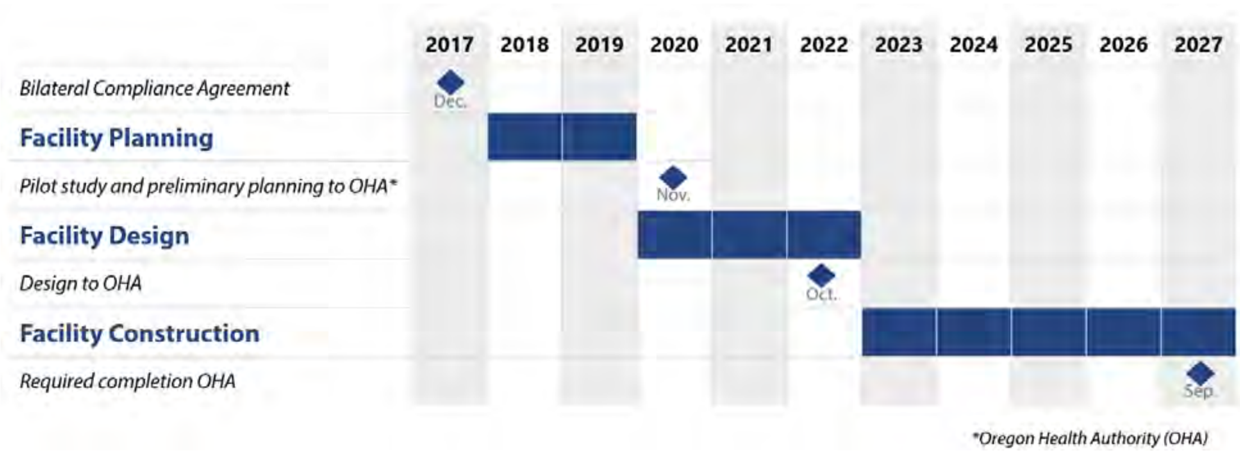

Figure 1-1. Schedule for Filtration showing key compliance milestones

Meeting OHA's compliance schedule will require an aggressive planning, design, and construction schedule. A strategic sequencing of tasks is needed to ensure worker safety, cost-effective operations, and completion and startup by the compliance date.

Key dates in the filtration facility's compliance schedule include:

- **November 30, 2020:** submit pilot study results and preliminary planning
- **October 31, 2022:** submit final construction plans and construction schedule
- **September 30, 2027:** water being served meets all surface water and *Cryptosporidium* treatment requirements

Although the pipelines are not required to be reported on to OHA nor do they have a compliance schedule, they are necessary to connect the filtration facility to the existing water system and enable the filtration facility to be in operation by the compliance deadline.

1.1.3 Filtration Benefits

PWB is committed to protecting public health by serving excellent water every minute of every day. Filtration is an investment in Portland's water infrastructure that will provide benefits now and for future generations. These benefits include:

- Providing significant public health benefit to the community by reducing susceptibility of the City's drinking water to *Cryptosporidium*, *Giardia*, viruses, and pathogens addressed by regulatory drinking water requirements.
- Reducing the region's susceptibility to turbidity events in the Bull Run Watershed by allowing PWB the option to operate through most storm events and after small to moderate landslides. Filtration is also anticipated to benefit the City during and after 1) a seismic event which may result in increased turbidity and interrupt use of groundwater, 2) larger landslides and flooding events, or 3) a watershed fire which may result in extended post-fire turbidity events.
- Reducing the region's susceptibility to other wildfire impacts. Wildfire is known to alter nutrient loading in burned watersheds, including both nitrogen and phosphorus. Biological activity in the Bull Run reservoirs tends to be phosphorus limited, so fire related increases in phosphorus, along with increased water temperatures, may increase the risk of harmful algal blooms. Although PWB has not detected the algal species associated with the production of cyanotoxins to date, increases in nitrogen and phosphorus would likely increase the risk of toxic cyanobacteria.
- Allowing PWB to better control changes in water quality by reducing the need to switch from surface water to groundwater sources during a high turbidity event. Regulations require that PWB switch to groundwater (typically, as the sole water supply) when turbidity exceeds 5 Nephelometric Turbidity Units (which to the human eye still looks very clear). Filtration will reduce the need to switch to groundwater during turbidity events. This will benefit water quality by allowing PWB to better control changes in water quality related to switching between surface water and groundwater sources, which have very different water chemistry (i.e., pH, alkalinity, and total dissolved solids).

- Providing more consistent water quality which is of value to the local economy as the Portland metropolitan area supports multiple international businesses such as sensitive computer and technology manufacturers, hospitals, and hard and soft beverage producers including brewers. These businesses are sensitive to water quality and rely on high-quality water and consistent-quality water.
- Reducing the solubility of lead from household plumbing by incorporating corrosion control treatment into the filtration facility design to reduce the concentration and solubility of lead and copper in drinking water at the customer's tap. Corrosion control treatment will include adjusting alkalinity and pH of Bull Run water delivered to customers to comply with the Lead and Copper Rule.
- Improving overall system reliability by building new large-diameter pipelines constructed to modern seismic standards to connect the filtration facility to the existing system and allow sections of existing, aging conduits to be removed from service.

1.1.4 Program Team

Following the August 2017 Resolution, PWB began pre-planning efforts to establish a preliminary understanding of the scope, schedule, and budget needs to construct the filtration facility and associated onsite facilities described in this report, as well as the necessary pipelines to connect the facility to the existing water system.

As part of this pre-planning work, PWB identified staffing needs to support successful delivery of the filtration facility and pipelines over the 10-year schedule. Along with identifying PWB staff to manage the Filtration project, this included a need for assistance from consultants and technical experts with experience in planning, design, and construction of filtration facilities.

Early on, it was known that these projects would be the largest and most complex undertaken by PWB to date. The projects' size and complexity led PWB to develop a program around these projects to ensure successful delivery by the compliance deadline. In 2018, Brown and Caldwell was selected to serve alongside PWB staff as part of a newly formed program team to guide delivery of both the filtration facility and pipelines. Along with setting up the program office and establishing project controls for delivery, the program team has been responsible for developing project definition of the filtration facility and pipelines.

To augment the program team, PWB will use separate consultants to provide design services and construction services for the filtration facility and pipeline projects.

In 2019, Stantec was selected to provide design services for the filtration facility. In February 2020, the request for proposal for a construction contractor for the filtration facility was advertised, with the expectation of selecting the contractor by the end of 2020. This timeline allows for the contractor to be onboard in time to provide feedback during the design process prior to the anticipated construction start in 2022.

In January 2020, the request for proposal for a pipeline designer was advertised, with the expectation of selecting a designer by the end of 2020. Pipeline design will be led by PWB's Engineering Services Group, Design Section, in close collaboration with the program team. It is anticipated that a pipeline contractor will be selected in 2021.

1.1.5 Filtration Overview Summary

This section described the regulatory drivers, compliance schedule, and early decisions leading up to the Filtration project. Key considerations from this section include:

- **Project Driver.** After multiple *Cryptosporidium* detections in 2017, PWB's variance for *Cryptosporidium* treatment requirements was revoked, and in August of 2017 PWB entered into a bilateral compliance agreement with OHA to treat the Bull Run supply.
- **Compliance Schedule.** PWB's compliance agreement with OHA includes a schedule with key dates for compliance tracking and reporting:
  - **November 30, 2020:** submit pilot study results and preliminary planning
  - **October 31, 2022:** submit final construction plans and construction schedule
  - **September 30, 2027:** water being served meets all surface water and *Cryptosporidium* treatment requirements
- **Filtration Benefits.** The Portland City Council selected filtration treatment both to comply with OHA's order to treat for *Cryptosporidium*, as well for the multiple public health and water quality benefits it provides.
- **Program Team.** During project pre-planning, PWB identified the need for a program team to help guide delivery of the filtration facility and pipelines, as well as additional consulting expertise and support needed for design and construction of the projects.

1.2 Goals and Values

Early in the Filtration project, the program team developed a set of goals and values using community and PWB staff input. The program team is using these goals and values to make decisions for the Filtration project that are consistent with the community that PWB serves.

This section includes the following content:

- Goals
- Values
- Goals and Values Summary

1.2.1 Goals

PWB's ongoing outreach has helped the program team develop goals for the Filtration project. The goals were developed during a series of workshops and are based on best management practices shared by staff and informed by public outreach efforts. These goals, as identified in the team's Program Charter, include the following:

- Continue to produce excellent tasting water while meeting regulatory requirements and water quality goals
- Engage operations and maintenance staff throughout the program life cycle to create an adaptable system with a successful handoff
- Advance social equity and disadvantaged, minority, women-owned, and emerging small businesses capacity

- Maintain transparent communication, stakeholder engagement, and good neighbor practices
- Emphasize a safe work environment and minimize lost work incidents
- Execute a sustainable project that minimizes its carbon footprint
- Complete construction well in advance of the compliance deadline to allow time for commissioning

1.2.2 Values

The values for the Filtration project were originally developed during the pre-planning phase of the project and are described in the *Filtration Plant Decision Process Technical Memorandum (TM)* (PWB, 2018). The values are based on public outreach and community engagement efforts and continue to inform decisions.

Outreach and community engagement for the Filtration project are guided by PWB's commitment to keeping customers and other stakeholders informed, engaged, and supportive of the Filtration project. The overall communications approach is described in the *Communications Framework* included in Appendix A (Barney & Worth, 2018). The framework is informed by public opinion research, best practices for digital and strategic communications, PWB communications, water quality and engineering staff, and the consultant communications team and will be periodically updated as a living document and supplemented with annual communications plans that describe individual activities, responsible parties, and schedule.

PWB conducted community outreach to identify values specific to the Filtration project. More than 1,600 customers provided input through an online survey. PWB also interviewed 20 stakeholder groups representing community organizations, high-volume customers, communities of color, low-income ratepayers, and public health agencies (Figure 1-2).

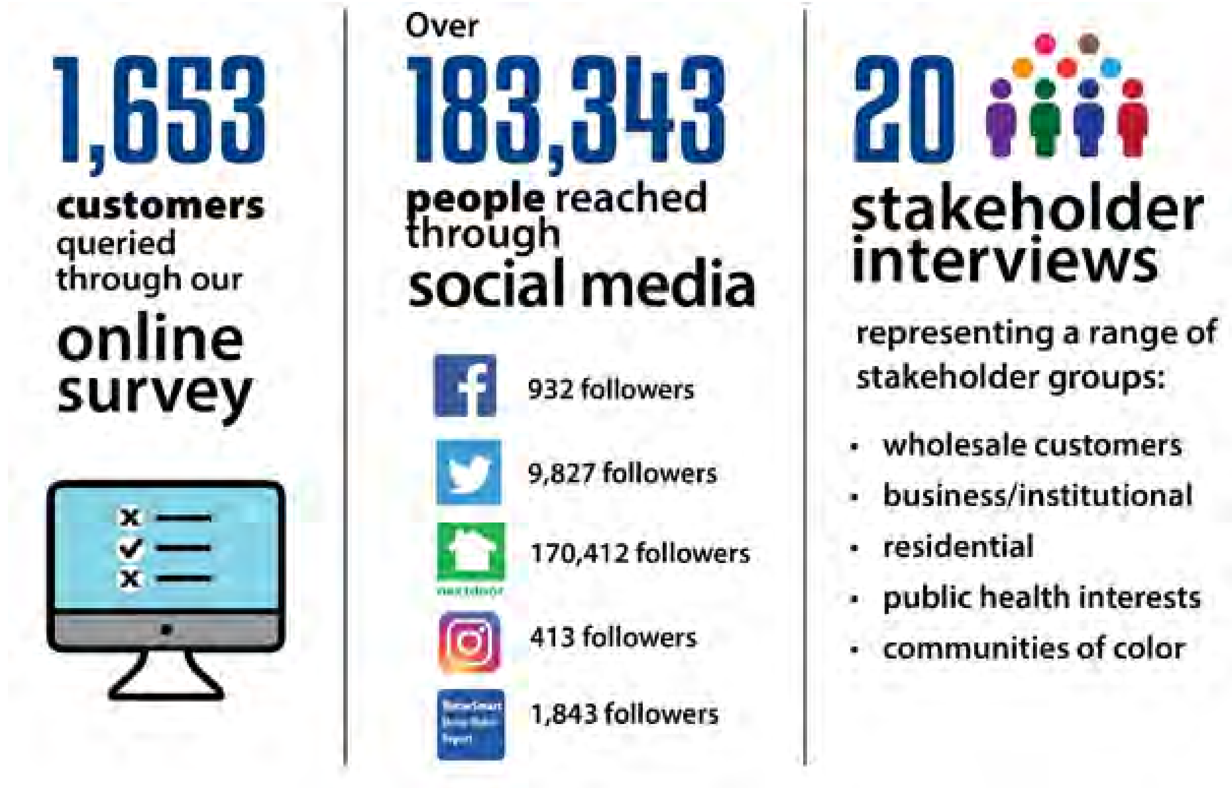

Figure 1-2. Community outreach activities informed development of project values

Community values drawn from this opinion research include:

- Public health and water quality
- Cost/benefit and impact to individual bills
- Appropriate treatment and chemicals
- Minimal environmental impacts
- Looking to future needs
- Reliability and resilience to earthquake and fires
- Consistent water quality (i.e., hospitals, breweries, and industrial customers)

The program team reviewed these community values and then cross-referenced and supplemented them with PWB values. Through multiple iterations in workshop settings, the eight values shown in Figure 1-3 below were identified as representing the most important elements to the community and PWB that must guide any decision.

Figure 1-3. Public outreach efforts identified community values

During project definition, as described in this report, the program team applied these values to characterize, understand, and communicate trade-offs in decision-making for key considerations and confirm alternatives were both technically feasible and aligned with the community PWB serves. These values will continue to be applied as guiding principles as the Filtration project progresses into design.

1.2.3 Goals and Values Summary

This section described the goals and values being used by the program team to shape recommendations for this project. Key considerations from this section include:

- **Values.** The values framework consists of eight criteria based on values developed during pre-planning. These values were used during project definition to evaluate the benefits of feasible alternatives and project options.

1.3 Filtration Background

To accelerate progress and establish early direction for the program team, PWB identified four areas of focus and procured short-term assistance during the project pre-planning phase from three consultant teams. These consultant teams brought experience in treatment planning, treatment design, and public outreach and supported work on: delivery method, facility capacity, facility location, and filtration technology. PWB evaluated these topics using values and constraints and recommended four foundational elements best-suited for Portland's drinking water. These elements, summarized in this section, formed the foundation of the project definition efforts described in this report.

This section discusses the following topics:

- Delivery Method
- Facility Capacity
- Facility Location
- Filtration Technology
- Filtration Background Summary

1.3.1 Delivery Method

The first focus area was to identify a Filtration project delivery method (i.e., how would the filtration facility be constructed). The delivery method significantly impacts what tasks are needed, how tasks are scheduled, and what actions and preparations are required legally (such as with City Council). Hence, this needed to be identified early on. Design-bid-build is historically the industry-standard delivery method; however, this is changing due to advantages associated with other more progressive and collaborative methods.

In order to minimize delivery risk and cost and schedule impacts, PWB evaluated alternative delivery methods as allowed under Oregon Revised Statute 279C.335. The process PWB underwent to evaluate the suitability of delivery methods is documented in the *Filtration Plant Project Alternative Delivery Methods TM* (PWB, 2018). Three potential alternative delivery methods were considered to construct and commission the filtration facility:

- Construction Manager/General Contractor (CM/GC)
- Fixed-Price Design-Build
- Progressive Design-Build

On January 30, 2018, an Alternative Delivery Workshop was held with HDR, Jacobs (CH2M), PWB, and City of Portland procurement staff. The purpose of the workshop was to describe the contractual arrangements for the three alternative delivery methods, differentiate the methods by their specific characteristics, and compare each method to PWB considerations under three main categories: project-specific attributes, PWB culture, and management and reporting.

Based on the expectation that CM/GC would provide the greatest benefits while still aligning with PWB's culture and management/reporting needs, CM/GC was recommended as the project delivery method and was authorized by City Council on August 29, 2018 (Ordinance 189146).

1.3.2 Facility Capacity

The second focus area was facility capacity, a complex decision that included consideration of multiple factors, including future demands, level of service goals (both quantity and quality), and capital and operating costs for different facility capacity and supplementary supply scenarios.

Prior to identifying the filtration facility capacity, PWB first needed to evaluate how it intended to use its sources of water, including groundwater, and how those uses would impact water management strategies. Depending on a variety of factors such as facility capacity, changes in demand, or impacts to operations, alternative management strategies may be used for managing the available water supply to meet water system demand.

However, PWB determined that it would be unwise to size a filtration facility or select treatment processes in such a manner that would require annual use of groundwater to meet summer seasonal demands, or be dependent on water curtailment or nearby wholesalers to provide supplemental water. This determination aligns with past input from PWB staff and the community indicating that Bull Run is the preferred water supply. Thus, the Bull Run supply will remain the primary supply once the filtration facility is constructed, and PWB will continue to employ water management practices as it has in the past and use groundwater to augment the Bull Run supply, such as during hot dry summers and to serve as a backup supply.

Initially in the facility capacity assessment, five capacity alternatives for the future filtration facility were identified. These alternatives and the evaluation process are documented in the *Filtration Plant Capacity Alternatives TM* (PWB, 2018). The alternatives were established based on a combination of the physical constraints of the existing Bull Run supply system and PWB's projected demands. Two of the five alternatives, the 100 and 200 million gallons per day (mgd) capacity alternatives, were found to be unsuitable and were eliminated.

The three remaining alternatives represented approximate filtration facility capacities of 115 mgd, 145 mgd, and 160 mgd. These three alternatives were evaluated using the values framework with measurable criteria to support the decision-making process. In all modeling scenarios, including sensitivity analysis scenarios, the 115 mgd alternative scored below the 145 and 160 mgd alternatives. Along with providing the fewest overall benefits in most of the evaluation scenarios, the 115 mgd alternative had the highest cost per unit value for all reviewed scenarios and was therefore removed from further consideration.

The same decision models indicated the scoring between the 145 and 160 mgd alternatives were very similar. After additional analysis, the two alternatives were merged into an initial facility capacity range of 145 to 160 mgd. This range was determined to provide adequate direction for the pre-planning phase, reflecting PWB's understanding of projected peak day demands, while providing flexibility for design in the years ahead knowing the capacity ultimately constructed might be smaller as a result of subsequent decisions.

The selected range of 145 to 160 mgd was accepted by City Council on December 12, 2018, (Resolution 37402). The choice of capacity then informed the choice of location and filtration technology.

1.3.3 Facility Location

The third focus area was facility location. The location was selected after the filtration facility capacity was identified, but before the filtration technology was determined. Based on previous studies, six sites were evaluated for their ability to host a filtration facility: Carpenter Lane, Lusted Hill (with expansion), Headworks, Larson's Ranch, Powell Butte, and Roslyn Lake (Figure 1-4).

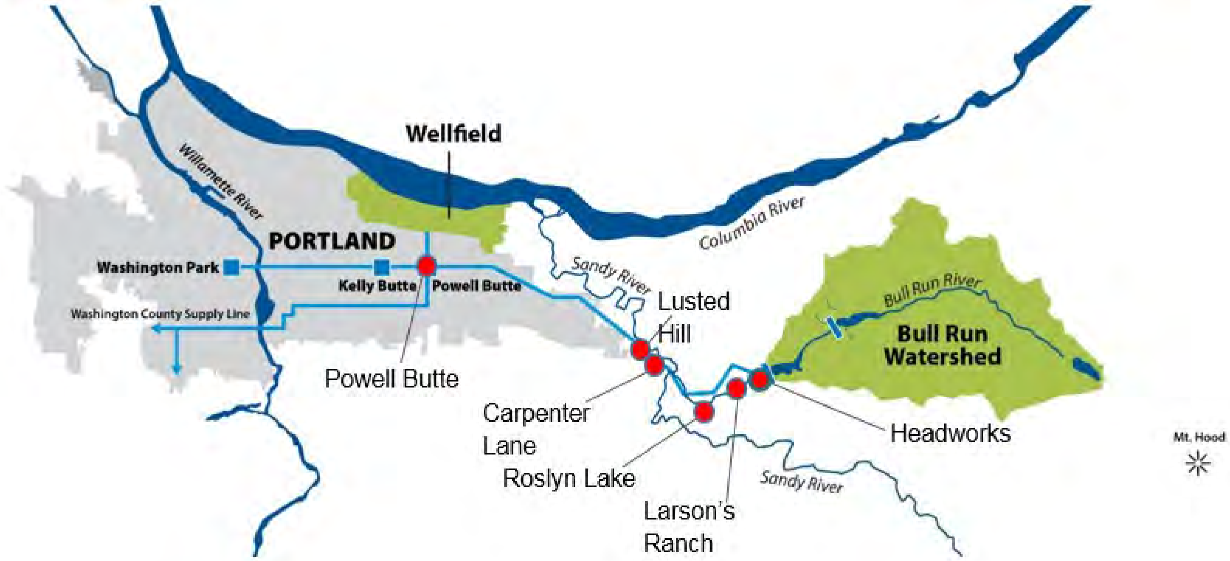

Figure 1-4. Approximate locations of the six potential filtration facility sites reviewed

The location decision was a difficult decision to make. Although the decision framework was used, the final two sites were similar in their value scores. Compounding this was the added difficulty of anticipating how the Bull Run supply transmission system may change in the future. HDR coordinated closely with PWB and their other consultants, Jacobs and Barney & Worth, to develop the criteria and performance scales that assisted the location decision.

Several major considerations exist that affected site choice such as cost/benefit impacts, meeting future needs, and regulatory compliance. Specific siting criteria were developed that supported these broader values, including maximizing gravity flow, site proximity to existing and future conduit rights-of-ways, site size, site slopes and geologic conditions, and impacts to the compliance schedule.

The six potential filtration facility sites were evaluated for their ability to meet five essential criteria. Sites needed to meet all essential criteria or else were considered to have a fatal flaw. Figure 1-5 below summarizes each site's ability to meet the essential criteria (using a pass/fail scoring). Site criterion with green partial shading indicated potentially mitigatable risks, but for which ability to mitigate was unpredictable. Four of the sites failed to meet all essential criteria. Two sites, Carpenter Lane and Lusted Hill, were advanced for further evaluation using the values framework.

	Carpenter Lane	Headworks	Larson's Ranch	Lusted Hill	Powell Butte	Roslyn Lake
Hydraulic Grade	Pass	Fail	Fail	Pass	Pass	Fail
Proximity to Pipes	Pass	Pass	Pass	Pass	Pass	Pass
Lot Size	Pass	Fail	Pass	Pass (with site expansion)	Pass	Pass
Geologic Hazards	Pass	Fail	Pass	Pass	Pass	Pass
Schedule	Pass	Pass	Pass	Pass	Fail	Pass

Figure 1-5. Pass/fail results of how well each initial site met the essential criteria

The evaluation results were discussed and scoring for both alternatives was close. However, further evaluation highlighted that the Lusted Hill site had two deficiencies: 1) the existing property was not large enough for a filtration facility without purchasing more property; 2) the land use zoning would be difficult to site a filtration facility; and the impact of these two conditions on the schedule made this site unacceptable. Therefore, the City-owned site near Carpenter Lane was recommended for the filtration facility location and was accepted by City Council on December 12, 2018 (Resolution 37402).

1.3.4 Filtration Technology

The fourth focus area was filtration technology. This decision is described in the *Filtration Plant Technology Assessment TM* (PWB, 2018). The filtration technology decision was considered after facility capacity and facility location were determined because these could have impacted the technology decision.

The EPA recognizes several filtration strategies for compliance with the LT2 Rule. These technologies include granular media filtration, membrane filtration, slow sand filtration, cartridge and bag filtration, and diatomaceous earth filtration. Of these filtration technologies, there are no known cartridge and bag or diatomaceous earth filtration facilities larger than 50 mgd in the United States. Therefore, PWB proposed to focus the evaluation on the remaining three technologies.

PWB identified a list of filtration benefits that would have measurable impact on evaluating the three technologies. These filtration benefits are based on the benefits originally described by PWB to City Council in the August 1, 2017, memorandum identifying the probable benefits of filtration over UV treatment. Potential benefits of filtration include:

- Provide pathogen removal for *Cryptosporidium, Giardia*, bacteria, and viruses
- Produce biologically stable water
- Reduce disinfection byproducts
- Increase supply reliability

- Reduce distribution system flushing and lower turbidity levels
- Reduce iron and manganese concentrations
- Improve water quality stability and reduce lead and copper release at customer taps
- Reduce water quality impacts from warmer weather (such as algae)
- Reduce organic discoloration events
- Improve ability to respond to changes in regulations
- Increase ability to meet critical service levels
- Improve ability to treat a sustained elevated turbidity event
- Reduce customer cost of water treatment at the tap

The three technologies were then evaluated for their ability to provide the above desired system benefits. For evaluation purposes, some pre- or post-treatment measures were assumed so that PWB could evaluate the full treatment system's ability to achieve the desired benefits of filtration. This was done to develop capital and operating costs so that decision-makers could fairly evaluate the alternatives. Actual pre- or post-treatment processes will be determined later. None of the treatment configurations for slow sand filtration provided a good or excellent rating for all filtration benefits. Therefore, it was recommended that only granular media filtration and membrane filtration be further evaluated for potential use on the Bull Run supply.

These two technologies were then compared using the decision model. In the different scenarios, granular media filtration resulted in higher performance. Granular media filtration provides higher value at a lower life cycle cost while providing the desired filtration benefits. The membrane filtration option had a higher life cycle cost than granular media filtration and provides less value. Granular media filtration was recommended as the filtration technology and was accepted on December 12, 2018 (Resolution 37402).

1.3.5 Filtration Background Summary

This section described technical analysis work that PWB completed during pre-planning for Filtration to make four critical decisions: delivery method, facility capacity, facility location, and filtration technology.

- **Delivery Method.** CM/GC was selected as the delivery method, allowing for contractor input during design to improve constructability and reduce cost and schedule risks.
- **Facility Capacity.** A range of 145 to 160 mgd was selected as the appropriate capacity for the filtration facility to consistently meet Portland's projected water system demands.
- **Facility Location.** Site selection was determined based on passing all of the siting criteria that included: works for gravity flow; reasonably close to existing and future pipelines; adequate area, reasonable slopes and suitable geologic conditions, and ability to meet the compliance schedule. The City-owned property near Carpenter Lane site was selected for the location.
- **Filtration Technology.** Granular media filtration was selected as a proven technology widely used at filtration facilities nationwide to provide excellent water quality.

These four early decisions were evaluated using a rigorous process founded in community values and form the basis for the project definition efforts described in this report.

1.4 Report Overview

The purpose of this report is to document key outcomes from the project definition effort related to the filtration facility. These outcomes include important technical constraints identified through field evaluations, conclusions based on engineering analysis related to key project considerations, and major scope decisions that informed project definition and development of the initial cost estimate for the filtration facility.

This section includes the following content:

- Report Organization
- Evaluation Process
- Report Overview Summary

1.4.1 Report Organization

This report is organized into seven chapters that describe the existing water system; present considerations related to water quality, planning, and design; identify treatment process alternatives suitable to treating the Bull Run supply; and describe support systems needed for the filtration facility.

Each chapter and major section of the report begins with an introduction or preview, followed by the main content (typically an evaluation process), and a summary. Supporting materials documenting the engineering effort associated with the report are included in the appendices.

The report uses specific words to identify:

- Assumptions that form the basis of the draft cost estimate
- Suggestions from the program team for further evaluation during design
- Preferences established through discussion with PWB staff
- Decisions validated during project definition

Assumptions were made based on best practices and professional judgment to support development of the initial cost estimate and are documented throughout this report and in the *Basis of Estimate Report* (Appendix B). Many of the assumptions identified in this report will be further evaluated during design.

While the primary purpose for this report is to document evaluations and recommendations to inform the design phase of the Filtration project, this report is also intended to be shared with the public to demonstrate transparency and communicate direction. As a result, the program team has worked to make content available to non-technical readers, while still presenting and conveying critical technical information.

1.4.2 Evaluation Process

This report documents the first phase of the project definition evaluation process, which included engineering analysis and evaluation of filtration facility alternatives performed by the program team.

- **Phase 1.** This process began with identification of key considerations that significantly affect the filtration facility performance or project cost. Technical consultants performed the initial engineering analysis and presented alternatives for each key consideration to the program team at one or more workshops. Based on feedback from the program team, additional engineering analysis was performed and presented. The workshops focused on the feasibility of available alternatives and their ability to meet level of service goals, as well as the benefits of the alternatives to Portland customers consistent with identified values, particularly water quality and reliability. Based on the engineering analysis and consideration of values, the program team reached an initial conclusion on feasible alternatives for each key consideration as described in this report.
- **Phase 2.** In the second phase of evaluation, the program team invited input from nationally recognized treatment and operations experts to help validate conclusions about filtration facility alternatives. The program team then packaged the screened alternatives into project options for the filtration facility and developed comparative costs for each option. These options were then taken to the PWB Commissioner and City Council for direction.

Collectively, these two phases of the project definition evaluation process were used to inform major scope choices and to establish a planning-level cost estimate for the filtration facility.

1.4.3 Report Overview Summary

This section provided an overview of the general structure and content of this report. Key considerations from this section include:

- **Report Organization.** The organization of this report generally follows the approach during project definition to gather information on constraints and opportunities, then perform screening level analysis using values and input from PWB to identify feasible alternatives for the filtration facility. The summaries at the conclusion of each chapter and chapter section identify key conclusions that informed project definition and assumptions used to develop the cost estimate.
- **Evaluation Process.** The evaluation process focused on identifying key considerations with the greatest impact on filtration facility performance and cost. For each key consideration, the program team identified and screened alternatives using engineering analysis and the values framework. The screened alternatives were then packaged into project options with comparative costs that were presented to the Portland City Council in November 2019.

1.5 Introduction Summary

This chapter introduced the regulatory driver for the Filtration project, the values that are informing project decisions, key outcomes from the pre-planning work, and the general structure of this report. Key considerations from this chapter include:

- **Filtration Overview.** The Filtration project is required by the federal Safe Drinking Water Act and must be completed by September 30, 2027, per a bilateral compliance agreement with OHA.
- **Goals and Values.** PWB is using values developed with input from community engagement and PWB staff to guide project decisions. These values include water quality, resilience, value and cost, and community and environment.
- **Filtration Background.** The project definition work described in this report builds on early planning efforts that identified four foundational elements: CM/GC as the project delivery method, a range of 145 to 160 mgd for the facility capacity, a facility location in eastern Multnomah County, and granular media filtration as the filtration technology.
- **Report Overview.** This report is organized into seven chapters that document the approach during project definition to gather information on constraints and opportunities, then perform screening level analysis using values and input from PWB to identify feasible alternatives for the filtration facility. The resulting outcomes from the engineering studies, investigations, and analyses were used to inform major scope decisions and to establish a planning-level cost estimate for the filtration facility.

References

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PWB, *Filtration Plant Decision Process Technical Memorandum*, September 5, 2018.[a]

PWB, *Filtration Plant Capacity Alternatives Technical Memorandum*, September 11, 2018. [a]

PWB, *Filtration Plant Project Alternative Delivery Methods Technical Memorandum*, February 27, 2018. [a]

PWB, *Filtration Plant Site Alternatives Technical Memorandum*, September 11, 2018. [a]

PWB, *Filtration Plant Technology Assessment Technical Memorandum*, September 18, 2018. [a]

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[a] *These references were previously aggregated into a single resource that is available at: portlandoregon.gov/water/77548 under Bull Run Preferred Alternatives Report*

Chapter 2
Existing Water System

Over the past century, the Portland Water Bureau (PWB) has developed and carefully managed a water system that starts at the protected Bull Run Watershed and flows via gravity to Portland. This chapter briefly describes PWB's existing water system, including discussion of the Bull Run Watershed and the current water storage and distribution infrastructure. Filtration will be a key component of building and maintaining a resilient water system that serves clean and safe water to nearly 1 million people. The new filtration facility and pipelines will need to integrate effectively with the vision and infrastructure of the existing system summarized in this chapter.

This chapter includes the following sections:

- 2.1 Background
- 2.2 Existing Infrastructure
- 2.3 Existing Water System Summary

2.1 Background

PWB has provided water to Portland area residents since 1895. PWB is the largest supplier of domestic water in Oregon and serves nearly one quarter of the state's population, either directly as retail customers or through wholesale contracts with 19 water providers in the region.

The primary water supply for Portland area residents is the Bull Run Watershed. The area draining to the water supply is 102 square miles. Between fiscal years 2013-14 to 2018-19, total Bull Run water produced has ranged from 30 billion gallons to slightly over 36 billion gallons. Located approximately 22 miles east of Portland in the Mount Hood National Forest, the protected watershed is managed by the United States Forest Service in cooperation with PWB. The collection, treatment, transmission, and distribution of drinking water from the Bull Run Watershed includes many assets over a large geographic area.

PWB has two large dam structures within the Bull Run Watershed, Dam 1 and Dam 2, that create two water storage reservoirs with a combined storage capacity of 16.9 billion gallons (with 9.9 billion gallons available for drinking water supply). Bull Run Lake, located in the higher elevations of the watershed, provides some additional water storage.

PWB owns structures and facilities for surface water treatment at two main sites: one where water supply is diverted from the Bull Run River into the conduits (within the Bull Run Watershed) and one near SE Lusted Road (south of Troutdale). These sites are referred to as Headworks and Lusted Hill, and are discussed in greater detail in Section 2.2.

Finished drinking water is sent to terminal storage at Powell Butte. From there, it is distributed to other major storage reservoirs, including Kelly Butte on the east side of Portland and to Washington Park (temporarily offline) on the west side of Portland. Total water storage capacity of Portland's system exceeds 200 million gallons. Water is distributed to wholesale customers from this system, which includes a significant connection on the west side of Portland via the Washington County Supply Line.

PWB has a supplemental groundwater supply, the Columbia South Shore Well Field (CSSWF), located south of the Columbia River and east of Portland International Airport. Water from the CSSWF is pumped to Powell Butte where it can be distributed directly or mixed with Bull Run water and distributed. Figure 2-1 shows this supply system. The CSSWF includes groundwater wells (such as production wells, monitoring wells, and test wells), a groundwater pumping station with a 2 MG onsite tank, and a groundwater collection system to convey water from each production well to the groundwater pumping station. The CSSWF has a baseline installed capacity of 95 mgd, and was originally sized to provide for winter season demand during turbidity events in Bull Run. The useable well field capacity for a 30-day duration use is approximately 80 to 85 percent of the installed capacity, or 76 to 81 mgd, which accounts for wells that are out of service for maintenance or of limited service due to water quality considerations such as elevated manganese.

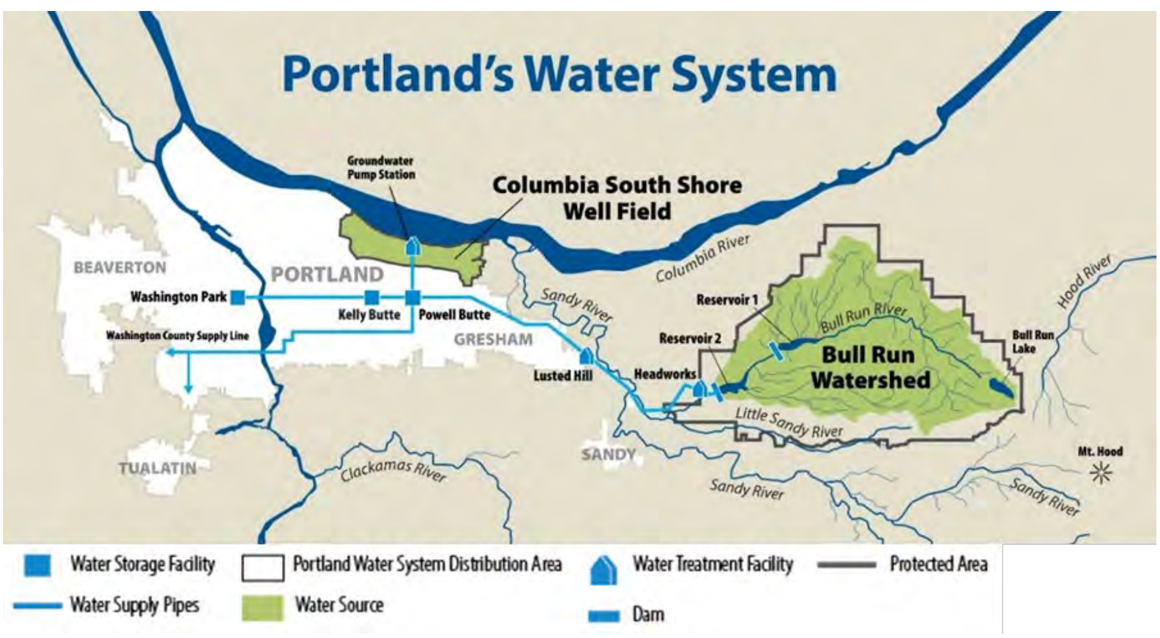

Figure 2-1. Overview of the existing water system

Once water is sent to a terminal reservoir, it is delivered via distribution transport mains to pump stations, tanks, and other distribution supply facilities. Water is then sent into local distribution mains (typically, smaller in diameter than transmission and distribution-transport mains) that deliver water directly to individual services or fire hydrants. Portland's distribution system includes more than 2,200 miles of distribution mains and more than 185,000 active retail distribution services.

2.1.1 Population and Development

In planning for Filtration, PWB considered projected water demands (from both Portland and wholesalers) to ensure the filtration facility is prepared to meet future supply and demand conditions. Currently, PWB serves water to approximately 950,000 people. This includes both Portland and wholesaler customers. Due to anticipated reductions in wholesale demands, it is estimated that approximately 912,000 people will be served in 2027 when the filtration facility is in operation.

Although population data indicates that Portland continues to attract new residents, per capita usage is declining for a variety of reasons, including plumbing code changes, customer behavior, and densification. These trends align with actual demand data in Figure 2-2, which shows the population growth from fiscal year (FY) 2007–08 to FY 2017–18 compared to steady and/or declining water consumption data.

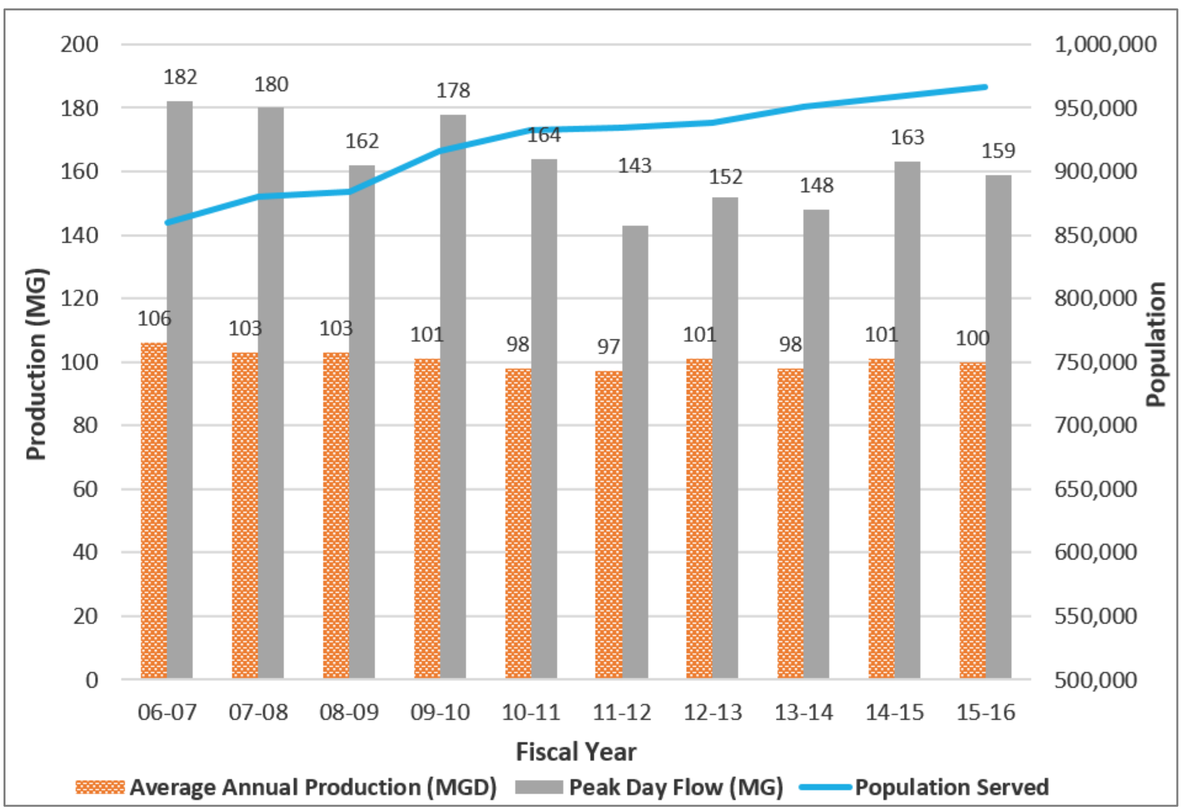

Figure 2-2. Historical water production trends show a slight increase in population with a slight decrease in average annual production and peak day flows

It was estimated in 2017 that the population served could increase to more than 1 million by 2045 with an approximate peak day demand of up to 160 million gallons per day (mgd). Figure 2-3 shows the potential for peak day demand as high as 160 mgd in 2045 given the projected variability in weather conditions, estimated in 2017 using weather data from 1940 to 2015. The initial 145 to 160 mgd capacity range of the filtration facility is sized to meet this upper end of the peak day demand range.

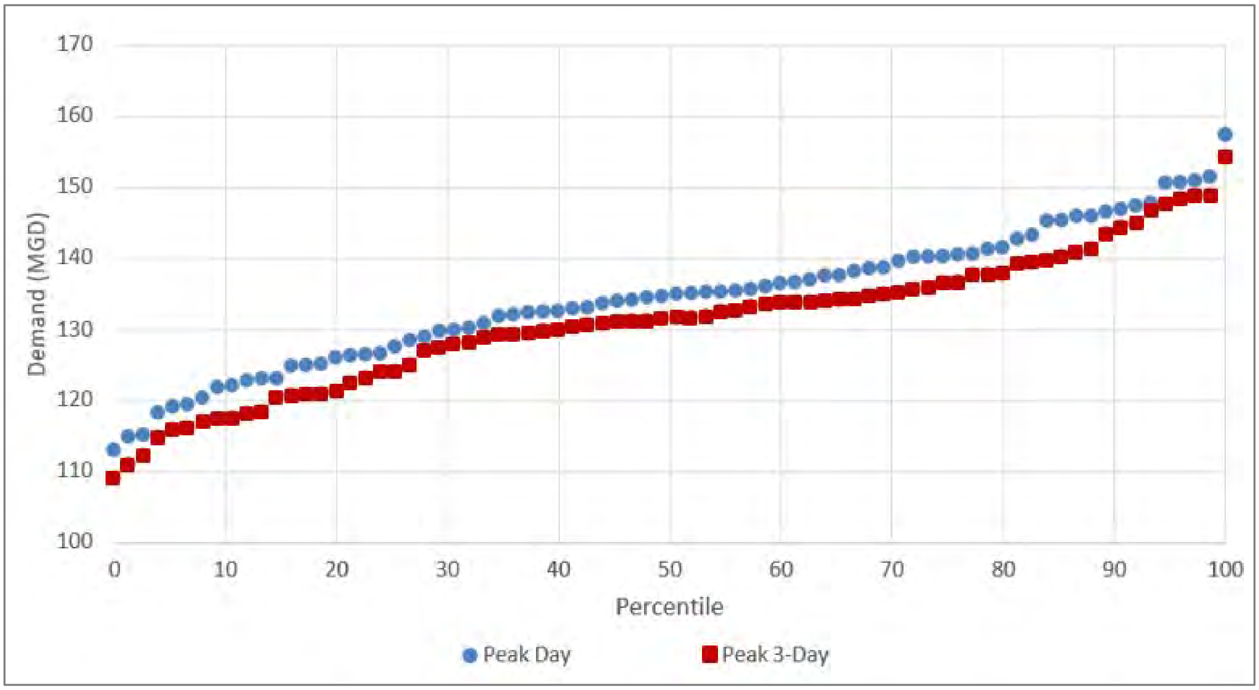

Figure 2-3. Potential peak day demand in 2045 over the range of variability in weather conditions, estimated in 2017 using weather data from 1940–2015

2.1.2 Service Areas

Supply and transmission of water to customers is generally organized into service areas. Service areas can be divided into retail service areas (internal to the City), and wholesale service areas (external to the City). The wholesale service areas are shown in Figure 2-4 below. Service areas may also have other connections, or interties, to neighboring systems.

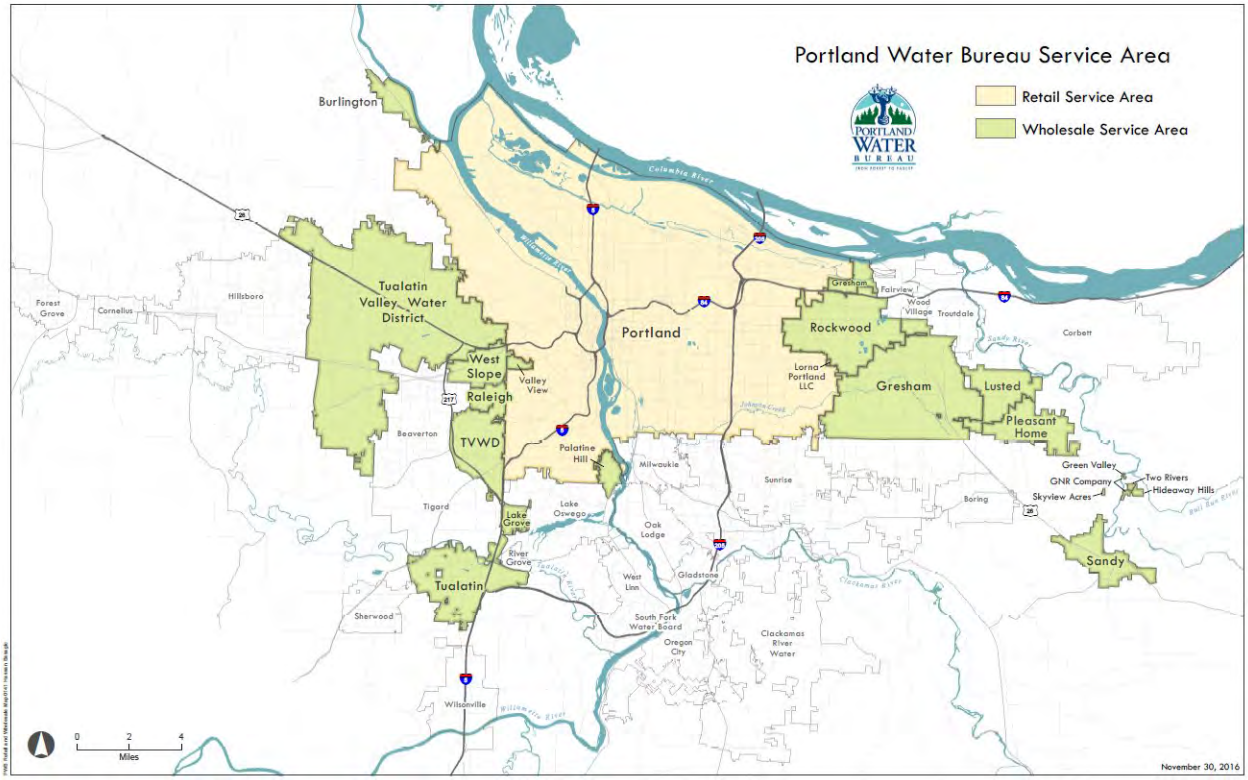

Figure 2-4. PWB's retail and wholesale service areas

Retail Service Areas

Retail service areas include 194 pressure zones representing approximately 610,000 customers. Retail service areas are almost exclusively downstream from the Powell Butte reservoirs and/or connected to the distribution system. The collective retail service area demand is accounted for in demand projections. For additional information on pressure zones and the distribution system please refer to the *Distribution System Master Plan* (PWB, 2007).

Wholesale Service Areas

Wholesale service areas represent a significant portion of PWB's water supply and include 19 wholesale customers. Wholesale demands are constrained by intergovernmental agreements that can be periodically renewed (or not renewed) and have the potential to impact supply planning. Individual wholesale demands are considered in PWB supply planning efforts. Table 2-1 below identifies PWB's 19 wholesalers and demands. Two wholesale customers have indicated they are considering other sources.

Table 2-1. Wholesale Customer Statistics (FY 2016–2017)[a]

Wholesaler	Consumption in 100 cubic feet[b]	Service Population Served by Portland[c]	Service Population Served by Other Sources[c]	Contract Expires
Burlington Water District	18,820	301	—	2026
GNR Water Company	3,222	47	—	2021
Green Valley Water Company	244	7	—	2021
Gresham, City of	2,839,686	69,596	5,929	2026
Hideaway Hills Water Company	2,079	51	—	2021
Lake Grove Water District	179,480	2,305	768	2026
Lorna Water Company	10,016	246	—	2021
Lusted Water District	68,517	1,078	—	2026
Palatine Hill Water District	158,148	1,525	—	2027
Pleasant Home Water District	75,785	1,463	—	2026
Raleigh Water District	241,771	4,227	—	2026
Rockwood Water People's Utility District	3,003,143	61,487	1,317	2026
Sandy, City of	206,238	3,451	7,421	2028
Skyview Acres Water Company[d]	—	—	37	2021
Tualatin Valley Water District	7,618,976	157,809	63,046	2026
Tualatin, City of	2,066,186	26,878	—	2026
Two Rivers Water Association	2,371	14	—	2021
Valley View Water District	58,742	949	—	2026
West Slope Water District	555,560	10,713	—	2026
Total Wholesale Customers	**17,108,984**	**342,147**	**78,518**	**—**

a. *Data is summarized from PWB Statistical Report for Fiscal Year 2016-2017, July 2018.*

b. *Consumption and revenue figures are adjusted for water sold to City customers.*

c. *Estimates are based on Portland State University service population forecasts. Population split is based on share of water purchased from Portland and other water sources as of June 30, 2017.*

d. *Skyview Acres Water Company is currently purchasing water directly from City of Sandy.*

Some of PWB's wholesale customers are connected upstream of the Powell Butte reservoirs, which presents challenges when supply planning as these customers may be impacted differently by system operations and hydraulics. Table 2-2 below lists wholesale customers who have connections to the supply system upstream of Powell Butte Reservoir along with any known alternate sources.

In the event that water transmission is disrupted along the conduits upstream of Powell Butte, these customers are anticipated to be supplied via finished water in the clearwell on site at the filtration facility. Of these wholesale customers, three were known to have alternate sources. In an emergency, these customers could potentially use their alternate sources to meet some or all of their supply needs, thereby reducing demand on PWB.

Table 2-2. Wholesale Customers Connected to Supply System

Wholesale Customer	Alternate Sources
City of Gresham	Cascade groundwater system
City of Sandy	Alder Creek and Brownell Springs
GNR Water Corporation[a]	None
Green Valley Water Company[a]	None
Hideaway Hills Water Company[a]	None
Lorna Water District	Main supply off of distribution system Backup supply is from Conduit 2
Lusted Water District	None
Pleasant Home Water District[a]	None
Rockwood Water PUD	Cascade groundwater system
Skyview Acres Water Company[a]	None
Two Rivers Water Association[a]	None

a. *These customers are served off of the SE Lusted Road distribution main.*

2.1.3 Background Summary

This section described PWB's service areas and the anticipated population growth in the region, which are important considerations for the filtration facility design and ultimate capacity.

- **Population and Development.** Based on PWB's assessment of projected demands, it is estimated that just over 1 million people will be served in 2045. The initial capacity range of the filtration facility (145 to 160 mgd) is sized to meet the upper end of this projected demand range.
- **Service Areas.** Some of PWB's wholesale customers draw water upstream of Powell Butte. In the event that water transmission is disrupted along the conduits upstream of Powell Butte, these customers are anticipated to be supplied with finished water stored at the filtration facility.

2.2 Existing Infrastructure

The Bull Run water system infrastructure consists of three major components: storage, transmission, and distribution. Storage includes the dams and reservoirs located in the Bull Run Watershed. Transmission includes the facilities that transport water from the Bull Run Watershed into Portland. Distribution includes the facilities involved in delivering water supply directly to homes and businesses. This section summarizes the storage and transmission components of the system as they pertain to the filtration facility.

2.2.1 Bull Run Watershed

The Bull Run Watershed is well known as one of the most protected drinking water watersheds in North America, if not the world. Ninety-six percent of the watershed is located on federal land and is part of the Mount Hood National Forest. Four percent of the watershed is owned by PWB. Public access to the watershed is prohibited except as part of strictly controlled tours. Recreation, commercial uses, and private development are not allowed in the watershed.

These protections allow the watershed to function as an intact forested ecosystem. The protected status of the watershed significantly reduces water quality risks and delivers very high-quality water into the drinking water system.

2.2.2 Bull Run Lake and Dam

The Bull Run Lake dam and dike are earthen and concrete structures located approximately 5 miles northwest of Mount Hood and provides additional storage at Bull Run Lake (Figure 2-5 below). The dam was originally constructed around 1915 to 1917. The dike is believed to be originally constructed between 1917 and 1921. Both have been repaired or rebuilt. Excessive seepage through the landslide that formed beneath the lake flows into the Bull Run River. The dike acts as a spillway releasing water during overflow events. Collectively, the structures and lake retain approximately 3 billion gallons of water which is available during the summer season supply. Bull Run Lake has a crest elevation of 3,184 feet. Outlet piping releases this water for supply into the Bull Run River.

Figure 2-5. Bull Run Lake Dam

2.2.3 Bull Run Reservoir and Dam 1

Reservoir 1, also known as Lake Ben Morrow, was created by the construction of Dam 1 (Figure 2-6). This dam is a concrete gravity arch structure built in 1929. Dam 1 retains approximately 11 billion gallons of water at an elevation of 1,050 feet (crest of the dam), approximately 10 billion gallons at an elevation of 1,045 feet (top of the flashboards), and approximately 8.7 billion gallons at an elevation of 1,036 feet (crest of the spillway).

Figure 2-6. Dam 1 showing Reservoir 1 drawn down during the early autumn before winter rains

The South Intake Tower on Dam 1 allows for water transfer between Reservoir 1 and the lower Bull Run River via the dam's needle valves. The North Intake Tower on Dam 1 provides the inlet for Powerhouse 1.

2.2.4 Bull Run Reservoir and Dam 2

Dam 2 was constructed in 1962 resulting in Reservoir 2 and retains approximately 6.8 billion gallons of water (Figure 2-7). The facility includes a spillway, north and south intake towers, control valving, and Powerhouse 2.

Figure 2-7. Overlooking the Diversion Pool

The North Intake Tower is used for water supply and power generation and can pass water to either Powerhouse 2, or to two 42-inch North Howell-Bunger Valves, which both discharge into the Diversion Pool at the downstream base of Dam 2.

The South Intake Tower serves as a secondary means to pass water through Dam 2 via the South Tower Tunnel. The tunnel can discharge to the Diversion Pool through the single 48-inch South Howell-Bunger Valve to the Primary Intake Structure, to Screenhouse 3, or directly into the Bull Run River. The tower is also used to manage the temperature of water that is diverted into the system and that flows downstream for Endangered Species Act-listed fish.

The Diversion Pool is a section of the Bull Run River located immediately below the base of Dam 2 and Powerhouse 2. The Diversion Pool was created when the diversion dam (a concrete weir) was constructed in the river channel in 1921. The dam was later raised bringing the crest of the dam to 749 feet.

2.2.5 Headworks

The facilities at Headworks are located in a small, flat area flanked by steep, wooded slopes and is immediately downstream of Dam 2 (Figure 2-8 below). Here Bull Run water is screened, chlorinated (chlorine is an oxidant that helps prevent harmful bacteria from growing in the water system), and enters three large-diameter pipelines (Conduit 2, Conduit 3, and Conduit 4) that transport the water approximately 10 miles downstream to Lusted Hill. The site includes facilities for water intake, chlorination, and operations and maintenance. Site expansion is restricted because of limited land availability.

Figure 2-8. Aerial view of Headworks

A small pilot facility was installed in May 2019 at Headworks due to its proximity to the existing inlet (assuring similar water characteristics) and staff operations (Figure 2-9). The pilot is testing potential treatment processes for their ability to satisfactorily treat Bull Run water and includes two trailers that are just to the side of Screenhouse 3.

Figure 2-9. Pilot testing facilities installed to study treatment processes

2.2.6 Conduits

The conduits are transmission pipelines that transport Bull Run water approximately 20 miles from Headworks to town (Figure 2-10). Along the way, there are several wholesale and retail connections. There are currently three conduits in service–Conduit 2, Conduit 3, and Conduit 4. Conduit 1 was taken out of service in the early 1950s when Conduit 4 was brought online. Remaining Conduit 1 sections are used in some areas for cathodic protection purposes as an anode.

Conduit 2 has a design capacity of 50 mgd and was originally constructed in 1911. Conduit 2 is Lockbar and riveted joint steel construction and ranges from 44 to 52 inches in diameter. The most recent interior inspection on Conduit 2 was in 2017 when the conduit was found to be in better-than-expected condition.

Conduit 3 has a design capacity of 75 mgd and was installed in 1925. Conduit 3 is Lockbar and riveted joint steel construction and ranges from 50 to 58 inches in diameter. Conduit 3 was inspected in 2018. Conduit 3 pipe between Hudson Road Intertie and Sam Barlow High School was found to be in the worst condition, with approximately 56 years remaining useful life. Cathodic protection was added to sections of the conduits beginning in 1979 and was substantially complete in 1983. Improvements to the cathodic protection system were made between 1987 and 1990. However, Conduit 2 was already 70 years old before cathodic protection was added.

Conduit 4 has a design capacity of 100 mgd and was installed in 1952. Conduit 4 is welded steel and ranges from 52 to 66 inches in diameter. There is a flexible joint every 80 feet and was constructed with thrust blocks at alignment ends. Cathodic protection has been in place on this pipe since the early 1980s when the pipe was still relatively young (about 30 years old).

PWB uses an impressed current cathodic protection system that is comprised of rectifiers, ground beds, and test stations. As of 2015, there are a total of 51 rectifiers along the conduit corridor, with more being installed. Each rectifier has a ground bed using either portions of abandoned Conduit 1, deep well anodes, or shallow, distributed anodes. Most of the cathodic protection anodes on the conduits are approximately 30 years old and are nearing maximum output. Deep well anodes and shallow, distributed anodes are increasing in resistance and output is expected to be reduced by half every 20 years. This indicates that these anodes are reaching the end of their useful life as the primary source of cathodic protection originally intended.

Figure 2-10. Conduits 2 and 4 at a bridge crossing

However, new anode installation would allow for reduced current output on older anodes, effectively extending their useful life. The approximately 150 cathodic protection system test stations are in the process of being upgraded to a new style as the existing style is obsolete.

Two large intertie facilities have been constructed in recent years and allow PWB to move flow between conduits. The Larson's Intertie facility was completed in 2002, and the Hudson Road Intertie was completed in 2006. The Hudson Road Intertie includes a 66-inch connection point for a future conduit and routes water to the City of Sandy, a PWB wholesale customer.

Lusted Hill

Lusted Hill, shown in Figure 2-11, is a water treatment facility that is located at SE Lusted Road and SE Cottrell Road, approximately halfway between Headworks and PWB's retail water distribution system. Here, ammonia is added to form chloramines, which have a longer life than chlorine and help to maintain disinfectant throughout the extensive distribution system. Constructed in 1991, the building originally served as the ammoniation facility (only). An expansion in 1996 incorporated sodium hydroxide for pH adjustment into the treatment process to provide corrosion control.

Also located on the Lusted Hill property is the surge tank for Conduit 3.

Figure 2-11. Lusted Hill facilities

Hosner Road Facilities

The Hosner Road facilities include the surge tanks for Conduit 2 and Conduit 4, a generator building, and water quality monitoring station (Figure 2-12 below).

Figure 2-12. Hosner Road facilities

2.2.7 Filtration Facility Site

The filtration facility site is a tax lot that has been owned by PWB since 1975. The site is approximately 95 acres and is located at the eastern terminus of SE Carpenter Lane in unincorporated Multnomah County, Oregon. The site consists of predominantly undeveloped agricultural land. The northeastern edge of the property along SE Dodge Park Boulevard is densely forested. Unimproved roads run along the property's northern, eastern, western, and southern boundaries as well as through the middle of the property to support agricultural operations.

Figure 1-3, from Chapter 1: Introduction, highlights that the most difficult site selection criterion to meet was a satisfactory hydraulic gradeline. The filtration facility site is at a favorable elevation for maintaining existing gravity flow hydraulics and thus passed this criterion. The site allows some flexibility with regard to facility elevation as it slopes from a higher elevation of 735 feet to a lower elevation of 690 feet.

Multiple site investigations have been performed to evaluate site suitability and potential concerns, including geologic hazards, cultural resources, archeologic resources, and other evaluations. A Phase I Environmental Site Assessment was performed in 2018. More detail is presented in Chapter 4: Planning Considerations.

2.2.8 Existing Infrastructure Summary

This section described key components of PWB's existing water system as it relates to the filtration facility and new pipelines.

Key considerations for project definition include:

- **Bull Run Watershed.** Continued protection of the Bull Run Watershed is a key component of PWB's stewardship of the Bull Run supply.

- **Bull Run Reservoirs.** There are two reservoirs on the Bull Run River that provide water storage. Each reservoir has intake towers that are used to draw water for the system.
- **Headworks:**
  - Consideration of chlorination and corrosion control treatment operations currently located at Headworks and Lusted Hill will be part of the design, commissioning, and startup of the filtration facility.
  - The pilot facility is currently located at Headworks and is anticipated to be re-located to the filtration facility as a resource for operators.
- **Conduits:**
  - There are three conduits currently in service. Conduit 2 has a design capacity of 50 mgd and was originally constructed in 1911. Conduit 3 has a design capacity of 75 mgd and was installed in 1925. Conduit 4 has a design capacity of 100 mgd and was installed in 1952.
  - The Hudson Road Intertie includes a 66-inch connection point for a future conduit and routes water to the City of Sandy, a PWB wholesale customer.
- **Filtration Facility Site:**
  - The filtration facility site is at a favorable elevation for maintaining existing gravity flow hydraulics and allows some flexibility with regard to facility elevation as the site slopes from a higher elevation of 735 feet to a lower elevation of 690 feet.
  - Multiple site investigations were performed during pre-planning to evaluate site suitability and potential concerns, including assessing geologic hazards and cultural and archeologic resources.

2.3 Existing Water System Summary

This chapter summarized the vision and infrastructure of PWB's existing system, which has been carefully managed and developed over the past century to reliably deliver high-quality drinking water to customers. The new filtration facility and pipelines will need to integrate effectively with the existing system, which requires understanding changes in demand, impacts to supply sources, as well as limitations to existing infrastructure.

- **Background.** Key considerations for project definition include:
  - The initial capacity range of the filtration facility (145 to 160 mgd) is sized to meet the upper end of the projected demand range based on serving 1 million people in 2045.
  - It is anticipated that finished water storage at the filtration facility will be available to serve wholesale customers upstream of Powell Butte in the event that water transmission is disrupted.
- **Existing Infrastructure.** Key considerations for project definition include:
  - Consideration of current chlorination and corrosion control treatment operations located at Headworks and Lusted Hill will be part of the design, commissioning, and startup of the filtration facility.
  - The pilot facility currently located at Headworks is anticipated to be re-located to the filtration facility as a resource for operators.
  - Three conduits are in service. Conduit 2 has a design capacity of 50 mgd and was originally constructed in 1911. Conduit 3 has a design capacity of 75 mgd and was installed in 1925. Conduit 4 has a design capacity of 100 mgd and was installed in 1952.
  - The Hudson Road Intertie includes a 66-inch connection point for a future conduit and routes water to the City of Sandy, a PWB wholesale customer.
  - The filtration facility site is at a favorable elevation for maintaining existing gravity flow hydraulics and allows some flexibility with regard to facility elevation as the site slopes from a higher elevation of 735 feet to a lower elevation of 690 feet.

References

Akana, *Phase I Environmental Site Assessment*, PWB, January 2018.

Bureau of Planning and Sustainability, *Growth Forecasts*, 2019. Accessed at: portlandoregon.gov/bps/76813

PWB, *Distribution System Master Plan*, June 2007.

PWB, *Filtration Plant Capacity Alternatives Technical Memorandum*, September 11, 2018.

Chapter 3

Water Quality Considerations

This chapter focuses on water quality considerations relevant to the Portland Water Bureau's (PWB's) filtration facility. These water quality considerations include historical water quality data, potential water quality risks, current and future water quality regulations, and water quality objectives. Together, these considerations inform the evaluation of treatment alternatives presented in subsequent chapters of this report.

This chapter is divided into the following sections:

- 3.1 Raw Water Quality
- 3.2 Finished Water Quality
- 3.3 Potential Future Raw Water Quality Risks
- 3.4 Current and Potential Future Regulations
- 3.5 Corrosion Control Program
- 3.6 Finished Water and Distribution System Water Quality Objectives
- 3.7 Water Quality Considerations Summary

The water quality constituents of primary focus relate to particulates, organics and their surrogates, microorganisms, select inorganic and aesthetic-based compounds, and constituents related to water stability and corrosion control. This water quality data will help inform design as PWB aims to meet more stringent water quality goals. Explanations and interpretations of water quality relationships, seasonal impacts, and other detailed water quality or regulatory nuances are included where needed to support understanding of the design and operation of the filtration facility. This information also provides context for discussions later in the chapter on optional performance programs. In certain circumstances, the ongoing pilot study will provide additional data and design criteria validation.

3.1 Raw Water Quality

Generally, the raw water quality of PWB's Bull Run supply is pristine and of excellent quality. This is demonstrated by average water quality data from 2007 to 2018 that shows low turbidity (0.6 nephelometric turbidity units [NTU]), low alkalinity (7.8 milligrams per liter [mg/L] as calcium carbonate [$CaCO_3$]), and low temperatures (9.8 degrees Celsius [°C]). Nutrients are low in the raw water with average nitrate and nitrite of 0.023 and 0.0026 mg/L, and average total phosphorus of 0.01 mg/L based on data from 2007 to 2018. Organics levels historically tend to increase in fall (i.e., the annual average total organic carbon [TOC] concentration of 1.1 mg/L increased to an average of 1.5 mg/L in fall from 2007 to 2018).

Seasonal impacts to water quality observed in the reservoirs are generally linked to turnover in Reservoir 2 and the onset of seasonal storms coinciding with fall leaf litter. At the intake, iron levels ranged seasonally with averages of 26.8 and 29.4 micrograms per liter (µg/L) in winter and spring, increasing to an average of 89.0 and 80.1 µg/L in summer and fall from 2007 to 2018. A similar seasonal fluctuation in manganese is observed at the intake with averages of 2.3 to 3.3 µg/L in the winter and spring, and 15.3 and 11.5 µg/L in the summer and fall from 2007 to 2018). In addition to seasonal changes, metals fluctuate by depth in the reservoir, typically increasing with depth. Iron ranged from 11.0 to 233.0 µg/L at the top of the water column (1 to 8 meters deep), and 17.4 to 363.0 µg/L at the bottom (27 to 32 meters deep) from 2007 to 2018. Manganese fluctuated similarly, with a range of 1.1 to 58.3 µg/L observed from 1 to 8 meters deep, increasing to a range of 1.0 to 309.0 µg/L from 2007 to 2018.

A range of algae generally typical of oligotrophic temperate waters are routinely observed in the reservoirs. The algae enumeration and speciation data identified later in this section is based on PWB's sampling and testing program, which uses different analytical approaches as compared to other literature sources. Although the algae values reported by PWB may differ in magnitude from other analytical approaches found in the literature, this historical data set can be interpreted qualitatively to inform the filtration facility design.

The following data are presented in the subsequent sections:

- Intake Water Quality Data
- Intake Algae Data
- Turbidity Data
- Reservoir 2 Water Quality Data
- Raw Water Quality Summary

3.1.1 Intake Water Quality Data

PWB monitors raw water quality at its Primary Intake Structure (described in Chapter 2: Existing Water System) to inform operational decisions. The Primary Intake Structure is located at the north end of the Headworks facility just upstream of the diversion dam. Figure 3-1 shows the conduits in the Primary Intake Structure that convey raw water from the diversion pool into the system. PWB is able to shut down the intake if water quality is not expected to meet current regulations for unfiltered water supplies, such as during turbidity events.

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Figure 3-1. Primary Intake Structure conduits at Headworks

Table 3-1 summarizes intake water quality data from 2007 to 2018 for a selection of water quality parameters, including conventional parameters like pH and turbidity, as well as metals, nutrients, algae, bacteria, and viruses.

Table 3-1. Raw Water Quality Data (2007–2018)[a]									
Parameter	**Units**	**10th Percentile**	**Average**	**90th Percentile**	**95th Percentile**	**Min–Max**	**No. of Samples**	**Method (MRL)**	**MCL or SMCL[a]**
Conventional Parameters									
Alkalinity	mg/L as $CaCO_3$	5.8	7.8	11.0	12.0	4.1–18.0	629	SM2320B (1 mg/L as $CaCO_3$)	—
Color	units	8.0	11.0	15.0	17.0	6.0–75.0	4,173	HM10048 (5.0 units)	15.0
pH	units	6.9	7.1	7.3	7.4	6.3–7.6	4,785	SM4500B (0.1 units)	6.5-8.5
Temperature	°C	2.2	9.8	15.3	16.5	2.2–18.7	4,833	SM2550B (0.1°C)	—
Total Hardness	mg/L as $CaCO_3$	5.4	7.1	8.7	9.4	3.0–12.0	30	SM2340B (0.1 mg/L as $CaCO_3$)	—
Total Suspended Solids	mg/L	0.25	0.5	1.0	1.3	0.25–16.0	662	SM2540D (0.5 mg/L)	—
Turbidity	NTU	0.3	0.6	0.9	1.2	0.1–>25[b]	4,556	SM2130B (0.05 NTU)	—
TOC[c]	mg/L	0.8	1.1	1.6	1.8	0.3–4.1	358	SM5310C (0.1 mg/L)	—
Dissolved Organic Carbon (DOC)	mg/L	1.2	2.6	4.3	4.9	1.1–6.6	20[d]	SM5310C (0.5 mg/L)	—
Ultraviolet $(UV)_{254}$	cm^{-1}	0.03	0.05	0.07	0.08	0.02–0.11	311	SM5910B (0.005 absorbance/cm)	—
Metals and Minerals									
Iron	µg/L	19.6	61.1	130.0	151.2	14.7–269.0	228	EPA[e] 200.8 (5 µg/L) starting in 2009, SM311B (100 µg/L) prior to 2009	300.0
Manganese	µg/L	1.7	9.2	24.6	28.4	1.1–55.7	247	EPA 200.8 (0.5 µg/L) after 2009, SM3111B (0.05 mg/L) in 2007, SM3111B (0.03 mg/L) from 2008–2009	50.0
Aluminum	µg/L	9.9	29.2	52.8	71.8	6.7–132.0	210	EPA 200.8 (0.5 µg/L)	50.0–200.0
Silica (Si)	mg/L as Si	3.6	4.2	4.9	5.2	2.9–5.9	334	SM4500-Si E (0.1 mg/L as Si)	—

Table 3-1. Raw Water Quality Data (2007–2018)[a]

Parameter	Units	10th Percentile	Average	90th Percentile	95th Percentile	Min–Max	No. of Samples	Method (MRL)	MCL or SMCL[a]
Nutrients, Chlorophyll, and Algae									
Nitrite as N	mg/L	<MRL	<MRL	<MRL	<MRL	<MRL–0.01	315	SM4500-NO3 F (0.005 mg/L)	1.0
Nitrate as N	mg/L	<MRL	0.023	0.050	0.059	<MRL–0.090	325	SM4500-NO3 F (0.01 mg/L)	10
Total Phosphorus	mg/L	0.005	0.006	0.008	0.010	<MRL–0.051	336	SM4500-P F (0.005 mg/L)	—
Chlorophyll	µg/L	0.2	0.7	1.3	1.4	0.03–2.6	647	SM10200H (0.01 µg/L)	—
Viable Algae	units/mL	30	647	1,371	1,532	2–3,310	540	SM10200A-F (1 unit/mL)	—
Protozoa and Bacteria									
Cryptosporidium[f]	oocysts/L	—	0.0009	—	—	0–0.18	2,518	—	—
Giardia[f]	cysts/L	—	0.0040	—	0.02	0–0.27	2,518	—	—
E.coli (EC)	most probable number (MPN)/100mL	0.5	1.3	2.0	4.1	0.5–56.0	3,630	SM9223 B QTY (MPN/100mL)	—
Fecal Coliform	colony-forming unit (cfu)/100mL	0.5	1.0	2.0	2.0	0.25–47.0	3,924	SM9222 D (cfu/100mL)	—
Total Coliform (TC)	cfu/100mL	10	59	115	179	0.5–1,733	3,630	SM9222 (cfu/100mL)	—

a. *Maximum contaminant level (MCL) or secondary MCL (SMCL). Results from 2007–2018 provided by PWB. Non-detect values were replaced with half the maximum reporting limit (MRL), unless otherwise indicated. Statistics were calculated comprehensively for source water, including all intake locations: Primary Intake Structure (labeled 2PIS in data file) and Screenhouse 3 (labeled 2P in data file).*

b. *Maximum daily turbidity was limited by the instrument setting, which varied over the dataset, therefore the value reported is ">," because the actual maximum value observed was greater than the recorded value. Maximum value reported from maximum daily turbidity at Headworks from Appendix of "Turbidity-flow relationship in the Bull Run Watershed" (Anderson, 2018).*

c. *Per OAR 333-061-0032 9d, enhanced coagulation to remove TOC is required if the source water TOC >2.0 mg/L as calculated quarterly as a locational running annual average (LRAA). As reported in this section, average TOC = 1.0 mg/L, suggesting enhanced coagulation and TOC removal is not mandatory from a regulatory perspective.*

d. *DOC is not routinely tested in the system; therefore, a limited number of DOC samples are available. DOC was collected from October–December 2010 as a part of a special sampling program at the intake (i.e., "WS WATERRF TC 04348"), followed by monthly sampling in 2011 starting in February as a part of the LT2 sampling program (i.e., "WS LT2 INTERIM INTAKE"). TOC data collected on the same day was typically lower than the DOC results, which is not expected and is likely related to filter contamination and/or clogging issues causing elevated DOC results. The DOC levels sampled in 2011 were highest in fall ranging from 1.95 mg/L DOC in February to 2.69 mg/L DOC in November.*

e. *United States Environmental Protection Agency (EPA).*

f. *Cryptosporidium and Giardia results (2010–2017) are based on a count of oocysts and cysts, respectively, divided by the sample volume. Oocyst counts for Cryptosporidium where either 0, 1, 2, or 3, while cyst counts for Giardia were 0, 1, 2, 3, 4, or 5, with sample volumes ranging from 10–55 L. In the dataset, 1.7% of the Cryptosporidium and 7.1% of the Giardia counts were >0. Values presented as "—" are 0. Non-detects were treated as 0.*

Table 3-2 shows how raw water quality varies seasonally for a selection of water quality parameters of interest to treatment considerations, including organics levels (TOC, DOC, and UV_{254}), iron, and manganese. TOC in the raw water intake increases as expected in the fall during periods of reservoir turnover, leaf shedding, and the onset of storms. Iron levels triple in summer and fall, as compared to levels in winter and spring. Average manganese concentrations are also higher in summer and fall (the summer average is approximately six times the winter average).

Table 3-2. Average Raw Water Quality Data by Season (2007–2018)

Parameter	Units	Winter[a] Jan–Mar	Spring[a] Apr–Jun	Summer[a] Jul–Sep	Fall[a] Oct–Dec
Alkalinity	mg/L as $CaCO_3$	6.2 (4.1–7.9)	6.9 (5.5–9.3)	9.6 (6.3–18.0)	8.4 (5.1–14.0)
Color	units	11.1 (6.0–75)	9.3 (6.0–26)	10.6 (6.0–20)	14.2 (7.0–33)
pH	units	7.1 (6.6–7.6)	7.0 (6.3–7.4)	7.0 (6.7–7.5)	7.1 (6.4–7.6)
Temperature	°C	5.1 (2.2–8.7)	8.9 (4.7–15.0)	14.6 (7–18.7)	9.8 (3.0–17.8)
Turbidity	NTU	0.7 (0.1– >25[b])	0.3 (0.2–1.5)	0.4 (0.2–3.0)	0.7 (0.2–4.3)
TOC	mg/L	1.0 (0.7–1.8)	0.9 (0.3–1.3)	0.8 (0.7–1.5)	1.5 (0.7–4.1)
DOC[c]	mg/L	1.7 (1.4–2.0)	1.6 (1.1–1.8)	1.4 (1.1–1.9)	3.4 (1.6–6.6)
UV_{254}	cm^{-1}	0.05 (0.03–0.11)	0.04 (0.02–0.07)	0.04 (0.03–0.08)	0.06 (0.03–0.10)
Manganese	µg/L	2.3 (1.2–15.0)	3.3 (1.5–25.0)	15.3 (1.3–45.5)	11.5 (1.1–55.7)
Iron	µg/L	29.4 (17.1–177)	23.6 (14.7–42.2)	88.7 (26.8–173)	79.0 (28.4–269)
Viable Algae	units/mL	449 (2–2800)	613 (14–1970)	(4–1970)	777 (6–3310)

a. *Results provided by PWB for 2007–2018. Non-detect values were replaced with half the MRL. Statistics were calculated comprehensively for PWB's source water, including all intake locations: PIS (labeled 2PIS in data file) and Screenhouse 3 (labeled 2P in data file). Minimum – maximum averages are shown in parentheses (Min-Max).*

b. *Maximum daily turbidity was limited by the instrument setting, which varied over the dataset; therefore, the value reported in this table is ">," because the actual maximum value observed is greater than the recorded value. Maximum value reported from maximum daily turbidity at Headworks from Appendix of "Turbidity-flow relationship in the Bull Run Watershed," (Anderson, 2018).*

c. *DOC is not routinely tested in the system; therefore, a limited number of DOC samples are available. DOC was collected from October–December 2010 as a part of a special sampling program at the intake (i.e., "WS WATERRF TC 04348"), followed by monthly sampling in 2011 starting in February as a part of the LT2 sampling program (i.e., "WS LT2 INTERIM INTAKE"). TOC data collected on the same day was typically lower than the DOC results, which is not expected and is likely related to filter contamination and/or clogging issues causing elevated DOC results. The DOC levels sampled in 2011 were highest in fall ranging from 1.95 mg/L in February to 2.69 mg/L in November. Generally, it is expected that nearly all the TOC in the raw water is in the dissolved form.*

Figures 3-2 through 3-18 below present time series data from 2007 to 2018 for a selection of parameters to characterize relevant trends in raw water quality, including alkalinity, color, pH, temperature, turbidity, organics (TOC, DOC, UV_{254}), iron, and manganese, as well as percentiles for some water quality parameters. This time series data showing the variation of water quality characteristics is of interest as it affects design and operation of the filtration facility.

Alkalinity

Alkalinity levels at the intake are summarized in Figure 3-2 and Figure 3-3. The results show a consistent seasonal fluctuation in alkalinity over the year with the lowest alkalinity typically between 4 to 6 mg/L as $CaCO_3$, and the highest around September to October from 9 to 18 mg/L as $CaCO_3$.

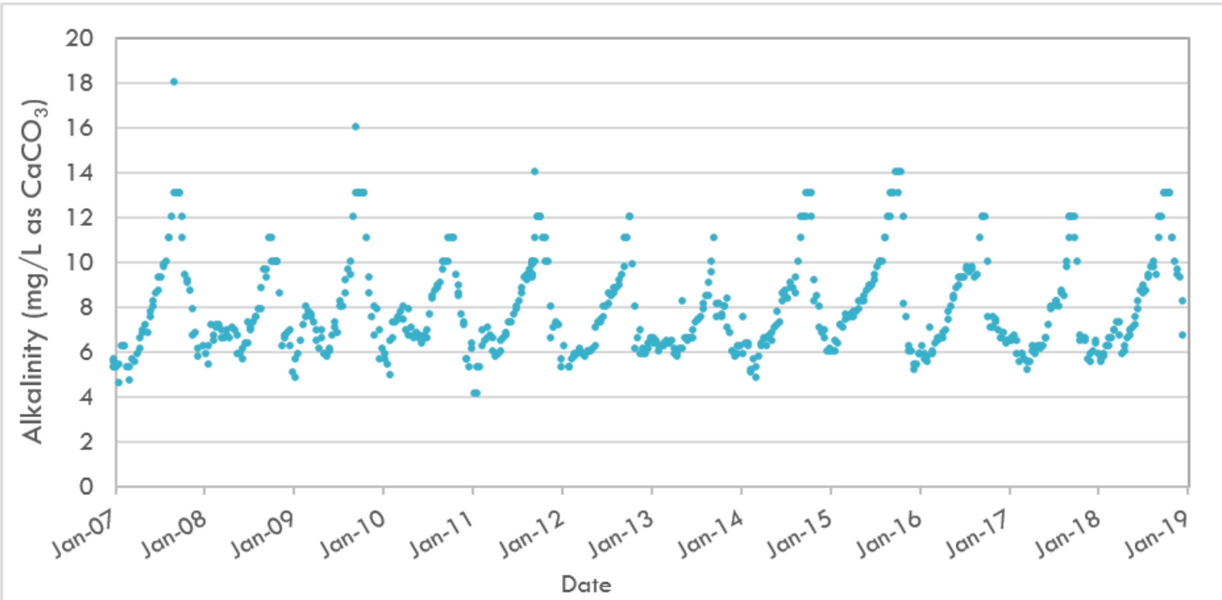

Figure 3-2. Raw water alkalinity data (2007–2018)

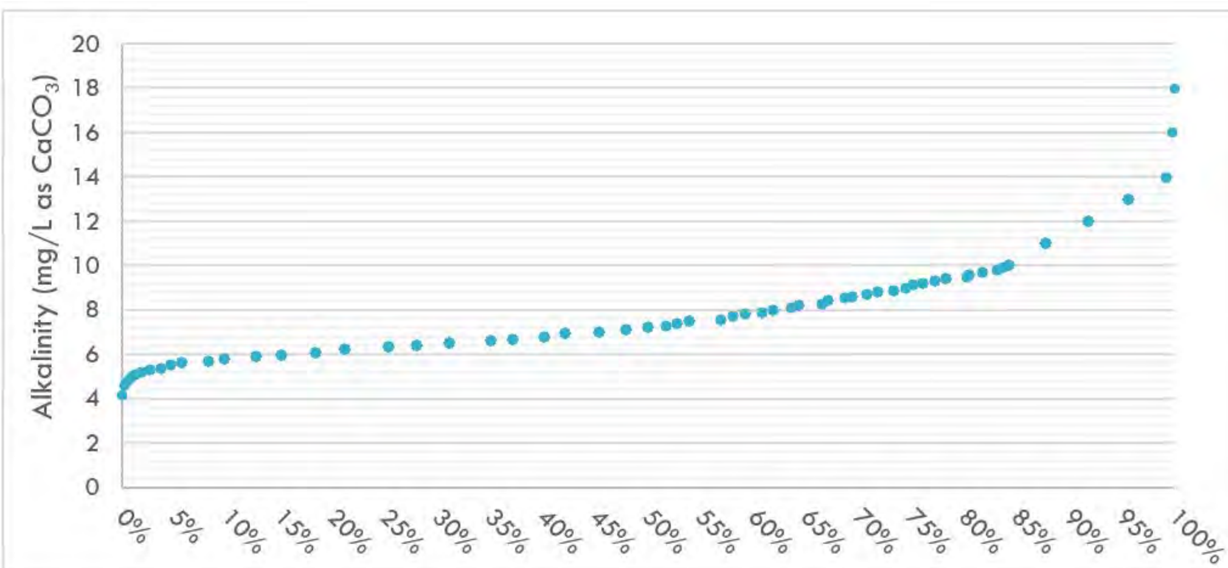

Figure 3-3. Raw water alkalinity percentile (2007–2018)

Color

Apparent color, shown in Figure 3-4 and Figure 3-5, consistently fluctuated between a low of 6 to 12 color units for the majority of the year to a high of 20 color units in late fall. The average apparent color was 11 color units, which is below the SMCL of 15 units. As seen in Figure 3-4, 87 percent of samples collected had a measured color value of 15 or less (i.e., 13 percent are above 15). As expected, color is elevated in fall during reservoir turnover, leaf shedding, and the onset of storms. Color has shown a consistent pattern except for an isolated event in January 2011, when color rose to a maximum of 75 color units. This corresponds to a record high flow with elevated turbidity (maximum of 16.4 NTU) that caused PWB to take the Bull Run supply offline in January 2011.

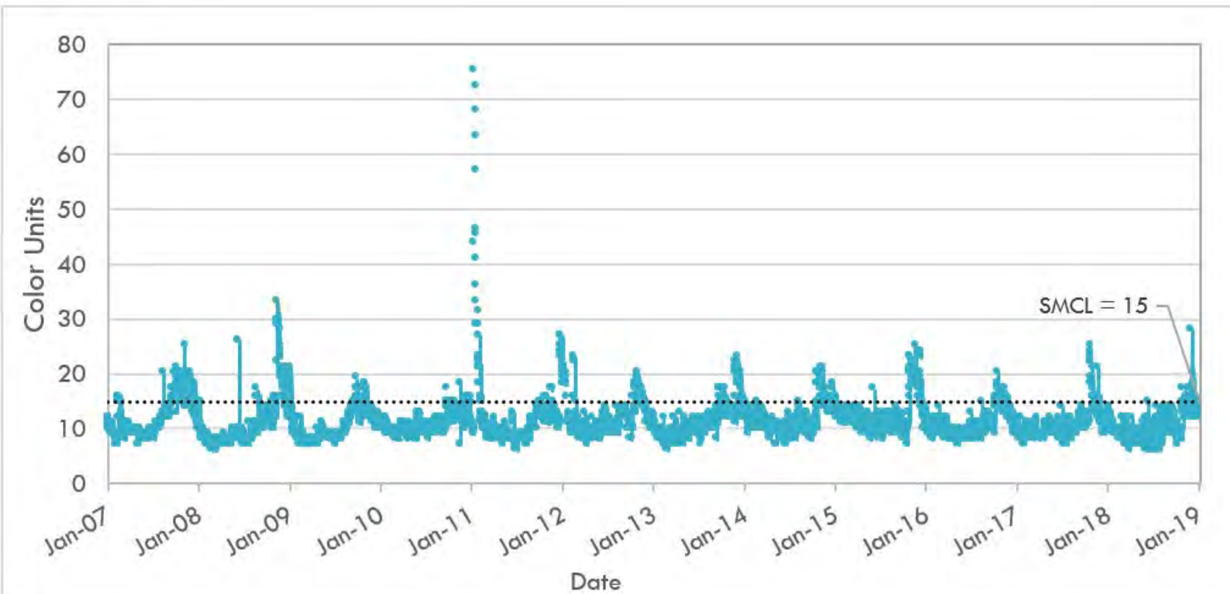

Figure 3-4. Raw water color data (2007–2018)

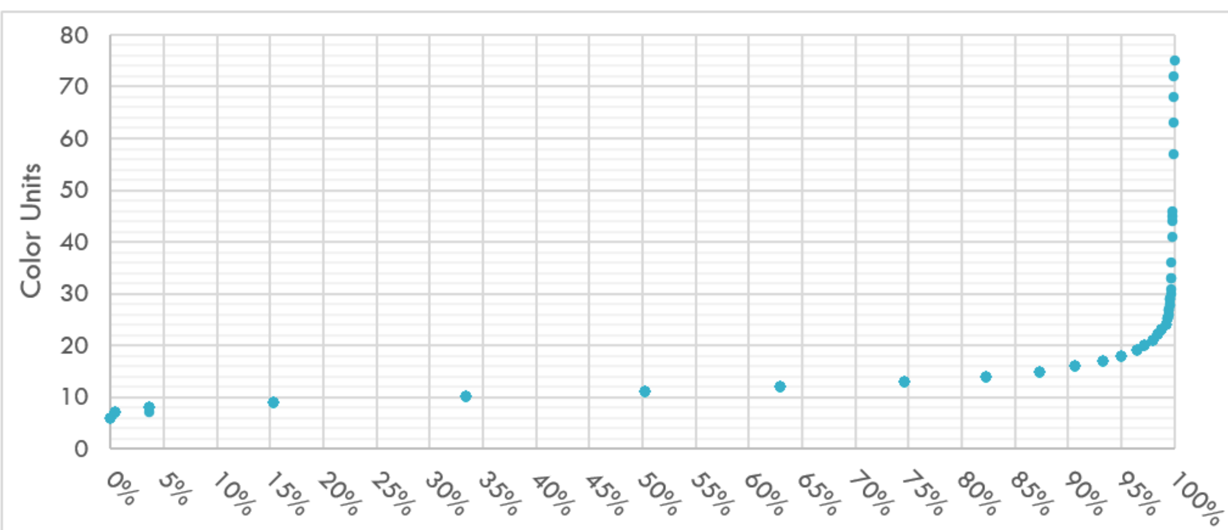

Figure 3-5. Raw water color percentile (2007–2018)

pH

Figure 3-6 shows pH levels at the intake. The average pH is 7.1, with a range from 6.3 to 7.8. All pH readings were below the high SMCL range of 8.5, and 99.9 percent of samples were above the low SMCL range of 6.5 with two exceptions in May and December of 2007 (Figure 3-7). Variation in pH is primarily due to seasonal changes in temperature and water quality, but pH also fluctuates if the water flows through the Howell Bunger Valves.

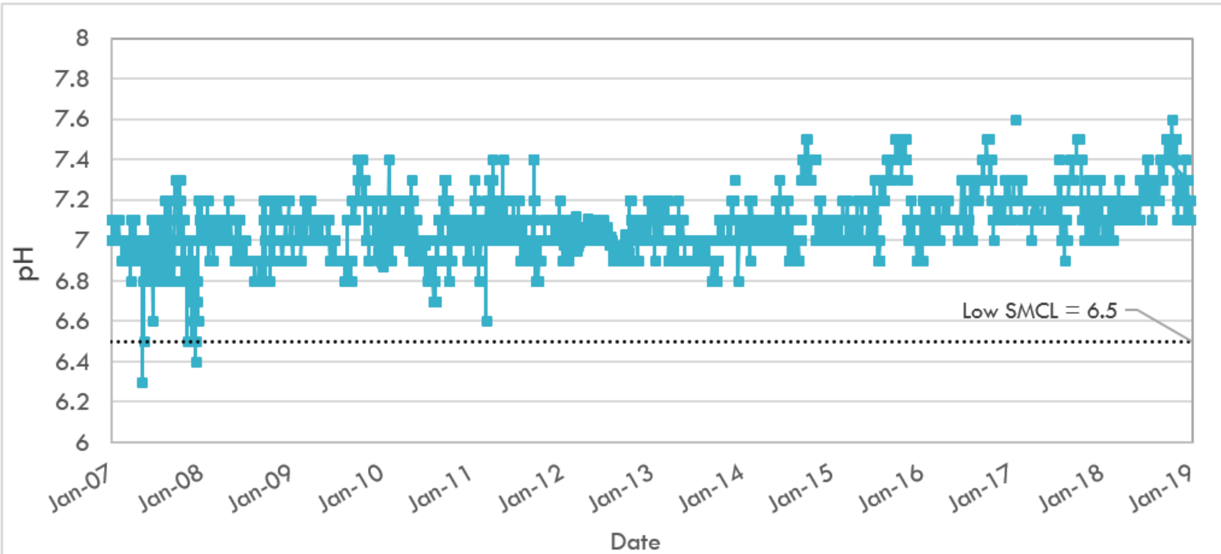

Figure 3-6. Raw water pH data (2007–2018)

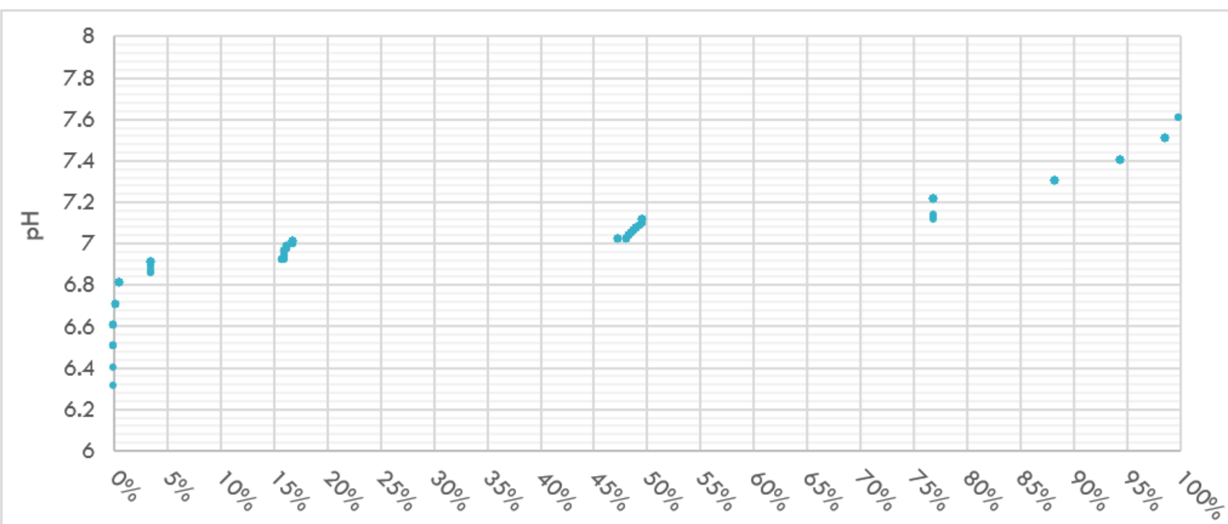

Figure 3-7. Raw water pH percentile (2007–2018)

Temperature

Temperatures at the intake fluctuated consistently with the lowest temperatures typically between January to February and highest temperatures in late September to early October (Figure 3-8). From 2007 to 2018, the temperatures ranged from a high of 18.7°C in September 2007 to a low of 2.2°C in February 2008, with an even distribution (Figure 3-9). Starting in 2014, temperature at the intake was measured continuously with online instruments. The data presented in Figure 3-8 starting in 2014 are the daily averages.

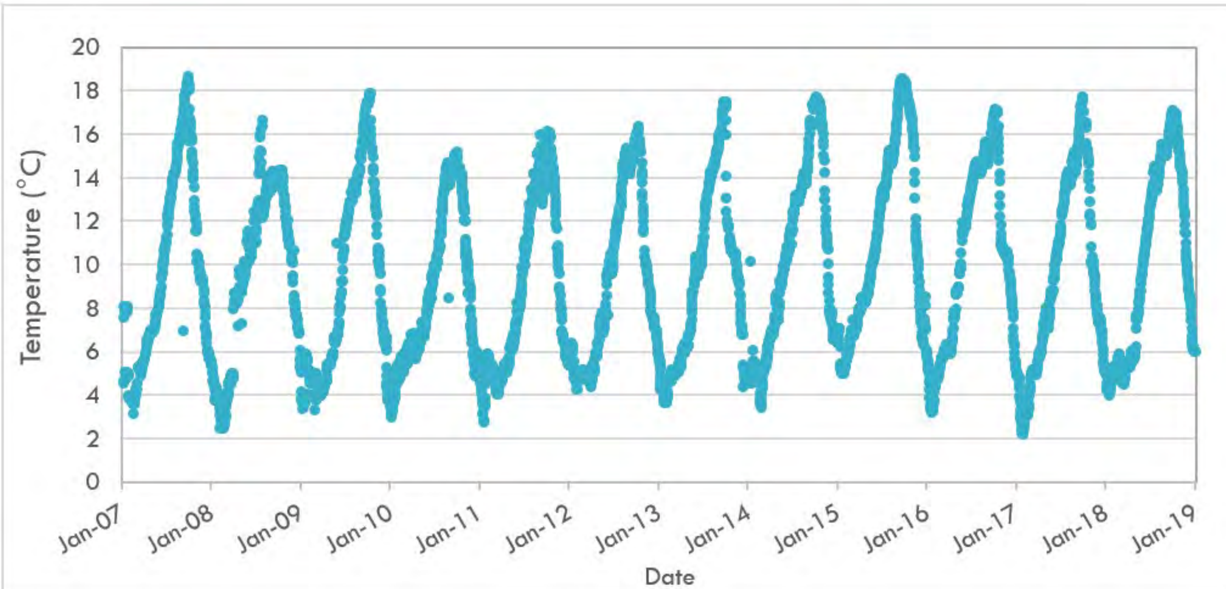

Figure 3-8. Raw water temperature data (2007–2018)

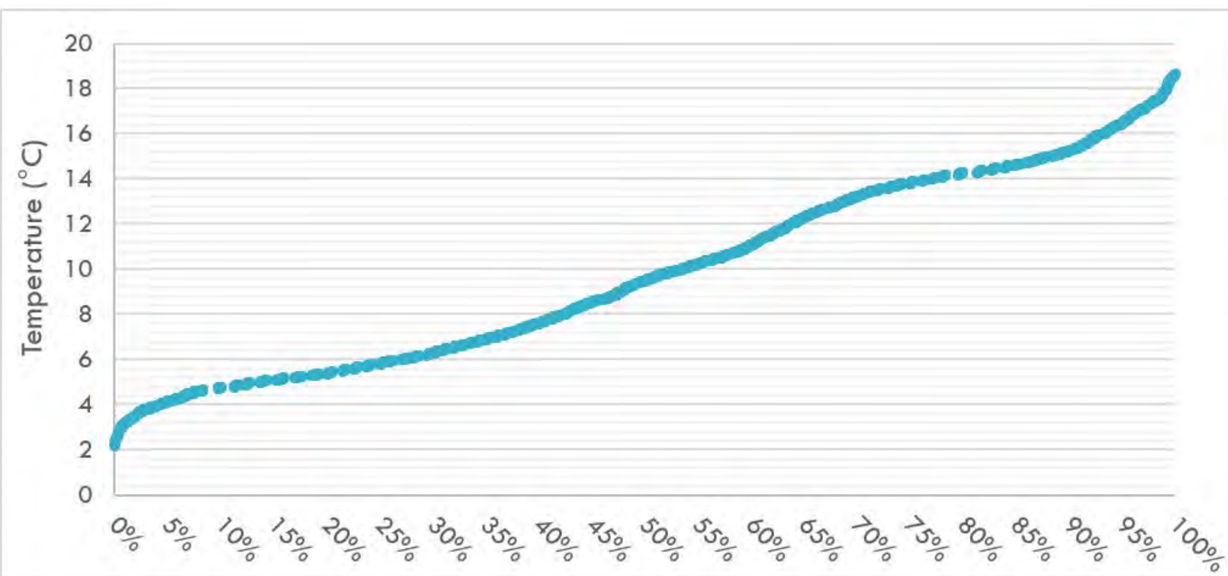

Figure 3-9. Raw water temperature percentile (2007–2018)

Turbidity

Turbidity values measured at the Headworks intake are presented in Figure 3-10, with Figure 3-11 showing corresponding percentiles. For the raw water sent to the Headworks intake, fewer than 2 percent of samples collected at the intake have turbidity values greater than 2 NTU (Figure 3-11). Outside of the shutdown periods, only one event surpassed 5 NTU in January 2011, when the sample exceeded the instrument limit with a measured value of 16.99 NTU. Turbidity values presented in Figure 3-10 are only indicative of how turbid the water entering through the intake was at the time of the sample event.

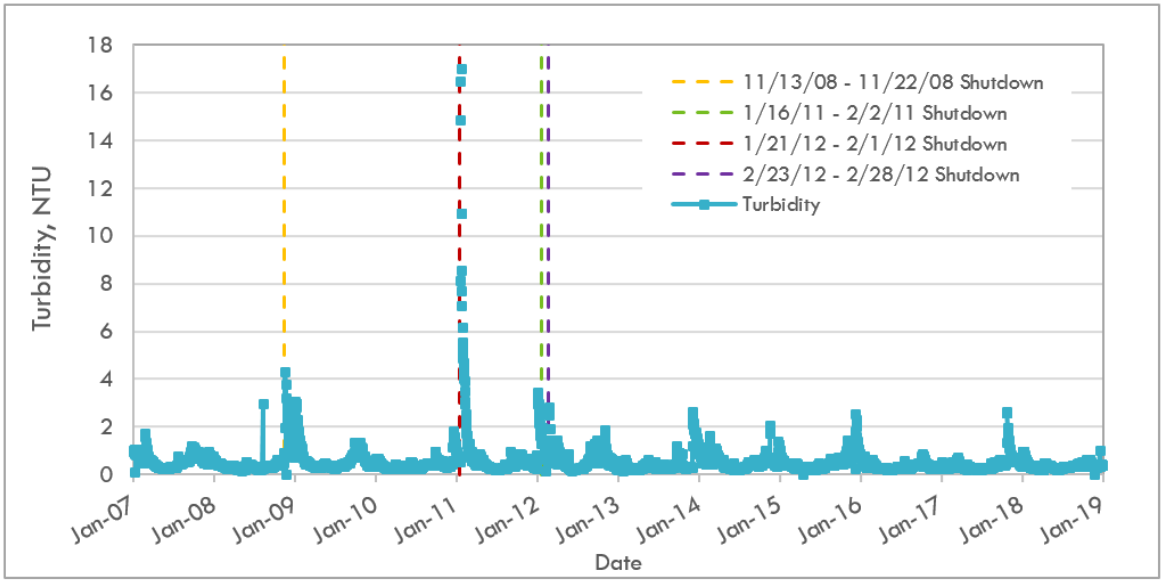

Figure 3-10. Daily average raw water turbidity data from the intake structure (2007–2018)

Daily turbidity from intake locations: Primary Intake Structure and Screenhouse 3.

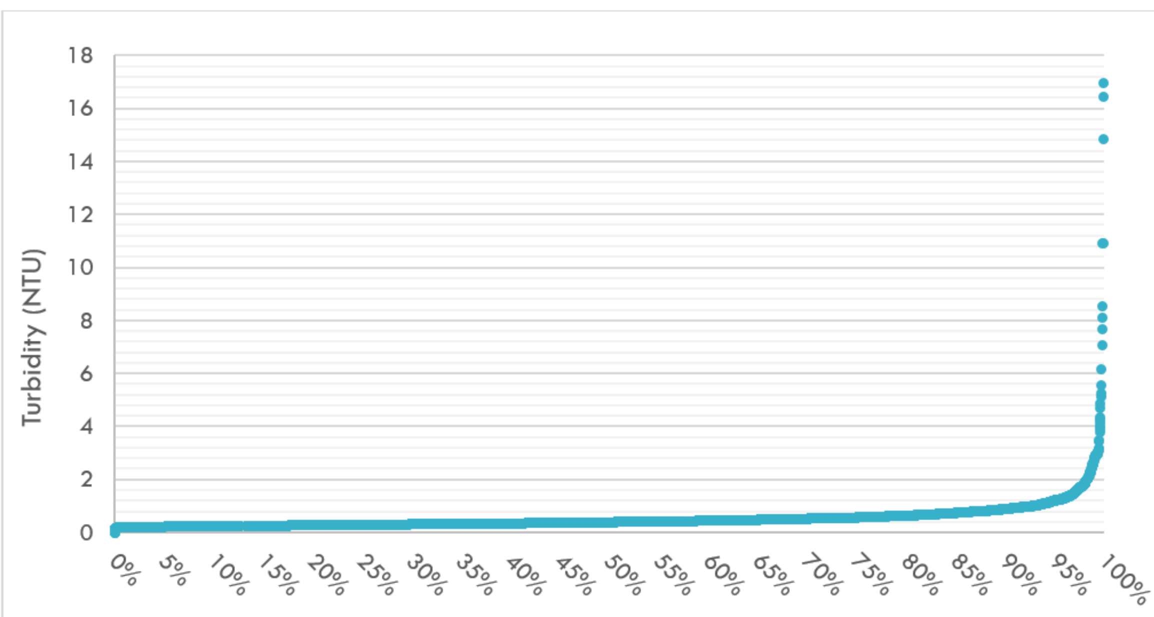

Figure 3-11. Raw water turbidity percentile (2007–2018)

During this time period, there were four events when the intake at Bull Run was shut down, and PWB switched to an alternative water source. A gap exists in the intake raw water quality data for turbidity due to these shutdowns and the placement of the intake turbidimeter, which is such that during a shutdown, the turbidity data collected is not representative of the water quality in the reservoir that triggered the shutdown. Hence, historical raw water turbidity might have been higher but was not able to be measured at the intake. Section 3.1.3 describes historical turbidity data for Reservoir 2 dating back to 1965, which is more representative of the range of turbidities expected from the Bull Run source than the intake data summarized in this section. As flow from Reservoir 2 is conveyed to the Headworks intake, water from Reservoir 2 is generally indicative of the water quality that can be expected at the intake, with the exception of the shutdown periods. The frequency of turbidity exceedances resulting in a source water change from the reservoir to the backup groundwater source is also provided.

Organics

Raw water organics data for TOC, DOC, and UV_{254} is presented in Figure 3-12 from 2007 to 2018, followed by a graph showing the abbreviated timeframe when DOC was evaluated to better correlate DOC and TOC in Figure 3-13. Note that TOC is a typical constituent directly proportional to the particulate and dissolved organic content of the water, whereas DOC refers to the fraction that is dissolved (i.e., passing a 45µm filter paper). Likewise, UV_{254} is a measure of a water sample's absorbance of UV radiation at a wavelength of 254 nm. These general constituents are considered reasonable surrogates for a certain class of precursor organic molecules that are prone to forming halogenated DBPs upon disinfection with chlorine.

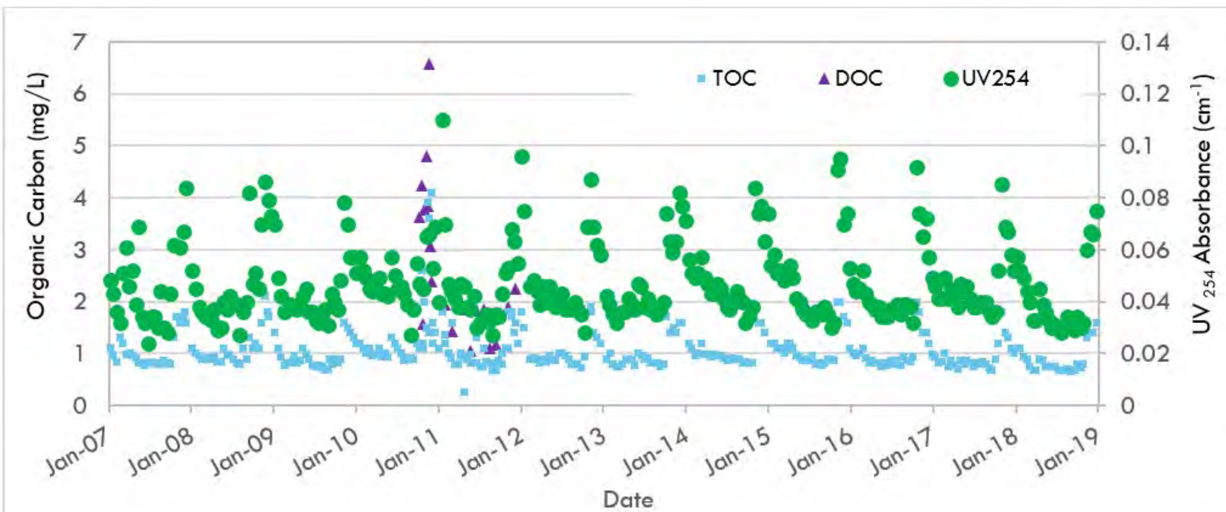

Figure 3-12. Raw water TOC, DOC, and UV_{254} data (2007–2018)

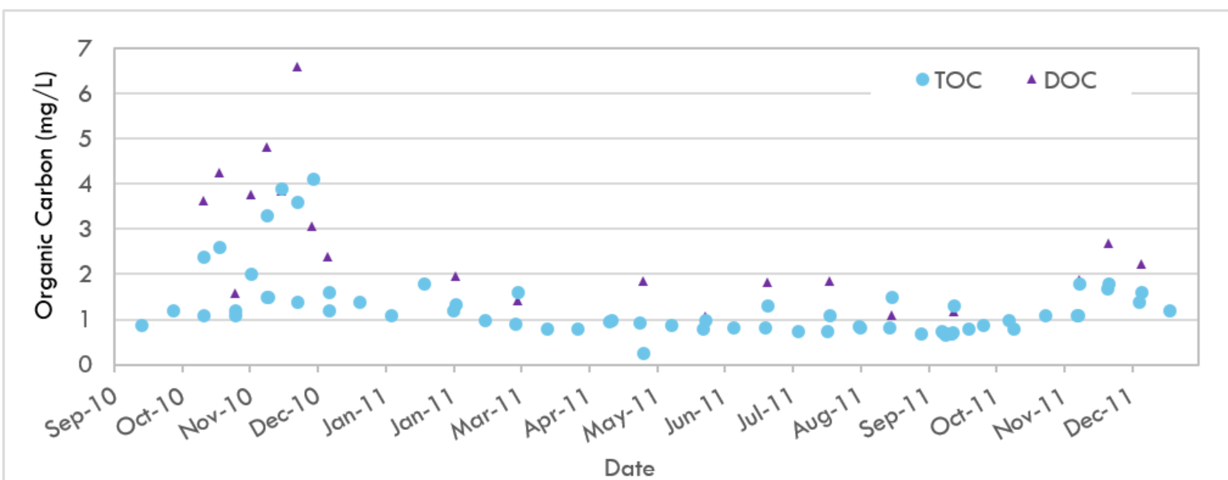

Figure 3-13. Raw water TOC and DOC data (September 2010–December 2011)

The DOC levels sampled in 2011 were highest in fall, ranging from 1.95 mg/L DOC in February to 2.69 mg/L DOC in November (Figure 3-13). DOC is not routinely tested in PWB's system; therefore, the results presented in Table 3-1 are for a limited number of samples from two sample programs. DOC was collected in fall 2010 from October to December as a part of a special sampling program at the intake (i.e., "WS WATERRF TC 04348"), followed by monthly sampling in 2011 starting in February as a part of the LT2 sampling program (i.e., "WS LT2 INTERIM INTAKE").

For TOC, more than 60 percent of samples have a measured value less than 1 mg/L, with a maximum value of 2.1 mg/L (Figure 3-14). TOC data collected on the same day was typically lower than the DOC results, which is not expected. Elevated DOC levels may be the result of errors in the DOC analyses from contamination (e.g., during some early sampling at the pilot plant facilities, contamination originated from the type of filter paper used in the analysis), or from losses during the sampling and analysis process. UV_{254} fluctuation correlated with increases in TOC and DOC seasonally. TOC, DOC, and UV_{254} were typically greatest in fall, which correlates to an increase in organic material in the reservoir at that time.

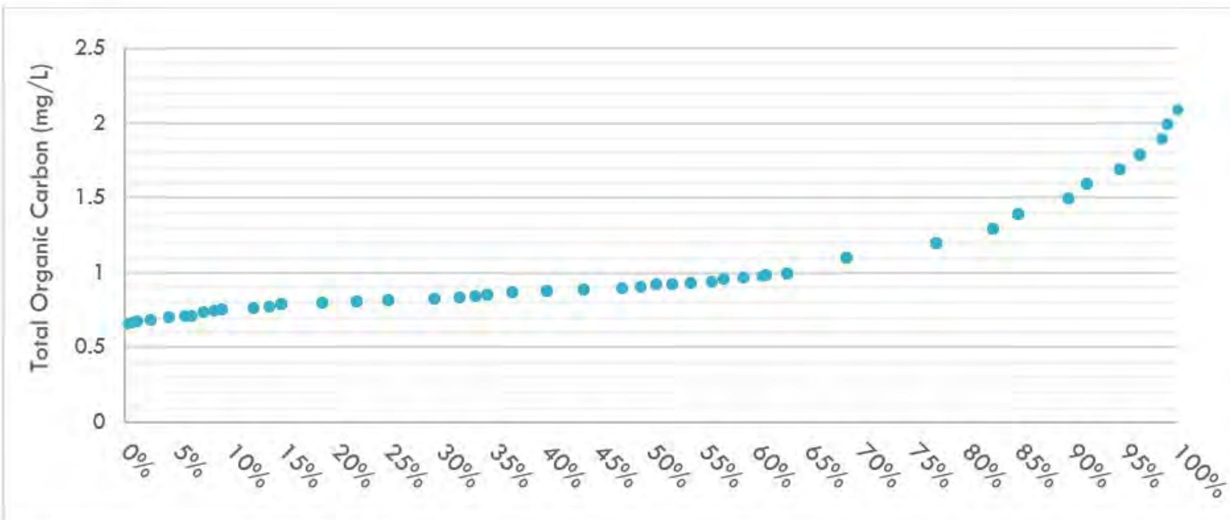

Figure 3-14. Raw water TOC percentile (2007–2018)

Iron

Iron levels measured at the intake are summarized in Figure 3-15, followed by the percentile rank in Figure 3-16. From 2007 to 2009, iron data were collected on a tri-annual basis and analyzed using the SM3111B method. All samples were below the MRL of 200 µg/L; therefore, results are presented as half the MRL (100 µg/L). Samples were collected more frequently starting in 2010 and analyzed by Inductively Coupled Plasma-Mass Spectrometry (EPA 200.8 method), with a lower MRL of 5 µg/L. Over the past decade, iron levels at the intake stayed below the SMCL, with the 90th percentile rank at 130 µg/L, and 50 percent of samples below 45 µg/L. Generally, iron tends to increase in fall during reservoir turnover, with average summer and fall levels three times the average winter and spring levels.

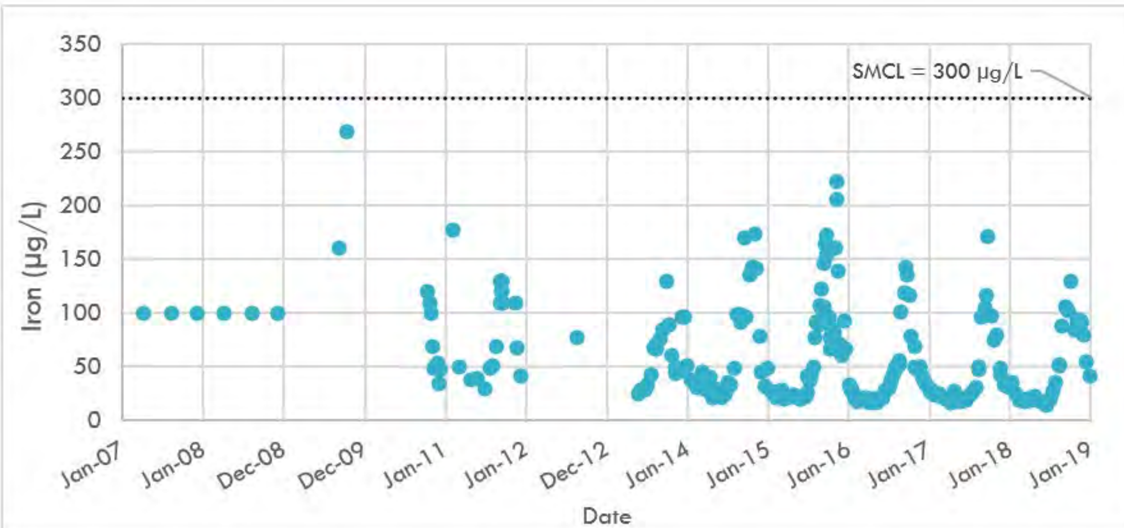

Figure 3-15. Raw water iron data (2007–2018)

Data were analyzed using method SM3111B from 2007 to 2009, with MRL of 100 µg/L. Data are reported as half the MRL if non-detect. Data were analyzed using the more sensitive EPA 200.8 method starting in 2010 with an MRL of 5 µg/L.

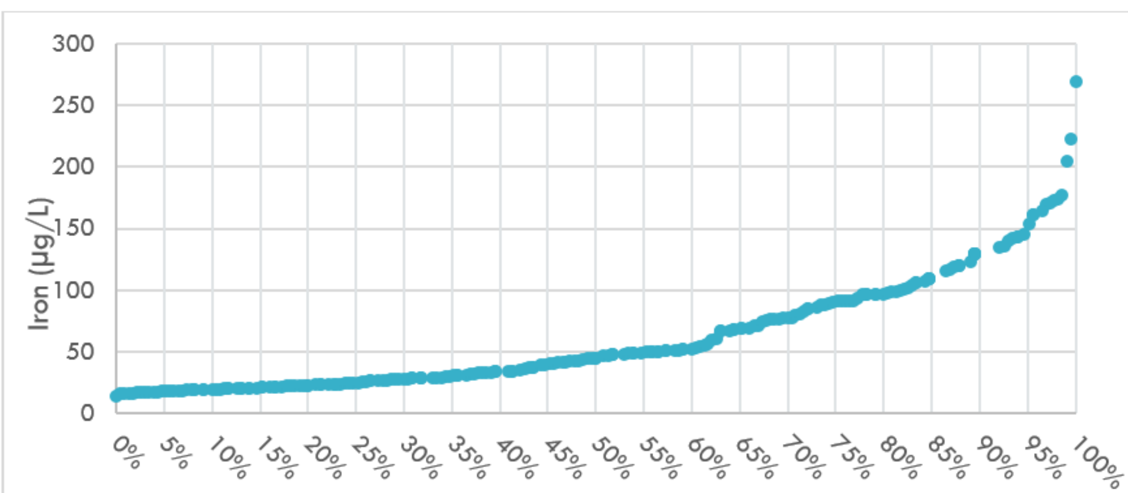

Figure 3-16. Raw water iron percentile (2010–2018)

Data collected prior to 2010 were excluded from the percentile chart, given the data were analyzed with method SM3111B, versus the newer more sensitive EPA 200.8 method.

Manganese

Manganese levels measured at the intake are summarized in Figure 3-17, followed by the percentile rank in Figure 3-18 below. Prior to 2008, samples were analyzed using the SM3111B method, with MRL of 50 µg/L. In 2008, MRL for the method decreased to 30 µg/L through 2009, presented in the figure as half the MRL. Manganese data were collected more frequently starting in 2010 and analyzed by Inductively Coupled Plasma-Mass Spectrometry (EPA 200.8 method with MRL of 0.5 µg/L). The datasets are combined in Figure 3-17.

The maximum total manganese concentration in the raw water exceeded the SMCL of 50 µg/L once with a value of 56 µg/L. Over the 11-year period, the 90th percentile level for manganese is 25 µg/L.

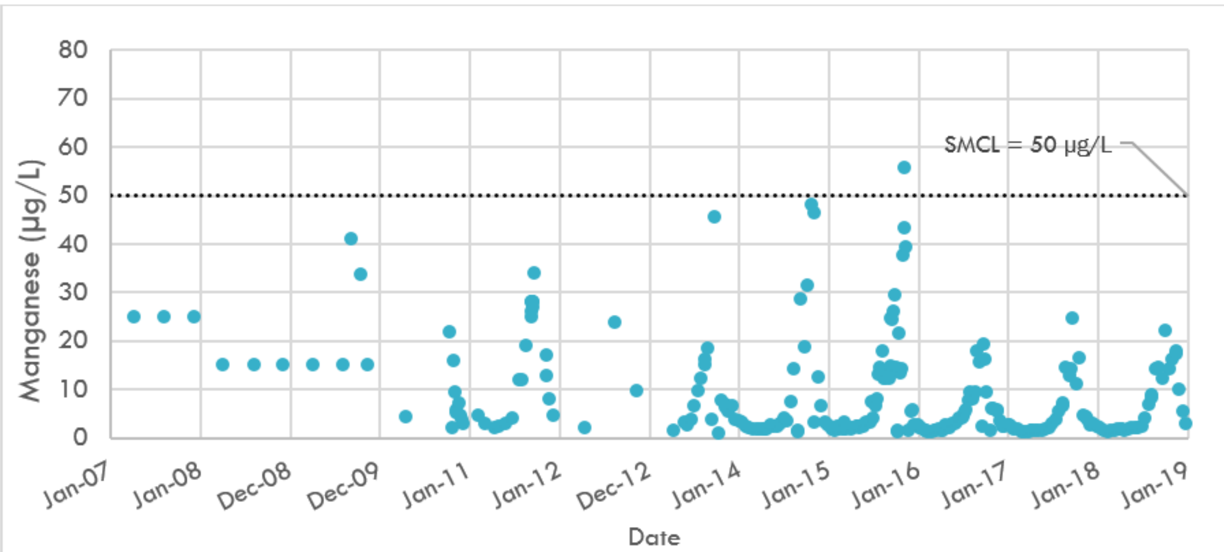

Figure 3-17. Raw water total manganese data (2007–2018)

Data were analyzed using method SM3111B from 2007 to 2009, with MRL of 50 µg/L in 2007 and 30 µg/L from 2008 to 2009. Data are reported as half the MRL if non-detect. Data were analyzed using the more sensitive EPA 200.8 method starting in 2010 with an MRL of 0.5 µg/L

For context, to prevent manganese deposit buildup in the distribution system, some systems have established a treatment goal of 20 µg/L (a future PWB treatment goal would likely be lower than this as aesthetic issues can be detected as low as 20 µg/L). Manganese increases in fall during reservoir turnover periods, as demonstrated by the summer average approximately six times the fall average. In addition, the manganese levels in the raw water are influenced by how the multi-level intake structure is operated (which feeds the diversion pool for the raw water intake). In general, manganese levels have increased in the bottom of Reservoir 2.

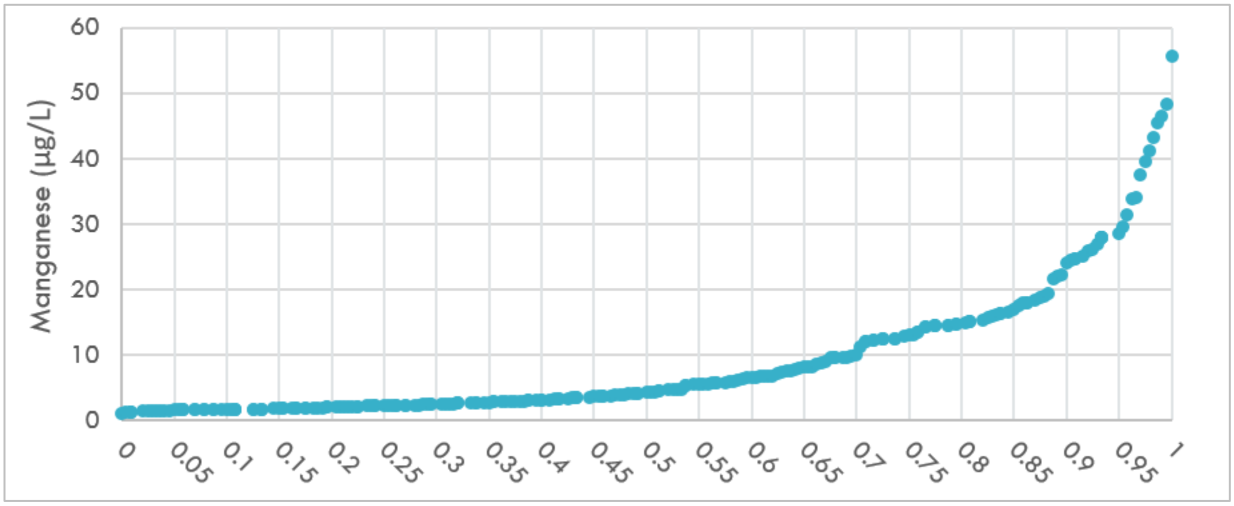

Figure 3-18. Raw water total manganese percentile (2010–2018)

Data collected prior to 2010 were excluded from the percentile chart, given the data were analyzed with method SM3111B, versus the newer more sensitive EPA 200.8 method.

3.1.2 Intake Algae Data

The presence, quantity and speciation of algae in the Bull Run reservoirs are important considerations for the filtration facility, particularly if there are algae known to cause filter clogging and/or taste and odor (T&O) issues, as well as blue green algae with the potential to release cyanotoxins and cause harmful algal blooms (HABs). Historical algae enumeration data from 2007 to 2018 are presented in Figure 3-19 in units/mL collected from the Headworks intake. Both the viable and total counts are provided from 2007 to mid-2013. The total count includes both dead and live cells, while the viable count includes the number of live cells only. Starting in mid-June 2013, results were reported for viable algae only. The average algae count over the time period is 650 units/mL, with a range of 2 to 3,310 units/mL. Note that PWB attributes the distinct step change from 2013 to 2014 to a change in the analyst and purchase of a new microscope.

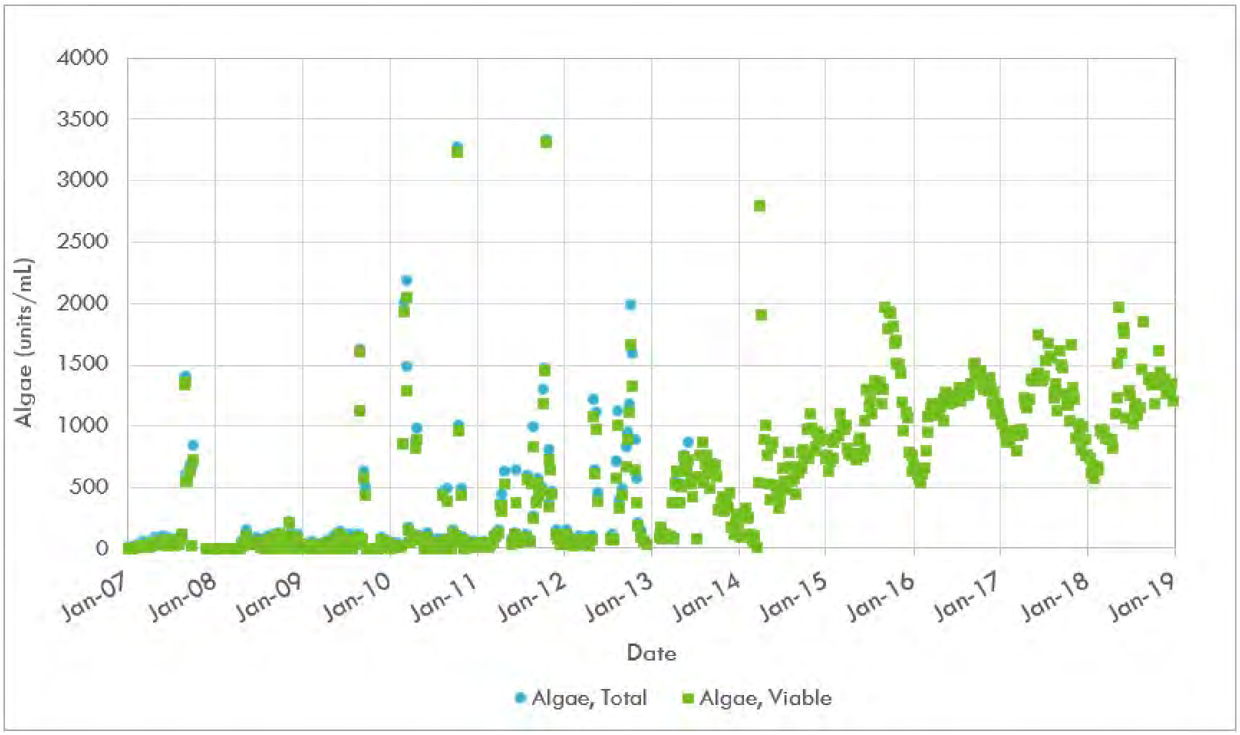

Figure 3-19. Algae data at the intake (2007–2018)

Figure 3-20 below presents algae enumeration data from June 2008 to 2018. The algae enumeration data is presented for the dominant genera, determined based on a ranking of the maximum algae count from the record. The top ten genera are included in Figure 3-20, with maximum counts around 500 to 3,000 units/mL. The maximum count was used to determine a subset of the data because water quality impacts are typically a result of algae that are present at higher concentrations.

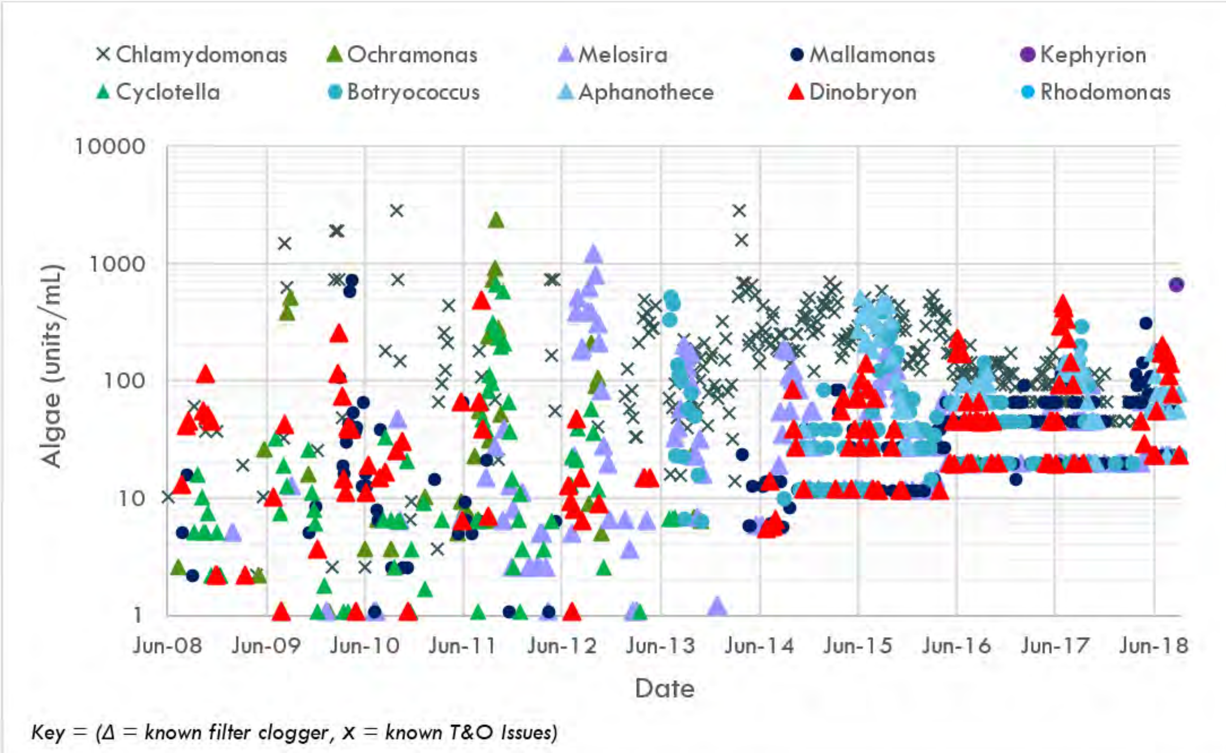

Figure 3-20. Algae enumeration data at the intake for dominant algae genera (2008–2018)

Historical speciation data from PWB LIMS database available starting in June 2008.

Key characteristics for the dominant algae genera are summarized in Table 3-3, indicating if the species has reported filter-clogging potential or T&O issues, and if there have been specific cases of filter issues related to outbreaks of the given algae species. Of the dominate algae species, two have past issues in PWB's system. *Dinobryon* is known to clog PWB customers' household filters and have T&O issues based on repeat customer complaints. In addition, *Ochromonas* was responsible for the bloom in 2005.

Melosira and *Cyclotella*, while not responsible for past issues in the Bull Run supply, are known to clog filters. Additionally, *Aphanothece* was the dominant algae in the City of Bellingham, Washington, filter clogging event of 2009, which significantly reduced Bellingham's ability to operate. The other dominant algae with known T&O issues are *Chlamydomonas, Mallomonas*, and *Ochromonas* (AWWA, 2010).

While review of potential water quality issues was limited to algae, there are also zooplankton that could be an issue in future. Specifically, *Holopedium* is a zooplankton present in the Bull Run reservoirs that is known to clog filters.

Table 3-3. Algae Species Descriptions and Known Impacts

Algae Genera	Description	Count Range (units/mL)	Filter Clogger	T&O Issues	Known Drinking Water/ Filter Impacts
Chlamydomonas	Green algae, unicellular	2–2,840	—	◆	T&O issues known from potent amines[a] ; Algae can reportedly pass through sand filters.[b]
Ochromonas	Chrysophyte, unicellular	1–2,370	◆	—	Dominant species in PWB 2005 bloom.[c]
Melosira	Centric diatom, chain-forming	1–1,220	◆	—	Listed as a filter-clogging algae in "Water Quality Indicators Guide."[d]
Mallomonas	Chrysophyte, autotrophic biflagellate unicellular	1–731	—	—	—
Kephyrion	Chrysophyte	181–665	—	—	—
Cyclotella	Centric diatom, chain-forming	1–664	◆	—	Listed as a filter-clogging algae in "Water Quality Indicators Guide."[d]
Botryococcus	Green algae, thick mucilaginous sheath	1–523	—	—	—
Aphanothece	Blue-green, colony-forming	12–510	◆	—	Dominant species in Bellingham filter-clogging event in 2009.[e]
Rhodomonas (Komma)	Cryptophytes, unicellular	6–496	—	—	—
Dinobryon	Chrysophyte, single colony-forming	1–492	◆	◆	Tied to repeat PWB customer complaints when there were increases of *Dinobryon*.

a. *T&O issues reported for Chlamydomonas (AWWA, 2010). None of the other top 10 algae species have reported T&O issues.*

b. *Testing shows Chlamydomonas can pass through filters (Whipple, 1990).*

c. *Presentation on PWB's algae monitoring program shows the bloom of 2005 was Chrysophyte (Richter, 2013).*

d. *Melosira and Cyclotella are both listed as potential filter cloggers (Terrell and Perfetti, 1991).*

e. *Bellingham experienced filter-clogging issues from Aphanothece (WALPA, 2012).*

An additional concern with algae is the release of cyanotoxins from cyanobacteria (both during periods where cells are lysed and by extracellular diffusion of these compounds). PWB collected bi-weekly samples for two of the more common cyanotoxins, cylindrospermopsin and microcystins, from July to September 2018 to comply with temporary rules Oregon Health Authority (OHA) issued summer 2018. Although not included in the timeframe of this data analysis, PWB also tested for cyanotoxins bi-weekly from June 10 to September 23, 2019, at the entry point to the distribution system (Lusted Hill Outlet) as part of monitoring required under the Unregulated Contaminant Monitoring Rule 4 (UCMR4). Results were non-detect and were measured by the enzyme-linked immunosorbent assay method, with MRLs of 0.11 and 0.3 µg/L for cylindrospermopsin and microcystin, respectively. However, PWB staff report that potential toxin-producing cyanobacteria have been found in the Bull Run Watershed at very low numbers.

Note that the Bull Run system is oligotrophic, and the water is cold for much of the year. The river and reservoirs are phosphorus limited. These conditions, in combination with constant water flows, moderate the potential for algal growth compared to many other water systems.

3.1.3 Turbidity Data

The historical compliance turbidity data from 1970 to 2018 measured at the intake are presented in Figure 3-21. The data represent maximum daily turbidity calculated based on six daily turbidimeter readings, recorded every 4 hours from 1996 to 2018. Prior to 1996, the data represents daily grab samples collected by operations staff. The data are limited by the calibration setting of the turbidimeter and by shutdown events (when there are high turbidity episodes) during which the measured turbidity at the intake is not representative of the maximum turbidity in the Bull Run reservoirs. In essence, many of the peak values of turbidity may not have been measured due to these circumstances.

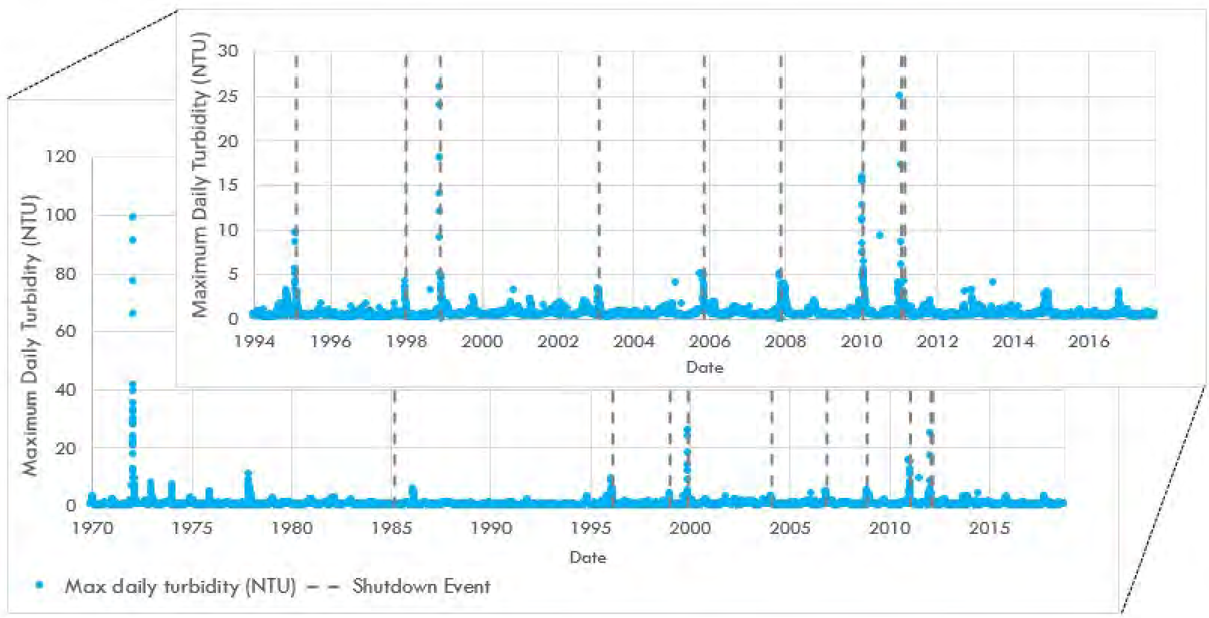

Figure 3-21. Maximum daily turbidity data (1970–2018)

Typically, a shutdown is initiated if there is indication that the raw water could have turbidity above 5 NTU, which cannot be served to customers. Past events are indicated on Table 3-4 and presented in Table 3-4, including the duration and maximum daily turbidity measured prior to the shutdown. Since 1986, the Bull Run source has experienced 10 shutdowns due to high turbidity events that lasted 4 to 22 days. The shutdown events over this timeframe represent approximately 1.2 percent of the operational time period. As a result of not capturing the actual turbidity in the reservoir during shutdown events, there is confidence in the understanding of the frequency of turbidity spikes and required shutdowns, but not in the magnitude of turbidity spikes.

Table 3-4. Intake High Turbidity Shutdown Events (1986–2012)

Start Date	Duration in Days	End Date	Maximum Daily Turbidity (NTU)[a]
2/25/1986	22	3/19/1986	5.8
2/7/1996	8	2/15/1996	>9.0
12/28/1998	5	1/2/1999	4.0
11/25/1999	19	12/14/1999	>25.0
1/29/2004	4	2/2/2004	3.4
11/7/2006	14	11/21/2006	>5.0
11/13/2008	9	11/22/2008	>5.0
1/16/2011	17	2/2/2011	16
1/21/2012	11	2/1/2012	>25.0[b]
2/23/2012	5	2/28/2012	7.4[b]

a. *Maximum daily turbidity at Headworks was limited by the instrument setting, which varied over the dataset, therefore values in this table with a ">" do not reflect the actual maximum turbidity that occurred (Anderson, 2018).*

b. *Maximum hourly turbidity data from 1996–2018 is from SCADA provided by PWB 11/15/2018. Data prior to 1996 were based on daily grab samples.*

3.1.4 Reservoir 2 Water Quality Data

Figures 3-22 through 3-26 below present data collected in Reservoir 2 upstream of the intake for manganese, iron, total nitrogen, total phosphorus, and UV_{254}. The data were separated into four depth ranges from 1 to 8 meters, 9 to 17 meters, 18 to 26 meters, and 27 to 32 meters. Prior to January 2014, raw water withdrawal was from the bottom of the reservoir. The installation of multi-level sluice gates for releasing water from various strata within the reservoir now allows PWB to adjust the withdrawal depth depending on the time of year. The withdrawal elevation location is determined by temperature changes to maintain proper temperature for downstream fish and water quality.

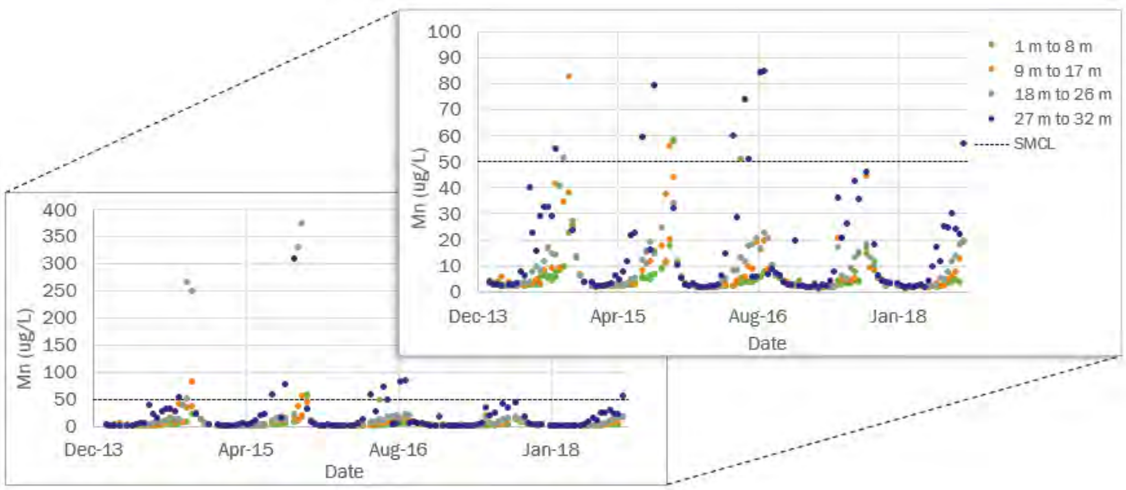

Figure 3-22. Manganese in Reservoir 2 by depth (2013–2018)

Note that SMCL refers to a standard to address issues of aesthetics, not health concerns.

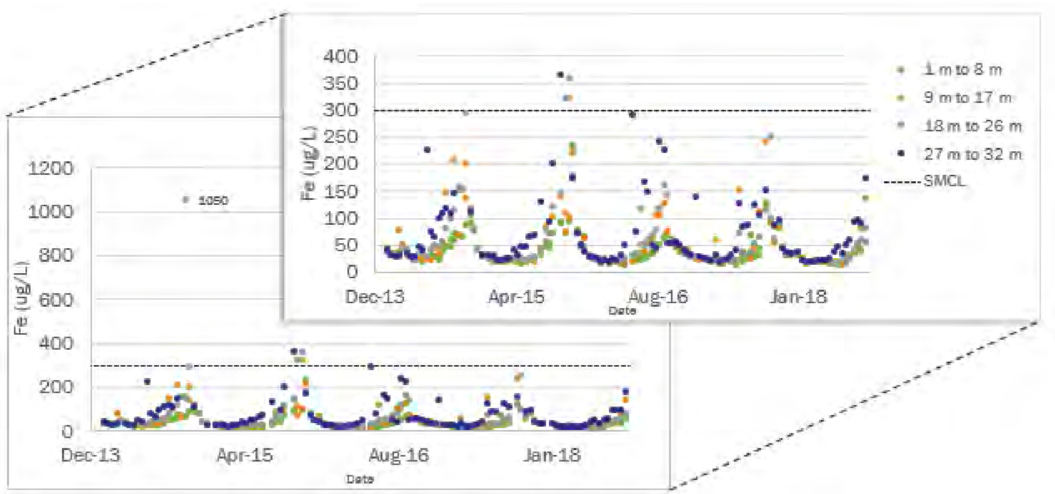

Figure 3-23. Iron in Reservoir 2 by depth (2013–2018)

Note that SMCL refers to a standard to address issues of aesthetics, not health concerns.

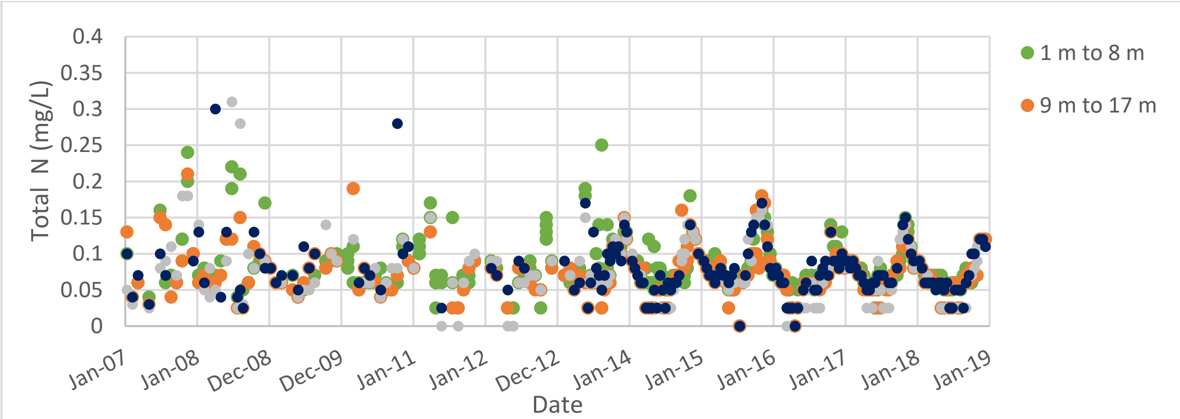

Figure 3-24. Total nitrogen as N in Reservoir 2 by depth (2007–2018)

Note that non-detect values were replaced by half the MRL (0.05 mg/L) and samples collected on 9/22/2015 with a value of 0.72 mg/L as N are not presented.

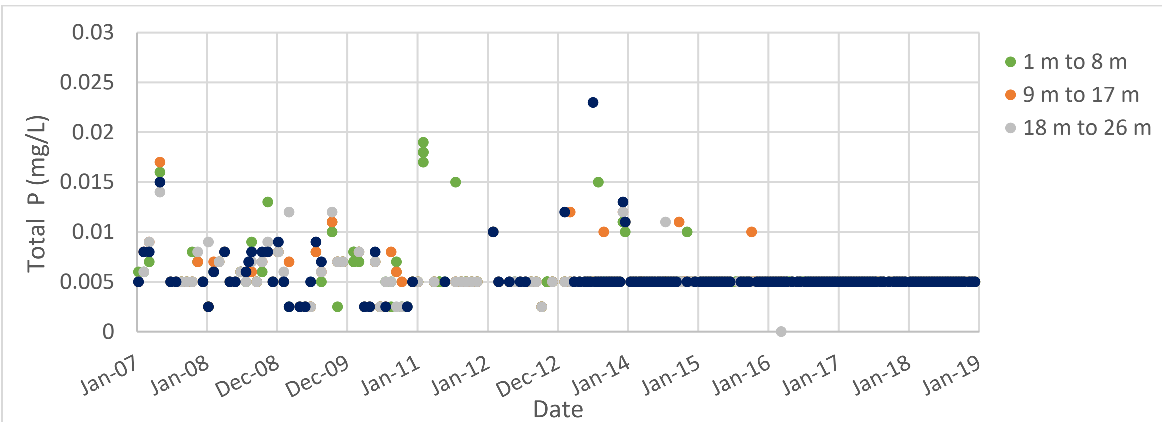

Figure 3-25. Total phosphorus in Reservoir 2 by depth (2007–2018)

Note that non-detect values were replaced by half the MRL (0.0025 mg/L or 0.005 mg/L depending on the method) and samples collected on 10/11/2010 with a value of 0.079 mg/L TP are not presented.

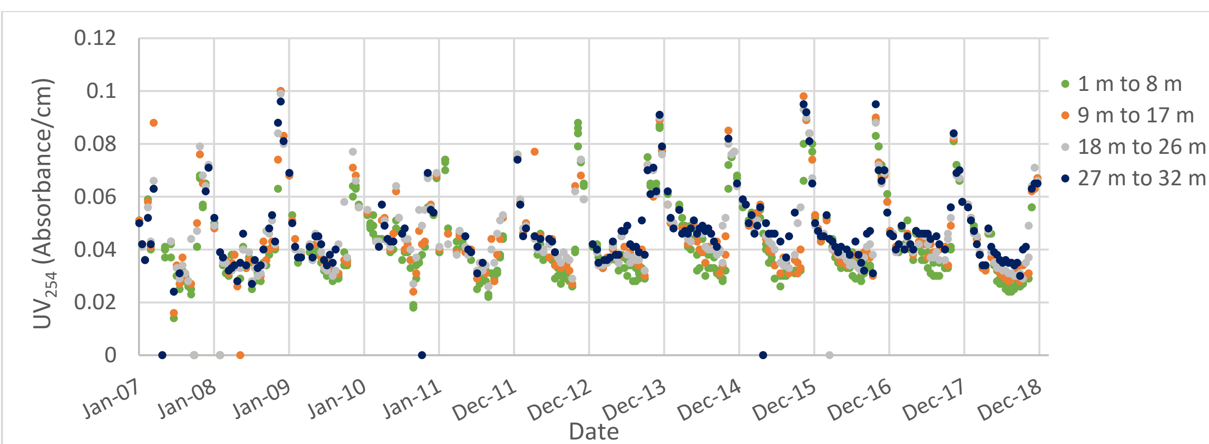

Figure 3-26. UV_{254} in Reservoir 2 by depth (2007–2018)

Some observations on the change in water quality by depth are as follows:

- Overall, water quality in the reservoir does not fluctuate consistently with depth, except during periods of reservoir stratification.
- Manganese levels vary by depth, with higher concentrations observed at the bottom between 18 and 32 meters in the summer and fall. Increased concentrations of manganese have been observed in fall, which is the result of localized depletions of dissolved oxygen that encourage a reducing environment. This trend is observed every year.
- Iron levels vary somewhat by depth, with higher concentrations observed in the deeper portion of the reservoir. Increased concentrations of iron have typically been observed in fall. However, the highest concentrations were observed in summer 2015 when at all depths the levels were greater than 300 µg/L. This observation may have been caused by a localized reservoir turnover and stirring of bottom sediments, as well as localized depletion of dissolved oxygen, which encourages a reducing environment. This type of seasonal event may occur again in the future.
- Nutrient levels are low in the reservoir at all depths. From 2007 to 2018, total phosphorus was typically around 0.01 mg/L with a maximum near 0.02 mg/L, while total nitrogen was 0.1 mg/L as N.
- Typically, algae concentrations are highest at the surface and 5 to 6 meters below the surface due to the presence of sunlight, but algae can migrate occasionally through the water column due to changes in their buoyancy at different times of the day, possibly resulting in elevated numbers at different depths.
- UV_{254} varies seasonally and by depth. The highest UV_{254} levels are observed during fall, with values up to 0.08 to 1.0 cm^{-1} around the time of reservoir turnover and when there are higher amounts of leaf litter present. UV_{254} levels drop considerably during winter and stay low through summer at levels around 0.035 to 0.045 cm^{-1}. UV_{254} levels are generally lower at the top of the water column than the bottom during the winter to summer months.

3.1.5 Raw Water Quality Summary

This section summarized raw water quality data from 2007 to 2018 for a selection of water quality parameters, including conventional parameters like pH and turbidity, as well as metals, nutrients, algae, bacteria, and viruses.

General trends in PWB's raw water are summarized as follows:

- TOC in the raw water intake increases as expected in fall during periods of reservoir turnover, leaf shedding, and the onset of storms.
- Iron and manganese levels increase in summer and fall, as compared to winter and spring levels, as a result of reservoir turnover. Iron and manganese tend to increase with depth.
- Algae is routinely detected in the intake with an average total algae count of 650 units/mL entering the Headworks intake. Known filter-cloggers that have been present in the intake in counts above 500 units/mL, but have not had reported issues in PWB's system, include *Melosira*, *Cycotella*, and *Aphanothece*. *Chlamydomonas* had the highest maximum count from the time period, and has been known to cause T&O issues in other water systems.

- The average turbidity at the intake is 0.6 NTU over the past decade, with a maximum measured turbidity of 25 NTU. One limitation of the measured turbidity is that during a high-turbidity shutdown event (greater than 5 NTU), the turbidimeters do not capture the actual turbidity level in the upstream water source due to a calibration setting to maximize data resolution at the lower values. As a result, much greater turbidity could have occurred than was measured. Since 1986, there have been 10 shutdowns due to elevated turbidity.
- Overall, water quality does not change significantly with depth, except during summer stratification. Algae is typically present at the surface in higher concentrations.
- Nutrients are low throughout the water column with an average total phosphorous of 0.01 mg/L and total nitrogen of 0.1 mg/L.

3.2 Finished Water Quality

This section provides details regarding historical finished water quality for PWB's distribution system. Data is provided for the Lusted Hill Outlet, where treated water enters the distribution system, and at regulated sample locations in the distribution system. Data presented is limited to disinfection byproducts (DBPs) and lead monitoring efforts, as those are a focus for the filtration facility and corrosion control treatment improvements.

This section is divided into the following subsections:

- Lusted Hill Outlet
- Distribution System DBP Levels
- Lead Monitoring
- Finished Water Quality Summary

3.2.1 Lusted Hill Outlet

Water quality data at the Lusted Hill Outlet, where treated water enters the distribution system, from 2007 to 2018 is summarized in Table 3-5 for a selection of parameters. The entry point is a non-compliance sampling location used to track water quality entering the distribution system.

A review of total chlorine residual at the Lusted Hill Outlet from 2007 to 2018 indicates that the target total chlorine residual levels vary seasonally. On average, the total chlorine residual is 2.0 mg/L. Since 2014, the target free chlorine residual for chlorine concentration time is 2.2 mg/L in winter and spring and 2.5 mg/L in summer and fall. The free chlorine dose at Headworks typically ranges from 3.0 to 3.5 mg/L, but peaks as high as 5.0 mg/L.

Table 3-5 also includes statistics for two DBPs, trihalomethanes (TTHMs) and haloacetic acids (HAA5s), at the entry point. DBPs form as a result of the disinfection process and are regulated for samples collected in the distribution system and reported as LRAA. The DBPs at the Lusted Hill Outlet are on average below the MCL for TTHM (80 µg/L) and HAA5 (60 µg/L), with an average TTHM of 14 µg/L and average HAA5 of 27 µg/L. The majority of DBPs form during free chlorine disinfection, prior to distribution, with some increase through the distribution system following chloramination at Lusted Hill.

Table 3-5. Finished Water Quality Data at Lusted Hill Outlet (2007–2018)

Parameter	Units	Average	90th Percentile	95th Percentile	Min–Max	No. of Samples
pH	units	8.0	8.2	8.2	7.6–9.2	7,373
Turbidity	NTU	0.46	0.77	1.02	0.0375–3.60	7,148
TOC	mg/L	1.1	1.7	1.8	0.66–1.9	34
Total Chlorine Residual	mg/L	2.0	2.4	2.5	0.9 –3.2	7,282
DBPs						
TTHMs	µg/L	13.6	24.7	31.9	4.8–34.0	38
HAA5s	µg/L	27.4	40.5	52.7	13.9–75.9	40

a. *Data provided by PWB 11/8/2018. Statistics were calculated comprehensively for Lusted Hill Outlet finished water sample locations, including TRLHC2LO, TRLHC3LO, and TRLHC4LO at the entry point to the distribution system.*

b. *Presented DBP data are for measured TTHM and HAA5 results in quarterly samples collected at TRLHC3LO, which is a non-compliance sample location for Stage 2 DBP sampling.*

3.2.2 Distribution System DBP Levels

In 2012, the Stage 2 Disinfectants and DBPs Rule (DBPR) became effective, requiring compliance monitoring throughout the distribution system for TTHMs and HAA5s and reporting of the LRAA at each location. The Stage 2 DBPR allows for reduced monitoring if the LRAA is half the MCL. The operational goal for the filtration facility was set at half the MCL as well, to provide a conservative goal for design and operation of the future facility.

Figure 3-27 and Figure 3-28 summarize TTHMs and HAA5s at five of the distribution system sample locations with the highest LRAAs from 2015 to the first quarter of 2019. Over the past 3 years, the LRAAs were generally less than half the MCL for TTHM and slightly higher than half the MCL for HAA5 at the five locations with the highest LRAAs.

Seasonal fluctuation of TTHM and HAA5 occurrence is visible in Figure 3-29 and Figure 3-30, which shows the quarterly TTHM and HAA5 results at the three locations with the highest DBP levels from 2015 to 2018. Based on this dataset, the highest month for DBPs is November when water age may be higher (linked to increase in free chlorine addition from summer to fall and seasonal drops in water demand associated with reduced irrigation) and most importantly, when water quality typically has the highest concentrations of naturally occurring organics.

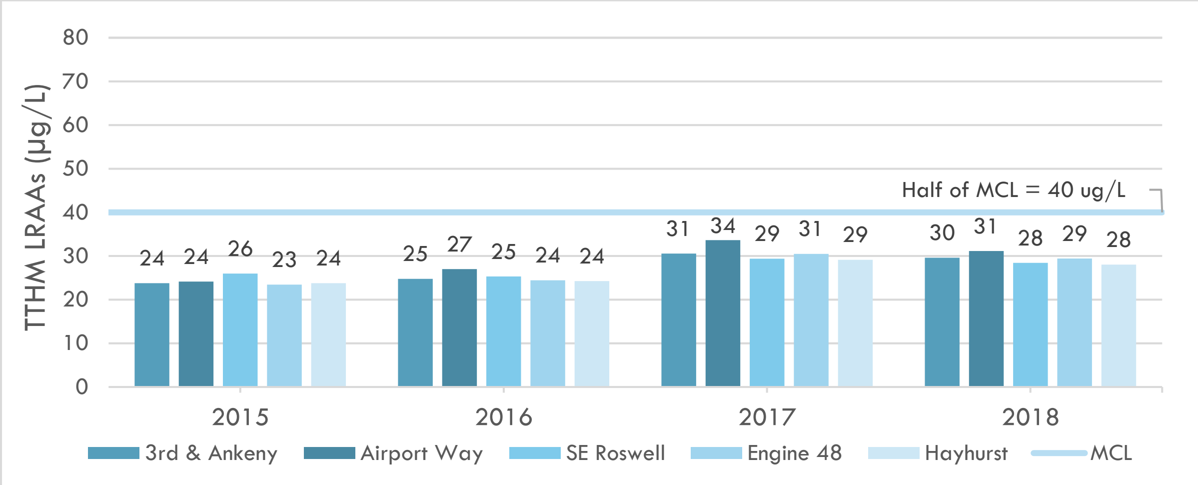

Figure 3-27. TTHM LRAA at sample locations with highest LRAAs (2015–2018)

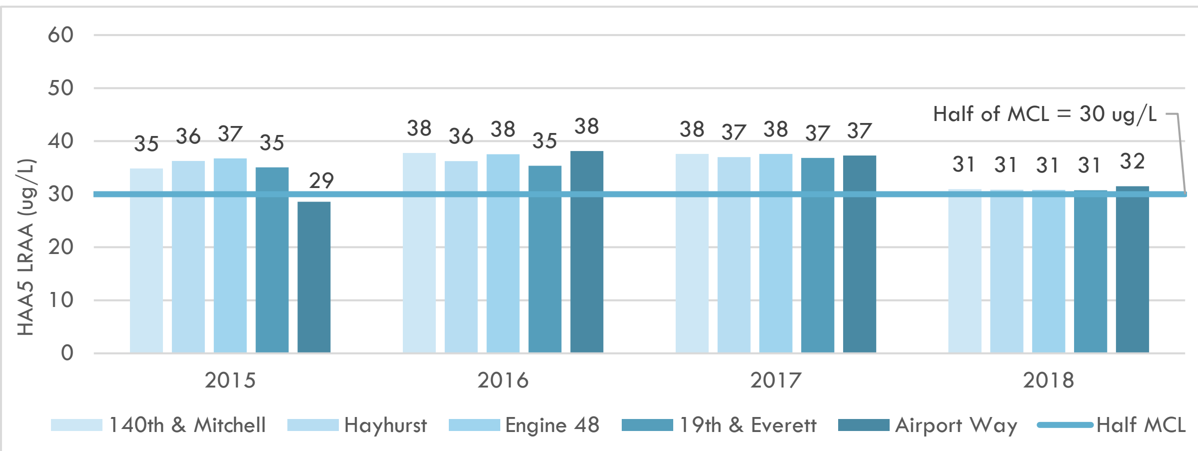

Figure 3-28. HAA5 LRAA at sample locations with highest LRAAs (2015–2018)

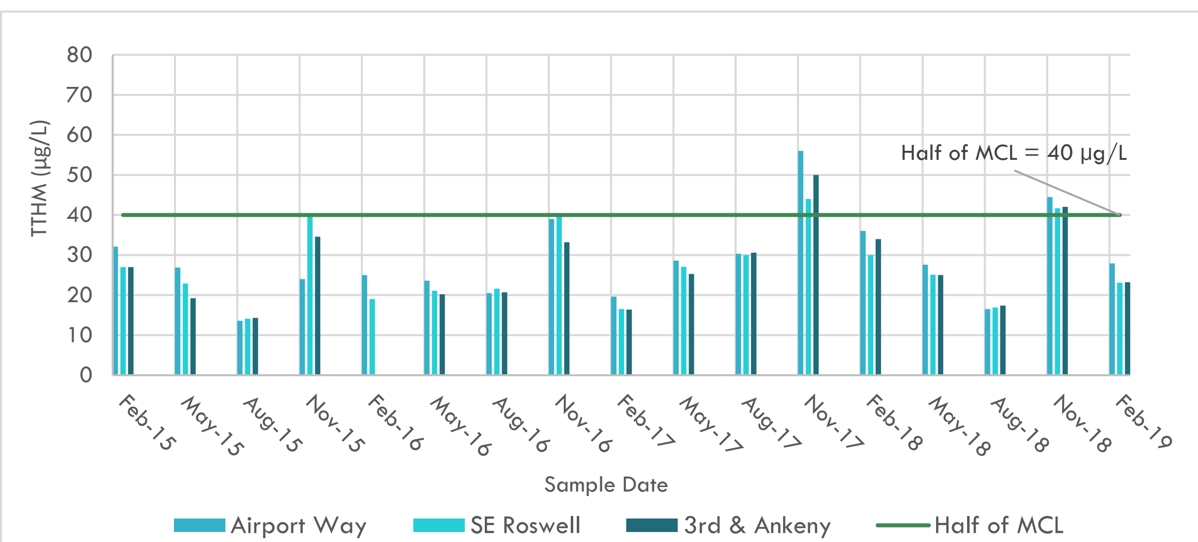

Figure 3-29. Quarterly TTHM results for top three TTHM sample locations (2015–2018)

Data not available from 3rd and Ankeny in February 2016.

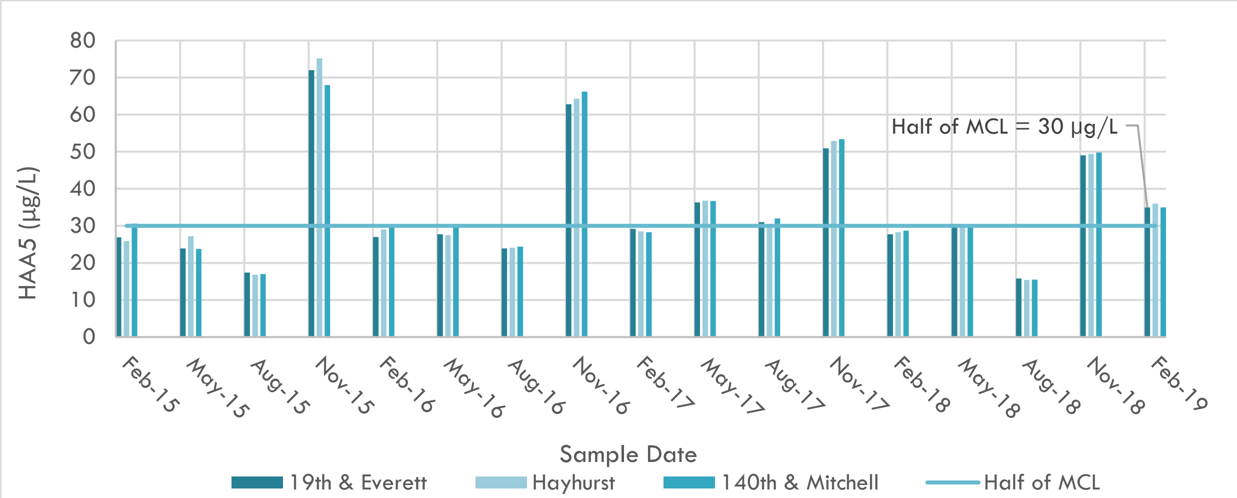

Figure 3-30. Quarterly HAA5 results at top three HAA5 sample locations (2015–2018)

In addition to monitoring for the Stage 2 DBPR, PWB participated in all four rounds of monitoring for the UCMR, with the most recent monitoring for unregulated brominated HAA species under the fourth rule (UCMR4), referred to as HAA9. Regulated HAA5s consist of dibromoacetic acid, dichloroacetic acid, monobromoacetic acid, monochloroacetic acid, and trichloroacetic acid. The four additional unregulated HAAs monitored under UCMR4 that make up the remainder of HAA9 include bromochloroacetic acid, bromodichloroacetic acid, chlorodibromoacetic acid, and tribromoacetic acid. For PWB, the dominant species of HAA is dichloroacetic acid, followed by trichloroacetic acid. Brominated HAAs are low- to non-detect as expected based on consistently measuring non-detects for bromide at the Headworks intake over the past 3 years and represent between 1 to 8 percent of the total HAA9.

Based on data from 2015 to 2018, the following conclusions about DBPs in the distribution system can be drawn:

- HAA5s are higher than TTHMs.
- HAA5 LRAAs tend to be at or slightly above half the MCL. There have been two individual sample exceedances above the MCL (60 µg/L) in November over the past 3 years.
- HAA5 levels peak in November when organics levels and water age tend to be higher.
- The majority of DBPs are formed near the Lusted Hill Outlet; there are lesser amounts formed in the distribution system after chloramination.
- Based on additional sampling for HAA9 to comply with UCMR4 monitoring, the dominant HAA species are dichloroacetic acid and trichloroacetic acid, and brominated HAAs make up between 1 to 8 percent of the total HAA9.

In addition to the conclusions summarized above based on the current DBP data, there is an additional consideration for DBPs related to future regulatory changes. If nitrogenous DBPs are regulated in future, these are the DBPs that could increase through the distribution system, influenced by factors such as ammonia addition for secondary disinfection chloramination, use of treatment chemicals such as polymers containing amine functional groups, and biological activity and nitrification within distribution system biomass.

3.2.3 Lead Monitoring

Twice each year, PWB and some of the regional water providers in the Bull Run service area test more than 100 high-risk homes to monitor the effectiveness of corrosion control for lead and copper in tap water. These high-risk homes are known to contain copper pipes and lead solder, which is more likely to contribute to elevated lead levels. These homes represent a worst-case scenario for lead in water.

If lead levels are more than 15 parts per billion (the action level established by the EPA to monitor the effectiveness of corrosion treatment) in more than 10 percent of these homes, PWB notifies its customers and performs outreach and education to those most at-risk for lead exposure. In the most recent round of monitoring, less than 10 percent of high-risk homes were above the action level for lead in water.

Figure 3-31 shows Portland joint monitoring 90th percentile lead levels at sampling locations. Note that the major change in lead levels was the result of adding corrosion control treatment in 1997, which directly reduced lead levels in finished water. Given that the lead action levels have periodically exceeded the action level of 15 parts per billion in the past, PWB will increase both the alkalinity and pH of water delivered to customers as part of the Improved Corrosion Control Treatment project to comply with the Lead and Copper Rule (LCR) by 2022.

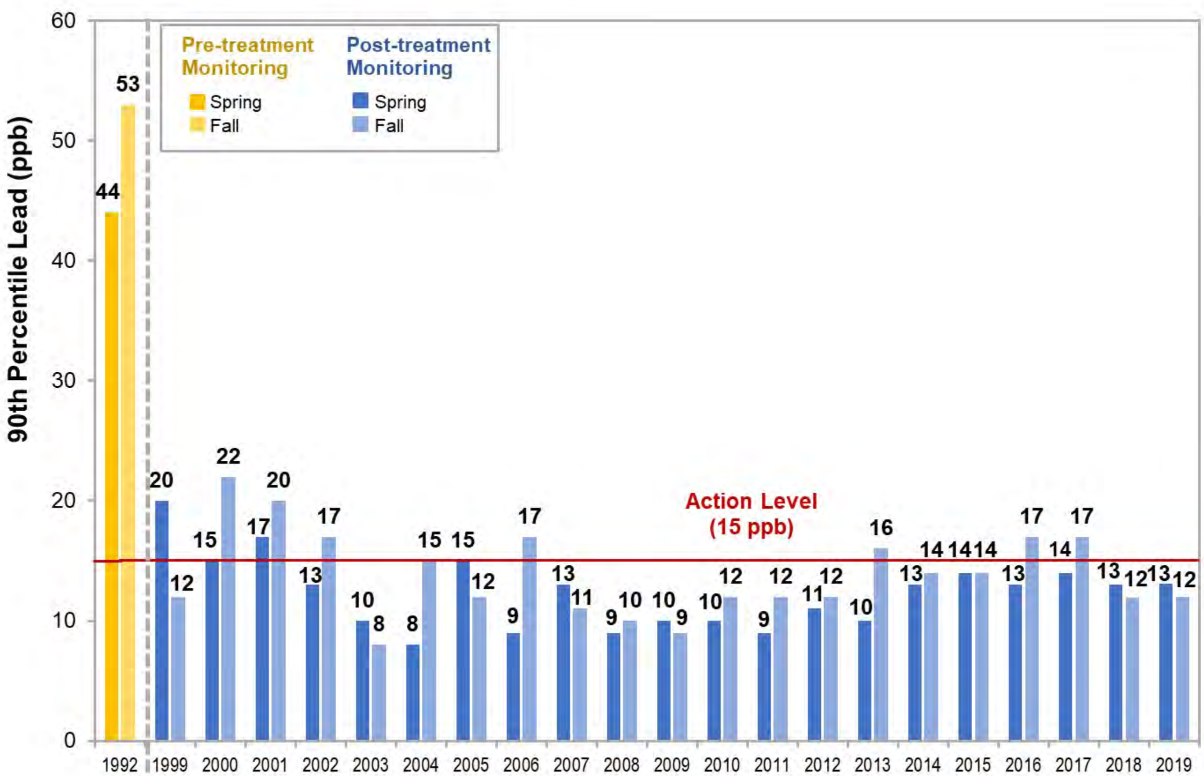

Figure 3-31. Finished water 90th percentile lead levels (1992–2019)

3.2.4 Finished Water Quality Summary

This section summarized historical finished water quality data for PWB's distribution system, focusing on DBPs and lead monitoring efforts.

The following are key characteristics of PWB's finished water quality:

- The majority of DBPs form before the entry to the distribution system during free chlorine disinfection.
- HAA5s tend to be at or above half the MCL, but still below the full MCL for the LRAA, with two exceedances above the MCL in November over the past 3 years.
- Based on data from 2015 to 2018, HAA5s are higher than TTHMs and HAA5 levels peak in November when organics levels and water age tend to be higher.
- PWB has been treating and monitoring the drinking water for lead and copper since 1997. Finished water has exceeded the lead action level twice in the past 4 years. PWB continues to improve corrosion control to reduce the potential for exceedances.

3.3 Potential Future Raw Water Quality Risks

While the Bull Run Watershed is protected, future source water quality may be negatively impacted by climate change and other natural events such as wildfires or droughts. The following sections describe key climate change trends and potential natural hazards that pose a risk to the Bull Run supply, along with the corresponding water quality impacts of such events. An overall summary of the water quality impacts is included along with a tabular summary of each of the impacts shown in Table 3-6 at the end of this section.

This section is divided into the following subsections:

- Climate Change Impacts
- Natural Hazards
- Potential Future Raw Water Quality Risks Summary

3.3.1 Climate Change Impacts

This section describes the primary climate change trends that will likely impact future water quality of the Bull Run supply.

This section includes discussion of the following content:

- Increased Air Temperature
- Changes to Precipitation

Increased Air Temperature

According to the National Climate Assessment, average Pacific Northwest air temperatures have increased by 2 degrees Fahrenheit (°F) since 1900. The Bull Run Watershed has experienced a 1.7°F increase in daytime temperatures and a 3.3°F increase in nighttime temperatures from 1907 to 2007. Recent years, including 2015 and 2018, were among the warmest on record. This warming trend is expected to continue and possibly accelerate through the next several decades and beyond, based on climate model projections for the region.

Increased air temperature will increase water temperature and could increase the potential for algae growth in the watershed. The potential for algae depends on other factors such as nutrient availability and loading (the watershed and reservoirs are currently phosphorus limited with low nutrient loading due to the geology, soils, and protected forest conditions), solar radiation, winds, algae predators, and reservoir depth withdrawal management (e.g., algae may concentrate on the surface and deeper depth withdrawal in the reservoirs may mitigate algae concentrations entering the filtration facility).

Specifically, toxic cyanobacteria (i.e., blue-green algae) tend to favor warmer water temperatures (assuming there are no limiting factors such as nutrient availability and solar radiation), which can result in the presence of cyanotoxins above EPA health advisory levels that would impact the reliability of the Bull Run source. Currently, EPA has established a 10-day health advisory for total microcystin and cylindrospermopsin. The EPA also included 10 cyanotoxins, including six microcystin variants, total microcystins, and cylindrospermopsin, as a part of the Unregulated Contaminant Monitoring Rule 4 (UCMR4). MCLs may be developed in the future for cyanotoxins.

While not all algae growth results in undue water treatment difficulties, there is a potential for increased growth of filter-clogging algae such as *Dinobryon* and other finished water quality problems related to T&O. In essence, increased air temperatures influenced by climate change may exacerbate algae growth, which can in turn impact water treatment and finished water quality. These factors need to be considered to develop a profile of possible raw water quality scenarios affecting design of the filtration facility.

In addition to algae growth, prolonged periods of warmer air temperatures could increase the risk of fire. The primary concern for Bull Run is the increased risk of a large crown fire spreading into the watershed from the surrounding area (Davis, 2018). More information is provided in the section on natural hazards.

Changes to Precipitation and Streamflow

Like other "transient rain-snow" watersheds across the Northwest, the Bull Run Watershed is expected to shift to a "rain-dominant" watershed by the end of the century, as warmer temperatures cause more rain and less snow to fall during winter months. Climate models also indicate the potential for increasing extreme precipitation in Oregon west of the Cascades (Mote et al., 2019). Intense storm events contribute heavy rainfall over a short duration (e.g., 1 day), resulting in slope instability and landslides.

PWB's hydrologic and localized climate modeling has simulated higher winter peak flows (above 5,000 cubic feet per second [cfs] on the main stem Bull Run River), which could increase the risk (frequency or severity) of storm- or landslide-driven turbidity in the watershed, both of which the Bull Run has experienced in the past (PWB, 2017). Elevated turbidity and organics are likely the primary water quality impacts to the Bull Run supply related to extreme precipitation, although the probability and magnitude of increased turbidity events over the next 30 years is uncertain (PWB, 2017). The mechanisms that can scour watershed soils and elevate turbidity levels can also contribute to elevated organic matter, particularly during the wet season.

Summer season flows in streams that feed the Bull Run reservoirs have declined over the last 40 to 50 years. Water temperatures in Bull Run streams are also warmer during the summer than they were 20 to 30 years ago in response to warmer air temperatures and lower summer streamflows. Water supply modeling through mid-century indicates the spring and summer dry season may lengthen in future, and that streamflows during these seasons are likely to decrease further (PWB, 2019). These climatic conditions may create a greater draw on reservoir storage to meet customer demand and regulatory commitments, and could increase the need to supplement Bull Run supply with groundwater frequently in the summer months and/or in greater amounts. Additionally, reservoir water temperature modeling suggests that warmer air temperatures would also increase water temperatures in the Bull Run reservoirs and downstream, making it more difficult to manage temperature for water supply and for fish habitat in the future (PWB, 2019).

3.3.2 Natural Hazards

The Bull Run Watershed is a wooded mountainous watershed in the vicinity of a large volcano (Mount Hood) and the Pacific Northwest coast, which brings about the risk of hazardous natural events such as wildfires, earthquakes, or volcanic eruptions with varying degrees of likelihood and consequence. The primary risk with these events is the potential for an influx of turbidity that could easily overwhelm the existing unfiltered Bull Run supply and could require a shutdown of the future filtration facility. Although an influx of turbidity could affect a filtered supply, impacts associated with these events are reduced or less frequent. The key types of events and the anticipated water quality impacts are summarized below.

Wildfire

Overall, the risk of a wildfire in the Bull Run Watershed is low. Bull Run's old-growth forests are naturally fire-resistant due to high amounts of precipitation, high humidity from moist westerly winds, deep organic layers that absorb and retain moisture, and moderate climatic conditions. Given the typical wet and humid conditions, and a strong fire prevention program, a high-severity fire greater than 2,000 acres is estimated to have a 1 percent probability based on historic data from the watershed (Davis, 2018). Future temperature increases and prolonged dry periods would be more conducive to wildfire in Bull Run forests. Ignition sources are, however, likely to remain unchanged. A fire burning into the watershed from drier forests during east wind events remains the greatest risk of a large fire in the watershed.

While large fires are naturally infrequent, the impact could be severe. Temperate rainforests, like those in the Bull Run Watershed, accumulate large amounts of biomass over centuries due to the natural lack of fire. When fires do burn under extreme weather conditions, they tend to be high-severity crown fires that can volatize much of the forest biomass (Davis, 2018). How the fire affects the forest floor and soils influences the effect on water quality.

Given that no large fires have occurred in the Bull Run Watershed during the 100-year history of PWB's water system, specific water quality effects from future fires are uncertain. Data from similar forest systems is also limited. Insights into the potential post-fire effects on water quality, therefore, must be drawn from forests that differ markedly from the Bull Run forest. Overall, based on the best available science, the water quality impacts of a fire are likely to be increases to turbidity, water temperature, nutrients, pH, metals such as manganese or iron, and organic carbon. These increases may in turn result in changes to water color and T&O.

Turbidity is expected to be the most challenging, costly, and persistent long-term water quality impact. Based on historic events in other watersheds, turbidity can exceed hundreds, likely thousands (even tens of thousands) of NTU in the first 1 to 3 years directly following a fire (Davis, 2018). These turbidity levels could be high enough to periodically curtail operation or even shut down the unfiltered Bull Run source as well as the filtration facility. Increases in turbidity are most likely to occur during heavy rains following a watershed fire, decreasing in magnitude and frequency with the number of storm events and in the years following the fire.

In addition, large sediment delivery downstream in an extreme event could partially fill one or both of the Bull Run reservoirs. Besides reducing supply capacity, shallower bathymetry and lack of access to lower dam gates could alter reservoir water quality by modifying limnological dynamics and/or limiting operations. The likelihood and severity of these impacts from an extreme wildfire are uncertain. Changes to pH and increases in organics, including DOC, inorganics, and nutrient concentrations from wildfire are also possible. These changes would likely be shorter duration than turbidity changes, potentially persisting for 1 to 2 years following the fire. There are some cases where TOC tapered after an event but stayed as high as 5 mg/L for years, necessitating addition of powdered activated carbon, as was seen in Santa Barbara after the 2007 Ventura Thomas Fire. Increases in these constituents are also most likely during rain events following a fire. Dissolved carbon is a concern because of an increased potential for the formation of DBPs resulting from chlorine reacting with dissolved carbon.

Changes to nutrients, pH, and organic carbon can affect the color and T&O of the water, and could result in increased HABs and formation of DBPs (Davis, 2018). Since the Bull Run reservoirs are phosphorus-limited, increases in phosphorus might increase HABs, when combined with warmer air and water temperatures. The risk of HABs is partially mitigated because increased nutrient loading is more likely during wet-season storm events when water temperatures are cooler and algal growth is slower.

Pathogens might also increase following a fire as a result of increased sediment loading and changes to the organic layers on the forest floor. Longer-term changes to pathogen levels are less certain. Wildlife are the key contributor of pathogens in the Bull Run Watershed. How a wildfire might affect post-fire wildlife species abundance and densities and subsequent pathogen levels is uncertain.

Note that a fire in an adjacent watershed could also impact the water quality of the Bull Run supply by depositing airborne ash and smoke particles. Although the extent of wildfire effects on the Bull Run Watershed is unclear, it is anticipated that a filtered source would significantly improve PWB’s ability to respond to, mitigate, and recover from these impacts.

Seismic Events

Earthquakes in the Pacific Northwest occur in response to active convergence of the Juan de Fuca oceanic plate and the North American continental plate in the Cascadia Subduction Zone (CSZ). CSZ megathrust events are considered to have the most hazard potential, generating large earthquakes with magnitudes ranging from 8.0 to 9.2 depending on rupture length. The recurrence intervals for CSZ events are estimated at approximately 500 years for the mega-magnitude full-rupture events (magnitude 9.0 to 9.2), and 200 to 300 years for the large-magnitude partial-rupture events (magnitude 8.0 to 8.5).

Current research indicates a 16 to 22 percent chance of an earthquake with a magnitude of 8.5 on the CSZ in the next 50 years (Goldfinger et al., 2019). In the event of a major earthquake, water quality could be impacted by a dam failure or catastrophic landslide, resulting in elevated levels of turbidity in the source water and potential damage to Headworks.

Mount Hood Eruption

The Bull Run Watershed is within Mount Hood's volcano hazard zone for volcanic ash and regional lava flows. An eruption could result in deposits of volcanic ash in the reservoirs and volcanic sediment entering the Bull Run River headwaters and washing downstream in the months to years after the eruption (USGS, 2014).

Longview, Washington, experienced that impact as a result of the Mount St. Helens eruption in 1980. Two decades after the eruption, sedimentation pushed water around an existing dam and carried volcanic ash down the Cowlitz River, where Longview draws its drinking water. Volcanic ash is abrasive and caused significant infrastructure damage, enough to drive Longview to change its water source. Mount St. Helens, which is approximately 52 miles north of the Bull Run Watershed, is not viewed as a significant threat to the Bull Run supply. There was minor ash deposition to the Bull Run River at the time of the 1980 eruption, but no significant impact to raw water quality.

A Mount Hood eruption could similarly impact the Bull Run supply, though it is difficult to quantify the extent of these impacts. The Lolo Pass ridge between Mount Hood and the Bull Run Watershed may protect the Bull Run supply from direct debris flows; however, the supply could be impacted by ashfall resulting from an eruption. This may reduce the useful life of assets or increase asset maintenance.

3.3.3 Potential Future Raw Water Quality Risks Summary

This section identified potential impacts to future raw water quality resulting from climate change and natural hazards. These potential impacts are summarized in Table 3-6 below. Through discussion at the Water Quality Goals workshops with the program team on November 27, 2018, and January 15, 2019, PWB identified the following high-priority water quality impact scenarios that the filtration facility needs to be designed to address:

- Turbidity from wildfires and other sources.
- Climate change-related temperature increases contributing to the potential for increased algae growth (assuming no limiting nutrients).
- Organics, nutrients, and metals from wildfires.

Table 3-6. Future Risk Impacts to Water Quality		
Potential Event	**Water Quality Change**	**Potential Impacts**
Air Temperature Increase	Potential for cyanotoxins from increased cyanobacteria blooms (e.g., anatoxin and microcystin) and related T&O issues leading to potential to trigger health advisories.	Maintenance issues due to filter-clogging algae; increased wildfire risk.
Precipitation Change	Potential for increased turbidity, inorganics and nutrients, and elevated DOC.	Debris flow from landslide or increased streamflow resulting in increased turbidity or infrastructure damage.
Wildfire	100s to >10,000 NTU turbidity 1–3 years post-fire, followed by periodic spikes Elevated turbidity, TOC, metals, and nutrients.	Temporary filter rate reduction or facility shutdown and use of alternate supply. Destabilized hillslopes in burned areas years after the fire causing landslides, which can bring sustained high turbidity and large woody debris downstream.
Earthquake	Potential for landslides that could result in elevated turbidity, algae, nutrients, and metals.	Infrastructure damage resulting in extreme water quality upset leading to service disruption, specifically dam crack potentially demolishing Headworks.
Mount Hood Eruption	Elevated turbidity and deposits of volcanic ash.	Increased turbidity could require periodic shutdowns of the filtration facility and use of an alternative supply.

3.4 Current and Potential Future Regulations

This section reviews current water quality regulations and the development of these rules. The EPA develops and implements drinking water regulations under the Safe Drinking Water Act (SDWA) of 1974 and subsequent 1986 and 1996 amendments. The SDWA regulates public drinking water systems as well as source waters such as rivers, lakes, reservoirs, and groundwater wells. States are able to develop their own regulations but must adhere to the EPA's minimum requirements.

In some cases, states develop more stringent requirements. Oregon's more stringent regulations are highlighted below. Per the SDWA requirements and the Code of Federal Regulations 40 CFR 142, Oregon has primary responsibility (primacy) to enforce EPA mandated rules 40 CFR 141 to 143 for drinking water, including protection of source water quality and design and operation of water system facilities. OHA is the state agency responsible for compliance with these regulations, and the OHA Public Health Division program of Drinking Water Services is charged with rule enforcement. Rules specific to drinking water are codified as Oregon Administrative Rules (OAR) to assist the OHA in implementing and interpreting their statutory authority. The relevant OAR for this project is OAR 333-061, which provides the regulations pertinent to water quality and to the design and performance of the filtration facility.

This section is divided into the following subsections:

- Current Federal and State Regulations
- Potential Future Federal Regulations
- Current and Potential Future Regulations Summary

3.4.1 Current Federal and State Regulations

This section provides an overview of federal regulations (adopted into OAR-333-061) focusing on source water, treatment, disinfection and DBPs, lead and copper, and microbial protection. This section is not intended to address all regulations in detail, but instead provides information pertinent to PWB's system.

This section includes discussion of the following topics:

- Surface Water Treatment Rules (SWTR)
- Stage 1 and 2 DBP Rules
- Lead and Copper Rule (LCR)
- Total Coliform Rule (TCR)
- Filter Backwash Recycling Rule
- Additional State-Specific Regulations

Surface Water Treatment Rules

Table 3-7 presents an overview of the SWTR progression from the original implementation in 1989, to the subsequent Interim Enhanced SWTR (IESWTR) in 1998, the Long-Term 1 Enhanced SWTR (LT1 Rule) in 2002, and the current version, the Long-Term 2 Enhanced SWTR (LT2 Rule) established in 2006.

PWB operated under a variance from the *Cryptosporidium* treatment requirements of the LT2 Rule until 2017, when monitoring indicated that *Cryptosporidium* was present above the low detection threshold established in the variance. Following exceedance of the *Cryptosporidium* levels, PWB entered into an agreement with OHA to have a filtration facility online by September 30, 2027. More detail on PWB compliance from 2006 to 2017 is included in Chapter 1: Introduction.

Table 3-7. Surface Water Treatment Rules

	SWTR, 1989[a]	IESWTR, 1998[b]	LT1 Rule, 2002[c]	LT2 Rule, 2006[d]
Who these rules apply to	All systems using surface water or groundwater under the direct influence of surface water (GWUDI).	All systems using surface water or GWUDI that serve <10,000 people.	All systems using surface water or GWUDI that serve <10,000 people.	All public water systems using surface water or GWUDI.
Established Removals	Established 3-log removal/inactivation of *Giardia* and bacteria and 4-log removal/inactivation for viruses.	Established 2.0-log removal in filtered system for *Cryptosporidium* and potential *Cryptosporidium* source control for unfiltered systems.	Set MCLG of 0 for *Cryptosporidium*. Added *Cryptosporidium* as indicator of GWUDI. Required systems using alternative techniques demonstrate 2-log removal of *Cryptosporidium*.	Requires monitoring and treatment for *Cryptosporidium;* log removal dependent on bin classification.[e]
Disinfection Credits	Required residual disinfection to be ≥0.2 mg/L at distribution system entrance and detectable throughout.	—	Required disinfection profile with daily inactivation measurements compiled over 1 year; established lowest monthly inactivation as the benchmark.	—
Monitoring Requirements	Required continuously monitored turbidity (<0.5 NTU in 95% of monthly samples). Established operational guidelines.	Established monitoring frequency requirements for combined (every 4 hours) and individual filters (every 15 minutes); established turbidity <0.3 NTU in 95% of monthly samples and 1 NTU maximum.	Established different effluent requirements dependent on treatment technology.	Assigns filtered systems to four possible bins based on *Cryptosporidium* concentrations with associated requirements.
Additional Rules	—	Required periodic sanitary surveys and cover for all new, finished water storage facilities.	—	Requires that water is stored in a covered reservoir or that reservoir discharge is treated to established log-removals.

a. *Rules are summarized from the Guidance Manual (EPA, 1991).*

b. *Rules are summarized from the IESWTR (EPA, 2001).*

c. *Rules are summarized from LT1 Rule Factsheet (EPA, 2002).*

d. *Rules, including bin classification and corresponding concentrations and treatment required for filtered systems, are summarized from the LT2 Rule Factsheet (EPA, 2006).*

e. *Filtered systems are given a bin classification based on source water monitoring results.*

Based on *Cryptosporidium* levels, the Bull Run source is considered Bin 1 classification based on source water monitoring results. Bin 1 does not require additional removal or inactivation for conventional and direct filtration facilities and is deemed adequate to provide 2.0-log *Cryptosporidium* removal to satisfy the requirements of OHA 333-061-0032 1aAiii. Bin classifications higher than Bin 1 require additional removal or inactivation credits per OHA 333-061-0032 4g.

Stage 1 and 2 Disinfection Byproduct Rules

Along with the SWTR, disinfectants and DBPs are regulated through the Stage 1 and Stage 2 DBPRs summarized in Table 3-8. The initial Stage 1 Rule was established in 1998 and then expanded with the Stage 2 Rule in 2006.

The initial Stage 1 Rule set primary MCLs for TTHM and HAA5 based on averaging distribution system sampling locations into one running annual average (RAA). Stage 1 also set rules for maximum residual disinfectant levels (MRDLs).

Stage 2 maintained the same MCLs for TTHM and HAA5 as Stage 1; however, the levels are based on averaging each location's quarterly sample results to calculate the TTHM LRAA. Stage 2 also included an initial distribution system evaluation requirement for water systems to characterize their distribution systems and identify monitoring sites where customers are likely to be exposed to high concentrations of DBPs. PWB is in compliance with the Stage 2 DPBR.

Table 3-8. Stage 1 and 2 DBPR Requirements[a]

Stage 1 DBPR (1998)	Stage 2 DBPR (2006)
• TTHM and HAA5 MCL calculated using RAA of all samples and system locations. • Locations based on maximum residence time.	• MCL calculated using LRAA for each monitoring location based on initial distribution system evaluation. • Monitoring requirement no longer dependent on number of facilities or wells.
• Reduced monitoring for TTHM and HAA5 available if RAA ≤0.040 mg/L and HAA5 ≤0.030 mg/L. • TOC removal dependent on source water TOC (2–4, 4–8, or >8 mg/L) and alkalinity (0–60, 60–120, or >120 mg/L) for conventional treatment facilities.	• Eligibility unchanged, but source water TOC samples required every 30 days for most systems and every 90 days for reduced monitoring systems.
• Regulated contaminants include TTHMs, HAA5, bromate, chlorite, chlorine/chloramines, and chlorine dioxide.	• If operational evaluation level is exceeded, DBP mitigation actions must be identified by the system. • New analytical method approved for bromate evaluation.
• MCLs defined: TTHMs (0.080 mg/L)[b], HAA5sb (0.060 mg/L), bromate (0.010 mg/L), chlorite (1.0 mg/L).	• No changes in MCLs. • MCLG expanded to include chloroform (0.07 mg/L) and monochloroacetic acid (0.07 mg/L) and lowered for trichloroacetic acid (0.2 mg/L).
• MRDL defined for the following disinfectants: chlorine and chloramines (4.0 mg/L as Cl_2) and chlorine dioxide (0.8 mg/L).	• No changes in MRDLs.

a. *Summarized from the DBPR Quick Reference Guide (EPA, 2010).*

b. *Although there is no collective MCLG for this contaminant group, there are individual MCLGs for some contaminants:*
Trihalomethanes: bromodichloromethane (0); bromoform (0); dibromochloromethane (0.06 mg/L): chloroform (0.07 mg/L).
HAAs: dichloroacetic acid (0); trichloroacetic acid (0.02 mg/L); monochloroacetic acid (0.07mg/L). Bromoacetic acid and dibromoacetic acid are regulated with this group but have no MCLGs.

Lead and Copper Rule

Lead and copper regulations and subsequent modifications derived from the LCR are summarized in Table 3-9, with MCLGs, ALs, and general requirements outlined. PWB has exceeded the AL for lead on several occasions, most recently in fall 2017. PWB also has an agreement with OHA to improve corrosion control treatment by 2022.

Table 3-9. Lead and Copper Rules[a]

Parameter	LCR (1991–1992)	Modified LCR (2000)	Modified LCR (2007)
MCLGs	MCLGs established for lead (0.0 mg/L) and copper (1.3 mg/L).	Maintained MCLGs.	Maintained MCLGs.
Action Levels	Established for lead (0.015 mg/L) and copper (1.3 mg/L) based on 90th percentile level of water samples.	Maintained ALs.	Maintained ALs.
Compliance	If action levels are exceeded, other requirements could be triggered, including water quality monitoring, corrosion control treatment, source water monitoring or treatment, public education, or lead service line replacements.	Clarified that demonstration of optimized corrosion control treatment and replacement of lead service lines is required.	Water systems required to re-evaluate service lines replaced through testing Compliance period defined as 3-year calendar period
Lead and Copper Monitoring	Required first draw samples for areas with high-risk occurrence; number of samples depends on system size; monitoring required every 6 months.	N/A	Requires 5+ samples per monitoring period for systems serving ≤100 people. If <5 taps for human consumption, requires 1 sample per tap. Defines monitoring period as specific period in which water systems conduct required monitoring.
Water Quality Monitoring	Required for systems serving ≥50,000 or if action level is exceeded.	N/A	Requires states obtain prior approval to add a new water source or change a treatment process prior to implementation when changes would have long-term impacts on water quality.
Reduced Monitoring	Allowed systems to qualify dependent on the system population as well as monitoring results.	N/A	N/A
Reporting and Education	Provide individual lead tap results to people who receive water from sampled sites; all systems required to provide education statement and report violations in Consumer Confidence Reports.	Required public education but allowed more flexibility in mode of delivery for public education, especially for smaller systems.	N/A

a. Summarized from the Quick Reference Guide (EPA, 2008).

In 2019, the EPA released a revised LCR (LCRR) proposal for comment. Once the final LCRR is promulgated, compliance will be required within 3 years, which may be as early as January 1, 2024. It is possible that compliance with the final rule may be required prior to startup of the filtration facility, impacting corrosion control decisions for this project.

Although it is unknown what aspects of the proposed changes will remain in the final LCRR, significant items in the proposal include the following:

- A 0.010 mg/L (at 90th percentile of tap sampling results) lead trigger level that, if exceeded, could require increased monitoring, public notification, re-evaluation of optimized corrosion control treatment (which will also require orthophosphate testing), and other actions (i.e., the trigger level exceedance will be similar to an action level exceedance).
- New sample site tier criteria.
- Utility involvement in lead monitoring of schools and licensed childcare facilities.
- Completion of a lead service line inventory.
- Changes in reduced monitoring criteria.
- “Find and fix” each/all lead site results greater than 15 µg/L.
- Additional public outreach requirements.

Total Coliform Rule

In 1989, the TCR was established to improve public health by reducing fecal pathogens to minimum levels through control of TC bacteria, including fecal coliforms and EC. In 2014, a revised TCR was released. OHA implemented provisions to this rule in 2016. This revision required a program submission to the EPA region along with the adoption of Level 1 and 2 assessment categories to reflect severity and frequency of a problem. The Revised TCR also included a treatment technique violation if a system fails to conduct an assessment, fails to correct sanitary defects from an assessment within 30 days, or a seasonal system fails to complete approved procedures prior to serving water to the public.

The sampling and monitoring requirements and violation and reporting requirements are defined in Table 3-10 below.

<table>
<tr><th colspan="3">Table 3-10. TCR and Revised TCR Requirements</th></tr>
<tr><th>Parameter</th><th>TCR (1989)[a]</th><th>Revised TCR (2014)[b, c]</th></tr>
<tr><td>Sampling Requirement</td><td>• TC sample must be collected at sites representative of distribution system and performed at regular intervals throughout the month.
• Sampling frequency based on population served.
• If positive routine sampling, three repeat samples collected and analyzed for TC: one sample from original tap, one sample within five service connections upstream, one sample within five service connections downstream.
• Each TC+ sample must be tested for fecal coliforms or EC.</td><td>• Sample site plan is required, including quarterly monitoring and annual identification of additional routine monitoring.
• Sampling frequency according to sample site plan; assessment and corrective action if system is considered vulnerable to contamination.
• Each TC+ must be tested for EC.</td></tr>
<tr><td>Violations</td><td>Monthly MCL Violations:
• Systems collecting <40 samples: Violation if >1 routine or repeat sample per month is TC+.
• Systems collecting ≥40 samples: Violation if >5% of routine or repeat samples per month are TC+.
Acute MCL Violations (all systems):
• Violations if fecal coliform or EC+ in repeat sample, or if fecal coliform or EC+ routine sample. and TC+ repeat sample.</td><td>Assessment Trigger:
• Non-acute MCL removed; violations >5% TC+ in monthly samples trigger Level 1 assessment instead.
Major MCL Violations (all systems):
• EC MCL violations replaced TCR's acute MCL.
Combinations resulting in violation:
<table><tr><th>Routine</th><th>Repeat</th></tr><tr><td>EC+</td><td>TC+</td></tr><tr><td>EC+</td><td>Any missed sample</td></tr><tr><td>EC+</td><td>EC+</td></tr><tr><td>TC+</td><td>EC+</td></tr><tr><td>TC+</td><td>TC+ (but no EC analysis)</td></tr></table></td></tr>
<tr><td>Reporting</td><td>Monthly Reporting:
• Public: Within 30 days.
• State: End of next business day.
Acute Reporting:
• Public: Within 24 hours.
• State: End of next business day.</td><td>For all systems, if any repeat sample is TC+ and EC+ must report to the state by the end of day.</td></tr>
</table>

a. *Summarized from EPA's Total Coliform Rule: A Quick Reference Guide.*

b. *Summarized from EPA's Revised Total Coliform Rule: A Quick Guide Reference.*

c. *Oregon-specific details adapted from OH's Revised Coliform Monitoring Requirements.*

Filter Backwash Recycling Rule

Along with the regulations detailed above, the Filter Backwash Recycling Rule in 2001 required streams of water returned to the facility be returned prior to primary coagulant addition. The EPA's *Filter Backwash Recycling Rule Technical Guidance Manual* recommends the recycle flow be at or below 10 percent of facility flow (EPA, 2002). The following information related to filter backwash must be submitted to OHA per the requirements specified in OAR 333-061-0032 10:

- Copy of recycle notification.
- List of recycled flows and return frequency.
- Average and maximum backwash rate and duration.
- Typical filter run length and how that is determined.
- Type of treatment for recycled flow.
- Data on treatment unit sizing with loading rates, chemicals used, and frequency of solids removal.

Additional State-Specific Regulations

Beyond the federal regulations discussed in the previous section, Oregon has moved to determine contamination occurrence and develop monitoring plans and regulations based on regional concerns such as per- and polyfluoroalkyl substances (PFAS) and cyanotoxins. Working with Oregon Department of Environmental Quality (DEQ), OHA is currently mapping potential PFAS contaminated sites, and their proximity to water systems, to determine risk. The United States Legislature is considering national legislation having implemented health advisories in 2016 based on the best available studies. Several states have also passed health advisory limits (i.e., New Jersey, California, Vermont, Michigan, and Minnesota). PFAS has not been detected in the Bull Run source; however, Portland International Airport has significant concentrations of PFAS, making it a consideration for PWB's groundwater source.

After a do-not-drink advisory was issued in Salem, Oregon, due to detections of cyanotoxins in finished water, OHA introduced a temporary order in June 2018 and a subsequent permanent order in December 2018 that requires cyanotoxin monitoring in public drinking water systems and established a set of Oregon action levels. The Oregon action levels were based on the following EPA health advisory limits:

- Total microcystins: 0.3 µg/L for vulnerable populations; 1.6 µg/L for ages 6 and older.
- Cylindrospermopsin: 0.7 µg/L for vulnerable populations; 3.0 µg/L for ages 6 and older.

While only systems with susceptible sources are required to sample under the permanent rule, when a raw water sample results in a detection of 0.3 µg/L total microcystins or 0.7 µg/L cylindrospermopsin , the raw water monitoring frequency increases to weekly samples and finished water samples are added. If there are finished water detections, the frequency increases to daily sampling. To return to routine monitoring, two consecutive non-detects are required—either 2 consecutive weeks for raw water, or 2 consecutive days for finished water (OAR 333-061-0510 to 0580).

A facility is considered susceptible if any of the following criteria are met:

- One or more HABs have been documented, or at least one cyanotoxin has been detected in source water.
- Water source upstream of the facility's source is listed as not meeting water quality standards for algae and aquatic weeds.
- The point of diversion into the water system is downstream of, or influenced by, a source susceptible to HABs or cyanotoxins.
- Source is on a water quality limited listing in the DEQ *Integrated Report and Clean Water Act* Section 303(d).

The Bull Run source has not experienced documented or known cyanobacteria blooms in either reservoir. PWB's source does not meet the criteria above and is therefore not considered susceptible to cyanotoxins under the OHA permanent rule. PWB voluntarily tested the Bull Run supply for cyanotoxins in 2005 and submitted samples according to OHA's temporary cyanotoxin monitoring rule in 2018. PWB also tested for 10 different cyanotoxins at the Lusted Hill Conduit 3 Outlet sampling location in summer and fall of 2019 as part of the federal UCMR4. All results were non-detect. PWB continues to test water routinely.

3.4.2 Potential Future Federal Regulations

Along with the regulations previously described, there are different stages that potential regulations can be categorized in. To provide more detail on these stages, this section is divided into the following:

- Six-Year Review
- Contaminant Candidate List (CCL)

Six-Year Review

The SDWA requires EPA to review, and revise if necessary, each drinking water regulation in a 6-year review cycle. This review considers health effects, changes in technology, and factors that will improve public health protection. Table 3-11 summarizes regulations the EPA is currently developing or reviewing.

Table 3-11. Potential Future Regulations

Contaminant	Overview	Status
LCRR[a]	While current LCR has reduced lead exposure, there is a need to strengthen its public health protection and clarify implementation requirements. LCR White Paper provides examples of options to improve the rule. The proposal released in Oct. 2019 includes new potential compliance requirements such as a 100 µg/L lead trigger level, inclusion of inhibitors if a corrosion study is necessary, and increased customer outreach (see text at end of section 5.1.1.3).	Under development. Final rule expected July 2020.
Use of Lead-Free Pipes, Fittings, Fixtures, Solder and Flux for Drinking Water[b]	EPA proposes modification of the definition of lead-free plumbing products (pipes fittings, fixtures) to conform to the 0.25% weighted averaged of lead content level as well as labeling requirements for devices that meet the new definition.	Under development.
Perchlorate[c]	Public input requested concerning perchlorate. Requesting comments on National Primary Drinking Water Regulation (NPDWR) established MCLG of 58 µg/L. Also, three regulatory options proposed: • MCL and MCLG set at 18 µg/L • MCL and MCLG set at 90 µg/L • Withdrawal of regulation	Under development.
Chromium (Total/Hexavalent Chromium)[d]	MCL of 100 µg/L previously established with plans to monitor selected systems' levels under UCMR3. Development of integrated risk information system (IRIS) assessment determined potential health effects with inhalation and ingestion of hexavalent chromium.	Being reviewed.

a. *Summarized from EPA material accessed at: epa.gov/dwstandardsregulations/lead-and-copper-rule-long-term-revisions*

b. *Summarized from EPA material accessed at: epa.gov/dwstandardsregulations/use-lead-free-pipes-fittings-fixtures-solder-and-flux-drinking-water*

c. *Summarized from EPA material accessed at: epa.gov/dwstandardsregulations/perchlorate-drinking-water*

d. *Summarized from EPA material accessed at: epa.gov/dwstandardsregulations/chromium-drinking-water*

Along with the above development and review, eight drinking water regulations are candidates for revision from the completion of the Six-Year Review 3: chlorite, *Cryptosporidium, Giardia lamblia,* HAA5, heterotrophic bacteria, *legionella,* TTHM, and viruses. The Six-Year Review 4 process has already begun, with completion anticipated in 2023.

Contaminant Candidate List

Independent from the Six-Year Review, EPA periodically publishes a CCL and establishes regulatory determination on at least five contaminants from the list of known or anticipated to occur contaminants in water systems. While these contaminants are not currently regulated, they are listed on the EPA's current CCL4 and may be subject to future regulation under the SDWA. These chemical and microbiological contaminants include pesticides, carcinogens such as DBPs (including halogenated and nitrogenous DBPs), chemicals used in commerce and pharmaceuticals, and waterborne pathogens such as *legionella*, *mycobacterium*, and *salmonella*. With the exception of perchlorate, all contaminants listed on CCL3 were carried forward. The EPA publishes the CCL every 5 years with nominations for the next list accepted approximately midway through the 5-year duration. For example, CCL4 was published in 2016 and the nomination deadline for CCL5 was December 2018 (EPA, 2016). According to OHA, cyanotoxins, manganese, and PFAS are contaminants of concern in Oregon (OHA, 2019).

Within the list, contaminants are separated as either ready for regulatory determination, or in need of further research to determine specifics such as health effects, treatability, analytical methods, and occurrence. Between versions of CCLs, listed contaminants can be removed if sufficient information determines no regulation is needed. Alternately, candidate contaminants can become regulated contaminants.

If categorized as needing further research, contaminants can fall under UCMR, a SDWA amendment that includes up to 30 unregulated contaminants that are monitored by public water systems. UCMR is based largely on the review of the CCL and development in coordination. Currently, UCMR4 monitoring occurs from 2018 to 2020 and includes:

- Nine cyanotoxins, one cyanotoxin group.
- Eight pesticides, one manufacturing byproduct.
- Three brominated HAA DBP groups.
- Three alcohols.
- Three semi-volatile organic chemicals.
- Two metals.

3.4.3 Current and Potential Future Regulations Summary

This section reviewed current water quality regulations and potential future regulations that may be applicable to the design of the filtration facility.

Key considerations for project definition include:

- **Current Federal and State Regulations:**
  - The relevant OAR for this project is OAR 333-061, which provides the regulations pertinent to water quality and to the design and performance of the filtration facility.
  - The Bull Run source is considered Bin 1 classification based on source water monitoring results. Bin 1 does not require additional removal or inactivation for conventional and direct filtration facilities and is deemed adequate to provide 2.0-log *Cryptosporidium* removal to satisfy the requirements of OHA 333-061-0032.
  - In 2019, the EPA released a LCRR proposal for comment. It is possible that compliance with the final LCRR may be required prior to startup of the filtration facility, impacting corrosion control decisions for this project.
- **Potential Future Federal Regulations:**
  - EPA undergoes a 6-year review cycle and maintains a CCL list of potential future regulated contaminants. A 6-year review cycle is underway, with completion anticipated in 2023.
  - OHA has identified cyanotoxins, manganese, and PFAS as contaminants of concern in Oregon.

3.5 Corrosion Control Program

In compliance with the LCR, PWB has been operating under an OHA-approved alternative optimized corrosion treatment program that consists of a Lead Hazard Reduction Program, including pH adjustment with sodium hydroxide to reach a target pH of 8.2.

On November 4, 2016, OHA approved PWB's proposed schedule to implement Improved Corrosion Control Treatment by September 30, 2022. PWB completed a water quality corrosion study in 2017 and a corrosion control pilot study in 2018. The corrosion pilot recommended water be treated to maintain a pH of at least 8.5 throughout the distribution system with an alkalinity between 25 to 40 mg/L as $CaCO_3$ when using 100 percent Bull Run water. OHA accepted PWB's treatment recommendation on October 15, 2018.

Design of Improved Corrosion Control Treatment at Lusted Hill is underway. The corrosion control treatment system at Lusted Hill will eventually be replaced by the filtration facility. Filtration and pre-treatment will modify the source water quality, requiring completion of a new optimized corrosion control treatment (OCCT) study.

3.6 Finished Water and Distribution System Water Quality Objectives

Beyond complying with the MCLs for regulated constituents, PWB aims to meet more stringent water quality goals. Some of the key desired qualitative water quality goals include:

- DBP reduction.
- Biostability improvement.
- Pathogen removal/inactivation.
- Lead reduction.
- Consistent water quality.
- Distribution sediment loading reduction.
- Manganese, iron, and secondary MCLs.
- T&O reduction.
- Color reduction.

There are several resources available to assist PWB in identifying and meeting these goals and which provide prescribed guidance documents. These water quality-based criteria and guidance are mentioned below and again in Chapter 5: Design Considerations. To stay proactive in surpassing water quality standards, it is assumed for project definition that water quality goals are equal to or less than half the MCL of regulated DBPs. This general MCL goal is subject to change as the pilot plant study and filtration facility design progress. The finished water and distribution system water quality objectives described in this section will help to inform the treatment process alternatives evaluation. Understanding and keeping water quality goals in mind will influence the cost-benefit assessment of enhanced treatment options.

This section is divided into the following subsections:

- Oregon Area Wide Optimization Program (AWOP)
- Partnership for Safe Water (PSW)
- Pilot Study Water Quality Goals

3.6.1 Oregon Area Wide Optimization Program

A network consisting of EPA, Process Applications, Inc., Association of Safe Drinking Water Administrators, and state participants such as Oregon have collaborated to implement a national AWOP. AWOP is a non-regulatory approach to determine and address performance-limiting factors at surface water treatment facilities with a focus on facility optimization. In Oregon, EPA Region 10 participates and assists in the focus on particle removal optimization (especially, with direct and conventional filtration facilities) to minimize pathogen exposure and maximize human health protection. To meet more stringent water quality goals, PWB will consider formal participation in Oregon AWOP while taking into account the information provided by the pilot study and full-scale facility operations.

Table 3-12 below lists the turbidity goals of the Oregon AWOP compared to OHA requirements.

Table 3-12. Oregon AWOP Goals Compared to OHA Requirements

Conventional Filtration			
Sedimentation	**AWOP Criteria**	**AWOP Turbidity Goals**[a]	**OHA Requirement**[b]
Settled Water Turbidity	If average annual raw water turbidity is >10 NTU	≤2.0 NTU, 95% of the time	No specific requirement
	If average annual raw water turbidity is ≤10 NTU	≤1.0 NTU, 95% of the time	No specific requirement
Conventional and Direct Filtration			
Filtration	**AWOP Criteria**	**AWOP Turbidity Goals**	**OHA Requirement**
Individual Filter Effluent (IFE) and Combined Filter Effluent (CFE)	Based on daily maximum values recorded during 4-hour increments (excluding 15 minutes following backwash).	CFE and IFE ≤0.10 NTU, 95% of the time	CFE ≤0.30 NTU, 95% of the time
		Max. CFE and IFE = 0.30 NTU	Max. CFE = 1.0 NTU
IFE Turbidity after Backwash	Applies to systems with and without filter-to-waste capability. Also applies to recovery period immediately after backwash.	Returns to ≤0.10 NTU within 15 minutes Max. spike = 0.30 NTU, returns to ≤0.10 NTU	No specific requirement

a. *Summarized from OHA's online surface water treatment resources. Accessed at: oregon.gov/oha/PH/HealthyEnvironments/DrinkingWater/Operations/Treatment/Pages/index.aspx#awop*

b. *Summarized from OAR 333-061-0030 3bA.*

3.6.2 Partnership for Safe Water

PWB may wish to consider formally joining PSW and electing to comply with PSW's design, operational, and administrative requirements. PSW focuses on optimization of facility operations and provides criteria for design and management of new facilities. PSW's program involves numerous agencies, including the EPA and American Water Works Association. The program aims to provide subscribers with resources such as software, manuals, and guidance reports for optimization of both potable water treatment and distribution systems.

PSW offers self-assessment and optimization programs so that operators, managers, and administrators have the tools to improve treatment facility performance above and beyond even proposed regulatory levels that are summarized in PSW's Program Fact Sheet. There are four basic phases for achieving PSW goals as follows:

- **Phase I: Commitment.** An honest commitment is required so that utilities are fully engaged in the optimization process.
- **Phase II: Baseline and Annual Data Collection.** Baseline and annual data are submitted for inclusion in PSW's annual report and to measure improvement.
- **Phase III: Self-Assessment.** The Self-Assessment Completion Report is the basis for the Directors Award and reviewed by treatment plant optimization experts.
- **Phase IV: Fully Optimized System.** Phase IV recognizes the highest level of optimization with exclusive Excellence in Water Treatment Award, and the President's Award, recognizing progress towards full optimization.

Phase IV recognizes facilities that have achieved the highest possible turbidity performance by meeting the specifics outlined in Table 3-13. These PSW turbidity goals are consistent with Oregon AWOP turbidity goals.

Table 3-13. PSW Performance Goals[a]

Parameter	Goal	Criteria
Settled Water Turbidity	≤2.0 NTU, 95% of monthly samples taken at 4-hour increments for each basin.	If average annual raw water turbidity is >10 NTU.
	≤1.0 NTU, 95% of monthly samples taken at 4-hour increments for each basin.	If average annual raw water turbidity ≤10 NTU.
Filtered Water Turbidity	≤0.1 NTU, 95% of the time Max. = 0.30 NTU	Based on values recorded at 15-minute intervals.
Filter Performance	96th, 97th, 98th, and 99th percentile values of filtered water turbidity.	Indicator of consistent filtration performance.
IFE Turbidity after Backwash	Returns to ≤0.1 NTU within 15 minutes.	No specific requirement.
CFE Turbidity	≤0.1 NTU 95% of the time	No specific requirement.
Disinfection	Concentration time values to achieve log inactivation of *Giardia* and virus.	No specific requirement.

a. Summarized from guidelines for Phase IV Application (AWWA, 2014).

PWB may also consider meeting the following PSW general data monitoring requirements:

- Daily raw water turbidity.
- Settled water turbidity, every 4 hours, for individual sedimentation basin.
- Continuous, online turbidity measurements for each filter.
- One turbidity profile, with backwash, from a filter run each month.
- Combined filter effluent every 4 hours.

3.6.3 Pilot Study Water Quality Goals

Table 3-14 below summarizes the pilot plant study's water quality treatment goals and benchmarks. Information in this table was submitted to and approved by OHA. The operational goals for turbidity are based on the PSW Phase IV Performance Goals (PSW, 2014) and Oregon AWOP. DBP goals are set as half the MCL. The performance benchmarks were determined based on minimum production needs and typical filter efficiency. Overall, finished water quality goals will be further refined following the pilot plant study.

Table 3-14. Pilot Study Water Quality Goals and Performance Benchmarks

Parameter	Location	Regulatory Requirement[a]	Operational Goal[b]	Comments on Operational Goal
Turbidity	Settled water[c]	No requirement	≤2.0 NTU, 95% of monthly samples taken at 4-hour increments for each basin[d]	• Overview of PSW and Oregon AWOP: - ≤1.0 NTU 95% of the time when source turbidity is ≤10 NTU - ≤2.0 NTU 95% of the time when source turbidity is >10 NTU
	IFE[e]	≤0.3 NTU, 95% of monthly samples[f]	≤0.10 NTU, 95% of the filter run time	• Goal matches PSW Phase IV Performance Goal but more stringent because it is applied to IFE • Oregon AWOP operational goal is <LT2 Microbial Toolbox credit of 0.15 NTU
	IFE[e]	≤1 NTU at any time[f]	Max = 0.30 NTU	• Goal matches PSW Phase IV Performance Goal; Oregon AWOP
Particle Counts	IFE[e]	No requirement	<50 particles/mL at 5–15 µm, 95% of the filter run time	• Particle count goals are surrogates for *Cryptosporidium* and *Giardia* removal
	IFE[e] compared to raw		2.0-log removal from raw water for 3–5 µm and 2.5-log removal for 5–15 µm[g]	• Particle count goals are surrogates for *Cryptosporidium* and *Giardia* removal • Assumes sedimentation in operation
TTHM	Simulated Distribution System (SDS)	MCL = 80 µg/L based on LRAA sampling	≤40 µg/L for chosen treatment scheme as measured by DBP SDS testing	• Goal based on LRAA; goal (half MCL) is also a trigger for reduced DBP monitoring
Sum of HAA5	SDS	MCL = 60 µg/L based on LRAA sampling	≤30 µg/L for chosen treatment scheme as measured by DBP SDS testing	
Minimum Unit Filter Run Volume (UFRV)[h]	Individual filter	No requirement	>6,500 gallon/sf-run 95% of the operational time	• Backwash based on turbidity, head loss, and run time triggers • Goal based on estimated minimum to meet water production goals
Filter-to-waste Cycle	Filter-to-waste	No requirement	≤5% of total UFRV 95% of the operational time	• Goal based on achieving overall filter efficiency of ≥95%

a. *Regulatory requirement meets federal and state requirements.*

b. *Operational goal is modeled from PSW and Oregon AWOP and is an internal PWB goal, not based on regulatory requirements.*

c. *Applicable when operating in conventional filtration mode.*

d. *Optimal turbidity will be determined based on producing filterable floc. Turbidities > 1 NTU were required when raw water turbidity was <10 NTU to optimize filtration.*

e. *IFE samples will be analyzed continually and recorded every 5 minutes.*

f. *Regulatory requirement based on CFE. Pilot plant study monitoring based on IFE.*

g. *When operating in direct filtration mode, 2.0-log removal from raw water for 3–5 µm range and 2.0-log removal for 5–15 µm.*

h. *Minimum UFRV is based on a filter loading rate of 12 gpm/sf and a desired facility production of 145 mgd with eight filters and one filter out of service.*

3.6.4 Finished Water and Distribution System Water Quality Objectives Summary

This section summarized the voluntary guidance of the Oregon AWOP and PSW and identified the pilot plant goals.

Key considerations for project definition include:

- Beyond complying with the MCLs for regulated constituents, PWB aims to meet more stringent water quality goals. It is assumed for project definition that water quality goals are equal to or less than half the MCL of regulated DBPs.
- The pilot plant operational goals incorporate Oregon AWOP and PSW standards. As design progresses, PWB will consider formal participation in these programs.
- Ongoing pilot plant testing will assist in defining the operational goals for full-scale treatment, taking into consideration turbidity, particle counts, organics and DBP reduction, filter performance, UFRV minimums, and disinfection contact time.

3.7 Water Quality Considerations Summary

PWB is looking to address today's water quality goals with the capability to adapt to meet tomorrow's more stringent and unknown needs. The information in this chapter provides the foundational understanding of historical water quality data and discusses both current and potential future regulations that should be considered in design and operation of the filtration facility. The characteristics of the raw water and finished water quality, along with PWB's desired water quality objectives will inform evaluation of treatment alternatives in subsequent chapters of this report.

Key considerations for project definition include:

- **Raw Water Quality.** General trends in PWB's raw water are summarized as follows:
  - TOC in the raw water intake increases as expected in the fall during periods of reservoir turnover, leaf shedding, and the onset of storms.
  - Iron and manganese levels increase in summer and fall and with depth.
  - Algae is routinely detected in the intake with an average total algae count of 650 units/mL entering the Headworks intake. Other known filter cloggers that have been present in the intake in counts above 500 units/mL, but have not had reported issues in PWB's system, include *Melosira*, *Cycotella*, and *Aphanothece*. *Chlamydomonas* had the highest maximum count from the time period, and has been known to cause T&O issues in other water systems.
  - The average turbidity at the intake is 0.6 NTU over the past decade, with a maximum measured turbidity of 25 NTU. One limitation of the turbidity measured at the intake is that during a high-turbidity shutdown event (above 5 NTU), the turbidimeters do not capture the actual turbidity level in the upstream water source due to a calibration setting to maximize data resolution at lower values. As a result, much greater turbidity could have occurred than was measured. Since 1986, there have been 10 shutdowns due to elevated turbidity.
  - Overall, water quality does not change significantly with depth, except during summer stratification.
  - Algae is typically present at the surface in higher concentrations.
  - Nutrients are low throughout the water column with an average total phosphorous of 0.01 mg/L and total nitrogen of 0.1 mg/L.
- **Finished Water Quality:**
  - The majority of DBPs form before the entry to the distribution system during free chlorine disinfection.
  - DBPs at the Lusted Hill Outlet are on average below the MCL for TTHM (80 µg/L) and HAA5 (60 µg/L), with an average TTHM of 14 µg/L, and average HAA5s at 27 µg/L.
  - In the distribution system, HAA5s tend to be at or above half the MCL, but still below the full MCL for the LRAA, with two exceedances above the MCL in November over the past 3 years. Since the LRAA is the compliance target not individual quarter samples,

  the exceedances in November did not trigger a violation but are noted to show that there are higher levels of DBPs in the fall.

  - PWB has treated the drinking water to reduce lead and copper since 1997. Finished water has exceeded the lead AL twice in the past 4 years. PWB continues to improve corrosion control to reduce the potential for exceedances.

- **Potential Future Raw Water Quality Risks:**
  - Climate change impacts predicted to affect the Bull Run supply include temperature increase and more extreme precipitation conditions.
  - Natural hazards of concern include an increased probability of wildfire events, seismic activity being within the CSZ, as well as a potential Mount Hood volcanic eruption.
  - Considering the expected climate changes and potential natural hazards, PWB's priority scenarios include:
    - An increase in metal, organic, and nutrient concentrations from wildfires.
    - An increase in turbidity from wildfires and other sources.
    - Temperature increases and subsequent proliferation of filter-clogging or cyanotoxin producing algae.
- **Current and Potential Future Regulations.** There are a variety of federal regulations, those pertinent to PWB's system include:
  - SWTP, developed by numerous revisions with the most recent guideline specified in the 2006 LT2 rule, with removals and disinfection credits defined. Having previously operated under a variance, PWB has an agreement with OHA to have a filtration facility online by 2027 to meet these requirements.
  - Stage 1 and 2 DBPRs set primary MCLs for TTHM and HAA5, along with MRDLs
  - LCR, at the time of writing this document, was most recently modified in 2007, establishes MCLGs, ALs, monitoring and educational requirements associated with lead and copper. Having recently exceeded the AL for lead, PWB has an agreement with OHA to provide improved corrosion control treatment by 2022. Following promulgation, PWB's corrosion control and compliance approach may change and plans for the filtration facility may be impacted.
  - TCR, established in 1989 and revised in 2014, improves public health by removing fecal pathogens and establishing sampling, monitoring, and reporting requirements.
  - Along with the current federal regulations, OHA and DEQ are currently mapping out PFAS contaminated sites and performing a risk assessment due to potential national future legislation.
  - Cyanotoxin monitoring and associated action levels were defined in 2018. This rule requires susceptible sources to monitor and sample regularly and Bull Run is not considered a susceptible source according to OHA's criteria.

- Along with the LCR currently under development, the following rules are under development with the EPA:
  - Perchlorate MCLG definition or withdrawal of regulation.
  - Use of lead-free plumbing products to conform to 0.25 percent weighted average as well as labeling requirements.
  - Review of 100 ug/L MCL for chromium, as well as monitoring plan and IRIS assessment.
  - 8 DW regulations are candidates for revision including: chlorite, *Cryptosporidium*, *Giardia* lamblia, HAA5, heterotrophic bacteria, *legionella*, TTHM, and viruses.
  - Independent from the 6-year review process, CCL4 was published in 2016 and the CCL5 nominations were closed December 2018.
  - UCMR4 monitoring, occurring from 2018 through 2020, include cyanotoxins, pesticides, brominated HAA DBP groups, alcohols, semi-volatile organic chemicals, and two metals.

- **Corrosion Control Program:**
  - PWB is currently operating under an OHA-approved alternative optimized corrosion treatment program with the proposed completion of September 2020.
  - Design of Improved Corrosion Control Treatment at Lusted Hill is underway and will eventually be replaced by the filtration facility.
- **Finished Water and Distribution System Water Quality Goals:**
  - Key desired improvements include improved stability, consistent water quality, and reductions in contaminants such as lead, metals, DBPs, as well as color and T&O.
  - PWB is considering formal participation in in Oregon AWOP and PSW.
  - Pilot plant testing will assist in refining the operational goals for full-scale treatment.

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Chapter 4
Planning Considerations

This chapter identifies planning considerations associated with the physical characteristics of the Portland Water Bureau's (PWB's) filtration facility site. This chapter aims to identify potential design and implementation issues and options for resolution. For each option, analysis is provided, along with potential supporting design standards, and a recommendation or identification of further analysis to be completed by the designer.

This chapter includes the following sections:

- 4.1 Site Description
- 4.2 Permitting Considerations
- 4.3 Cultural Resources Protection
- 4.4 Environmental Assessment
- 4.5 Traffic Considerations
- 4.6 Acoustic Considerations
- 4.7 Geotechnical Considerations
- 4.8 Hydraulic Considerations and Ultimate Capacity
- 4.9 Facility Overflow Management
- 4.10 Planning Considerations Summary

4.1 Site Description

The filtration facility site is in unincorporated Multnomah County just north of the Clackamas County line and approximately 3 miles east of the City of Gresham urban boundary. The site is zoned Multiple Use Agricultural (MUA-20) in the Rural Plan Area West of Sandy River.

PWB has operated facilities in the area for more than 100 years and has owned the filtration facility site since 1975 for the purpose of future water facilities. The site consists of two adjacent lots: a 56.87-acre lot to the west and a 36.62-acre lot to the east. The approximately 95-acre site consists of predominantly undeveloped agricultural land. The northern portion of the property along SE Dodge Park Boulevard is forested. The remaining portions of the property are cleared and leased for agricultural operations as a commercial tree nursery. It is PWB's understanding that since purchasing the property in 1975, the land has been leased for commercial production of nursery crops. Access roads along the site's northern, eastern, western, and southern boundaries and through the middle of the property support agricultural operations and access to the existing water towers to the south.

The historical development of the property since the early 1900s is summarized below and further described in the *Phase II Environmental Site Assessment Report* included in Appendix C (Assessment Associates, Inc., 2020):

- **1910s to 1950s:** Prior to the late 1950s, the property was predominantly undeveloped and partially wooded, with several cleared pasture areas. A small cluster of buildings, most likely a residence and a few agricultural outbuildings, occupied the north-central portion of the property between the late 1940s and early 1980s, and an additional residence and barn occupied the southwestern portion of the property between the late 1940s and mid-1970s. The surrounding land was wooded or agricultural, with a few widely scattered single-family residential structures. One water tower was present across the south-central border of the property on a separate tax lot.
- **1960s to 1970s:** A residential structure was constructed immediately east of the eastern property boundary by 1970. Agricultural and grazing lands on the eastern side of the property were converted to nursery crops by 1975, at which time an access road was constructed on the northeastern portion of the property.
- **1980s to 2017:** The residential and agricultural structures on the western border of the property were removed, and an additional water tower was constructed immediately south of the central portion of the property by 1982. The property has not changed appreciably since then.

The site consists of a hill on the eastern portion of the property with the existing grade gently sloping downward to the northwest, southwest, and southeast (Figure 4-1).

Figure 4-1. Aerial view of the filtration facility site showing existing site contours

4.2 Permitting Considerations

This section presents the findings from a preliminary review of permitting needs for the filtration facility by Winterbrook Planning. The findings address local, state, and federal permits and their implications for design, construction, and operation of the filtration facility. The environmental constraints related to relevant permit types for the filtration facility site are illustrated in Figure 4-2.

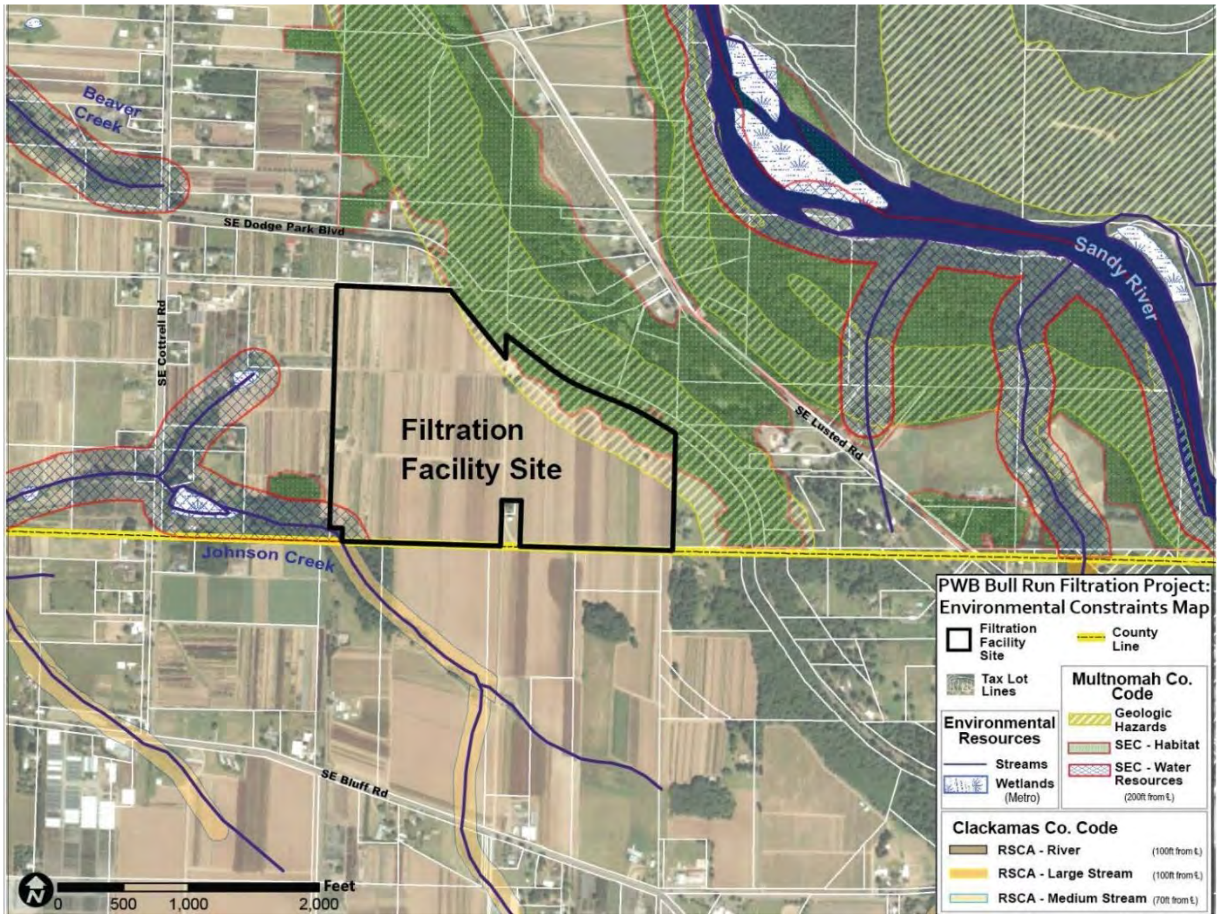


Figure 4-2. Environmental constraints for filtration facility permits

The agency permitting needs further described in the following sections include:

- Multnomah County Permits
- Clackamas County Permits
- Oregon Department of Environmental Quality (DEQ) Permits
- Oregon Water Resources Department (OWRD) Dam Safety Permit
- Wetland and Waterway Permits
- Oregon State Historical Preservation Office (SHPR) Archaeological Permit
- Drinking Water Program Plan
- Permitting Considerations Summary

In addition to the procedural steps described below, there are additional actions that may support the permitting process and long-term project success. These actions include:

- Active and ongoing public outreach and involvement, with direct outreach to nearby residents and businesses. This work has been underway for some time and public feedback is being incorporated into planning efforts.
- Early coordination with regulatory agency staff. This coordination began in 2018 with outreach to county planning staff. It will continue in the form of informal meetings with agency staff, pre-application conferences, and forums such as Portland Streamlining Committee meetings (the Streamlining Committee is made up of local, state, and federal agency staff that meet monthly to review City of Portland projects).

4.2.1 Multnomah County Permits

The filtration facility will require land use permits from Multnomah County. The key land use permit for the filtration facility is Multnomah County’s Type III Conditional Use/Community Service. Community Service uses require a Type II Design Review. Concurrent Multnomah County reviews may include environmental, geologic hazard, and transportation-related permits, depending on the design and layout of planned facilities. The potential permits are listed below and summarized in the following discussion.

- Type III Conditional Use/Community Service Review
- Type II Design Review
- Type II Significant Environmental Concern-habitat (SEC-h) Review
- Type II Significant Environmental Concern-water (SEC-w) Review
- Type II Geologic Hazard Permit

The permits listed above will typically be consolidated into one Type III review, which means a staff recommendation and a public hearing before the Multnomah County land use hearings officer. A pre-application conference is required and should occur less than 6 months prior to land use application submittal. Once the application is submitted and deemed complete, a final decision should (under state law) be issued within 150 days.

Conditional Use/Community Service Review. The filtration facility is a “utility facility” and therefore designated a Community Service Use per Multnomah County Code (MCC) 39.7520.A.6. In the MUA-20 zone that governs the filtration facility site, Community Service Use is allowed if approved through a Type III conditional use review, per MCC 39.4320.A. Note that the MUA-20 zone technically is not an “agricultural” zone subject to statewide rules for agricultural lands. The main review for the filtration facility site will be a combined Community Service/Conditional Use review.

The relevant approval criteria for the Community Service Use review are in MCC 39.7515. A few key criteria are highlighted below, with some initial design considerations noted. The criteria include the following:

1. **Is consistent with the character of the area.** The site design should include generous setbacks, landscaped buffers, and buildings that are limited in height and scale.
2. **Will not adversely affect natural resources.** SEC zones apply to the forested slopes on the northeast portion of the property and a stream near the southwest corner of the property. Early conceptual plans show no impacts in these areas, but potential impacts should be monitored as design and construction plans develop.
3. **The use will not: 1) Force a significant change in accepted farm or forest practices on surrounding lands devoted to farm or forest use; nor 2) Significantly increase the cost of accepted farm or forest practices on surrounding lands devoted to farm or forest use.** The review will assess potential impacts on farming, if any.

A new communications tower is proposed for the filtration facility. This tower will be reviewed concurrently as part of the Conditional Use/Community Service Review.

Design Review. Design review is required as part of the Community Service/Conditional Use review, per MCC 39.7505.B. This code includes review of buildings and structures on the filtration facility site. Consideration should be given to minimizing the height of proposed structures, increasing building and structure setbacks from the edges of the site, and buffering the boundaries of the site or the edges of buildings with landscaping, as appropriate. In general, the design should inventory and consider view impacts looking toward the filtration facility site and its structures from neighboring properties and rights-of-way.

SEC Review. The filtration facility site is affected by two SEC overlay zones: SEC-h for habitat, and SEC-w for water resources. The habitat overlay applies to the forested area on the northeast edge of the site, which covers approximately 6 acres (6.5 percent) of the site. The water resources overlay applies within 200 feet of a stream located near the southwest corner of the site. This water resources overlay area covers approximately 1.2 acres (1.4 percent) of the site. Conceptual plans for the filtration facility show no development within either of these zones, but potential impacts should be monitored as design and construction plans develop.

Geologic Hazard Permit. The northeast edge of the site is covered by a geologic hazard overlay zone. This overlay zone, which covers approximately 12.6 acres (13.8 percent) of the site, protects steeply sloped areas “from public and private losses due to earth movement hazards in specified areas.” A geologic hazard permit will be needed if proposed development enters this zone. Current conceptual plans for the filtration facility avoid development in the zone.

Other County Requirements. One of the baseline requirements for a Multnomah County land use review is that development occur on a legal lot of record. PWB records show that the subject site meets the lot of record requirements. A current title report will be required as part of the land use application.

As part of the land use process, Multnomah County will review proposed local street access to the filtration facility site. Multnomah County traffic and access impacts will also be reviewed.

4.2.2 Clackamas County Permits

If site access is proposed from the south, similar Clackamas County requirements as those described above for Multnomah County would apply. The Clackamas County traffic and access impacts will be reviewed. There is also the potential for access road improvements to trigger a Clackamas County Stream Conservation Area (SCA) permit. Potential SCA implications will be reviewed in more detail as site access and design plans are further defined.

4.2.3 Oregon Department of Environmental Quality Permits

The following permits will be required by DEQ for this project:

- **National Pollution Discharge Elimination System (NPDES) 1200-C General Construction Permit.** DEQ issues this permit to regulate stormwater runoff from construction activities that affect surface waters. The basic permit requirements are twofold: obtain a Land Use Compatibility Statement from Multnomah County, and submit a full-sized, completed erosion and sediment control plan to DEQ.
- **Other NPDES Permits.** The NPDES permit for overflow and drain flows may be covered under PWB's existing NPDES Permit 101617 if it is modified to include the new locations. PWB is in discussions with DEQ about the extension or modification of the NPDES Permit. Among other modifications, blow-offs from new piping will need to be added to the NPDES Permit 101617 as new outfalls if the permit is not materially changed. In addition, Oregon has a general permit (200-J) applicable to filtration backwash that covers other filtration facilities. This is a separate NPDES permit that may apply to this project.

4.2.4 Oregon Water Resources Department Dam Safety Permit

Construction of overflow basins at the site may trigger a dam safety permit (also known as reservoir storage permit) from the OWRD. State dam safety statutes and requirements apply to storage of 3 million gallons (MG) or more and dams 10 feet or more in height. Preliminary indications are that planned overflow basins may meet both thresholds and trigger a dam safety permit. Other basins, such as the clearwell, are not expected to need OWRD approval because they are partly in the ground and the height and volume thresholds only apply to storage above ground elevation.

4.2.5 Wetland and Waterway Permits

Johnson Creek flows immediately to the southwest of the site, with one northern tributary located on nursery land to the west of the site. In its lower reaches, Johnson Creek supports coastal cutthroat trout and winter-run adult steelhead among other sensitive and listed species. Current conceptual site plans for the filtration facility show no development near these waterways. However, state and federal permits related to jurisdictional waters could be triggered for certain activities such as construction of overflow pipelines from the filtration facility stormwater and overflow basins to nearby waterways. As design develops, the program team will meet with the Portland Streamlining Committee to review potential impacts.

4.2.6 Oregon State Historic Preservation Office Archeological Permit

SHPO manages programs for the protection of the state's historic and cultural resources. A SHPO archaeological permit would be required if archaeological investigations were needed to assess the significance of an archaeological resource within the filtration facility site that could not be avoided or that was unexpectedly encountered during construction. An archaeological survey of the filtration facility site was conducted in March 2019. The survey did not result in the discovery of prehistoric Native American cultural materials or deposits. Indications of an earlier farmstead were found on the filtration facility site, including a small concrete foundation. The buildings were removed by 1980. Based on recent surveys, no archeological permits are anticipated to be needed for the project. However, an inadvertent discovery plan outlining construction protocols and notification procedures should be prepared and followed during construction in the event of inadvertent discovery of archaeological materials.

4.2.7 Drinking Water Program Plan

Modification to the City of Portland's drinking water system will require Drinking Water Program Plan review by the Oregon Health Authority (OHA). PWB has had initial meetings with OHA to discuss the general approach to project improvements. Follow up communications will continue as design progresses. The Drinking Water Program Plan review for the project will occur at 100 percent design.

4.2.8 Permitting Considerations Summary

This section summarized the preliminary review of local, state, and federal permitting needs for the filtration facility. Key considerations for project definition include:

- **Multnomah County Permits.** The filtration facility will require permits from Multnomah County, including conditional use, design review, environmental (SEC), geologic hazard, and transportation-related permits.
- **Clackamas County Permits.** If site access is provided from the south, there is the potential for Clackamas County SCA and transportation-related permits.
- **State and Federal Permits:**
  - State regulations will trigger NPDES permits, OHA plan review, and dam safety permits.
  - State and federal archeological, wetland/waterway, and endangered species regulations are unlikely to be triggered, but review thresholds will be monitored as design progresses.
  - PWB was invited to apply for federal funding under the Water Infrastructure Finance and Innovation Act; therefore, the projects must comply with federal laws and regulations, including preparing impact analyses for the following:
    - National Environmental Policy Act of 1969 for environmental resources.
    - National Historic Preservation Act for cultural resources.
    - Farmland Protection Policy Act for significant agricultural lands.

4.3 Cultural Resources Protection

An archaeological survey, including both desktop and field investigations, was conducted by Heritage Research Associates to identify potential cultural resources at the filtration facility site.

This section includes discussion of the following topics:

- Archaeological Survey
- Archaeological Records Search
- Archaeological Pedestrian Survey
- Cultural Resources Protection Summary

4.3.1 Archaeological Survey

An archaeological surface survey was conducted at the filtration facility site in March 2019 to determine if prehistoric or historical archaeological sites that are listed in, or may be eligible for inclusion in, the National Register of Historic Places (NRHP) are present within the Area of Potential Effect for the project. The archaeological survey did not result in discovery of prehistoric Native American cultural materials or deposits at the site; however, ground-disturbing activities will be conducted under the terms of an inadvertent discovery protocol.

Although no prehistoric materials were found during the survey of the filtration facility site, indications of a previous farmstead were found. The structures appear to have been demolished less than 50 years ago and so do not qualify as an archaeological site. Because this farmstead may date to 1911 or earlier, ground-disturbing activities in the immediate vicinity of the farmhouse will be conducted under the terms of an inadvertent discovery protocol and/or archaeological monitoring in the event that buried archaeological features related to use of the farmstead, such as a privy or a well, are encountered.

A rock retaining wall was found offsite on SE Dodge Park Boulevard northeast of the filtration facility site. If this wall is impacted by the project, a more in-depth recording and assessment of the feature should be conducted.

The archeological survey ensures compliance with state and local land use laws requiring identification and protection of significant resources, defined as those eligible for the National Register of Historic Places. This survey also meets federal cultural resources compliance requirements under Section 106 of the National Historic Preservation Act of 1966 (as amended).

4.3.2 Archaeological Records Search

Archaeological site and project records on file at the SHPO were reviewed to identify previous cultural resources investigations conducted in the project vicinity and archaeological sites recorded in the area. The historical Sanborn Insurance maps do not include the current project area, but other historical maps that include the project vicinity were inspected.

Twelve previous archaeological investigations on file at the SHPO have been conducted within approximately 1 mile of the filtration facility site and are summarized in Table 4-1. Only one archaeological site was recorded as a result of these projects; it is nearly a mile away and will not be affected by the Filtration project.

Table 4-1. Summary of Previous Investigations Near the Site

Project Description	Results	Reference
Archaeological Context Statement for Portland Basin	Regional background research	Ames, 1994 (SHPO 20025)
Portland General Electric Bull Run Hydroelectric Project Traditional Cultural Property literature review	Background ethnographic research	French et al., 2000 (SHPO 17038)
Portland General Electric Bull Run Hydroelectric Project Archaeological Investigations	Survey and 200 probes, found three sites (35CL265 in project vicinity)	Oetting, 1999 (SHPO 17039)
Portland General Electric Bull Run Hydroelectric Project evaluation of three sites	Test excavations, one site National Register of Historic Places-eligible, two not eligible (35CL265 not eligible)	Oetting, 2003 (SHPO 18627)
Diack’s and Sester’s ponds survey, Bull Run Water System	Survey, negative results	McDaniel, 2005 (SHPO 20312)
Sandy River conduit relocation survey	Survey and 18 probes, negative	Buchanan and Fagan, 2008 (SHPO 21825)
Bonneville Power Administration transmission line survey at three locations	Survey and 10 probes, negative	Oliver and Schmidt, 2011 (SHPO 24087)
Bonneville Power Administration transmission line survey at 16 impairment locations	Survey and one probe in two locations near current project, negative	Roulette and Harris, 2014 (SHPO 26263)
Oxbow Regional Park survey	Survey and six probes, negative	Chapman et al., 2016 (SHPO 27409)
Oxbow-PWB Restoration Project survey, Oxbow Regional Park	Survey, negative	Musil and Oetting, 2017 (SHPO 28797)
Oxbow Restoration Project survey, Oxbow Regional Park	Survey and eight probes, one historical site (35MU275), determined not eligible	Windler, 2017 (SHPO 29245)
Dunn's Corner telecommunications facility survey	Survey and two probes, negative	Goodwin, 2018 (SHPO 29740)

4.3.3 Archaeological Pedestrian Survey

A pedestrian survey of the approximately 95-acre facility site was completed by two archaeologists walking parallel transects north and south across the property. The site is a working tree nursery, so the individual survey transects were spaced roughly 65 feet apart and oriented along the rows of plantings, covering 100 percent of the site.

The pedestrian survey did not result in the discovery of any evidence of prehistoric archaeological materials or deposits. No additional subsurface probing was deemed necessary because large areas of bare ground were consistently exposed across the site, providing good-to-excellent ground visibility due to tilling.

The only observed archaeological feature was a large concrete foundation on the western edge of the property where a barn was part of an earlier farmstead (Figure 4-3). Early United States Geological Survey (USGS) quadrangles show a building at the location of this farmstead as early as 1911. Later USGS aerial photographs indicate that the house and barn were removed sometime between 1975 and 1980. The trees and concrete foundation are the only physical remains of the farmstead still visible. Given the early age of the farmstead, it is possible that features such as privies or a groundwater well may be present below the disturbed deposits.

Figure 4-3. Photograph from 1952 depicting an early farmstead at the filtration facility site

Source: USGS, 1952

4.3.4 Cultural Resources Protection Summary

This section summarized the findings of desktop and field investigations to identify potential cultural resource at the filtration facility site. Key considerations for project definition include:

- Although no prehistoric materials were found during the site survey, ground-disturbing activities will be conducted under the terms of an inadvertent discovery protocol.
- Indications of an earlier farmstead were found, but the structures appear to have been demolished less than 50 years ago and so do not qualify as an archaeological site. Ground-disturbing activities in the immediate vicinity of the farmhouse will be conducted under the terms of an inadvertent discovery protocol in the event that buried archaeological features related to use of the farmstead, such as a privy or a well, are encountered.
- A rock retaining wall was found offsite on SE Dodge Park Boulevard northeast of the filtration facility site. If this wall is impacted by the project, a more in-depth recording and assessment of the feature will be conducted.

4.4 Environmental Assessment

A Phase I Environmental Site Assessment (ESA) completed by Akana reviewed the current and historical use of the filtration facility site and potential environmental contamination (Akana, 2018). The Phase I ESA identified potential environmental impacts at the property based on the possibility of pesticide residues in the near-surface soils from its historical use as a nursery. Thus, additional environmental investigation was recommended through a Phase II ESA.

This section includes discussion of the following topics:

- Phase II ESA
- Environmental Assessment Summary

4.4.1 Phase II ESA

The recommended Phase II ESA was conducted by Assessment Associates, Inc., at the filtration facility site in February 2019. The *Phase II ESA Report* is included in Appendix C (Assessment Associates, Inc., 2020). Soil samples from two depths (0- to 6-inches deep and 6- to 12-inches deep) were collected from locations evenly spaced throughout the roughly 80 acres of the property currently or historically used for agricultural purposes. Soil samples were collected in accordance with DEQ's guidelines for evaluating residual pesticides on agricultural land. The samples were analyzed for pesticides, chlorinated acid herbicides, and 13 priority pollutant metals using United States Environmental Protection Agency (EPA) methods.

Because the laboratory detection limits for some of these parameters were above the DEQ clean fill screening levels, an additional investigation was performed in December 2019. Additionally, a geophysical survey was conducted in December 2019 near several previously demolished structures at the site in an effort to locate and identify subsurface anomalies that could interfere with future excavation work. The results of both investigations are summarized below and in the supplemental assessment report included in Appendix C (Assessment Associates, Inc., 2020).

February 2019 Phase II ESA Investigation:

- **Pesticides.** None of the composited soil samples exhibited concentrations of pesticides at levels greater than the laboratory detection limits. The detection limit for Aldrin, dichlorodiphenyldichloroethane, dichlorodiphenyldichloroethylene, dichlorodiphenyltrichloroethane, Dieldrin, and Toxaphene exceeded the DEQ clean fill screening levels but were well below both the DEQ Occupational Risk-based Concentrations (RBCs) and EPA's Industrial Regional Ingestion Non-Cancer Screening Levels.
- **Chlorinated Acid Herbicides.** None of the composited soil samples had concentrations of herbicides at levels greater than the laboratory detection limits. The detection limit for 2-methyl-4-chlorophenoxy acetic acid and 2-methyl-4-chlorophenoxy propionic acid exceeded DEQ clean fill screening levels, but the 2-methyl-4-chlorophenoxy acetic acid was well below DEQ's Occupational RBC. Two-methyl-4-chlorophenoxy propionic acid does not have a DEQ RBC but has a similar mammalian toxicity profile as 2-methyl-4-chlorophenoxy acetic acid.

- **Metals.** Arsenic, beryllium, chromium, copper, lead, mercury, nickel, and zinc were detected in all of the composited soil samples. However, all concentrations were well below the applicable DEQ clean fill screening levels. Antimony, cadmium, selenium, silver, and thallium were not detected at concentrations greater than the laboratory reporting limits in the composited soil samples. Arsenic exceeded the DEQ Occupational RBC but was below the published background and clean fill reference levels.

December 2019 Supplemental Phase II ESA Investigation:

- **Pesticides.** Results for the composite sample collected at 0 to 6 inches showed that dichlorodiphenyldichloroethylene, dichlorodiphenyltrichloroethane, and dieldrin concentrations slightly exceeded DEQ clean fill screening levels but were well below DEQ residential risk-based screening levels. Results for the composite sample collected at 6 to 12 inches slightly exceeded the DEQ clean fill screening level only for dieldrin but were well below the DEQ residential risk-based screening level.
- **Chlorinated Acid Herbicides.** The composited soil samples did not have concentrations of herbicides that exceeded the DEQ clean fill levels.
- **Metals.** Antimony, arsenic, cadmium, selenium, and silver were detected in the composited soil samples; however, concentrations were below DEQ clean fill screening levels.

December 2019 Geophysical Survey:

A geophysical survey was performed at the former locations of:

- A home and barn along the west-central property line
- A farm structure in the northern portion of the site near an existing stand of trees
- A small shed along the southern portion of the property north of the water tanks

Several subsurface anomalies were detected though none are likely to be underground storage tanks. The interpretations of these include:

- Vertical pipe (interpreted as a well casing) near the former home along the west-central property line.
- Building debris and a water supply pipe near the former farm structure in the northern portion of the site.

The survey report, completed by Pacific Geophysics, includes methods, findings, and figures showing the area surveyed.

4.4.2 Environmental Assessment Summary

This section describes the results of the Phase II ESA of the filtration facility site. Key considerations for project definition include:

- The Phase II ESA determined that metal and herbicide concentrations are below the DEQ clean fill screening levels. While a few pesticide concentrations are slightly above the DEQ clean fill screening levels, they are well below DEQ residential risk-based screening levels.
- Based on preliminary discussions with DEQ, excavated materials from the near-surface layer (upper 12 inches) of the site are suitable for onsite reuse and are likely suitable for a clean fill determination and a DEQ permit exemption for offsite reuse; excavated materials below the near-surface layer are suitable for a clean fill determination.

4.5 Traffic Considerations

A traffic impact analysis was conducted by Global Transportation Engineering to evaluate suitable site access options, existing transportation facilities, and to identify potential offsite improvements required to adequately serve the filtration facility. This analysis is based on the standards established by Multnomah County and Clackamas County for each jurisdiction's roadways. The following information summarizes the *Traffic Impact Analysis TM* included in Appendix D (Global Transportation Engineering, 2019).

This section includes discussion of the following topics:

- Existing Conditions
- Potential Site Access Locations
- Traffic Impact Analysis
- Traffic Considerations Summary

4.5.1 Existing Conditions

Roadway functional classification, existing transportation facilities, and intersections were reviewed along SE Carpenter Lane, SE Dodge Park Boulevard, SE Altman Road, SE Cottrell Road, SE Bluff Road, and SE Proctor Road (Figure 4-4).

Modes of travel, including pedestrian, bicycle, transit, and motor vehicle, were evaluated. No available sidewalks, bike lanes, on-street parking, or transit routes were identified within the study intersections. Based on the *Multnomah County Transportation System Plan*, SE Dodge Park Boulevard, SE Cottrell Road, SE Bluff Road, and SE Altman Road have been designated as freight routes with no additional restrictions on vehicle size. The *Clackamas County Transportation System Plan* does not designate roadways in the study area as freight routes.

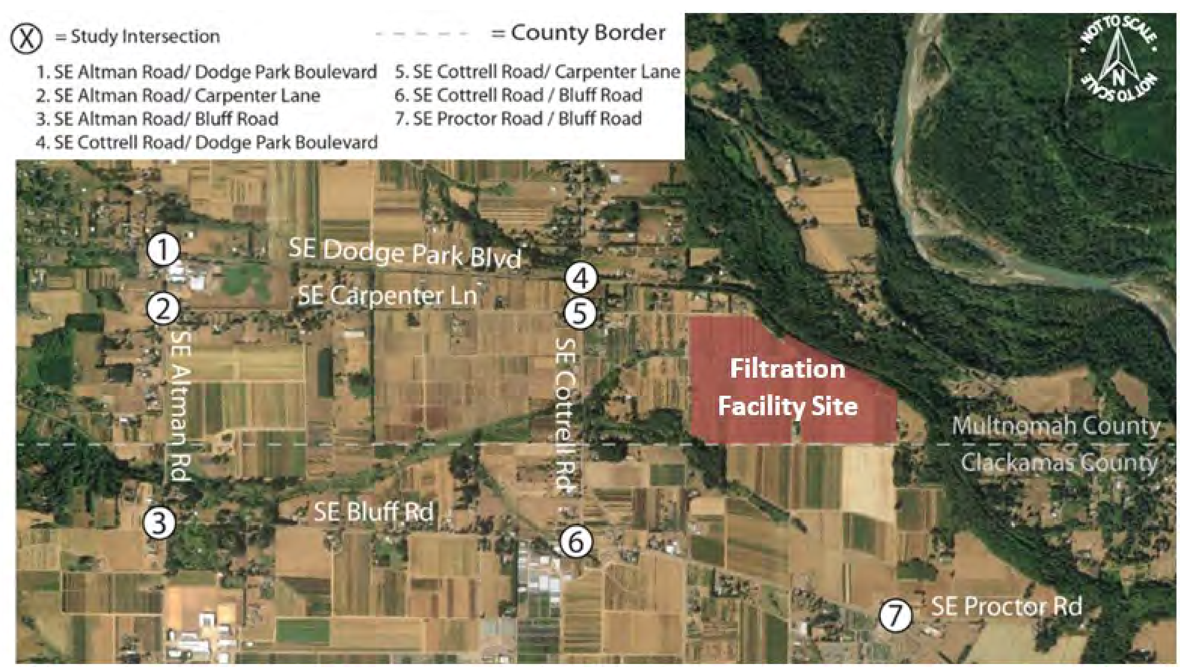


Figure 4-4. Roadways and intersections evaluated near site

4.5.2 Potential Site Access Locations

The filtration facility site is proposed to be accessed from one location from the north and a second location from the south. Four potential site accesses and seven intersections were evaluated using the standards established by the American Association of State Highway and Transportation Officials and the requirements of Multnomah and Clackamas counties. The four potential site access points are shown in Figure 4-5 and include:

North Access Points:

- SE Carpenter Lane
- SE Dodge Park Boulevard
  - Access A: Located 2,765 feet east of SE Cottrell Road
  - Access B: Located 1,460 feet east of SE Cottrell Road

South Access Point:

- SE Bluff Road

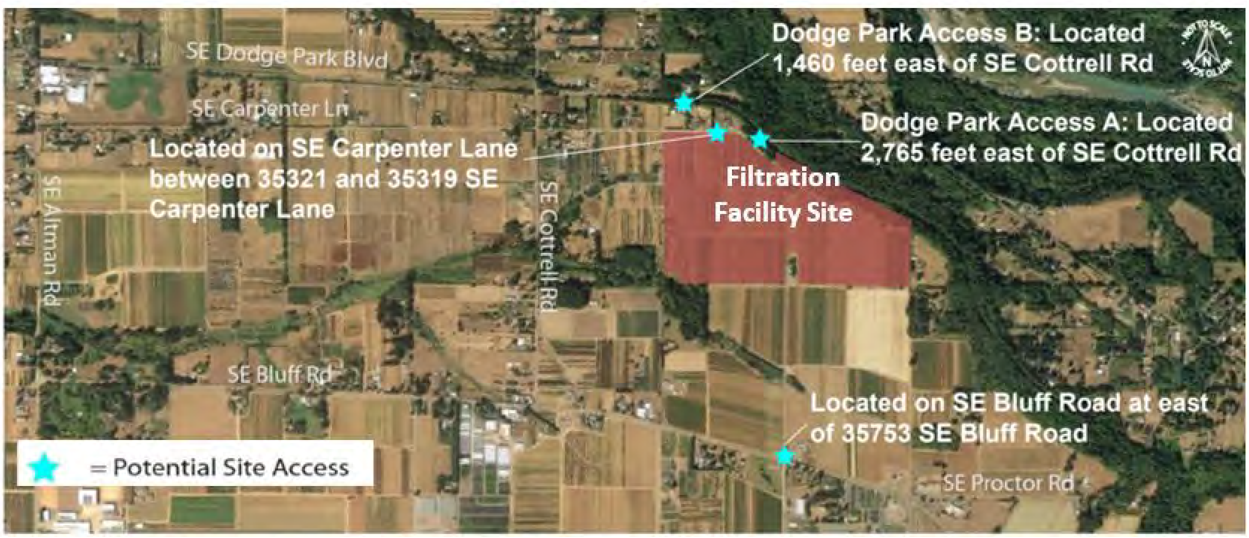


Figure 4-5. Four potential access points to access the site were evaluated

A summary of the field evaluation of the four potential site accesses and nearby roadway characteristics follows:

- Access on SE Carpenter Lane
  - Low speed limit. Slower travel speeds should be beneficial to residents living along the alignment and safer for truck traffic.
  - No pavement markings present. Pavement marking improvements will be necessary.
  - Narrow roadways. May require some widening or shoulder work to carry an increase in traffic.
  - Pavement structure. Structural upgrades are likely required to both SE Carpenter Lane and SE Cottrell Road to accommodate an increase in trucks.

- Access on SE Dodge Park Boulevard (Access A)
  - SE Dodge Park Boulevard with its higher functional classification can support site truck traffic.
  - Winding roadway limits location of access along SE Dodge Park Boulevard due to sight distance constraints.
  - Inclined, winding roadway with dense overhead tree canopy may be hazardous during inclement weather.
  - Access is located within a geological hazardous overlay zone. The Geotechnical Technical Advisory Committee will need to determine the feasibility of this access location. The permitting team will need to coordinate with Multnomah County to determine if this overlay zoning issue can be resolved.
  - Access is located within an environmental habitat overlay zone. The permitting team will need to coordinate with Multnomah County to determine if this overlay zoning issue can be resolved.
  - Requires new road and access easement from SE Dodge Park Boulevard to the northern site access.
  - Requires extensive slope excavation, as well as slope and vegetation management.
- Access on SE Dodge Park Boulevard (Access B)
  - SE Dodge Park Boulevard with its higher functional classification can support site truck traffic.
  - Access will need to be located no further than 1,460 feet from the SE Cottrell Road centerline to maintain the minimum sight distance requirements to the east.
  - Requires new road and access easement from SE Dodge Park Boulevard to the north entrance of the site. Discussions with Multnomah County will be needed to determine if access will be considered a new roadway or driveway.
  - Based on Multnomah County design standards, the following applies:
    - If this access is classified as a local commercial/industrial street, the minimum public intersection spacing between the intersecting roadways of SE Carpenter Lane and SE Dodge Park Boulevard is 200 feet and will require a 150-foot intersection spacing between the proposed access and SE Cottrell Road.
    - If the access is classified as a private access driveway, it will require a 100-foot setback from SE Cottrell and will require a minimum 50-foot access driveway spacing along SE Dodge Park Boulevard.
  - Requires vegetation management.
- Access on SE Bluff Road
  - SE Bluff Road with its higher functional classification can support site truck traffic.
  - Access road improvements to trigger a Clackamas County SCA permit.
  - Requires vegetation to be trimmed east of the access and SE Bluff Road intersection.
  - Requires a new access and an easement from the site to SE Bluff Road.

4.5.3 Traffic Impact Analysis

A post-construction traffic impact analysis was conducted to assess the potential impact of new vehicle traffic generated by the filtration facility to the local transportation network. Existing traffic volumes, traffic operations, and lane configurations in the study area were reviewed. The analysis used the existing 2019 traffic volumes and assumed an annual background traffic growth rate of 2 percent to forecast future 2040 traffic conditions.

To assess the future condition, peak hour trips for filtration facility operation were developed. Site trips that may be generated include facility operations and maintenance (O&M) staff, chemical deliveries, solids off-hauling, and public tours. Public tours are likely to occur outside of peak traffic periods and are therefore not included in the peak period traffic projections. Table 4-2 summarizes an estimate of site-generated trips with an estimated 23 morning peak hour (7 to 9 a.m.) trips and 20 afternoon peak hour (4 to 6 p.m.) trips added to the local transportation network.

Table 4-2. Summary of Anticipated Average Daily Weekday Site-generated Trips

	7 to 9 a.m. Peak Hour			4 to 6 p.m. Peak Hour			Off-Peak Hour		
	Enter	**Exit**	**Total**	**Enter**	**Exit**	**Total**	**Enter**	**Exit**	**Total**
Facility O&M Staff[a]	16	4	20	4	16	20	0	0	0
Chemical Deliveries and Solids Off-Hauling	3	0[b]	3	0	0	0	0	3	3
Total	**23**			**20**			**3**		

a. *Of the site-generated trips, 80 percent will enter the site and 20 percent will exit during the morning peak hours while 20 percent will enter the site and 80 percent will exit during the afternoon peak hours.*

b. *Assumes truck trips will leave the site outside the peak periods.*

A performance analysis of the study intersections was conducted to determine the expected traffic conditions once the filtration facility is operational. The morning and afternoon peak periods and intersection level of service were analyzed. The study intersections were evaluated for the following scenarios:

- 2019 Existing Conditions
- 2040 Background Traffic Conditions (no-build)
- 2040 Total Traffic Conditions (site buildout)
  - Scenario A: SE Carpenter Lane
  - Scenario B: SE Dodge Park Boulevard
  - Scenario C: SE Bluff Road

A collision analysis of crash data determined the study intersections do not exceed the 1.0 crashes per million entering vehicle safety threshold. No crash pattern was found, and safety mitigation is not required for the study intersections.

Left and right turn warrants were evaluated for the unsignalized study intersections and potential site accesses. Left and right warrants are developed based on traffic volumes for when to install turn lanes. Signal warrants were also reviewed for the unsignalized study intersections. Signal warrants are used to determine whether installation of a traffic control signal is justified at a particular location.

To evaluate the worst case for each scenario, it was assumed site-generated trips would use the identified access. However, site-generated trips are expected to use SE Dodge Park Boulevard and SE Bluff Road since these roadways are county-designated truck routes, have wider lane widths, are classified as collectors, and provide connections to US 26. In addition, SE Bluff Road provides direct access to the City of Sandy.

4.5.4 Traffic Considerations Summary

This section summarized the preliminary analysis of existing traffic conditions, potential access routes, and post-construction traffic impacts from the filtration facility.

Key considerations for project definition include:

- The potential impacts of the filtration facility on the local transportation system were evaluated using existing and future conditions at study intersections. The analysis indicates that the study intersections and four potential site accesses will operate at an acceptable level of service meeting both Multnomah County and Clackamas County standards under each access scenario.
- Left-turn lane, right-turn lane, and signal warrants for the proposed accesses and seven study intersections were evaluated under all scenarios. Based on this analysis, turn lane and signal improvements are not required to serve the filtration facility adequately. Further analysis is not required to determine whether a traffic control device or other improvement is justified.
- Intersection sight distance for the north and south proposed access points for the filtration facility were evaluated. The site distance measurement assumes stop control operations at the proposed site accesses.
- The evaluation of traffic safety near the filtration facility determined there are no existing roadside features that are anticipated to impact intersection or stopping sight distance at the four potential accesses or within the study area. Also, the study intersections do not exceed the safety threshold as indicated by the crash data for a 5-year period.
- Additional analysis of construction traffic impacts will be conducted during the design phase.

4.6 Acoustic Considerations

This section describes baseline sound measurements taken by Greenbusch Group and associated code requirements for the filtration facility site. The following information summarizes the *Acoustic Design Criteria and Baseline Measurement TM* in Appendix E (Greenbusch Group, 2019).

This section includes discussion of the following topics:

- Regulatory Criteria
- Existing Sound Levels
- Design Criteria
- Sound Mitigation
- Acoustic Considerations Summary

The auditory response to sound is a complex process that occurs over a wide range of frequencies and intensities. Decibel levels (dB) are a means of expressing this range of intensities with a numerical scale. The minimum sound level variation perceptible to a human observer is about 3 dB. Sound level meters and monitors use a filtering system to approximate human perception of sound, which is less sensitive to sounds outside the speech frequency range. Measurements made using this filtering system are referred to as “A weighted” and are called “dBA.” Table 4-3 summarizes the dBA of common sound levels. These levels serve as a comparison point for the sound measurements taken at the site as described in this section.

Table 4-3. Weighted Levels of Common Sounds[a]

Sound	Sound Level (dBA)	Approximate Relative Loudness[b]
Jet Plane at 100 ft.	130	128
Rock Music with Amplifier	120	64
Thunder, Danger of Permanent Hearing Loss	110	32
Boiler Shop, Power Mower	100	16
Orchestral Crescendo at 25 ft.	90	8
Busy Street	80	4
Interior of Department Store	70	2
Ordinary Conversation at 3 ft.	60	1
Quiet Car at Low Speed	50	1/2
Average Office	40	1/4
City Residence, Interior	30	1/8
Quiet Country Residence, Interior	20	1/16
Rustle of Leaves	10	1/32
Threshold of Hearing	0	1/64

a. *Data is summarized from United States Department of Housing and Urban Development, Aircraft Noise Impact Planning Guidelines for Local Agencies, November 1972.*

b. *As compared to ordinary conversation at 3 ft.*

4.6.1 Regulatory Criteria

Multnomah County Code (MCC) and Clackamas County Code (CCC) include requirements that limit sound emissions from the filtration facility. The site is located in Multnomah County on the border of Clackamas County. Sound levels received at properties within Multnomah County are governed by MCC, while sounds received at properties within Clackamas County are governed by CCC requirements. Both MCC and CCC contain requirements for sound measurement equipment.

Multnomah County Code. MCC 15.269 includes permissible sound levels at or within the boundary of properties containing noise sensitive units. MCC 15.266 defines noise sensitive units as follows:

> Any building or portion thereof, vehicle, boat, or other structure adapted or used for the overnight accommodation of persons, including, but not limited to individual residential units, individual apartments, trailers, hospitals, and nursing homes.
>
> *MCC 15.266*

Table 4-4 provides the maximum permissible sound levels defined in MCC 15.269.A. Note that MCC does not specify the noise metric the limits are based on.

Table 4-4. MCC Maximum Permissible Sound Levels

Daytime (7 a.m.–10 p.m.)	Nighttime (10 p.m.–7 a.m.)
60 dBA	50 dBA

MCC 15.269.B also prohibits sound from being plainly audible between 10 p.m. and 7 a.m. within a noise sensitive unit or on a public right-of-way at a distance of 50 feet or more from the sound source.

MCC 15.270.B exempts sounds caused by emergency work, or by the ordinary and accepted use of emergency equipment, vehicles, and apparatus, whether or not the work is performed by a public or private agency, upon public or private property.

Clackamas County Code. CCC 6.05.040 prohibits sound levels from exceeding the limits shown in Table 4-5.

Table 4-5. CCC Maximum Permissible Sound Levels

Daytime (7 a.m.–10 p.m.)	Nighttime (10 p.m.–7 a.m.)
60 dBA	50 dBA

The limits shown in Table 4-5 are applied 3 feet from windows and doors of noise sensitive units. CCC 6.05.020 defines noise sensitive units as follows:

> Any building or portion thereof, currently and regularly used for the overnight accommodation of persons, including, but not limited to individual residential units, individual apartments, hospitals, and nursing homes.
>
> *CCC 6.05.020*

CCC 6.05.050 provides exemptions from the sound level limits listed in CCC 6.05.040, including the following:

- Sounds caused by emergency work, or by the ordinary and accepted use of emergency equipment, vehicles, and apparatus, whether or not such work is performed by a public or private agency, upon public or private property (CCC 6.05.050.B).
- Sounds caused by industrial, commercial, timber-harvesting, or utility organization or workers during normal operations (CCC 6.05.050.F).

4.6.2 Existing Sound Levels

Continuous sound level measurements were made at six locations along the perimeter of the filtration facility site between Friday, April 5 and Monday, April 8, 2019 (Figure 4-6). The intent of these measurements was to document existing background sound levels near the site prior to project construction and to aid in development of design criteria. Local noise sources included street traffic, aircraft, and nature sounds. Additional information about the background sound level measurements is included in the *Acoustic Design Criteria and Baseline Measurement TM* in Appendix E (Greenbusch Group, 2020).

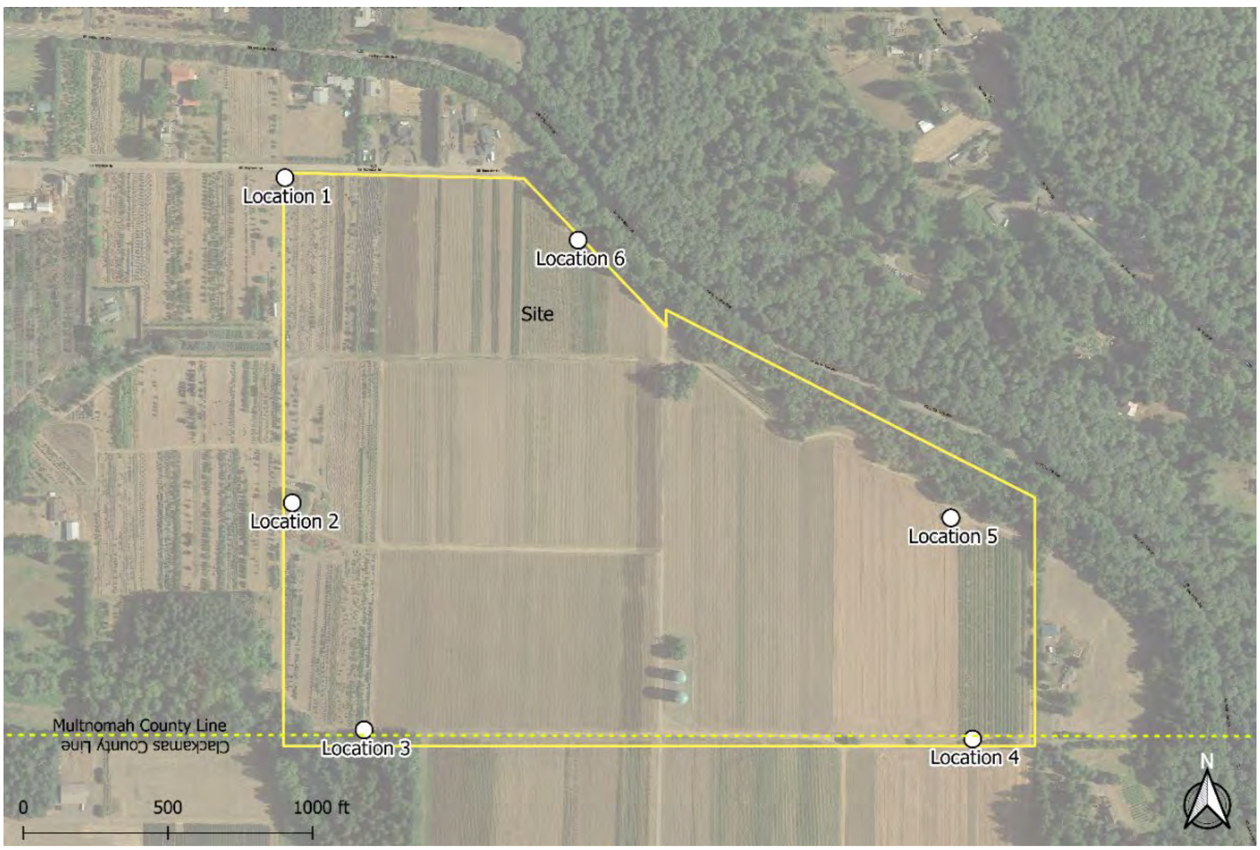


Figure 4-6. Acoustic measurements were taken at six locations along the site perimeter

Measurement locations at the north and northeast sides of the site were generally higher than those at the west and south portions of the site. This is likely due to the proximity of SE Carpenter Lane and SE Dodge Park Boulevard.

Note that MCC and CCC do not specify the noise metric for the sound level limits. Therefore, noise metrics designated in the Oregon Administrative Rules (OAR) were used. The measured hourly Equivalent Sound Levels (L_{eq}) are also reported for information purposes.

Measured sound levels are summarized in Table 4-6.

Table 4-6. Measured Hourly Sound Levels, min-max (median), dBA

Location	Daytime (7 a.m.–10 p.m.)				Nighttime (10 p.m.–7 a.m.)			
	L_{01}	L_{10}	L_{50}	L_{eq}[b]	L_{01}	L_{10}	L_{50}	L_{eq}[b]
Location 1	47–70 (58)	41–55 (49)	37–46 (41)	41–58 (47)	39–61 (46)	34–54 (42)	33–47 (38)	33–50 (40)
Location 2	51–65 (57)	42–54 (50)	38–51 (44)	41–52 (48)	40–60 (51)	35–54 (48)	33–49 (44)	34–51 (45)
Location 3	48–59 (56)	40–53 (49)	36–49 (44)	41–51 (47)	36–59 (50)	33–51 (45)	31–48 (42)	34–50 (43)
Location 4	48–66 (57)	46–58 (48)	46–50 (46)	46–55 (48)	46–55 (48)	46–48 (46)	46–47 (46)	46–47 (46)
Location 5	48–62 (57)	44–59 (51)	38–54 (45)	42–55 (48)	40–56 (50)	36–51 (44)	34–47 (41)	35–49 (42)
Location 6	56–65 (61)	48–58 (54)	47–53 (49)	48–57 (52)	47–66 (56)	47–57 (52)	46–52 (49)	46–56 (50)
MCC and CCC Code Limits[a]	60				50			

a. Noise metric not defined in MCC or CCC limits; therefore, OAR metrics were used.

b. Measured hourly L_{eq} sound levels are also reported for information purposes.

Existing median hourly daytime L_{50} sound levels at the filtration facility site range between 41 to 49 dBA and median nighttime L_{50} levels range between 38 to 49 dBA. Average sound levels at the site are below daytime and nighttime MCC and CCC sound level limits. This data was used to develop the acoustical design criteria for the site.

4.6.3 Design Criteria

The filtration facility must comply with sound level limits established by MCC and CCC. Therefore, the design criteria cannot be louder than 60 dBA during daytime hours (7 a.m. to 10 p.m.) and 50 dBA during nighttime hours (10 p.m. to 7 a.m.).

Since normal operations at the site after construction are anticipated to be constant in nature, these limits are applied as hourly L_{50} for the environmental analysis of facility operations. MCC and CCC limits are independent of ambient conditions.

Table 4-7 below presents the increase to the ambient sound environment if MCC and CCC limits are used as the design criteria. These estimate cumulative statistical sound levels under the assumption that existing and project sound levels would combine logarithmically, based on a steady-state condition.

As shown in Table 4-7, if MCC and CCC limits are used to establish the design criteria, the ambient noise environment is expected to increase by an average of 15 dB during daytime hours and 8 dB during nighttime hours. Ambient sound levels at Location 1 are anticipated to increase by up to 19 dB during daytime hours and 12 dB during nighttime hours.

Sound levels at the site are required to be quieter than MCC and CCC limits. Beyond these limits, it is recommended the designer identify feasible opportunities to reduce noise emissions from the site.

Table 4-7. Increase to Ambient Conditions from Code Limits, Hourly L_{50}

Location	Daytime (7 a.m.–10 p.m.)				Nighttime (10 p.m.–7 a.m.)			
	Existing	Code	Total	Increase	Existing	Code	Total	Increase
Location 1	41	60	60	19	38	50	50	12
Location 2	44	60	60	16	44	50	51	7
Location 3	44	60	60	16	42	50	51	9
Location 4	46	60	60	14	46	50	51	5
Location 5	45	60	60	15	41	50	50	9
Location 6	49	60	60	11	49	50	52	3
Median	45	60	60	**15**	43	50	51	**8**

4.6.4 Sound Mitigation

Sound mitigation will likely be required to comply with codified sound limits. Sound mitigation may include the following:

- Installing noise walls and/or berms between the facility and adjacent properties.
- Locating equipment as far from property lines as feasible.
- Using acoustical louvers and/or sound traps in areas containing loud equipment or operations.
- Using acoustically absorptive materials in spaces housing loud equipment or operations.
- Enclosing outdoor mechanical equipment within sound enclosures.
- Limiting deliveries or material hauling to daytime hours, as feasible.
- Using ambient sensing broadband alarms, where safe to do so.
- Upgrading exterior doors and seals.
- Upgrading mufflers for vehicles used on site.

4.6.5 Acoustic Considerations Summary

This section summarized the baseline acoustic measurements and code requirements applicable to the filtration facility site.

Key considerations for project definition include:

- The site is required to comply with sound level limits established by MCC and CCC.
- Codified sound limits are an average of 15 dB louder than existing daytime (7 a.m. to 10 p.m.) sound levels and an average of 8 dB louder than existing nighttime (10 p.m. to 7 a.m.) sound levels.
- Sound mitigation will likely be necessary for the site to meet codified sound level limits.

4.7 Geotechnical Considerations

This section summarizes preliminary geotechnical considerations based on work performed by RhinoOne Geotechnical, including information summarized from the *Geotechnical Data Report* in Appendix F (RhinoOne Geotechnical, 2020). This preliminary work was based on a review of geologic and hazard mapping reports, site reconnaissance, previous subsurface explorations, and explorations conducted for project definition.

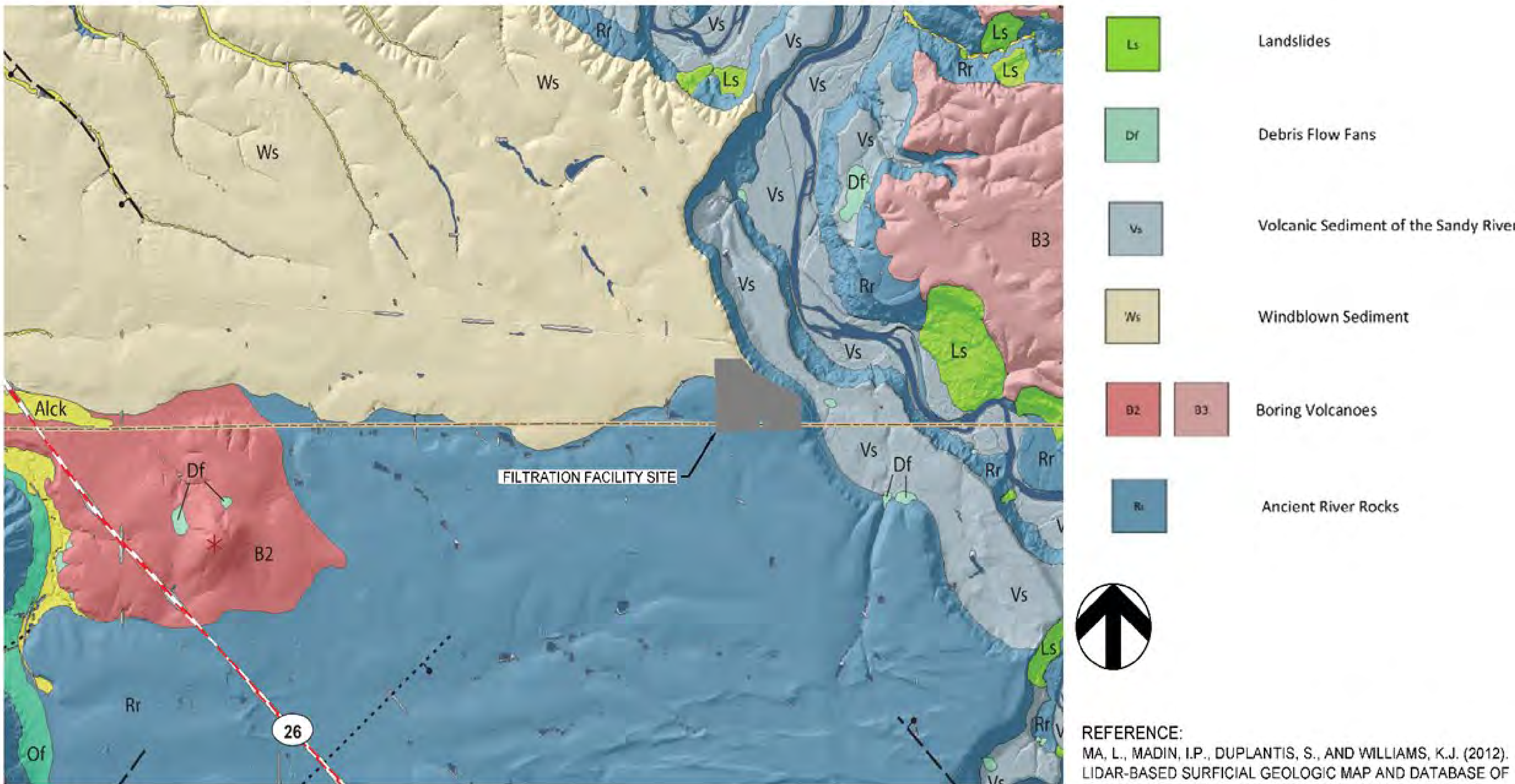
This section includes discussion of the following topics:

- Geology
- Soil
- Groundwater
- Quaternary Faults
- Earthquake Induced Landslide Probability
- Liquefaction Probability and Lateral Spreading
- Geotechnical Considerations Summary

4.7.1 Geology

The filtration facility site is near the eastern extent of the Portland Basin, west of the Sandy River drainage near the foothills of the Cascade Mountains (Figure 4-7 below). The site is on Ancient River Rock deposits between Boring Lava basalt flows to the west and the foothills of the Cascade Mountains to the east. The site is approximately 600 feet above the Sandy River. An ancient river terrace is located roughly 300 feet down steep valley walls just east of the site, before dropping another 300 feet to the current Sandy River floodplain. The area is part of the larger Puget Sound-Willamette Valley physiographic province, a tectonically active lowland situated between the Coast Range to the west and the Cascade Mountains to the east (Orr and Orr, 1999).

The Ancient River Rock unit consists of sandstones, siltstones, and conglomerates from sediment deposits left by ancient rivers that flowed through the region. Basement rock in the vicinity of the filtration facility site are similar to those exposed in the adjacent Boring Lavas and foothills of the Cascade Mountains, which consist primarily of the Miocene (20 to 10 million years before present) Columbia River Basalt Group. This basalt group consists of thick flows that have been folded and faulted from regional compressional tectonics.



REFERENCE:
MA, L., MADIN, I.P., DUPLANTIS, S., AND WILLIAMS, K.J. (2012). LIDAR-BASED SURFICIAL GEOLOGIC MAP AND DATABASE OF THE GREATER PORTLAND AREA, CLACKAMAS, COLUMBIA, MARION, MULTNOMAH, WASHINGTON, AND YAMHILL COUNTIES, OREGON, AND CLARK COUNTY, WASHINGTON. OREGON DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES. OPEN-FILE REPORT O-12-02

Figure 4-7. Oregon Department of Geology and Mineral Industries map showing regional geology near the filtration facility site

4.7.2 Soil

The preliminary geotechnical work included subsurface field explorations consisting of three borings and nine test pits at the filtration facility site. Topsoil was encountered in each of the borings and test pits to depths ranging from 1.5 to 2 feet below ground surface (BGS). The topsoil consists of soft to stiff silty clay with some sand. This is the nursery till zone. The topsoil is underlain by red-brown to brown silty clay with trace fine sand to sandy silt with clay (alluvium) to depths between 20 to 40 feet BGS. The clay is generally medium-stiff to stiff with medium to high plasticity, while the silt is generally soft and interbedded with clay. Sands with silts (decomposed sedimentary rocks) were observed below the clay and silt layers to the maximum depth explored of 100.2 feet BGS. The sands are arranged in layers of silty sand to sands with silt and are generally loose to very dense with increasing density with depth. Between depths of 65 to 75 feet BGS, the sands are described as relict coarse sands and gravel. Below a depth of 75 feet, the sands are described as decomposed conglomerate.

4.7.3 Groundwater

Information provided by the USGS *Estimated Depth to Groundwater Study of the Portland Metropolitan Area*, along with a review of existing well logs and previous geotechnical investigations in the area, indicates the groundwater table is likely at a depth greater than 150 feet (Figure 4-8 below). Perched groundwater was observed in each of the borings at depths between 25 and 33 feet BGS. Seepage was also observed in Test Pit 2 and Test Pit 4 between depths of 6.5 to 8 feet. Based on the relatively low infiltration rates and the presence of shallow groundwater, infiltration may not be feasible at the site.

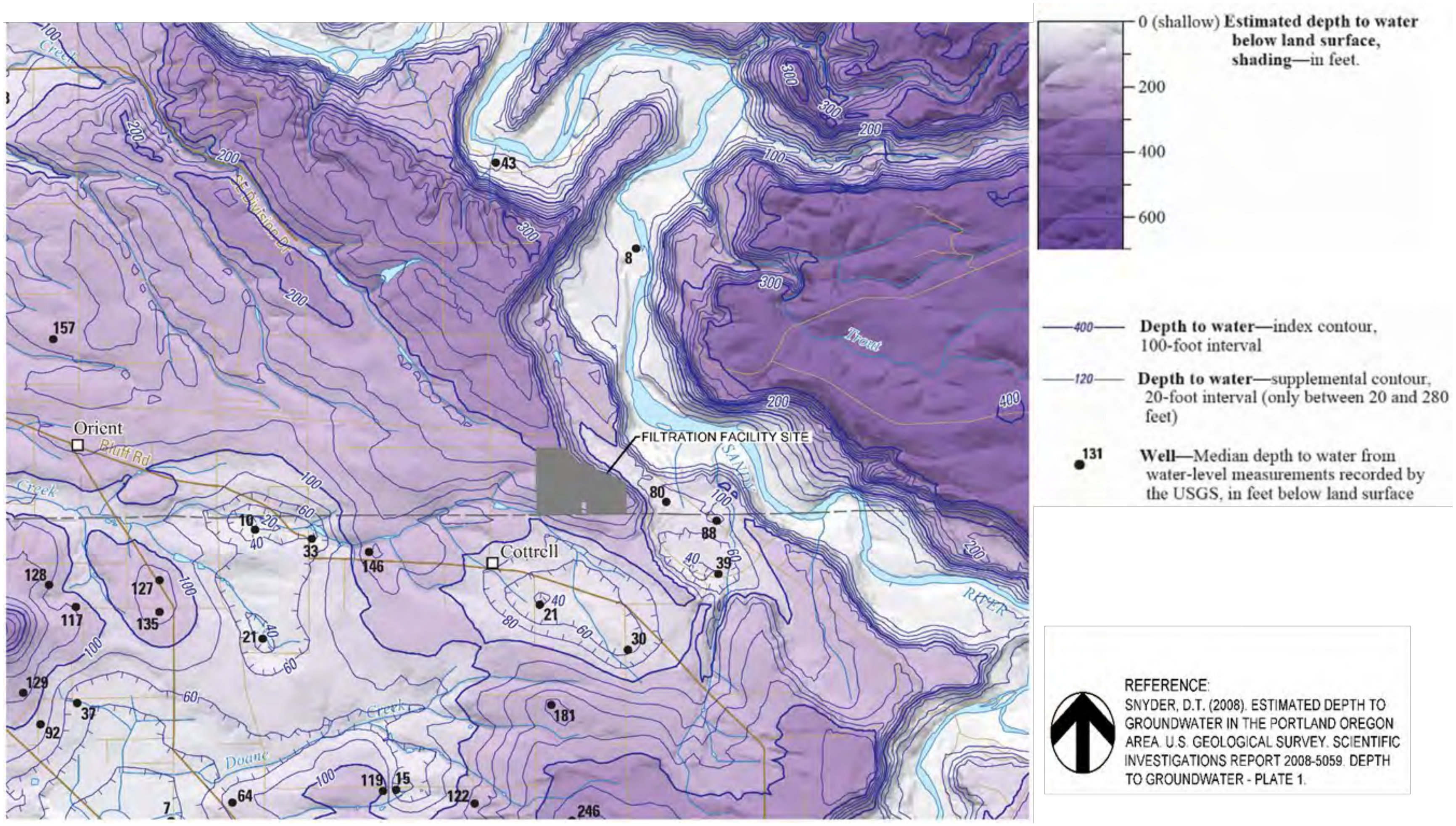


Figure 4-8. USGS map showing estimated depth to groundwater

4.7.4 Quaternary Faults

The USGS Quaternary Fault and Fold Database and a study by Geomatrix indicate that the Damascus-Tickle Creek fault zone, shown in blue on Figure 4-9, is mapped west of the site. This fault zone consists of numerous short northeast- and northwest-trending faults that form a broad, northeast-trending fault zone. These faults fold and offset rocks of the Pliocene Troutdale Formation, Plio-Pleistocene Springwater Formation, and Pleistocene Boring Lava. Most of these faults are thought to be near-vertical reverse faults with a significant component of right-lateral strike-slip. There are no known historic earthquakes attributed to these faults.

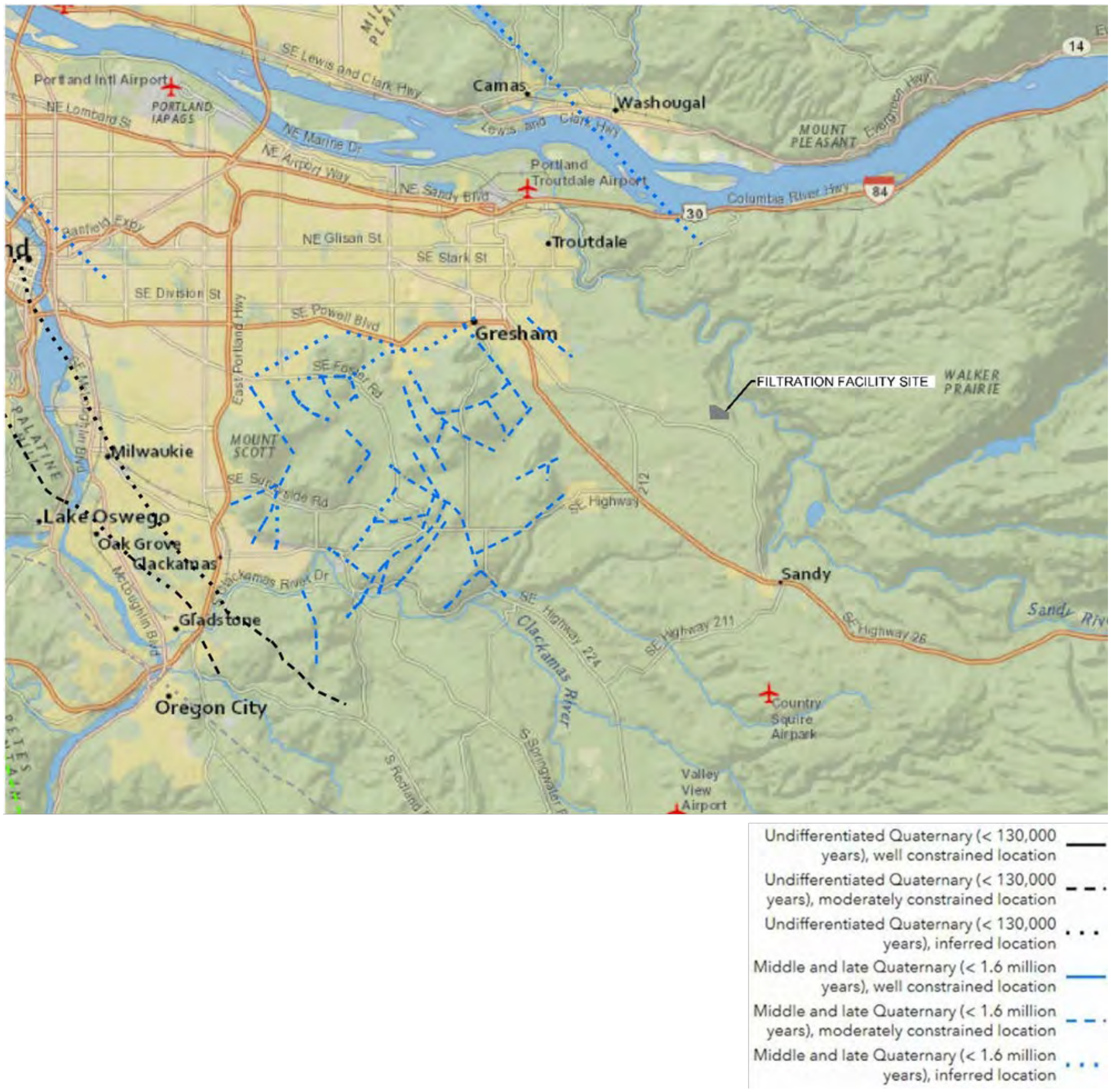


Figure 4-9. Quaternary fault map showing regional fault zones in relation to the filtration facility site

4.7.5 Earthquake Induced Landslide Probability

Landslide probability mapping suggests that the gentle slopes of the ancient river terrace and the rolling plains do not generally exhibit significant landslide potential. Figure 4-10 and Figure 4-11 below show the earthquake-induced landslide hazard map for the facility site. The steeper slopes above and below the ancient river terrace are mapped as having a high to very high landslide probability in the event of an earthquake. The hazard map is supported by landslide inventory mapping completed by the Oregon Department of Geology and Mineral Industries. A series of smaller landslides are mapped along the steep terrace riser slopes below the facility site and along SE Dodge Park Boulevard further southeast.

Larger landslides are present along the lower terrace riser above the Sandy River adjacent to SE Lusted Road northeast of the site. Two small areas mapped as debris flow fans exist along SE Dodge Park Boulevard near the intersection with SE Proctor Road. The debris deposited near the base of the steep slope would have originated from slope failures within stream drainages. The steep slopes below the facility site on SE Dodge Park Boulevard and SE Lusted Road are mapped with a high to very high earthquake induced landslide probability.

The landslide probability mapping hazards are based on calculations expected from a magnitude 9.0 Cascadia earthquake. However, preliminary seismic evaluation indicates that the landslide risk is likely surficial and local, rather than deep-seated and global. If structures are sited with a slope setback of approximately 200 feet, it is anticipated that the risk of damage from an earthquake-induced landslide will be substantially mitigated. Additional analysis will be performed during design to determine if ground improvements are needed to further mitigate the risk.

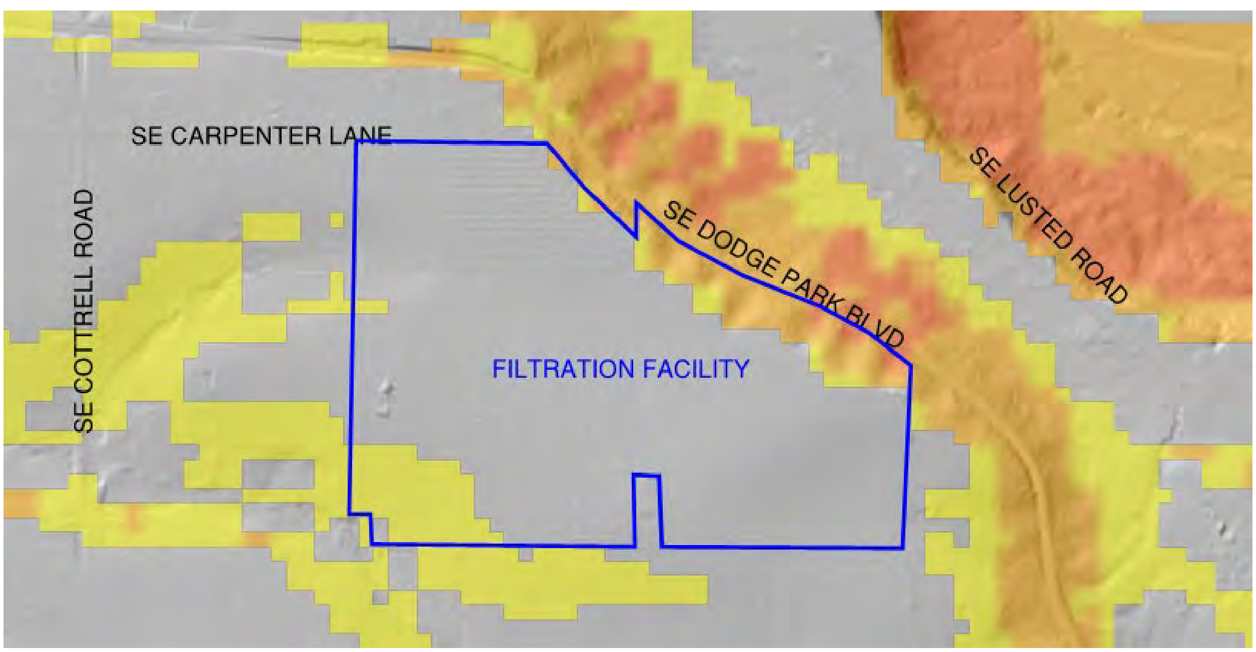


Figure 4-10. Landslide probability map showing some medium probability areas near the filtration facility site

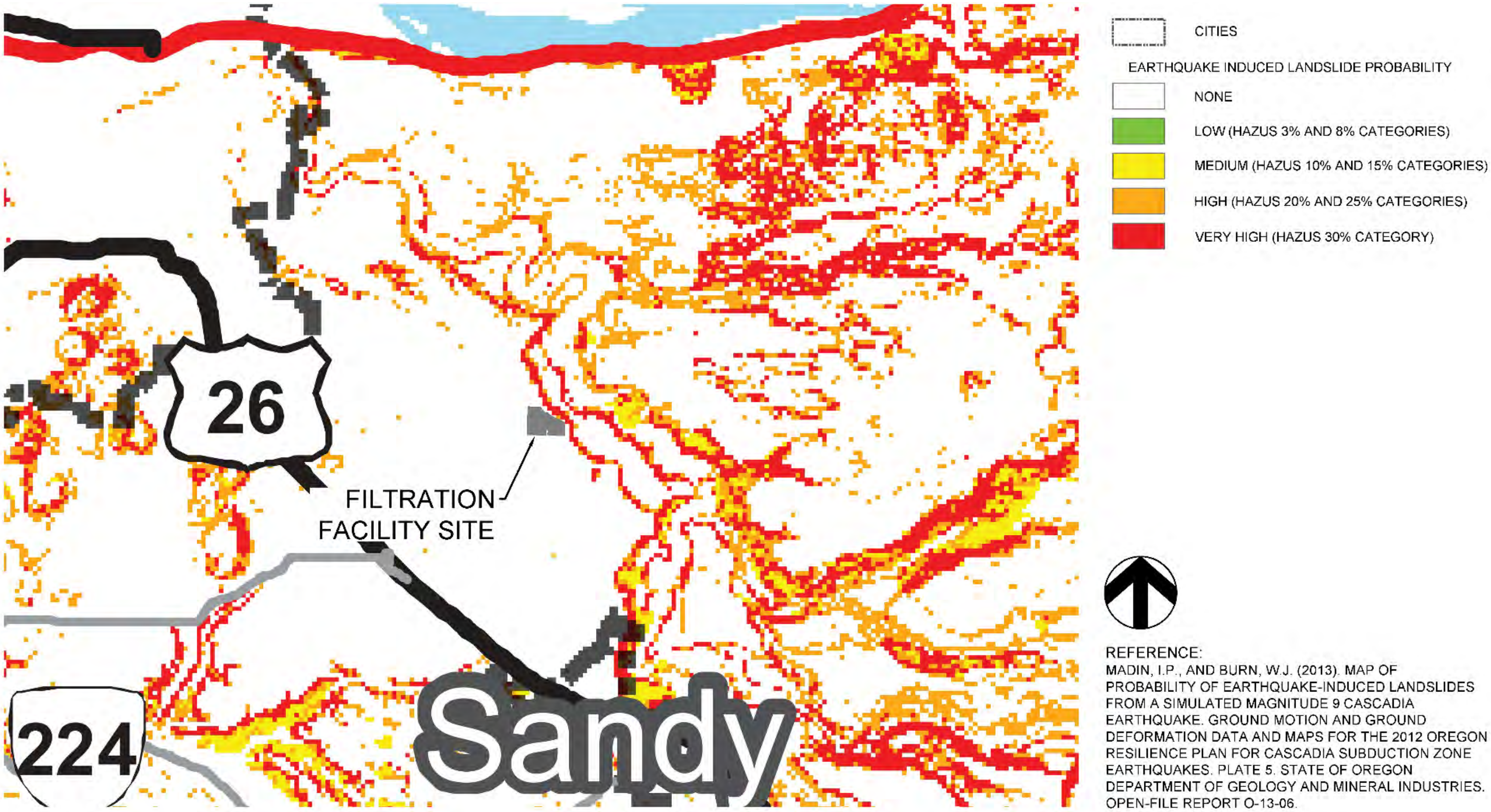


Figure 4-11. Landslide probability map showing medium and high probability areas located northeast of the filtration facility site

4.7.6 Liquefaction Probability and Lateral Spreading

The regional liquefaction probability and liquefaction-induced lateral spreading permanent ground deformation maps show 0 to 5 percent probability for liquefaction for the floodplains along the Sandy River (Figure 4-12 below). The mapping estimates the probability of liquefaction expected from a magnitude 9.0 Cascadia earthquake. Similarly, the lateral spreading induced permanent ground deformation is mapped as none to low (0 to 4 inches). It is possible that discrete zones of loose, saturated sand layers are present beneath the ancient river terrace that could liquefy during an earthquake. Locations closer to the lower terrace may be at higher risk of ground deformation from differential settlement and/or lateral spreading.

On the filtration facility site, the softer sand and silt layer below the 25- to 40-foot depth, where groundwater is present, shows potential for liquefaction and settlement on the order of 2 to 6 inches under seismic loading. Preliminary seismic evaluation indicates that there is a risk of seismic-induced liquefaction of soft soil layers. It is likely that the major structures and utilities will need foundation support (e.g., piles, stone columns, or other ground improvement). The details of this foundation support will be developed further in design.

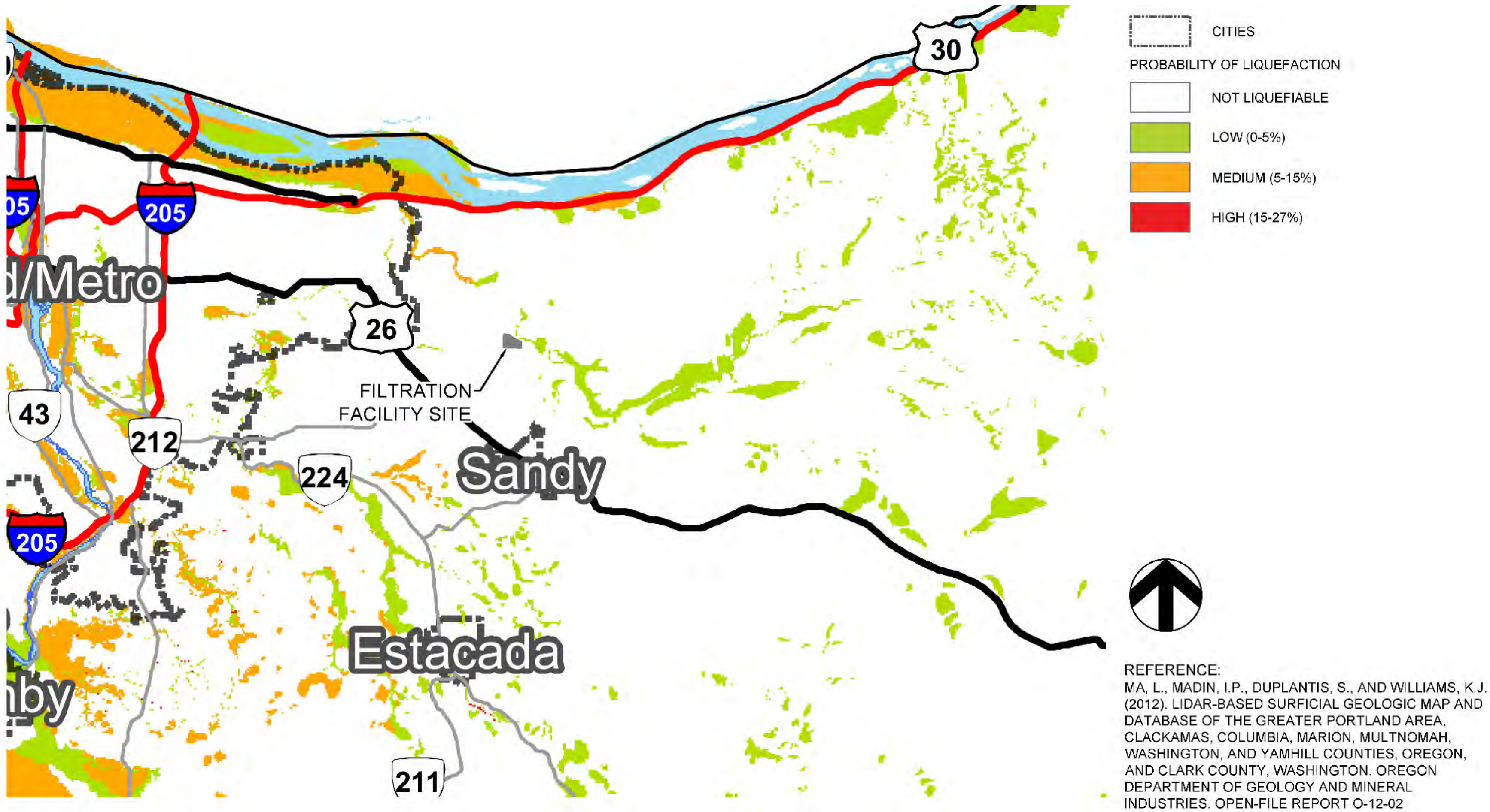


Figure 4-12. Liquefaction probability map

4.7.7 Geotechnical Considerations Summary

This section summarized preliminary geotechnical studies and research conducted to characterize the suitability of the site for the filtration facility. Based on review of existing data sources and preliminary geotechnical explorations and data, the preliminary conclusions of the geotechnical engineering team and the Geotechnical Technical Advisory Committee include:

- The site has a layer of softer sand and sandy silt at 20 to 40 feet depth that will probably be prone to settlement in an earthquake.
- The steep slope northeast of the site has localized landslides, but is likely not at risk of a larger global seismic-induced failure. The slope and terrace are considered stable overall.
- The lower steep slope above the Sandy River (north of SE Lusted Road) is much higher risk, as the river is undermining the slope. Key facilities should not be located near this slope.
- It is likely that buildings and structures will need piles or other foundation improvements to prevent vertical settlement. However, the risk of seismic-induced lateral spread is considered low.

The designer will further develop and quantify these hazards during design, with oversight from the program team.

4.8 Hydraulic Considerations and Ultimate Capacity

While PWB's early analysis during the site selection identified an approximate and preliminary hydraulic grade line (HGL) range to and from the site, the analysis did not specify a recommended HGL on the site or consider ultimate capacity or buildout that may occur.

The following sections summarize the evaluation of HGL on the site to assess impacts to pumping and site layout and subsequently consider how the planned ultimate capacity will affect the filtration facility planning, layout, design, and cost.

This section includes discussion of the following topics:

- HGL Options
- Ultimate Capacity Options
- Hydraulic Considerations and Ultimate Capacity Summary

The existing HGL of the Bull Run conduit system from the Diversion Pool at Headworks allows delivery of 210 million gallons per day (mgd) by gravity to the Powell Butte reservoirs. The elevation of much of the filtration facility site is above the existing HGL. Flow of raw water by gravity from Headworks to the site between 100 to 160 mgd appears feasible, depending on the selected elevation of the filtration facility with respect to the HGL.

Figure 4-13 was developed during the project pre-planning and illustrates the existing and proposed HGLs at the site. Note the figure shows elevation on the y-axis and distance on the x-axis and is not to scale.

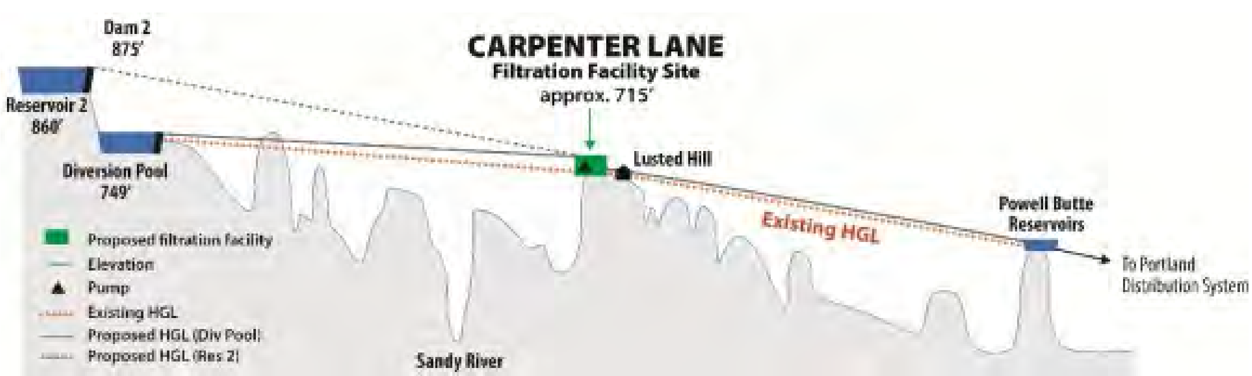


Figure 4-13. Existing and proposed HGLs shown with 160 mgd flow rate

An initial alternatives assessment compared gravity and pumped raw water HGL options during the Ultimate Capacity and HGL Workshop with the program team on December 11, 2018. These two options were further refined to incorporate three ultimate capacity expansion options.

- Gravity raw water HGL:
  - 160 mgd ultimate capacity
  - 240 mgd ultimate capacity
  - 320 mgd ultimate capacity

- Pumped raw water HGL:
  - 160 mgd ultimate capacity
  - 240 mgd ultimate capacity
  - 320 mgd ultimate capacity

The options to increase to 320 mgd capacity were considered unlikely, as a third dam would likely need to be built to enable reliable delivery of that amount of water from the Bull Run Watershed. However, the 320 mgd options are included in this analysis for comparative purposes.

These alternatives were evaluated using existing available information and preliminary analysis including:

- Preliminary hydraulic modeling of the Bull Run conduit system.
- Preliminary filtration facility hydraulic profile.
- Comparative costs developed to Association for the Advancement of Cost Estimating Class 5 standard.
- Conceptual filtration facility layout based on conservative assumptions.
- Site mapping based on existing Light Detection and Ranging data.
- 3D Civil earthwork volumes generated from the conceptual facility site layout.

The evaluation of these alternatives is discussed in detail in the sections below.

4.8.1 Hydraulic Gradeline Options

Figure 4-14 below illustrates the schematic hydraulic profiles of the filtration facility for gravity (solid line) and pumped (dashed line) options under the Diversion Pool head of 749 feet elevation.

The following assumptions were used in the comparison analysis of gravity versus pumping HGL options:

- The filtration facility treatment process uses 25 to 30 feet of the available HGL
- Two miles of new pipeline will be installed from the Hudson Road Intertie to the filtration facility

To achieve a capacity of 160 mgd, the new raw water pipe connections were assumed to be two 90-inch diameter pipes. Pipe size may vary depending on factors such as the alignments selected, pipeline phasing, and hydraulic model calibration.

To achieve 240 mgd delivery of raw water to the filtration facility, additional infrastructure would be needed. One option is to harden or replace the existing three conduits from Headworks to the Hudson Road Intertie with one or two new pipelines capable of sustaining the higher pressure of the Dam 2 reservoir. The existing Diversion Pool, with a head range of 747 to 753 feet, could be bypassed and the higher Dam 2 head, ranging from 830 (mean low) to 860 feet (high operating level), could be used. This would allow delivery of 240 mgd to the filtration facility by gravity.

Gravity HGL Option. The existing filtration facility site elevations are generally 690 to 735 feet (Figure 4-1). The site consists of a hill on the eastern portion of the property with the existing grade sloping gently downward to the northwest, southwest, and southeast. Preliminary modeling suggests that the optimum site inlet elevation would be approximately 715 feet above sea level as this would maximize gravity flow potential through the site and allow gravity delivery of approximately 160 mgd of raw water to the filtration facility (Figure 4-14 below).

Setting the filtration facility inlet elevation at 715 feet would limit the site layout area to the lower west side and require larger earthwork volumes for construction. This would increase project capital costs, but lower long-term operating costs. The gravity option would require larger raw water and finished water pipelines to reduce head loss. This would increase project capital costs but would avoid construction costs and long-term O&M costs associated with a pump station. Increasing capacity up to 240 mgd will require additional pipeline infrastructure as noted above.

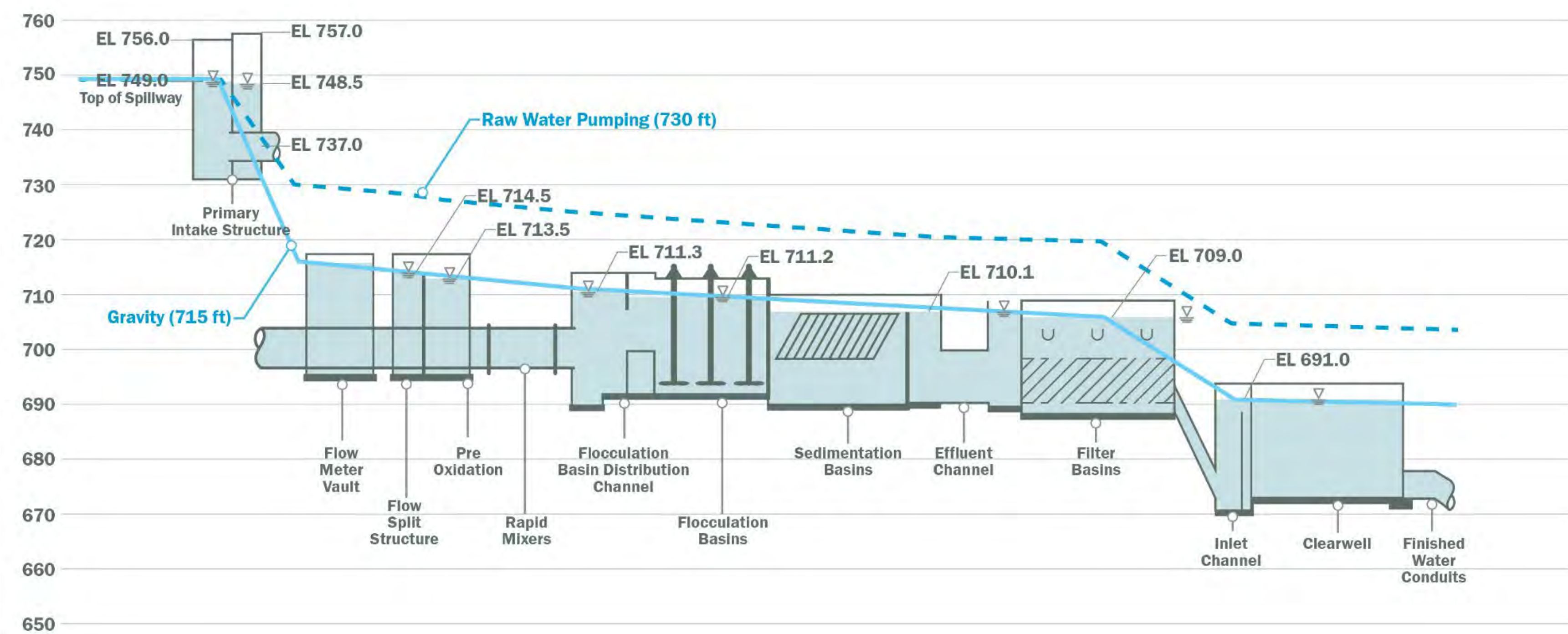


Figure 4-14. Gravity versus pumped HGL options based on 160 mgd

Pumped HGL Option. Another option considered would be to set the filtration facility inlet elevation at 730 feet. While this would optimize the cut and fill volumes for construction and reduce the cost of earthwork, it would also require pumping. The pumped option would allow for smaller finished water pipes, due to a higher filtration facility outlet elevation, and for smaller raw water pipes due to the higher pumped head available. An inlet elevation of 730 feet would limit gravity delivery to 105 mgd. When demands exceed 105 mgd, the raw water would have to be pumped. The addition of a pump station would increase capital, maintenance, and energy costs. For purposes of the initial analysis, it was assumed that water would be pumped when demands exceed 105 mgd.

Considerations for the gravity (715 feet) and pumped (730 feet) HGL options are summarized in Table 4-8. If the flow into the filtration facility is not hydraulically limited, both pumped and gravity options will allow delivery of 160 mgd of finished water from the filtration facility to the Powell Butte reservoirs by gravity.

Table 4-8. HGL Options at Filtration Facility Inlet

Option	Features
Gravity	• Filtration facility inlet elevation 715 ft. • Allows for conveyance of approximately 160 mgd raw water without building new conduit to Headworks • Limits primary process trains to lower west side of site • Requires deep pipeline(s) across the east half of the filtration facility site • No additional land needed for a pump station • Higher reliability in an earthquake or other disaster that interrupts electric power • Reduces energy and O&M costs for the lifetime of the facility • Lower visual profile • Life cycle cost is essentially the same as pumped option, for the same ultimate capacity
Pumped	• Filtration facility inlet elevation 730 ft. • Requires use of pump station when water demand is >105 mgd • Allows more flexibility in site layout • Minimizes site work by matching existing grade surface • Necessitates additional property acquisition along raw water pipeline (this could potentially be avoided by locating a deeper and more expensive pump station on the site) • Increases energy needs and O&M costs for the lifetime of the facility • Lower reliability in an earthquake or other disaster that interrupts electrical power • Higher visual profile • Life cycle cost is essentially the same as gravity option, for the same ultimate capacity

Within the accuracy of the comparative costs developed, there was no significant life cycle cost difference between the gravity and the pumped options, at the corresponding ultimate capacity. That is, the 160 mgd pumped option life cycle cost was approximately the same as the 160 mgd gravity option, considering earthwork, site piping and utilities, pump station energy needs, and O&M costs, and raw and finished water pipes. Therefore, the choice between gravity and pumped options focused on other comparative advantages and disadvantages.

4.8.2 Ultimate Capacity Options

During the project pre-planning as described in Chapter 1: Introduction, PWB recommended a capacity range for the filtration facility of 145 to 160 million gallons per day (mgd). The upper end of that range, the facility capacity of 160 mgd, is therefore used in the analysis throughout this report.

While the 145 to 160 mgd capacity range meets projected demands into the foreseeable future, the range does not account for ultimate site capacity and buildout, which extend beyond today's planning horizons and demand projections. Depending on the ultimate capacity, greater spacing of support facilities will be required to leave room for treatment train additions, additional treatment processes, greater mass excavation (cost efficient now), some upsizing of site piping and utilities, and potential upsizing of finished water pipelines.

Three options were evaluated for facility ultimate capacity. These options correspond to the assumed maximum demand on the system within the lifetime of the facility. For planning purposes, the lifetime of the facility is assumed to be 100 years.

- **160 mgd capacity.** Assumes the initial capacity of 160 mgd will be sufficient for the next 100 years, or that alternative water sources will be found for demands over 160 mgd.
- **240 mgd capacity.** Assumes the future filtration facility capacity will need to be increased by 50 percent within 100 years.
- **320 mgd capacity.** Assumes the future filtration facility capacity will need to be increased by 100 percent within 100 years.

Planning for potential future capacity expansion will increase initial construction cost, due to:

- **Mass earthwork**. Mass earthwork for the future footprint is assumed to be completed in the initial construction. It would be possible to defer this cost to a later phase, but at greater cost.
- **Site piping and utilities**. Site piping and utilities will have longer length pipe, conduit, and wiring to leave space for construction of future process basins.
- **Pump station**. The raw water pump station for the pumped option would not be expanded to 240 or 320 mgd, because it is assumed that new conduit will be built by the time the future expansion is needed. Pumping would become an unnecessary, stranded investment, since the new conduit will provide enough gravity flow capacity for assumed expansions.

The various pumped versus gravity HGL options and the 160, 240, and 320 mgd capacities were evaluated using a conceptual site layout (Figure 4-15 below). The conceptual site layout follows existing topographical "fall lines" where there are existing pathways for water to flow off site by gravity. These existing fall lines are useful to decide general facility orientation so that gravity flow potential is maximized and earthwork re-grading is minimized to help control costs. From the center of the site, one fall line is roughly northwest to southeast, and another is northeast to southwest. The northeast to southwest fall line is assumed for the conceptual site layout referenced throughout this report.

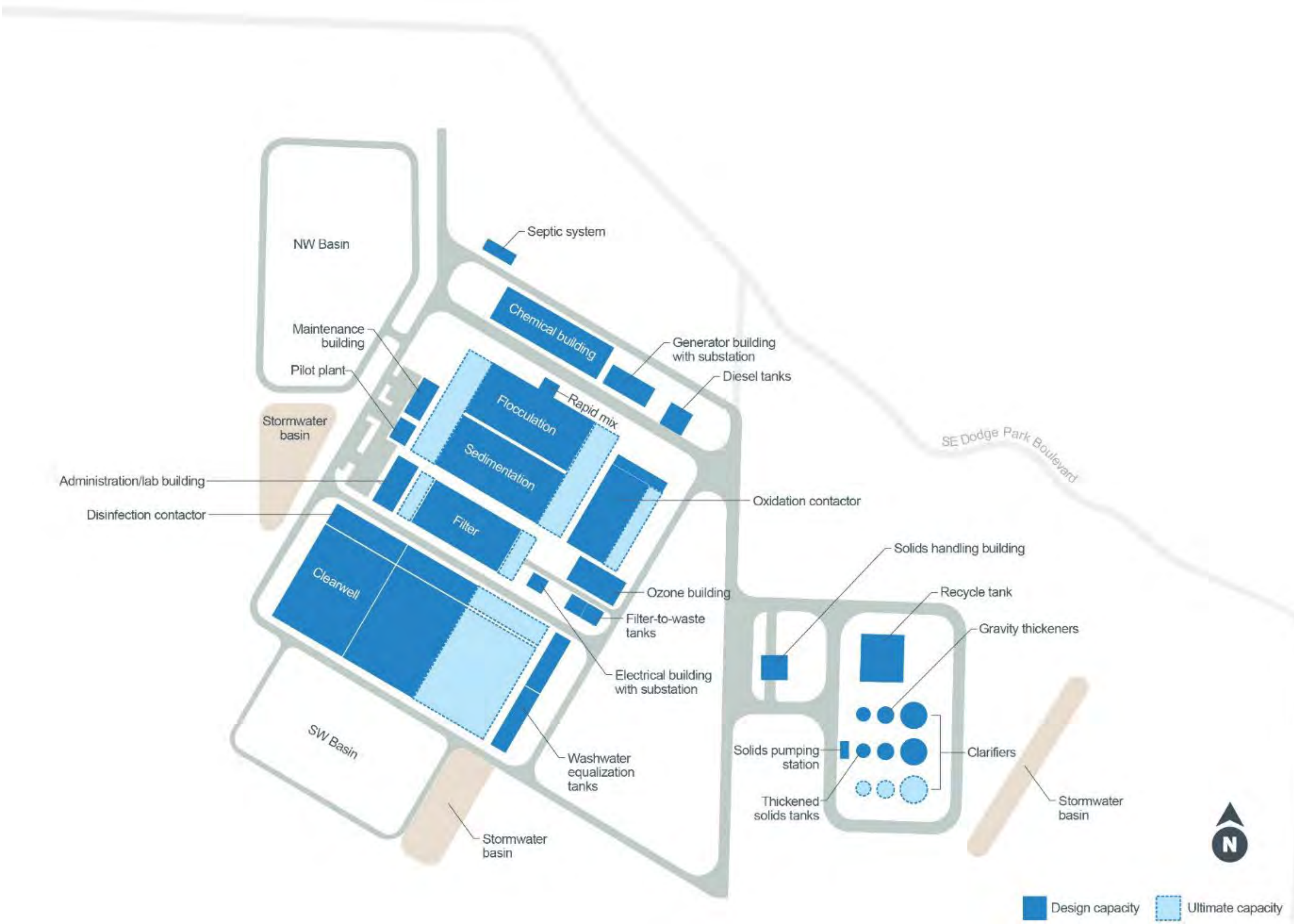


Figure 4-15. Conceptual site layout showing treatment processes and design capacity assumed for project definition evaluations

The conceptual site layout reflects assumed unit processes for a conventional filtration facility and was developed for illustrative purposes only. The site layout shows the initial 160 mgd design capacity with the ability to expand. The illustrated fall line alignment and conceptual layout of the facility are assumed for project definition and initial cost estimate development and subject to change during design.

To evaluate pumped and gravity options for different capacities, comparative layouts for each capacity option were developed, and comparative excavation and infrastructure costs were estimated. The pumped options require less excavation using the existing site grade while the gravity options require more excavation to provide an inlet water surface elevation of 715 feet. The pumped options assumed that a 160 mgd pump station would be needed that would be operated 3 months out of each year, during peak demand season.

As described in Table 4-8, the gravity options have significant advantages compared with the pumped options. At the same time, the pumped options had no offsetting cost advantages. At a workshop held December 11, 2018, PWB recognized the advantages of the gravity options and elected to proceed with further evaluation of only the gravity options.

Under gravity flow, three alternatives were considered:

- **160 mgd, gravity**: plan and design for a filtration facility with ultimate capacity of 160 mgd. Additional capacity could be built in the future, but it would be more difficult and more expensive than if the capacity was planned from the inception.
- **240 mgd, gravity:** plan and design for a filtration facility with ultimate capacity of 240 mgd. This would add initial cost, but would make future expansion simpler and less costly. This alternative would be much more flexible to respond to future needs than the 160 mgd alternative.
- **320 mgd, gravity:** plan and design for a filtration facility with ultimate capacity of 320 mgd. This alternative would add initial cost over the 240 mgd alternative. It is unlikely that the Bull Run Watershed could ever produce more than 240 mgd, unless a third dam is built at some time in the future; therefore, the potential benefit of planned capacity greater than 240 mgd is doubtful.

4.8.3 Hydraulic Considerations and Ultimate Capacity Summary

This section described the evaluation of HGL and ultimate capacity alternatives for the filtration facility. Based on the comparative analysis of pumped and gravity options using a conceptual site layout, the recommended HGL option and ultimate capacity option are as follows:

- **HGL recommendation** is a filtration facility inlet of 715 feet elevation to support maximum gravity capacity. This requires more site excavation than an HGL of 730 feet but avoids pumping. This recommendation also allows approximately up to 160 mgd by gravity from Headworks initially. The following summarizes key advantages of the gravity option:
  - Higher likelihood that the filtration facility may still be able to provide water after a seismic event or other major disaster.
  - Operational similarity to the existing gravity system.
  - Lower filtration facility profile helps reduce visual impacts to site neighbors.

  - Reduces energy requirements and minimizes the land required for an additional pump station facility.
  - Life cycle costs are approximately (within the accuracy of planning-level estimating) the same as the pumped option. Increased pipeline and earthwork cost for the gravity option is offset by the pump station capital and energy costs.
- **Ultimate capacity recommendation** is 240 mgd. The following summarizes advantages of setting ultimate capacity at 240 mgd:
  - Addresses anticipated future potential needs within the facility life cycle.
  - Can fit reasonably within the existing site. Provision for future 240 mgd capacity can be made at a reasonable cost.
  - Can be easily integrated with the existing system.
  - Larger capacity than 240 mgd is unlikely to have long-term benefit, given the physical and regulatory constraints of the Bull Run Watershed.

4.9 Facility Overflow Management

Overflow basins support facility O&M and provide relief for potential facility emergency overflows. Since basins can require a large site footprint and affect the layout of other facilities, it is important to size them early in the project. This section describes overflow management options for the facility, identifies overflow locations within the facility, calculates the associated volume at each location, determines the probability of occurrence or concurrence, and provides a recommendation of total minimum overflow volume.

For clarification, the following discussion refers to the existing conduits upstream of the filtration facility as the raw water *conduits* and to the new section of pipelines connecting those conduits to the facility as the raw water *pipelines*. Consequently, the existing conduits downstream of the filtration facility are the finished water *conduits* and the new section of pipelines connecting those conduits to the facility are the finished water *pipelines*.

This section includes discussion of the following topics:

- Overflow Management Alternatives
- Preliminary Onsite Overflow Basin Layout
- Operations and Maintenance
- Preliminary Basin Sizing
- Emergency Overflows
- Expansion Considerations
- Lined versus Unlined Overflow Basins
- Facility Overflow Management Summary

4.9.1 Overflow Management Alternatives

The following facility overflow management site alternatives were considered:

- Offsite Discharge to Sandy River
- Offsite Discharge to Local Drainage
- Onsite Basins

Maximum overflows would be the capacity of the raw water conduit delivery system, or 160 mgd (240 mgd for the ultimate expansion scenario). If there is insufficient room on site to handle required overflow functions and volumes, then offsite piping and facilities may be needed, which would present additional cost and permitting difficulty. If there is enough room on site, offsite alternatives will not be needed.

Offsite Discharge to Sandy River

Discharging overflow to the Sandy River would require a large-diameter (54- to 66-inch) pipeline from the site to the Sandy River (Figure 4-16). The most direct route to the Sandy River would be approximately 2,500 feet long, down steep slopes, and dropping approximately 600 feet from the site to the river. Such a pipeline would cross high seismic risk slopes, be very difficult to construct, and have high environmental impacts, including crossing heavily wooded terrain. The pipeline would include the following considerations:

- More costly than onsite overflow management.
- Significant environmental impacts to natural resources.
- Permit concerns relative to water treatment chemicals potentially entering the environment.
- Right-of-way acquisition.
- New pipeline in a seismic hazard area.
- Difficult to permit due to impacts on steep slopes and environmental overlay zone, and the need for a discharge permit to the Sandy River for 160 mgd.
- Onsite stormwater detention would still be needed as required by MCC.

Based on these considerations, this alternative was removed from further consideration.

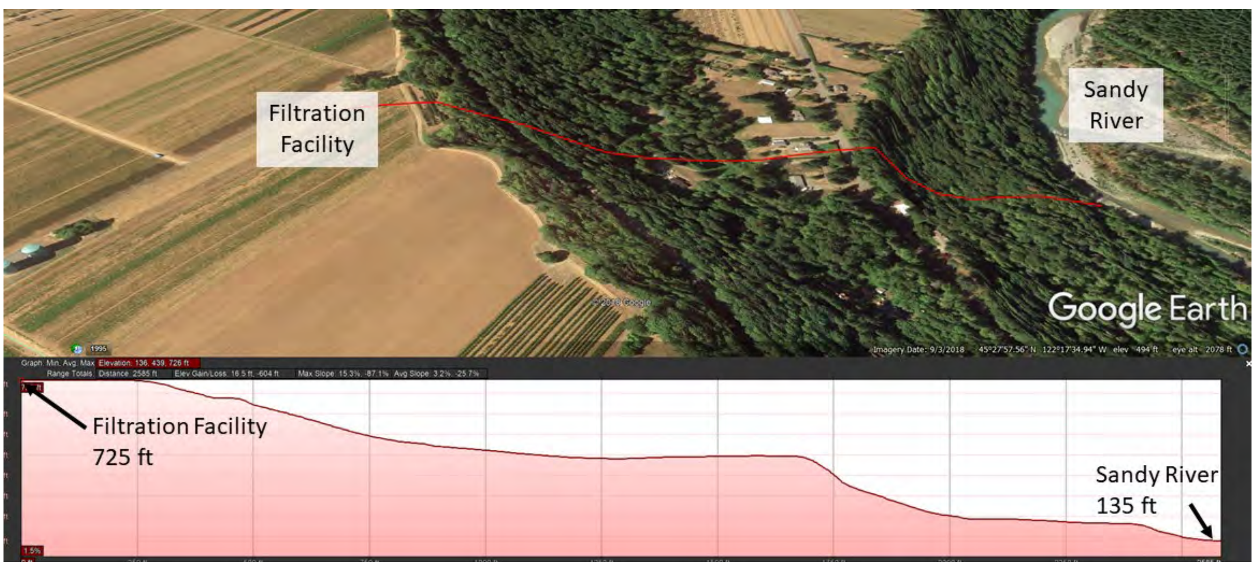


Figure 4-16. Overflow discharge to the Sandy River alternative

Offsite Discharge to Local Drainage

Discharging overflow to the local drainage, which is the headwaters of Johnson Creek (Figure 4-17) would include the following considerations:

- Volume (maximum 160 mgd) would overwhelm local drainage, which only carries 15 mgd in a 10-year storm.
- Onsite stormwater detention would still be needed as required by MCC.
- Not permittable at the flows anticipated because the overflow volumes are much larger than the base flow in the drainage.

- Permit concerns relative to water treatment chemicals potentially entering the environment.

This alternative has a fatal flaw, in that the flow allowed to the local drainage basin is significantly lower than the maximum overflow. Large detention basins would be required on site to attenuate the overflow before discharge. Based on these considerations, this alternative was removed from further consideration.

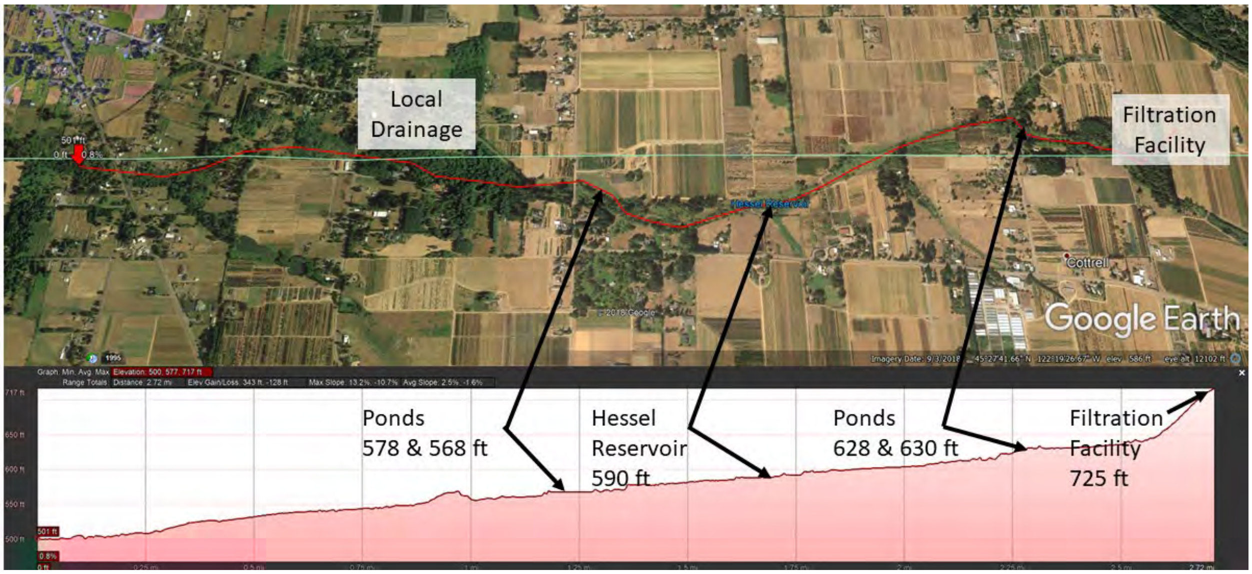


Figure 4-17. Overflow discharge to local drainage alternative

Onsite Basins

Onsite basins include the following considerations:

- Onsite handling is feasible and economical.
- No significant permitting issues.
- No significant environmental issues.
- Require considerable site area.

Based on the above considerations, onsite basins to handle overflows are the preferred alternative and will be further evaluated.

4.9.2 Preliminary Onsite Overflow Basin Layout

The filtration facility hydraulic grade is assumed to be 715 feet at the inlet with existing site elevations from 690 to 740 feet. The overflow basins need to be at a lower elevation on the site to accommodate each potential overflow area and drain each unit process by gravity. Based on these requirements, one overflow basin is shown in the northwest corner (NW Basin) and the other lower elevation basin in the southwest corner (SW Basin) of the conceptual site plan (Figure 4-18).

The two basins are different sizes and at different elevations to capture potential facility overflows by gravity flow. The NW Basin is larger (16 MG capacity) and will be dedicated to a correspondingly larger portion of the treatment facilities that may overflow to that area: the raw water pipeline and processes that have the required hydraulic grade to drain to the NW Basin (i.e., everything upstream of the filters).

Figure 4-18. Conceptual site plan illustrating potential overflow sources

The SW Basin is constrained by the natural site contours sloping down to the southwest and is, therefore, smaller (5 MG capacity) and at a lower elevation than the NW Basin. The SW Basin will be dedicated to a small portion of the treatment facilities that may overflow to that area and to processes that cannot drain to the NW Basin (i.e., everything downstream of, and including, the filters). Water in the SW Basin could be pumped to the NW Basin, if needed. The volume contributions from potential overflow sources to each overflow basin are discussed in the following sections.

4.9.3 Operations and Maintenance

General maintenance will include periodic draining of accumulated rain in the overflow basins, cleaning of sediment, weed control, and inspection and repair of liner, valves, and gates. Typical seasonal maintenance will involve taking one or more of the treatment process basins offline during low-flow periods, draining, inspecting, performing maintenance, and then refilling to repeat the maintenance activities for other process basins. Full facility shutdowns are expected to occur rarely; therefore, it is not necessary to design the overflow basins to simultaneously receive process basin drainage and the peak overflow.

If PWB were to implement pigging for raw water pipeline maintenance in the future, the overflow basins should be sized to accommodate this activity. Pigging is typically conducted at velocities of 3 to 5 feet per second. The maximum length of pigging for the new raw water pipelines would be 11,000 feet (starting at Hudson Road Intertie and ending at the filtration facility). With a pipe diameter of 96 inches, this would result in a total volume of 4.1 MG. Since this is less than the volume needed for emergency overflow, and the two would not coincide, provision for pigging will not govern the size of the basins.

4.9.4 Preliminary Basin Sizing

The conceptual site plan assumes the following volumes would drain from the unit processes (Figure 4-19 below).

- NW Basin (O&M process drainage requirements only):
  - Ozone contact basin: 1.2 MG
  - Flocculation basins: 3.0 MG
  - Sedimentation basins (full conventional): 3.8 MG

  TOTAL: 8.0 MG
- SW Basin (O&M process drainage requirements only):
  - Filters—negligible (assuming no off-spec water, see below)
    - Filters are typically drawn down to the filter media before shutting down.
    - In the event of a water quality upset that requires wasting filtered water, the overflow located downstream of the filters will give operators a place to send off-spec water. Diversion of off-spec filtered water could be implemented by simply closing the clearwell inlet gates and allowing off-spec water to overflow. This diverted water can result in a significant amount of water. This could lead to the need to waste the combined volume of the flocculation basins, sedimentation basins, and ozone contact basin, approximately 7.2 MG. This overflow would go initially to the SW Basin but could be pumped to the NW Basin.
  - Clearwell—negligible
    - Not included in these calculations; a clearwell/finished water reservoir is typically considered part of the distribution system and remains online to provide operational and emergency storage during a facility shutdown.
  - The maximum maintenance volume required in SW Basin is negligible, resulting in a minimum recommendation of 1.0 MG.

  TOTAL: 1.0 MG

For project definition, the worst-case scenario of a full facility shutdown is assumed.

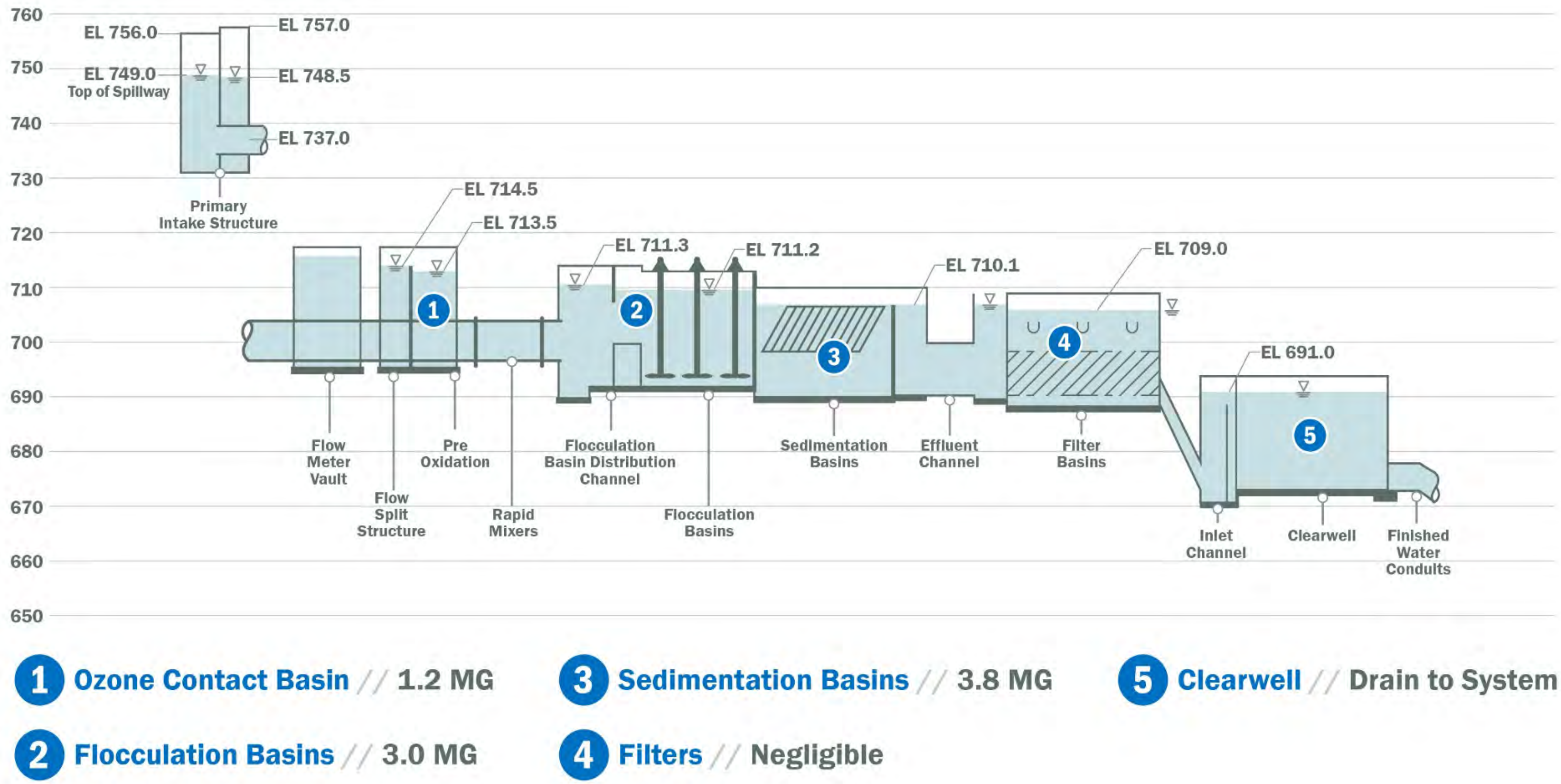


Figure 4-19. O&M overflow volumes

4.9.5 Emergency Overflows

Best practice filtration facility design includes gates and valves to isolate basins and/or full process trains for capacity turndown and routine maintenance. Closing valves and gates either inadvertently or purposefully can cause a basin or process train to overflow.

- Inadvertent overflows may be due to operator or equipment error such as:
  - Closing a gate or valve by accident.
  - Forgetting to open a gate or valve when adjusting facility flow.
  - Instrumentation failure.
- Intentional overflows are often initiated to protect water quality from:
  - Incorrect chemical feed system dosing resulting in problems meeting finished water quality goals.
  - Unstable or high turbidity values in the settled water feeding the filters that can lead to excessive particle loading and reduce filter productivity.
  - Filter breakthrough leading to elevated finished water turbidity.

Instrumentation should provide an alarm in the event of an overflow, limiting the overflow duration to no more than 15 minutes. Overflow channels, weirs, and pipes are also part of protecting filtration facility structures and should be included at each major process basin or basin group. Typically, filtration facilities have at least three overflow locations: the flocculation basin inlet, settled water channel, and clearwell outlet. Based on Bull Run conduit operations and a desire to meet Partnership for Safe Water goals, the conceptual site plan includes five overflow locations:

- Raw water conduits (unique to PWB)
- Flocculation basin inlet—pre-treatment inlet
- Settled water channel—pre-treatment outlet
- Clearwell inlet (unique to PWB)
- Clearwell outlet

Similar to O&M overflows, emergency overflows are dedicated to either the NW Basin or SW Basin based on the location of the overflow in the hydraulic profile. These five overflow locations are described in the following section.

Raw Water Conduits—NW Basin

Most systems have a pressurized raw water conduit that is controlled at the filtration facility and do not need a raw water overflow. However, the current operation of conduit flow is controlled at Headworks by throttling the inlet valves. This results in sections of open channel flow in the pipes and keeps the pipeline pressures low enough that the pressure rating of the conduits is not exceeded. The existing conduits cannot be subjected to the pressures that would result from attempting to control flow downstream at the filtration facility.

Flow control at Headworks is expected to continue when the filtration facility is online; therefore, an overflow for the raw water conduits must be provided at the filtration facility to allow for controlled valve closure to prevent surges (dynamic volume), and for a minimum volume to drain after the upstream valve is closed (static volume). The dynamic and static volumes are the total overflow volume for a raw water conduit overflow.

A preliminary ramp-down curve for the existing conduit system was developed for review using the parameters provided by PWB operations staff (Figure 4-20).

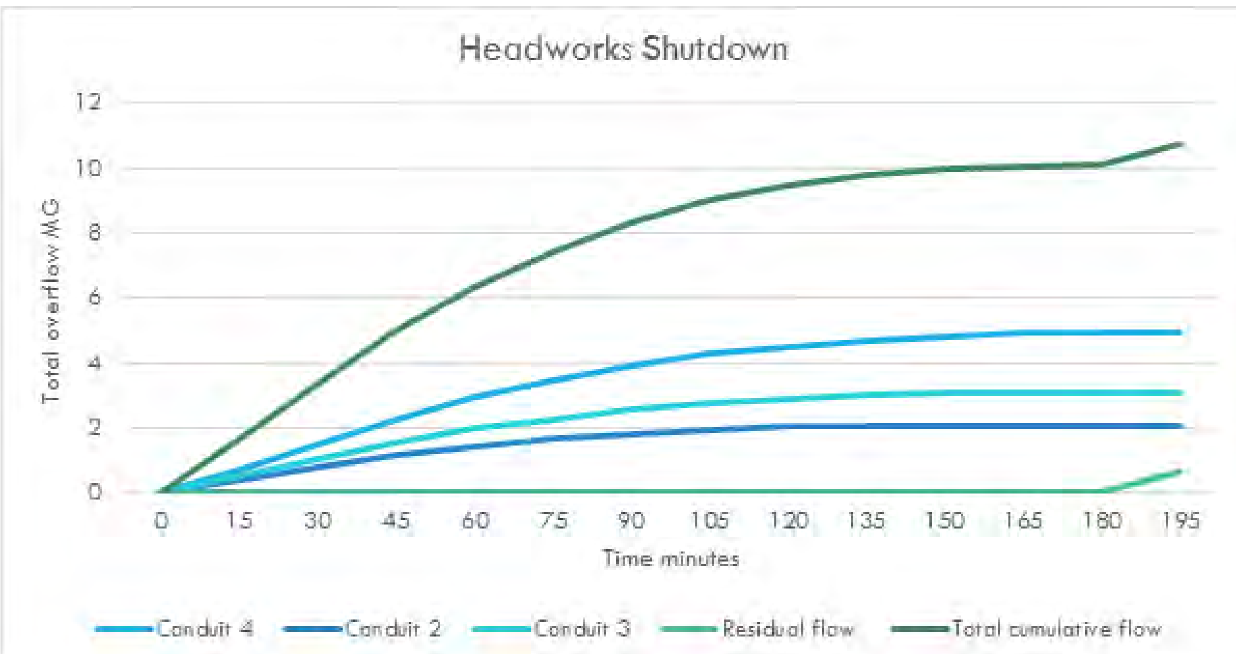


Figure 4-20. Headworks shutdown curve

Table 4-9 below shows a gradual ramp-down consistent with current Headworks operations. Sudden closure is avoided to prevent damage to the conduits. The resulting flow table and total volume curve are shown below. The overflow calculation shows a total of 10.7 mgd, allowing for a maximum of 165 minutes for the conduit shutdown (Conduit 4 takes the longest), plus 30 minutes for the filtration facility operator to react and contact the Headworks operator to initiate the shutdown. It is probably not necessary to allow more than 30 minutes reaction time, because the basin will have a 50 percent safety factor.

Table 4-9. Dam 2 Headworks Shutdown (Flow to Filtration Facility at 715 ft Elevation)														
Conduit	**Time (minutes)**													**Total Overflow (MG)**
Conduit 4	**15**	**30**	**45**	**60**	**75**	**90**	**105**	**120**	**135**	**150**	**165**	**180**	**195**	**—**
Flow	73	73	73	63	53	43	33	23	18	13	8	3	0	—
Flow per time period MG	0.76	0.76	0.76	0.66	0.55	0.45	0.34	0.24	0.19	0.14	0.08	0.03	0.00	—
Cumulative Flow	0.76	1.52	2.28	2.94	3.49	3.94	4.28	4.52	4.71	4.84	4.93	4.96	4.96	**4.96**
Conduit 2	**15**	**30**	**45**	**60**	**75**	**90**	**105**	**120**	**135**	**150**	**165**	**180**	**195**	**—**
Flow	37	37	37	27	22	17	12	7	2	0	0	0	0	—
Flow per time period MG	0.39	0.39	0.39	0.28	0.23	0.18	0.13	0.07	0.02	0.00	0.00	0.00	0.00	—
Cumulative Flow	0.39	0.77	1.16	1.44	1.67	1.84	1.97	2.04	2.06	2.06	2.06	2.06	2.06	**2.06**
Conduit 3	**15**	**30**	**45**	**60**	**75**	**90**	**105**	**120**	**135**	**150**	**165**	**180**	**195**	**—**
Flow	50	50	50	40	30	25	20	15	10	5	0	0	0	—
Flow per time period MG	0.52	0.52	0.52	0.42	0.31	0.26	0.21	0.16	0.10	0.05	0.00	0.00	0.00	—
Cumulative Flow	0.52	1.04	1.56	1.98	2.29	2.55	2.76	2.92	3.02	3.07	3.07	3.07	3.07	**3.07**
Residual flow after valves are closed (gravity flow of volume above 715 ft)														**0.65**
Total of all conduits and residual volume above 715 ft.														**10.7**

Figure 4-21 below provides a profile of the raw water conduit elevation from Headworks to the filtration facility site. Volume during shut down is assumed as follows:

- Dynamic volume: 10.0 MG
  - Volume that flows from dynamic head at the source during the Headworks valve shutoff
  - Assume valve cycle time as shown in Table 4-9 and full facility flow rate from 160 to 0 mgd over that time
- Static volume: 0.7 MG
  - Volume that flows out of the high points of the conduits after the Headworks valves are shut off
- Total volume: 10.7 MG

The probability of a raw water overflow occurrence is extremely low. Potential raw water overflows would most likely occur following a finished water pipeline disruption or when a water quality event at the filtration facility required an immediate flow reduction. It is assumed that the raw water pipelines would overflow to the NW Basin.

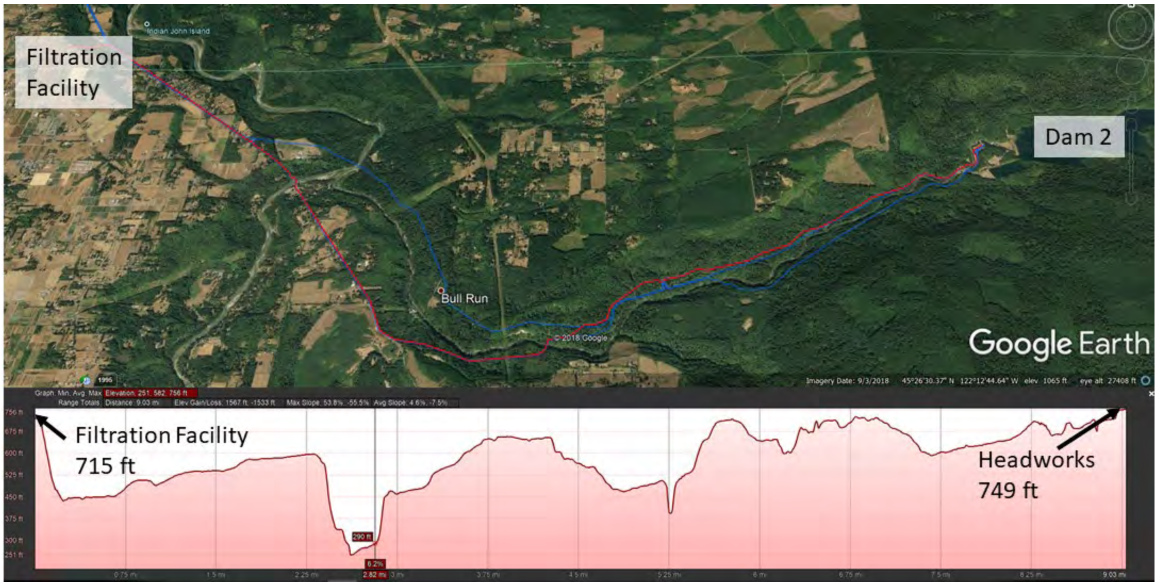


Figure 4-21. Raw water conduit elevation from Headworks to the filtration facility

Flocculation Basin Inlet—NW Basin

The conceptual site plan shows four flocculation and sedimentation trains with each train divided into two basins. Each train or basin will have an isolation gate to open when the unit is online and close when the unit is offline. The overflow pipe should be sized for maximum facility flow, but typical overflow volumes will be limited to the hydraulic capacity of one train or basin. Instrumentation should provide an alarm to limit duration to no more than 15 minutes.

- Maximum design capacity: 160 mgd
- Typical overflow rate and volume:
  - 20 mgd for 15 minutes
  - 0.2 MG

The probability of occurrence is low. An overflow can occur here if a large flow change is made without opening an additional gate or if the raw water conduit control malfunctions. The response to basin overflows will be further developed during design and the O&M engagement process. It is assumed flocculation/sedimentation basins would overflow to the NW Basin.

Settled Water Channel—NW Basin

The conceptual site plan shows 12 filters for the 160 mgd facility capacity. Each filter will have an inlet isolation valve to open when the filter is online and close when the filter is offline. The overflow pipe should be sized for maximum facility flow, but typical overflow volumes will be limited to the capacity of one filter.

- Maximum design capacity: 160 mgd
- Typical overflow rate and volume:
  - 20 mgd for 15 minutes
  - 0.2 MG

The probability of occurrence is low to moderate. An overflow can occur here if a large flow change is made without bringing an additional filter online, or if a filter experiences sudden premature turbidity breakthrough and has to be taken offline immediately. The overflow duration will be limited by reducing raw water supply at the Headworks, or by bringing on additional filter capacity. It is assumed that the settled water channel would overflow to the NW Basin.

Clearwell Inlet—SW Basin

An overflow should be provided between the combined filter effluent and the clearwell inlet. Design should include an overflow pipe sized for maximum facility flow, but typical overflow volumes will be limited in duration as instrumentation (e.g., water quality and flow instrumentation) should provide an alarm to limit overflow duration to no more than 15 minutes if possible.

- Maximum design capacity: 160 mgd
- Typical overflow rate and volume:
  - 160 mgd for 15 minutes
  - 1.7 MG

The probability of occurrence is typically extremely low. However, PWB's finished water goals make the clearwell inlet overflow location a valuable tool to prevent off-spec water resulting from a filter breakthrough from entering the clearwell. In this case, the probability of occurrence would increase from extremely low to low. For purposes of this discussion, extremely low would be once in several years, where low would be once or twice per year, as a deliberate water quality strategy. It is assumed that the clearwell inlet would overflow to the SW Basin.

Clearwell Outlet—SW Basin

The overflow pipe should be sized for maximum facility flow, but typical overflow volumes will be limited to the difference between flow into and out of the clearwell.

- Maximum design capacity: 160 mgd
- Typical overflow rate and volume: 0 mgd

The probability of occurrence is extremely low. The operational storage volume of the clearwell is intentionally sized to prevent this condition. However, the clearwell still needs an overflow weir and pipe to meet OHA and building code requirements. This will discharge to the SW Basin. In an emergency such as a sudden pipe break downstream of the clearwell or a failure of associated chemical feed systems, the facility would need to be shut down.

Assuming 15 minutes to shut down, or to re-direct the water flow to the NW Basin (shutting off basins upstream of the filters), the volume would be 1.7 MG. It is assumed that the clearwell outlet would overflow to the SW Basin.

Emergency Overflow Summary

The NW Basin should have a minimum emergency overflow volume of 10.7 MG for the worst-case condition of a raw water conduit shutdown. The SW Basin should have a minimum emergency overflow volume of 1.7 MG for the worst-case condition of sudden filter breakthrough.

The O&M and emergency uses for overflow basins are mutually exclusive, therefore the overflow volumes are not additive. O&M activities are planned activities that assume the basin will be empty; however, it is common for a filtration facility to be at least partially shut down during these activities, which would limit the volume of a potential overflow. Should an emergency overflow occur prior to a planned O&M activity, that activity should be postponed until volume in the basin is recovered.

- NW Basin minimum volume (160 mgd capacity):
  - Operational: 0.4 MG
  - Emergency: 10.7 MG
  - Total Volume (Emergency) = 10.7 MG
  - Factor of Safety = 1.5
  - Recommended Minimum Volume = 16 MG (rounded to the nearest 500,000 gallons)
- SW Basin minimum volume (160 mgd capacity):
  - Operational: 1.7 MG
  - Emergency: 1.7 MG
  - Total Volume (operational or emergency) = 1.7 MG
  - Factor of Safety = 2 (larger factor of safety for unknowns related to clearwell inlet overflows)
  - Recommended Minimum Volume = 3.0 MG (rounded to the nearest 500,000 gallons)

4.9.6 Expansion Considerations

This section revises recommended volumes based on the ultimate filtration facility capacity of 240 mgd and the criteria described above. Note that the NW Basin is sized based on emergency overflow. When 240 mgd is reached, it is assumed that a new pipeline or pipelines from Headworks to the filtration facility would be built. These pipelines could be designed to withstand pressure imposed by valve closure at the downstream end; therefore, the overflow amount to the NW Basin could be the same or even reduced from the initial 160 mgd facility.

- NW Basin minimum volume:
  - Operational: 0.4 MG (same basin size as 160 mgd, but more basins)
  - Emergency: 10.7 MG
  - Total Volume (emergency) = 10.7 MG (rounded to the nearest 500,000 gallons)
  - Factor of Safety = 1.5
  - Recommended Minimum Volume = 16 MG
- SW Basin minimum volume:
  - Operational: 2.5 MG

- Emergency: 2.5 MG
- Total Volume (operational or emergency) = 2.5 MG
- Factor of Safety = 2 (larger factor of safety for unknowns related to clearwell inlet overflows)
- Recommended Minimum Volume = 5.0 MG (rounded to the nearest 500,000 gallons)

4.9.7 Lined versus Unlined Overflow Basins

For the purposes of project definition, it is assumed that the overflow basins will be lined. This topic is to be examined in more detail during design to identify if lining is warranted based on the chemicals to be used at the filtration facility. If the basins are not lined, costs will be reduced, and by enabling percolation, it may be possible to divert a portion of incidental rainfall into the overflow basins from being recycled to the head of the facility for treatment.

As an alternative to the earth embankment with liner configuration identified for project definition, the overflow basins could be concrete basins with vertical walls. This would likely increase cost but may reduce the footprint. The choice of final configuration and construction details should be addressed during design to determine the best fit with the final site layout.

4.9.8 Facility Overflow Management Summary

The necessary overflow handling volumes that could be accommodated in two basins are listed in Table 4-10 for 160 and 240 mgd facilities:

Table 4-10. Recommended Overflow Basin Volumes[a]

Overflow Basin	160 mgd	240 mgd
NW Basin[b]	16 MG	16 MG[c]
SW Basin[d]	3 MG	5 MG

a. *Basin volumes are rounded to the nearest MG.*

b. *Safety factor of 1.5 for emergency overflow.*

c. *At 240 mgd, the emergency overflow will be reduced, because the future upstream conduits can be designed for flow control at the filtration facility. Therefore, the basin will not need to be enlarged.*

d. *Safety factor of 2 for emergency overflow.*

4.10 Planning Considerations Summary

This chapter summarized planning considerations for the filtration facility that informed development of a conceptual site plan and evaluations performed as part of the project definition efforts presented in subsequent chapters of this report, especially Chapter 7: Filtration Facility Support Systems.

Key considerations for project definition discussed in this chapter include:

- **Permitting.** Close coordination between the permitting team, permitting agencies, designers, and community members is critical to support the permitting process and long-term project success.

- **Cultural Resources**. Although no archaeological artifacts were found in the field surveys, ground-disturbing activities on the site should be conducted under the terms of an inadvertent discovery protocol and/or archaeological monitoring if buried archaeological features are encountered.
- **Environmental Assessment.** Based on the Phase II ESA results, excavated materials from the near-surface layer (upper 12 inches) of the site are suitable for onsite reuse and are likely suitable for a clean fill determination and a DEQ permit exemption for offsite reuse. Excavated materials below the near-surface layer are suitable for a clean fill determination.
- **Traffic**. The analysis of post-construction traffic from the filtration facility indicates that the study intersections and potential site accesses will operate at an acceptable level of service meeting both Multnomah County and Clackamas County standards and do not require traffic improvements. Additional analysis of construction traffic impacts will be conducted during the design phase.
- **Acoustics.** Noise mitigation will likely be needed to meet codified sound level limits.
- **Geotechnical.** The steep slope along the northeast edge of the facility site is likely vulnerable to seismically induced local slides and deformation. This can be mitigated by using a setback from the northeastern slope for critical facilities. For purposes of conceptual site layouts, the setback is 200 feet. Seismic-induced settlement under structures and utilities will likely need to be mitigated by pile supports or ground improvements. The exact extent of the recommended setback and potential ground improvements will be further evaluated in design.
- **Hydraulics**. Setting the facility elevation to allow full-time gravity flow is preferred over the higher elevation that would require pumping.
- **Ultimate Capacity**. The maximum capacity of the filtration facility is assumed for project definition to be 160 mgd initial and 240 mgd ultimate.
- **Overflow Management**. The need for onsite overflow management and initial sizing of overflow basins is established at 16 MG (NW Basin) and 3 MG (SW Basin) for the 160 mgd peak day design condition.

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Chapter 5
Design Considerations

This chapter identifies general design considerations that will be used to frame project definition for treatment processes and facility support systems discussed in subsequent chapters. These considerations include both required and elective design guidance and are presented as conservative suggestions based on industry guidelines, best practices, engineering judgment, and known constraints. This chapter presents an overview of design considerations, including:

- 5.1 System Resilience
- 5.2 Process Layout Efficiency for Potential Expansion
- 5.3 Chemical Safety and Beneficial Solids Reuse
- 5.4 Sustainability Goals
- 5.5 Design Regulations and Guidance
- 5.6 Pilot Plant Study
- 5.7 Good Neighbor Agreement
- 5.8 Design Considerations Summary

In this chapter, treatment processes will be introduced and briefly described to orient the reader to the importance of general design considerations. A more complete description of specific treatment processes is included in Chapter 6: Treatment Process Alternatives. Many of the suggested design considerations have been incorporated as assumptions for project definition and initial cost estimate development and will be further refined and evaluated to assess feasibility during project design.

5.1 System Resilience

Features that provide for system resilience in the form of operational reliability, and flexibility for key water treatment unit processes are a vital aspect of filtration facility design. This design consideration is essential for providing operators with necessary tools to optimize facility operations under different conditions, including periods of rapidly changing raw water characteristics, facility upsets, or unexpected changes in water demand. Ultimately, the driver for this set of design considerations is to help maximize public health protection and enhance finished water quality aesthetics while meeting water demands. This section briefly describes the approach used to integrate system resilience into the filtration facility design.

This section includes discussion of the following topics:

- Seismic Resilience
- Unit Process Selection and Sizing
- Yard Piping and Conveyance Channels
- Pipeline Reliability
- Ease and Safety of Facility Operations and Maintenance (O&M)
- System Resilience Summary

5.1.1 Seismic Resilience

The filtration facility and new pipelines will be built to modern seismic standards to withstand an emergency such as a major earthquake. The new pipelines will also replace nearby aging pipelines that do not meet current seismic standards and are at risk of failure due to poor condition. Since the new infrastructure associated with the filtration facility will be seismically resilient, an earthquake is more directly a risk to existing infrastructure and subsequent access limitations, with secondary water quality impacts.

In 2013, the State of Oregon developed the *Oregon Resilience Plan* to prepare for the magnitude 9.0 Cascadia earthquake event, with a goal to provide water to 80 to 90 percent of the water system after a 9.0 earthquake (Oregon Seismic Safety Policy Advisory Commission, 2013). Re-establishing water service is a crucial element in the overall recovery of communities after a major earthquake and is considered a top priority by the Oregon Seismic Safety Policy Advisory Commission, which set goals, referred to as target states of recovery, for the time required to achieve different levels of service for the water system. The target states of recovery identified in the *Oregon Resilience Plan* require a high degree of reliability from the backbone water system (i.e., key supply, treatment, and transmission elements). According to the *Oregon Resilience Plan*, the backbone should be 80 to 90 percent operational within 24 hours following the earthquake.

For project definition, it is assumed the filtration facility will implement recommendations in the *Oregon Resilience Plan*, including recovery standards applicable for structural, electrical, mechanical, and civil components where possible.

5.1.2 Unit Process Selection and Sizing

The core design philosophy for sound filtration facilities starts with proper selection and sizing of key unit treatment processes, followed by robustness of the design to accommodate both normal and potential future operational challenges. Based on general filtration facility design principles and engineering judgement, it is suggested to consider designing slightly oversized or parallel critical piping and process systems to provide additional flexibility and resilience for unknown circumstances for little added cost.

It is also suggested to select treatment processes, process mechanical equipment, and auxiliary support systems that have been successful in similar applications. A project of this significance should rely on technologies and systems that have been sufficiently proven. Note this consideration does not limit use of new technologies or innovation but merely systems with a limited record of success.

For project definition and initial cost estimate development, each key potential unit process is assumed to consist of at least two major parallel units to provide system reliability and flexibility. This includes:

- Pre-oxidation
- Rapid mixing
- Flocculation
- Sedimentation
- Filtration
- Disinfection/clearwell storage
- Residual solids handling
- Recycle water systems

This concept extends to specific process mechanical equipment (e.g., pumps, blowers, and mixers) and associated support systems.

In the case of basins and key conveyance channels and pipelines, it is suggested to include a sufficient number of parallel units sized for peak day flow conditions, with the ability to take units out of service during periods of reduced demand or when extraordinary conditions warrant. Based on general facility design guidance, and for project definition and initial cost estimate development, it is suggested that the filtration facility configuration enable half the facility (or discrete portions or specific unit processes), including major piping and conveyance channels, to be taken out of service for maintenance or other reasons. This would typically occur during low flow demand and/or modest raw water quality periods (e.g., winter season when turbidities are not excessively high), while continuing to produce high-quality finished water at reduced capacity. This approach could also facilitate filtration facility optimization trials with two parallel treatment trains running together to evaluate different operational modes. This dual facility (bifurcation) concept is suggested for further evaluation during design.

An example of the bifurcation approach relates to filters. The total required filtration area (filter area-square feet [sf]) is dependent on the design peak day flow rate, coupled with the maximum unit filtration rate (gallons per minute [gpm] per sf). The number of actual filter units is based upon limitations of filter dimensions and support systems such as underdrains and backwashing system. Thus, the total number of filters must consider the number of filters that would be out of service during backwashing or other outage scenarios. Ultimately, the number of filters can range from an impractical, but theoretical, minimum of two filters to a large array of 20 or more filters. The number and size of filters will be a value somewhere in between that balances cost and complexity (especially as filters are taken out of service routinely) with production demands, water quality, system reliability, and process flexibility to handle expected and unexpected operating scenarios.

In the case of process mechanical equipment, a complete standby unit is suggested for each system, as the probability of failure of process mechanical equipment is higher than that of more passive items such as concrete basins or static sedimentation settling plates. A similar concept is suggested with electrical power supply, where the use of a standby power system to support operations of key processes is to be provided. The specifics of these approaches are further discussed in Chapter 7: Filtration Facility Support Systems. This concept of a duty/standby configuration for supporting mechanical and electrical systems has been employed for project definition based on general filtration facility design guidance.

5.1.3 Yard Piping and Conveyance Channels

As with the treatment process-related design considerations described above, it is suggested that critical yard piping and conveyance channels be designed with an attention to system resilience. The basic unit processes need to be accompanied by equally robust and sound pipelines and channels. Channel division is the concept of splitting channels so that maintenance can occur on one channel while facility flow continues through the other channel. This capability is often lacking in the design of older water treatment facilities, with maintenance or repair for certain important systems only possible during facility-wide outages. It is suggested that during design, critical pipelines and channels be evaluated to be slightly oversized when possible and include at least two separate pathways for reliability and to allow for inspection and repairs during planned outages. It is also suggested that design includes provisions to enable maintenance entry into large-diameter pipes and other critical areas of the filtration facility where inspections and repair work must be conducted periodically.

Additional considerations for the designer to evaluate during sizing and design of these pipelines and channels, include the following:

- Pipe materials
- Minimum and maximum flow velocities
- Convenient isolation valves and gates
- Properly installed flow meters
- Pipe venting
- Surge control, where applicable
- Drainage
- Flushing systems

For project definition and initial project cost estimation, dual or looped pipelines with segment isolation valves are assumed for critical functions. Additionally, major conveyance channels are divided with appropriate motorized or manual fixed isolation slide gates (Figure 5-1), crane-actuated removable bulkheads (Figure 5-2), or equivalent segmented stoplogs, used for less frequent applications.

Figure 5-1. Typical fixed isolation slide gate

Figure 5-2. Example removable bulkhead

Specific examples of system resilience include a dual pipe or conduit system for rapid mixing, dual or looped feed for backwash water entering the filter gallery (the facility could be shut down should a single backwash conduit fail), and dual unit process feed and collection channels. If implemented, these approaches will permit maintenance of structures when portions of the system are taken out of service during low flow periods.

The yard piping and conveyance channel features described in this section are based on general filtration facility design guidance and have been assumed for project definition and initial cost estimate development.

5.1.4 Pipeline Reliability

The Filtration project includes design and construction of new raw water pipelines to connect the existing conduits to the filtration facility and new finished water pipelines to convey treated water from the filtration facility to the existing conduits. To address reliability and maintenance concerns, PWB recommends building two raw water pipelines and two finished water pipelines.

PWB currently has three conduits that provide supply to the system. Conduit 2 (44- to 52-inch diameter), Conduit 3 (50- to 58-inch diameter), and Conduit 4 (56- to 66-inch diameter) were constructed in 1911, 1925, and 1953, respectively. Operating with multiple pipelines has allowed PWB to take one or two lines out of service to perform both routine and major maintenance. Multiple pipelines also provide system reliability during unforeseen events such as a landslide or pipeline failure. PWB prefers to maintain the flexibility of multiple pipelines with the new filtration facility system.

For project definition and development of preferred raw water pipeline and finished water pipeline alignments, it is assumed that PWB will proceed with construction of dual pipelines.

5.1.5 Ease and Safety of Facility O&M

A significant design consideration is increasing reliability by accommodating future facility O&M activities. This is a broad topic that will be further evaluated and incorporated using Reliability Centered Design principles during design. The basic project definition philosophies are described in this section.

This topic relates to how easily each system can be operated and accessed for maintenance. Complexity does not necessarily improve ease of operations and can occasionally do the opposite. Systems should be simple to understand, have numerous “work-arounds” in case of unexpected events or failures, be easy to adjust, and provide means to monitor behavior and performance. For example, simple vertical flocculation mixers with variable speed drives (as identified in Chapter 6) provide added flexibility to operators and allow visual access into the flocculation basins to gauge floc development. The same applies to maintenance access for equipment, including the ability to take systems out of service.

Floor space needs to be adequate to slide out and remove equipment components. Buildings must include adequate means and egress to lift and remove heavy equipment items for major maintenance and may include fixed or permanent hoisting devices (i.e., bridge cranes, monorails, or crane davits). In addition, design features to promote equipment accessibility and improve overall operator circulation, including interconnecting galleries and tunnels housing process piping, electrical conduits, and other support utilities, will be beneficial for facility O&M. This is especially true if critical passageways are designed to permit forklift entry as needed for equipment and piping system disassembly.

Other significant design concepts to improve ease of operations could include locating the Administration Building and, more importantly, the Control Center to provide a commanding view of key unit processes, while also being adjacent to facility access and circulation points such as aboveground walkways and catwalks and belowground utility corridors, galleries, or tunnel systems. Figure 5-3 illustrates a typical filtration facility gallery and tunnel system.

Figure 5-3. Everett, Washington, water treatment facility filter gallery

It is also suggested the design consider safety features for the protection of staff, visitors, and neighbors. Potential hazards range from chemical contact to slips, trips, and falls; electrical shocks; and vehicle accidents. There are many governing safety codes applicable to the filtration facility along with general design practices to promote overall system safety that are suggested for consideration by the designer. Some of these codes are as follows:

- International Building Code
- International Fire Code
- National Electrical Code
- International Plumbing Code
- Life Safety Code
- National Fire Protection Association Codes
- Occupational Safety and Health Act
- Uniform Mechanical Code

5.1.6 System Resilience Summary

This section described design considerations suggested to enhance overall system resilience. These suggestions have been adopted for project definition and initial cost development and are based on general filtration facility design guidance.

Providing system resilience against significant risks is of critical importance to PWB and its customers and a priority value. As such, PWB will continue to assess costs and benefits of investments to increase system resilience during design of the filtration facility.

- **Seismic Resilience.** For project definition, it is assumed the filtration facility will implement recommendations in the *Oregon Resilience Plan*, including recovery standards applicable for structural, electrical, mechanical, and civil components where possible.
- **Unit Process Selection and Sizing.** For project definition and initial cost estimate development, key unit processes are assumed to consist of at least two major parallel units to provide system reliability and flexibility. It is suggested that a bifurcation approach to allow part of the facility to be taken offline without disrupting service be further evaluated during design.
- **Yard Piping and Conveyance Channels.** It is suggested that provisions for maintenance of critical pipelines and channels, such as access and slight oversizing of critical piping, be further evaluated during design to increase system reliability and to allow for inspection and repairs during planned outages.
- **Pipeline Reliability.** For project definition and development of preferred raw water pipeline and finished water pipeline alignments, it is assumed that PWB will proceed with construction of dual pipelines.
- **Ease and Safety of Facility Operations.** A significant design consideration is increasing reliability by accommodating future facility O&M activities in the design. This topic will be further evaluated and incorporated using Reliability Centered Design principles during design.

5.2 Process Layout Efficiency for Potential Expansion

One measure of an efficient process layout is how well it promotes centralized operations, minimizes excessive facility head loss, provides easy staff circulation pathways, inspection, and maintenance access, and reduces overall cost. The suggested layouts depicted in this report are based on these core principles. Note that the layouts depicted here are design suggestions and form the basis of the initial project cost estimate. The designer will provide their own layout concepts that will also need to address these industry best practices.

Another key aspect of an efficient site layout is anticipating and accommodating for orderly and minimally disruptive potential future facility expansions. Scenarios where this attention to detail in the design effort is ignored or minimized can lead to haphazard additions of non-symmetrical unit process elements (e.g., additions of odd-sized treatment basins that complicate uniformity in unit process water quality and hydraulic distribution). Moreover, a poorly laid-out facility may require costly and disruptive construction for expansion retrofits. This important design suggestion, affecting both expansion of unit process elements and major piping and conveyance channels, is to be carefully considered in design.

Chapter 4: Planning Considerations, identifies the filtration facility capacity assumptions for project definition purposes as a design capacity of 160 million gallons per day (mgd), expandable to 240 mgd. This section describes how this initial and future capacity can be efficiently accommodated in the project design.

This section includes discussion of the following topics:

- Unit Process Expansion to Ultimate Capacity
- Sizing Site Piping and Conveyance for Ultimate Flows
- Process Layout Efficiency for Potential Expansion Summary

5.2.1 Unit Process Expansion to Ultimate Capacity

Mitigating the impacts of future expansion-related disturbance requires understanding site space and future construction constraints, nearby utilities, and to a degree, facility symmetry. This is especially true if parallel unit processes are to be maintained after facility expansion. This means that the initial design and construction must plan for future expansion. Ways to facilitate future expansion could include knockout walls or extension dowel placement in the initial concrete pours, added pipe rack or electrical cable tray space for future utilities, advance preparation of underground soils, and other sensible construction practices.

For example, if four parallel flocculation basins are selected for the initial 160 mgd design, adding two more basins will provide the necessary 240 mgd ultimate capacity. Figure 5-4 below illustrates two potential geometrical options to achieve this expansion. Selection of the preferred expansion geometry will depend on final facility layout, the space set aside for expansion, and potential disruption to existing operation. Note that future expansion can be subdivided in stages, if the desired next incremental step is less than a 50 percent facility expansion. In the preceding example, the addition of only one flocculation basin theoretically increases facility peak day capacity by 25 percent to 200 mgd.

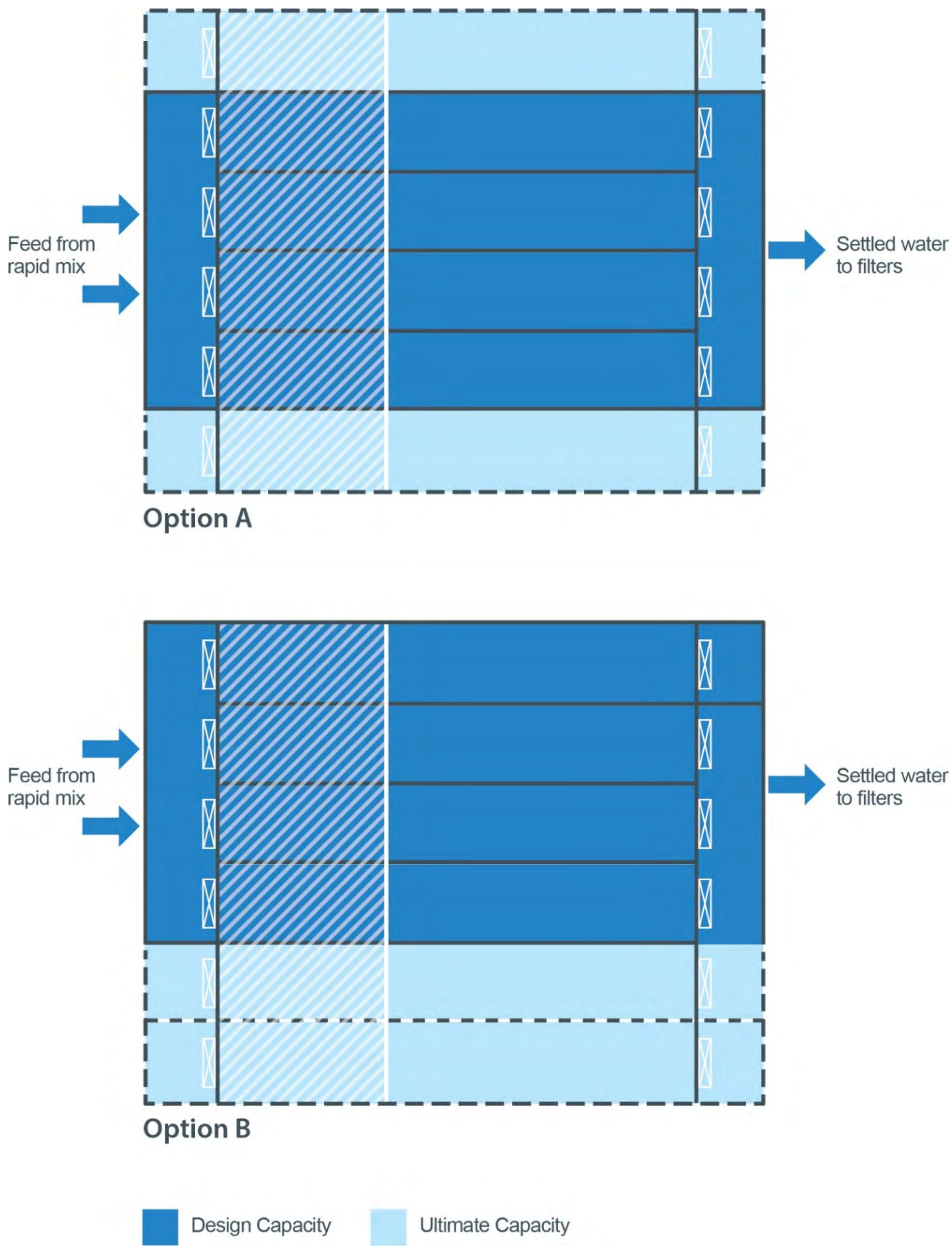


Figure 5-4. Two conceptual options to achieve a 50 percent increase in flow for future flocculation sedimentation basin expansion

Evaluating the impacts of future filtration facility expansion during the initial design is a valuable concept to apply to each of the major unit processes and key support systems (e.g., standby engine generators). Note that parts of the facility may not need expansion as capacity increases in the future. This often applies to fixed assets such as the administration and maintenance buildings, or other systems where incremental expansion provides limited value. It is important that the design of unit processes be compartmentalized into a sufficient number of parallel units to promote flexibility and sensible expansion but not at the expense of unnecessary layout complexity or cost.

5.2.2 Sizing Site Piping and Conveyance for Ultimate Flow

With respect to sizing of critical pipelines, many will be located within the central part of the filtration facility and will not be easily expandable due to their location. Therefore, it is suggested that critical and large-diameter piping be designed to accommodate the range of initial design capacity and ultimate capacity flow scenarios. The size and selection of these pipelines must take into consideration the range of flow velocities, hydraulic head losses, and the presence of settleable solids in certain conduits. It is undesirable to oversize pipelines that must bear settleable solids that can clog systems during low flow periods.

For these reasons and for project definition, open channels or major piping systems feeding, collecting, and bypassing specific unit processes are suggested to be designed to accommodate flows necessary to produce 240 mgd of finished water during peak day conditions. These systems must also consider the effects of filtration facility recycling and the necessary boost in unit process hydraulic loading as a result. To produce a net finished water flow of 240 mgd to the distribution system, it is necessary to convey more than 240 mgd of raw water through the treatment process to account for water that is “lost” or recycled due to backwashing filters, removing solids from the bottom of the sedimentation basins, or operating filters in a filter-to-waste mode after each backwash cycle. The amount of recycled water can vary as a percentage of net finished water production, with typical values ranging from 2 to a recommended maximum of 10 percent per the Filter Backwash and Recycle Rule. Backwash water piping systems may be designed to backwash only one filter at a time (as is assumed for project definition), regardless of future filters added to accommodate the ultimate expansion.

5.2.3 Process Layout Efficiency for Potential Expansion Summary

This section described design considerations to plan for potential future facility expansion. Key considerations for project definition include:

- **Unit Process Expansion.** It is important that the design of unit process facilities incorporates flexibility for future expansion, but not at the expense of unnecessary layout complexity or cost.
- **Sizing Site Piping and Conveyance for Potential Expansion.** Open channels or major piping systems feeding, collecting, and bypassing specific unit processes are suggested to be designed for easy future expansion and to accommodate flows necessary to produce the ultimate design capacity of finished water during peak day conditions.

5.3 Chemical Safety and Beneficial Solids Reuse

This section addresses design considerations related to chemical safety and beneficial solids reuse and briefly describes the approach used to integrate these considerations into the filtration facility design. These considerations align with project values such as maximizing public health and water quality with minimal treatment chemicals and environmental impacts. As part of the design process, PWB will be evaluating inherently safer technologies that minimize risk to the public and facility staff and assessing opportunities for beneficial reuse.

This section includes discussion of the following topics:

- Chemical Safety
- Beneficial Solids Reuse
- Chemical Safety and Beneficial Solids Reuse Summary

5.3.1 Chemical Safety

A measure of sustainability is to manage the types and quantities of chemicals used to treat Bull Run water. It is important to select and use chemicals in quantities that are effective to help meet finished water quality goals, safe for operators to use, and meet the requirements of the NSF 60 and other appropriate quality regulations and guidance. It is also important to know the fate of these chemicals, their potential impact on community safety (chemical transport and usage), and their potential impact on the environment. The types of water treatment chemicals assumed for project definition are listed in Table 5-1. These listed chemicals are commonly used in water treatment for their effectiveness, safety, and cost.

Table 5-1. Assumed Water Filtration Treatment Chemicals

Chemical	Purpose	Form
Sodium hypochlorite (bleach)	Used for disinfection, pre-oxidation	Liquid
Oxygen	Used to form ozone	Cryogenic liquid
Ozone[a]	Used for oxidation	Gas formed on site
Alum, polyaluminum chloride, aluminum chlorohydrate, and ferric chloride	Prime coagulant	Liquid
Coagulant aid polymer	Assists with coagulation	Polymer solution or emulsion
Powdered activated carbon[b]	Provides adsorption	Powder
Filter aid polymer	Assists with filtration	Polymer emulsion or dry powder
Calcium thiosulfate (quenching agent)	Quenches residual ozone	Liquid
Ammonium hydroxide (ammonia water)	Chloramination	Liquid
Soda ash	Corrosion control	Powder
Carbon dioxide	Corrosion control	Cryogenic liquid
Thickening aid polymer	Assists with thickening for solids handling only	Polymer emulsion or dry powder
Dewatering aid polymer	Assists with dewatering for solids handling only	Polymer emulsion or dry powder

a. Potassium permanganate may be considered as an alternative oxidant.

b. May be used on occasion.

The specific chemicals that work best with the Bull Run supply are being identified through the pilot plant study. These chemicals will be optimized to reduce operating costs while promoting effective water treatment. In addition, it is assumed for overall safety reasons that gaseous chlorine will not be used at the filtration facility and sodium hypochlorite will be used instead. During design, it is suggested that the use of onsite generated sodium hypochlorite and oxygen be considered from an environmental and sustainability perspective to mitigate chemical manufacturing and transport impacts. Note that the use of onsite generated sodium hypochlorite still requires delivery of salt and use of onsite electricity to generate the sodium hypochlorite solution. Similar investigations related to overall sustainability of other chemicals are suggested to be conducted during design (e.g., use of onsite generated oxygen in lieu of transporting LOX for ozone production). Chemicals will be obtained from reputable sources and suppliers in compliance with NSF-60 certification.

Safety of both the community and facility operators is a top priority. The filtration facility will meet rigorous state and federal safety standards for chemical storage and transportation that are designed to protect workers and the public. For example, chemicals stored on site will meet regulatory requirements for secondary containment to capture any chemical if a tank leaks. PWB will develop risk management and emergency action plans for the filtration facility and will follow industry best practices for preparing for potential emergency situations.

5.3.2 Beneficial Solids Reuse

Residual solids from water treatment are comprised mostly of inert clays and silts. The residual solids produced at the filtration facility will be processed and dewatered to a state of dryness suitable for economical trucking off site. For project definition, it is assumed residual solids will be transported to municipal landfills to be used for daily cover, as practiced at the Joint Water Commission and Seattle Tolt water treatment facilities. For planning purposes, it is anticipated that options for reuse of filtration facility residual solids will be limited—residual solids from water treatment possess few of the biological nutrients found in municipal wastewater treatment solids and are thus not as well suited for many agricultural uses. However, with direct operating experience and improved understanding of residual solids characteristics, options for beneficial reuse can be further investigated and may include the following:

- Use as an amendment for industrial processes, including:
  - Concrete manufacturing or brick making (practiced at the Vancouver, British Columbia, water treatment facility).
  - Topsoil blending.
- Use as an agricultural or forest land soil amendment where addition of clay and silt is desirable, including:
  - Forest land application (practiced at the Everett, Washington, water treatment facility),
  - Local agricultural application with lime added to increase alkalinity (practiced at the Asheville, North Carolina, water treatment facility).

- Co-blend with municipal wastewater treatment biosolids, if practical.
- Direct discharge of liquid sludge or solids streams (without thickening and dewatering) to a wastewater treatment plant (practiced at the Bellingham, Washington, and Lake Oswego, Oregon, water treatment facilities but likely impractical here as nearby sewer connections are not available).

Typically, recipients of residual solids test the composition and moisture content to determine how they can optimize use of solids in their own processes and may refuse solids that do not meet their own quality control standards. Thus, it is important to produce a steady stream of reasonably consistent and dry residual solids that can be economically trucked from the site for either landfill disposal or other subsequent beneficial reuse options (either in part or in whole).

5.3.3 Chemical Safety and Beneficial Reuse Summary

This section described consideration of design options to enhance chemical safety and beneficial reuse that are suggested for further evaluation during design. Decisions on specific water treatment chemicals and residuals solids handling strategies will be made during design.

Key considerations for project definition include:

- **Chemical Safety:**
  - For project definition, use of the chemicals listed in this section for water treatment are assumed.
  - It is assumed for overall safety reasons that gaseous chlorine will not be used at the filtration facility and sodium hypochlorite will be used instead.
  - During design, it is suggested that the use of onsite generated sodium hypochlorite and oxygen be considered from an environmental and sustainability perspective to mitigate chemical manufacturing and transport impacts.
- **Beneficial Solids Reuse.** It is assumed for project definition that residual solids from the water treatment process will be dewatered and taken to a landfill for use as daily cover.

5.4 Sustainability Goals

Sustainability is both a requirement and a stated desire of PWB for this project. Portland's Green Building Policy mandates sustainability requirements for any new City of Portland-owned building, including both occupied and unoccupied structures. One of the goals strongly expressed during the discussion of sustainability with the program team was to achieve energy efficiency through high-quality design rather than by buying carbon offsets. The Energy Trust of Oregon (ETO) provides useful guidance for project sustainability considerations with an emphasis on energy-efficient design. In addition, it provides financial incentives that are matched to various measures pursued in the design of a project. The ETO program can be used in concert with the Portland Green Building Policy to achieve sustainability goals.

Both the ETO and the Portland Green Building Policy are discussed in detail below. Another program under consideration is the Envision framework by the Institute for Sustainable Infrastructure. This program could potentially be used in concert with the standard sustainable building programs mandated by Portland's Green Building Policy. It would also work well in combination with the ETO program. As with any sustainable building program, early identification of program goals will be essential to success.

This section includes discussion of the following topics:

- Portland Green Building Policy
- Energy Trust of Oregon
- Sustainability Goals Summary

5.4.1 Portland Green Building Policy

Portland's Green Building Requirements stipulate that all new, City-owned buildings will incorporate sustainable features. The City has a comprehensive guideline for sustainable measures to be considered and specific minimum baselines. These items should be an integral part of the design process as they impact planning and cost considerations. These requirements are specifically laid out in the City of Portland *Green Building Policy*. It is suggested that a member or members of the design team be tasked to coordinate and track the implementation of these requirements. Some specific requirements are listed below along with some potential project-specific applications:

- Register and certify for one of the following sustainable building programs:
  - United States Green Building Council Leadership in Energy and Environmental Design (LEED) Building Design and Construction at the Gold level.
  - Earth Advantage Commercial certification at the Gold level.
  - Design, build, and operate to achieve Living Building Challenge status.
- Achieve 15 percent energy savings beyond Oregon Energy Efficiency Specialty Code.
- Incorporate onsite renewable energy systems and meet Oregon's 1.5 percent for Green Technology requirement.
- Earn or meet LEED's energy commissioning credit requirements.

- Use native or non-invasive drought-tolerant plants and do not use potable water for routine and ongoing irrigation. Raw water at the filtration facility could be a potential alternative.
- Select WaterSense-labeled products to reduce total potable water use.
- Cover roofs of all buildings with ecoroofs (exemptions to this requirement can be approved on a case-by-case basis; the site has land available for stormwater infiltration, which could be a factor in favor of an exemption).
- Incorporate Salmon Safe certification recommendations during construction and after project completion.
- Incorporate measures to reduce bird strikes, including treatment of glass and lighting design. Any such measures would not increase the amount of exterior lighting.
- Provide no more than the minimum car parking required by code. Additional parking shall be limited to the minimum shown in a parking demand analysis approved by the Bureau of Transportation. City fleet vehicle parking is exempt from this requirement. As the site is in a remote rural location some negotiation of these requirements based on the needs of facility operations should be possible.
- Provide covered and secure bicycle parking for employees and visitors. As a remote rural location the amount of bicycle parking provided at the facility would be subject to negotiation based on the anticipated usage.
- Pre-wire charging stations for City-owned electric vehicles.
- Follow construction waste prevention guidelines to meet the City's 85 percent waste diversion goal.
- Follow space allocation standards and space planning guidelines outlined in the *Green Building Policy*.

5.4.2 Energy Trust of Oregon

The ETO is committed to providing comprehensive, sustainable energy efficiency and renewable energy solutions to customers in southwest Washington and Oregon. ETO prioritizes energy efficiency and clean energy by providing incentives for the installation of energy-efficient equipment and energy savings measures. For the filtration facility, these can be divided between industrial, new building, and small office energy reduction measures. The industrial measures would likely be the most impactful. The small office incentives would primarily apply to the Administration Building.

For the new building and industrial incentives, the ETO has a program that is specific to water and wastewater treatment facilities. The program includes overall building design measures as well as specification of energy-efficient equipment. Selected measures should be evaluated to make sure they do not affect process reliability. Overall cost of measures and corresponding ETO incentives will depend on the what measures are chosen. Some of the measures and energy-efficient equipment that may be appropriate for this project include:

- Added wall, roof, attic insulation
- Hot water pipe insulation
- Compressed air

- Motors and drives
- Welding equipment
- Custom water and wastewater incentives
- Strategic Energy Management (a holistic approach to reducing energy use)
- Custom measures

Other new building incentives that ETO assists customers with include:

- Energy Star
- Solar ready design and construction
- Solar development assistance
- Whole building approaches such as energy modeling, United States Green Building Council's Leadership in Energy and Environmental Design Workbook, net zero
- Full building commissioning

To be eligible for small office incentives, per ETO's *Small Office Incentive Workbook*, there are required base measures and optional elective measures. Once installed, incentives are calculated based on a per-sf basis, the amount of energy saved, or a bonus percentage, and will depend on the equipment selected and the number of elective options installed. Optional measures must have a minimum of a 1-year payback and must pass the program's cost-effectiveness test.

The workbook's measures and associated incentives are grouped as either required or optional. Table 5-2 presents the base incentive amount for each tier, as well as optional elective measures and their additional incentives, if installed.

Table 5-2. ETO Required Measures

Tier	Description	Incentive
Required Base Measures		
Good	• 15% internal lighting power density (LPD) reduction • Code required automatic lighting controls • Advanced power strips at all workstations	$0.20/sf
Better	• 25% LPD reduction • Automatic lighting controls in all spaces • Advanced power strips at all workstations	$0.30/sf
Best	• 45% LPD reduction • Automatic lighting controls in all spaces • Advanced power strips at all workstations	$0.40/sf

Table 5-2. ETO Required Measures		
Tier	**Description**	**Incentive**
Optional Elective Measures		
Prescriptive	• Power management software on desktops • Server closer mini-split A/C • Economizer on 3-, 3.5-, or 4-ton A/C or heat pump unit • Condensing boiler • Additional standard measures (ETO Form 5205)	Standard incentive +15% bonus
Calculated	• 50% LPD reduction • 60% LPD reduction • Variable frequency drive on supply fans (Units <110,000 Btu/hour) • Uninterruptable power supplies • Special measures (energy efficiency measures that require a custom calculation to demonstrate potential energy savings)	\$0.03/sf \$0.07/sf \$100/ton \$0.25/kW or \$100/therm

Given the scale of this project and its multiple building uses, it is advised that the designer and PWB meet with ETO early in design. By doing so, the designer can identify likely measures to use and further define specific incentive amounts. This helps assure that energy reduction measures are incorporated into the design from the outset. The early design assistance meeting qualifies for incentive funding; however, it must take place during preliminary design, before final design is complete.

5.4.3 Sustainability Goals Summary

This section described sustainability policies, goals, and certifications that may be relevant to the filtration facility design. The filtration facility will be built in accordance with Portland's *Green Building Policy*, which outlines requirements for resource efficient design, construction, and operational practices for City-owned facilities and publicly funded development. Along with meeting the commitment to green building practices outlined in the City's policy, PWB is also continuing to assess the applicability of additional sustainability guidance and practices, such as the measures identified by ETO.

Key considerations for project definition include:

- **Portland Green Building Policy.** The policy stipulates that all new, City-owned buildings will incorporate sustainable features and outlines specific minimum baselines that will need to be considered during design. It is suggested that the Envision framework be considered early in design for potential use in concert with the Green Building Policy.
- **ETO.** ETO has a program that provides incentives for energy reduction measures. A design assistance meeting will need to be held during preliminary design of the filtration facility to quality for incentive funding.

5.5 Design Regulations and Guidance

In addition to general design considerations identified in this chapter, there are a set of more thoroughly prescribed regulatory and guidance documents that will significantly affect the design of the filtration facility. Some of these documents are regulations-based and others are suggested guidance. This section briefly discusses the nature and applicability of these forms of design regulation and guidance, including:

- Oregon Health Authority (OHA) Regulations (regulations-based)
- Oregon Area Wide Optimization Program (AWOP) Guidance
- Partnership for Safe Water (PSW) Guidance
- Building Codes (regulations-based)
- Specific Design Guidance Summary

The water quality-based regulations and guidance discussed in Chapter 3: Water Quality Considerations, will drive the design by emphasizing finished water quality and facility performance standards.

5.5.1 Oregon Health Authority Regulations

OHA is the responsible state agency for related facility design criteria, with Drinking Water Services (DWS) in charge of enforcement. An initial discussion of the OHA requirements codified in Oregon Administrative Rules (OAR) 333-061 is presented in Chapter 3 primarily as it relates to water quality issues. This legislative citation describes obligatory aspects of filtration facility design that are to be followed for this project. Specific information on construction standards is provided in OAR 333-061-0050.

5.5.2 Oregon Area Wide Optimization Program Guidance

Oregon AWOP is a non-regulatory approach to enhance facility performance with corresponding design emphasis, especially with respect to pathogens. Chapter 3 describes the nature and importance of the Oregon AWOP and its guidance related to water quality goals and facility performance. With respect to treatment and facility design criteria, the Oregon AWOP webpage offers guidance for key unit processes, including flocculation, sedimentation, filtration, and disinfection. These goals and criteria are being used for project definition, and it is expected that the subsequent design of the filtration facility will follow suit. The pilot plant study is also following these suggested protocols and goals. PWB will further evaluate formal participation in the Oregon AWOP as the project progresses.

5.5.3 Partnership for Safe Water Guidance

PSW provides guidance and specific goals related to water quality, facility design, facility optimization, and management procedures and practices. PSW's volunteer program is further described in Chapter 3 and is in part, sponsored by the United States Environmental Protection Agency and the American Water Works Association.

As with the Oregon AWOP, the water quality goals and design criteria used for project definition and the pilot plant study are influenced by PSW goals and guidance to ensure the highest finished water quality and public health protection. It is anticipated that the filtration facility design will also follow these guidelines. PWB will further evaluate formal participation in PSW as the project progresses.

5.5.4 Building Codes

The following list reflects the most significant current building codes that will be applicable to the filtration facility design. It is assumed that the design will be permitted by the City of Gresham Building Department as are other buildings in unincorporated Multnomah County located east of Gresham. Since new codes are typically adopted every 3 years, it is anticipated that changes to these building codes may occur by the time the building permit is submitted. Changes to the building code will need to be incorporated so that the design is up to code when submitted.

- **2019 Oregon Structural Specialty Code.** Based on the 2018 International Building Code.
- **2019 Oregon Zero Energy Ready Commercial Code.** Based on American Society of Heating Refrigerating and Air-Conditioning Engineers Standard 90.1-2016.
- **2019 Oregon Mechanical Specialty Code.** Based on the 2012 International Mechanical Code and the International Fuel Gas Code.
- **2020 Oregon Electrical Specialty Code.** Based on the 2020 National Electrical Code.
- **2020 Oregon Plumbing Specialty Code.** Based on the 2021 Uniform Plumbing Code.

Additionally, maintenance and other non-public areas at the filtration facility will need to follow state and federal Occupational Safety and Health Administration regulations.

5.5.5 Specific Design Guidance Summary

This section outlined applicable state and federal water quality regulations and local codes and ordinances that will guide design of the filtration facility. Given the 7-year timeline for project design and construction, some of these codes may undergo updates during the life of the project. Beyond meeting applicable codes and regulations, PWB is continuing to evaluate applicability of non-regulatory guidance such as the Oregon AWOP and PSW.

Key considerations for project definition include:

- **OHA Regulations.** OHA requirements relevant to the filtration facility design are codified in OAR 333-061.
- **Guidance.** Oregon AWOP and PSW goals and criteria are being used to guide project definition and the pilot plant study.
- **Building Codes.** It is assumed that the design will be permitted by the City of Gresham Building Department as are other buildings in unincorporated Multnomah County located east of Gresham.

5.6 Pilot Plant Study

The pilot plant study is part of project definition and will inform the treatment processes selected for the filtration facility. The study will provide useful information for unit process design criteria and to assess the performance of significant treatment processes that will affect water quality and cost. For example, the pilot plant study will assess the benefits of oxidation (e.g., pre-chlorination and pre-ozonation), sedimentation, unit filtration rates, and, possibly, corrosion control measures. As an ongoing activity, the study will also examine direct filtration in parallel with conventional filtration. Direct filtration is a process described in Chapter 6 consisting of rapid mix, flocculation, and filtration. Conventional filtration is a process similar to direct filtration; however, with sedimentation added immediately upstream of filtration. The pilot plant study will meet the requirement to provide preliminary planning and study results to OHA by November 30, 2020, as part of the overall compliance schedule.

The main objectives of the pilot plant study include:

- Support development of a sound, buildable, and operable basis of design that meets regulatory goals.
- Inform treatment process selection.
- Optimize operations to inform design parameters and seasonal operating parameters.
- Evaluate data for the Oregon AWOP and PSW.
- Support operator education by broadening treatment process understanding.

After the pilot plant study is complete, PWB will continue to operate the pilot plant as a training tool for operators and to further refine operating conditions or conduct research.

This section includes discussion of the following topics:

- Pilot Plant Work Plan
- Pilot Plant Study Summary

5.6.1 Pilot Plant Work Plan

The *Pilot Plant Work Plan* outlines pilot plant testing to validate proposed treatment approaches, inform design criteria, and assist the transition to full-scale operation (Appendix G). The work plan includes a study to evaluate specific unit processes with respect to removal of particulates and organics, unit process loading, treatment efficiency and productivity, and the treatment train's overall ability to comply with current regulatory requirements. Upon completion of the pilot plant study, it is anticipated that PWB will have a better understanding of how treatment processes will impact Bull Run water to help optimize treatment design.

The pilot plant study will potentially consist of two phases. The first phase is focused on evaluating specific treatment processes and technologies, such as pre-treatment (rapid mix, flocculation, sedimentation, and oxidation) and filtration, to optimize treatment of Bull Run water and will also study the effects of stress testing the pilot facilities with spiking studies related to elevated turbidity and parameters simulating extreme watershed events. The second phase may evaluate post-treatment disinfection and corrosion control strategies with a corrosion control study (once granular media filtration has been optimized in Phase 1). The pilot plant study will evaluate water quality over four seasons to test the full spectrum of raw water conditions expected from the Bull Run Watershed.

Phase 1 will be used to evaluate media types and unit filtration rates with conventional filtration operations and to understand the benefits of various oxidation agents. Pre-ozonation and intermediate ozonation will be tested for their ability to improve filtration performance and to evaluate how ozonation followed by biofiltration impacts the removal of natural color, organics, inorganics, and disinfection byproduct precursors. Other pre-oxidants, like chlorine, will also be evaluated for their ability to improve filtration performance and remove color and inorganics, such as manganese.

The following treatment processes will be tested in the pilot plant study using two equivalent and parallel process trains:

- Pre-treatment
  - Rapid mix
  - Flocculation (direct filtration)
  - Flocculation and sedimentation (conventional)
  - Ozonation (pre-ozonation and intermediate ozonation) and other pre-oxidants (chlorine)
- Granular media filtration (anthracite/sand and granular activated carbon [GAC]/sand media)

Figure 5-5 below shows the process flow diagram for the pilot plant study. Two parallel treatment trains with three filters each are used to evaluate filter media and upstream processes. Each treatment train is served by a flocculation-sedimentation system, ozone module, and filtration skid. Two flocculation and sedimentation units are used so direct filtration and conventional filtration can be compared simultaneously. The ozone unit is configured so that pre-ozonation and intermediate ozonation can be tested in either train. Six filters are used to test different media configurations.

The pilot plant includes testing the impacts of high-rate filtration (defined as filters with a maximum unit filtration rate exceeding 6 gpm/sf) on these processes. Facilities designed for a maximum unit filtration rate not exceeding 6 gpm/sf are generally exempt from mandatory pilot plant studies. Filtered water will not be routinely chlorinated during Phase 1 of this study; therefore, the filters will be allowed to operate as biologically active filters.

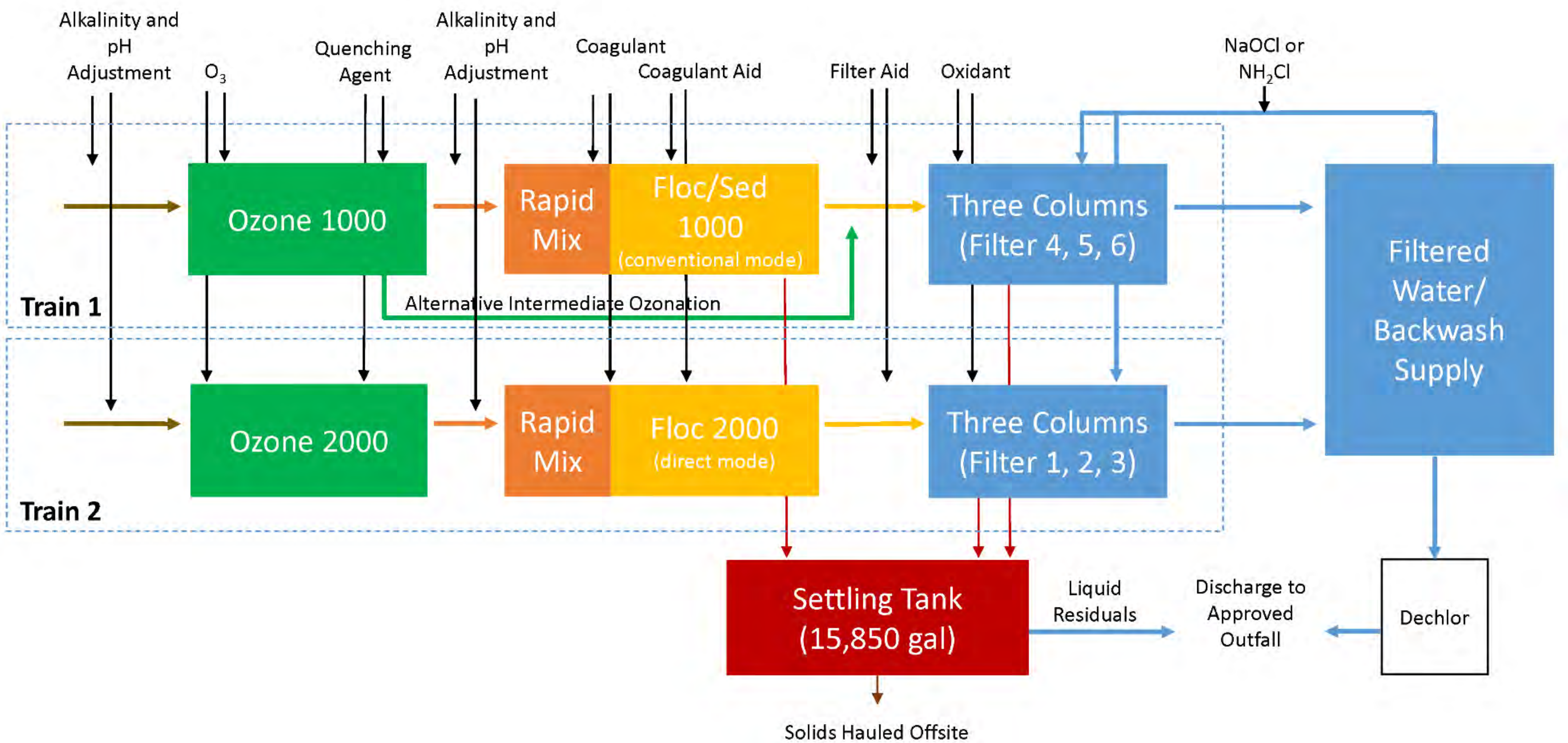


Figure 5-5. Flow diagram showing the pilot plant study dual treatment train

Table 5-3 shows the initial six-filter configuration. There are three filters with GAC/sand media and three filters with anthracite/sand media of different media depths and sizes. The results from the initial round of testing will be used to identify preferred filter media configurations. The preferred configurations will then be used for subsequent investigations.

Table 5-3. Filter Media Profile Overview

Media Type	Depth (inches)	Effective Size (mm)	Uniformity Coefficient	Specific Gravity	L/d ratio
Filters 1 and 6 – Anthracite, total media depth of 72 inches					
Anthracite	60	1.30	1.26	1.64	1,170
Sand	12	0.65	1.39	2.65	470
Total	72	—	—	—	1,640
Filter 2 – GAC, total media depth of 60 inches					
GAC, Calgon Filtrasorb 816	48	1.30	1.36	1.39	940
Sand	12	0.55	1.37	2.65	550
Total	60	—	—	—	1,500
Filters 3 and 4 – GAC, total media depth of 72 inches					
GAC, Calgon Filtrasorb 816	60	1.30	1.36	1.39	1,170
Sand	12	0.55	1.37	2.65	550
Total	72	—	—	—	1,730
Filter 5 – Anthracite, total media depth of 60 inches					
Anthracite	48	1.20	1.31	1.64	1,020
Sand	12	0.60	1.37	2.65	510
Total	60	—	—	—	1,530

5.6.2 Pilot Plant Study Summary

This section summarized elements of the pilot plant study work plan. The pilot plant study will investigate the seasonal water quality variations expected in the raw water and will also use limited duration spiking trials to simulate and evaluate potential future water quality conditions such as increased turbidity from storm events or a wildfire.

The pilot plant study is expected to inform the following design criteria:

- Alkalinity and pH adjustments ahead of coagulation as needed to optimize coagulant performance.
- Oxidant type, dose location, contact time, and dose.
- Coagulant and coagulant aid type and dose range, and addition points for coagulant aid.
- Sedimentation basin surface overflow rate.
- Filter aid type and dose range.
- Unit filtration rate.
- Filter media type and bed configuration.
- Primary disinfectant (chlorine with the possibility of credit for oxidation upstream of filtration) and secondary disinfectant (chloramine) dose, demand and decay behavior, a disinfection byproduct formation potential.

5.7 Good Neighbor Agreement

During design, PWB will continue working with a Site Advisory Group to develop a Good Neighbor Agreement for the filtration facility. The Site Advisory Group is comprised of adjacent property owners, farm operators, local school representatives, area water utilities, environmental organizations, and others. The group was formed to establish an ongoing opportunity for neighbors of the filtration facility to meet with the program team and to foster open communication by identifying and resolving community concerns early in the project.

The Good Neighbor Agreement will be an agreement reflecting PWB's commitments to filtration facility neighbors that addresses concerns while achieving a successful project. The agreement is intended to be used by the designer to ensure the filtration facility is well adapted to its rural setting. The Good Neighbor Agreement is anticipated to be completed early in 2021. The Site Advisory Group will meet monthly as the Filtration project progresses to help develop the agreement.

PWB is committed to keeping neighbors informed and involved throughout the project. The Good Neighbor Agreement will ensure that neighbors' concerns are considered through the design and construction processes and into ongoing operations. Outreach also includes mailers, an e-newsletter, doorhangers, information sessions, "backyard" visits, and other outreach events.

5.8 Design Considerations Summary

This chapter discussed design considerations for treatment processes and support facilities that reflect both required and voluntary guidance and goals. These design considerations are presented as conservative suggestions based on industry guidelines, best practices, engineering judgment, and known constraints. The design considerations suggested in this chapter reflect a commitment to the project values, including water quality and resilience, and form the foundation of project definition information described in later chapters of this report.

Key considerations for project definition include:

- **System Resilience:**
  - It is assumed the Filtration project will implement recommendations in the *Oregon Resilience Plan*, including recovery standards applicable for structural, electrical, mechanical, and civil components where possible.
  - Key unit processes are assumed to consist of at least two major parallel units to provide system reliability and flexibility.
  - It is assumed that PWB will proceed with construction of dual raw water and finished water pipelines.
- **Process Layout Efficiency for Potential Expansion.** Open channels or major piping systems feeding, collecting, and bypassing specific unit processes are suggested to be designed to accommodate flows necessary to produce the ultimate design capacity of finished water during peak day conditions.
- **Chemical Safety and Beneficial Solids Reuse:**
  - During design, it is suggested that the use of onsite generated sodium hypochlorite and oxygen be considered from an environmental and sustainability perspective to mitigate chemical manufacturing and transport impacts.
  - It is assumed for project definition that residual solids from the water treatment process will be dewatered and taken to a landfill for use as daily cover.
- **Sustainability Goals:**
  - The Portland *Green Building Policy* stipulates that all new, City-owned buildings will incorporate sustainable features and outlines specific minimum baselines that will need to be considered during design.
  - The ETO has a program that provides incentives for energy reduction measures. A design assistance meeting will need to be held during preliminary design of the filtration facility to quality for incentive funding.
- **Specific Design Guidance:**
  - Oregon AWOP and PSW goals and criteria are being used to guide project definition and the pilot plant study.
  - It is assumed that the design will be permitted by the City of Gresham Building Department.

- **Pilot Plant Study.** PWB is conducting a pilot plant study to validate proposed treatment approaches, inform design criteria, and assist the transition to full-scale operation.
- **Good Neighbor Agreement.** PWB is currently working with a Site Advisory Group to develop a Good Neighbor Agreement that is anticipated to be completed in early 2021. The agreement will document PWB's commitments to site neighbors.

Many of the suggested design considerations have been incorporated as assumptions for project definition and initial cost estimate development, and will be further refined and evaluated to assess feasibility during project design.

References

City of Portland, *ENB-9.01 - Green Building Policy, Binding City Policy Resolution No. 37122*, adopted by City Council April 22, 2015.

Energy Trust of Oregon, *Small Office Incentive Workbook*, "How to Use This Workbook," 2019.

Energy Trust of Oregon, *Custom Capital Fact Sheet*, 2019

OHA Public Health Division, *Oregon Administrative Rules, Chapter 333, Division 61 Drinking Water*. Oregon Seismic Safety Policy Advisory Commission (OSSPAC), *Oregon Resilience Plan: Reducing Risk and Improving Recovery for the Next Cascadia Earthquake and Tsunami*, Report to the 77th Legislative Assembly, Salem, Oregon, February 2013.

Chapter 6
Treatment Process Alternatives

The filtration facility will consist of a series of discrete unit processes that sequentially treat Bull Run water to achieve the finished water quality objectives described in Chapter 3: Water Quality Considerations. The filtration facility will be designed to meet regulatory requirements set forth by the Oregon Health Authority (OHA) in Oregon Administrative Rules (OAR) 333-061. Each unit process has a specific function to assist with the removal, conversion, or inactivation of undesirable constituents found in the Bull Run supply, and to enhance the water quality of the finished water.

This chapter describes treatment process alternatives considered for the Bull Run supply. The purpose and nature of these processes are described, along with a summary of engineering analysis and evaluation performed by the program team. The initial analysis described in this chapter documents the process to screen feasible alternatives and identify key considerations meriting additional discussion.

This chapter is organized into the following sections:

- 6.1 Oxidation
- 6.2 Rapid Mix
- 6.3 Flocculation
- 6.4 Powdered Activated Carbon (PAC) Adsorption
- 6.5 Sedimentation
- 6.6 Filtration
- 6.7 Disinfection
- 6.8 Clearwell
- 6.9 Corrosion Control
- 6.10 Solids Handling
- 6.11 Treatment Process Alternatives Summary

The process flow diagram in Figure 6-1 below illustrates the treatment process as described above.

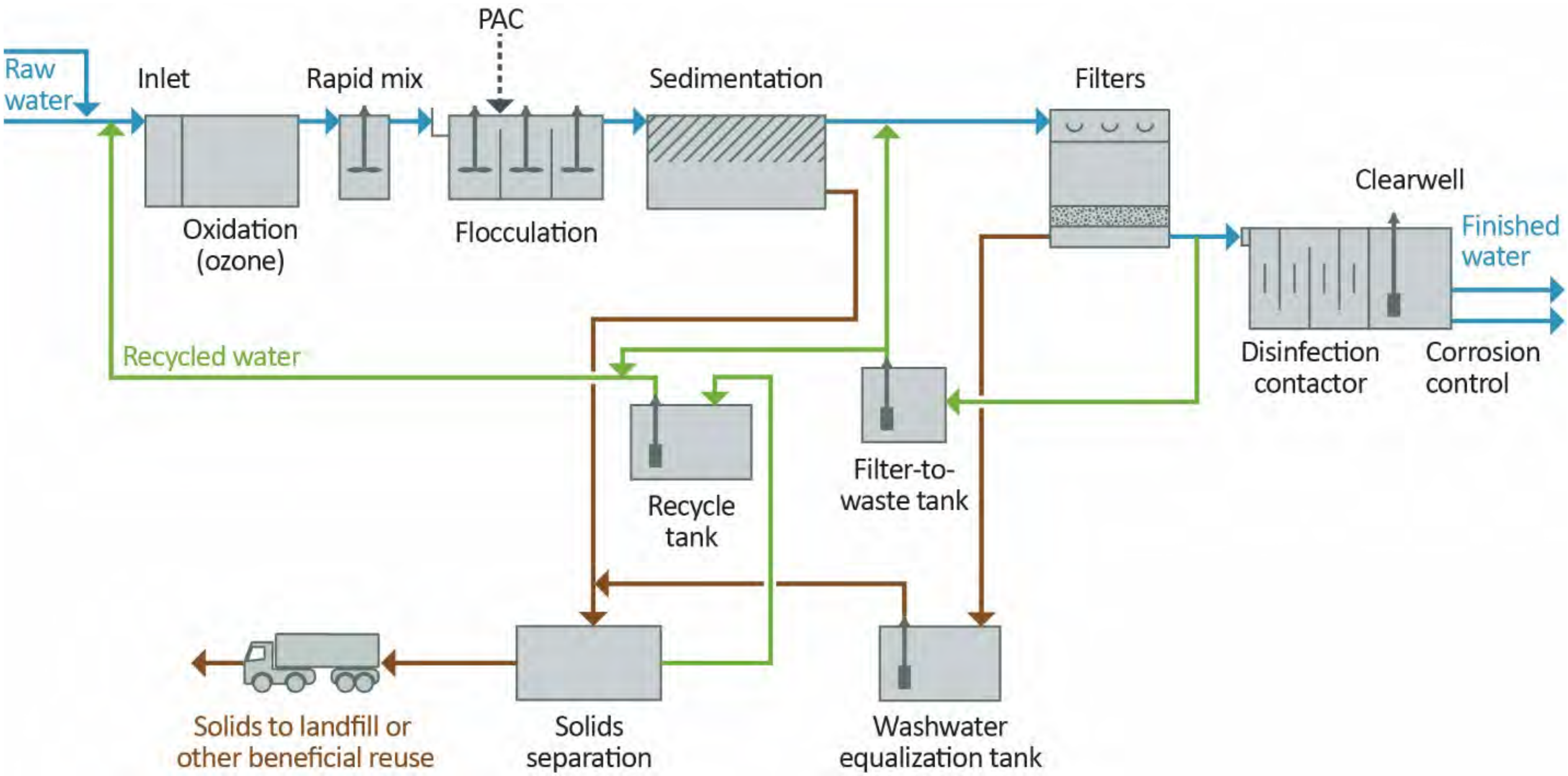
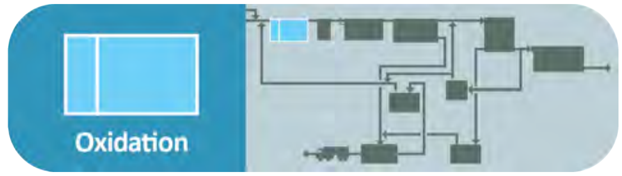


Figure 6-1. Process flow diagram for filtration unit processes

Note that PAC is not used in conjunction with ozone and that pre-ozonation is shown for illustration purposes.

6.1 Oxidation



The oxidation process alters the charge, or valance state, of floc particles or dissolved ions and breaks chemical bonds to convert larger molecules to smaller molecules that have more desirable chemical and physical properties.

Oxidation is often used ahead of flocculation to improve "microflocculation," the filterability of floc particles that promotes longer filter runs. Although the microflocculation benefit is driven by chemical mechanisms not fully understood, the results are well documented (MWH, 2012). Oxidation can also break up and destroy molecules responsible for taste and odor (T&O), disinfection byproduct (DBP) precursors, organics, color, and algal toxins. An example of the oxidation of soluble inorganics is oxidation of dissolved manganous ions (Mn^{+2}) to a solid and filterable form (Mn^{+4}), which is commonly precipitated as a type of manganese dioxide (MnO_2). MnO_2 is an effective adsorbent for iron, radium, and other trace inorganics. MnO_2 is formed as a red or black precipitate that must be removed, otherwise these precipitates will be deposited in the distribution system or home plumbing fixtures.

Choices related to oxidation include the type of oxidant, injection location within the treatment process, contact time, and design features of the contact vessel.

This section discusses the following topics:

- Chemical Oxidant Alternatives
- Dosing Location
- Oxidation Summary

Table 6-1 below presents a summary of important issues associated with oxidation that were considered to arrive at project definition assumptions.

Table 6-1. Important Issues for Oxidation

Important Issues	Description
Raw water quality characteristics	• Pre-oxidants can improve water filterability, especially for water with low turbidity levels such as the Bull Run supply. • Organic and inorganic matter may be successfully oxidized into more favorable forms for improvements to water quality. - Oxidation of organics may reduce DBPs and unfavorable T&O compounds. - Soluble inorganics such as manganese can be oxidized to a filterable/removable form.
Chemical type and dose	Chemicals such as chlorine, ozone, permanganate, chlorine dioxide, and hydrogen peroxide can be considered. Each has advantages and disadvantages in terms of effectiveness, public health protection, and safety.
Application point	Pre-oxidation ahead of flocculation is typically used to enhance filterability of low turbidity water and to oxidize organics and inorganics. Intermediate oxidation following sedimentation is used for water with higher organics concentrations. Sedimentation removes a portion of the oxidation burden, allowing oxidizer dosages to be reduced.
Concentration Time (CT)	CT is a function of the type of oxidant and the concentration of substances requiring oxidation. The range of contact time is generally 1 to 10 minutes. This will be verified during the pilot plant study.

6.1.1 Chemical Oxidant Alternatives

The oxidation process can be accomplished with a variety of chemical oxidants, each with specific chemical handling criteria, dosages, and contact time characteristics. This section provides screening guidance for oxidant alternatives considered for this project, including:

- Chlorine
- Ozone
- Potassium Permanganate
- Chlorine Dioxide
- Hydrogen Peroxide

Chlorine

Chlorine is the traditional and most commonly used oxidant. Chlorine will be used at the filtration facility to disinfect filtered water. Providing an additional feed system to meter chlorine for oxidation is inexpensive. Chlorine is commonly used as a primary pre-oxidant or as a backup to other pre-oxidants. Chlorine functions well as a pre-oxidizing agent but does introduce concerns of potentially elevating the concentration of regulated halogenated DBPs (i.e., total trihalomethanes [TTHMs] and halogenated acetic acids [HAA5s]), depending on the concentration and nature of precursor compounds found in the raw water. Under some circumstances, use of chlorine can lyse algal cells and potentially increase the risk of elevated algal toxins released to the water.

It is prudent to maintain flexibility to add chlorine at various injection locations. Chlorine used for oxidation can be sourced from either of the common feed systems (typically, gaseous chlorine or sodium hypochlorite) selected for primary disinfection. A chlorine solution would be prepared and injected via diffusers or mixers within the treatment process. For project definition, the use of a delivered bulk liquid sodium hypochlorite solution is assumed. PWB currently uses gaseous chlorine at Headworks. For public and operator safety relative to transport and use of this gas, most utilities are switching from gaseous chlorine to liquid hypochlorite. For added conservatism in project definition, it is assumed the electrical distribution system sizing is sufficient to provide power for onsite sodium hypochlorite generation, should that decision be favored during the design process.

Ozone

Ozone is a strong oxidant and disinfectant that has been used for more than 100 years in Europe and 80 years in the United States. Ozone is used at more than 290 drinking water treatment facilities in the United States and Canada. As of 2015, this included use of ozone in 42 states and an estimated 30 percent of the water treated in the United States (AWWA, 2015). Some of the water treatment facilities toured by PWB staff that include ozonation systems are identified in Table 6-2.

Table 6-2. Water Treatment Facilities Toured by Program Team that Use Ozone

Utility	State	Water Treatment Facility	Size (mgd)[a]
Lake Oswego Tigard Water Partnership	Oregon	Lake Oswego Tigard	38
Tacoma	Washington	Green River	150
Seattle	Washington	Tolt	120
Weber Basin Water Conservancy District	Utah	Davis North	46
Metropolitan Water District of Salt Lake and Sandy	Utah	Point of the Mountain	70
Central Utah Water Conservancy District	Utah	Don A. Christiansen Regional	100
East Bay Municipal Utility District	California	Sobrante	60
San Francisco Public Utilities Commission	California	Harry Tracy	140
San Diego	California	Miramar	215
San Diego	California	Alvarado	200
San Diego County Water Authority	California	Twin Oaks Valley	100
Metropolitan Water District of Southern California	California	Joseph Jensen	750
Metropolitan Water District of Southern California	California	Robert A. Skinner	350
Los Angeles Department of Water and Power	California	L.A. Aqueduct	600

a. Million gallons per day (mgd).

Ozone has historically been used to enhance flocculation, address T&O concerns, serve as a primary disinfectant, reduce halogenated DBPs, and improve coagulation and filtration (MWH, 2005). Ozone is also considered the Best Available Technology for its ability pharmaceutical and personal care products, endocrine-disrupting compounds, and contaminants like cyanotoxins or harmful algal blooms (HABs) often exacerbated by post-fire impacts.

Ozone is most often used before flocculation/sedimentation “pre-ozone” or after “intermediate ozone.” It is seldom used after filtration “post-ozone.” Ozone has also been shown to be effective as part of a multiple barrier for inactivation of microorganisms and *Cryptosporidium* and is considered a toolbox option (in conjunction with filtration) for inactivation in the event *Cryptosporidium* concentrations increase beyond present levels (OAR 333-061-0032 17b). Note that the use of ozone can form bromate in finished water; a DBP regulated to not exceed 10 µg/L. However, owing to the low concentrations of bromide ions found in the raw water, the formation of bromate is expected to be insignificant.

Ozone is generated on site for immediate use because its short half-life precludes storage. Ozone is generated from oxygen, either from ambient air or from a higher concentration oxygen source such as liquid oxygen (LOX). Feed gas is typically created by vaporizing LOX stored on site. Alternately, oxygen can be manufactured on site with vacuum swing adsorption systems. Oxygen gas is fed to the ozone generator, passing through a gap between a dielectric tube and a stainless-steel housing. High voltage passes through the dielectric to a grounded screen/plate. This process splits oxygen molecules into single oxygen atoms (O). These single oxygen atoms combine with oxygen molecules (O_2) to form ozone (O_3) at concentrations of 10 to 12 percent by weight. Ozone in the gas is transferred to the water using either fine-bubble diffusers or side-stream injection to establish a dissolved residual.

Ozone residual decay can occur within several minutes to over an hour, depending on water quality, temperature, and the initial dose. A concrete basin is typically constructed to provide appropriate contact time and contain undissolved ozone gas. Undissolved ozone off-gas must be collected and drawn through destruct equipment to convert it back to oxygen before release. Electrical power consumption may be somewhat high for oxygen generation (if used) and ozone generation, requiring increased electrical system capacity. Figure 6-2 shows ozone generators from the program team’s tour of East Bay Municipal Utility District’s facility.

Figure 6-2. Ozone generators from East Bay Municipal Utility District facility

El Sobrante, California

Ozone can be used to oxidize manganese; however, in practice, there are some complications in a typical surface water treatment facility. A milder oxidizer such as chlorine or potassium permanganate will oxidize manganous ions to form a manganese dioxide precipitate that can be filtered. These manganese oxide precipitates are retained in the filter until they are backwashed out. Ozone is a more powerful oxidant and has the potency to continue to oxidize manganese dioxide further to a soluble permanganate form, thus defeating the purpose of removing manganese as a filterable particle. This can be managed by controlling ozone dosing and contact time, or by quenching ozone residuals before too much contact time with manganese dioxide precipitates occurs.

Potassium Permanganate

Potassium permanganate ($KMnO_4$) is often used to control T&O and to remove iron and manganese. Potassium permanganate is commercially available in crystalline form and can be fed directly into the process stream using a dry chemical feeder, or metered into the process stream in a concentrated solution prepared on site. Liquid solutions of sodium permanganate ($NaMnO_4$) are available for general ease of handling; however, costs may favor the use of potassium permanganate.

Potassium permanganate oxidizes soluble forms of iron and manganese into a solid, filterable form via a change in valance state. If potassium permanganate is overdosed in the 0.05 mg/L range, the non-reduced (unreacted) permanganate produces a pink color in the water. Under most treatment conditions when used for manganese removal, soluble manganese (manganous ion form) is reduced to insoluble manganese dioxide (MnO_2). Typically, MnO_2 removal is achieved through conventional clarification or filtration.

Potassium permanganate is commonly added at the head of the treatment process to allow sufficient time for oxidation, and the removal of pink color and MnO_2 precipitates.

Chlorine Dioxide

Chlorine dioxide (ClO_2) is another strong oxidizing agent for oxidation of organics and inorganics such as manganese. Like ozone, chlorine dioxide dissipates quickly and must be generated on site. Typically, two separate chemical feed systems are needed where a solution of sodium chlorite is reacted with chlorine to form chlorine dioxide gas. This gas is then dissolved in water to form a chlorine dioxide solution for metering to specific injection points. This process involves use of two chemical feed systems; however, the chlorine system used to prepare chlorine dioxide will already be required at the filtration facility for disinfection purposes.

Unreacted chlorite is a regulated DBP of concern, and the use of chlorine dioxide creates subsequent chlorate ions that, while not currently regulated, are listed as a general health concern. Moreover, there is a small hazard risk with handling sodium chlorite solution, should it ever spill and evaporate to form potentially explosive anhydrous sodium chlorite crystals if subsequently exposed to organic materials (e.g., crystals coming into contact with oils or greases).

Chlorine dioxide has been shown to be effective as part of a multiple barrier for inactivation of microorganisms and *Cryptosporidium* and is considered a toolbox option (in conjunction with filtration) for inactivation in the event *Cryptosporidium* concentrations increase beyond present levels (OAR 333-061-0032 17a).

Chlorine dioxide would provide no benefit for turbidity events (including post-fire) or algal toxins. Additionally, chlorine dioxide cannot maintain a residual in the distribution system so chlorine and ammonia (chloramines) would still be needed. Chlorine dioxide is used at few facilities in North America and is typically used for pre-oxidation of organics as a substitute for chlorine or permanganate, rather than disinfection for pathogens.

Hydrogen Peroxide

Hydrogen peroxide can serve as an oxidizing agent for a variety of applications. For example, in the use for manganese removal, it can convert manganous Mn^{+2} ions to filterable Mn^{+4} ions forming MnO_2, by acting as an electron acceptor in this reaction. Hydrogen peroxide can also serve as a reducing agent. For example, it can reduce chlorine residual (HOCl) to chloride salt (Cl^-) by acting as an electron donor in this reaction. It is purchased as a liquid in a variety of strengths. Commercially, a 35 percent strength solution is most common for water treatment applications. This chemical is not commonly used to treat water similar to the Bull Run supply. This is partly because of cost, especially when compared to the relative cost-benefit of chlorine. Hydrogen peroxide is often considered for treating groundwater impacted by high concentrations of sulfur or other compounds of industrial origins.

A summary comparison of the advantages and disadvantages of each oxidant is considered in Table 6-3.

Table 6-3. Oxidants Summary Comparison

Advantages	Disadvantages
Chlorine (assuming use of sodium hypochlorite)	
• Gaseous chlorine currently used at Headworks. • Inexpensive. • Known to be effective for pre-oxidation purposes, including manganese. • Provides additional barrier for microorganisms (bacteria, *Giardia*, and viruses). • Commonly used to enhance flocculation and filtration performance. • Widely used where total organic carbon (TOC)/dissolved organic carbon (DOC) or DBP formation potential values are low. • May reduce color.	• Can increase concentrations of halogenated DBPs. • Can rupture (lyse) algae cells and release various algal toxins. • Not commonly used for organics control.

Table 6-3. Oxidants Summary Comparison

Advantages	Disadvantages
Ozone	
• Reduces halogenated DBP formation potential by oxidizing organic carbon. • Provides additional barrier for microorganisms (bacteria, *Cryptosporidium*, *Giardia*, and viruses) and cyanotoxins. • Suitable for additional *Cryptosporidium* inactivation credits in conjunction with filtration if Bin 2 or higher. • Improves T&O, reduces color. • Provides significant process enhancements (microflocculation). • Reduces coagulant dose (pre-ozone only). • Improves settled and filtered water quality. • Improves filter performance (longer runs). • Facilitates biofiltration, further reducing T&O, DBPs, and CECs.	• Capital costs for equipment, buildings, and basins (partially offset by other process savings). • Operating costs for power and LOX (partially offset by other process savings). • Complex system operations and monitoring. • Excess unreacted ozone gas must be converted to oxygen. • May promote formation of bromate if bromide levels in source water are high (not applicable to Bull Run source). • Requires ozone residual control or quenching upstream of filtration if manganese oxidation and removal is required.
Potassium Permanganate	
• Effective as a pre-oxidant for manganese. • Inexpensive. • Used for periodic control of T&O. • Minor reduction of halogenated DBP formation potential. • May reduce color.	• Risk of pink water if overdosed. • Not as powerful an oxidizer as chlorine.
Chlorine Dioxide	
• Known to be effective for pre-oxidation purposes, including manganese. • Provides an additional barrier for microorganisms (bacteria, *Giardia*, *Cryptosporidium*, and viruses). • Shown to be effective for inactivation of *Cryptosporidium*; considered toolbox option for inactivation per OAR 333-061-0032 17a. • Does not create halogenated DBPs. • Less expensive than ozone. • May reduce color.	• Requires two reagents (sodium chlorite and chlorine). • Can result in unreacted chlorite ions. Chlorite MCL is 1.0 mg/L per OAR 333-061-0030 2b. • One byproduct is the formation of chlorate ions (ClO_3^-), an unregulated DBP. Although not currently regulated by OHA, chlorate concentrations in drinking water are a concern due to creating thyroid problems, and other agencies have suggested maximum concentration guidelines (e.g., World Health Organization guideline of 0.7 mg/L and EPA Health Advisory Limit of 0.21 mg/L).
Hydrogen Peroxide	
• Typically, used with oxidants such as chlorine, ozone, or UV to produce hydroxyl radicals used for difficult-to-treat pollutants of industrial origin. • May reduce color.	• Not commonly used for raw water similar to the Bull Run source due to cost.

6.1.2 Dosing Location

This section further discusses how oxidation can be accomplished with these chemical oxidants, including potential injection locations within the treatment process based on the specific chemical handling criteria and contact time characteristics of each oxidant. The methods of chemical injection and the need for adequate contact time are similar, but the purpose and chemical dosing ranges may differ.

This section includes the following topics:

- Pre-oxidation
- Intermediate oxidation
- Contact Time

Pre-oxidation

There are two nominal locations for the application of oxidants in the treatment train; the first location is pre-oxidation upstream of rapid mix (Figure 6-3). Pre-oxidation enhances flocculation and filtration effectiveness as described above, with a secondary focus on oxidation of inorganic and organic compounds. This process is particularly useful for cold raw water with low turbidity levels that are difficult to coagulate/flocculate and filter efficiently. Many water treatment facilities, especially those that treat cold, low turbidity Cascade Range water, use some form of pre-oxidation to improve performance. This process is currently practiced at the water treatment facilities for Seattle (using ozone), Everett (using chlorine), and Tacoma (using ozone) where operators indicate that pre-oxidation improves flocculation and filtration effectiveness compared to periods when pre-oxidation is temporarily suspended. In addition, the pilot plant study conducted for PWB in the 1990s indicated that pre-oxidation with ozone produced significant benefits to aid flocculation and filtration of Bull Run water.

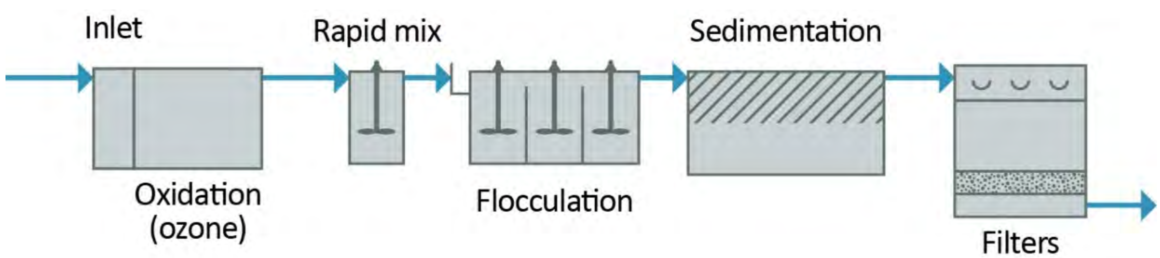


Figure 6-3. Dosing location showing pre-oxidation upstream of rapid mix

Intermediate Oxidation

The second location for application of oxidants is intermediate oxidation downstream of sedimentation and upstream of filtration (Figure 6-4 below). Intermediate oxidation is applied to conventional filtration processes only. Because the flocculation/sedimentation process has removed materials that can react with the oxidant, the oxidant demand in this arrangement is reduced. For this reason, intermediate oxidation is generally considered to mitigate overall oxidant usage when raw water contains high concentrations of oxidizable materials. Intermediate ozonation is practiced at the Lake Oswego Tigard water treatment facility. Note that intermediate oxidation does not provide the flocculation and filtration benefits associated with microflocculation.

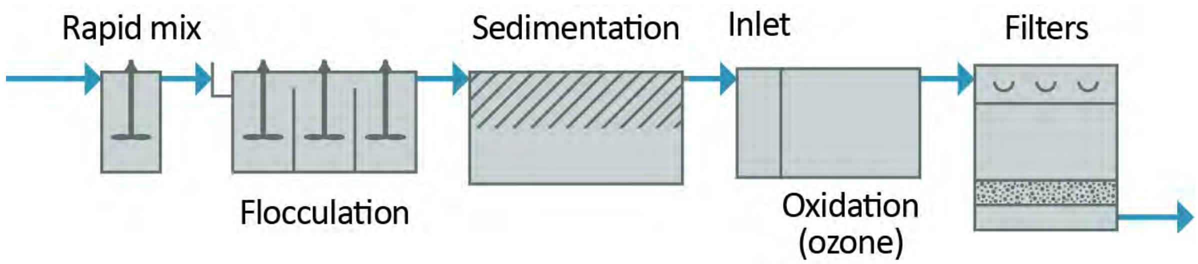


Figure 6-4. Dosing location showing intermediate oxidation upstream of filtration

Given PWB's raw water characteristics, use of oxidizers at similar facilities to enhance flocculation and filtration, results of the 1990s pilot plant study, and interim results of the ongoing pilot plant study, pre-oxidation is the most logical injection point. In the future, if raw water conditions change significantly (e.g., increased concentrations of algal toxin and undesirable organics) consideration may be given to intermediate oxidation to help reduce overall oxidant use.

Contact Time

The contact time for oxidation is assumed to be 10 minutes for both pre- and intermediate oxidation, which is a conservative assumption for project definition and consistent with ozone decay values (MWH, 2012). This value is to be revised once future pilot plant study data has been analyzed. For ozonation, a dedicated, totally enclosed contactor is selected. For pre-oxidation with either chlorine or potassium permanganate, multiple feed lines will be provided to feed these chemicals in the flocculation or sedimentation basins or the raw water pipelines upstream of the filtration facility.

Contact times and reactor configurations will be determined once pilot plant study data is available, however a 10-minute contact time is initially assumed. Note that ozone contactors are specialized and, typically, made of concrete and enclosed to collect and convey off gasses to subsequent neutralization (Kawamura, 2000, and Ten States Standards Section 4.4.7.4.6). Other oxidants can be more easily added to conveyance piping, feed channels or open vessels (e.g., flocculation or sedimentation basins) where off gasses are not present to any large degree.

6.1.3 Oxidation Summary

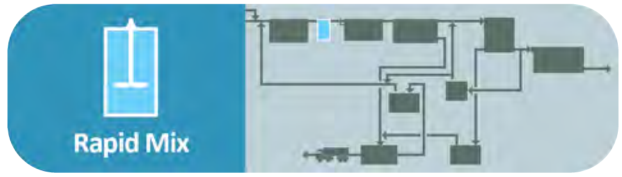
This section summarized advantages and disadvantages of oxidizing agents and associated dosing locations to treat raw water similar to the Bull Run supply. While not all filtration facilities require oxidation, discussion with the Technical Advisory Committee (TAC) in December 2017 and February 2018 workshops demonstrated that oxidation helps further manage existing raw water quality with respect to manganese, T&O, and DBP precursors. Chapter 3 discussed the presence of manganese and organic substances in the Bull Run supply and the treatment goals to reduce their impacts. Based on this information, it is prudent to evaluate using oxidation to further remove these substances from the raw water. Moreover, the benefits of oxidation are more significant should raw water quality degrade over time. The

1990s pilot plant study illustrated the benefits of oxidation. The current pilot plant study will further examine oxidation to identify benefits and validate design criteria.

Conclusions of the program team on oxidants were as follows:

- **Chemical Oxidants Alternatives:**
  - Ozone was selected for subsequent detailed analysis owing to its significant value from a water quality perspective. Ozone has a superior ability to improve filtration performance and more effectively oxidize organics, including algal toxins and a broad range of T&O and other potentially future regulated compounds. Based on this value and feedback provided by the TAC, the program team believes ozone will play a role in the treatment of Portland's drinking water.
  - Given the cost and the feed and contact system needs, implementation of ozone might be delayed. Thus, chlorine and potassium permanganate were also advanced for consideration as oxidants. These oxidants would be used until a future ozone process could be constructed.
  - It was determined during the TAC workshops that the use of PAC would not be necessary for control of organics if ozone is used.
  - Use of chlorine dioxide and hydrogen peroxide as oxidants for Bull Run raw water is not recommended. This assessment is based on the effectiveness of the oxidants, common use in the industry for this purpose, and overall concerns of water quality. This conclusion was confirmed with input from the TAC.
  - During the TAC workshops, it was also judged that chlorine or permanganate oxidation would not be as effective as ozone.
- **Dosing Location:**
  - Pre-oxidation will likely offer significant water quality benefits with respect to improving filtration performance, oxidizing undesirable organics and inorganics (e.g., T&O, DBP precursors, algal toxins, other unregulated organics, and manganese), and providing a multiple barrier for control of microbials.
  - Pre-oxidation with chlorine and potassium permanganate warrant consideration during design. In particular, if ozone is not included, pre-oxidation for improved filter performance and manganese removal will be needed. These pre-oxidants (e.g., chlorine) may also offer benefits when used in conjunction with ozone.
  - Note that it may be hydraulically feasible to consider an oxidation reactor that can be configured for either pre- or intermediate oxidation. This concept is suggested for further examination during design.
- **Contact Time.** The contact time for oxidation is assumed to be 10 minutes for both pre- and intermediate oxidation.

6.2 Rapid Mix



The rapid mixing process, also referred to as flash mixing, provides rapid addition of water treatment chemicals (e.g., coagulants and coagulant aids) into the raw water.

Rapid mixing helps the flocculation process achieve effective particle destabilization by quickly and uniformly dispersing coagulants with enough mixing energy to initiate coagulation. Rapid mixing is especially critical when treating cold water such as the Bull Run supply with low turbidity, organics, and alkalinity. Effective rapid mixing minimizes chemicals used to optimize flocculation and coagulation. With metal salt coagulants, rapid mixing can save 25 to 30 percent on coagulant required (Kawamura, 2000).

Table 6-4 presents a summary of important issues associated with rapid mixing that were considered to arrive at project definition assumptions.

Table 6-4. Important Issues for Rapid Mixing

Important Issues	Description
Type and dose of pre-oxidants, coagulants, and coagulant aids	• Types and dose of coagulants and coagulant aids will be evaluated during the pilot plant study and will influence detention time and velocity gradients (i.e., mixing energy) used for design of rapid mix systems. • Metal salt coagulants are commonly used primary coagulants. Some water treatment facilities also feed a polymeric coagulant aid with the primary coagulant. • Optimal coagulation may depend on pre-oxidation to improve effectiveness of floc formation and settleability and help convert soluble and colloidal compounds to particulates for removal.
Raw water quality characteristics	• Temperature, water density, and viscosity will be important features affecting rapid mix design. Raw water, such as the Bull Run supply, with low turbidity, temperature, organics, and alkalinity impacts the effectiveness of flocculation.
Type of rapid mixing	• Several types of rapid mixing technologies are available and are suggested to be evaluated during design.

The most common types of rapid mixing include:

- In-line static mixing
- In-line mechanical mixing
- In-tank/basin mixing
- Pumped diffusion mixing
- Hydraulic mixing

6.2.1 Rapid Mixing Summary

This section briefly described the rapid mix process and listed rapid mix alternatives that can be considered during the design process for use at the filtration facility. For project definition, pumped diffusion rapid mixing is assumed. Pumped diffusion, selected on the basis of performance, is commonly used at large water treatment facilities and offers the benefits of minimal system head loss, adjustable energy inputs, excellent performance, and commonly available equipment that is easily maintained. Note that selection of the preferred rapid mix will not greatly affect the project definition cost estimate, and the final selection of rapid mix facilities will be made during design.

Figure 6-5 illustrates a typical pumped diffusion rapid mixing system.

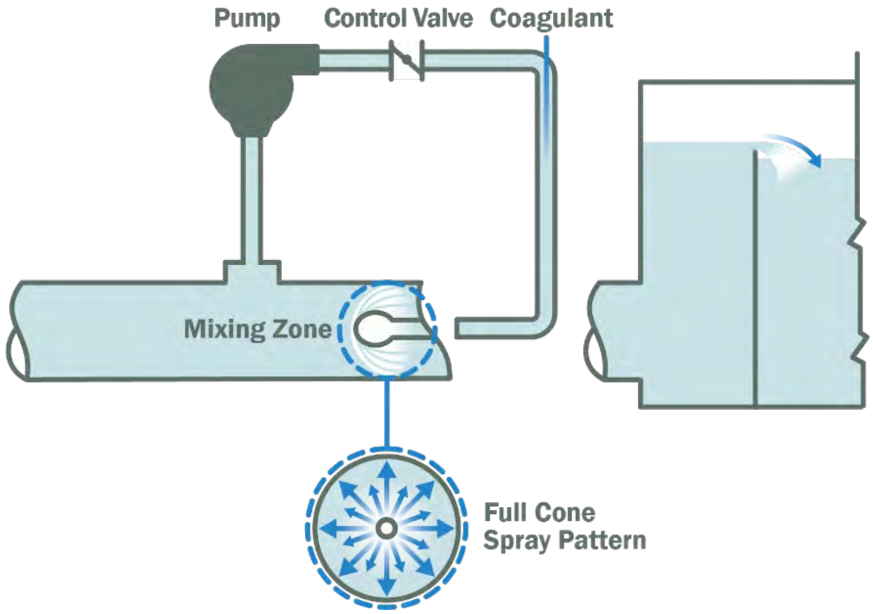
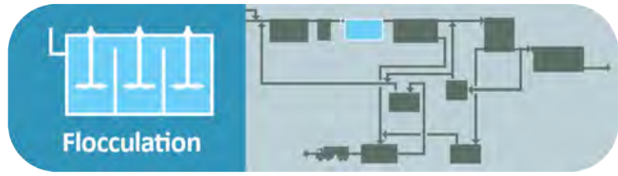


Figure 6-5. Example pumped diffusion rapid mixing

6.3 Flocculation



The flocculation process relies on a combination of gentle mixing and stirring energy and time to produce a suitable floc.

Flocculation provides operators with process flexibility to enhance overall treatment performance. Operators can optimize complex water chemistry by their choice of coagulant chemicals, mixing energy input, and reaction time to destabilize colloids and form floc.

The mixing energy and detention time characteristics of flocculation help particulate matter collide and react with coagulants and initiate destabilization. This creates a more settleable or filterable floc by building particles of optimum size and density to be removed in the downstream sedimentation or filtration processes. Flocculation typically has two to four discrete mixing energy stages (Kawamura, 2000), with typical detention times varying from 15 to 45 minutes depending on raw water quality and temperature. As water passes through each stage, the mixing energy input incrementally decreases to reduce shearing of forming particles.

The optimal floc size is dependent on the downstream treatment unit. For direct filtration, smaller floc can penetrate deeper into the filter media and thus avoid blinding the filter or accumulating excessive head loss. For conventional filtration, larger floc settle out better in the sedimentation basin and minimize the solids loading onto the filter.

Table 6-5 presents a summary of important issues associated with flocculation that were considered to arrive at project definition assumptions.

Table 6-5. Important Issues for Flocculation

Important Issues	Description
Raw water quality characteristics	• Quality of raw water and type of pre-oxidants, chemical coagulants, and coagulation aids can impact flocculation performance. Constituents such as turbidity, pH, alkalinity, algae, temperature, organics, and inorganics can impact flocculation characteristics.
Range and variability of facility flows	• Flow range and variability can significantly impact detention time and hydraulic mixing requirements for optimal flocculation. The effectiveness of upstream rapid mixing directly affects flocculation performance.
Downstream sedimentation or filter operation	• Type of filtration system and mode of operation, including direct or conventional filtration, impacts the desired type of floc particle. Direct filtration uses a “pin” type floc formation that can be more readily filtered, as compared to conventional filtration, which uses a larger, settleable floc.
Type of flocculation system	• Several flocculation technologies are available and are suggested to be evaluated during design. Variable and tapered mixing energy input is highly desirable.

Flocculation is achieved by vertical or horizontal mechanical devices or by hydraulic means alone. For mechanical flocculation, horizontal paddles and vertical mixers (e.g., paddles, turbines, and hyperboloid impellers) are general configurations for consideration (Kawamura, 2000). These devices are frequently coupled with variable speed drives to regulate mixing energy input. No particular configuration dominates in hydraulic flocculation. Selection of a mechanical flocculation option is driven by multiple factors, including effectiveness in energy transfer without undue floc shear, the nature of downstream processes that affect filterability (i.e., sedimentation and filtration), ease and flexibility of maintenance, and operational preferences and level of available operator expertise. Common types of flocculation mixing include:

- Vertical shaft flocculators in each flocculation stage (mechanical mixing).
- Horizontal shaft flocculators in each flocculation stage (mechanical mixing).
- Baffled channel flocculation tanks (hydraulic mixing).

6.3.1 Flocculation Summary

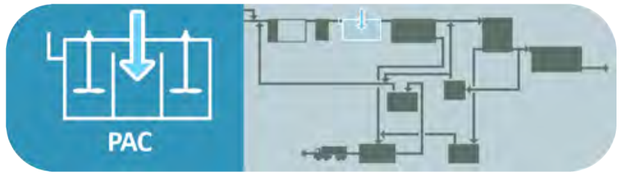
This section briefly described the flocculation process and listed flocculation alternatives that can be considered during the design process for use at the filtration facility. The vertical shaft flocculation system is assumed for project definition. Vertical shaft flocculation is commonly used at newer filtration facilities such as the water treatment facilities for Tacoma and Metro Vancouver (Kawamura, 2000). Vertical shaft flocculation with variable speed drives also offers the benefits of a high degree of mixing energy and speed (velocity gradient) adjustability, equipment redundancy, and easy access for equipment maintenance that does not require the flocculator to be dewatered. Note that selection of the preferred flocculation system will not greatly affect the project definition cost estimate, and the final selection of flocculation systems will be made during design.

Figure 6-6 shows a vertical shaft flocculator drive from the program team's tour of the Joint Water Commission's water treatment facility.

Figure 6-6. Vertical shaft flocculator drive at Joint Water Commission facility

Hillsboro, Oregon

6.4 PAC Adsorption



The adsorption process in water treatment consists of a sorbent (typically, a solid bulk material) that exerts weak attractive forces (e.g., covalent bonding, van der Waals, and electrostatic) to physically attract and retain specific dissolved sorbate molecules for removal from water.

The adsorption process is rate-limited and influenced by chemical equilibria, sorbate concentration gradients, and sorbent capacity. There are several types of common sorbents used in water treatment (e.g., activated alumina, chitosan, zeolite, clay minerals, and activated carbon). Only activated carbon is considered in this chapter because of its effective affinity for the specific organic molecules of concern related to T&O, DBP formation, and certain types of algal toxins. Activated carbon is commonly used in water treatment for adsorption and is made from a variety of materials, including wood, coconut shells, lignite, and coal. Activated carbon can be furnished in a fine, powdered form known as PAC, or in a granular form known as granular activated carbon (GAC).

PAC is used when a contaminant of interest varies in concentration and season (such as T&O-causing compounds and cyanotoxins), while GAC is generally used when the contaminant is present year-round (such as contaminated groundwater plumes or baseline TOC reduction). As such, PAC is anticipated to be more suitable than GAC for treating Bull Run water.

PAC is typically fed during rapid mix or flocculation on an as-needed basis when cyanotoxins or T&O are present. To avoid blinding filters with the use of PAC and reducing filtration facility productivity, sedimentation is commonly used to largely remove PAC before it can reach the filters. The PAC particles adsorb organic compounds encountered in the water which then settle out in the sedimentation basin or are captured in the filters.

GAC is also used in treating water via adsorption but is generally applied in dedicated contact vessels near the end of the treatment process. In this application, water flows past GAC granules and the degree of adsorption is commonly controlled by detention time, characterized by the key design criterion of empty bed contact time. GAC adsorption is not included in the project definition scope because the Bull Run raw water quality does not have sufficient concentrations of organic materials to require use of this relatively expensive unit process. However, GAC is discussed as a filter media in subsequent sections of this chapter. In that capacity, GAC is used as a physical filter media and its adsorptive qualities are not considered.

Table 6-6 below presents a summary of important issues associated with PAC that were considered to arrive at project definition assumptions.

Table 6-6. Important Issues for PAC

Important Issues	Description
Flexibility	• PAC dosage can be adjusted as water quality changes.
Delivery form	• PAC can be purchased and stored in 50-pound bags, 1–2-ton supersacks, or delivered by bulk trucks and fed as a powder using a dry feed system. PAC can also be purchased in bulk and fed as a slurry using metering pumps and mixing tanks.
Storage	• Storage in bags requires dust suppression control systems.
Dosing location	• PAC should be added prior to oxidants or disinfectants since oxidants such as chlorine and potassium permanganate have a negative effect on PAC removal of T&O.
Use of sedimentation	• Sedimentation is used to collect spent PAC before it can reach the filters, preventing early filter clogging and reductions in productivity.
Solids production	• Use of PAC will proportionally increase solids production at the filtration facility.

PAC is an effective adsorbent for organic compounds. Water treatment facilities may use PAC seasonally to improve removal of problematic organic compounds. For example, this process was used seasonally at the Don A. Christiansen water treatment facility in Utah before pre-ozonation was installed.

The most common application is for seasonal removal of T&O-causing compounds. These compounds can derive from natural processes in forested watersheds; for example, organics causing earthy-musty odors such as geosmin and 2-methylisoborneol are released by actinomycetes and cyanobacteria (Lalezary et al., 1986). Because these compounds are released by natural processes, their presence in the source water is often transient. When the T&O-causing compounds are present, PAC can be fed at appropriate doses to adsorb the target organics; after the event passes, the PAC dose can be lowered or eliminated.

Another common usage for PAC is removal of cyanotoxins. PAC is effective at removing extracellular cyanotoxins, although the required dosage depends on the form and concentration of the cyanotoxin, and the type of PAC applied. Knowledge of the expected types and concentrations of cyanotoxins in the source water is critical for sizing PAC feed equipment. Site-specific testing is suggested to develop removal isotherms for the type(s) of PAC to be used in the source water matrix.

Beyond the targeted removal of specific organic compounds such as those discussed above, PAC may be used for general reduction of TOC to improve the performance of downstream treatment processes and to remove DBP precursors.

6.4.1 Implementation

PAC is used for removal of targeted organic compounds in source water. Due to concerns about future raw water quality changes, removal of organics concentrations higher than those currently experienced must be considered. The function of PAC partially overlaps with the function of ozone treatment. As discussed in the previous section on oxidation, higher concentrations of organics may be removed with chemical oxidation (e.g., chlorine, ozone, or permanganate). Ozone is considered more powerful and versatile for handling raw water organics, and unlike PAC, ozone provides microflocculation benefits that improve filter productivity without increasing residual solids production.

Alternatively, these organics may be adsorbed and removed by PAC. Based on discussion at the TAC workshops, the use of ozone would provide adequate means to address organics without the need to use PAC; however, if ozonation is not provided, the use of PAC is preferred. Moreover, sedimentation is typically needed if PAC is used to help reduce filter clogging and control related filter productivity losses.

If PAC is used as the primary treatment technology to remove targeted organic compounds, the ability to successfully feed PAC will be vital to facility operations. PAC systems are operated like other chemical addition systems; PAC is either added through a dry feed system or concentrated slurry. Important criteria for selecting the point of application include 1) effective mixing, 2) sufficient contact time, 3) minimal interference of other chemicals with adsorption, and 4) no degradation of finished water quality. Adsorption kinetics and equilibrium capacity are dependent on the type of adsorbate, the type of PAC, and the competition from background organic matter. If contact time is insufficient, it must be compensated with an overdose of PAC.

If ozone is not provided, it is suggested the PAC storage and feed facilities be designed with full redundancy, including two or more storage tanks, two or more feed pumps or feeders, and redundancy in feed piping to allow for continued feed in the event of clogging.

If the facility is constructed as a direct filtration facility, the ability to feed PAC will be highly limited by the potential solids loading that can be applied to the filters. PAC adsorbs organic compounds to carbon particles, but these organics are not removed until the PAC particles are removed. Typically, a majority of the PAC particles are removed during sedimentation.

Table 6-7 summarizes advantages and disadvantages of PAC.

Table 6-7. PAC Summary Considerations

Advantages	Disadvantages
• Inexpensive capital costs. • Lower capital costs compared to GAC. • Ability to adjust PAC dosage as water quality changes, used when needed. • Effective against algae, T&O, low concentrations of pesticides, and other organic micropollutants. • Widely used for control of periodic T&O and other issues.	• High operating costs if high PAC doses are required for long periods of time. • Inability to regenerate. • Lower organics removal than ozone. • Increased difficulty of sludge disposal. • Difficult to remove PAC particles completely from the water. • Low removal of TTHMs and volatile organic compounds relative to ozone. • May require dust control depending on the feed system; some concern of potential dust explosions. • Can clog filters and decrease run times. • Typically, paired with sedimentation.

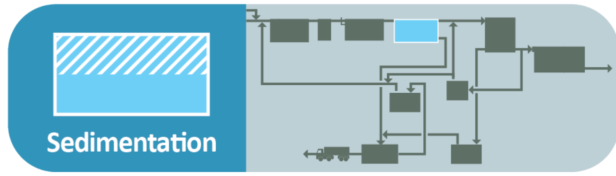
6.4.2 PAC Adsorption Summary

This section described considerations for implementing a PAC system at the filtration facility. For project definition, a PAC system is suggested if ozonation is not included.

Conclusions of the program team on PAC adsorption were as follows:

- **Implementation:**
  - PAC is used when a contaminant of interest varies in concentration and season (such as T&O-causing compounds and cyanotoxins), while GAC is generally used when the contaminant is present year-round. As such, PAC is anticipated to be more suitable than GAC for treating Bull Run water.
  - If the filtration facility is constructed with ozone feed capabilities, there will be little to no need to add storage and feed facilities for PAC. Conversely, if the filtration facility is constructed without the ability to apply ozone, then PAC will be needed for periodic removal of target organic compounds.
  - Sizing of the PAC system will be highly dependent on the expected contaminant loads in Bull Run source water. Relatively high dosages of PAC may be needed to respond to T&O-causing compounds or cyanotoxins and to achieve finished water targets and log-removal requirements.
  - Without sedimentation, the use of PAC would result in high solids loading onto the filters. This may preclude the use of PAC as an option for removal of some organic compounds, including certain cyanotoxins.

6.5 Sedimentation



The sedimentation process removes flocculated and settleable material from the liquid stream prior to filtration.

Sedimentation diminishes the solids burden on the filters, improving filtered water quality and filter productivity with longer filter runs and less water needed for filter backwashing. Additional benefits associated with sedimentation include 1) providing 1 to 2 hours more contact time for colloidal and dissolved substances to either be adsorbed onto floc particles or oxidized, 2) providing more time for operators to react to raw water quality changes or production challenges, and 3) providing the capability to treat higher turbidity water than direct filtration as may occur after a large storm or fire.

This section discusses the following topics:

- Direct and Conventional Filtration Comparison
- Basic Sedimentation Alternatives
- Full-sized and Hybrid Sedimentation Comparison
- Sedimentation Summary

Table 6-8 presents a summary of important issues associated with sedimentation that were considered to arrive at project definition assumptions.

Table 6-8. Important Issues for Sedimentation

Important Issues	Description
Raw water quality characteristics	• Raw water quality and flocculation performance can impact sedimentation performance. • Constituents such as turbidity, pH, alkalinity, algae, temperature, organics, and inorganics can impact sedimentation characteristics.
Flexibility	• Sedimentation can accommodate large swings in raw water turbidity and provides a location to settle PAC (if used).
Type of sedimentation systems	• Types of sedimentation can be considered relative to cost, footprint, and effectiveness at handling potential large increases in raw water turbidity.
Footprint	• Footprint and unit hydraulic loading rates must be selected appropriately to match the specific type of sedimentation system selected.

6.5.1 Direct and Conventional Filtration Comparison

The consideration of sedimentation as part of the treatment process is associated with evaluation of the two basic treatment configurations applicable to the filtration facility:

- Direct filtration (excludes sedimentation)
- Conventional filtration (requires sedimentation)

Direct filtration is commonly used for raw water with low, stable turbidities and organic concentrations. During periods of elevated and sudden raw water turbidity increases, direct filtration facilities often struggle to maintain finished water quality and production goals. Conversely, conventional filtration is typically used when raw water exhibits corresponding higher or more erratic characteristics.

The selection of direct filtration or conventional filtration is driven by an understanding of the raw water quality. From the data summarized in Chapter 3, PWB's existing raw water quality is generally stable with low turbidity, organics concentrations, and alkalinity. However, there are occasions where raw water turbidity at the Headworks has been observed as high as 25 NTU (capped by the maximum value read by the turbidimeter), and the historical record for turbidity in Reservoir 2 indicates occasional values on the order of 100 NTU (capped by the maximum value read by the turbidimeter). Moreover, as discussed in the TAC workshops, there is a possibility of future raw water quality degradation resulting from climate change, algal blooms, wildfires, and landslides.

Typical raw water quality guidance for direct and conventional filtration is shown in Table 6-9 below. As seen in Table 6-9, the Bull Run source appears reasonably amenable to direct filtration under current normal conditions; however, occasional raw water turbidity increases may compromise direct filtration operations and concerns about future changes to raw water quality and watershed integrity indicate that conventional filtration be considered.

This section includes more detailed discussion of the following topics:

- Direct Filtration
- Conventional Filtration
- Direct and Conventional Filtration Comparison

Table 6-9. Typical Raw Water Quality Guidance

Feature	Raw Water Quality Criteria[a]		Historical Intake Raw Water Quality			Potential Post-Fire Water Quality
	Direct Filtration	Conventional Filtration	Mean	95th percentile	Max.	
Typical Turbidity (NTU[b])	<5 (75% of the time)	>20–50	0.6–1.6[c]	1.3–4.6[c]	—	>50
Max. Turbidity (NTU)	Max. 20 (for 3 hrs.) or Max. 30–40 (instantaneous)	1,000–3,000	—	—	>20 (0.2% of time) >20 (0.9% of time) North Tower	100–1,000, possibly as high as 10,000
Max. Color (color units)	<20	>20	11	18	75	>20
Max. TOC (mg/L)	<4.0	>4.0	1.0	1.7	4.1	>4.0
Total Algae Count (asu/mL)	<2,000	>2,000	287[d]	1,195[d]	3,340[e]	>2,000
Iron (mg/L)	<0.3	>0.3	0.063	0.161	0.269	>0.3
Manganese (mg/L)	<0.05	>0.05	0.0099	0.0312	0.056	>0.05

a. Data is largely summarized from Utah regulatory standards.

b. Nephelometric turbidity units (NTU).

c. First value represents maximum daily turbidity for compliance from 1970–2014 measured from grab samples at Headworks. Second value represents maximum hourly turbidity from SCADA collected at the Reservoir 2 North Tower from 2013–2018.

d. PWB Comprehensive Raw Water Quality Data (2007–2017).

e. Total algae data collected by PWB but may not be in units identical to asu/mL. One asu is an algal clump viewed under microscope with an area of 400 µ2.

Direct Filtration

A water treatment facility that excludes sedimentation is identified as a direct filtration facility (OAR 333-061-0020 59). A general process schematic illustrating the arrangements of direct filtration is shown in Figure 6-7.

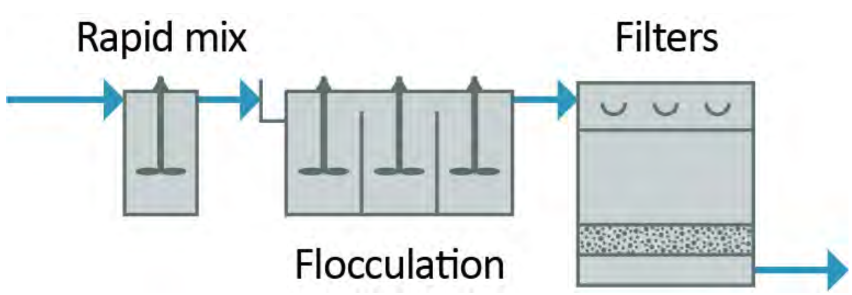


Figure 6-7. Process schematic for typical direct filtration

The historical Bull Run water quality appears amenable to direct filtration; however, direct filtration is less capable of reacting to sudden swings or spikes in turbidity or organic concentrations (e.g., T&O, DBP precursors, algae, and algal toxins). During these episodes, finished water production may drop due to shorter filter run cycles and higher amounts of water may be needed for filter backwashing, potentially compromising the filtration facility's ability to meet demand targets. These types of sudden swings or spikes in raw water quality can affect performance of conventional filtration systems, but not to the same degree or magnitude as would be experienced by direct filtration systems. Inability to meet flow production targets may require PWB to augment supply with groundwater sources.

In addition, as this facility will likely have no sewer system to convey liquid wastes from residual solids dewatering processes, all liquid wastes need to be returned to the head of the facility for treatment. Direct filtration is not as protective as conventional filtration with respect to treating these recycle streams, which may convey harmful bacterial and protozoan microorganisms (e.g., *Cryptosporidium* and *Giardia*).

Conventional Filtration

A water treatment process that consists of rapid mix for coagulants, followed by flocculation, sedimentation, and filtration resulting in "substantial particulate removal" is identified as a conventional filtration facility (OAR 333-061-0020 59). A general process schematic illustrating the arrangements of conventional filtration is shown in Figure 6-8.

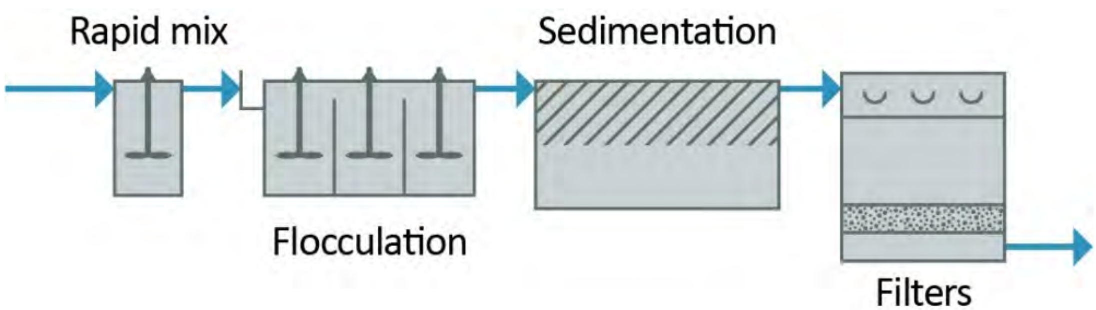


Figure 6-8. Process schematic for typical conventional filtration

Conventional filtration requires liquid stream solids separation between the flocculation and filtration processes. This sedimentation process provides many added benefits, especially for future raw water quality concerns; however, this process also increases facility construction cost. The benefits of sedimentation include:

- Providing operational reliability and flexibility due to increased detention time.
- Increasing the ability to treat turbidity and organics from most storms or fires without losing capacity.
- Managing risks from algae and algal toxins.
- Reducing levels of DBPs, manganese, and color.

The added 1 to 2 hours of detention time provided by sedimentation can 1) improve adsorption efficiency of dissolved or colloidal constituents onto floc particles, 2) provide more time for chemical oxidation, and 3) provide more time for operators to react to unexpected treatment challenges.

Direct and Conventional Filtration Comparison

During periods of increased turbidity or organics concentrations, sedimentation can help divert materials away from the filters by settling solids in the sedimentation basins. However, this typically requires increased coagulant dosing and generates additional residual solids.

During periods of elevated constituents in raw water, notably organics, temporary addition of PAC for adsorption is common when ozonation is not implemented. While it is also possible for direct filtration facilities to be supplemented in this manner, residual solids added to the process (i.e., PAC) will increase the particulate matter burden on the filters and potentially reduce filter productivity. A sedimentation process to settle spent PAC is suggested to mitigate the impact of dosing this additional treatment agent.

The filtration facility must provide a total of 3.0-log of removal or inactivation of *Giardia* and 4.0-log removal or inactivation of virus (OAR 333-061-0032 1aA; 0050 4cB and 0050 4cD). Some of these log credits are granted by the treatment technique itself, and some must be obtained via post-filtration disinfection. Table 6-10 illustrates the array of log credits available for direct and conventional filtration.

Table 6-10. Filtration Credits for Microbial Removal

Filtration Treatment Technology	Maximum Logs of Credit for Physical Removal			Minimum Logs of Inactivation Needed by Disinfection		
	Giardia	Virus	*Cryptosporidium*	*Giardia*	Virus	*Cryptosporidium*
Direct	2.0	1.0	2.0	1.0	3.0	0.0[a]
Conventional	2.5	2.0	2.0	0.5	2.0	0.0[a]

a. Assuming raw water Cryptosporidium bin concentrations are <0.075 oocysts/L

As shown in Table 6-10, twice as much *Giardia* disinfection inactivation credit is typically required for direct filtration as opposed to conventional filtration, with a 50 percent increase needed for virus inactivation. This translates directly to higher chlorine dosing or larger clearwells or disinfection contact basins for the direct filtration process. Note that this partially offsets the cost savings of constructing full-sized sedimentation basins, as direct filtration systems will require correspondingly larger clearwells. For project definition, it is assumed additional disinfection credits for pre-disinfection with ozone or chlorine are not included. Table 6-10 also indicates that a maximum of 2.0 logs of *Cryptosporidium* removal credit can be provided by the use of either direct or conventional filtration. If the raw water *Cryptosporidium* bin concentrations are 0.075 oocysts/L or greater, higher levels of *Cryptosporidium* removal or inactivation are required as specified in OAR 333-061-0032 4f.

The possibility of constructing direct filtration initially and designing for future sedimentation was examined in discussions with the TAC. There, it was mentioned that since this facility is intended to operate as a zero liquid discharge (ZLD) facility (no available sewers to convey liquid waste dewatering streams), there is some risk that direct filtration without ozone would be less protective against protozoan contamination (especially, *Cryptosporidium*) due to recycling the waste streams.

Based on input from the TAC, in the event of a high-turbidity event, the benefits of constructing some form of sedimentation outweigh the operational and treatment risks associated with delaying (for years) construction of a sedimentation basin.

Other relative comparisons can be made between conventional filtration and direct filtration systems with respect to operational risks. A summary comparison of conventional filtration and direct filtration is shown in Table 6-11.

Table 6-11. Conventional Filtration and Direct Filtration Summary Comparison

Feature	Direct Filtration	Conventional Filtration
Ability to handle elevated raw water turbidity and organics (e.g., T&O, DBP precursors, color)	Limited to turbidities of 20-40 NTU.	Well suited.
Ability to handle elevated algae and algal toxins	Limited benefit.	Provides added benefit.
Ability to oxidize and remove iron and manganese	Less benefit compared to conventional filtration.	Provides added benefit due to additional contact time with oxidizing agent.
Disinfection contact time	Requires larger clearwell.	Can use smaller clearwell.
Sensitivity to rapidly changing raw water or facility upset conditions (e.g., post-fire)	Limited benefit.	Provides added benefit.
Improve facility productivity (e.g., filter run time)	Limited benefit during periods of elevated raw water quality.	Added benefit during periods of elevated raw water quality.
Detention time buffer for chemical reactions and facility operations	No added detention time.	Approximately 1 or more added hours provided.
Compatibility with PAC	Limited compatibility.	Compatible.
Costs	Reduced capital and operating costs.	• Greater capital and operating costs. • Capital cost increases are offset by reduced clearwell size (disinfection).
Chemical use and solids production	Less than conventional filtration during periods of elevated raw water quality.	Increased during periods of elevated raw water quality.
Facility footprint	Smaller footprint.	Larger footprint.

6.5.2 Basic Sedimentation Alternatives

This section examines the following basic types of sedimentation alternatives:

- Low-rate Gravity Sedimentation
- High-rate Gravity Sedimentation
- Ballasted Sedimentation
- Dissolved Air Flotation (DAF)

Low-Rate Gravity Sedimentation

A traditional low-rate gravity sedimentation basin is typically conservatively designed to provide a large hydraulically tranquil (quiescent) zone to permit gravity settling of heavy, settleable floc particles. Sludge collected in the sedimentation basins is removed either manually or using mechanical devices (i.e., scrapers and vacuum suction devices) and is then pumped or delivered by gravity to residuals solids handling processes.

Typical design criteria for low-rate gravity types of basins are shown as follows (Sanks, 1980):

- **Surface overflow rate (SOR):** 0.3 to 1.0 gpm/square feet (sf)
- **Side water depth:** 10 to 20 feet
- **Detention time:** 1.5 to 4 hours

High-Rate Gravity Sedimentation

High-rate gravity sedimentation is commonly achieved by adding contact surface area to a sedimentation basin through use of tube settlers or Lamella settling plates. Based on current industry trends and the known durability of stainless-steel Lamella plates compared to plastic tube settlers, Lamella plates are assumed for project definition (Kawamura, 2000).

These tube settler or plate systems provide additional surface area to a sedimentation basin without increasing the overall basin dimensions. For Lamella plates, the projected surface area of a single plate is equal to the width multiplied by the length of the plate and the cosine of the angle of the plate from the horizontal. The total projected plate surface area required for each sedimentation basin is equal to the basin flow divided by the plate settler surface loading rate (SLR). Typically, plate settlers are not considered 100 percent effective in practice, and the SLR is de-rated using an efficiency factor between 80 to 90 percent. The selected design SLR and efficiency factor depends on water quality parameters such as temperature and turbidity.

Typical design criteria for high-rate gravity sedimentation basins with Lamella plates are as follows:

- **SOR:** 2.0 to 4.0 gpm/sf for light floc
- **SLR:** 0.25 to 0.35 gpm/sf with an efficiency factor of 80 to 90 percent, based on the projected area of the Lamella plates
- **Side water depth:** 15 to 20 feet
- **Detention time:** 15 to 25 minutes

Ballasted Sedimentation

Ballasted sedimentation (e.g., Actiflo) uses coagulant-coated sand particles to enhance solids flocculation and settleability (MWH, 2012). In ballasted sedimentation, the process stream is dosed with coagulants such as alum, ferric chloride, or ferrous sulfate, and directed to an injection tank. In the injection tank, polymers and fine silica sand are added under continuous mixing to a ballasted floc around the sand particles. The flow then enters a maturation tank where gentle stirring further enhances flocculation. From the maturation tank, the flow is directed to a settling tank where the ballasted floc is removed via plate settlers. The settled solids are then pumped through hydro-cyclones where the high-density sand is separated from the flocculated solids. The sand is then concentrated and recycled back into the head of the process, while the solids are disposed of as settled sludge.

Typical design criteria for ballasted sedimentation systems are as follows:

- **SOR:** 20 to 30 gpm/sf
- **Microsand feed:** 0.15 to 0.4 percent of influent flow rate

Although ballasted flocculation systems reduce facility footprint to a greater extent than high-rate gravity sedimentation, they are more mechanically complex and may not be as well suited as high-rate gravity sedimentation for removal of floc and colloidal material (e.g., originating from turbidity events). Ballasted flocculation systems rely on more polymer use, may generate more residual solids, and have life cycle costs equivalent to, and in some cases greater than, high-rate gravity sedimentation.

Dissolved Air Flotation

DAF units are an alternative to sedimentation, removing particulate matter at the top of the unit process as opposed to the bottom as with gravity sedimentation. With DAF, air is used to float solids in the flocculated water. The bubbles rise, attaching to solids particles in their path, and float to the water surface. The foam blanket that forms on the reactor surface is collected either hydraulically or by a skimming device that drags the floated solids over a beach plate to a collection trough. This trough conveys the “float” to a chamber from which the thickened float or sludge is discharged by gravity or pumped to downstream processing.

A portion of the DAF effluent is recycled (approximately 8 to 12 percent of the process flow stream) for dissolving and injecting air into the DAF process. The recycled flow gets pressurized and super-saturated with air and is then re-introduced to the full flow. This re-introduced water then releases the super-saturated air when pressure is released.

Polymers may be added to DAF influent to enhance particle flocculation. Polymers have a minimal impact on the DAF thickened solids concentration but improve the capture rate substantially.

DAF is most effective in solid-liquid separation applications involving 1) separation of low-density particulate matter such as algae, 2) supplies with moderate dissolved organic matter (natural color), 3) low to moderate turbidity waters that, after coagulation and flocculation, produce low density floc, and 4) low temperature waters. Some of these conditions (e.g., conditions 1 and 3), may be applicable to the Bull Run supply.

Typical design criteria for DAF systems are as follows:

- **SOR:** 10 to 16 gpm/sf
- **Rise rates:** 4 to 20 gpm/sf

Table 6-12 summarizes advantages and disadvantages of sedimentation options.

Table 6-12. Sedimentation Options Summary Considerations

Advantages	Disadvantages
Low-Rate Gravity Sedimentation	
• Mechanically simple and less expensive to operate than ballasted sedimentation or DAF. • System reacts quickly to rapid changes in raw water quality turbidity. • Suitable to handle large turbidity events up to 500 NTU and higher.	• Large footprint.
High-Rate Gravity Sedimentation	
• Mechanically simple and less expensive to operate than ballasted sedimentation or DAF. • System reacts quickly to rapid changes in raw water quality turbidity. • Suitable to handle large turbidity events up to 500 NTU and higher.	• More compact than low-rate gravity sedimentation.
Ballasted Sedimentation	
• Effective at removing material that tends to float (e.g., algae). • Small footprint; more compact than high-rate sedimentation. • System reacts quickly to rapid changes in raw water quality turbidity.	• Unable to handle high turbidity loads (e.g., turbidities >100–500 NTU resulting from a large fire). • Mechanically complex and expensive to operate (high energy cost component). • May yield more residual solids than other processes. • May not be as effective for high turbidity consisting of very small and light particulate matter.
DAF	
• Effective at removing material that tends to float (e.g., algae). • More compact than high-rate sedimentation.	• Unable to handle higher turbidity loads (e.g., turbidities >20 NTU). • Mechanically complex and expensive to operate (high energy cost component).

Based on information contained in Table 6-12, use of the high-rate gravity sedimentation process is assumed for project definition.

6.5.3 Full Sized and Hybrid Sedimentation Comparison

This section examines use of conventional filtration with full-sized sedimentation as compared to hybrid sedimentation, which uses smaller process basins.

This section includes the following discussions:

- Full Sedimentation (sized for all flows)
- Hybrid Sedimentation (sized for all but high summer season flows)

Full-Sized Sedimentation

The full-sized sedimentation process is assumed to include Lamella plate settlers that would be sized to provide effective sedimentation year-round, including the summer months. For this situation, the sedimentation basins are sized for the peak day flow of 160 mgd using the following typical design criteria:

- **SOR:** 3.0 gpm/sf based on the overall basin area
- **SLR:** 0.34 gpm/sf with an 80 percent efficiency factor, based on the projected area of the inclined plates
- Total active surface area: 38,900 sf (estimated)

Note that these design criteria are applied to the summer season peak flows where typical water temperatures are approximately 16°C.

Hybrid Sedimentation

A hybrid sedimentation process would allow operation as a conventional filtration facility with sedimentation during seasons when raw water turbidity is likely to be elevated, and operation as a direct filtration facility during seasons when raw water turbidity is low. This hybrid approach, used at Tacoma’s Green River Filtration Facility, reduces facility construction costs and can be used if raw water quality follows a predictable pattern.

When raw water quality follows historical behaviors, it may be possible to bypass sedimentation, operate in a direct filtration mode, and still achieve finished water quality and production goals, provided a separate flocculated water bypass channel is provided. If a separate bypass channel is not provided, flocculated water must flow through downstream hybrid sedimentation basins. Under this latter circumstance, there will be periods at high flow rates (typically summer months) where the flows through the hybrid sedimentation process would exceed the nominal design SOR and SLR values for full sedimentation. In this scenario and at these high flows, facility operations would be classified as direct filtration. If the facility is deemed to operate in a direct filtration mode via the means described above, a higher level of disinfection log removal credits in the clearwell will be necessary unless other means of disinfection credits are obtained. In this case, chemical coagulant dosing could be reduced commensurate with direct filtration operations and the sludge removal mechanisms could be turned off. This mode of operation is expected to reduce chemical consumption and residual solids production.

The hybrid sedimentation process assumes that summer season raw water quality will permit direct filtration and that sedimentation basins will be in use for the rest of the year. For this determination, winter season conditions will be used as the governing sizing condition for hybrid sedimentation, with peak winter season flows estimated at 97 mgd and typical water temperatures slightly below 5°C. As a result, the following typical design criteria are selected by a proportional adjustment of Stokes' Law (Kawamura, 2000):

- Hybrid sedimentation (non-summer) SOR: 2.2 gpm/sf based on the overall basin
- Hybrid sedimentation (non-summer) SLR: 0.25 gpm/sf with a 80 percent efficiency factor, based on the projected area of the inclined plates
- From this, the total active surface area of the hybrid sedimentation basins is estimated to be 32,200 sf

Although the hybrid sedimentation basins are sized for flows that are 40 percent less than the full-sized sedimentation process, the hybrid sedimentation basins are somewhat similarly sized and are only 17 percent smaller than the full-sized sedimentation basins. This unique ratio of capacity versus size is due principally to two factors 1) the relatively small ratio between projected peak summer day and peak winter day flows, 2) the lower unit sedimentation basin design loading criteria impacted by the relative narrow range of water temperatures. Cold water temperatures, and corresponding higher viscosities, reduce particle settling velocities and sedimentation process performance.

Hybrid sedimentation basins, sized for winter-time scenarios shown above, can still be used during the summer months. When used in the summer months, the hybrid sedimentation basins would operate at the unit loading conditions shown as follows:

- Hybrid sedimentation (summer) SOR: 3.6 gpm/sf
- Hybrid sedimentation (summer) SLR: 0.41 gpm/sf with 80 percent efficiency factor

Note the hybrid sedimentation summer hydraulic loading design rates are higher than the full sedimentation summer hydraulic loading design rates as a result of smaller hybrid basin sizing. OHA may determine that summer operations with hybrid sedimentation will still be designated as direct filtration. In that case, additional disinfection credits will be necessary as described above.

6.5.4 Sedimentation Summary

This section summarized the advantages and disadvantages of sedimentation alternatives to treat the Bull Run raw water quality. Filtration with sedimentation maximizes treatment benefits and provides a strong measure of public health protection.

- **Direct and Conventional Filtration Comparison:**
  - It is suggested to proceed with conventional filtration to treat the Bull Run supply and eliminate direct filtration from further consideration.
  - Sedimentation provides numerous benefits such as the ability to react to high turbidity, high organics episodes, and algal blooms caused by storm and fire events and potential climate change impacts.
  - Sedimentation provides an active barrier to control water quality concerns and help maintain filter productivity over a broad range of raw water quality changes.

- **Basic Sedimentation Alternatives:**
  - High-rate gravity sedimentation systems are generally more versatile at treating a range of raw water quality concerns especially because these systems offer longer contact time for processes such as oxidation and adsorption with PAC. As cited earlier, this longer contact time provides an added cushion during facility upset conditions.
  - High-rate gravity sedimentation systems with Lamella plates are assumed for project definition based on high reliability and flexibility, low operating cost, and being a proven technology.
- **Full-sized and Hybrid Sedimentation Comparison.** Two basic configurations, full versus hybrid sedimentation, are carried forward for subsequent unit process evaluation. A full-sized sedimentation system is assumed for project definition and cost development purposes.

Figure 6-9 depicts a high-rate sedimentation basin from the program team's tour of the Joint Water Commission's water treatment facility.

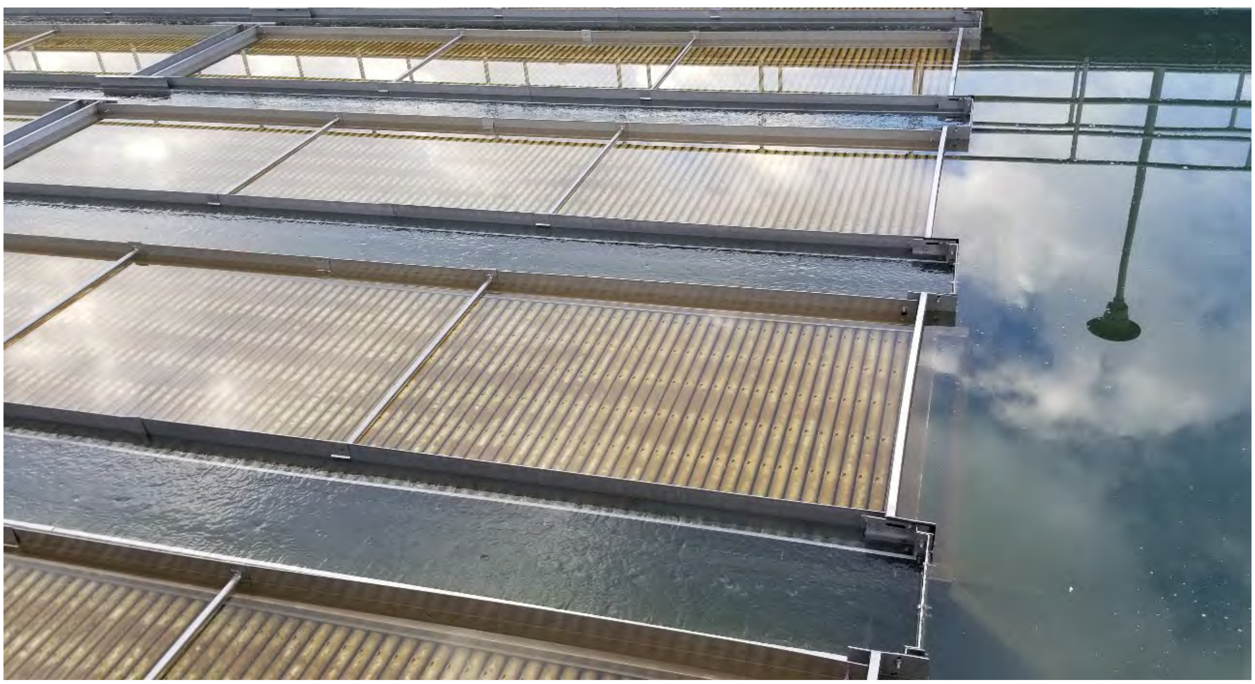
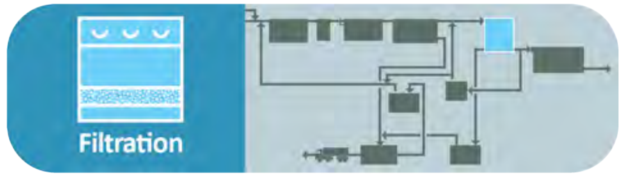


Figure 6-9. High-rate plate sedimentation basin at Joint Water Commission's facility

Hillsboro, Oregon

6.6 Filtration



The filtration process removes particulates naturally occurring in raw water and those formed during flocculation and sedimentation. Filtration also provides a critical barrier against transmission of waterborne diseases by physically removing microbials. The effectiveness of filtration is directly tied to the effectiveness of forming easily filterable floc particles through proper addition of coagulants, as well as the physical and hydraulic characteristics of the filters.

This section discusses the following topics:

- Filtration Alternatives
- Filtration Summary

PWB selected granular media filtration during pre-planning as described in Chapter 1: Introduction. Building on this initial decision, Table 6-13 presents a summary of important issues associated with filtration that were considered to arrive at project definition assumptions.

Table 6-13. Important Issues for Filtration

Important Issues	Description
Raw water quality and pre-treatment type	• Raw water quality and the type of pre-treatment will impact design and operation of a filtration system. • Important raw water quality factors affecting pre-treatment and filtration include turbidity, pH, alkalinity, temperature, and organics concentrations.
Filtered water quality	• Removal of particulates and other constituents are important for regulatory compliance and to monitor the effectiveness of filtration.
Filter productivity	• Overall filter performance and productivity is measured by the unit filtration run volume and the net efficiency of filtration when considering backwash and filter-to-waster volumes.
State regulatory guidelines for filter loading rate	• Filtration systems must be designed and operated to meet the governing regulations found in OAR 333-061-0032 4 and 5. • The filtration system design must be in keeping with OHA's accepted standard engineering references and guidance, such as the Great Lakes Upper Mississippi River *Recommended Standards for Water Works ("the ten states standards"),* which indicates "typical filtration rates are from 2–4 gpm/sf." • The pilot study for this project tests loading rates from 6–12 gpm/sf (supported by recent local experiences and OHA approval at similar facilities).
Filter washing system	• Air scour and hydraulic backwashing systems are needed.
Mode of filter operation	• Constant rate filtration will be the default, but other modes may be acceptable.

Table 6-13. Important Issues for Filtration

Important Issues	Description
Filter media design, including type, size, and media depth	The pilot plant study is testing several dual-media configurations (silica sand with anthracite and silica sand with GAC) as follows: • Silica sand: 12-inch depth, effective size ranging from 0.55–0.65 mm, depending on specific parings with anthracite effective size. • Anthracite: 48–60-inch depth, ranging from effective size 1.2–1.3 mm, depending on specific parings with silica sand effective size. • GAC: 48–60-inch depth, effective size 1.3 mm. • Media length/diameter (L/d) ratio: 1,530–1,730.
Chemical application points	• Filter aid will be dosed upstream of filtration.
Filter size and configuration	• Filters can be single, or bifurcated with two cells each to reduce instantaneous backwashing demands. • The number of filters and the total filter area should meet peak day flow demands at the maximum permitted unit filtration rate with one filter out of service (n+1).

6.6.1 Filtration Alternatives

The following applicable filtration topics and operational options are discussed below:

- Granular Media Filtration
- GAC Cap
- Biologically Active Filtration (BAF)

The primary difference between these media configurations is their ability to remove organic compounds via adsorption or biological oxidation. These configurations perform similarly with respect to physical particle removal via filtration. Other granular filtration technologies exist, such as slow sand filtration; however, this technology was ruled out during the pre-planning work in 2018.

Granular Media Filtration

Granular media filtration is the most commonly used filtration technology (MWH, 2005). The Portland City Council selected granular media filtration as the filtration technology in 2018 as described in Chapter 1. Typical design is dual media, which includes a layer of anthracite coal or GAC and a bottom layer of sand. Both anthracite and GAC media have a larger effective size than the sand media and thus allow floc particles to penetrate, serving as a particle storage layer. The sand has a smaller effective size (compared to anthracite or GAC) and serves as a final particle barrier. This filter configuration has a long and proven record of providing high-quality water with simple operation and maintenance.

For project definition, dual media is assumed. Specifics on media configuration and type (anthracite versus GAC) will be selected during design and after pilot plant study data becomes available. During design, pilot plant study results for these two types of media will be evaluated. Generally, GAC media requires a reduced unit flow for backwashing compared to anthracite media due to a lower specific gravity. However, GAC has less physical longevity (it is a more friable media than anthracite and is assumed to need replacement on a 7-year cycle to maintain an acceptable range of effective size) and a higher unit cost than anthracite.

Of particular interest for project definition is the maximum unit filtration rate and the depth of the media. This rate is used as the chief criterion for sizing the filters. As described in Table 6-5, typical filtration rates can range from 6 to 12 gpm/sf, with a proportional relationship between the filter size and the facility construction cost. The permissible maximum unit filtration rate will be determined by OHA based on the submitted pilot plant study report. For the purposes of project definition, a conservative value of 6 gpm/sf is selected. Also, for project definition, a deep bed, dual media configuration is assumed with silica sand on the bottom and anthracite or GAC media as the top layer. There is a growing interest in deep bed dual media filtration, which is defined as having at least 4 to 6 feet of media as the top layer (AWWA, 1999). Deep bed filters provide more media for increased particle removal and filter productivity between backwashes; however, this tends to increase the height and cost of the filter boxes. Various filter media configurations of depth and anthracite versus GAC media are being tested in the pilot plant study.

In addition to reducing turbidity by removing particulates and organic materials that are adsorbed or bonded to floc particles (including DBP precursor molecules), granular media filters can be designed to effectively manage particulate oxidized manganese dioxide and soluble manganese. As discussed in Chapter 3, manganese is present in the raw water and can exceed the secondary standard MCL on infrequent occasions, although 95 percent of the measured historical concentrations are below the MCL. The goal for manganese removal as stated in Chapter 3 is a finished water concentration equal to or less than half of the MCL. Dissolved manganese is naturally positively charged, and the filter media negatively charged, thus, under optimized operations, dissolved manganese may be attached or adsorbed onto the filter media. Particulate manganese (e.g., manganese dioxide) can also be directly removed by the filters.

GAC Cap

As a variation of the granular media options describe above, GAC filter caps are installed as the top layer (typically, atop an anthracite bed) and are designed to support the periodic removal of T&O compounds and seasonal organic carbon. Generally, this layer is on the order of a few inches deep or more and functions as an adsorber with a short empty bed contact time. This approach may be more effective in some circumstances than addition of PAC, but once the GAC cap is exhausted with respect to saturation adsorption of organics, it must be replaced. As the GAC cap is relatively small and contact times are short, it is expected that bed exhaustion as an adsorber would occur relatively quickly and replacement of the media or this adsorber function would be uneconomical. Use of a GAC cap is not suggested for the reasons cited in this section and Section 6.4; however, a GAC cap may be re-examined during design.

Biologically Active Filtration

BAF is the promotion of biological growth in the filter media to remove constituents via biological oxidation. There is increasing awareness and acceptance of BAF systems in water treatment as the organisms that colonize the filter media are not pathogenic and are easily inactivated with post filtration disinfection. BAF has been used for organic and inorganic degradation, metal oxidation, and nutrient removal, in addition to traditional particle removal through the filter media. In this mode of operation, disinfectant, or chlorine residual, is not present in the filter to allow naturally occurring biological growth. Often, quenching agents are used to remove disinfectant residuals in either the feed to the filters or to the water used for backwashing. Note that the absence of an oxidizing residual may hinder the formation of manganese dioxide precipitates, thus possibly reducing overall manganese removal.

GAC may perform better than anthracite with respect to biological growth and constituent removal in BAF systems because its microporous structure may provide more attachment surfaces for microbiological colonization (MWH, 2005); however, anthracite has also been shown to perform adequately in this regard. For project definition, it is assumed the filtration system will be designed so that future facility operations can be adjusted to BAF if desired.

An additional consideration for the use of BAF is based on the use of intermediate ozonation. When ozone is used for oxidation and destruction of T&O and other organic compounds, BAF may be implemented to reduce concentrations of ozonation byproducts, such as assimilable organic carbon that can fuel biofilm growth in the distribution system. Generally, systems with higher raw water organics levels may consider use of intermediate ozonation immediately downstream of sedimentation to reduce overall ozone consumption by relying on sedimentation for partial organics removal. In these cases, concentrations of these ozone byproducts may be higher. Ozone breaks down organic carbon into byproducts such as assimilable organic carbon which, if allowed to pass through filtration and into the distribution system, results in downstream biological re-growth. BAF can effectively reduce assimilable organic carbon in the filters, preventing this regrowth potential. If ozonation is not used, the need for BAF may be diminished.

Table 6-14 below summarizes advantages and disadvantages of granular filtration media configuration and operational alternatives, primarily relating to removal of organics.

Table 6-14. Filtration Media Configuration Options Summary Considerations	
Advantages	**Disadvantages**
Granular Media	
• Proven technology with a long track record of use at facilities worldwide. • Most commonly accepted form of filtration, especially for particulate removal. • Performance functions are well understood and can be optimized relatively easily. • Materials are generally quite durable. • Can promote biological filtration.	• Anthracite media may have less ability to adsorb or support biological activity to oxidize organics compared to full bed GAC media, GAC cap, or BAF operations.
Granular Activated Carbon Cap	
• Higher efficiency than PAC. • Provides DBP control by lowering natural organic matter (NOM) and TOC. • Excellent adsorption capacity. • Reactivation possible. • Provides adsorption barrier and filtration without significant capital costs, new construction, or impacts to filter head loss.	• Its adsorptive capacity is consumed continuously as opposed to PAC addition to periodically treat for specific T&O episodes, as an example. • To minimize potential filter head loss, GAC layer is typically only a few inches thick, resulting in small empty bed contact time and less efficient organics removal than a full-sized post-filtration GAC contactor. • Most effective for controlling T&O when organics episodes are predictable and short, and when a program for seasonal GAC cap installation can be implemented. • Once adsorptive capacity is reached, cap must be replaced with new material if remaining biological oxidation is inadequate to affect desired removals. This increases operating costs compared to media that is not being used for physical adsorption.
Biologically Active Filtration	
• Anthracite or GAC media can be effectively used. • Extends GAC bed life, if GAC is needed as an adsorber. • Can remove contaminants not otherwise removed by granular media operations described above. • Provides opportunity to support biological colonies to further oxidize background organic matter. • Loading rates are similar to conventional filtration.	• Operational challenges. • Efficiency varies with temperature and raw water concentrations of organics and nutrients. • Concentration of microorganisms can be higher in effluent than influent. • Run times can be lower than traditional media filtration due to increased head loss and filter loading from biomass. • Can only be achieved if ozone is part of the treatment process.

6.6.2 Filtration Summary

This section summarized considerations for filtration process alternatives considered to treat the Bull Run supply. For project definition, granular media filtration, as described above, is selected. Figure 6-10 depicts a cross section of a typical granular media filtration system.

Key considerations for project definition include:

- A dual media configuration is suggested, including silica sand and either anthracite or GAC media.
- For project definition, a maximum unit filtration rate of 6 gpm/sf is assumed.
- A total bed depth of 60 inches of media atop 12 inches of silica sand, an n+1 filter arrangement (one filter assumed to be out of service), and filter cleaning with air scour and hydraulic backwashing is also assumed for project definition.
- Additional validation of filter media configuration will be forthcoming from the ongoing pilot study.
- It is suggested to include provisions that allow the filters to operate as biologically active filters, should that mode of operation be advantageous in the future. These provisions could include piping flexibility and/or quenching agent feed points to ensure disinfectant residuals can be managed to low levels for both filter influent flows and backwash water.

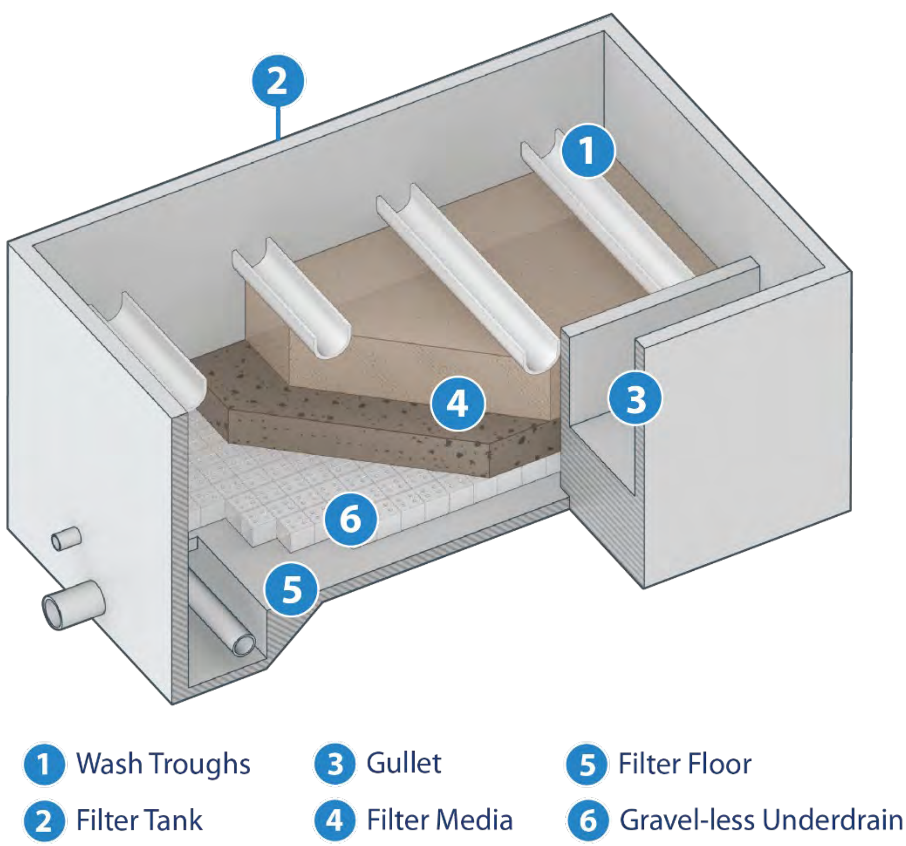
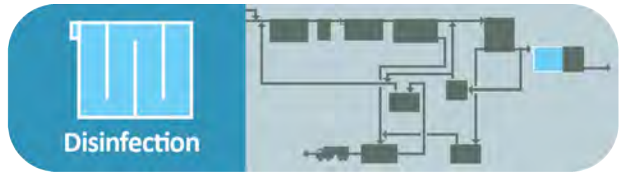


Figure 6-10. Example granular media filtration system

6.7 Disinfection

The disinfection process is required at the filtration facility to achieve microbial inactivation requirements specified in EPA and OHA drinking water regulations.



Disinfection will consist of 1) a primary step upstream of the filters or immediately post filtration as water enters the clearwell and 2) a secondary step for residual maintenance in the distribution system.

Aside from pre-oxidizing particles and organic matter and possibly improving coagulation and filtration processes, disinfection chemicals must be used to achieve regulated levels of primary disinfection (generally, applied post-filtration) and secondary disinfection residuals.

This section discusses the following topics:

- Regulatory and Finished Water Criteria and Goals
- Primary Disinfection
- Secondary Disinfection
- Disinfection Summary

Note that this section describes the nature of disinfection with a focus on disinfectant agents. A discussion of the location where disinfection is applied is in Section 6.8 Clearwell.

6.7.1 Regulatory and Finished Water Criteria and Goals

Table 6-15 below presents a summary of important issues associated with disinfection that were considered to arrive at project definition assumptions. These items are also identified in OAR 333-061-0032.

Table 6-15. Important Issues for Regulated Microorganisms and DBPs							
EPA Regulation	**Compliance Timeline**	**Regulatory Requirements**					
		Virus	***Giardia***	***Cryptosporidium***	**TTHMs**	**HAA5s**	**Bromate**
Surface Water Treatment Rule (SWTR)	1993	99.99% Removal/ Inactivation (4.0-log)	99.9% Removal/ Inactivation (3.0-log)	N/A	N/A	N/A	N/A
Interim Enhanced SWTR	2002	N/A	N/A	99% inactivation (2.0-log)	N/A	N/A	N/A
Stage 1 Disinfectants and DBPs Rule (DBPR)	2002	N/A	N/A	N/A	80 µg/L[a] (RAA[b])	60 µg/L[a] (RAA[b])	10 µg/L (RAA[b])
Stage 2 DBPR	2012	N/A	N/A	N/A	80 µg/L[a] (LRAA[c])	60 µg/L[a] (LRAA[c])	N/A
Long-Term 2 Enhanced SWTR (LT2 Rule)	2012	N/A	N/A	No additional treatment assumed beyond filtration based on Bin 1[d] status.	N/A	N/A	N/A

a. *Finished water goal per Chapter 3 are half of the values shown.*

b. *Running annual average (RAA).*

c. *Locational running annual average (LRAA).*

d. *The current historical mean value for Cryptosporidium is 0.0007 oocysts/L. The current threshold for unfiltered systems requires 2.0-log Cryptosporidium inactivation if the mean count is 0.01 oocysts/L or greater per OAR 333-061-0032 3.*

Table 6-16 presents a summary of the different bin classifications for *Cryptosporidium*, and the required additional treatment beyond compliance with existing regulations.

Table 6-16. Important Issues for *Cryptosporidium* Removal[a]					
Cryptosporidium **Concentration (oocysts/L)**	**Bin Classification**	**Additional Treatment: Conventional Filtration**	**Additional Treatment: Direct Filtration**	**Additional Treatment: Slow Sand or Diatomaceous Earth Filtration**	**Additional Treatment: Alternative Filtration Technology**
<0.075[b]	1	No additional treatment	No additional treatment	No additional treatment	No additional treatment
≥0.075 and <1.0	2	1.0-log treatment	1.5-log treatment	1.0-log treatment	As determined by the state
≥1.0 and <3.0	3	2.0-log treatment	2.5-log treatment	2.0-log treatment	As determined by the state
≥3.0	4	2.5-log treatment	3.0-log treatment	2.5-log treatment	As determined by the state

a. *OAR 333-061-0032 4gA.*

b. *The current historical average value for raw water Cryptosporidium is 0.0007 oocysts/L (Chapter 3).*

Based on the EPA and OHA regulations presented above, Table 6-17 summarizes the regulatory requirements for inactivation and removal of viruses, *Giardia,* and *Cryptosporidium* for the filtration facility. Based on *Cryptosporidium* sampling results from the Bull Run supply, PWB is in Bin 1.

Table 6-17. Key Criteria for Regulated Microorganism Control

Criteria	Virus	*Giardia*	*Cryptosporidium*
Total required removal/inactivation.	4.0-log	3.0-log	2.0-log
Maximum removal credits provided for conventional filtration.	2.0-log	2.5-log	2.0-log credit for Bin 1
Maximum removal credits provided for direct filtration.	1.0-log	2.0-log	2.0-log credit for Bin 1
Additional disinfection inactivation needed after conventional filtration credits.	2.0-log	0.5-log	0-log for Bin 1
Additional disinfection inactivation needed after direct filtration credits.	3.0-log	1.0-log	0-log for Bin 1

As PWB is assigned to the Bin 1 condition for C*ryptosporidium* (<0.075 oocysts/L), per OAR 333-0061-0032 4g, the filtration facility will achieve the necessary 2.0-log C*ryptosporidium* removal credit, and no additional inactivation will be required by disinfection. Although considered unlikely based on historical raw water quality data, if the filtration facility is ever required to achieve additional C*ryptosporidium* treatment, it would either need to come from ozone disinfection or the application of other microbial toolbox techniques described in the LT2 Rule, such as increasing watershed controls, adding pre-treatment, enhancing treatment, or adding or improving post-treatment. The specific values of additional credits that may be required in the future via the microbial toolbox are described in OAR 333-0061-0032 12.

6.7.2 Primary Disinfection (pre- or immediately post-filtration)

Primary disinfection kills or inactivates bacteria, viruses, and other potentially harmful organisms to prevent diseases such as typhoid fever, hepatitis, and cholera. Some disinfectants are more effective than others at inactivating certain, potentially harmful, organisms. Primary disinfection of Bull Run water is currently achieved by contact with free chlorine at Headworks and is designed to achieve necessary disinfection log inactivation credits for *Giardia* and viruses.

Historically, free chlorine has been widely used as a primary disinfectant, but some utilities have been moving away from its use due to the formation of DBPs. Additionally, chlorination and chloramination are not effective at inactivating *Cryptosporidium*. Other commonly used primary disinfectants include ozone and chlorine dioxide, which can be effective at inactivating *Cryptosporidium*. The following section considers primary disinfectant alternatives, including:

- Free Chlorine Disinfection
- Ozone Disinfection
- Chloramine Disinfection
- Chlorine Dioxide Disinfection

Note that these chemicals can also serve as chemical oxidants as described in Section 6.1.

Free Chlorine Disinfection

Free chlorine disinfection using chlorine gas or sodium hypochlorite is the most traditional approach for water treatment facilities to provide pathogen disinfection (MWH, 2005). Free chlorine is known as an excellent disinfectant, but under conditions of high concentrations of dissolved NOM, halogenated DBPs can form that may exceed regulatory standards. Note that the historical water quality data presented in Chapter 3 indicates the existing raw water is low in organic compounds, and DBP formation potential with free chlorine is expected to be lower than the goals set forth for the filtration facility. For project definition, bulk delivery of 12.5 percent strength (equivalent Cl_2 by weight) sodium hypochlorite is assumed, with consideration of onsite generation systems suggested during design. Onsite generation is based on electrolysis of a brine solution to create a sodium hypochlorite solution that is typically 0.8 percent strength (equivalent Cl_2 by weight) for onsite uses. Salt is delivered to the site as a precursor to making a brine solution, and electrical power is needed to generate sodium hypochlorite.

For free chlorine disinfection, the CT requirement is governed by *Giardia* inactivation over that for virus inactivation. The CT typically required for a 0.5-log *Giardia* inactivation credit after conventional filtration (Table 6-17) ranges from approximately 13 to 39 mg-min/L for the temperature ranges shown in Table 3-1 (assuming a typical pH of 7.5 and a free chlorine residual of 1.2 mg/L).

Ozone Disinfection

Ozone is commonly used in drinking water treatment for disinfection as well as for oxidation benefits described previously. Ozone is a strong oxidant that can inactivate microorganisms, including *Cryptosporidium, Giardia,* and viruses, and does not form regulated halogenated DBPs. Ozone must be generated on site using oxygen-fed, or less efficient air-fed, ozone generators.

Although EPA disinfection credit requirements do not specify where disinfection is achieved, the Oregon construction standard (OAR 333-061-0050 4cD) requires disinfection credit occur after the filtration process for a conventional filtration facility. An active group of Oregon drinking water utilities has been working with OHA to define methods in which water treatment facilities can possibly be awarded pre-filtration disinfection credit with ozone. In October 2019, OHA approved City of Wilsonville with an intermediate ozone contact time credit under a waiver that includes a requirement for covered filters. The concept of covering filters to qualify for additional ozonation disinfection credits is to ensure that once ozone disinfection is provided, water downstream from this disinfection process cannot be re-contaminated by external sources. If intermediate ozonation is practiced at the filtration facility, then covering the downstream filters would be consistent with the OHA approval of the City of Wilsonville's intermediate ozonation treatment system. Note that the use of pre-ozonation to gain ozonation disinfection credits would likely require covering downstream processes (e.g., rapid mix, flocculation, sedimentation, and filtration).

For ozone disinfection, the CT requirement is governed by virus inactivation over that for *Giardia* inactivation. The CT typically required for a 2.0-log virus inactivation credit after conventional filtration (Table 6-17) ranges from approximately 0.26 to 0.83 mg-min/L for the temperature ranges shown in Table 3-1.

It may be possible with OHA approval to partially substitute ozone for free chlorine at the filtration facility. Due to the complexity of an ozone system exclusively used for primary disinfection, provisions should be included to allow free chlorine to perform this same function, should the ozone system be off line.

Chloramine Disinfection

Chloramines are disinfectants that are produced by combining free chlorine (or hypochlorite) with ammonia to produce a long-lasting, stable disinfectant residual. Three forms of chloramines can be produced, including monochloramine, dichloramine, and trichloramine, depending on the relative concentrations of ammonia and chlorine, and the pH and temperature of the water. Monochloramine is preferred for disinfection purposes. Chloramines are often used in situations where high concentrations of dissolved NOM are present or where utilities have a concern of violating the Stage 2 DBPR, as chloramines produce far fewer halogenated DBPs than free chlorine. Again, this situation is not likely for the Bull Run supply. If chloramines are considered as a primary disinfectant for pathogens, long contact times are required.

For chloramine disinfection, the CT requirement is governed by virus inactivation over that for *Giardia* inactivation. The CT typically required for a 2.0-log virus inactivation credit after conventional filtration (Table 6-17) ranges from approximately 342 to 1,147 mg-min/L for the temperature ranges shown in Table 3-1 (assuming a typical pH between 6.0 to 9.0).

Chlorine Dioxide Disinfection

Chlorine dioxide is a synthetic, yellow- to brown-colored gas when at room temperature and atmospheric pressure. It has been used as an alternative disinfectant in drinking water treatment to control DBPs. Chlorine dioxide is generated on site through the combination of chlorine gas and sodium chlorite. Additionally, if gaseous chlorine is not used, sodium hypochlorite can be mixed with an acid, such as hydrochloric acid, to form gaseous chlorine and then mixed with sodium chlorite. Note the numerous water quality and safety concerns associated with chlorine dioxide described in Section 6.1.1.

For chlorine dioxide disinfection, the CT requirement is governed by *Giardia* inactivation over that for virus inactivation. The CT typically required for a 0.5-log *Giardia* inactivation credit after conventional filtration (Table 6-17) ranges from approximately 2.8 to 8.6 mg-min/L for the temperature ranges shown in Table 3-1 (assuming a typical pH between 6.0 to 9.0).

Table 6-18 below presents general information on the advantages and disadvantages of each type of primary disinfectant.

Table 6-18. Primary Disinfectant Options Summary Considerations	
Advantages	**Disadvantages**
Free Chlorine	
• Effectively inactivates viruses and *Giardia.* • Provides a residual that is easily measured and controlled. • Economical compared to other disinfectants. • Extensive track record of use. • Can oxidize some cyanotoxins. • Effective for manganese oxidation.	• High reactivity with naturally occurring organic and inorganic compounds to produce halogenated DBPs. • Safety hazards associated with transporting and handling gaseous chlorine. Hazards mitigated with the use of sodium hypochlorite. • Potential T&O events if overdosed, but easily identified and corrected.
Ozone	
• Also used for T&O control. • Effectively inactivates *Cryptosporidium*, *Giardia*, and viruses. • May be granted disinfection credits if used for chemical oxidation upstream of filtration. • Effective for oxidation of most cyanotoxins. • Can be effective for manganese oxidation. • Provides microflocculation benefit if added upstream of flocculation.	• Highly reactive, does not maintain a residual over time. • Requires an additional disinfectant that will maintain a residual in the distribution system. • Potential formation of bromate and other brominated DBPs; however, unlikely due to low bromine concentrations in raw water supply. • Requires significant capital expenditures for onsite ozone generation and ozone contact basin. • OHA approval for ozone disinfection credit ahead of filtration (where it would benefit organics removal) is anticipated to be allowed by waiver application. • A means to dose free chlorine is still required to maintain either a free chlorine or a chloramine detectable residual in the distribution system.
Chloramine	
• Most stable distribution system residual • Little reactivity with NOM, which is the precursor for halogenated DBPs.	• Increased operational complexity of controlling chlorine to ammonia ratio. • Potential for nitrification. • Potential formation of dichloramine and trichloramine that can cause aesthetic issues in distribution system. • Not commonly used for primary disinfection owing to long CT requirements for *Giardia* and virus inactivation. • Not effective for oxidation of cyanotoxins or manganese.
Chlorine Dioxide	
• Little reactivity with NOM, which is the precursor for halogenated DBPs. • Effectively inactivates *Cryptosporidium*, *Giardia*, and viruses. • Effective for oxidation of manganese.	• Residual dissipates quickly. • Requires two chemicals (typically, chlorine and sodium chlorite, or sodium hypochlorite with hydrochloric acid). • Potential to form chlorite, a DBP regulated by the EPA at a concentration of 1 mg/L. • Generates chlorate ions in finished water, a water quality health advisory concern not yet regulated by OHA. • Not effective for oxidation of cyanotoxins. • A means to dose free chlorine is still required to prepare chlorine dioxide and maintain either a free chlorine or a chloramine residual in the distribution system.

From the information listed in Table 6-18, the primary disinfectant assumed for project definition is free chlorine, although further investigation during design is suggested into the use of ozone to obtain partial primary disinfection credits. Other primary disinfectants options are either more costly, introduce operational and water quality difficulties, or may be difficult to permit.

6.7.3 Secondary Disinfection (at the distribution system entrance)

For secondary disinfection used to maintain a disinfectant residual throughout the distribution system, PWB has examined the option of remaining with chloramines or potentially switching to a free chlorine residual. The following section summarizes these disinfection application options, along with common disinfectants used for each purpose.

Secondary disinfection is required to provide adequate public health protection for water in the distribution system as well as assist in meeting multiple regulatory requirements. Secondary disinfection provides disinfection from the facility through the distribution system to customers' taps. OHA requires that a detectable chlorine residual is maintained and recorded throughout the distribution system (OAR 333-061-0036) and a disinfectant residual entering the distribution system be maintained at greater than 0.2 mg/L at all times (OAR 333-061-0040). Per this regulatory criteria, free chlorine and chloramines are the only disinfectants allowed for secondary disinfection. When properly implemented, free chlorine and chloramines will remain in the distribution system providing the final barrier for public health. Chloramines (specifically, monochloramine) have long-lasting properties that are desirable in secondary disinfection.

Following the implementation of filtration, it may become easier to achieve a detectable residual with either chlorine or chloramines, due to the reduced chlorine demand from the upstream removal of dissolved and particulate inorganic and organic material. Filtration, however, will not necessarily diminish chlorine demand inherent to the nature and condition of distribution system piping. Moreover, filtration is expected to reduce halogenated DBP precursors prior to the introduction of free chlorine for disinfection.

As PWB has used chloramines successfully for many decades, a change to use free chlorine as a new secondary disinfectant is not warranted and may be problematic. There are concerns associated with switching the secondary disinfectant residual related to existing pipe scale stability, T&O control, DBP formation, maintaining a free chlorine residual in the predominantly unlined cast iron piping in the system, and other associated water quality impacts. Changing to free chlorine would also impact groundwater facilities (which currently use chloramines) and require chlorine booster stations, thereby requiring additional system modifications.

A brief summary of benefits and risks in converting the distribution system from chloramines to a free chlorine residual is provided in Table 6-19 below.

Table 6-19. Benefits and Risks of Switching to Free Chlorine for PWB's Distribution System

Advantages	Disadvantages
• Simplified process with one less chemical to add and control.	• Potentially increases halogenated DBPs.
• Decreased chance of nitrosamine formation (a non-regulated but potential carcinogen that may be formed from amine-based functional groups associated with chloramines or with amine-based coagulants and coagulant aids).	• Initial challenge of converting to chlorine may cause difficult-to-manage breakpoint chlorine reactions in the system.
• Improved metals stability in the distribution system with a given finished water pH and alkalinity adjustment strategy (e.g., potentially lower lead levels). • Formation of Pb(IV) scale is less soluble than typical Pb(III).	• Temporary equilibration phase throughout distribution system may increase metals concentrations.
• Improved compatibility with some regional supplies	• Free chlorine residual may give an objectionable, chlorinous T&O to delivered water (depending on residual level).
• As chlorine is a more powerful disinfectant than chloramines, less contact time is required for control of microorganisms.	• Residual chlorine levels may require strategic boosting throughout the distribution system to maintain appropriate residual levels throughout the distribution system and may require additional process control measures at unstaffed booster stations and other infrastructure improvements.
• No nitrification events.	• Potential impacts on *Legionella* control regionally (some studies suggest chloramines offer better control in building systems).

From the information contained in Table 6-19, the secondary disinfectant selected for further project definition is chloramination, which is supported by guidance obtained during the TAC workshops. This is PWB's longstanding practice and it would lessen the number of new impacts and changes experienced by customers when the filtration facility becomes operational. Moreover, chloramination mitigates DBP formation potential and maintains a long-lasting residual with fewer chlorine booster stations.

Additionally, the cost of an ammonia feed system is relatively low compared to the rest of the project. PWB can cease use of ammonia feed to revert to a free chlorine system in the future if desired, after securing regulatory approvals and making necessary modifications to distribution sampling and monitoring protocols for microbials and lead/copper.

6.7.4 Disinfection Summary

This section described the advantages and disadvantages of primary disinfection alternatives for use to treat the raw water quality characteristics of the Bull Run supply. This section also summarized the current practices for secondary disinfection.

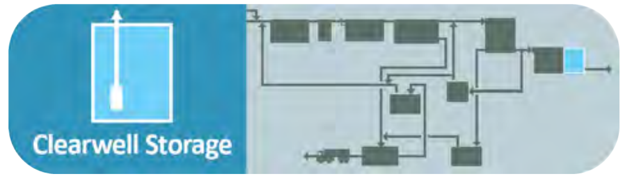
Key considerations for project definition include:

- **Regulatory and Finished Water Criteria and Goals:**
  - PWB is currently assigned to the Bin 1 condition for C*ryptosporidium* (<0.075 oocysts/L), per OAR 333-0061-0032 4g. The filtration facility will achieve the necessary 2.0-log C*ryptosporidium* removal credit. No additional inactivation will be required by disinfection.
- **Primary Disinfection:**
  - For project definition, chlorine and ozone are the two most likely candidates for primary disinfection at the filtration facility.
  - Use of sodium hypochlorite addition for free chlorine primary disinfection is assumed for project definition.
  - Further investigation during design is suggested into the use of ozone to obtain partial primary disinfection credits. It is suggested that PWB consider the costs and benefits of using pre-filtration ozonation as a primary disinfectant (either in whole or in part in combination with post-filtration chlorine), and meet with OHA to determine the operational requirements necessary to receive a waiver for ozone disinfection.
- **Secondary Disinfection:**
  - There is no clear reason to switch to free chlorine for secondary disinfection at this time. This is especially true if it is determined to use ozone as a primary disinfectant (to achieve the maximum benefit in DBP reduction).
  - Chloramination is assumed as the secondary disinfectant for project definition.

Potential next steps to evaluate these issues could include:

- DBP issues could be examined during the pilot plant study to determine if there would be a change in TTHMs and HAA5s if the system were to be switched from chloramines to free chlorine in the future. DBP testing will be evaluated for chloraminated systems. PWB may wish to perform DBP testing with free chlorine in a separate future study.
- Corrosion control testing will evaluate chloraminated systems initially. PWB may wish to perform corrosion control testing with free chlorine in a separate future study.

6.8 Clearwell



The term clearwell as used in this report refers to a large structure sized to handle disinfection, operational, and emergency storage.

The clearwell serves two primary functions 1) chemical and 2) hydraulic. The entrance to the clearwell provides a location to add a disinfectant to initiate primary disinfection. At the entrance, there may be a dedicated and baffled primary disinfection basin, separate from the rest of the clearwell, or the clearwell may be designed as a singular basin to accommodate both disinfection and hydraulic storage. After primary disinfection is complete, the remaining portion of the clearwell can be used for secondary disinfection chemical addition (i.e., ammonia). A clearwell also provides local operational storage to decouple facility flow from distribution system demand to provide stable operations. Ideally, facility flow changes are limited to one or two per day with the clearwell absorbing instantaneous flow changes such as peak-hour demand. Another potential function of a clearwell is to provide water storage for ancillary processes, including backwash source water and facility process water.

A clearwell and a finished water reservoir can be the same structure or two separate structures. A clearwell is typically a smaller structure sized for primary disinfection and minor operational volume like filter backwashes and facility process water (water used at the filtration facility for domestic use, chemical dilution, or wash down) as described above. A finished water reservoir is typically larger as it is sized for operational storage for distribution system equalization and for emergency storage. For project definition, it is assumed that the existing Powell Butte reservoirs and other related storage facilities provide adequate volume for traditional finished water reservoirs for PWB. The storage component of the new clearwell will provide localized storage as described above, including adequate volume to serve the nominal storage fluctuation needs associated with water demands for customers between the filtration facility and the Powell Butte reservoirs identified in Chapter 2: Existing Water System.

Clearwell storage provides the contact time needed to achieve disinfection requirements prior to water entering the distribution system. This function is typically provided in a baffled basin to minimize hydraulic short circuiting. The disinfection contact portion of the clearwell can be constructed as a basin adjacent to the storage basin, a baffled area internal to the clearwell, or accounted for in the design of the entire active volume of the clearwell. The various geometries of optimal clearwell configurations will be examined during design. For project definition, the clearwell will be described as a large single basin, with a baffled inlet zone dedicated to primary disinfection. This baffled inlet zone is assumed to include an exit weir hydraulically separating the disinfection section from the fluctuating storage section of the clearwell, ensuring a fixed contact volume for primary disinfection.

This section discusses the following topics:

- Clearwell Sizing
- Clearwell Summary

Table 6-20 presents a summary of important issues associated with clearwell storage that were considered to arrive at project definition assumptions.

Table 6-20. Important Issues for Clearwell

Important Issues	Description
Purpose for clearwell	• Effective disinfection contact time for *Giardia* and virus inactivation
Construction materials	• Reinforced concrete
Mixing and detention time	• Necessary contact time, adjusted for basin baffling factors, as a function of pH, chlorine residual, and temperature
Operability considerations	• Provide dual basins for redundancy and ease of maintenance

6.8.1 Clearwell Sizing

The literature suggests that a clearwell include a minimum of two cells for maintenance, with a minimum volume of 10 to 15 percent of facility capacity (Kawamura, 2000). For project definition, an initial volume of 10 percent, or 16 MG for a 160 mgd facility, is assumed. The following discussion identifies potential benefits for other clearwell volumes that can be further considered during design.

There are four primary functions to consider when sizing a clearwell:

1. Disinfection volume (residual disinfectant concentration multiplied by time)
2. Operational storage
3. Emergency storage
4. Fire flow

When PWB provides water to wholesale customers, storage for fire flow to those customers is furnished by the wholesale provider. Powell Butte Reservoir and other distribution system reservoirs provide storage for fire flow for PWB's retail connections. For project definition, it is assumed the fire flow requirements at the filtration facility are relatively minor. Contingency has been included in the preliminary selection of the clearwell volumes, and this will be further evaluated by the designer. The clearwell volume is generally determined by selecting the largest resulting volume needed from:

- Adding volume factors 1 (disinfection) + 2 (operational storage)
- Volume factor 3 (emergency storage)

Disinfection Volume

Required disinfection is a function of the overall treatment process. Per OAR 333-061-0032-1, the filtration facility will need to provide 3.0-log removal for *Giardia* and 4.0-log removal for viruses. The pre-treatment process selected (i.e., rapid mix, flocculation, and sedimentation) will influence the amount of inactivation required by disinfection—direct filtration will provide 2.0-log and 1.0-log removal, respectively, and conventional filtration with sedimentation will provide 2.5-log and 2.0-log removal, respectively. The remainder must be achieved with disinfection within a clearwell.

Virus inactivation is easier to accomplish than *Giardia* inactivation, so even though log-removal requirements are higher for viruses, *Giardia* typically governs. In this instance, the requirements for primary disinfection in the clearwell are as follows:

- Direct Filtration: 1.0-log inactivation, *Giardia*
- Conventional Filtration: 0.5-log inactivation, *Giardia*

Actual disinfection is a function of dose and residual, flow, temperature, and pH. Table 6-21 shows disinfection volumes assuming a 2.0 mg/L free chlorine residual leaving the clearwell, monthly flow patterns scaled to a July peak flow of 160 mgd (January flow of 100 mgd) and applied historical temperature and pH values. Current free chlorine residuals at Lusted Hill are controlled to 2.5 mg/L in the summer and 2.2 mg/L in the winter; however, a lower value is assumed for project definition owing to the expected reduction in chlorine demand due to filtration. Note that disinfection requirements are easier to meet at higher temperature and lower pH, so pH adjustment for corrosion control is assumed to occur after disinfection credit has been met to minimize the required disinfection volume. Also included in the table are the range of baffling factors for the clearwell, or T_{10}/T where:

- T = time, measured in minutes, that the water is in contact with the disinfectant
- T_{10} = detention time at which 90 percent of the water passing through the unit is retained
- Baffling Factor = T_{10}/T ratio

Table 6-21. Volume Requirements to Meet Disinfection Year-round[a]

T_{10}/T	Conventional 0.5-log *Giardia* Volume (MG)	Hybrid 1.0-log *Giardia* Volume (MG)
0.15	8	16
0.3	4	8
0.5	2.5	5
0.7	2	3.5

a. Winter conditions govern the sizing of the volume needed for disinfection assuming a flow of 100 mgd, and an historical typical temperature of 40℃, and a pH of 7.1.

Assuming the clearwell is designed with internal baffling meeting a T_{10}/T of at least 0.5, the required disinfection volume is 5 MG for a hybrid treatment option. Smaller volumes can be obtained if the design is based on a recommended value T_{10}/T of 0.7 (Kawamura, 2000).

A hybrid sedimentation treatment option would have smaller sedimentation basins sized for peak day non-summer flows. However, preliminary discussions with OHA indicate they will require disinfection reporting based on the mode in which the facility is operating at the time of the sample. With a hybrid sedimentation basin operating in a direct filtration mode during the summer season, the increased CT requirements for disinfection will require either a larger contact disinfection volume, or use of a higher chlorine dose.

Operational Storage

The spatial distribution of demand along the distribution system will have a direct effect on the demand pattern at the clearwell. Demand fluctuations that result in large facility flow changes will require correspondingly larger storage volumes in the clearwell. The two 50 MG reservoirs at Powell Butte and the more than 200 MG of storage within the distribution system will buffer these effects at the filtration facility because more than 95 percent of system demand is downstream of Powell Butte. Future development could change this, but with the exception of Sandy, land east of Gresham is outside of the urban growth boundary and will likely not see significant growth in the near future.

Customers upstream of Gresham (City of Sandy, Lusted Water District, Pleasant Home Water District, and other smaller customers) draw approximately 4.5 mgd to meet average demand (Table 6-22). The conduits from Headworks to Powell Butte operate with open-channel flow, and the conduits between the filtration facility and Powell Butte will also operate with open-channel flow, which effectively buffers demand changes at the clearwell. Flow from the filtration facility to Powell Butte will be large in comparison to the 4.5 mgd of demand between the filtration facility and Powell Butte, allowing the conduits to serve as equalization for operational storage.

Table 6-22. Average Daily Use for Customers between Powell Butte and the Site

Customer/Location	Use
City of Sandy	3.0 MG
Lusted Water District	0.4 MG
Pleasant Home Water District	0.4 MG
Miscellaneous	0.7 MG
Total	**4.5 MG**

Open-channel flow and the Powell Butte reservoirs reduce the need for distribution system operational storage volume at the filtration facility to a virtually negligible amount. Facility operations, like utility water systems and backwashes, will be supplied from the clearwell and will exert intermittent demand.

If a separate backwash tank is not part of the final design, the clearwell needs to retain at least 1.5 MG for the backwash water volume, assuming two backwashes to start up the facility after a shutdown. Note that the operational volume required for backwashing is not additive to the required disinfection volume. This is because the use of 1.5 MG of volume would diminish finished flow in the clearwell, negating the need for additional flow proportional contact time.

Emergency Storage

Emergency storage is needed to meet demands when sources are interrupted by unexpected events (i.e., emergency facility shutdown, power outages, or equipment failures). As an example, Jordan Valley Water Conservancy District, a large water district in Utah, set a service goal of standby storage equal to 12 hours of peak day demand. Twelve hours is a common goal as it allows operators time to make repairs or changes without disrupting service. The greater the storage volume, the more time operators have to resolve an emergency situation. Based on Table 6-22, a 12-hour emergency storage goal is approximately 2.25 MG (half the total daily value shown in the table).

6.8.2 Clearwell Summary

This section summarized the sizing considerations for the clearwell process at the filtration facility. For project definition, the assumed clearwell size is 10 percent of facility capacity, or 16 MG, to provide conservatism and flexibility for operational and emergency situations. The minimum required clearwell volume is determined by selecting the largest resulting volume needed from:

- Adding volume factors 1 (disinfection) + 2 (operational storage) = 5 MG
- Volume of factor 3 (emergency storage) = 2.25 MG

Because disinfection volume governs, the filtration facility should have two 5 MG cells so that one cell can be offline for maintenance while the other cell allows the filtration facility to continue meeting disinfection requirements. This results in a minimum required volume of 10 MG, divided into two cells or separate structures; therefore, the assumed volume of 16 MG is sufficient (with two 8 MG cells), and additional storage is not recommended.

The clearwell will need an outlet weir to retain the required disinfection volume at all times (at least for the disinfection contact portion of the clearwell), with a lower outlet gate or valve to use the volume as emergency storage. Figure 6-11 depicts a typical concept for a clearwell configuration.

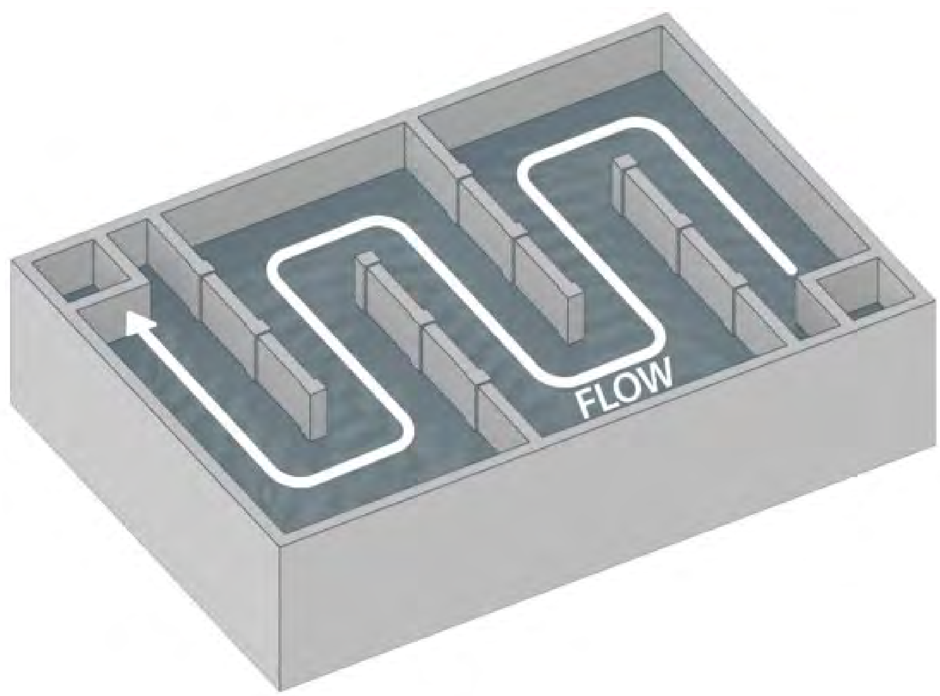
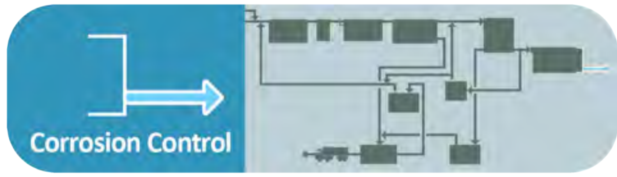


Figure 6-11. Example clearwell configuration with baffles providing both disinfection and storage

6.9 Corrosion Control



The Lead and Copper Rule requires water systems to implement optimal corrosion control treatment to minimize lead and copper release to drinking water.

The Improved Corrosion Control Treatment (ICCT) project at Lusted Hill is a critical part of the implementation to meeting EPA requirements and protect public health. ICCT consists of facilities for feeding powdered soda ash and carbon dioxide (carbonic acid solution) to maintain a pH of 8.5 and alkalinity between 25 and 40 mg/L as $CaCO_3$ throughout the distribution system when serving 100 percent Bull Run water. This treatment approach was recommended and accepted by OHA following completion of a corrosion control pilot study on unfiltered Bull Run water (Confluence Engineering, 2018).

The long-term corrosion control approach at the filtration facility will be re-evaluated with a corrosion control study using filtered water. For project definition and cost estimate development, it is assumed that soda ash and carbon dioxide processes will be incorporated at the filtration facility.

There are numerous approaches to corrosion control allowed under the current Lead and Copper Rule. The two most common approaches are:

- pH/alkalinity adjustment
- Corrosion inhibitor addition

The chemicals implemented to meet the selected corrosion control approach will be evaluated and selected as part of the corrosion control study and/or detailed design process. Treatment approaches previously evaluated by PWB are briefly described below.

6.9.1 pH/Alkalinity Adjustment

Corrosion control through carbonate passivation is typically accomplished through pH/alkalinity adjustment. The dissolved inorganic carbon concentration, which characterizes the level of carbonate alkalinity, is an important water quality parameter when using this corrosion control strategy. EPA guidance indicates that systems should have a minimum of approximately 5 mg/L as carbon (C) for lead corrosion control. Overall, it is a balance of dissolved inorganic carbon and pH that theoretically impacts lead and copper solubility; therefore, both parameters are important to control. Chemicals that can be used for pH/alkalinity adjustment include:

- Sodium hydroxide (caustic soda)
- Sodium carbonate (soda ash)
- Sodium bicarbonate
- Hydrated lime
- Quicklime
- Carbon dioxide

PWB's recently completed corrosion control study that identified ICCT compared the advantages and disadvantages of each of these chemicals. The evaluation results will be re-visited during the corrosion control study and/or detailed design.

6.9.2 Corrosion Inhibitor Addition

Corrosion inhibitors are phosphate- or silicate-based. Inhibitors form a protective, relatively insoluble film on the inside of the pipe that acts as a barrier between the water and the pipe surface. Commonly used corrosion inhibitors include chemicals such as orthophosphate, different poly- and blended phosphates, and silicates. In the recently completed ICCT study, these chemicals were not found to provide any benefit over pH/alkalinity control. Furthermore, some are not available in the region. The corrosion control study will determine their applicability (e.g., orthophosphate is typically most beneficial for lead service lines, which are not present in PWB's system) and if they require testing.

6.9.3 Corrosion Control Summary

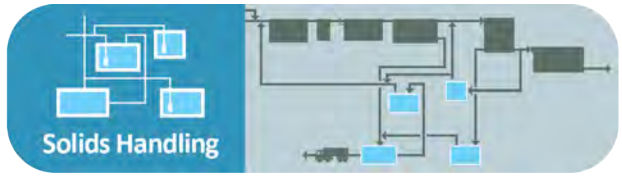
For project definition and initial cost estimate development, it is assumed soda ash and carbon dioxide will be added to maintain a pH of 8.5 and alkalinity adjustments between 25 and 40 mg/L as $CaCO_3$ throughout the distribution system. As noted in Chapter 5, the long-term corrosion control approach for the filtration facility will be evaluated. The results may differ from the approach assumed for this report.

Figure 6-12 depicts an example of a corrosion control soda ash feed system.

Figure 6-12. Example corrosion control soda ash feed system

Source: Alpine Industrial Systems

6.10 Solids Handling



Both sedimentation and filtration treatment processes generate residual solids. These solids need to be removed regularly to maintain proper operation of the treatment process. The residual solids at the filtration facility will consist of solids present in the raw water, along with solids generated from chemical precipitants and additives used for treatment. These solids must be concentrated and partially dewatered for economical truck transportation off site.

This section describes several solids handling considerations, including:

- Solids Characteristics and Origins
- Solids Treatment Strategies
- Mechanical Solids Handling Alternatives
- Solids Handling Summary

The two basic means to manage residual solids from water treatment facilities are using non-mechanical (e.g., lagoon-based) or mechanical (e.g., clarifiers, thickeners, and dewatering devices) systems.

Table 6-23 presents a summary of important issues associated with solids handling that were considered to arrive at project definition assumptions.

Table 6-23. Important Issues for Solids Handling

Important Issues	Description
Influence of raw water quality on residual solids	• Low turbidity water contains low concentrations of suspended and colloidal solids and will produce low quantities of residual solids. • High turbidity episodes (e.g., post-fire) or periods of high organics concentrations, in the absence of ozonation, may require the use of PAC, which can significantly increase solids production.
Type, characteristics, and quantities of solids produced	• Residual solids will be mostly metallic salt hydroxyl precipitates coupled with raw water suspended and colloidal solids, plus the presence of coagulant polymers. • Use of PAC will impact solids production proportionally. Solids produced from the treatment process will initially be light and somewhat difficult to thicken and dewater without proper treatment. • Quantities of solids will be affected by flow, suspended and colloidal solids concentrations, and treatment chemical applied for particulate removal and chemical adsorption.
Basic solids handling steps	• After solids-rich streams are isolated, residual solids must be thickened and dewatered for economical transportation and disposal off site (landfill disposal is assumed for project definition).

Table 6-23. Important Issues for Solids Handling	
Important Issues	**Description**
Non-mechanical and mechanical based handling systems	• Two basic practices include non-mechanical lagoon-based storage, thickening, and dewatering along with mechanical equalization, thickening, and dewatering systems. • Lagoon-based solids handling systems are strongly affected by available land area and meteorological conditions of precipitation versus localized evaporation. • Hybrid scenarios are also possible, for example, a mechanical process can be coupled with a storage pond to provide a greater degree of equalization, resulting in smaller equipment sizing for mechanical handling systems. For the purposes of project definition, these numerous hybrid permutations will not be considered, but will be deferred to the design process.
Recycle streams	• All recycle streams bearing any solids or particulate matter are to be conveyed to the head end of the filtration facility after proper flow equalization consistent with the requirements of OAR 333-061-0032-10 and the EPA *Filter Backwash Recycling Rule Technical Guidance Manual* (EPA 816-R-02-014). • Due to the absence of nearby sewers, this facility is assumed to be designed as a ZLD facility.

6.10.1 Solids Characteristics and Origins

Solids present in the raw water include both suspended and colloidal solids. These solids may be inorganic (e.g., clays and silts) or organic (e.g., organic detritus and algal cells). Organic solids in most watersheds derive from NOM and are quantified in terms of total organic carbon. Overall, the raw water solids are quantified in terms of total suspended solids (TSS).

The portion of the solids produced at the filtration facility that is not comprised of raw water solids derives from chemicals added as part of the treatment process. Precipitated solids from the coagulation process will be a major constituent. These solids generally consist of metal hydroxides such as aluminum hydroxide (from alum coagulation) or ferric hydroxide (from coagulation with ferric sulfate or ferric chloride). While the formation of the precipitated solids is directly proportional to the coagulant dose, the actual quantity depends on the underlying chemistry and will differ based on coagulant type and concentration.

Other chemicals added directly contribute to the solids removed by treatment. For example, PAC added for the adsorption of dissolved organics will be either settled out in the sedimentation basins or removed during filtration. Therefore, every pound of PAC added will directly result in a pound of residual solids. Similarly, polymers added for treatment (e.g., coagulation aids, filter aids, thickening aids, and dewatering aids) generally produce residual solids on a one-to-one mass basis.

The following section summarizes the residual solids management concepts that are assumed to be applicable to the filtration facility. Residual solids are assumed to be generated from the sedimentation basins and backwashing filters. Solids removed from sedimentation processes typically range from 0.1 to 2.0 percent solids, while washwater (water used to backwash the filters) will generally be less than 0.05 percent solids.

Assumed Solids Production

The production of residual solids is influenced by multiple factors, including:

- Total facility flow.
- Influent raw water suspended and colloidal solids (i.e., turbidity) and water chemistry and physical characteristics.
- Coagulant dose.
- Polymer dose.
- Other additives (such as PAC).

Generally, unit solids production at water treatment facilities is more heavily influenced by the raw water solids characteristics and loadings, including coagulant dose, than by changes in total facility flow demands. However, from a review of historical raw water quality data for the Bull Run supply, it is assumed that raw water solids characteristics and loadings will be generally stable at the filtration facility. The following criteria were assumed to project solids production for project definition:

- Total peak day facility flow = 160 mgd
- Annual average flow = 97 mgd
- Influent raw water solids = 0.5 mg/L TSS
- Coagulant dose = 10 mg/L alum (as 17.1 percent Al_2O_3) is assumed for project definition.
- Polymer dose = 1 mg/L (as product) is assumed for project definition for both coagulant and filter aid use.
- Other additives (such as PAC) = 0 mg/L (under normal circumstances).

Based on these criteria, the corresponding peak day solids production would be 7,910 dry solids-lbs./day. This value will be used to size solids treatment systems at the filtration facility. The annual average solids production is estimated to be approximately 4,800 dry solids-lbs./day. The annual average unit solids production rate is estimated to be 50 pounds dry solids/MG water produced.

Using the values listed above, the filtration facility residual solids are estimated to be comprised of approximately:

- Nine percent raw water solids.
- Seventy-four percent aluminum hydroxide based on the assumed use of alum as a prime coagulant for project definition.
- Seventeen percent polymer.

The solids production listed above represents assumed maximum solids production under anticipated normal conditions. This includes the assumption of ozone pre-treatment, which is why the assumed PAC dose is selected at 0 mg/L. If ozone pre-treatment is not included, it is assumed that the facility would need to occasionally feed PAC for removal of organics and T&O-causing compounds in the event of changes brought on by climate change or wildfires, which would correspondingly increase solids production. Similarly, the assumed maximum day solids production does not include extraordinary events, such as fire damage in the watershed, which would likely increase raw water solids and coagulant demand. Finally, considerations have been made for the ultimate facility capacity to make sure there is adequate space for future expansion of solids handling facilities.

Note that the assumed maximum day solids production should not be extrapolated to determine annual solids production. It is anticipated that the average day coagulant dose may be lower than 10 mg/L. Therefore, although the residual solids treatment processes will be sized for a peak day solids production, the average day throughput will be considerably lower.

Sedimentation basin solids are typically removed via a mechanical solids collection system (e.g., chain-and-flight collectors or hoseless sludge collectors), and the resulting residuals stream is commonly referred to as sedimentation basin blowdown or sludge.

The solids that accumulate in the filter beds are removed via backwashing. During the backwash event, the backwash water used to wash this filter is converted to solids-laden washwater, which must be managed. Together, the sedimentation basin sludge and washwater contain the majority of the residual solids generated by the filtration facility.

Solids removed from sedimentation processes are assumed to typically range from less than 0.1 to 2.0 percent solids, while washwater will generally be assumed to be less than 0.05 percent solids. At these concentrations, the solids are too dilute for direct mechanical dewatering, although some forms of non-mechanical dewatering, such as dewatering lagoons, could directly receive sedimentation sludge and washwater.

In addition to the two residuals streams discussed above, a small percentage of the solids produced at the filtration facility will be contained in the filter-to-waste stream. Filter-to-waste is a practice designed to discharge the small amount of turbidity released from the filters during their ripening period following a backwash event before the filter is put back into service. Therefore, this waste stream consists of low-turbidity water and is often discharged or recycled directly without additional solids treatment.

6.10.2 Solids Treatment Strategies

Solids handling is most often a five-step process, including:

1. Flow Equalization
2. Clarification and Thickening
3. Dewatering
4. Disposal
5. Recycle Stream Treatment

These basic unit processes are to be used regardless if solids handling is conducted via non-mechanical or mechanical means. The solids produced at the filtration facility will be in the form of a slurry or suspension, the concentration of which will depend on the type of residual. Unless the solids are to be trucked in liquid form for disposal at a wastewater facility or elsewhere, most water treatment facilities use solids treatment processes to concentrate the solids into a dewatered cake, which can then be more efficiently and cost-effectively trucked off site for reuse or disposal (Kawamura, 2000).

There are many types of solids handling technologies commonly used to concentrate and dewater water treatment facility solids (Kawamura, 2000). The two main categories of solids handling technologies are:

- **Non-mechanical systems** that use passive methods such as drainage, decanting, and solar drying in lagoons to separate free water from the residual solids.
- **Mechanical solids handling systems** that input energy to force free water out of the thickened sludge.

Many water treatment facilities with mechanical solids handling systems use a clarifier or gravity thickening process to concentrate the solids into a thickened sludge, resulting in a typical solids concentration of 2 to 5 percent. This thickened sludge, while still a pumpable liquid, is thick enough to be subsequently treated by most dewatering processes.

It is assumed most solids handling systems for filtration facility residual solids can produce a dewatered cake that is at least 20 percent solids.

The analysis of solids handling options included discussions of non-mechanical and mechanical alternatives, along with best practices for solids disposal and recycle stream treatment options.

Non-Mechanical Solids Handling

The non-mechanical solids handling treatment train is less complicated than that for mechanical solids handling. Filter-to-waste is treated the same as in mechanical dewatering and is recycled either to the head of the process or to the front of the filters. Washwater and sedimentation basin solids are both sent directly to the non-mechanical solids handling structures (i.e., lagoons), where they would settle and consolidate. Decant from lagoons would be recycled to the head of the process. After the residual solids are sufficiently dewatered in the lagoons, they would be removed via heavy equipment and transported off site for beneficial use or disposal.

Non-mechanical lagoon-based solids handling is generally recognized as a less-expensive process. After solids are loaded into a lagoon or sand drying bed, the only operational attention required is that associated with decanting the clear liquid from the top of the lagoon (supernatant). Operational and energy inputs are low to negligible until evaporative drying has sufficiently dewatered the solids to the target cake concentration, at which point the solids are removed. However, this process is time-intensive and is subject to local precipitation and evaporation influences, so the filtration facility needs to have a sufficient number of lagoons or beds, or sufficiently sized lagoons or beds, to receive new solids while the full lagoons or beds are drying.

Drying bed technologies share a common design concept: the cover material (bed) is installed over an underdrain consisting of gravel and perforated pipe. Drying bed technologies differ in the type of supporting material used for the bed surface (e.g., sand) and in whether external forces such as vacuum are used to promote separation of solids. Initially, water percolates through the bed and is collected by the underdrain and discharged. Additional dewatering then occurs via evaporation. Sludge drying beds may be classified as: 1) conventional sand, 2) paved, 3) artificial media, and 4) vacuum assisted. Many design variations are possible. Key design considerations include the layout of drainage piping, the thickness and type of layers, and construction materials. Solids concentrations resulting from a typical sand drying bed can range between 20 to 25 percent. Figure 6-13 illustrates a cross section of a drying bed.

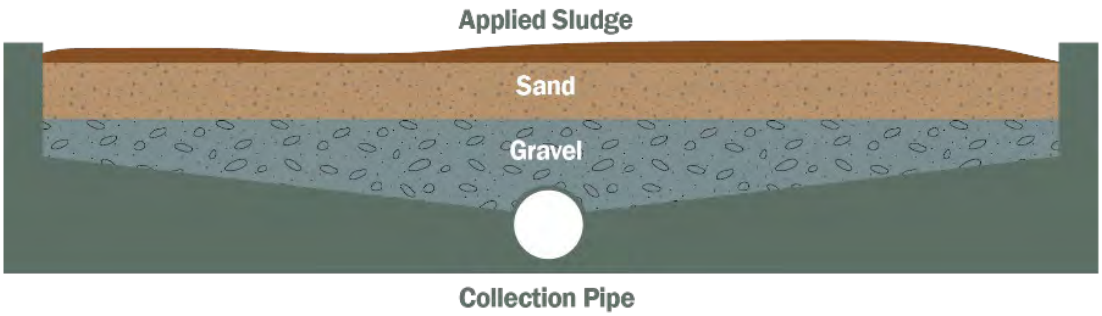


Figure 6-13.Typical drying bed configuration

Meteorological Limitations. Non-mechanical dewatering depends on evaporative drying to achieve its final dewatered cake solids concentration. It is assumed properly sized non-mechanical dewatering processes can achieve solids concentrations of at least 20 percent. Unlike mechanical dewatering, which generally cannot dewater past 35 to 40 percent solids, non-mechanical dewatering processes can produce very dry solids (greater than 50 percent solids concentration) if given sufficient drying time. However, even if the majority of the free water, on a mass basis, is removed via decanting, it is generally not possible to achieve a solids concentration of more than 10 to 12 percent in a non-mechanical dewatering lagoon or sand bed without evaporative drying. Since the net annual evaporation at the facility site is low, the residual solids will need longer residence time in the dewatering lagoons or sand beds, which will increase the total area of lagoons or sand beds required. Preliminary sizing indicates that the eastern portion of the site, approximately 18 acres (nearly the entire available eastern portion of the site), would need to be covered with non-mechanical dewatering lagoons to treat the projected solids production when the facility is operating under the 160 mgd flow condition with average annual climatic conditions (i.e., 5 inches of net evaporation per year given the annual average precipitation of 36 inches per year).

Figure 6-14 illustrates a conceptual facility layout of this option, based on the ultimate future 240 mgd facility flow and 5 years of average weather conditions. During years with above-average precipitation or below-average evaporation, this 18-acre drying lagoon area may be insufficient to dewater solids produced by the filtration facility.

Lagoons have a limited ability to adapt to conditions that increase solids loading. For example, as discussed in Chapter 3, a watershed fire would likely increase runoff and raw water turbidities and may require addition of PAC (if ozone pre-treatment is not installed at the facility) to remove the increased organic loading in the raw water. The lagoons may not have sufficient size to account for this additional volume of solids.

Figure 6-14. Conceptual site layout of filtration facility using non-mechanical solids handling

Mechanical Solids Handling

The conceptual mechanical solids handling treatment train assumes that filter-to-waste would not be treated and would be recycled either to the head of the filtration facility or directly back to the filter influent. Washwater would be collected in a washwater equalization tank, and then released over time to hybrid gravity thickeners that would clarify and thicken the solids in the washwater, or to separate clarifiers and gravity thickeners. Gravity thickeners would also receive solids directly from the sedimentation basins, which would be treated with a thickening polymer. Supernatant from the clarifiers/gravity thickeners would flow to an overflow tank, before being pumped back to the head of the facility to be recycled. Thickened sludge would be pumped to thickened solids tanks to homogenize and equalize the sludge prior to dewatering. Sludge would be pumped from those tanks to the mechanical dewatering process, from which the dewatered cake would be transported off site for beneficial use or disposal. The liquid residuals from the dewatering process (centrate or pressate) would be returned to the gravity thickeners for clarification and thickening.

Mechanical dewatering does not require a large footprint for treatment and is generally unaffected by local weather influences. Mechanical dewatering physically presses and drains excess water from solids. Solids are processed within days, if not hours, of when they are produced. There is much less need for solids storage, which reduces the associated footprint. However, a number of supporting processes are required to facilitate mechanical treatment, including equalization storage tanks, clarifiers, gravity thickeners, homogenization tanks, and polymer feed systems. These systems increase the amount of operator attention required and add capital and maintenance costs. There are also higher operational costs associated with mechanical dewatering because of power and chemical inputs.

The assumed conceptual site layout (Figure 6-15) for a mechanical solids handling option includes two equalization tanks for washwater along with space for a future third equalization tank, two clarifiers and gravity thickeners along with space for a future third clarifier and gravity thickener, a recycled water basin, a dewatering building, and two solids homogenizing basins with space for a future basin. The final configuration of these unit processes will be determined during design if a mechanical solid handling process is selected. Moreover, the final design may consider the use of lagoons (or even the mostly empty overflow lagoons) to provide flow equalization for a mechanical solids handling system, resulting in smaller sizes for downstream solids handling equipment designed to address reduced flows from equalization tanks or basins.

Figure 6-15. Conceptual site layout of filtration facility using mechanical solids handling

Non-mechanical and Mechanical Solids Handling Comparison

Several site characteristics warrant consideration when evaluating solids handling alternatives. First, there is no sewer access and waste discharge to the environment is deemed impractical due to permitting issues, so all liquid streams from the solids handling processes will need to be recycled back to the head of the facility. Second, the site is fairly large, with 95 acres available for potential site buffers and treatment processes. Finally, net evaporation at the site is low due to high annual rainfall. Based on data from the past 5 years, the net annual evaporation is only 5 inches per year. When evaluating mechanical versus non-mechanical solids handling systems, the evaporation consideration is the most relevant. A particularly wet year can strongly influence lagoon dewatering if lagoon sizing is based only on average weather conditions.

Table 6-24 presents a summary of non-mechanical and mechanical solids handling considerations. For project definition, mechanical solids handling systems are assumed.

Table 6-24. Non-Mechanical and Mechanical Solids Handling Considerations

Advantages	Disadvantages
Non-mechanical	
• Simple operations, potentially lower capital and O&M costs.	• Land intensive process that relies on evaporative drying (i.e., weather dependent). • May be overwhelmed by a significant raw water quality event requiring large uses of PAC.
Mechanical	
• Smaller footprint, does not rely on environmental inputs.	• More complex, requiring more supporting processes (e.g., washwater equalization, clarification, or gravity thickening). • Potentially higher capital and O&M costs.

6.10.3 Mechanical Solids Handling Alternatives

There are numerous typical options available to process solids using mechanical systems. A discussion of these options follows.

- Flow Equalization
- Clarification and Thickening
- Dewatering
- Disposal
- Recycle Stream Treatment

Figure 6-16 below depicts the treatment process for solids streams using a typical mechanical solids handling process. The solids treatment steps are described in the following sections.

Flow Equalization

To reduce the size of solids handling equipment, flow equalization of washwater is often considered. For project definition, two concrete equalization tanks are assumed to contain at least three typical backwash volumes combined (conservative assumption). During design, other tank configuration options or storage volumes may be considered.

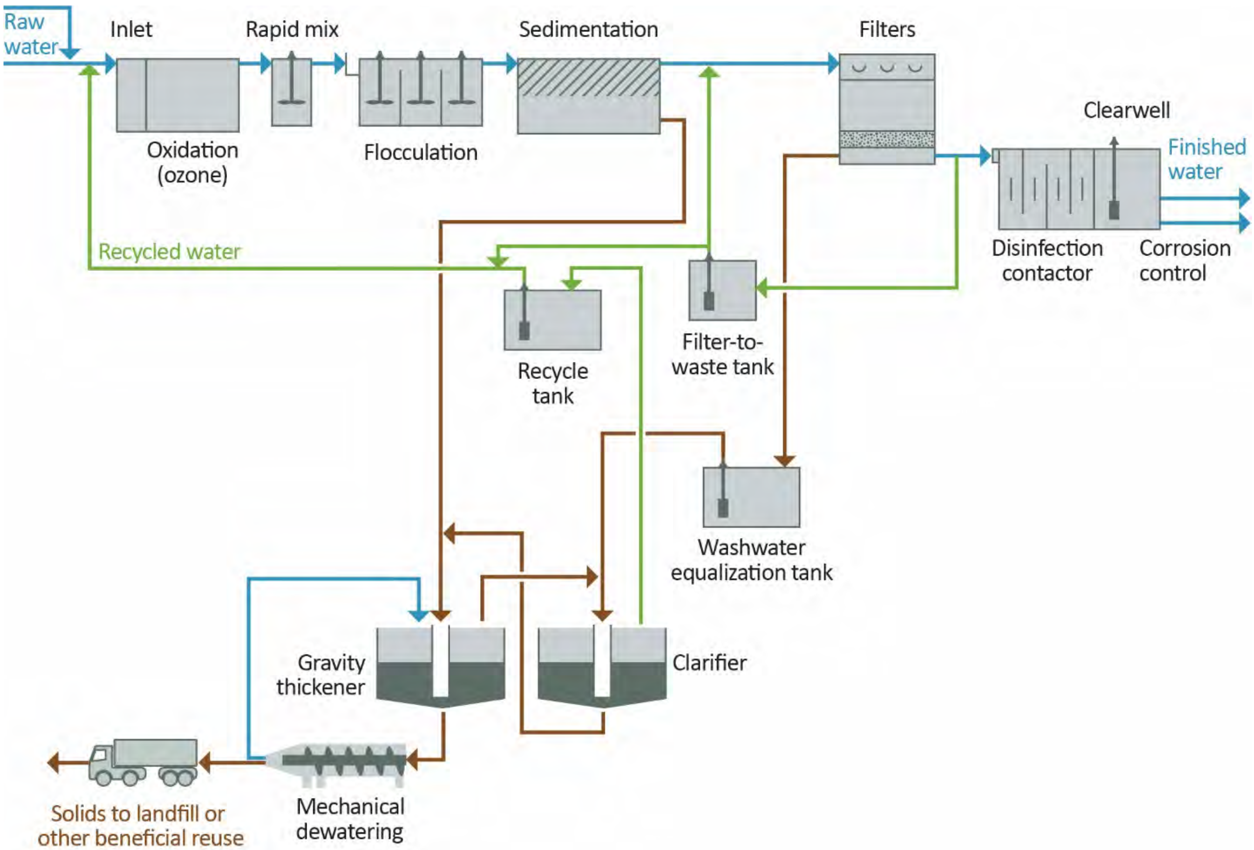


Figure 6-16. Treatment process showing a typical mechanical solids handling system

Clarification and Thickening

Residual streams from water treatment processes (sedimentation solids and washwater) are typically less than 1 percent solids and must be thickened to be economically dewatered. Initial consideration of residuals treatment focused on solids handling hybrid gravity thickeners that would both clarify washwater and thicken solids from the washwater and sedimentation basins. It was assumed that a more conservative approach would be to size dedicated solids stream handling clarifiers for the washwater. Blowdown from these clarifiers would be combined with blowdown from the sedimentation basins and sent to dedicated gravity thickeners. This has the effect of changing the limiting factor for gravity thickener sizing from the hydraulic loading rate (for the hybrid gravity thickeners) to the solids loading rate (for the dedicated gravity thickeners). This reduces the size of the gravity thickeners but requires the addition of washwater clarifiers.

For project definition, the use of gravity clarifiers to concentrate washwater solids prior to thickening is assumed. It may be possible to eliminate separate clarifiers and thickeners in lieu of the use of larger gravity thickeners, but this refinement is deferred to design.

If ozone is not added to the primary treatment train, the filtration facility will need to allow for additional solids from adding up to 20 mg/L or more of PAC. However, for project definition purposes, the use of ozonation is assumed without the need for PAC.

Depending on the thickening process, the solids concentration for treated coagulation residuals can be between 1 to 4 percent. There are several options for separate thickening processes. Common thickening systems are discussed in more detail below, including:

- Gravity Thickening
- Flotation Thickening
- Mechanical Thickening

Gravity Thickening. Gravity thickening uses the natural tendency of higher-density solids to settle out of liquid to concentrate the solids. The residuals stream is fed into a tank through a center well, which releases the solids at a low velocity near the surface of the tank. The solids then settle to the bottom of the tank and are removed by scrapers. Many systems also use a skimmer to collect and remove floatables that accumulate at the top of the tank. Design considerations include the tank shape and size, inclusion of a conical bottom fitted with collectors or scrapers, and the need for static conditions to induce settlement. Gravity thickening typically yields a solids concentration of 2 to 3 percent (MWH, 2005). Figure 6-17 shows a typical gravity thickener.

Figure 6-17. Typical gravity thickener

Source: Goble Sampson

Flotation Thickening. Flotation thickening achieves solid-liquid separation by introducing air bubbles into the flotation process system. This is referred to as dissolved air flotation thickening (DAFT) and has been demonstrated to be a successful separation technology (MWH, 2005). In DAFT, pressurized air is injected into recycled drinking water and added to the residuals feed. When the pressure on the injected water is released, it allows the super saturated air to escape into the residuals as small bubbles that cause turbulence. The small bubbles mix with and adhere to the solids suspended in the residuals stream so that they float rather than sink. The floating material (thickened solids) is then skimmed off.

Figure 6-18 shows a typical DAFT system.

Figure 6-18. Typical DAFT system

Source: Goble Sampson

Mechanical Thickening. Gravity belt thickeners (GBTs), often used for this purpose, are constructed from a porous belt (metal mesh) that allows water to drain through while retaining the solids. The recirculating belt travels through solids removal and wash sections before returning to service. Considerations, such as belt material and applied loading, influence separation efficiencies, with solids concentrations for residual solids ranging from 2.5 to 4.5 percent.

Figure 6-19 below illustrates a gravity belt thickener.

Figure 6-19. Typical mechanical gravity belt thickener

Source: Goble Sampson

Table 6-25 provides a summary of advantages and disadvantages of gravity, flotation, and mechanical thickening processes.

Table 6-25. Thickening Options Summary Considerations	
Advantages	**Disadvantages**
Gravity Thickeners	
• Simple to operate and maintain. • Low operating costs compared to DAFT or GBTs. • May not need polymer as a thickening aid. • May achieve approximately 95% solids recovery.	• Requires more land area than mechanical GBTs and DAFTs. • Produces solids concentrations (typically, 2–3% solids by weight) that are usually lower than DAFT or GBTs. • Long retention times cause scum buildup. • Generates a return flow with small solids quantities for reprocessing at the head of the filtration facility.
Dissolved Air Flotation Thickeners	
• Higher loading rates equate to smaller footprint than gravity thickening (MWH, 2005). • Adjustable air flow allows for variable solids loading. • Can remove low density particles that require long settling periods. • Produces typical solids concentrations of 2–4%. • May achieve approximately 95% solids recovery.	• Higher capital, energy, and operating costs. • Air loading and recycle rates are key design considerations that can affect DAFT performance. • Generates a return flow with small quantities of solids for reprocessing at the head of the process. • Usually requires addition of a thickening aid polymer.
Mechanical Gravity Belt Thickeners	
• Typically used if space is a constraint. • Typically used if gravity settling or flotation do not provide desired solids thickening. • GBTs are simple designs with minimal operator oversight. • May achieve approximately 95% solids recovery.	• Generates a return flow bearing some small quantities of solids for reprocessing at the head of the filtration facility. • Usually requires a thickening aid polymer. • Requires maintenance.

Dewatering

Dewatering is a critical step for economical disposal of residual solids. Dewatering processes can be characterized as either mechanical or non-mechanical (gravity) and can achieve up to 45 percent solids concentrations (AWWA, 1999). Typical processes are discussed in more detail below, including:

- Belt Filter Press
- Centrifuge
- Screw Press

Belt Filter Press. Belt filter presses use pressure to force water out of the residuals through a porous belt while retaining the separated solids on the belt. Treatment residuals are deposited on the dewatering belt and drained in the free drainage zone. The remaining solids/water are sandwiched between two porous belts and passed over/under a series of different diameter rollers. The rollers impart low and high pressure on the belts, squeezing the additional water from the solids and through the porous belt. The more extensive the belt travel, the drier the filter cake. Key design considerations include the size of the belt press and solids conditioning prior to the dewatering process. Belt presses are sized on the basis of weight or volume of solids to be dewatered rather than the water flow of the facility. Solids must be conditioned with polymer to ensure optimum performance. Polymer feed points should be designed at several locations. Solids concentrations can be between 1 to 20 percent for low TSS waters, for example where solids are primarily made up of aluminum hydroxide floc, and up to 50 percent for higher TSS content. Situations where 1) coagulants that could be used (e.g., polyaluminum chloride or aluminum chlorohydrate) to generate smaller quantities of difficult-to-dewater hydroxyl sludges as compared to the larger quantities of floc produced with the use of alum, or 2) where PAC is used, will create scenarios where solids will be easier to dewater and may produce higher cake solids concentrations.

Figure 6-20 depicts belt filter press dewatering from the program team's tour of Metro Vancouver's water treatment facility.

Figure 6-20. Belt filter press dewatering at Metro Vancouver's Seymour Capilano facility

Vancouver, British Columbia

Centrifuge. Centrifuges use centrifugal force to separate suspended solids from water. The amount of force applied to the residual solids depends on the centrifuge's rotational speed. The solid bowl centrifuge is the principal type of centrifugal separator used to dewater treatment residuals (AWWA, 1999). The bowl centrifuge has two moving parts: the bowl and the scroll. As centrifugal force pushes the solids to the edge of the spinning bowl, a rotating scroll moves the dewatered solids along a horizontal axis to a collection point.

Key design considerations are the size of the centrifuge, rotational speed (often many thousands of revolutions per minute) and centrifuging time, as well as solids pre-conditioning. Centrifuges are sized based on the weight or volume of solids to be dewatered, the force applied, and the centrifuging time. Centrifuges perform better with the addition of a conditioning agent; thus, they are rarely operated without the addition of a polymer to the residual suspension. Typical solids concentrations for centrifugal separators are between 20 to 30 percent.

Screw Press. Screw presses operate similarly to centrifuges; however, screw presses use direct mechanical squeezing force, rather than centrifugal force, to push free water from the sludge in a tapered device (akin to a meat grinder used for sausage making). Whereas a centrifuge may operate at many thousands of revolutions per minute, a screw press moves very slowly, typically at a few revolutions per minute. Sludge entering a screw press must first be conditioned with dewatering aid polymers for effective operations.

Figure 6-21 illustrates a cross section of a screw press.

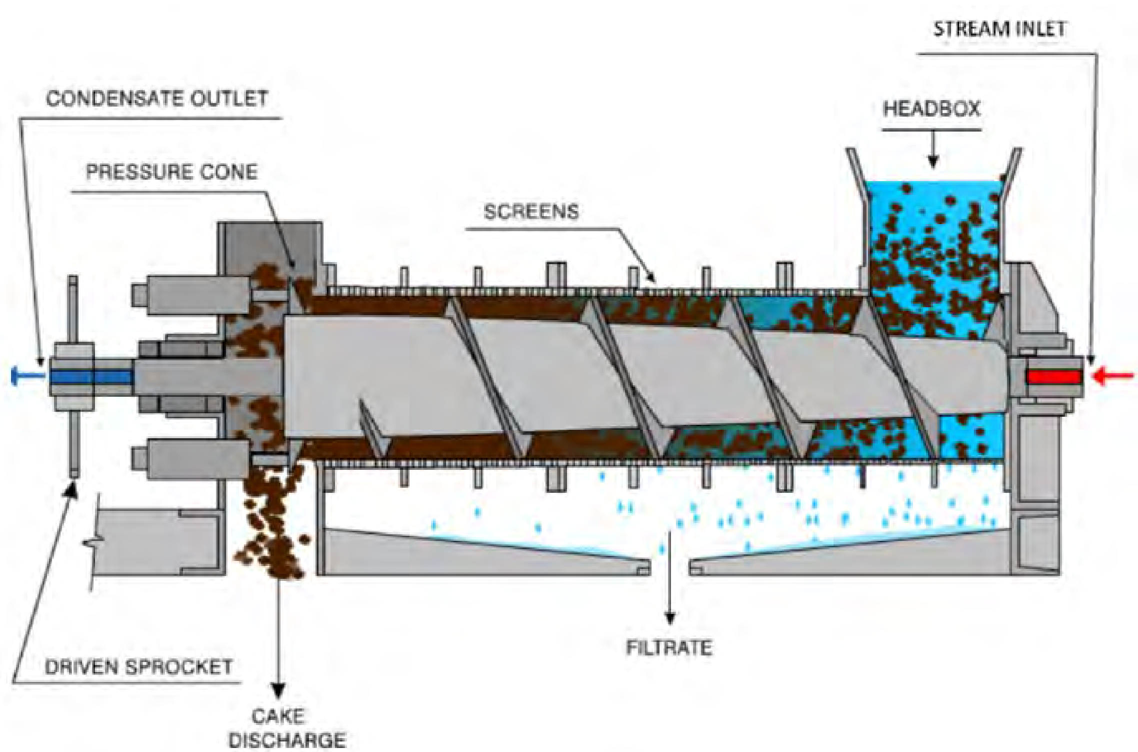


Figure 6-21. Typical screw press

Source: FKC Screw Press

Table 6-26 provides a summary of advantages and disadvantages of belt filter press, centrifuge, and drying bed dewatering processes.

Table 6-26. Dewatering Options Summary Considerations	
Advantages	**Disadvantages**
Belt Filter Press	
• Simple maintenance. • Solids concentrations up to 50% can be achieved, especially for lime softening residuals because their more granular structure can withstand higher pressures.	• Belts require washing, which generates another residual stream. • Usually requires a dewatering aid polymer.
Centrifuge	
• Lower O&M costs and better performance than conventional belt filter presses. • Small footprint. • Easier to clean than a belt filter press.	• Operational problems and energy costs increase proportional to the centrifuge size. • High rotational speed. • Higher energy consumption than other devices. • More complex maintenance and cleaning needed for these high-speed machines. • Usually requires a dewatering aid polymer.
Drying Beds	
• Low capital cost and energy consumption. • Low to no chemical consumption. • Low attention required compared to other mechanized systems. • Low sensitivity of sludge variability.	• Large footprint. • Rate of evaporation depends on local climate, residual solids characterization, and extent of external drainage enhancement. • Thin layers dry more quickly, but also increase operating costs.
Screw Press	
• Slow rotating device. • Relatively low energy consumption. • Easier to clean than a belt filter press.	• Can be expensive. • Usually requires a thickening aid polymer.

Disposal

After treatment, dewatered residual solids are taken off site for beneficial use (e.g., concrete making or land application) or to a landfill for use as daily cover. During project definition, the TAC provided input regarding best practices for solids disposal at other filtration facilities. These beneficial use options include:

- Concrete or brickmaking
- Land application to bind phosphorus and reduce nutrient runoff
- Roadway and road bank construction and fill
- Blending with compost as a bulking agent

These beneficial use options will require addition investigation once the characteristics of the residual solids are known (pending finalization of design criteria, pilot study data, and operational assumptions). For project definition, residual solids used as daily cover at a landfill is assumed.

Recycle Stream Treatment

Free water removed during thickening and dewatering (unless removed via evaporative drying) of residual solids ("residual liquids") also needs to be managed. Common options for residual liquids include:

- Discharge to the environment via a National Pollutant Discharge Elimination System-permitted outfall.
- Disposal to a nearby sanitary sewer.
- Recycle to the head of the facility's primary treatment train.

Due to the lack of sanitary sewer lines in the vicinity of the site and the assumed difficulty in obtaining a discharge permit to convey liquid waste streams to the environment, it is assumed that it will be necessary to recycle all liquid residuals streams. The filtration facility will be assumed to operate as a ZLD facility. Discussion during the TAC workshops regarding potential for recycle of cyanotoxins and/or *Cryptosporidium* strengthens the need for use of ozone pre-treatment in the primary treatment train, instead of using ozone to specifically treat the recycle stream. It was also recognized that organics recycled in the liquid residual streams will potentially increase DBP precursor material in the facility influent.

6.10.4 Solids Handling Summary

This section summarizes the anticipated characteristics of residuals solids at the filtration facility and the advantages and disadvantages of solids treatment strategies and mechanical solids handling alternatives.

- **Solids Treatment Strategies:**
  - Mechanical solids handling is preferred to non-mechanical solids handling because it can better respond to unplanned changes in solids loading rates such as those following storm or wildfire events.
  - There are also concerns about the ability to sufficiently dewater the filtration facility residual solids in the space available using non-mechanical lagoon-based dewatering during years with above-average precipitation and/or below-average evaporation.

- **Mechanical Solids Handling Alternatives:**
  - For project definition, the following items are assumed for mechanical solids handling:
    - Concrete washwater flow equalization tanks
    - Separate clarifiers for initial solids separation
    - Separate gravity thickeners
    - Screw press dewatering
  - It is assumed the mechanical dewatering options (i.e., belt filter press, centrifuge, and screw press) will have similar footprint requirements and budget-level capital costs. The types of mechanical thickening and dewatering will be subject to re-evaluation and refinement during design.
  - For project definition, it is assumed residual solids will be used as daily cover at a landfill, although other options are to be evaluated during design.
  - It is further assumed that due to the lack of nearby sewers, all recycle streams from solids handling processes are to be returned to the head of the filtration facility for additional processing consistent with a ZLD design.

6.11 Treatment Process Alternatives Summary

This section summarized outcomes from the initial evaluation of treatment process alternatives considered for treating Bull Run water (Figure 6-22 below). The alternatives evaluations described in this chapter focused on issues and considerations that significantly affect treatment performance, cost, or footprint of the filtration facility. The outcomes from these evaluations reflect project values, best practices, and engineering judgment and are based on technical workshops and additional analysis completed by the program team.

The following assumptions were used for project definition and conceptual cost estimate development and will be further evaluated during design:

- **Oxidation.** For project definition, pre-oxidation using ozone is assumed, with chlorine as a backup pre-oxidant.
- **Rapid Mix.** Pumped diffusion rapid mixing is assumed for project definition as the initial technology selection.
- **Flocculation.** Vertical shaft flocculation with adjustable speed drives is assumed for project definition as the initial technology selection.
- **PAC.** Further evaluation pending determination of ozone inclusion.
- **Sedimentation.** Conventional filtration best provides the ability to meet water quality and resilience goals. Full-sized high-rate sedimentation basins with plate settlers are assumed for project definition.
- **Filtration.** The filter design should not preclude future use of biological filtration. Provisions could include measures to manage disinfectant residuals going onto the filters. For project definition, deep bed dual media configuration using silica sand and either anthracite or GAC media with a maximum unit filtration rate of 6 gpm/sf is assumed. The filtration rate may be set to the standard rate of 6 gpm/sf, or increased based on pilot study results.
- **Disinfection.** Continued use of chloramines is selected for secondary disinfection and will be used for simulated distribution system testing in the pilot study. For project definition, use of chlorine for primary disinfection is assumed. Further evaluation during design of using ozone to meet a portion of primary disinfection needs.
- **Clearwell.** Clearwell volume within the feasible range of 10 to 16 MG. For project definition, a total volume of 16 MG with two 8 MG cells is assumed.
- **Corrosion Control.** For project definition, mid-range pH and alkalinity adjustments consistent with the ICCT project are assumed. This may be refined pending the outcome of the corrosion control study on filtered water and potential changes to the Lead and Copper Rule.

- **Solids Handling.** Solids handling systems should not rely wholly on non-mechanical (lagoon) systems because of their vulnerability to weather and inability to respond to water quality emergencies that increase solids production. Additional evaluation of specific solids handling technologies and residuals disposal will be conducted during design; the technologies are anticipated to have similar footprint requirements and budget-level capital costs.
- For the initial cost estimate, the following processes are assumed:
  - Concrete washwater flow equalization tanks.
  - Separate clarifiers for initial solids separation.
  - Separate gravity thickeners.
  - Screw press dewatering.
  - Solids used as landfill daily cover.
  - Recycle of solids handling liquid waste streams to the head of the process.

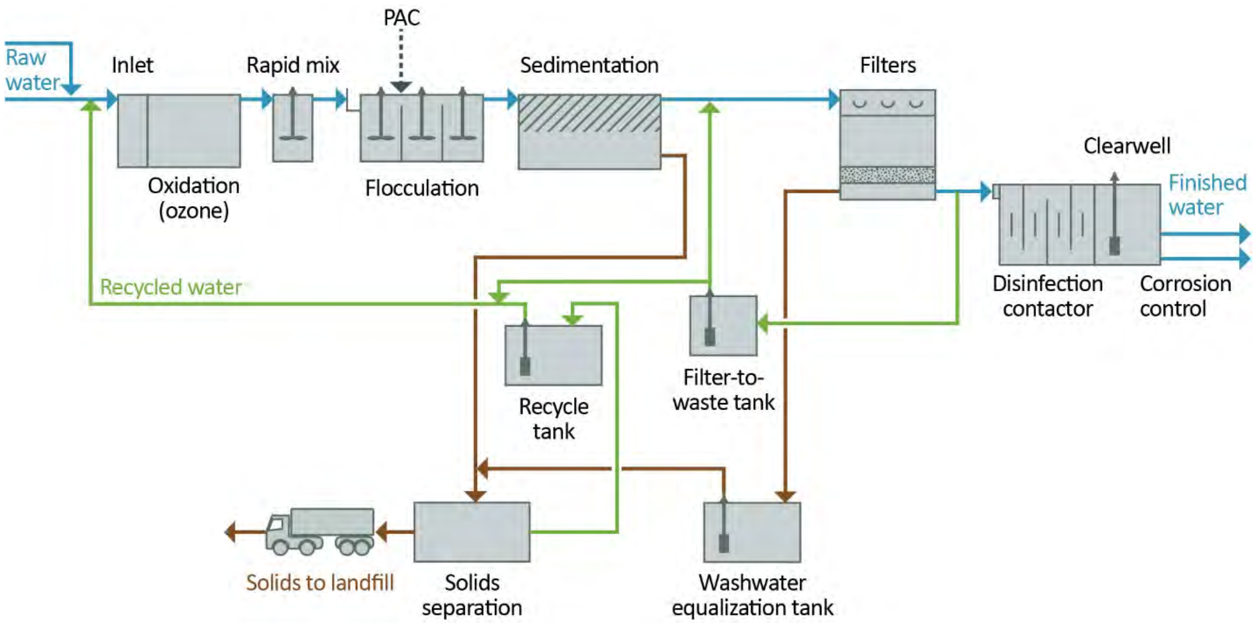


Figure 6-22. Process flow diagram for filtration unit processes

Note that PAC is not used in conjunction with ozone and that pre-ozonation is shown for illustration purposes.

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Chapter 7
Filtration Facility Support Systems

This chapter describes key support system features for the filtration facility. The Portland Water Bureau (PWB) is committed to implementing best practices to limit community impacts from the filtration facility and is working with the community on a Good Neighbor Agreement and other measures to address potential impacts. This includes considerations related to designing site grading, screening, landscaping, and architecture to fit within the rural area as described in this chapter.

This chapter includes the following sections:

- 7.1 Site Development
- 7.2 Utilities
- 7.3 Architectural Programming
- 7.4 Filtration Facility Support Systems Summary

7.1 Site Development

This section expands on Chapter 4: Planning Considerations, by describing design features of the filtration facility conceptual site plan shown in Figure 7-1 below that are related to the site development and by providing design guidance for the following:

- Site Grading and Earthwork
- Slope and Soil Mitigation
- Overflow Basins
- Stormwater
- Onsite Wastewater Management
- Site Access and Traffic Management
- Site Development Summary

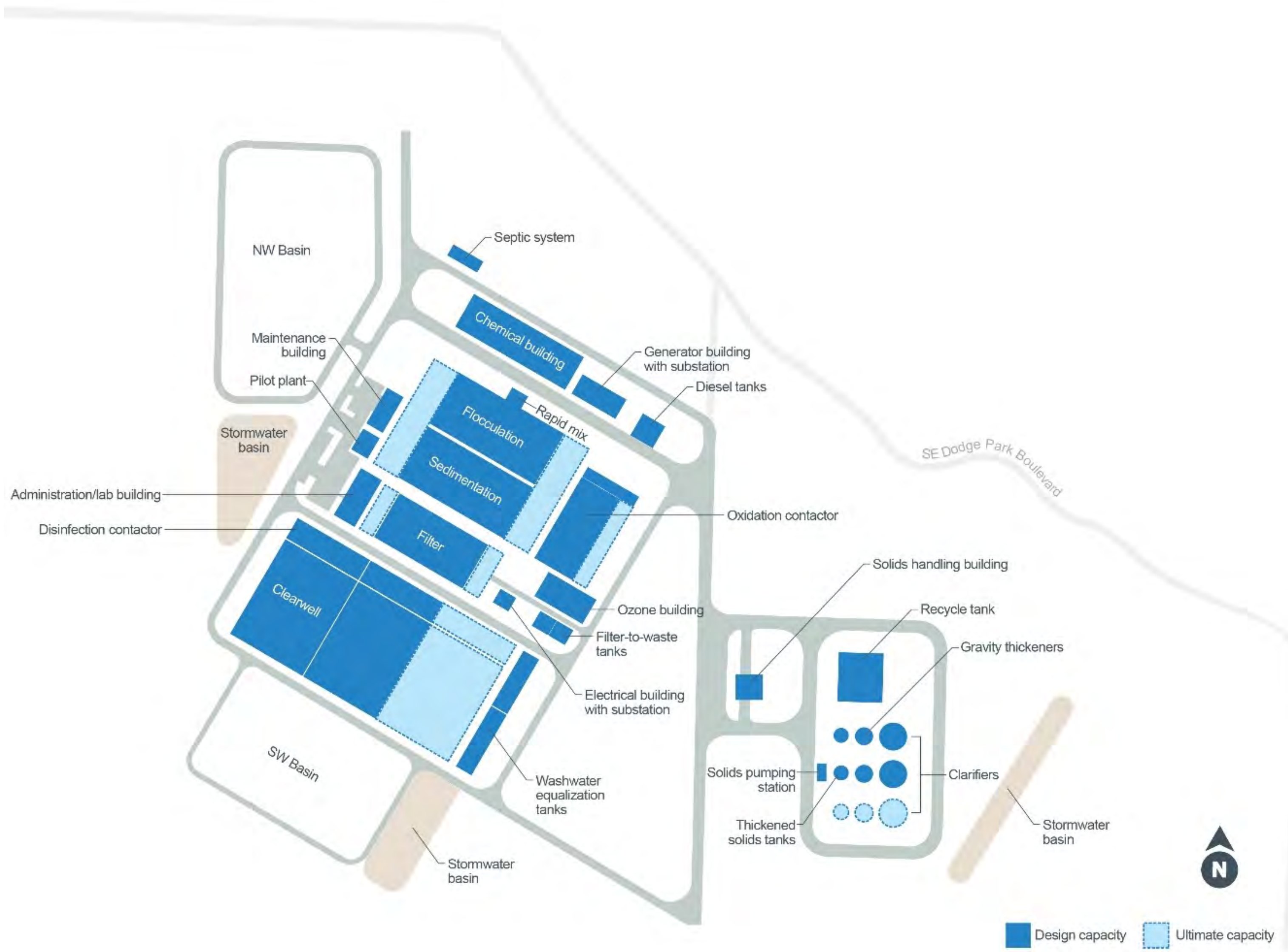


Figure 7-1. Conceptual site plan

7.1.1 Site Grading and Earthwork

The goal of gravity raw water delivery at the filtration facility requires that the inlet basin water surface be set at 715 feet elevation. This in turn sets the hydraulic profile of the filtration facility, ranging from 715 feet at the inlet to 691 feet maximum water surface at the clearwell (Chapter 4). It is common for the basins to be exposed above grade 3 to 5 feet to minimize excavation and provide adequate site drainage. The existing ground surface ranges from approximately 690 to 725 feet elevation immediately surrounding the treatment basins and clearwell, and the finish grade will need to be excavated down to 680 to 710 feet. This means that an overburden averaging 15 to 20 feet in depth will have to be removed from the site prior to structural excavation for the basins.

The conceptual mass grading may remove approximately 840,000 bank cubic yards of material. The conceptual structural excavations may remove approximately 385,000 bank cubic yards of additional material. Excavated cut slopes for mass grading are generally not likely to exceed 3:1, while the structural excavations will likely be shored and ultimately backfilled against the structures. In the event of inadvertent discovery of archaeological materials during excavation, an inadvertent discovery plan outlining construction protocols and notification procedures will be followed.

Native materials may be used on site for some backfills, berms, and shaping, but most of the excavated material will need to be hauled off site. The total volume anticipated is approximately 1,225,000 cubic yards. Structural backfill will generally be imported granular material. The required volume of imported granular material will be approximately 87,000 bank cubic yards.

Preliminary geotechnical information is summarized in the *Geotechnical Data Report* in Appendix F (RhinoOne Geotechnical, 2020). The site has layers of soft sand and silt varying between 20 to 40 feet depth that is generally saturated with perched groundwater. For some structures, such as the clearwell, excavation may be deep enough to get to a sound foundation material. In other cases, it may be necessary to over-excavate the soft material, or to provide foundation improvements such as piles or stone columns, to prevent settlement of major structures. The presence of perched groundwater at 20 to 40 feet depth means that design and construction must consider dewatering for major excavations.

7.1.2 Slope and Soil Mitigation

Based on preliminary geotechnical information at the filtration facility site summarized in Chapter 4, potential geotechnical issues include slope stability and the seismic behavior of the soft layer of sand and silt. Evaluation of seismic deformation risks is in progress and will continue during design. The soft water-saturated layer of sand and silt is expected to be present 20 to 40 feet under some major structures. For project definition, ground improvements or deep foundations (piles) soil were assumed to be necessary for major structures and overflow basins. For potential slope hazards, a setback of approximately 200 feet from the crest of the steep slope northeast of the site above SE Dodge Park Boulevard has been identified as the primary mitigation. Further evaluation of the slope setback and soil mitigation will be continued during design.

7.1.3 Overflow Basins

Facility overflows from operation and maintenance (O&M) activities and emergencies will be managed by two overflow basins—the NW Basin and the SW Basin. While unlikely, if a facility overflow occurs, the overflow volume will be recycled to the head of the filtration facility for volume recovery. Table 7-1 summarizes overflow basin volume recommendations for both 160 and 240 million gallons per day (mgd) facility capacities. When determining the overflow basin to build initially, the following should be considered:

- Overflow basins will likely be considered dams due to their storage volume and depth, which may make permitting future modifications difficult (state dam safety statutes and requirements apply to storage of 3 million gallons [MG] or more and dams 10 feet or more in height—both thresholds have to be met).
- Locations of the basins may constrain the construction of future capacity.
- Basin pumping facilities will likely need to be modified for future facility expansion, due to likely depth and dimension changes.

Based on the discussion in Chapter 4, it is recommended to build the overflow basins for the ultimate design capacity to avoid additional cost related to future expansion of the basins and to provide operational flexibility.

Table 7-1. Recommended Overflow Basin Volumes

Overflow Basin	160 mgd	240 mgd
NW Basin	16 MG	16 MG
SW Basin	3 MG	5 MG

For project definition, a 16 MG NW Basin and a 5 MG SW Basin are assumed. During design, the following items should be further evaluated:

- Dechlorination of discharge
- Safe embankment design (dam permit)
- Overflow control
- Access ramp for maintenance
- Drainage pipe from NW Basin to SW Basin
- Pump from SW Basin to NW Basin and facility inlet

7.1.4 Stormwater

This section identifies code requirements for onsite stormwater management and describes the conceptual stormwater system design. Construction of the filtration facility will increase the impermeable area of the site compared to the existing agricultural use. Stormwater management will need to conform with Multnomah County requirements. The following information summarizes the *Stormwater Conceptual Design Technical Memorandum (TM)* in Appendix H (Akana, 2020).

Code Requirements

The facility site is zoned Multiple Use Agricultural (MUA-20) in the "Rural Plan Area" west of the Sandy River. The site is currently used as a nursery and is planted with grasses, shrubs, and small trees. A stormwater system in not currently in place for the site or the general area.

The Multnomah County Zoning Code (MCC 39.6235) requires that "the system shall be designed to ensure that the rate of runoff for the 10-year 24-hour storm event is no greater than that which existed prior to development at the property line or point of discharge into a water body." The stormwater system will be planned and sized to handle the difference between the existing site stormwater runoff volume and the post-construction stormwater runoff volume. The stormwater system will meet the *Multnomah County Design Standards Design Manual Part One* Section 5, and MCC 39.6235.

The conceptual design does not increase the volume of stormwater runoff onto the Multnomah County right-of-way on SE Carpenter Lane or SE Dodge Park Boulevard (the only County right-of-way within the project area). If the design increases the stormwater discharged into a County right-of-way, a 25-year 24-hour storm event calculation would be used for stormwater design in that area. Additional permitting requirements per *Multnomah County Road Rule Standard* Section 26.000 would then need to be incorporated into the design for that specific discharge. Figure 7-2 shows the existing flow pattern of stormwater runoff from the site.

Figure 7-2. Anticipated existing stormwater runoff based on site topography

Stormwater and Overflow Basin Evaluation

Stormwater detention basins can be combined with facility overflow basins or kept separate. The facility overflow basins will generally contain process water and rainwater that will eventually be pumped to the head of the filtration facility. Stormwater runoff from impervious surfaces and landscape areas has the potential to be contaminated with chemicals and motor oils. These stormwater contaminants are not recommended to be introduced to the filtration treatment process. Therefore, it is suggested that the stormwater basins be physically separated from the overflow basins so that the treatment process does not receive direct stormwater runoff. The civil design should include green infrastructure to limit surface runoff contaminants from reaching the stormwater basins.

Conceptual Stormwater System Design

The stormwater system will focus on capturing, detaining, and releasing stormwater from the added impervious areas from the site in conformance with the latest Multnomah County design standards. The stormwater management concept is based on the *Multnomah County Design Standards Design Manual Part One* Section 5, and MCC 39.6235. The stormwater system will be planned and sized to handle the difference between the existing site stormwater runoff volume and the post-construction stormwater runoff volume.

The following stormwater system design assumptions were used:

- A 10-year 24-hour storm event with a rainfall of 4 inches (NOAA Atlas 2, Volume X, Isopluvials of 10-year 24-hour precipitation map).
- Infiltration is highly unlikely based on current geotechnical review showing soils of low permeability.

Figure 7-3 below shows the conceptual stormwater design layout described in the following sections.

Figure 7-3. Stormwater conceptual design

Stormwater Collection System

The conceptual site layout creates approximately 686,100 square feet (sf) of new impervious surfaces with 370,800 sf of roadway and 315,300 sf of structures. For project definition, it is assumed that roadways are shed sections without curb and gutter, and that stormwater from roads will be collected in a roadside ditch and culvert network. There may be sections within the site that include curb and gutter/catch basin systems as part of the final design determination.

Stormwater from structure roofs, between buildings, and parking lot catch basins will be collected in downspouts and trapped in catch basins to remove solids.

The facility area will drain towards the northwest and southwest of the site, while a portion of the solids handing area will drain towards the southeast.

Stormwater Detention Basins

The conceptual site layout creates approximately 686,100 sf of impervious surfaces with 1,571,700 sf of landscaping. An additional 509,300 sf will be open tanks and will not contribute to the stormwater post-development area. The new impervious surfaces generally include circulation roads, parking areas, roofed structures, and other hardscaped areas.

Stormwater calculations were based on the 10-year 24-hour Santa Barbara Urban Hydrograph in accordance with County code. To comply with the *Multnomah County Stormwater Management Code*, stormwater detention basins need to be sized so that the post-development flow released from the 10-year 24-hour storm would be equal to or less than the pre-development flow. Table 7-2 provides a breakdown of the modeling calculation results.

Table 7-2. Stormwater Modeling Calculations[a]

Location	West Basin	Southwest Basin	South Basin	Total
Pre-Development				
Pervious Area (sf)	1,109,100	1,045,100	612,900	2,767,100
Max Flow (cfs)	33.6	32.8	20.0	—
Post-Development				
Pervious Area (sf)	488,800	591,100	491,800	1,571,700
Impervious Road Area (sf)	190,950	97,300	82,515	370,765
Impervious Structure Area (sf)	99,150	211,600	4,585	315,335
Open Tank (sf)	330,200	145,100	34,000	509,300
Max Flow (cfs)	26.6	31.9	19.3	—

a. Assumptions:

Conceptual design is based on an ultimate capacity of 240 mgd.

Flocculation/sedimentation basins, filters, filter-to-waste tanks, washwater equalization tanks, recycle tank, clarifiers, and overflow basins are assumed to be pervious and not contributing additional stormwater volumes.

Predevelopment CN value for areas is 85. Post-development CN value for the paved area and roofed structures is 98 and for the landscaped area is 86. Pervious areas within a basin boundary are assumed to have the same CN.

The stormwater calculations for the conceptual site plan determined the stormwater flow rate of the post-development conditions is less than the pre-development conditions. Therefore, stormwater detention is not required for the conceptual site design. The reduced post-development stormwater flow is due to additional stormwater being captured by the open process tanks and overflow basins. In other words, the total stormwater captured by the open basins removes more stormwater than what is added by the impervious area of the post-development conditions.

Because of the large impact that the open process tanks and basins have on this calculation, the volume of post-development compared to pre-development runoff will be sensitive to certain design choices such as:

- Roofed or open process tanks (e.g., filters and flocculation/sedimentation basins)
- Size of overflow basins
- Extent of ecoroof technology
- Total developed impervious area

Smaller process tanks, roofed process tanks and filters, and/or smaller overflow basins, may increase the post-development runoff to the point where stormwater basins will be required. Therefore, space has been reserved on the conceptual site plan for three potential stormwater basins in case stormwater facilities are required as design progresses. If basins are required, discharge locations and their release patterns need to be located to follow the current natural water flow to avoid changes in waterway alignment and prevent erosion. If during design, post-development flows are greater than pre-development, a metering discharge structure such as a weir or orifice plate will be constructed at the outlet of each stormwater basin to control discharge rates to match the existing pre-development flow rates. The location and flow path of the outlet structure should have an armored discharge point (with riprap) to prevent erosion and help direct flow. The soil on site is high in clay content, limiting infiltration. Infiltration testing should be completed prior to design to determine soil infiltration capability.

Stormwater Summary

The conceptual stormwater design includes allocated space for three stormwater basins to provide flexibility during design. The stormwater management system for the site ultimately depends on the total impervious and pervious area of the design. The initial calculation based on the filtration facility concept shows no net increase, primarily because of the large area of open tanks and basins that do not contribute to stormwater runoff. Because of the large impact that the open process tanks and basins have on this calculation, the volume of post-development compared to pre-development runoff resulting from the final design will be sensitive to certain design choices:

- Roofed or open process tanks (e.g., filters and flocculation/sedimentation basins)
- Size of overflow basins
- Extent of ecoroof technology
- Total developed impervious area

Smaller overflow basins or process tanks, or covered process tanks and filters, may increase the post-development runoff to the point that stormwater basins will be required.

7.1.5 Onsite Wastewater Management

The site requires a septic system to handle domestic wastewater, since urban services (normally sanitary sewer and water service) are not allowed in rural areas per the Oregon State Planning Goals 3, 11, and 14. At this time, there is no sanitary sewer system developed or planned for the area. The closest system is in the City of Gresham urban growth boundary approximately 3.5 miles to the west and is sized to support development in the expanded growth boundary only. Connection to Gresham's sanitary system is not viable since the site is:

- Outside of the urban growth boundary and consequently not serviceable.
- Not within a future expansion area outside the growth boundary.
- Fails to meet criteria for sanitary sewer service.

This wastewater design section includes a summary of the conceptual onsite wastewater design system and recommendations discussed in the *Septic System Conceptual Design TM* in Appendix I (Akana, 2019). The septic tank and drainage field sizing and design are based on DEQ Chapter 340, Division 71, Onsite Wastewater Treatment Systems, the preliminary geotechnical report, and current site conditions. The filtration facility will generate the following wastewater, which is anticipated to be disposed of as follows:

- Sanitary, shower, and kitchen wastewater will discharge to the septic system.
- Process wastewater will be recycled to the head of the filtration facility.
- Laboratory wastewater, floor cleaning (chemical/solids buildings), online instrument reagents, and eye wash wastewater are anticipated to be conveyed to a holding tank that will be pumped off site. Discharge must comply with MCC 36.7990. Neutralization may be an option and should be reviewed during design.

The septic system will be planned and sized to accept domestic sanitary sewer and kitchen waste flows. The daily sewage flow has been estimated based on the following:

- Shift employee count of 18 (maximum 24 FTEs).
- OAR 340-071-0220 Table 2, Quantities of Sewage Flows, Factories with showers listing of 35 gallons per capita per day. The 35 gallons per capita per day has been applied to all FTEs.

The estimated daily sewage flow for the site using these assumptions is 840 gallons per day (gpd). The following section discussed conceptual sizing of each wastewater component used in the cost estimate.

Septic Tanks

There will be sanitary facilities in several buildings at the facility, including:

- Administration Building
- Maintenance Building
- Chemical Feed Building
- Solids Handling Building

While the Administration Building will house the majority of domestic wastewater flow production, toilet facilities and hand wash sinks are proposed for each building and will be connected to the septic system. Laboratory sampling, eyewash, and emergency shower facilities are not anticipated to flow into the sanitary system, but will instead be conveyed to a holding tank that will be pumped off site. The laboratory hand and dish wash facilities will be connected to the septic system.

Based on the topography and the conceptual site grading, the septic system area is anticipated to be located on the north portion of the site adjacent to SE Carpenter Lane and east of the access drive. The Chemical and Solids Handling buildings are anticipated to include small holding tanks (500 gallon). A pump chamber (either separate or inclusive to the tank) would be installed at the Chemical Building to pump wastewater from the Solids Handling and Chemical buildings. A large (1,500 gallon) tank with a pump chamber would be constructed at the Administration Building to handle the flow from this building and the nearby Maintenance Building. The conceptual septic design layout is described in the following sections.

Wastewater Piping

Piping from the Solids Handling Building should be installed in a gravity flow condition. Gravity flow from the Chemical Building may be possible depending on final building elevations, either to the Administration Building or directly to the recirculating wastewater filters. However, at this time, it is projected as pumped flow to minimize the depth of the septic system. Piping from the Maintenance Building to the Administrative Building would be by gravity flow.

Wastewater Pumps

A variety of pumps are available for these types of installations. While the flow is anticipated to be mainly liquid, solids capability should be anticipated. Head conditions will be low, so high-head pumps will not be required. A standard float tree would be installed to manage on and off cycles and a local control panel would be installed with a warning light in case of power interruption. It is suggested to include a local disconnect and direct power plug, or else include the system in the emergency power system in case of a power outage.

Recirculating Wastewater Filter and Tank

DEQ Chapter 340, Division 71, Onsite Wastewater Treatment Systems is prescriptive on the type of recirculating filters for onsite septic systems (OAR 340-071). The bottom area of the filter should be sized based on the maximum organic load during design. The conceptual gravel filter design is rectangular with a total length of 40 feet, a total depth of 3 feet 9 inches, and a total width of 10 feet. The vertical cross section (top to bottom) will consist of a 3-inch distribution pipe zone, a 3-foot gravel filter zone, and a 6-inch collection pipe zone. The distribution pipe zone will contain multiple perforated pipes running in parallel, spaced evenly in the distribution zone. The collection zone contains a couple of perforated pipes centered between the above distribution pipes. The wastewater will be pumped into the gravel filter through a recirculation/mixing tank and cycled through the dosing tank within the recirculation tank before being released into the drainfield.

Dosing Tank

A dosing tank will be provided after the gravel filter to meter flow into the drainfield. A portion of the recirculated wastewater will be released to the drainfield. The remaining wastewater will recirculate back through the recirculating filter. The tank will require a float control switch system to activate the pump or discharge valve, and a high-water alarm panel should be installed outside of the tank.

Drainfield

Site soil condition information was collected from the USGS Estimated Depth to Groundwater Study of the Portland Metro Map, existing well logs, and recent geotechnical investigations in the area and on the site as described in the *Geotechnical Data Report* in Appendix F (RhinoOne Geotechnical, 2020). This data indicates the groundwater table is likely more than 150 feet deep. Perched groundwater was encountered at the filtration facility site during geotechnical boring at a depth of 25 to 33 feet. It is assumed the site consists of Soil Group C. The topsoil ranges from 1 to 2 feet and consists of a silt clay till zone from previous onsite agricultural operations. The soil layer below the topsoil ranges from 20 to 33 feet deep and is comprised of red-brown silty clay with traces of fine sand. The clay is medium stiff to stiff and has a high plasticity. At a depth of 25 to 33 feet BGS, a loose sand layer starts and continues to a depth of up to 100 feet BGS. Percolation testing has not been performed; however, due to the soil conditions, the infiltration rate is expected to be low. The design of the filter and drainfield should be based on percolation tests performed in the area designated for the drainfield.

A drainfield conceptual design has been developed based on DEQ 340-71. Based on the expected lack of permeability available, a mound system is recommended for the site with a single, perforated drainpipe centered in the mound. The mound is proposed to consist of a lower sand base, discharge gravel layer, upper sand layer, and topsoil cap.

7.1.6 Site Access and Traffic Management

The site access routes and associated traffic management will be developed based on PWB's preferred site access locations presented in Chapter 4. For purposes of cost estimating at this conceptual stage, two access locations are assumed, one from the north and one from the south. The northern access route is assumed to be for staff vehicles, public access, and emergency vehicles. The southern access route is assumed to be for general delivery trucks, chemical delivery trucks, solids handling trucks, and other large vehicles. Onsite roadways shown are conceptual and will be finalized by the designer. The major goals for the road system should consider the following:

- Primary and secondary entrances.
- Parking for facility staff, tours, and the public.
- Access and road grades suitable for delivery trucks.
- Onsite circulation for operations vehicles, maintenance vehicles, and a tour bus.
- Site security and access.

7.1.7 Site Development Summary

This section summarized site development considerations for the filtration facility design. Key considerations for project definition include:

- **Site Grading and Earthwork.** Mass excavation is required to achieve a hydraulic profile capable of gravity flow. Careful site layout during design is critical in minimizing the amount of soil removed from the site.
- **Slope and Soil Mitigation.** A minimum setback of critical facilities from the crest of the steep slope northeast of the site and soil mitigation will need to be further reviewed by the designer.
- **Overflow Basins.** Facility overflows from O&M activities and emergencies are anticipated to be managed by overflow basins sized for ultimate capacity. This avoids future modifications of these basins that will likely considered to be dams due to the storage volume and depth.
- **Stormwater:**
  - Space is allocated for three stormwater basins in the conceptual site layout; however, the initial calculation shows no net increase in stormwater from the filtration facility, primarily because the open process tanks and basins do not contribute to stormwater runoff.
  - The volume of post-development compared to pre-development runoff resulting from the final design will be sensitive to certain design choices, including:
    - Roofed or open process tanks (e.g., filters and flocculation/sedimentation basins).
    - Size of overflow basins.
    - Extent of ecoroof technology.
    - Total developed impervious area.
  - Smaller overflow basins or process tanks, or covered process tanks and filters, may increase the post-development runoff to the point that stormwater detention basins will be required.
- **Onsite Wastewater Management.** Onsite sanitary facilities are required to handle domestic wastewater because urban services are not allowed in rural areas. The wastewater management system includes a septic system, holding tanks, and potential for neutralization. The system will need to be further reviewed by the designer.
- **Site Access and Traffic Management**. Two site access routes are assumed. Site access to the filtration facility will need to consider right-of-way, permitting, safety, and community feedback and will be further coordinated with PWB during design.

7.2 Utilities

This utilities section describes the design features of the conceptual site plan related to utilities and provides design guidance for utilities at the filtration facility site. This section includes discussion of the following topics:

- Site Piping
- Power Supply
- Instrumentation and Controls
- Utilities Summary

7.2.1 Site Piping

This section describes support systems for the facility design related to site piping. This section includes discussion of the following topics:

- Potable Water
- Fire Suppression
- Natural Gas
- Raw Water and Finished Water Connections
- Yard Piping Connections
- Utility Relocation and Existing Easement Modifications

Potable Water

Potable water to supply a facility-wide distribution system can be pumped from the clearwell or from the finished water pipeline (after the water has achieved its full disinfection contact time). A distribution grid will need to be installed at each major building and process basin for washdown and other needs. The designer will size the system based on anticipated demands at each building and structure. Redundant potable water pumps should be provided. It is likely that hydropneumatic tanks will be required to sustain pressure in the system.

Fire Suppression

The fire suppression system can use raw water instead of potable water to avoid oversizing potable water pumps and the distribution system. Diesel engine-driven pumps can be used for reliability in a power outage. Fire suppression piping should be routed to each building and major structure. Buildings will likely be sprinklered, while hydrants should be set at locations accessible to emergency equipment. Use of raw water will require strainers and screens at the point of pumping. Raw water can be pumped from the inlet basin or from the raw water pipelines.

Natural Gas

Natural gas is available along SE Carpenter Lane. The available capacity for the facility is based on a feasibility study initiated by PWB contacting NW Natural. The cost estimate assumes the facility heating needs are met using natural gas instead of electric. However, the design should consider the use of non-fossil fuel sources for the initial installation. Moreover, it is suggested that if the facility design includes fossil fuel-sourced heating systems, that should not preclude future conversion to non-fossil fuel sources.

Raw Water and Finished Water Pipeline Connections

The filtration facility site work will include connections to the new raw water and finished water pipelines. Two raw water pipelines and two finished water pipelines are assumed. The assumed connection locations are identified in the *Basis of Estimate Report* (Appendix B).

Yard Piping Connections

The connections from the yard piping to the structures will require a degree of flexibility. A typical approach is to provide a pipe spool with two flexible connections, harnessed to restrain against thrust and seismic motion. If movement and deformation are too great for this approach, a flexible ball-joint fitting may be needed. Seismic deformation and shaking movement will be further evaluated by the designer.

Utility Relocation and Existing Easement Modifications

A survey of existing utilities at the filtration facility site is anticipated. The known existing utilities at the site include 12-inch-diameter Pleasant Home Water District pipelines, Portland General Electric (PGE) utilities, and nursery irrigation pipelines (Figure 7-4 below). The conceptual design shows the solids facilities within the area of the existing water pipelines and associated easement, so these utilities will likely need to be relocated. Relocation will depend on the design. A PGE easement is shown on the western property boundary and may need to be adjusted depending on the final location of the finished water pipelines shown in the conceptual site plan. The nursery is anticipated to remove their irrigation pipelines when their lease ends. The irrigation equipment, pipelines, and associated existing easements along the southwest property boundary may provide irrigation water to nearby private property and are anticipated to remain as-is. Improvements to this area along the southwest property boundary should be avoided since this area is within the water resources buffer based on proximity to the Johnson Creek Headwaters.

The facility access routes are currently in review (Chapter 4). Depending on the preferred access, easements may need to be obtained.

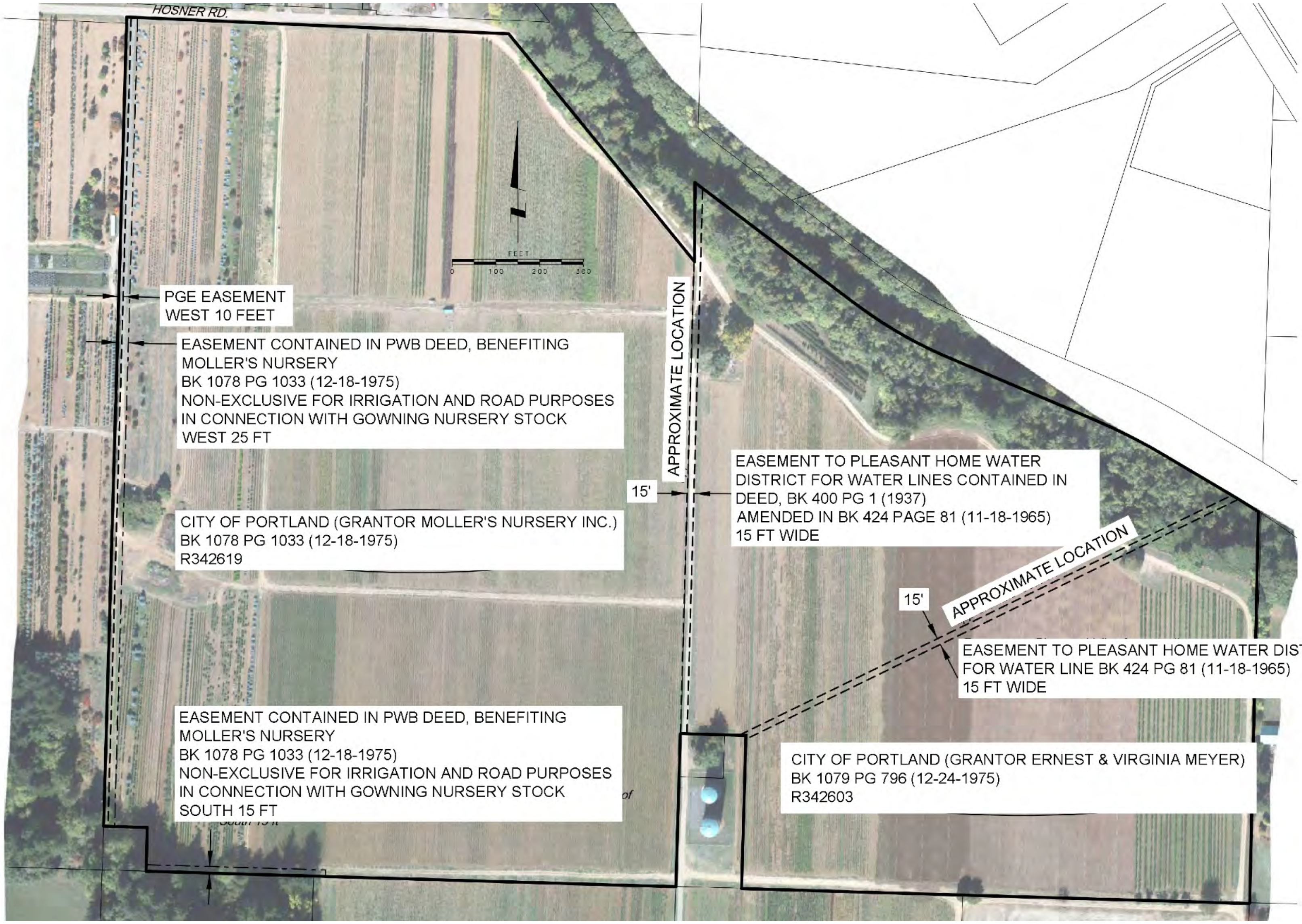


Figure 7-4. Existing utility and easement locations

7.2.2 Power Supply

The following sections describe electrical supply considerations to provide for efficient distribution of power at the filtration facility, sufficient redundancy to manage risk, and flexibility to accommodate future load growth needs.

This section includes discussion of the following topics:

- Supply Source
- Standby Power
- Site Power Distribution
- Solar Power

The recommendations for PGE power sourcing for this facility as well as onsite standby power are as described in the following section. The National Electrical Code (NEC) Article 708 has electrical requirements for facilities judged to be critical operations power facilities by the authority having jurisdiction. The Oregon Electrical Specialties code has specific amendments to the 2017 NEC that assign responsibility for classifying what falls under the definition of critical to the owner. Full facility compliance with NEC 708 would be cost prohibitive; therefore, the suggested approach is to review the requirements and identify specific equipment, systems, or areas of the facility to which these requirements should apply. This may include conducting an overall facility risk assessment to identify what level of risk mitigation is warranted. Since there are several electrical distribution choices relating to the level of facility criticality and defined risks, it is recommended that the risk assessment be conducted early in the design.

Supply Source

PGE is the serving power utility company for the area where the filtration facility will be located. PGE's Orient power substation is approximately 1.5 miles west of the site on the southeast corner of SE Dodge Park Boulevard and SE Altman Road (Figure 7-5 below). A distribution line runs east from the substation along SE Dodge Park Boulevard to within a few hundred feet of the facility site. This will be the primary power source for the facility.

The estimated peak day power demand for the facility design capacity of 160 mgd is 4 megawatt (MW). Table 7-3 below shows anticipated facility loads based on current information. These loads are conservative to identify worst-case electrical equipment, space, and cost scenarios. Based on these facility loads and considering the potential for future facility expansion to up to 240 mgd, it is recommended that power service be taken at the PGE distribution service voltage, which is listed as 13 kV nominal and assumed to be 12.47 kV.

The facility electrical planning covers the potential worst-case power demands. As such, the loading assumptions vary in some cases from overall equipment assumptions made elsewhere. For example, while liquid oxygen delivery or bulk delivery of hypochlorite may be assumed for purposes of facility layout, onsite oxygen generation and hypochlorite generation is assumed for purposes of worst-case power demand. It is also more than likely that the actual facility loads will be less than what is tabulated here due to equipment operating less than nameplate ratings and the diversity of simultaneous operation of loads. The projected loading for the 240 mgd facility capacity is estimated to be 4,700 kilowatts peak load and 4,200 kilowatts standby peak load. These conservatively derived values are expected to lower when design advances.

The 12.47 kV voltage level will allow for more efficient distribution of power in the filtration facility while providing a robust electrical backbone that covers flexibility for load growth. PGE has provided a preliminary rough order of magnitude cost of $600,000 for this primary line service to the filtration facility.

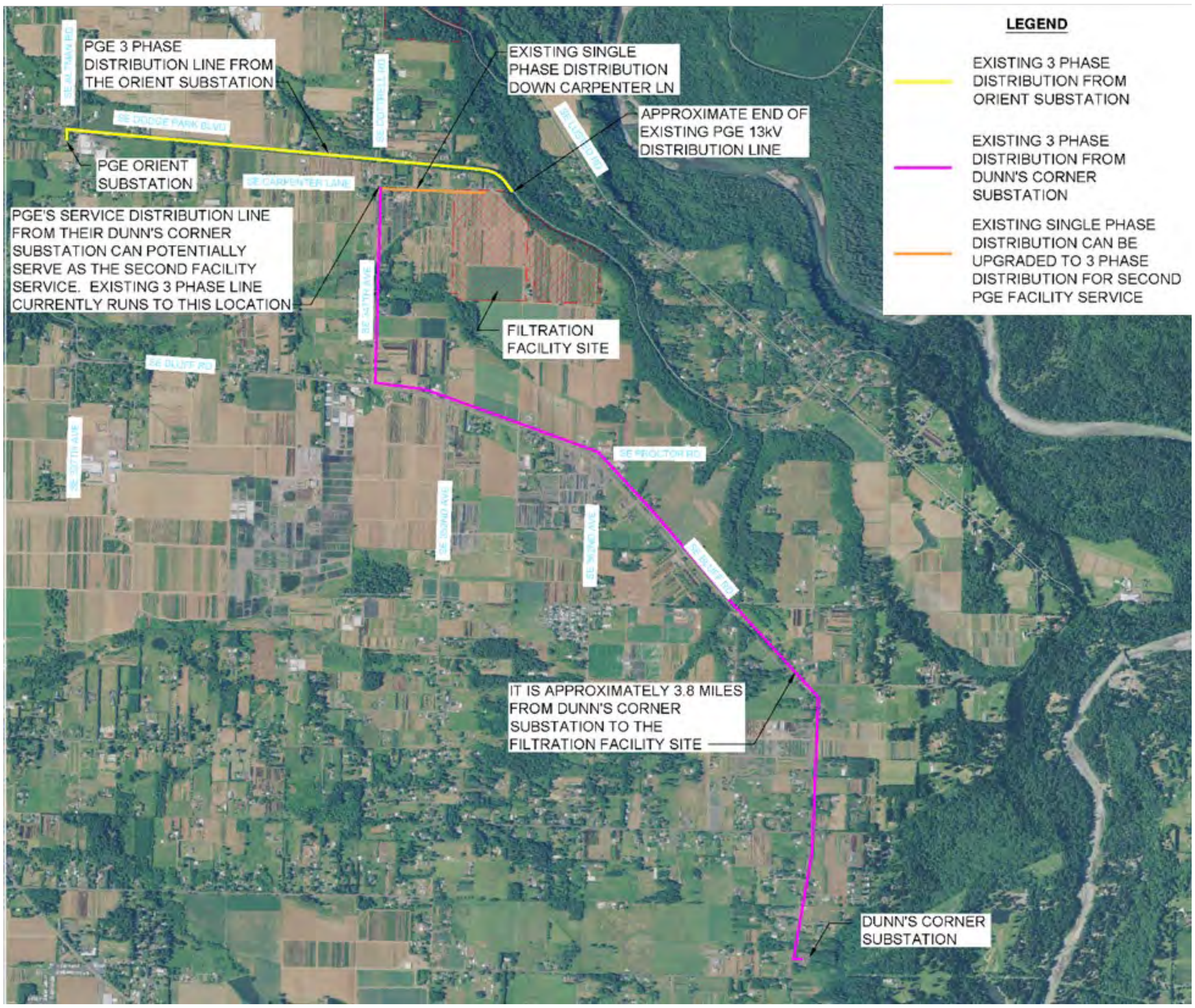


Figure 7-5. Existing nearby PGE power substations and distribution lines

A further option to consider with respect to PGE service is whether a second power line service is desirable. The decision to include a second power utility company source is generally based on criticality of service. Data centers, as a rule, have dual power services and standby generators. For this facility, PGE has indicated that a second line service is available for a preliminary estimated cost of $1,025,000, which covers the cost of extending this second line service plus an initial one-time assessment to cover the cost of reserve line capacity. According to PGE, the second line service will be sourced from their Dunn’s Corner substation, which is located approximately 4 miles from the site near the intersection of SE Dunn Road and SE Bluff Road. The distribution lines from this substation exist along SE Cottrell Road at the intersection with SE Carpenter Lane.

Table 7-3. Facility Loads Estimate (160 mgd) (kilowatts)

Facility Area	Process Equipment	Lighting/ Receptacle Loads	HVAC[f]	Misc Loads	Subtotal Estimated Load Peak	Subtotal Estimated Load Standby
Ozone[a, b]	1000	5	40	25	1,070	1,045
Rapid Mix	22.5	1.5	—	2	26	18.6
Flocculation	120	2.5	—	2	124.5	92.3
Sedimentation	12	2.5	—	2	16.5	11.3
Filtration[c]	825	5.5	—	35	865.5	845.3
Chemical Feed[a]	30	30	45	35	140	88.8
Hypochlorite Generation[d]	600	—	—	—	600	600.0
Solids Handling[a, e]	120	10	42	40	212	146.5
Recycle	37.5	1	—	1	39.5	29.1
Filter-to-Waste	11	2.5	—	5	18.5	12.0
Washwater Equalization	37.5	2.5	—	5	45	31.9
Overflow	190	—	—	—	190	142.5
Administration Building	—	20	214	80	314	210.5
Maintenance Building	—	5	32	100	137	76.5
Main Electrical/Generator Building	—	6	24	30	60	36
Electrical Building	—	2	10	15	27	16
Galleries	—	14	88	17	119	81.5
Estimated Total Facility Load (kW)					**4,004**	
Estimated Total Facility Standby Load (kW)						**3,484**

a. Includes building loads.

b. Assumes ozone generation includes oxygen generation on site (conservative power estimate).

c. Includes backwash pumps and backwash air scour blowers.

d. Hypochlorite generation load is included for peak load calculation as a worst-case load basis as opposed to bulk delivery assumed for site layout.

e. Includes Solids Handling Building loads plus gravity thickener loads and thickened solids tank loads.

f. HVAC loads assume that heating load is primarily natural gas, except for galleries.

For project definition, a single incoming service to the filtration facility, with backup provided by onsite standby power generators for reliability, is assumed. The conceptual electrical layout provides flexibility for accommodating either one or two incoming PGE services along with onsite generation. The ultimate decision as to what level of power redundancy and capacity is required and when, can therefore be made in design without incurring as significant impact to cost and space. The suggested facility risk analysis could look at reliability and quality of service of the two PGE sources versus the power capacity and onsite fuel storage requirements for standby generators to help determine what combination would be most appropriate for the expected facility level of service needs.

The recommended configuration for the facility main switchgear is a 12.47 kV dual line up in the Main Electrical/Generator Building. Major distribution switchgear and motor control centers will be configured as dual bus for reliability and flexibility for safe maintenance. Dual substations to transform voltage from 12.47 kV to 480 volts utilization service will be located at the Main Electrical/Generator Building, the Solids Handling Building, and the Electrical Building (Figure 7-6 below).

Standby Power

The estimated standby power requirement, based on current loads information, is 3.5 MW to allow the 160 mgd facility to run at peak capacity upon loss of utility power. The basis for this load is included in Table 7-3. The NEC classification for this standby power will be NEC Article 702 Optional Standby Systems. This classification assumes facility standby power is not used to meet legally mandated or critical life support requirements. This distinction of standby power classification is electrically significant as it avoids the need for onerous electrical protection requirements. Note that if any or all of the filtration facility is declared “critical” per the definition of NEC 708, then the generators classifications should be changed in tandem with that classification.

The suggested voltage for standby power generators is 12.47 kV. This will enable connection of these generators to the 12.47 kV main facility switchgear for consolidation of main utility and standby power in the Main Electrical/Generator Building. The recommended configuration of these generators is paralleling operation both between the generators and between the generators and the PGE incoming service. The paralleling capability to the PGE source will facilitate implementation of PGE’s Dispatchable Standby Generation program described below, as well as allow for facility load to be used for generator exercising. This consolidation of standby power at the 12.47 kV level means that standby power is available throughout the site. Rather than installing local generators at each facility with the inherent maintenance for that distributed generation, the critical power maintenance will be largely centralized. Standby power consolidation will also facilitate starting large loads on the generators. While this is the baseline configuration for project definition, it is understood that distributed low-voltage generators and local load banks for exercising generators may be a preferred option by PWB maintenance staff. If this is the case, it is suggested this be reviewed early in design since this will impact space requirements for buildings with substation-level distribution.

Two 1.75 MW generators will meet the 3.5 MW standby backup power capacity. These will be configured to run in parallel via the 12.47 kV main switchgear. The Main Electrical/Generator Building, which houses these generators, will have available space to accommodate generators up to 2 MW each if greater capacity is required for future facility expansion. In addition, space is reserved to allow for the building to be expanded in the future if a third generator is added. Bulk fuel tank capacity for operation of two generators at full load for 5 days would be 30,000 gallons. This is an initial estimated capacity that can be reassessed during design based on criticality of service needs and assessment of fuel availability following the worst-case failure of service scenario. Load banks for generator exercising are cumbersome at this capacity so the recommended standby power layout assumes that facility load can be used for generator exercising.

PGE offers a Dispatchable Standby Generation program where they support significant cost associated with standby generators for the benefit of having dispatchable access to that generation if supplemental power is needed on their grid. Although PGE is not accepting new capacity now, they have indicated that they will be looking for new subscribers during the project timeframe. It is suggested that potential subscription to PGE’s Dispatchable Standby

Generation program be further evaluated during design. Note that if the facility is subscribed, the Dispatchable Standby Generation program will address exercising needs for generators.

Site Power Distribution

The conceptual site electrical distribution for the filtration facility is shown in Figure 7-6 below. The main incoming service from PGE will enter the facility from a PGE power pole located on SE Dodge Park Boulevard. Service will be extended to the Main Electrical/Generator Building, which will be the primary power distribution center for the filtration facility. From this location, 12.47 kV dual distribution feeders will serve 12.47 kV to 480 volt distribution substations located at an electrical building near the filters and at the Solids Handling Building. For project definition purposes, it is assumed the substations will be outdoors, which is generally the standard arrangement for treatment facilities due to the cost of building space and the need to address the transformers heat rejection. If consideration is to be given to indoor substations, it is suggested this be addressed early in design due to the impact to building space layout requirements.

The Electrical Building will house 480 volt distribution that can serve the ozone building, washwater equalization loads, clearwell loads, filter-to-waste loads, south overflow basin pumps, and electrical building loads. The Solids Handling Building will have outdoor 12.47 kV to 480 volt substations to accommodate power needs in the Solids Handling Building and surrounding area, including the recycle tank loads, clarifiers, gravity thickener loads, thickened solids tanks loads, and solids handling pump station. The Chemical Feed Building, rapid mix units, flocculators, sedimentation loads, filters loads, Administration Building, Maintenance Building, north overflow basin pumps, and the Generator Building loads can be served from low-voltage distribution equipment in the Main Electrical/Generator Building.

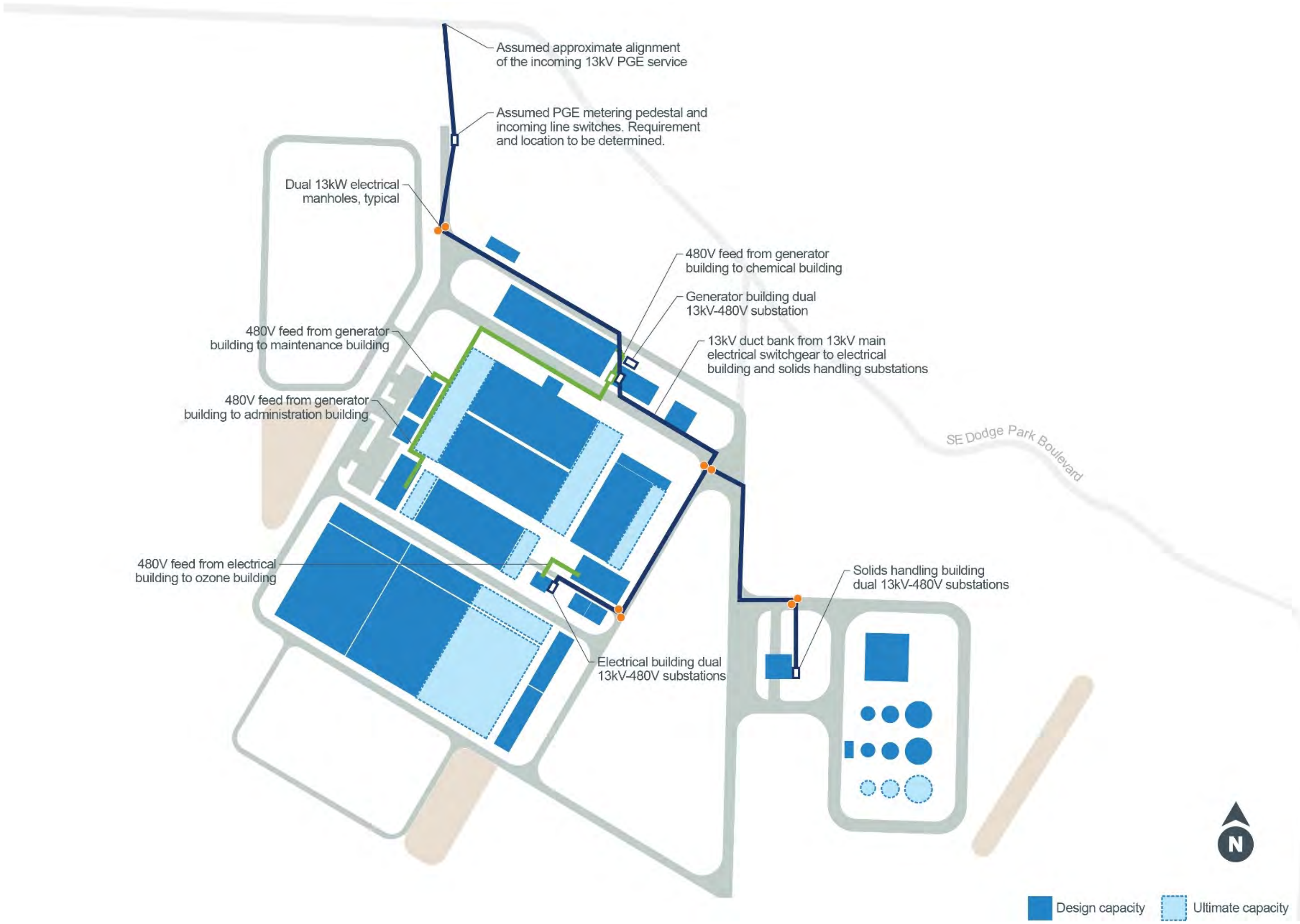


Figure 7-6. Electrical distribution

Solar Power

A conceptual site analysis was conducted to evaluate the possibility of using photovoltaic power generation (solar) at the filtration facility site. The *Solar Analysis Technical Summary* is included in Appendix J and summarized as follows.

Solar analysis was conducted using the National Renewable Energy Laboratory System Advisor Model software to simulate different options. The following four conceptual array options were reviewed:

- Option 1: Administration Building rooftop space only, 229 kW direct current (DC)
- Option 2: Clearwell rooftop space only, 2.2 MW DC
- Option 3: East side of site only, 5.4 MW DC
- Option 4: Maximum solar array that combines options 1, 2, and 3, 7.8 MW DC

Figure 7-7 summarizes the results of the analysis of energy production potential for each scenario compared to the estimated energy consumption of the facility (shown in MW-hours).

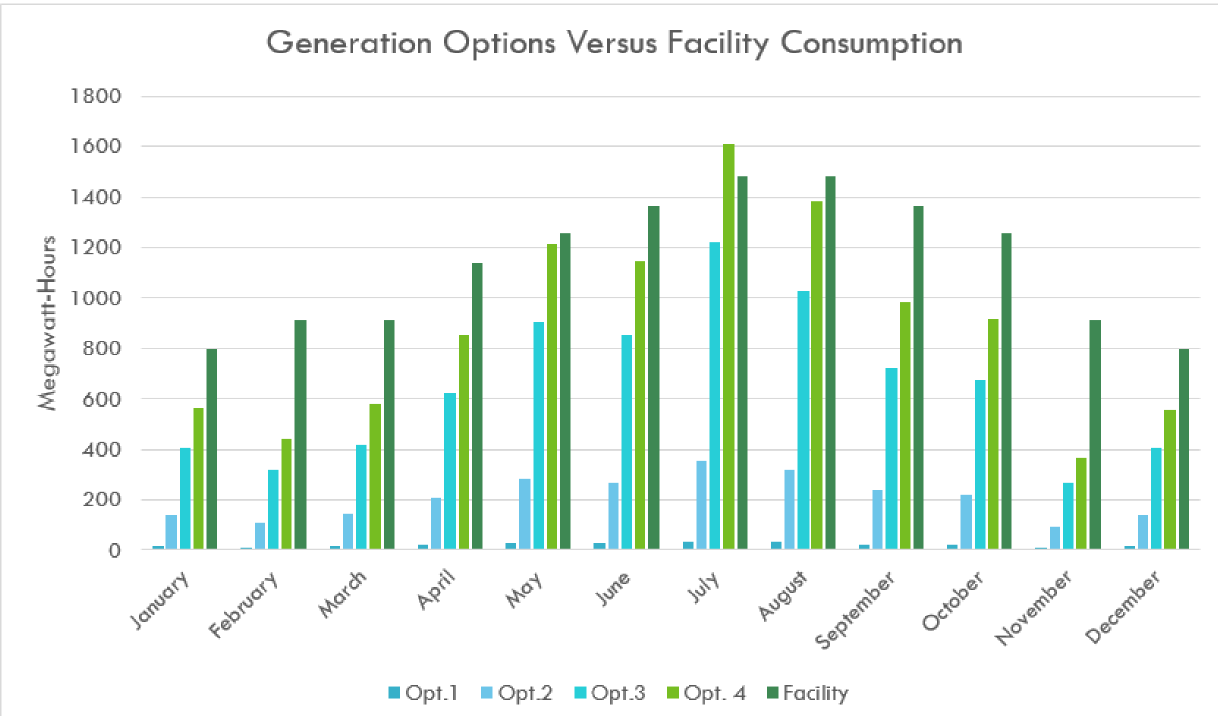


Figure 7-7. Summary comparison of four solar array installation scenarios at the site

Based upon the analysis, an array installed on the clearwell space shows the greatest potential for the facility and would allow for maximum use of PGE's net metering schedule. The size of the array brings the cost for power generated down to a level that while above conventional power cost, will reach parity with the utility in the future.

When comparing the static costs of solar generation versus escalated costs of conventional power (assuming a 2.5 percent escalation rate), solar generation reaches approximate parity with conventional power between years 15 and 20 depending upon the installed array size.

If the intent is for the filtration facility to use a greater percentage of renewable energy sources, then further design effort can consider gaining the most energy density, reducing allocated space, and reducing costs through design choices.

7.2.3 Instrumentation and Controls

PWB's existing supervisory control and data acquisition (SCADA) and instrumentation and control (I&C) systems are well established. The filtration facility will be integrated with existing systems to support effective operation. The following sections describe the existing systems and how these systems will be modified and expanded as needed to include the filtration facility.

This section discusses the following topics:

- SCADA
- Telecommunications
- Information Management
- Site Security

SCADA

The filtration facility will be fully integrated with PWB's existing SCADA system. SCADA provides secure and reliable real-time data monitoring, collection and archiving of historical data, and remote manual and automatic control of various distributed facilities. SCADA consists of centrally located file servers, local and remote computer workstations, field located remote terminal units (RTU), and programmable logic controllers (PLCs) maintained by PWB. This equipment is connected across PWB's system by a wide area network (WAN) kept physically separated from the City's business networks.

The Interstate Water Control Center (WCC) houses the primary SCADA file servers connected by a local area network (LAN). These servers, currently running Aveva OASyS software, include redundant real-time data servers (i.e., polling, alarming, and short-term trending), a decision support server (i.e., long-term trending and business intelligence), a web server (web client sessions for City network clients), and a data server (access to historical data for City network clients). WCC also stores program and data files on a shared disc array. A Backup Control Center (BCC) at Lusted Hill provides a subset of the servers and workstations at WCC for redundancy should the Interstate Building equipment fail. The BCC includes direct links (bypassing WCC and microwave radio systems) to the field-located RTUs and PLCs so that SCADA will remain operational without the WCC.

The BCC equipment (SCADA backup servers and master polling radios) is assumed to be relocated to the filtration facility and integrated with the facility's SCADA system. Direct links to the field-located RTUs and PLCs will also be moved to the filtration facility. During design, the Bureau of Technology Services (BTS) will conduct a line of site survey for new tower and fiber network options to inform further evaluation of relocating existing BCC equipment to the site.

Telecommunications

The SCADA servers at WCC and BCC are connected via LAN to multiple design and operator workstations running OASyS Human-Machine Interface (HMI) software and via WAN at other key PWB locations where operations and engineering staff remotely monitor operation of the water distribution system. RTU hardware at water facilities such as reservoirs and pressure reducing valves provide the remote interface to monitor levels, flows, and valve positions through SCADA. PLC hardware at more complex facilities such as Headworks and pumping stations provide automatic control functions as well as SCADA connectivity.

The SCADA WAN consists primarily of fiber optic cables managed by BTS. This includes City-owned Integrated Regional Network Enterprise fiber, as well as fiber owned by Comcast and PGE. In addition to SCADA, these fiber optic links include the City's business network managed by BTS. Within each facility, BTS maintains the business LAN while PWB maintains the SCADA LAN. Microwave radio systems, maintained by BTS, provide a redundant control network path to Headworks as well as links to PWB's master polling radios at elevated locations around the system.

PWB manages SCADA telecommunication technologies to RTUs via licensed master polling radios, unlicensed spread-spectrum radios, and leased telephone lines. Typically, RTU equipment is connected via licensed master polling radios. However, some four-wire leased telephone lines are still in use. SCADA polls each RTU in turn, and the RTU responds with the requested information. Phone lines communicate at 1200 baud and radios at 9600 baud.

For cyber-security, PWB encrypts radio and SCADA WAN traffic. Firewalls are used at sites to monitor and segment SCADA traffic. Cyber-security for telecommunication systems will rely on existing BTS policies to provide multiple layers of protection through a combination of physical, electronic, and procedural safeguards. SCADA network security will be based on the International Electrotechnical Commission 62443 (International Society of Automation 99) and National Institute of Standards and Technology 800-82 standards.

Information Management

The SCADA system is used primarily to monitor operation of distributed facility sites and to provide alarming, trending, and data acquisition through RTUs. At sites where complex automatic control functions are required, such as the filtration facility, PLCs are used. PWB has standardized on redundant Schneider Modicon M580 or M340 controllers programmed via the latest version of Unity software. These controllers will communicate via MODBUS protocol within the SCADA system. Where equipment using MODBUS transmission control protocol/internet protocol (TCP/IP) communication is not available, MODBUS RTU or MODBUS RS-485 serial may be used. HMI displays will be provided on main PLC control panels to assist with troubleshooting and provide backup control capabilities when the SCADA WAN is not available. PWB has standardized on Schneider Magelis series displays programmed with Vijeo Designer.

Site Security

The site security system will provide access control, intrusion detection, and video surveillance. The following security systems and measures are conceptual and are suggested for further review by the designer:

- Exterior building doors will have door contacts to indicate when a door is open. These contacts will input to the access control system.
- An intrusion detection system will consist of motion detectors, door contacts, and other intrusion detection devices as required and developed during the design process.
- Access to exterior building doors and PWB-indicated interior building doors will be via electronic access control (proximity card readers and cards or fobs). Exterior and interior doors indicated by PWB will also include CyberLock electronic lock cylinders. Restrooms for public meeting areas will need to be reviewed and should try to avoid separation by electronic access control doors.
- The security system will be Genesis, the new City of Portland standard. This system, along with related hardware, will be maintained by City Facilities Services. Cameras will be maintained by BTS Comm/Net.
- Security fence gate systems and measures:
  - Powered gate operators will provide primary access to the site; gates will operate with the same electronic access control system as the building access control. Card readers located outside the fence will permit entry by verified cardholders.
  - For visiting vehicles or deliveries by non-employees, a two-way intercom will provide communications from outside the gate to inside the facility. Gates will be operable remotely by onsite staff or security personnel as determined during design.
  - Sensors will provide automatic gate operation for exiting vehicles.
  - PWB’s Security Group will administer access through these gates.
  - The security system will be backed up by a UPS system, which in turn will be backed up by standby generator power, thus providing continuous power to security systems.
  - Card readers will be on the same system and programmable such that a staff person can access areas they are authorized to enter using a single card.
  - Auxiliary access gates without card readers will be padlocked with CyberLock electronic lock cylinders. The designer should carefully review with PWB the intended use, as card readers are preferred by users, and the cost savings is maximized when electronics are installed during facility construction as compared to retrofitting.
- A closed-circuit television (CCTV) system will consist of “pan, tilt, and zoom” and fixed cameras supplying video to monitors at the Administration Building and may consist of the following video coverage:
  - CCTV cameras will cover the gate area with coverage to be determined by PWB during design. The cameras should cover the vehicle driver, license plate, and gate approach, or a combination of the above for security and verification.
  - CCTV cameras will also be used for facility operations. The type, number, and location of cameras will be determined during design in close coordination with PWB security

and operations staff. The camera locations will need to include accessibility for future repair or replacement, such as using a boom truck or other means.

  - PWB's security personnel will control CCTV cameras. Remote monitoring locations will allow personnel to view the cameras at locations to be determined during design.
  - CCTV cameras will be internet protocol cameras. Cameras will be specified with BTS.
  - Video recordings will be stored on a network-based server for viewing through Genesis and will be maintained by City Facilities Services. The current video system is Pelco Video Expert. Depending on the timing of filtration construction activities and the Genesis system rollout, it may be necessary to accommodate some aspects of the existing Pelco system at the filtration facility to support seamless operations with other system facilities, such as Headworks and Lusted Hill.

- Further discussion will be needed during design to determine the type of security facilities around the site perimeter and evaluate alternatives, such as a potential "Clear Vu" type of product, that would be in keeping with the site's visual aesthetics while also protecting against attempts to breach the site.

7.2.4 Utilities Summary

This section summarized considerations for utility support systems at the filtration facility.

Key considerations for project definition include:

- **Site Piping:**
  - A redundant potable water supply system pumped from the clearwell or finished water pipeline is anticipated to serve each major building and process basin.
  - The fire suppression system is anticipated to use raw water to provide fire suppression at each major building, major structure, and throughout the site using diesel-driven pumps in case of a power outage.
  - Availability of natural gas along SE Carpenter Lane will need to be coordinated with NW Natural during design.
  - Facility site work is assumed to include connections to new raw water and finished water pipelines.
  - Yard piping connections to structures will need to be flexible to address seismic deformation and shaking and are suggested for further evaluation by the designer.
  - There are minimal existing utilities at the site, but some will require relocation and associated easement modifications.
- **Power Supply:**
  - Incoming power supply from PGE is anticipated to be a 12.47 kV distribution service.
  - Main incoming service is from the PGE Orient substation with an optional second service from PGE Dunn's Corner substation.
  - Two standby power generators will provide 3.5 MW at 12.47 kV for filtration facility backup power if PGE service is lost.

   - The filtration facility 12.47 kV main switchgear will be located in the Generator Building with redundant 12.47 kV feeders routed to treatment process locations and stepped down to 480 volts via local substations.
   - Various solar power options were assessed at the filtration facility and are suggested for further evaluation with PWB during design.
- **Instrumentation and Controls:**
   - SCADA. The SCADA system will be fully integrated with PWB's existing SCADA system.
   - Telecommunications:
      - Primary telecommunications to PWB and other City systems will be via redundant fiber optic pathways managed by BTS.
      - Backup telecommunications to critical PWB facilities will be via microwave links also managed by BTS.
      - Telecommunications to some remote PWB sites will be via master polling radios managed by PWB.
      - The business network will be managed by BTS and the process network will be managed by PWB.
      - Telephones will be voice over internet protocol.
   - Site Security:
      - Security systems will include access control, intrusion detection, and video surveillance.
      - Security systems will be managed by City Facilities Services.
      - Exterior doors and security gates will be monitored and access controlled via cards or fobs.
      - Interior doors identified by PWB will also be access controlled.
      - Doors and gates identified by PWB will also have CyberLock electronic keys.

7.3 Architectural Programming

Architectural programming is used to determine the functional and operational requirements for each building and facility at the filtration facility. This includes identifying the required facilities at the site, spatial requirements, adjacencies, staffing, building materials, and aesthetic considerations. This section describes the process of developing the overall architectural program, then focuses on each building and facility program.

This section includes discussion of the following topics:

- Architectural Approach
- Facility Adjacencies and Site Program
- O&M Staffing
- Building Materials
- Acoustics
- Lighting
- Odor
- Non-Process Building Concepts
- Architectural Programming Summary

7.3.1 Architectural Approach

The architectural program was developed using several sources, including review of similar buildings at existing water treatment facilities, building code requirements, sustainability goals, Portland green building standards, treatment process and operational information, and a Facilities Workshop on March 26, 2019. Applicable building codes, Portland's *Green Building Policy*, and Energy Trust of Oregon incentives are further described in Chapter 5: Design Considerations.

At the Facilities Workshop, the following goals for the filtration facility were discussed:

- Provide safe and high-quality drinking water to customers
- Be good stewards of ratepayer funds
- Provide a good and functional place to work
- Design for more than 50 years of operation
- Consider future expansion
- Provide a safe, secure site that is welcoming to visitors
- Be a good neighbor
- Provide a positive public interface
- Design for safety, security, and resilience

These goals served as a guide to develop the architectural program. The Facilities Workshop also identified types and capacity of new facilities at the site, initial programming information, preferred site adjacencies, and other factors that will influence design. Example floor plans were then developed using the initial room sizes and adjusted considering the overall program for each building.

The overall site design should both minimize the visual impact of the filtration facility and complement the rural character of the area. Methods for achieving this will include the character of the structures on the site, as well as use of landscaping and site topography. It is suggested that site design be informed by ongoing input from the community.

The site has naturally hilly terrain, and it is suggested to consider use of the topography in the site layout and landscape design to potentially screen views of facility structures. The natural topography could be augmented with berms, and structures could potentially be built at or below grade to lessen the visual impact of the facility.

The design of site structures is also suggested to use elements in keeping with local structures in the area and to respond to local environmental needs. Generic industrial boxes are to be avoided. The height of structures on the site should be kept as low as possible while allowing for operational needs. It is suggested that during design, photo simulations of structures be used to model view lines and be shared with site neighbors as part of project outreach.

Landscaping can both screen the facility and augment the natural beauty of the site. A landscape buffer will be incorporated at the edges of the site. It is suggested the designer consider the site edges in development of a cohesive landscape design of green rolling hills interspersed with groves of trees in keeping with the character of the site. Landscape material and plantings will be determined during design and should be complementary to the existing area vegetation.

7.3.2 Facility Adjacencies and Site Program

Adjacency requirements were identified to improve facility circulation and ease of O&M. The following is a list of adjacency considerations for developing the site program:

- Layout should support core filtration facility functions and maintain staff safety and site security.
- The control room should be in the Administration Building adjacent to and with a bird's eye view of the filters and, if possible, facility entrances.
- The shop area in the Maintenance Building should be near the control center and administrative area in the Administration Building to limit redundant spaces (i.e., lockers and breakroom) and to promote face-to-face communication and team building.
- The Maintenance Building should be near the Chemical Feed Building .
- The Chemical Feed Building should be near chemical-intensive unit processes such as flocculation basins, the clearwell, and solids handling; this could be a centralized building with pumped feed lines to day-use tanks, or multiple buildings.
- Chemicals should be stored inside a building except for liquid oxygen or compressed gas, which should be stored outside.
- The Chemical Feed Building should be convenient for delivery truck access and allow for multiple, simultaneous chemical deliveries.
- Electrical buildings/rooms should be near equipment with high electrical loads, including ozone generation and filter backwash pumps.
- Facilities should be oriented to minimize potential visual (day and night) and noise impacts.

- Overflow basins should be at lower elevations than unit processes to promote drainage and prevent potential flooding of process units or buildings in the event of an overflow.
- Truck delivery routes and activities should limit impacts to community roadways.
- Tour movement on the site should avoid truck delivery activities.
- Site design should support guided educational tours via driving and walking.
- Site design should provide for a convenient circulation pathway for O&M staff.
- A means to walk from one process location to another in inclement weather should be provided.
- Corridors should be considered for accessible placement of utility and chemical piping, along with electrical conduits to reduce dependency on buried utilities.

7.3.3 O&M Staffing

The filtration facility will require a staffing plan and support facilities for O&M. PWB uses separate trades for O&M activities, and support facilities will need to serve both trades. The staffing analysis assumes a 160 mgd peak day capacity facility. PWB operators will staff the filtration facility 24/7 while maintenance staff will work daytime shifts Monday through Friday. There will also be a need for laboratory and administrative support at the filtration facility usually during the daytime shift Monday through Friday. The staffing numbers are intended for planning purposes and will be updated during design.

The staffing levels and types of support facilities described in this chapter are based on a February 2019 Staffing Workshop with PWB staff and a survey of other utilities to gather information on staffing structures and processes, and the types of work facility staff perform.

To understand how other water utilities are staffing their treatment facilities, a questionnaire was sent to more than a dozen water treatment facilities in the Pacific Northwest. The questions included:

- How are your shifts and staffing set up?
- How many staff are on each shift? In what roles?
- Maintenance full-time equivalents (FTEs): mechanical/instrumentation and controls
- Operations FTEs: superintendent/supervisor/operators
- Do you contract out any laboratory services? If so, which services?
- Do you contract out any O&M services? If so, which services?

The information from responding utilities relative to staffing numbers at other water treatment facilities is shown in Table 7-4 below.

Table 7-4. O&M FTEs per mgd

Utility[a]	Capacity (mgd)	(M) Mechanical	(M) Instrumentation & Controls	(M) Subtotal FTE	(OPs) Superintendent	(OPs) Supervisor	(OPs) Operator	(OPs) Subtotal FTE	Total FTEs O&M	FTEs per mgd
A	65	2	1	3	1	1	10	12	**15**	**0.23**
B	75	2	1	3	1	1	10	12	**15**	**0.20**
C	100	—	—	1	1	1	7	9	**11**	**0.11**
D	120	4	—	4	—	—	10	10	**14**	**0.12**
E	70	—	—	—	—	1	6	7	**7**	**0.10**
F	476	9	—	9	1	1	20	22	**34**	**0.07**
G	120	1	1	2	1	1	4	6	**10**	**0.08**
H	143	8	—	8	1	1	8	10	**18**	**0.13**
I	150	6	1	7	1	1	8	10	**15**	**0.10**
Average	147	4	1	4	1	1	10	11	**15.4**	**0.13**

a. Survey results are presented anonymously.

(M) = maintenance group

(OPs) = operations group

Based on the information gathered from the utility surveys and the program team's experience developing staffing plans for other similarly sized facilities, it is anticipated that approximately 22 to 26 FTEs will be needed. This projection was developed using the following guidelines for staffing needs.

- **Operations:** 0.07 FTE/mgd
  - This equals approximately 10 FTEs
  - For planning purposes, it is recommended to use 12 to 14 FTEs.
    - This will account for facility management, laboratory technicians, process operators, and other staffing nuances yet to be defined.
- **Maintenance:** 0.05 FTE/mgd
  - This equals approximately 8 FTE
  - For planning purposes, it is recommended to use 10 to 12 FTEs.
    - This will account for a maintenance lead and other staffing nuances yet to be defined (landscaping labor is not included and could be provided by outside contractors if desired by PWB).
    - Also assumes overhaul and repair of large equipment will be sent off site, not completed by facility maintenance staff.
- **Total:** 22 to 26 estimated FTEs or 0.14 to 0.16 FTE/mgd

These staffing estimates should be re-evaluated during design once specific types of equipment and the overall facility layout are determined to identify if changes are needed.

The process for developing the staffing numbers and the range of FTEs was discussed with PWB O&M management during the Staffing Workshop, and it was agreed that 22 to 26 FTEs was an appropriate range and that 26 staff should be used for planning the filtration facilities. It was also noted that these staffing requirements would not necessarily result in the need for additional staff, since staff from other PWB facilities may be transferred to the filtration facility. Therefore, an assumption was made that at least 12 of the 26 staff are already employed by PWB and do not represent additional cost. Preliminary costs estimated the need for 14 additional staff; these numbers will be validated during design.

7.3.4 Building Materials

It is suggested that buildings at the filtration facility be constructed to maximize the use of long-lasting, durable, and low maintenance materials. These materials may include seismically resistant reinforced concrete masonry units, poured-in-place concrete, and steel frame structures; resin and heavy gauge metal panels for exterior cladding; and ecoroofs and heavy gauge metal roofing. The materials selection and the siting and massing of buildings should be consistent with the character of the area, reduce visual impacts, and respect the rural nature of the site. Buildings will incorporate sustainable materials where possible, and may include the following design strategies: ecoroofs, sustainable power generation, passive solar heating and cooling, natural daylighting, and energy saving measures.

7.3.5 Acoustics

The acoustic impacts of the filtration facility must be mitigated to meet code requirements and to be a good neighbor. Chapter 4 includes a summary of the baseline noise measurement, code requirements, and acoustical design criteria identified in the *Acoustic Design Criteria and Baseline Measurement TM* included in Appendix E (Greenbusch Group, 2020).

The conceptual site plan incorporates noise mitigation measures by concentrating noisier processes at the site interior and providing a setback for structures from the property line. The initial cost estimate also includes acoustic mitigation for buildings housing generators and air scour blowers and for use of sound-dampening building materials.

7.3.6 Lighting

The lighting design for the filtration facility needs to meet code requirements and provide adequate lighting to support facility operation, site security, and staff safety (emergency and egress lighting). In addition, lighting should avoid impacting nearby neighbors, and be consistent with the existing lighting environment. Design features to modulate lighting, such as the ability to turn vital lighting systems on or off from the Administration Building, are suggested for further evaluation during design.

The lighting design should evaluate methods to minimize glare, skyglow, and other forms of light pollution that may impact the surrounding community. The basic methods to reduce light pollution, as formulated by the International Dark-Sky Association, include:

- Only light areas that need to be lit when they need to be lit.
- Use lighting that is no brighter than necessary.
- Minimize blue light emissions by using lower temperature lighting sources.
- Fully shield and point lighting downward.

The Multnomah County Zoning Code includes Dark Sky standards that address these points. Potential design strategies that may be evaluated during design include low-level ambient lighting along facility walkways, switchable and/or motion-activated task and security lighting, dimmable light-emitting diode lighting, and locating light sources at the interior of the facility where they can be screened by structures (MCC 39.6850).

7.3.7 Odor

Odors are not a nuisance associated with filtration facilities and therefore no mitigations are required.

7.3.8 Non-Process Building Concepts

The sections below describe the specific programmatic requirements of the non-process buildings at the filtration facility. As discussed in the O&M staffing section, preliminary facility staffing ranges from 22 to 26 FTEs. Part-time staff may include visiting staff from PWB or other agencies who are working on projects. The building programs and the sizes of supporting areas provided in the conceptual floor plans are based on this staffing range with an assumed maximum of 18 people working at the same time on the same shift. An *Architectural Administration and Maintenance Building Programming* that identifies the approximate square footage of rooms assumed for cost estimate development is included in Appendix K (Convergence Architecture, 2019).

During design, discussions on the feasibility of the following items are suggested:

- Relocating some functions from other facilities, such as Headworks and Sandy River Station, to the filtration facility site.
- Locating satellite restrooms and/or tour facilities for combined public and staff use throughout the site.

Note that the figures and layouts in the following section are conceptual and included for illustrative purposes only. The design drawings will be developed during design.

This section includes the following content:

- Occupancy Types
- Administration Building
- Maintenance Building
- Chemical Feed Building
- Ozone Building
- Solids Handling Building
- Main Electrical/Generator Building
- Electrical Building
- Storage

Occupancy Types

For buildings containing chemical storage, the occupancy class is determined by the type and quantity of materials. The occupancy classification can affect many aspects of building design, including layout, construction materials and assemblies, ventilation requirements, and the fire suppression system and egress.

The occupancy classifications at this site are shown in Table 7-5. In general, high hazards (H) occupancies have more design requirements compared to other occupancies such as storage (S) and factory (F) occupancies.

Table 7-5. Occupancy Classification

Group	Hazard Classification	Description
Hazard		
H-2	High	Materials within are at risk of deflagration or accelerated burning.
H-3		Materials within readily support combustion. Of lower risk than H-2.
H-4		Materials within are health hazards with toxic and corrosive materials.
Factory		
F-1	Moderate	Industrial uses that include combustible materials.
F-2	Low	Industrial uses that use primarily non-combustible materials.
Storage		
S-1	Moderate	For storage of materials that are combustible but not classified as hazardous—also includes motor vehicle repair shops.
S-2	Low	For storage that is primarily non-combustible—also includes parking garages.

Administration Building

The Administration Building is the center of the non-process facilities at the filtration facility and contains many functions, including:

- Control Center
- Pilot Plant Facilities
- Laboratory
- Offices and Workspaces
- Multi-purpose Room and Conference Room
- Supporting Spaces
- Public Lobby and Educational Spaces

The upper floor of the Administration Building houses the Control Center and permits a view of the treatment processes. The upper floor provides direct access to the top of the filters from the Control Center and lobby, while the lower floor provides direct access to the filter gallery.

Control Center. The Control Center will control treatment processes at the filtration facility, communicate with the rest of PWB's system, and serve as a backup for the primary Interstate Building WCC. The backup WCC is a "virtual" environment where operators can readily switch from viewing and controlling treatment processes to viewing and controlling, if necessary, the entire water control system. The Control Center should include large control screen monitors with individual user lighting controls to adjust from dim to bright. A breakout room, supervisor office, and approximately four workstations should be within or adjacent to the Control Center (Figure 7-8 below). The SCADA and server room are shown on the first floor of the Administration Building.

A key design direction is for the control room to have visual and close physical access to the filters to facilitate efficient O&M circulation at the site. A two-story structure provides direct access from the Control Center to the top of the partially covered filters Only the operating deck where the operating consoles are located are assumed to be covered. Covering the actual filters themselves to provide a barrier from atmospheric contamination (e.g., airborne pesticides) is to be determined during design. The primary access to the top of the filters will be from the elevator lobby. A separate access will be provided for staff working in the Control Center. A stairway to the underground filter gallery and utility corridor should be adjacent to the Control Center (Figure 7-9 below). The utility corridor is anticipated to have forklift access to the filter gallery via a ramp at a separate entrance.

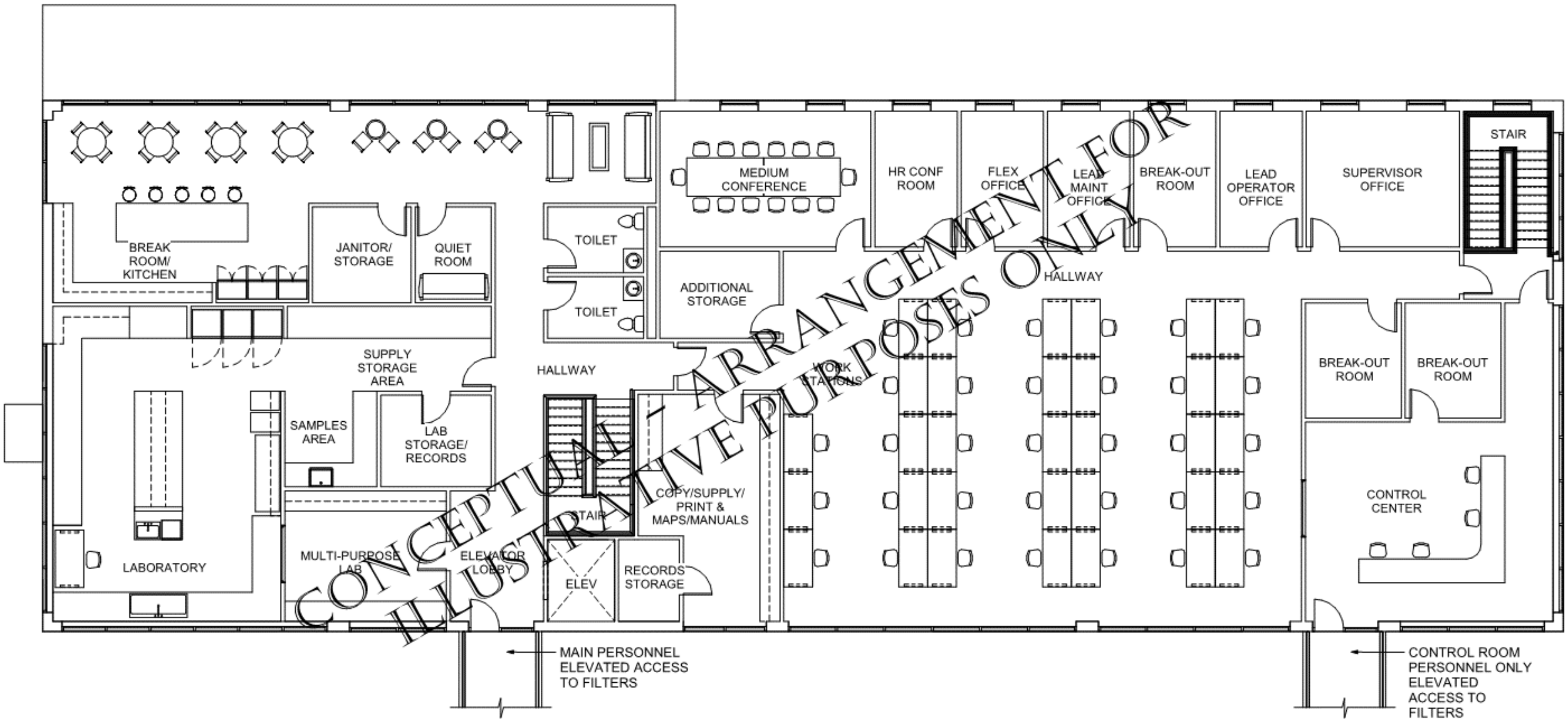


Figure 7-8. Potential concept for second floor of Administration Building

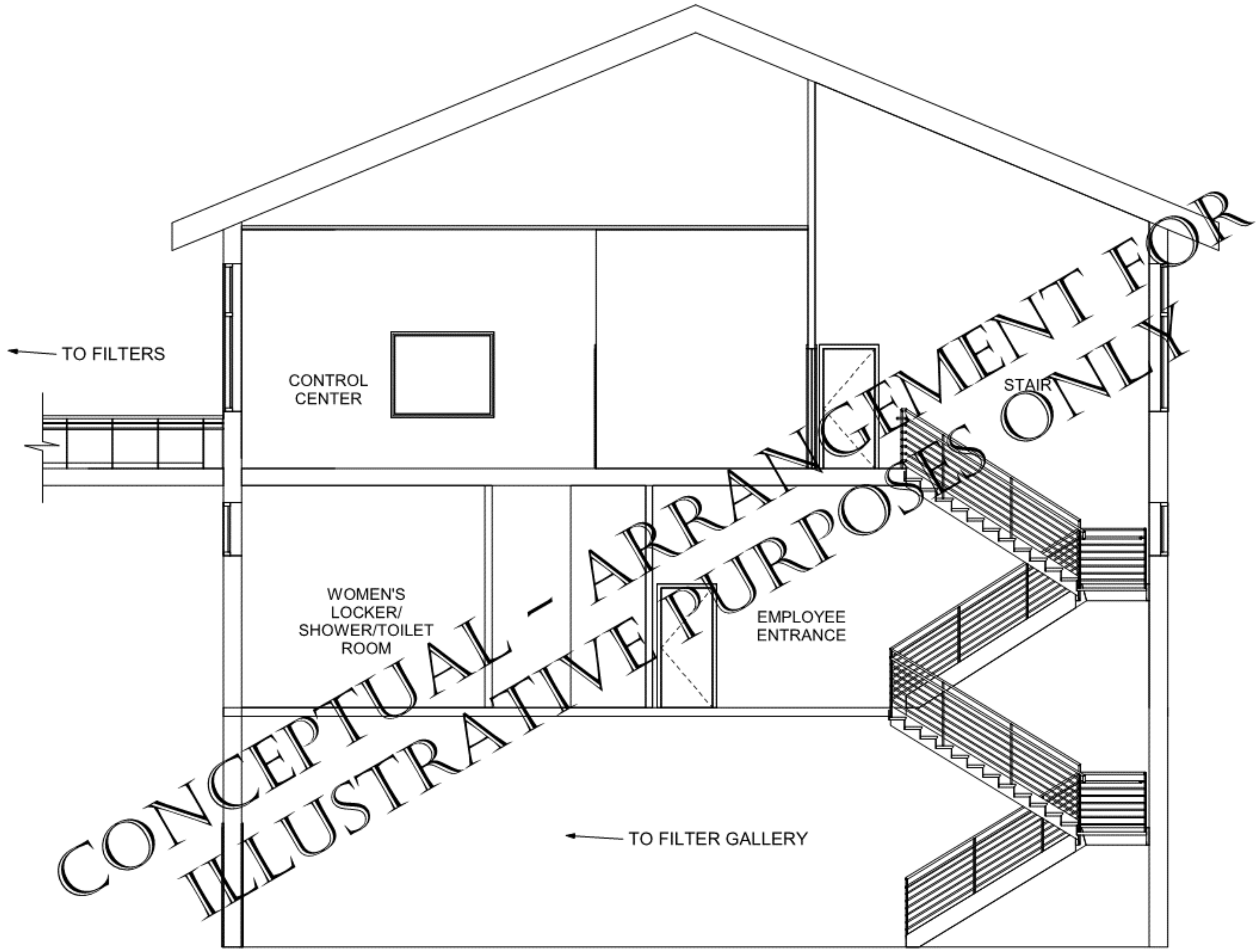


Figure 7-9. Potential concept for Administration Building section

Pilot Plant Facilities. The pilot plant facilities are currently located at Headworks and housed in several portable enclosures (Figure 7-10). The pilot facilities include dual treatment trains, flocculation/sedimentation units, pre-ozonation and intermediate ozonation units, six filtration column units, and ancillary equipment. The intent is to place the pilot plant facilities near or within the Administration Building and Laboratory at the filtration facility to allow operators easy access to these facilities.

Figure 7-10. Existing pilot plant enclosures

Laboratory. For project definition, the laboratory is assumed to be for use by facility operators to analyze process samples and will not be an accredited laboratory (Figure 7-11 below). However, the laboratory design should not preclude the ability to convert these facilities to an accredited status in the future. Samples taken at the facility laboratory will be used for routine process monitoring and adjustment. The Interstate laboratory will serve as an accredited laboratory where water quality testing procedures are conducted with the same rigorous practices suitable for reporting water quality results to the Oregon Health Authority and the public. The pilot plant facilities used during design will be relocated from Headworks to be adjacent to or within the Administration Building for easy access to the laboratory. Laboratory facilities for resource protection are not expected at the site. During design, it is suggested to further evaluate how potential educational tours may access part of the laboratory either directly or via a viewing gallery.

It is suggested the laboratory design consider a layout with a central island and perimeter bench space. Benches should be stainless steel and provided with rear drains. The central island should have upper storage shelving with a pass-through space below. The laboratory area will also need: a sink with continuous water samples for each critical process and a jar testing and Threshold Odor Number analysis area, a records room, a freezer for ice, two refrigerators (to store samples and reagents separately), storage space, dishwasher, vacuum supply system, three-compartment warewashing sink, and one vent hood. As samples will be sent to the Interstate laboratory, a dedicated counter space near the refrigerator for outgoing samples is needed. Storage shelving that will fit standard laboratory supply shipping boxes and sample coolers (27-inch depth minimum) should be provided. A separate laboratory manager office is not required, but a workstation for data entry is needed.

Offices and Workspaces. Offices and workspaces are suggested to be provided for both full-time and visiting workers. Dedicated offices for a supervisor, lead operator, lead maintenance person, and flex or future offices located near each other are anticipated for impromptu meetings and discussions. Laptop and tablet docking stations are anticipated for maintenance staff and a workstation for other staff. A private room is recommended to allow for human resources meetings. During design, additional discussions of workplace culture and future capacity are recommended to determine whether staff have dedicated or shared workstations.

Multipurpose Room and Conference Room. The Multipurpose Room provides flexibility for classroom training, large meetings, tour group presentations, and emergency response staging (Figure 7-12 below). The multipurpose room is sized to comfortably seat 40 people and provide a welcoming space acoustically and visually for large group presentations. Since operations staff will be on site 24/7, it is suggested to provide storage area to house emergency supplies in case of a disaster. During emergency situations, the Multipurpose Room may be used as temporary sleeping space—storage space with cots and other supplies should be located nearby. A separate medium-sized conference room for 15 to 20 people and huddle rooms of various sizes allow multiple meetings to occur simultaneously.

Supporting Spaces. Supporting spaces such as lockers, breakrooms, and restrooms are provided in the Administration Building. Centralizing these supporting spaces increases efficiency and supports a cohesive culture so staff in different areas of the facility will not be isolated.

Locker room size will be equally divided between female and male and provide gender neutral facilities. During design, additional discussion of potential shared locker room facilities with individual changing and shower rooms is suggested. Currently, PWB provides two lockers per person with some facilities using dividable 30-inch lockers. A mudroom and laundry facilities will provide an area for washing down boots and soiled clothing, respectively.

The kitchen and breakroom provide space for a maximum of 20 occupants with at least three refrigerators, a microwave oven, and an oven/range to support 24/7 operation. Other facilities include a lactation room, wellness rooms, exercise room, dedicated storage, janitorial room, and miscellaneous building auxiliary rooms.

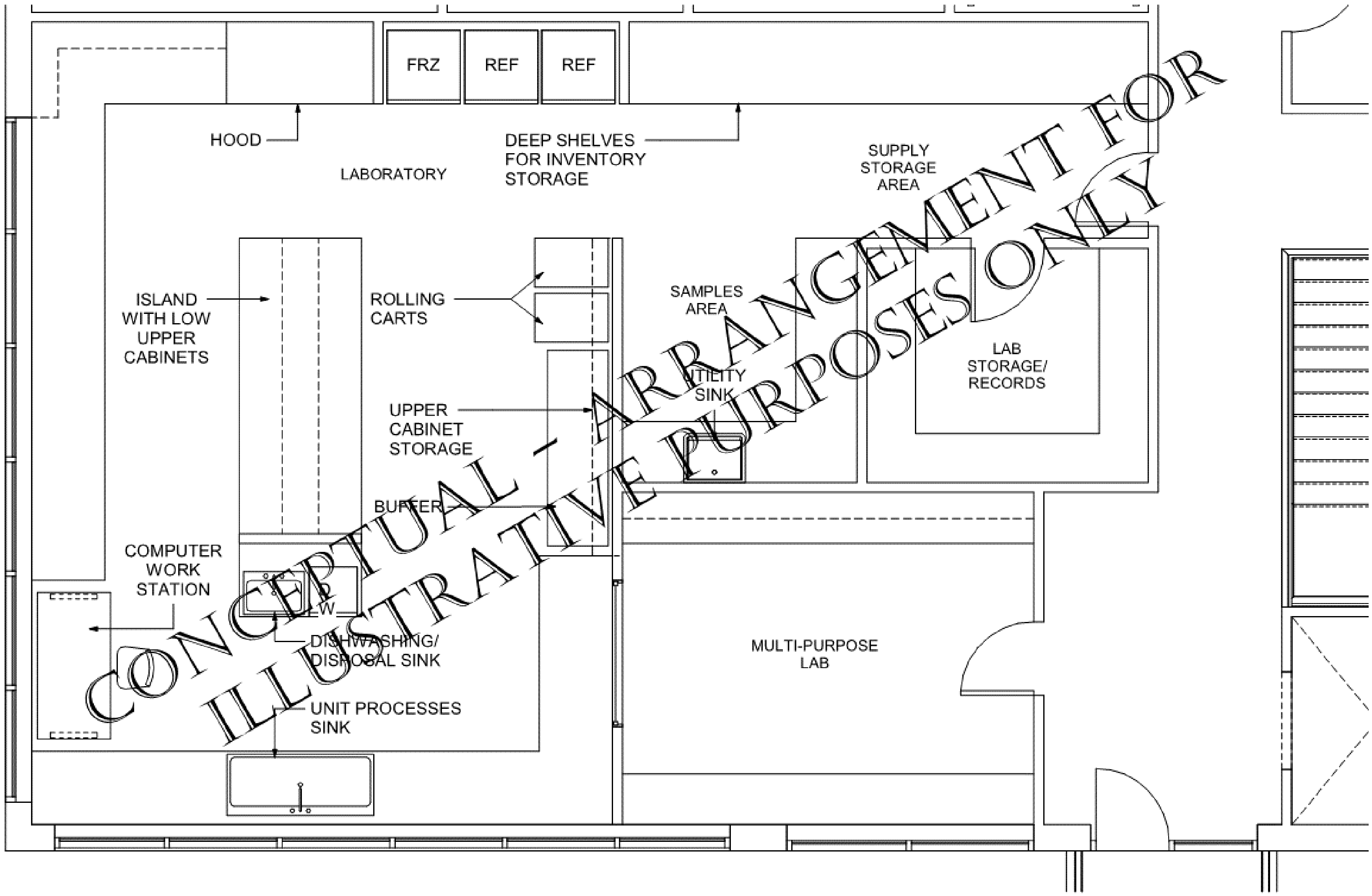


Figure 7-11. Potential concept for laboratory floor plan within Administration Building

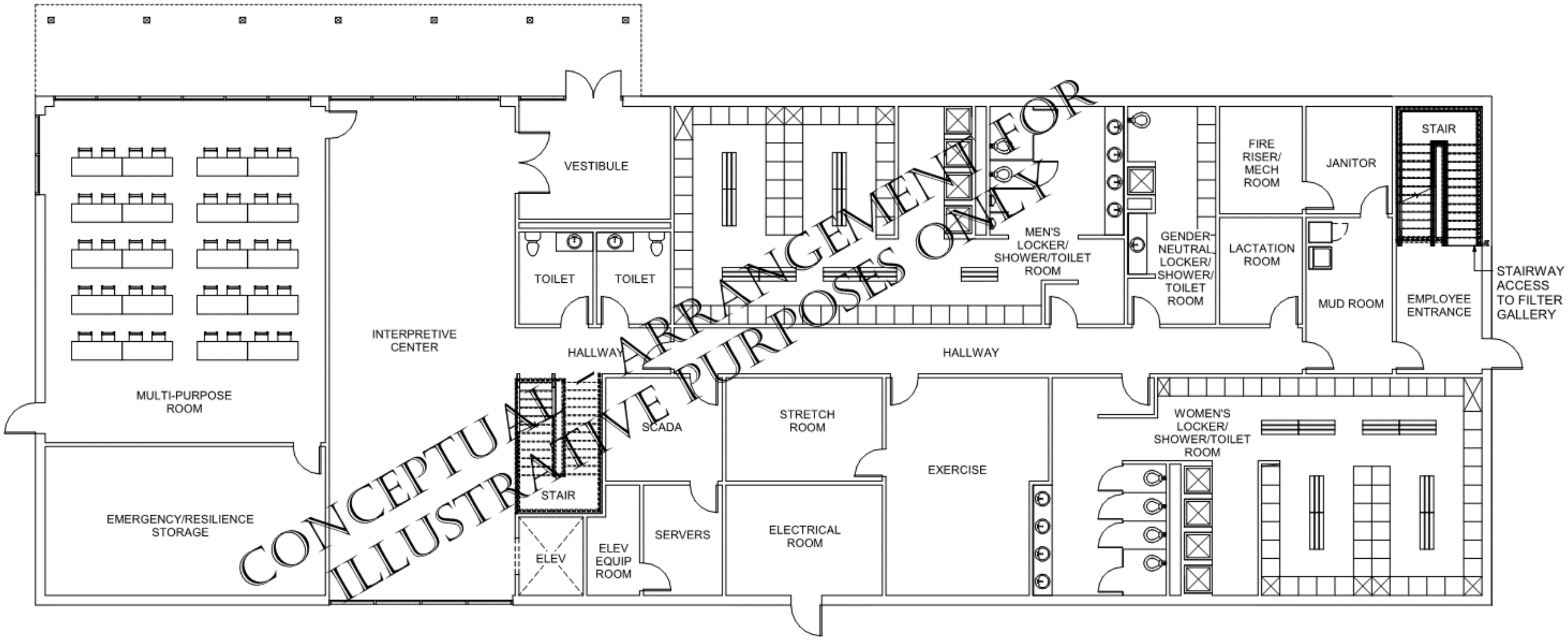


Figure 7-12. Potential concept for first floor of Administration Building

Public Lobby and Educational Spaces. Initial discussions considered the possibility of guided educational tours focused on drinking water treatment processes. During design, the possibility of both driving and walking tours should be further evaluated to allow PWB to effectively and authentically expose a variety of types of visitors to the important work that happens at the filtration facility. These tours would allow for viewing major treatment processes and would potentially include meeting in the Multipurpose Room and/or visiting the laboratory, as well as accessing the public lobby and restrooms. Consideration should be given to serving up to 40 visitors at a time.

To engage the greater community, the public areas of the facility may include educational displays and other design features related to local history and the environment. Further discussion is suggested to assess potential inclusion of publicly accessible spaces, such as nature trails, outside of the secured area of the filtration facility. During design, additional discussions are suggested to consider balancing public access, minimizing operations interruptions, and maintaining site security.

Maintenance Building

The Maintenance Building will house a warehouse, parts inventory, workshops, and facilities required to support maintenance and repair activities throughout the filtration facility (Figure 7-13 below). Most of the building will be open maintenance bays assuming that maintenance on large equipment will be performed by outside contractors. If maintenance of large equipment is to be done on site, cranes and other infrastructure may be needed in the building program. During design, additional discussion is suggested to determine the largest anticipated equipment to be repaired on site and the amount of onsite welding and fabrication. Another topic for design discussion is potential addition of a paint room or paint spray booth to the Maintenance Building. The design of these spaces should take into consideration noise that might occur during the day when maintenance staff are making repairs to equipment.

The Maintenance Building will also include a flexible shop space for minor welding, a metal shop, and a tool shop with a dedicated room for instrument repair. A dedicated air-conditioned room in a parts warehouse is suggested to store maintenance supplies and spare equipment parts. Tools will be stored in the Maintenance Building with a check-out system for tracking tools used in other structures on the site. In addition to these maintenance work areas, there will also be a shared office and workspace, restroom, small huddle/conference/break area, additional storage, mud room, and auxiliary rooms. A Level B personal protection equipment storage room with a washdown area and suit removal area is needed. The Maintenance Building is anticipated to be a work area only; therefore, locker rooms and other supporting facilities will be located in the nearby Administration Building.

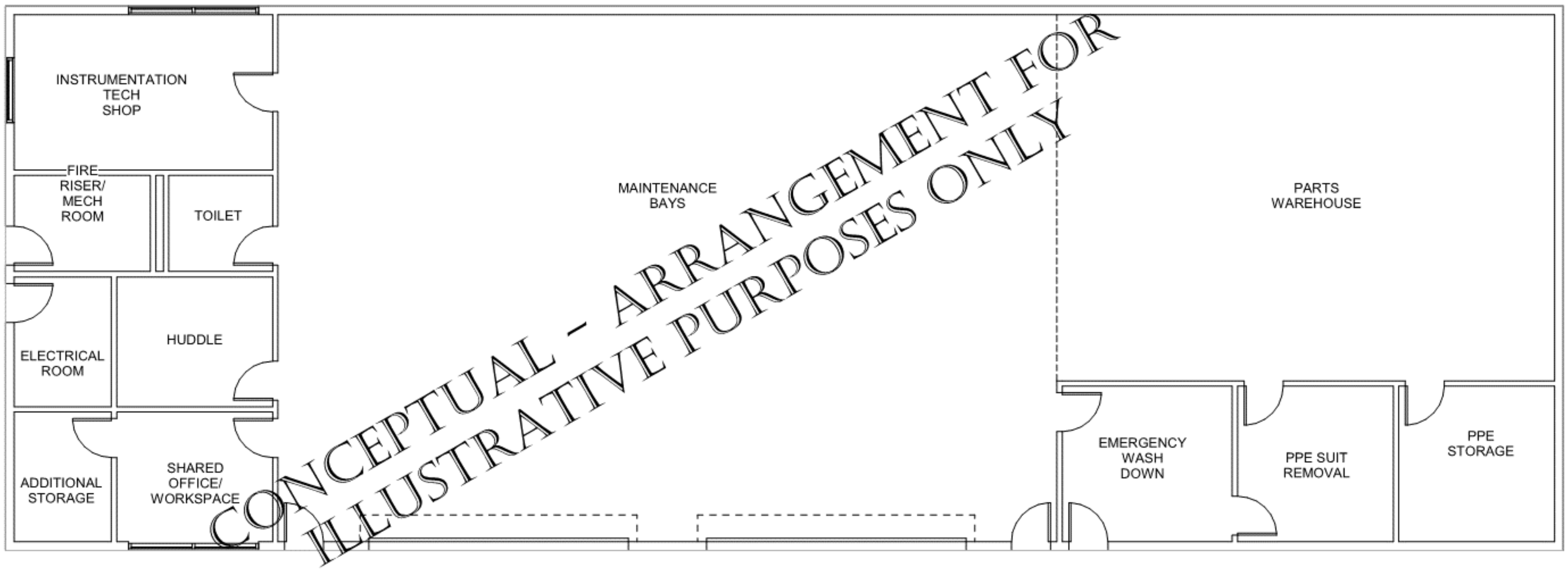


Figure 7-13. Potential concept for Maintenance Building floor plan

Chemical Feed Building

The Chemical Feed Building houses the chemical storage and feed systems for the majority of chemicals at the filtration facility with the exception of oxygen storage and ozone generation equipment (Figure 7-14 below). The following chemicals are provided as examples and assumed for project definition:

- Sodium hypochlorite used for disinfection and potentially pre-oxidation.
- Aluminum sulfate (alum) used as a primary coagulant.
- Ammonium hydroxide used for disinfection (formation of chloramines).
- Calcium thiosulfate used as a quenching agent for ozone.
- Carbon dioxide (stored outside building) used for pH adjustment for corrosion control.
- Potassium permanganate used for oxidation and/or algae control.
- Sodium carbonate (soda ash) used for corrosion control.
- Powdered activated carbon used for taste and odor control (if ozone is not provided).
- Cationic polymer used as a coagulant aid.
- Non-ionic or anionic polymer used as a filter aid.
- Space for spare chemical feed systems.
- Additional chemicals stored at the Solids Handling Building.

The approximate average demand storage volume and occupancy class information is shown in Table 7-6 below based on 30 days of storage at average chemical consumption rates or 7 days of storage at peak chemical consumption rates, whichever is greater. Incompatible chemicals should be stored in separate rooms and separate full-scale liquid spill containment should be provided for each liquid chemical. For indoor liquid containment, the containment volume is the volume of the largest tank plus 20 minutes of fire suppression water.

The Chemical Feed Building will also include a chemical control room, electrical room, office, breakroom, restroom, and auxiliary facilities. The building should have multiple chemical delivery truck unloading systems (one per tank) needed to fill each storage vessel. A tunnel extension or utilidor concept is suggested to convey process chemicals from the Chemical Feed Building to the main liquid stream tunnel system.

Table 7-6. Chemical Feed Building Chemical Storage and Occupancy Class

Function (assumed chemical)[a]	Assumed Annual Average Storage Volume[b]	Occupancy Class	HVAC Requirements
Disinfectant (sodium hypochlorite)	68,000 gallons	H-4	Ventilation
Primary Coagulant (aluminum sulfate)	28,000 gallons	H-4	Ventilation
Ammonium Agent (ammonium hydroxide)	10,000 gallons	H-4	Ventilation
Quenching Agent (calcium thiosulfate)	6,000 gallons	S1 or S2	—
pH Adjustment (carbon dioxide)	16,000 gallons (stored outside)	S1 or S2	Ventilation in case of leak
Auxiliary Oxidizing Agent (e.g., potassium permanganate)	830 $ft.^3$	H-3	Keep cool/ventilation
Alkalinity Adjustment Agent (sodium carbonate)	13,000 $ft.^3$	S-1	Weathertight/no moisture
Adsorbent (powdered activated carbon if ozone not provided)	15,000 $ft.^3$	H-2	Dust control
Coagulant Aid Cationic Polymer	4,600 gallons	S1 or S2	—
Filter Aid Cationic Polymer	460 gallons	S1 or S2	—

a. Not including spare chemical storage to be determined during design.

b. Volume assumes 30 days of storage for each chemical.

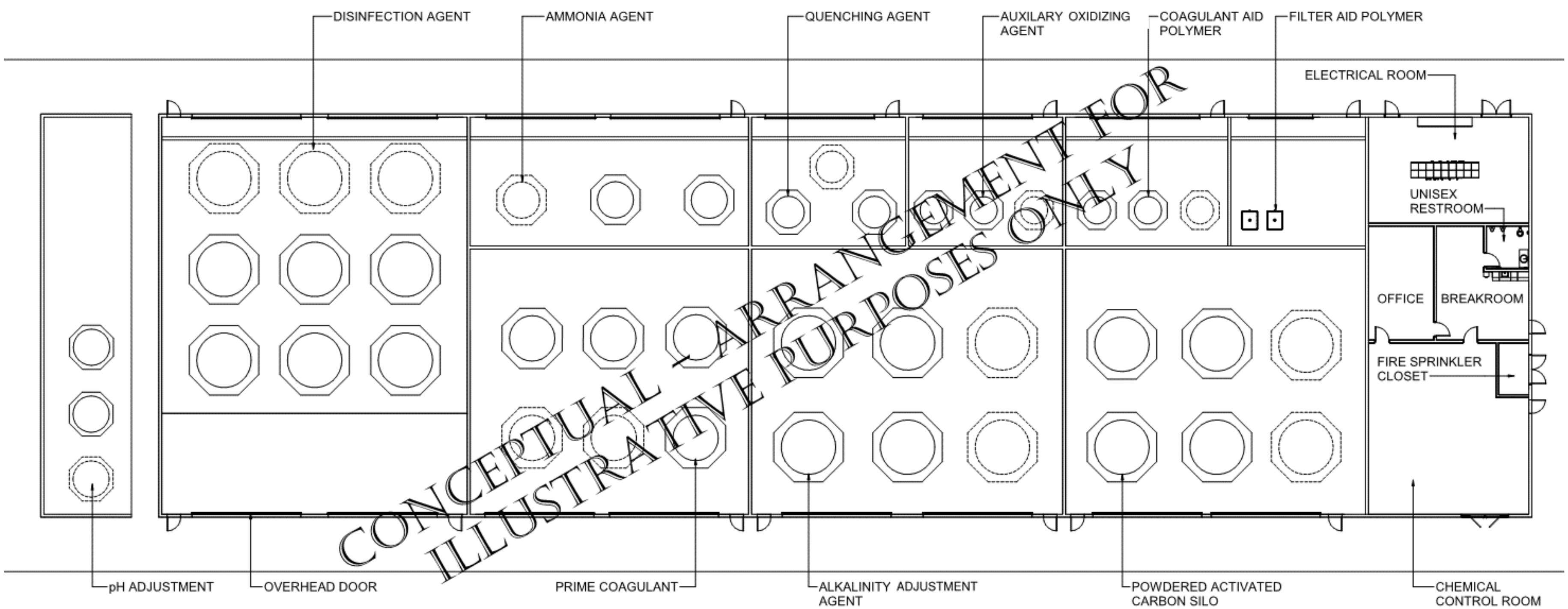


Figure 7-14. Potential concept for Chemical Feed Building floor plan

Ozone Building

The Ozone Building will house the ozone generators, nitrogen boost system, and ozone generator cooling systems, mechanical room, electrical room, and auxiliary facilities (Figure 7-15 below). For the purposes of project definition, it is assumed that ozone will be handled in a separate building (to avoid processing this oxidizing chemical in the presence of other chemicals) and that ozone solution is assumed to be transferred to the ozone contactors for sidestream injection. Liquid oxygen tanks and evaporators will be located outside the building with adjacent truck access for unloading.

The approximate average demand storage volume and occupancy class information for oxygen is shown in Table 7-7.

Table 7-7. Ozone Building Chemical Storage and Occupancy Class

Chemical	Assumed Annual Average Storage Volume (gallons)[a]	Occupancy Class	HVAC Requirements
Oxygen	48,000	H-3	—

a. Volume assumes 30 days of storage for each chemical.

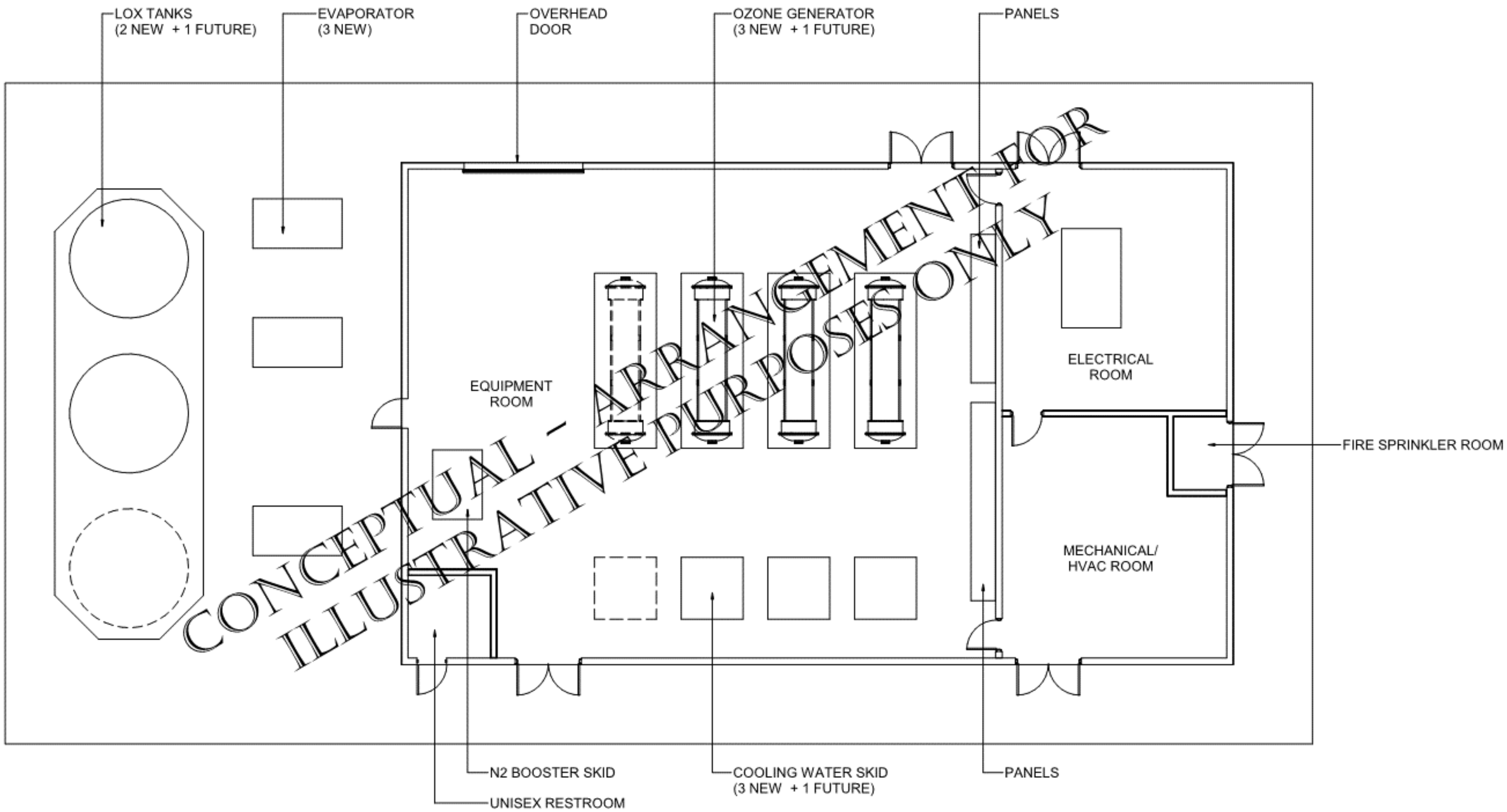


Figure 7-15. Potential concept for Ozone Building floor plan

Solids Handling Building

The Solids Handling Building will house the polymer feed equipment, mechanical room, electrical room, restroom, and auxiliary facilities on the first floor with dewatering equipment located on the second floor. Drive-through truck access on the first floor permits direct gravity truck loading of dewatered cake. During design, it is suggested the building height be considered during siting to minimize potential visual impacts (Figure 7-16 below).

The approximate average demand storage volume and occupancy class information for both emulsion polymers to aid solids thickening and dewatering are shown in Table 7-8.

Table 7-8. Solids Handling Building Chemical Storage and Occupancy Class

Chemical	Assumed Annual Average Storage Volume (gallons)[a]	Occupancy Class	HVAC Requirements
Polymer - thickening aid	150	F1 or F2	Low humidity storage and dust control for dry polymer; freeze protection for emulsion and solution polymers
Polymer - dewatering aid	150	F1 or F2	Low humidity storage and dust control for dry polymer; freeze protection for emulsion and solution polymers

a. Volume assumes 30 days of storage for each chemical.

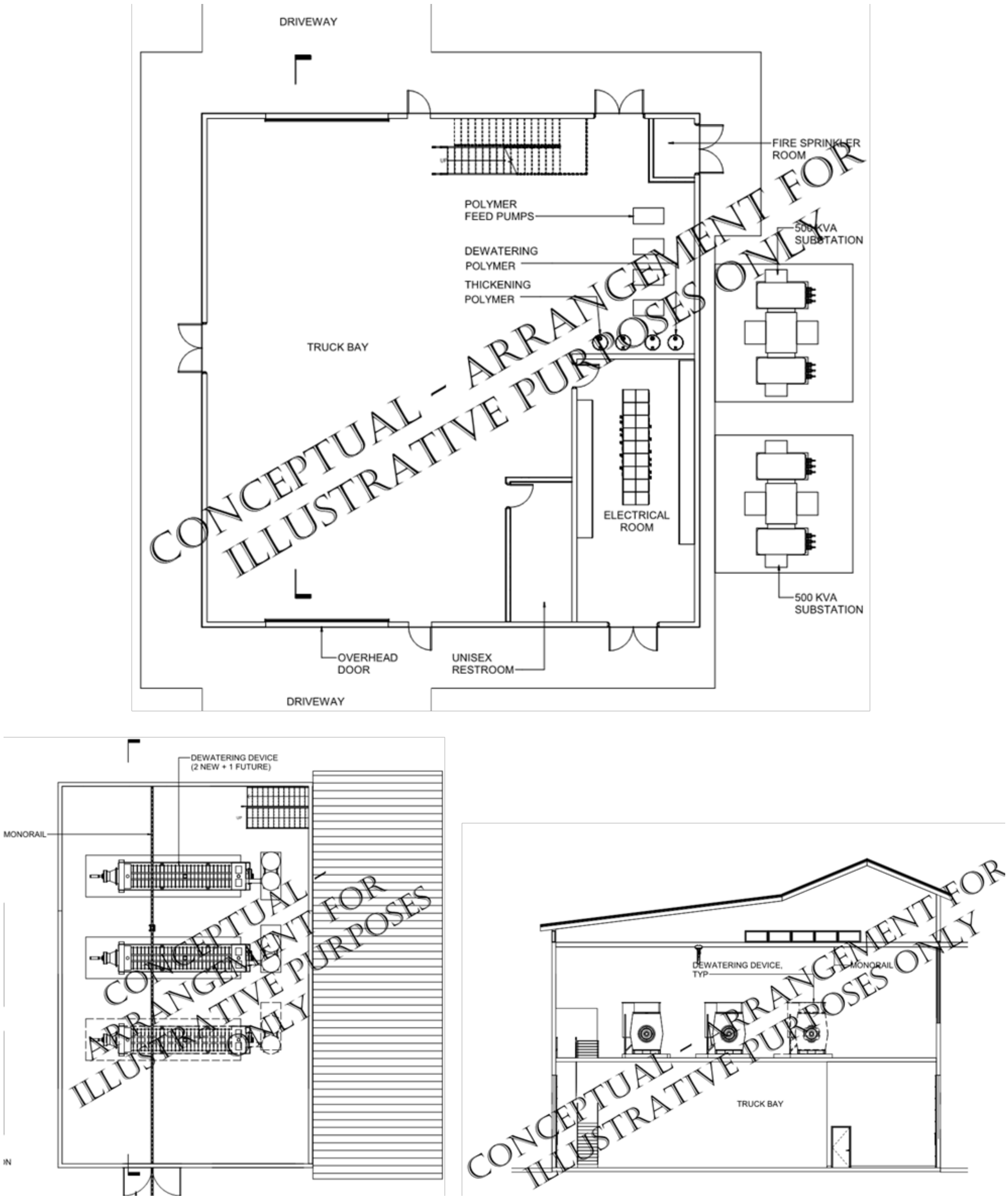


Figure 7-16. Potential concepts for Solids Handling Building

First floor plan and section (top)
Second floor plan (bottom)

Main Electrical/Generator Building

The Main Electrical/Generator Building will house the primary electrical distribution for the filtration facility (Figure 7-17 below). The electrical room will include 12.47 kV switchgear that receives power from PGE and provides power distribution to substations in the filtration facility main power center locations. The generator room will include two each 12.47 kV, 1.75 MW standby diesel generators to meet facility standby power requirements. Local low voltage requirements will be met using two 12.47 kV to 480 volt, 2000 kVA (actual capacity to be confirmed in final design) outdoor oil-filled substations just outside this building and 480 volt switchgear in the electrical room. This switchgear will in turn distribute low voltage power to local motor control centers for local load requirements.

These local loads are expected to include the Chemical Feed Building, Administration Building, Maintenance Building, rapid mix units, flocculators, sedimentation loads, north overflow basin pumps, and the Generator Building loads. The actual distribution of loads between this main electrical building and the second electrical building located near the filters will be determined during design when the load requirements and locations are finalized. The bulk fuel storage is located outside of the Main Electrical/Generator Building and has a placeholder capacity of 5 days fuel storage at full generators load.

The final decision on fuel storage capacity is suggested to be based on factors such as single or dual services from PGE and worst-case scenarios for refueling considering potential seismic or flooding conditions. During design, the potential sound impact of the generators on both facility staff and neighbors should be considered in the building design and siting.

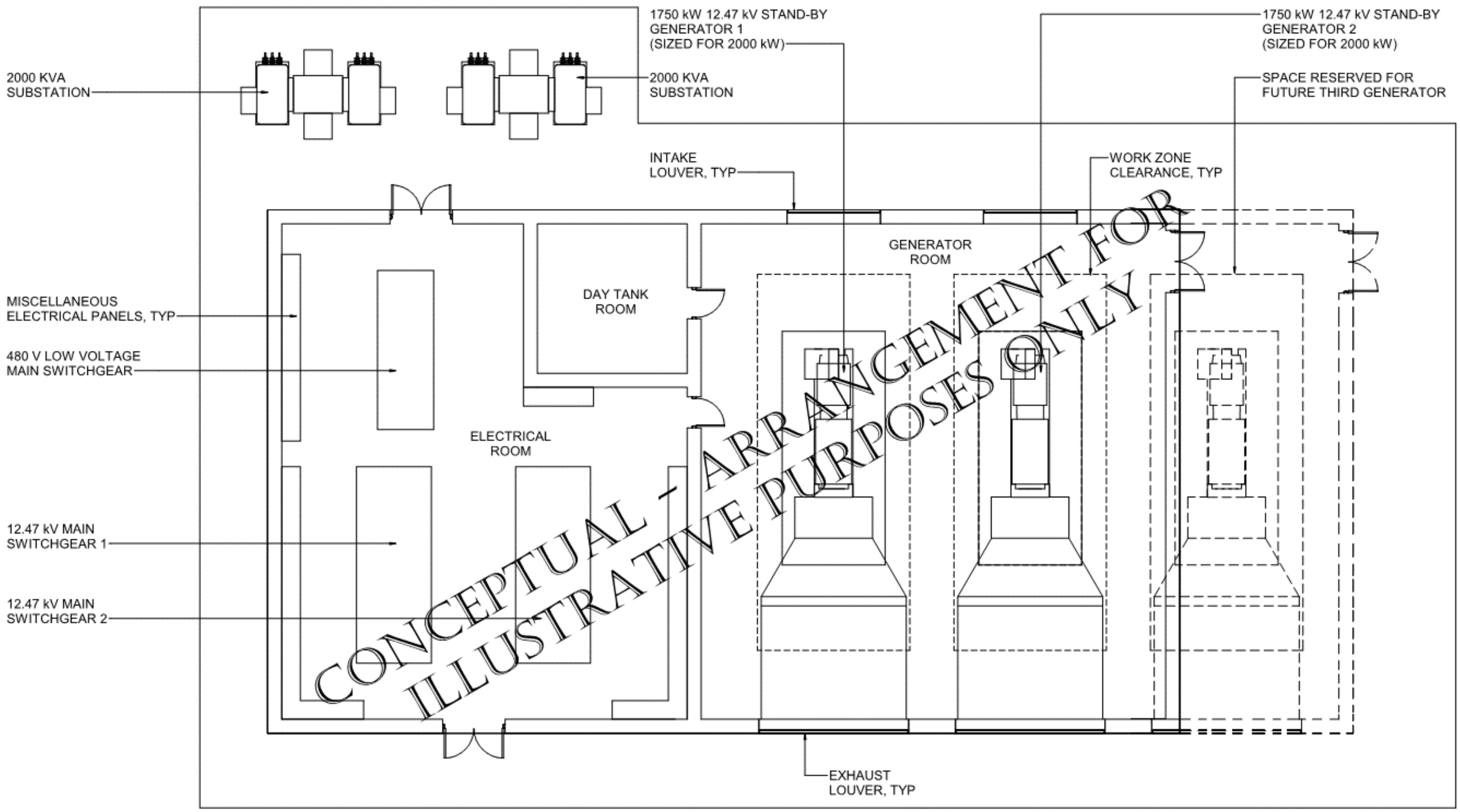


Figure 7-17. Potential concept for Main Electrical/Generator Building floor plan

Electrical Building

The Electrical Building provides a power distribution center near major loads such as ozone and the clearwell (backwash pumps and blowers) (Figure 7-18). These are large enough loads that local 480 volt distribution is warranted rather than routing voltage level distribution from the Main Electrical/Generator Building. Outdoor 12.47 kV to 480 volt, 2500 kVA (actual capacity to be confirmed during design) oil-filled substations outside of the Electrical Building will step down voltage from the feeders from the Main Electrical/Generator Building to 480 volts. The 480 volt switchgear and motorized control centers in this location will then provide low voltage distribution to other nearby load centers, including the Ozone Building, filter-to-waste tanks, washwater equalization loads, and south overflow basin pumps. As noted above for the main electrical building, it is anticipated that the final distribution of facility loads between these two electrical building distribution panels will be determined during design.

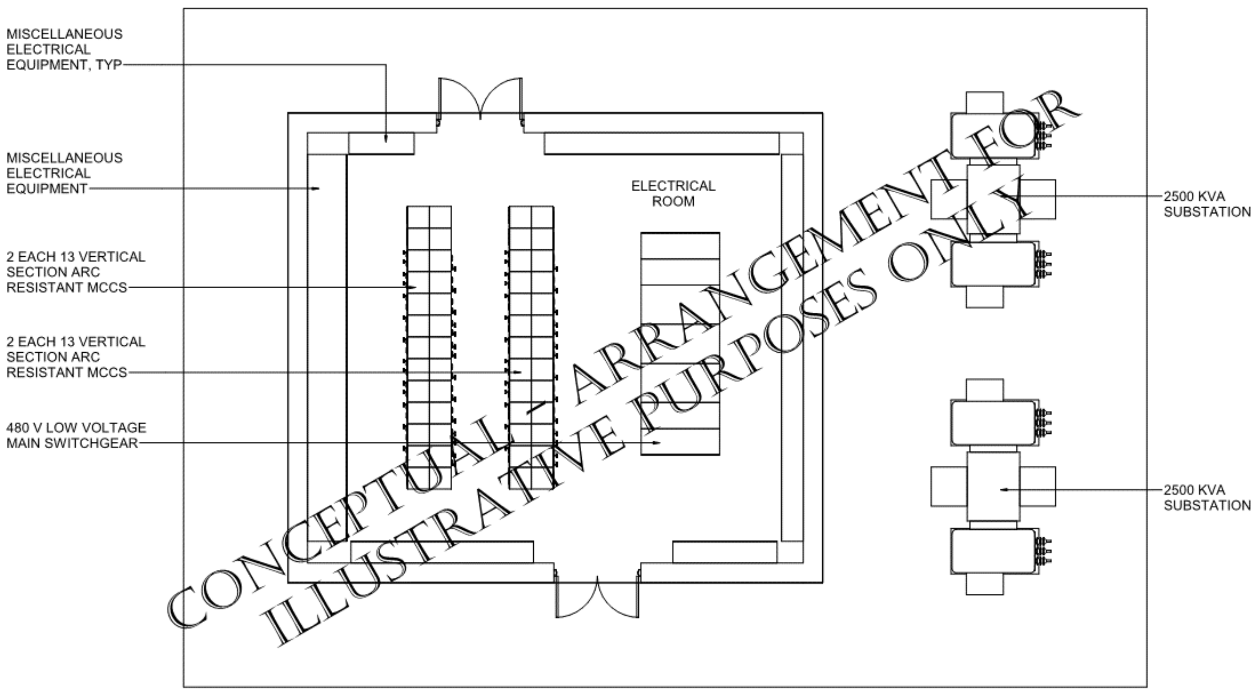


Figure 7-18. Potential concept for Electrical Building floor plan

Storage

The filtration facility will need onsite transportation vehicles for O&M staff. Vehicles should be stored in an area that is convenient for staff. Depending on the type of vehicles selected, vehicle storage could be a covered area and would most likely include charging stations for onsite electrical vehicles.

Outside storage that is enclosed for large equipment and supplies should be near the Maintenance Building for ease of access and a canopy should be considered to protect large, oversized stored items from the weather. The storage area should be kept to a manageable size to discourage accumulation of excess equipment while also considering future storage requirements.

7.3.9 Architectural Programming Summary

This section summarized the overall architectural programming for the filtration facility, including information about building programs for non-process structures at the site.

Key considerations for project definition include:

- **Architectural Approach.** The architectural programming reflects the project goals and an intent to develop an overall site concept that minimizes visual impact and complements the rural character of the site.
- **Facility Adjacencies and Site Program.** Adjacency considerations were identified to improve circulation for ease of O&M of the filtration facility.
- **O&M Staffing.** The filtration facility is anticipated to be staffed 24/7 while maintenance, laboratory and administration staff will work daytime shifts through the work week. The facility will require a staffing plan and support facilities for O&M.
- **Building Materials.** It is suggested the filtration facility be constructed using long-lasting, durable, seismically resistant, sustainable, and low maintenance materials consistent with the character of the rural area.
- **Acoustics.** The acoustical impacts of the filtration facility must be mitigated to meet code requirements and be a good neighbor.
- **Lighting.** The lighting design needs to meet code requirements, meet Dark Sky standards, provide adequate lighting to support facility staff safety, secure the site, and avoid impacting neighbors.
- **Odor.** Odors are not a nuisance associated with filtration facilities and therefore no mitigations are required.
- **Building Concepts.** Figures and layouts are conceptual only, the designer will be developing the design drawings.
  - **Administration Building.** The center of the non-process facilities at the filtration facility and contains many functions, including control center, laboratory, offices and workspaces, multi-purpose room and conference room, supporting spaces, public lobby and educational spaces. This multilevel building needs to be designed to facilitate staff activities and efficient access to the treatment processes, ground surface, and filter gallery to support facility O&M.
  - **Maintenance Building.** Supports the maintenance staff activities by housing a warehouse, parts inventory, and workshops.
  - **Chemical Feed Building.** Houses most of the chemical storage and feed systems for the filtration facility except for oxygen storage and ozone generation equipment.
  - **Ozone Building.** Houses the ozone generators, nitrogen boost system, and ozone generator cooling systems, mechanical room, electrical room, and auxiliary facilities.
  - **Solids Handling Building.** Houses the polymer feed equipment, mechanical room, electrical room, restroom, and auxiliary facilities with dewatering equipment located on the second floor to allow direct gravity truck loading via drive-through.

- **Main Electrical/Generator Building.** Houses the primary electrical distribution for the filtration facility, including switchgear, power distribution to local substations, and standby diesel generators.
- **Electrical Building.** Provides a power distribution center near major loads such as ozone and the backwash pumps and blower at the clearwell.
- **Miscellaneous.** The conceptual site plan includes space for the pilot plant facilities, vehicle storage, outside storage, and other buildings.

7.4 Filtration Facility Support Systems Summary

This chapter described basic site development, utility, and building needs to support the treatment processes for the filtration facility. The assumptions identified in this chapter inform the initial cost estimate and provide the foundational direction for future design development.

Key considerations for project definition include:

- **Site Development:**
  - Mass excavation is required to achieve a hydraulic profile capable of gravity flow. Conceptual mass grading may remove approximately 840,000 bank cubic yards of material. Conceptual structural excavations may remove approximately 385,000 bank cubic yards of additional material.
  - Native materials may be used on site for some backfills, berms, and shaping, but most of the excavated material will need to be hauled off site. The total volume anticipated is approximately 1,225,000 cubic yards. The required volume of imported granular material will be approximately 87,000 bank cubic yards.
  - A minimum setback of critical facilities from the crest of the steep slope northeast of the site and soil mitigation will need to be further reviewed during design.
  - Overflow basins are assumed to be lined and are anticipated to be sized for the ultimate facility capacity. This sizing will avoid future modifications of these basins, which may likely be considered dams due to storage volume and depth.
  - Space is allocated for three stormwater basins; however, the initial calculation shows no net increase in stormwater runoff from the filtration facility.
  - The volume of post-development compared to pre-development runoff resulting from the final design will be sensitive to certain design choices, including:
    - Roofed or open process tanks (e.g., filters and flocculation/sedimentation basins)
    - Size of overflow basins
    - Extent of green-roof technology
    - Total developed impervious area
  - It is assumed the stormwater basins will be unlined.
  - Onsite sanitary facilities are required to handle domestic wastewater. A septic system with holding tanks is assumed.
  - Two site access routes are assumed. Site access to the filtration facility will need to consider right-of-way, permitting, safety, and community feedback and will be further coordinated with PWB during design.
- **Utilities:**
  - A redundant potable water supply system pumped from the clearwell or finished water pipeline is anticipated to serve each major building and process basin.
  - The fire suppression system is anticipated to use raw water and include diesel-driven pumps in case of a power outage.

- Availability of natural gas along SE Carpenter Lane will need to be coordinated with NW Natural during design.
- Yard piping connections to structures will need to be flexible to address seismic deformation and shaking and are suggested for further evaluation during design.
- There are minimal existing utilities at the site, but some will require relocation and associated easement modifications.
- The main incoming service is assumed to be from the PGE Orient substation with an optional second service from the Dunn's Corner substation. Incoming power supply from PGE is anticipated to be a 12.47 kV distribution service.
- Two standby power generators will provide 3.5 MW at 12.47 kV for filtration facility backup power.
- The filtration facility 12.47 kV main switchgear will be located in the generator building with redundant 12.47 kV feeders routed to treatment process locations and stepped down to 480 volts via local substations.
- Various solar power options were assessed at the filtration facility and are suggested for further evaluation with PWB during design.
- Facility SCADA will be fully integrated with PWB's existing SCADA system.
- Primary telecommunications will be via redundant fiber optic pathways. Backup telecommunications to critical PWB facilities will be via microwave links. Telecommunications to some remote PWB sites will be via master polling radios.
- Security systems will include access control, intrusion detection, and video surveillance. Exterior doors and security gates, as well as identified interior doors, will have controlled access.

- **Architectural Programming:**
  - Facility structures, including building materials and acoustic and lighting design, should be consistent with applicable code requirements, functional needs, and the character of the rural area.
  - The filtration facility is anticipated to be staffed 24/7 and will require a staffing plan and support facilities for O&M.
  - Separate buildings are assumed for the following processes and equipment: administration, maintenance, chemical feed, ozone, solids handling, main electrical/generator, and electrical. The conceptual site plan also includes space for the pilot plant facilities, vehicle storage, and outdoor storage.

References

Clackamas County, *Clackamas County Code Title 6 Public Protection Section 05 Noise Control (CCC 6.05).*

DEQ, *Oregon Administrative Rules, Chapter 340, Division 35 Noise Control Regulations* (OAR 340-035).

DEQ, *Oregon Administrative Rules Chapter 340, Division 71 Onsite Wastewater Treatment Systems, Section 0220 Standard Subsurface Systems* (OAR 340-071-0220).

Environmental Protection Agency, Environmental Impact Statement Guidelines, General Guidelines Section III, 1973.

Multnomah County, *Multnomah County Code Vol. 2 Land Use Ordinances*, Chapter 39: Zoning Code, Updated 11.08.2019.

Multnomah County, *Multnomah County Design Standards, Design Manual Part One*.

Multnomah County Transportation Division, *Multnomah County Road Rules, Section 26.000 Stormwater and Drainage,* March 27, 2018.

National Fire Protection Agency, *National Electrical Code, Article 702 Optional Standby Power* (NFPA 70), 2017.

Chapter 8

Pipeline Alternatives

New large-diameter water transmission pipelines will connect the filtration facility to the existing Bull Run Conduits. Raw water pipelines (RWPs) will connect to the existing conduits along SE Lusted Road at Hudson Road Intertie and farther downstream at a new intertie. The RWPs will convey untreated water by gravity along a generally northwest route to the filtration facility. Finished water pipelines (FWPs) will convey treated water from the filtration facility by gravity along a generally northwest route and connect to the existing conduits at new interties.

This chapter includes the following sections:

- 8.1 Planning Considerations
- 8.2 Alternatives Evaluation Process
- 8.3 Initial Pipeline Alternatives
- 8.4 Refined Pipeline Alternatives
- 8.5 Optimized Pipeline Alternatives
- 8.6 Interties
- 8.7 Pipeline Alternatives Summary

8.1 Planning Considerations

This section identifies planning considerations for new pipelines to connect the filtration facility with the existing water system. PWB conducted various site investigations to characterize conditions along potential pipeline routes and inform evaluation of the pipeline alternatives.

This section includes the following content:

- Pipeline Study Area Description
- Permitting Considerations
- Environmental Assessment
- Cultural Resources Protection
- Geotechnical Considerations
- Reliability
- Hydraulic Considerations and Ultimate Capacity
- Traffic Considerations
- Surveying
- Right-of-Way
- Trenchless Applications
- Planning Considerations Summary

8.1.1 Pipeline Study Area Description

The RWPs will connect with the Bull Run Conduits along SE Lusted Road and convey untreated water by gravity to the filtration facility. The FWPs will convey treated water from the facility by gravity to the existing conduits.

PWB identified an initial pipeline study area where potential RWP and FWP routes could be located. The study area stretches generally from the intersection of SE Lusted Road and SE Hudson Road toward the intersection of SE Altman Road and SE Oxbow Drive (Figure 8-1). The filtration facility site in the center of the study area is about half a mile south-southwest of the Bull Run Conduits.

Figure 8-1. Conceptual pipeline route study area

The pipeline study area spans two relatively shallow sloped areas, the upper and lower terraces, separated by narrow steep sloped areas (Figure 8-2). The RWP alternatives start on the lower terrace and run to the filtration facility on the upper terrace, whereas the FWP alternatives run entirely on the upper terrace from the filtration facility to the connection locations.

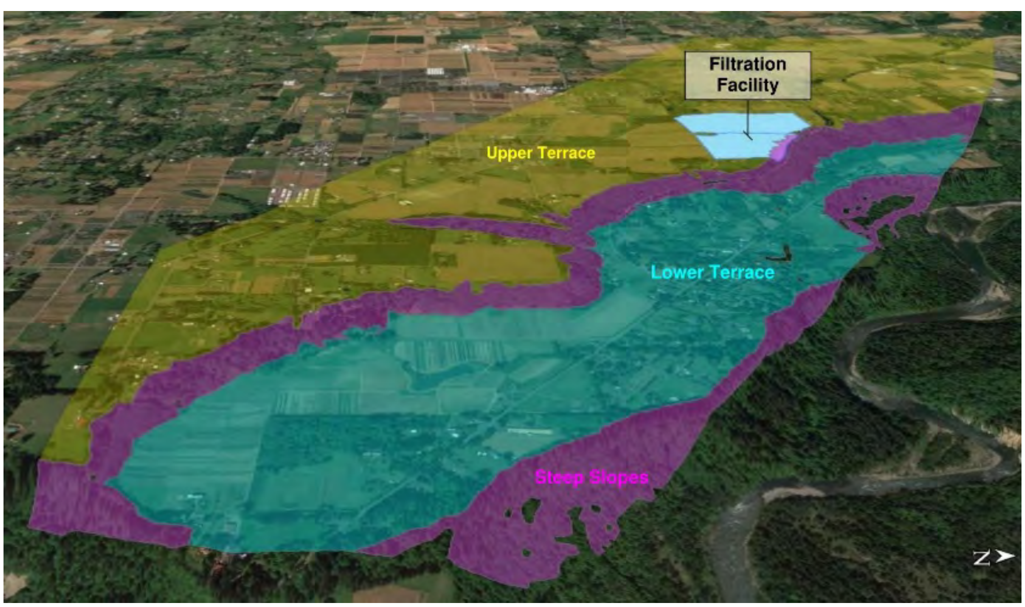


Figure 8-2. Pipeline study area, looking west, showing the upper and lower terraces separated by steep slopes

The existing Bull Run Conduits–Conduit 2, Conduit 3, and Conduit 4–have a combined capacity of approximately 210 million gallons per day (mgd) under the current hydraulic conditions of full flow from Headworks to the Powell Butte reservoirs.

Once the filtration facility is operational, all supply from the Bull Run reservoirs will be treated using filtration. The segments of the Bull Run Conduits between the new RWP and FWP connections will be abandoned or potentially repurposed. There have been preliminary discussions of using segments of an existing conduit, possibly Conduit 4, to connect to the existing Lusted backfeed distribution line that supplies customers upstream of the Lusted Hill facility, mostly on SE Lusted Road.

8.1.2 Permitting Considerations

This section presents findings from a preliminary review of pipeline permitting needs completed by Winterbrook Planning. The findings address local, state, and federal permits and their implications for design, construction, and operation of the pipelines. Consideration is needed for land use zoning, steep slopes, stream and wetland area protections, and wildlife habitat in both Clackamas and Multnomah counties (Figure 8-3.).

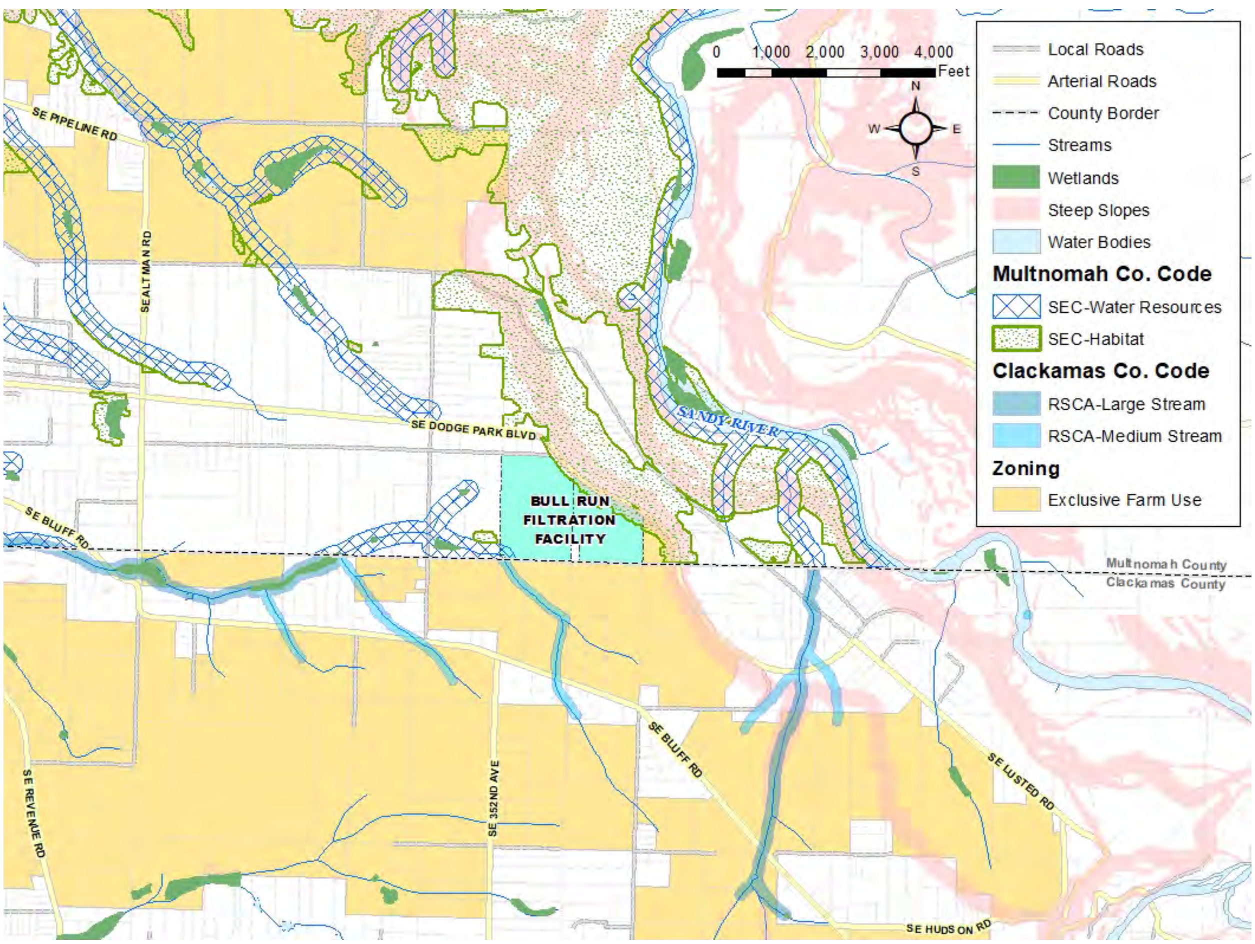


Figure 8-3. Permitting constraints for pipelines

Most of the RWP alternatives begin in Clackamas County, run generally northwest, and cross the Multnomah County boundary before entering the filtration facility site. As such, planning for the RWP alternatives considers the different permitting requirements for Clackamas and Multnomah counties. In Clackamas County, RWP alternatives are in the right-of-way or in Rural Residential Farm Forest 5-Acre or Exclusive Farm Use (EFU) zoning areas and cross River and Stream Conservation Areas (RSCA). In Multnomah County, RWP alternatives entering the facility site from the east are in Rural Residential, Multi-Use Agricultural (MUA-20), or EFU zoning areas and cross Significant Environmental Concern Habitat areas.

The FWP alternatives are completely within Multnomah County and run generally northwest from the filtration facility site to connect with the existing conduits. FWP alternatives are in MUA-20 or EFU zoning areas and cross Significant Environmental Concern Habitat or Water Resource areas. However, the FWP alternatives use right-of-way for most of the alignments, with the exception of a segment crossing private MUA-20 land between SE Dodge Park Boulevard and SE Lusted Road. No significant permanent impacts to the area are anticipated.

Pipeline crossings on EFU lands will be buried and existing surface conditions will be restored following construction. Proposed pipeline interties are located on non-EFU lands.

8.1.3 Environmental Assessment

Phase I and Phase II Environmental Site Assessments (ESAs) were conducted by Assessment Associates, Inc., along the potential pipeline routes in September 2019 and April 2020.

The Phase I ESA was conducted in conformance with ASTM Standard Practices and found no evidence of Recognized Environmental Conditions in connection with the project area. Phase II soil sampling along final pipeline alignments was recommended, and a Phase II ESA was subsequently performed along potential RWP and FWP alternatives. The *Phase II ESA Report* is included in Appendix C (Assessment Associates, Inc., 2020).

Soil samples from two depths (0 to 6 inches deep and 6 to 12 inches deep) were collected along the potential alignments at approximately 200 linear-foot intervals. In addition, 12 soil borings were advanced to depths ranging from 18 to 20 feet below ground surface.

Phase II ESA Results

Soil sample test results for pesticides, herbicides, and metal concentrations were below established background levels or the most conservative Oregon Department of Environmental Quality (DEQ) Risk-Based Concentrations for soil ingestion, dermal contact, and inhalation for occupational, construction worker, and excavation worker receptor scenarios.

Pesticides were detected above DEQ Clean Fill levels along private property between SE Dodge Park Boulevard and SE Lusted Road, and practical quantitation limits for soils tested south of SE Dodge Park Boulevard were above DEQ Clean Fill levels. Further testing may be needed if these soils (likely only topsoil) will be disposed of off site, which is not expected.

Since soils tested below their respective Risk-Based Concentrations, the soils would qualify for unrestricted reuse following approval of a solid waste permit exemption application. It is anticipated that native topsoil used for farming operations will be stockpiled and replaced above the pipe trench zone within private property limits.

8.1.4 Cultural Resources Protection

Heritage Research Associates (HRA) conducted archaeological pedestrian surveys in September 2019 and April 2020 to identify potential cultural resources along the potential pipeline alignments.

No evidence of prehistoric or early historic artifacts or deposits was observed on the ground surface or in shallow shovel probes sampled during the pedestrian survey of the pipeline project area. Access restrictions prevented investigation on some properties north of SE Dodge Park Boulevard along RWP alternatives, but the proposed alignments avoid structures visible from survey investigations of adjoining properties and in historic aerial imagery. As mentioned in Chapter 4: Planning Considerations, a rock retaining wall was found along SE Dodge Park Boulevard northeast of the filtration facility site. A more in-depth recording and assessment of the feature would not be required unless an RWP alternative along SE Dodge Park Boulevard is selected.

Archaeological site and project records on file at the State Historic Preservation Office were reviewed to identify previous cultural resources investigations and archaeological sites recorded in the pipeline project area. The same 12 previous archaeological investigation records reported in Chapter 4 Table 4-1 were identified within approximately 1 mile of the pipeline study area. Only one archaeological site was recorded as a result of these 12 previous investigations; the site is nearly a mile away from the pipeline study area and will not be impacted by pipeline construction.

The development of initial pipeline alternatives considered ways to avoid conflicts with existing structures to reduce direct and indirect above ground impacts. Refinement of the initial alternatives incorporated direction from HRA regarding properties and structures previously determined to meet National Register of Historic Places (NRHP) criteria and properties that could potentially meet NRHP criteria. However, even if properties along the proposed pipeline alternatives are formally determined to be NRHP eligible, it is unlikely that the project will adversely impact those properties, as the pipelines will be below ground and will not visually obstruct, change, or demolish NRHP eligible properties or structures. The location of above ground appurtenances required for pipeline operation (i.e., combination air valves and blowoffs) will be determined during detailed design with continued input from HRA to avoid impacts to NRHP properties.

The archeological survey complies with state and local land use laws requiring identification and protection of significant resources, defined as those eligible for the NRHP. The survey also complies with federal cultural resources requirements under Section 106 of the National Historic Preservation Act of 1966 (as amended).

8.1.5 Geotechnical Considerations

This section summarizes preliminary geotechnical considerations based on work performed by RhinoOne Geotechnical, including information summarized from the *Geotechnical Data Report* in Appendix F (RhinoOne Geotechnical, 2020). This preliminary work was based on a review of geologic and hazard mapping reports, site reconnaissance, previous subsurface explorations, and explorations conducted for project definition and described in Chapter 4.

In addition to geotechnical explorations at the filtration facility site, RhinoOne Geotechnical performed the following explorations and testing, illustrated on Figure 8-4, between March 11, 2019, and March 24, 2020.

- Twenty-two drilled borings and two rotary-sonic borings
- Installation of groundwater level monitoring devices in seven borings
- Laboratory testing of moisture content, grain size, Atterberg limits, and chemical composition

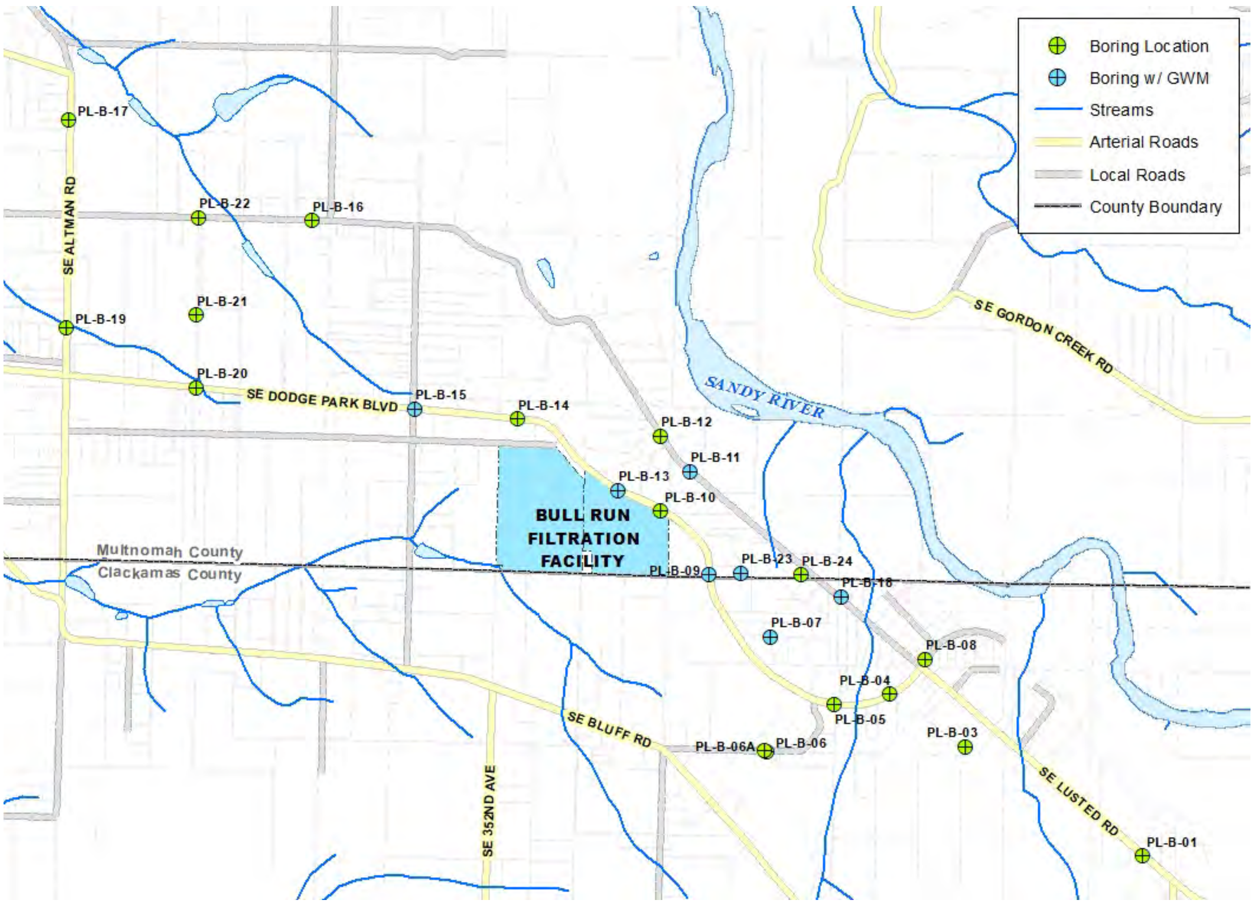


Figure 8-4. Pipeline geotechnical exploration borings and groundwater monitoring locations

Recommendations from the Geotechnical Technical Advisory Committee (GTAC), created for the filtration facility assessment, were considered when performing a fatal flaw screening and when evaluating pipeline alternatives. The GTAC consists of regional subject matter experts that include geologists and geotechnical engineers. The GTAC helped develop the geotechnical field exploration plan that was the basis of the geotechnical analysis completed for planning level study. After field testing, the GTAC met to review and discuss the data and provided input on pipeline routing, including expert feedback on fatally flawed alternatives.

Analysis of the collected data suggests areas of instability near the steep slopes below the upper terrace along SE Dodge Park Boulevard and lower terrace east of SE Lusted Road (approximately 1,500 feet north of the county line). This analysis, guided by GTAC input, is based on low blow count materials observed in geotechnical borings, soil characterizations, water table levels, and historic landslide records. The GTAC concluded that shallow failures are expected along the terrace shelves and should be considered significant hazards during a Cascadia Subduction Zone event. The GTAC recommended that pipelines avoid the hazards with trenchless methods, or provide seismic mitigation (e.g., ground improvements or pile supports) if open cut installation is selected, to reduce seismic-induced settlement and lateral spreading near these sloped areas. Additional geotechnical field work and analysis, including further evaluation of the seismic risks and mitigation measures will be evaluated in detailed design.

8.1.6 Reliability

PWB has historically relied on multiple pipelines as a key resilience strategy to provide reliable service. As discussed in Chapter 2: Existing Water System, PWB currently has three conduits in service. This allows one conduit to be out of service for repair or planned inspection and maintenance, while still providing sufficient capacity for all but peak summer demand.

The new pipelines must provide a similar level of resilience and reliability. Therefore, the initial assessment of potential pipeline routes screened alternatives using the assumption that two RWPs and two FWPs would be built, either initially or in phases. While building a second pipeline in the future is a viable alternative, it will cost more to re-mobilize teams for planning, permitting, design, and construction and will bring additional disturbance to the community. In addition, phased construction would require the initial (i.e., single) pipeline to be oversized to meet design capacity.

For project definition and development of preferred RWP and FWP alternatives, construction of two pipelines is assumed. Along the RWP, multiple existing conduits may be used within SE Lusted Road downstream of the Hudson Road Intertie. Isolation valves will be provided where the RWPs and FWPs connect to the Bull Run Conduits to allow the systems to be inspected and maintained when needed.

8.1.7 Hydraulic Considerations and Ultimate Capacity

With the elevation of the filtration facility site optimized based on the hydraulic grade line (HGL) of the existing system, a maximum of 160 mgd can be delivered to the facility by gravity using existing and new pipelines. To meet the hydraulic constraints and capacity goals, while maintaining gravity flow, the RWP must remain below elevation 715 feet (top of pipe), and the FWP must remain below the minimum operating level of the clearwell. These hydraulic constraints will result in deep open cut excavations where the FWP leaves the filtration facility generally east of SE Cottrell Road.

The new pipelines will be sized to handle the design capacity of the filtration facility and meet the ultimate capacity without upsizing newly installed pipe. The minimum acceptable capacity of the new pipelines is 145 mgd, but some piping configurations that add reliability may also provide higher capacity.

Ultimate capacity will require future improvements to the Bull Run Conduits upstream and downstream of the new RWP and FWP connections (e.g., upsizing or replacement of existing conduit, connecting to Dam 2 head, or raising the elevation of the diversion pool).

Hydraulic modeling of the water system is being conducted to inform sizing of the new pipelines. RWP diameter is expected to be 72 inches (+/- 6 inches). FWP diameter is expected to be 66 inches (+/- 6 inches). The RWP size must be larger due to the required capacity and the available hydraulic head. Pipe sizes will be confirmed after the transmission conduit model is calibrated in late 2020.

Flow control for the FWP will be handled downstream of the filtration facility at the intertie connections between the new FWP and the Bull Run Conduits.

8.1.8 Traffic Considerations

As summarized in Chapter 4, a post-construction traffic impact analysis was conducted by Global Transportation Engineering. In addition to evaluating post-construction traffic impacts and site access locations for the filtration facility, Global will also evaluate construction traffic impacts for the pipelines and filtration facility work as contractor schedules are better understood during detailed design.

Both RWP and FWP construction will impact traffic in the local vicinity and require lane closures and extended duration road closures (up to 1 year in some cases). It is expected that traffic impacts for RWP construction will be less impactful than for FWP construction, as most RWP alternatives will likely be located in easements on private property. Some road closures will be required for short pipeline roadway crossings and intertie connections on SE Lusted Road.

Construction of the FWPs will be more impactful to traffic, as most of the FWP alternatives are located within right-of-way. During FWP construction, these roads will likely be closed to through traffic, with detours to nearby roads. Roads that are closed to through traffic will remain open in a limited capacity for residents within the closure limits. Road closures will be sequenced and phased to reduce detour lengths and community impacts.

Road closures improve safety for both construction workers and the public and are preferred to lane closures that require flagging traffic through the construction zone. Eliminating the hazard of moving vehicles from the work zone will greatly reduce the risk of accidents on the project. Road closures also allow space for a wider construction work zone, which improves productivity and reduces construction duration.

Traffic control plans will be developed during design and will be submitted to the governing agencies for review and approval before construction.

8.1.9 Surveying

Field survey and utility investigations were conducted along potential pipeline routes to assist the evaluation process for alignment selection. Survey crews located existing public and private utilities in the right-of-way, staked existing PWB easements on private property, and surveyed geotechnical exploration locations.

8.1.10 Right-of-Way

Temporary and permanent construction easements are anticipated to be needed for pipeline construction and will be further evaluated during design after selection of the RWP and FWP alternatives.

Over 30 years ago, PWB acquired a number of permanent easements in anticipation of a future pipeline being constructed near SE Lusted Road downstream of the Hudson Road Intertie. These easements are generally 100-feet wide and traverse private property west of SE Lusted Road.

For the RWP alternatives, use of these existing easements is being considered. However, the proposed alternatives will require some additional permanent easements to connect the existing conduits to the filtration facility.

Most of the FWP alternatives are located within existing rights-of-way. Exceptions include anticipated need for permanent easements from the filtration facility site to SE Dodge Park Boulevard and across private property between SE Dodge Park Boulevard and SE Lusted Road. Additional permanent easements may be needed next to roads where existing right-of-way is too narrow to install the pipelines.

Permanent easements or property acquisition will also be needed at intertie locations where RWPs and FWPs connect to the Bull Run Conduits.

Temporary construction easements will be needed to provide construction staging areas and access for installing trenchless crossings of steep slopes and streams for the RWPs and FWPs.

8.1.11 Trenchless Applications

Staheli Trenchless Consultants completed a feasibility study investigating trenchless construction methods (Appendix L). The study evaluated available trenchless methods, expected geotechnical conditions, shaft types, risks, basis of sizing and costs, and design considerations.

Many of the RWP alternatives include a significant tunnel between the lower terrace adjacent to SE Lusted Road and the filtration facility site on the upper terrace. The tunnel would be approximately 15 feet in diameter and 1,500 feet long. A shallow shaft would be constructed on the lower terrace and a deep shaft, approximately 150 to 160 feet deep, would be constructed on the upper terrace within the filtration facility site. Inside the concrete-lined tunnel, two 72-inch-diameter RWPs would be installed parallel to each other. For cost estimating purposes, Staheli Trenchless Consultants assumed a tunnel boring machine would be used by the contractor; however, a conventional tunnel without a tunnel boring machine may also be feasible.

Staheli also evaluated trenchless methods for potential stream and wetland crossings on the RWP and FWP alternatives. These shorter trenchless crossings are expected to be installed using auger boring methods. The cost estimates provided in the *Trenchless Memo* (Appendix L) do not reflect the expected final construction costs. The estimated costs provided in the *Basis of Estimate* (Appendix B) include assumed markups to the trenchless subcontractor costs.

8.1.12 Planning Considerations Summary

This section summarized preliminary planning considerations for the new pipelines. Key considerations for project definition include:

- **Permitting.** Close coordination between the permitting team, permitting agencies, designers, and community members is critical to the permitting process and long-term project success. Pipeline crossings on EFU-zoned land will be buried and existing conditions will be restored following construction. Pipeline interties will be located on non-EFU lands.
- **Environmental Assessment.** Based on the Phase II ESA results, some excavated materials from the near-surface layer (upper 12 inches) of private property between SE Dodge Park Boulevard and SE Lusted Road are suitable for onsite reuse and are likely suitable for a clean fill determination and a DEQ permit exemption for offsite reuse. However, it is anticipated that native topsoil located above the pipeline will be stockpiled and replaced as backfill material above the pipe trench zone within private property limits. Excavated materials below the near-surface layer are suitable for a clean fill determination.
- **Cultural Resources.** Although no archaeological artifacts were found in the field surveys, ground-disturbing activities along pipeline alignments should be conducted under the terms of an inadvertent discovery protocol and/or archaeological monitoring if buried archaeological features are encountered. A rock retaining wall was found on SE Dodge Park Boulevard but will be avoided. A more in-depth recording and assessment of the feature was not recommended unless an alternative is selected in this immediate area.
- **Geotechnical.** GTAC analysis of field borings recommended avoiding the steep slopes below the upper and lower terraces. Pipelines located within these areas will require seismic mitigation. The designer will be responsible for conducting additional field explorations and evaluating hazards along the selected alignments. The GTAC will be retained for peer review of the future data and recommendations in detailed design.
- **Reliability.** For project definition and development of preferred RWP and FWP alternatives, construction of two RWPs and FWPs is assumed to provide resilience and reliability.
- **Hydraulics.** To meet the hydraulic constraints and capacity goals, while maintaining gravity flow, the RWPs must remain below elevation 715 feet (top of pipe) and the FWPs must remain below the minimum operating level of the clearwell. RWP diameter is expected to be 72 inches (+/- 6 inches). FWP diameter is expected to be 66 inches (+/- 6 inches). Pipe sizes will be confirmed after the transmission conduit model is calibrated in late 2020.
- **Traffic.** Road closures with detours are expected for RWP and FWP construction. Traffic control plans will be developed during detailed design and will be submitted to the governing agencies for review and approval before construction.
- **Right-of-way.** Temporary and permanent construction easements are anticipated to be needed for pipeline construction and will be further evaluated during detailed design.
- **Trenchless Applications.** Trenchless methods will likely be used for stream and wetland crossings. Trenchless methods will also likely be used to avoid significant seismic hazards, for example, the steep slope between the upper and lower terrace along the RWP.

8.2 Alternatives Evaluation Process

This section describes the process used to evaluate pipeline alternatives. The process included field investigations, engineering analyses, and technical workshops to first identify initial pipeline alternatives in the study area, then perform alternatives evaluations informed by findings from field investigations, and ultimately identify preferred RWP and FWP alternatives. Throughout the pipeline planning process, project values listed in Table 8-1 were used to help characterize benefits and trade-offs of feasible pipeline alternatives and guide decision-making.

Over the course of 2 years of pipeline planning, three significant pipelines workshops were held to advance the evaluation process (***initial***, ***refined***, and ***optimized***). The workshops started broad and inclusive, then became more focused. Alternatives were evaluated against the values and criteria supported by increasingly detailed field investigations and analysis.

- **Initial Alternatives Evaluation.** The pipelines workshop in December 2018 discussed initial alternatives and minimum requirements for the pipelines (i.e., single versus dual), refined the alternatives evaluation criteria, and performed a preliminary fatal flaw screening. During the refinement of the evaluation criteria, it was decided that the public health and water quality value was not a differentiator between pipeline alternatives since this value applies primarily to water treatment.
- **Refined Alternatives Evaluation.** At the pipelines workshop in May 2019, the RWP and FWP alternatives were refined and further evaluated using findings from preliminary field investigations. The results of the evaluation identified paired RWP alternatives and paired FWP alternatives for additional evaluation.
- **Resolution 37460.** In November 2019, the Portland City Council adopted Resolution 37460, which provided direction to construct two pipelines to and from the filtration facility. This decision was applied to subsequent evaluations where paired pipeline alternatives were considered.
- **Optimized Alternatives Evaluation.** The pipelines workshop in July 2020 further evaluated optimized variations of RWP and FWP alternatives based on new information obtained from field investigations. These variations were designed to maximize resilience and reliability, facilitate pipeline integration, and minimize environmental and community impacts.

8.2.1 Pipelines Evaluation Process

Throughout the pipeline evaluation process, the project values were used to guide analysis of the various alternatives. Initial pipeline alternatives were assessed using project values to identify fatal flaws and possible revisions. Each refined and optimized pipeline alternative was **rated** independently on a scale of 1 through 5 (a “1” rating being most negative and a “5” rating being most positive) for each project value, with the exception of comparative cost which was left in dollars and the public health and water quality value which was not scored for pipeline alternatives.

Table 8-1. Pipeline Alternatives Evaluation Criteria[a]

Criteria	Value
Resilience/ Reliability	• Earthquake (including the Cascadia Subduction Zone event) • Geologically active areas (potential seismic hazards) • High consequence foreign utilities (e.g., major water, gas, power, sewer, or communications lines) • Accessibility after a seismic event • Gravity capacity • Replacing existing conduits in poor condition
Integration	• Safety and operations • Other infrastructure ramifications
Comparative Cost	• Comparative cost of construction
Implementation	• Ease of construction • Implementation complexity • Land use permits • On- and off-site ownership • Available property • System hydraulics • Available right-of-way, available property, or existing PWB-ownership • Construction and maintenance access • Need for traffic control during construction
Meets Future Needs	• Hydraulic capacity • Available gravity capacity
Community Interests	• Local impacts (property, significant road closures, visual and noise impacts) • Safety
Environmental Impacts	• Electricity usage • Construction and operations fuel consumption • Wetland and waterway impacts • Environmental Species Act listed or sensitive species impacts • Archeological and cultural resources impacts

a. Modified criteria developed during initial pipeline workshop.

The ratings were summed and the alternatives **ranked** with the highest scoring alternative given a rank of 1. In the event of a tie, the higher rank was given to the more cost-effective alternative.

8.2.2 Alternatives Evaluation Process Summary

This section summarized the evaluation process applied to the pipeline alternatives to identify preferred RWP and FWP alternatives. Key considerations for project definition include:

- **Evaluation Process**. PWB engaged in a series of three workshops over a 2-year period to evaluate potential pipeline alternatives and identify preferred RWP and FWP alternatives.
- **Evaluation Criteria**. The project values and criteria were refined to better fit the pipelines evaluation.

8.3 Initial Pipeline Alternatives

In the first phase of the pipeline alternatives evaluation, initial pipeline alternatives for both the RWP and FWP alternatives were identified within the pipeline study area. At the pipelines workshop in December 2018, the initial alternatives were presented, the evaluation criteria were refined, and the alternatives were then screened for fatal flaws. This section describes the initial alternatives considered and the fatal flaws screening process.

This section includes the following content:

- Initial Raw Water Alternatives
- Initial Finished Water Alternatives
- Initial Pipeline Alternatives Evaluation
- Initial Pipeline Alternatives Summary

8.3.1 Initial Raw Water Alternatives

Six initial RWP alternatives were identified, as described in Table 8-2 and shown on Figure 8-5 below.

Table 8-2. Initial RWP Alternatives

Alternative	Length (ft)	Description
RW Alt 1	10,600	• Connection at Hudson Road Intertie • Alignment within PWB easements
RW Alt 2	7,300	• Connection at a new intertie located on SE Lusted Road near the intersection with SE Dodge Park Boulevard • Alignment on SE Dodge Park Boulevard
RW Alt 3	11,800	• Connection at Hudson Road Intertie • Alignment partially on PWB easements, SE Proctor Road, SE Bluff Road, and along a potential access road to the filtration facility site
RW Alt 4	1,400	• Connection at a new intertie located on SE Lusted Road • Most direct and shortest alignment between existing conduits and the filtration facility • Alignment southwest to the filtration facility site
RW Alt 5	12,800	• Connection at Hudson Road Intertie • Alignment on SE Lusted Road, SE Dodge Park Boulevard, SE Proctor Road, SE Bluff Road, and a new access road to the filtration facility site
RW Alt 6	14,700	• Connection at Hudson Road Intertie • Longest alignment option • Alignment due south to SE Hudson Road, SE Bluff Road, and a potential access road to the filtration facility site

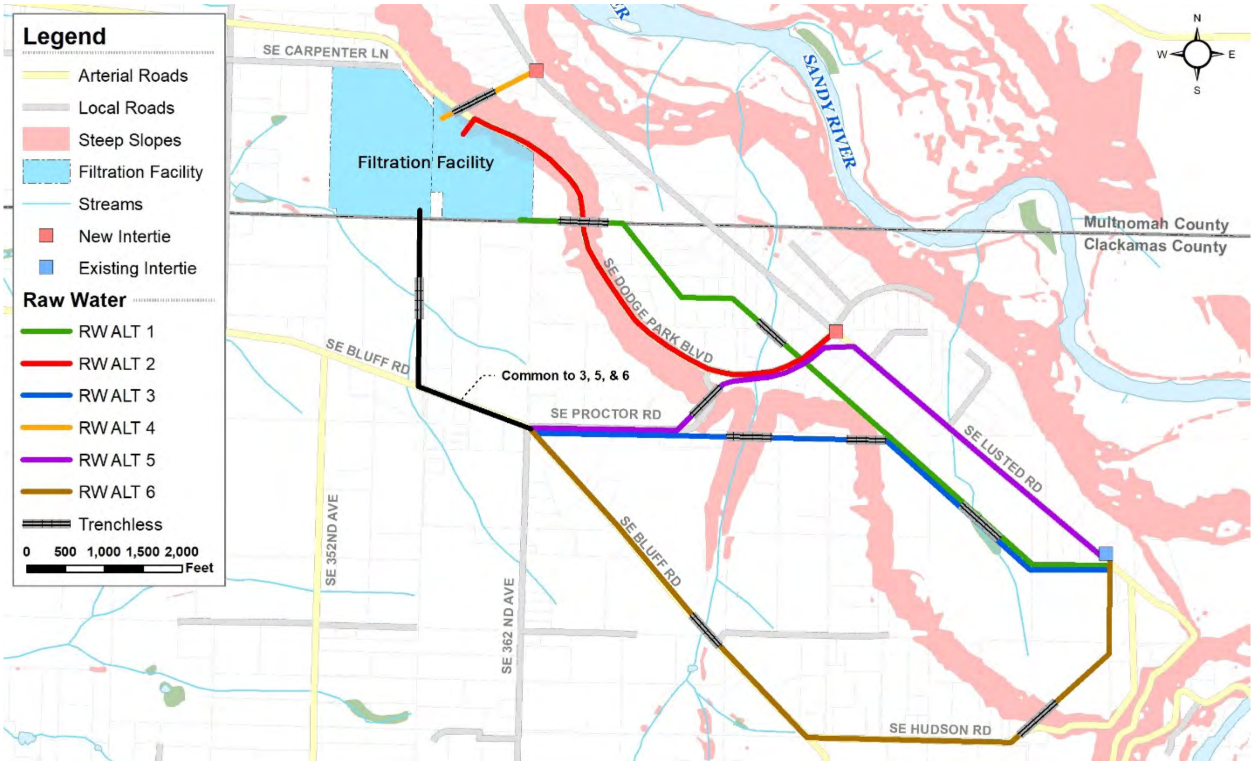


Figure 8-5. Initial RWP alternatives

8.3.2 Initial Finished Water Alternatives

Four initial FWP alternatives were identified, as listed in Table 8-3 and shown on Figure 8-6 below.

Table 8-3. Initial FWP Alternatives		
Alternative	**Length (ft)**	**Description**
FW Alt 1	8,200	• Alignment on SE Dodge Park Boulevard, SE Cottrell Road, SE Lusted Road, and SE Hosner Road • Connections to Conduit 3 on SE Cottrell Road, and to conduits 2 and 4 on SE Hosner Road
FW Alt 2	8,400	• Alignment on SE Dodge Park Boulevard, private property, and SE Hosner Road • Connections to Conduit 3 within private property boundaries, and to conduits 2 and 4 on SE Hosner Road
FW Alt 3	13,800	• Alignment on SE Dodge Park Boulevard and SE Altman Road • Connections to conduits 2, 3, and 4 on SE Altman Road
FW Alt 4	1,600	• Alignment directly northeast • Connections to conduits 2, 3, and 4 on SE Lusted Road

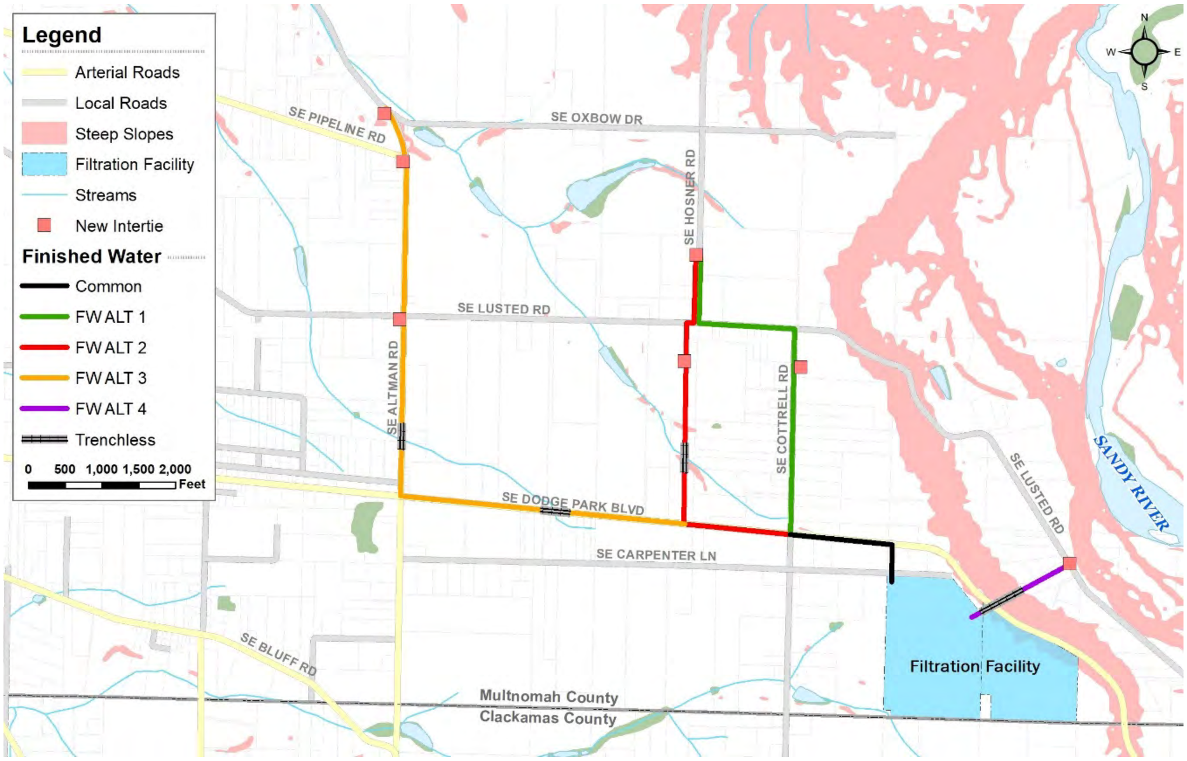


Figure 8-6. Initial FWP alternatives

8.3.3 Initial Pipeline Alternatives Evaluation

During the first pipelines workshop, fatal flaw screening criteria were developed and applied to the initial RWP and FWP alternatives. The alternatives were evaluated using the following fatal flaw criteria:

- Single point of failure
- Inadequate gravity capacity
- High cost due to excessive length
- Geologic and seismic hazards
- Onerous right-of-way acquisition
- Major utility conflicts

RW Alt 5 Revised

RW Alt 5, as initially conceived, was eliminated due to major conflicts with critical existing utilities in the SE Lusted Road right-of-way, including the Bull Run Conduits. These conflicts include:

- **Insufficient physical space.** The 60-foot-wide right-of-way already contains three PWB pipelines (12-, 52-, and 66-inch-diameter), and PWB has an adjacent 20-foot-wide easement that contains an additional conduit (58-inch-diameter). The new RWP requires a clear corridor at least 20-feet wide that does not exist.
  - The only way to make room for a RWP within the right-of-way would be to remove one of the existing conduits and build the RWP in its place; however, this would

unacceptably reduce the capacity and reliability of the Bull Run supply for the duration of construction. Further, there is risk, which can only partially be mitigated, of damage during close construction to the adjacent conduits. This partial mitigation would likely entail temporarily taking an adjacent conduit out of service, further reducing capacity and reliability of the supply until construction is complete.

 - Increasing the SE Lusted Road corridor width by acquiring an easement on the north side of SE Lusted Road would require purchase and removal of multiple residential structures. An alignment even further north of SE Lusted Road could avoid some local residences but would be too close to the steep slopes of the Sandy River where high geologic and seismic risks have been identified.
 - Moving to the south side of the SE Lusted Road corridor in an easement, while avoiding residential structures, is essentially the route covered by RW Alt 1.

- **Unacceptable compromise to long-term reliability and resilience**. By the time the RWP is installed and operational, the Bull Run Conduits will range in age from 75 to 115 years old—long before the recent understanding of the region's seismic hazards and modern seismic pipeline design practices were developed. Failure of any one of these conduits from any cause would pose a risk to the other adjacent conduits and the RWP. It is vitally important to the reliability and resilience of the Bull Run supply that the RWP (and FWP) are placed on alignments separated from the existing conduits.

At the workshop, the upstream connection for RW Alt 5 was relocated to the intersection of SE Lusted Road and SE Dodge Park Boulevard, and the revised alternative was carried forward for further analysis.

RW Alt 6 Eliminated

RW Alt 6 was eliminated because of inadequate gravity supply and high cost due to excessive length. Based on SE Hudson Road and SE Bluff Road being higher elevation than the supply, the alternative was fatally flawed and removed due to hydraulic grade and capacity issues. The top of the RWP must remain below elevation 715 feet to provide enough gravity supply to the filtration facility. Ground surface elevations on SE Hudson Road generally exceed elevation 780 feet. Additionally, there would likely be conflicts with the existing 24-inch Sandy water supply line in SE Hudson Road.

Additional Considerations

The segment of Conduit 3 near the initial FWP alternatives is in poor condition and strong consideration should be given to replacing this section of pipe. According to the *Conduit 3 Condition Assessment and Improvement Plan (*Pure Technologies, 2018):

> A remaining useful life evaluation using the ultrasonic thickness data was conducted to determine when the pipeline average thickness is expected to reach the limit thickness indicated in the M11 design analysis. The evaluation was split by intertie and concludes that the conduit between Headworks and Larson's Intertie has an average remaining life of 89 years, the conduit between Larson's Intertie and Hudson Intertie has an average remaining life of 78 years, and the conduit between Hudson Intertie and Sam Barlow High School has an average remaining life of 25 years, assuming all loading conditions apply.

At the time of filtration facility commissioning in 2027, the estimated average remaining design life of Conduit 3 within the project limits of the FWP and RWP will be about 15 years. Therefore, FW Alt 5 was added based on the recommendation to identify an alternative that replaces Conduit 3 upstream of SE Altman Road. This alternative parallels FW Alt 2 to SE Lusted Road, turns west to SE Altman Road, then turns north and connects with existing conduits and ends at the intersection with SE Oxbow Drive, as described in Table 8-4 and shown on Figure 8-7.

Table 8-4. Added FWP Alternative

Alternative	Length (ft)	Description
FW Alt 5	14,100	• Alignment on SE Dodge Park Boulevard, within private property boundaries, SE Lusted Road, and SE Altman Road • Connections to conduits 2, 3, and 4 on SE Altman Road

Figure 8-7. Added FW Alt 5

8.3.4 Initial Pipeline Alternatives Summary

This section described the first phase of pipeline alternatives evaluation during which initial pipeline alternatives in the study area were identified and screened for fatal flaws. The screened alternatives were then advanced for further evaluation.

Key considerations for project definition include:

- Initial alternatives were screened for fatal flaws related to single point of failure, inadequate gravity capacity, high cost due to excessive length, geologic or seismic hazards, onerous right-of-way acquisition, and major utility conflicts.
- Six initial RWP alternatives within the pipeline study area were identified for evaluation. As a result of fatal flaw screening, RW Alt 6 was eliminated from consideration. The upstream connection location for RW Alt 5 was modified and the revised alternative was carried forward for further evaluation.
- Four initial FWP alternatives within the pipeline study area were identified for evaluation.
- Based on consideration of the poor condition of nearby existing conduit, FW Alt 5 was added for further evaluation.

8.4 Refined Pipeline Alternatives

In the second phase of pipeline alternatives evaluation, the alternatives advanced from the initial evaluation were further refined using findings from geotechnical explorations, permitting regulations assessment, preliminary cost estimates, hydraulic modeling, and feedback from community outreach.

Each refined pipeline alternative was evaluated comparatively to the other alternatives assuming a single pipeline within the route. After each alternative was scored, viable paired alternatives (for construction of two pipelines) were further evaluated considering benefits, trade-offs, and comparative costs.

This section includes the following content:

- Refined Raw Water Alternatives
- Refined Finished Water Alternatives
- Refined Pipeline Alternatives Fatal Flaw Screening
- Refined Pipeline Alternatives Evaluation
- Refined Pipeline Alternatives Summary

8.4.1 Refined Raw Water Alternatives

Six refined RWP alternatives were identified as described in Table 8-5 and shown on Figure 8-8 below. RW Alts 1 and 3 start at the existing Hudson Road Intertie, while RW Alts 1A, 2, 4, and 5 require a new intertie farther downstream on SE Lusted Road. RW Alt 1A was added to the evaluation, as this alignment would re-use the existing conduits within SE Lusted Road, which adds significant economic value for a second pipeline connection.

Table 8-5. Refined RWP Alternatives

Alternative	Length (ft)	Description
RW Alt 1	10,600	• Connection at Hudson Road Intertie • Alignment on PWB easement
RW Alt 1A	3,600	• Connection at a new intertie on SE Lusted Road • Direct route west to filtration facility site
RW Alt 2	7,300	• Connection at a new intertie on SE Lusted Road near the intersection with SE Dodge Park Boulevard • Alignment on SE Dodge Park Boulevard
RW Alt 3	12,400	• Connection at Hudson Road Intertie • Alignment partially on PWB easement, SE Dodge Park Boulevard, SE Proctor Road, SE Bluff Road, and along a new access road to the filtration facility site
RW Alt 4	3,900	• Connection at a new intertie on SE Lusted Road • Alignment directly southwest to the filtration facility site; includes replacement of approximately 2,000 feet of existing upstream conduit for seismic mitigation
RW Alt 5	8,300	• Connection at a new intertie on SE Lusted Road near the intersection with SE Dodge Park Boulevard • Alignment on SE Dodge Park Boulevard, SE Proctor Road, SE Bluff Road, and along a new access road to the filtration facility site

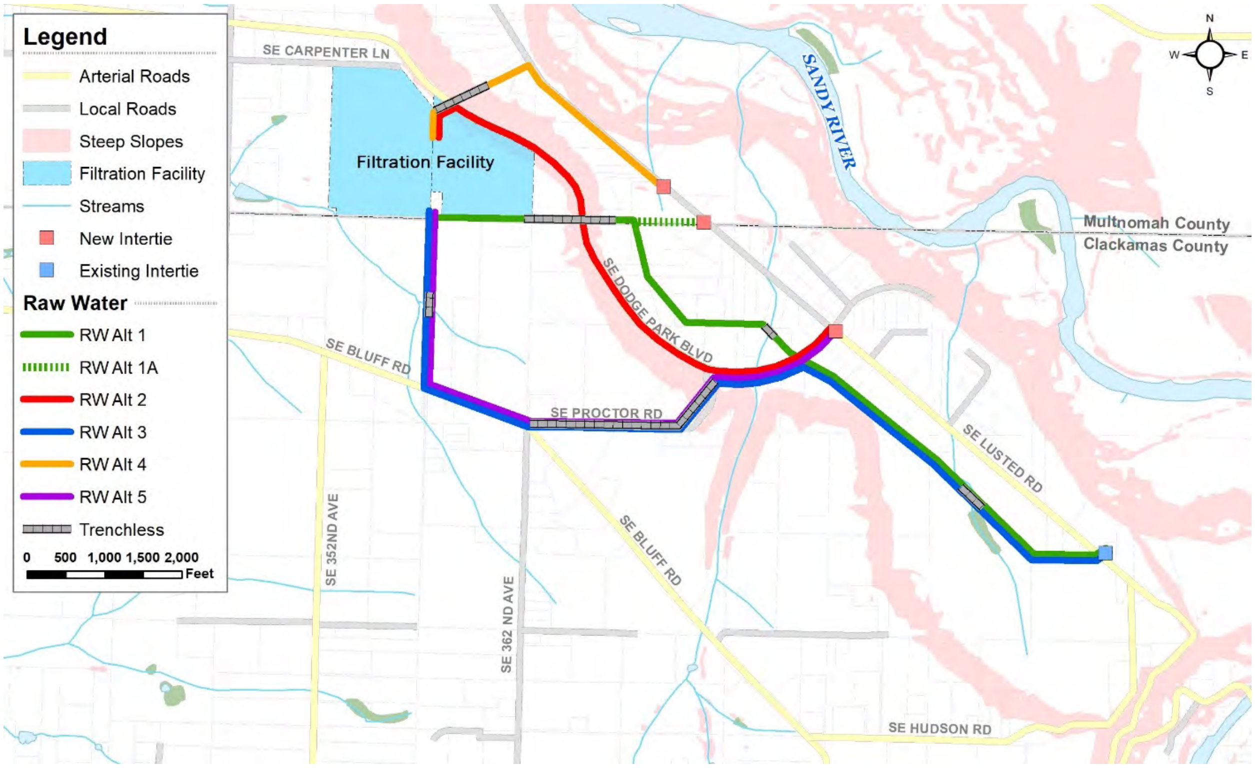


Figure 8-8. Refined RWP alternatives, including major trenchless crossings

Significant RW Trenchless Crossings

Trenchless construction needed for the raw water alternatives is summarized here, as it has a large impact on overall cost. Both the RW 1 and 1A paired alternative and the RW 3 and 5 paired alternative require a significant trenchless segment to avoid the geologic hazard risks, as discussed above, and for stream and wetland crossings. Two locations (Alts 1 and 1A and Alts 3 and 5) have been evaluated for this major crossing and are shown on Figure 8-8 above.

Each of the trenchless segments will require a deep reception shaft on the upper terrace, approximately 140 to 160 feet deep and a shallower launch shaft on the lower terrace, approximately 15 to 25 feet deep. Both shafts will need to be set back from the slope in accordance with further analysis of the selected alternatives by the pipeline designer. Boulders have been identified in existing borings and are expected within the pipeline tunneling profile.

During detailed design, trenchless methods will be evaluated, but due to the presence of boulders, a conventional tunnel has been assumed for project definition and cost estimate development. Obstructions such as boulders require access to the face of the tunneling machine, which is not possible with microtunneling equipment.

RW Alts 3 and 5 require a second trenchless tunnel along SE Proctor Road to keep the pipeline below the HGL. This trenchless crossing can be kept shallow and likely will avoid boulders, so for project definition and cost estimate development, a microtunnel has been assumed.

8.4.2 Refined Finished Water Alternatives

Five refined FWP alternatives to connect the filtration facility to the existing downstream conduits were identified, as described in Table 8-6 and shown on Figure 8-9. Each alternative, with the exception of FW Alt 4, will require interties at downstream connections.

Table 8-6. Refined FWP Alternatives

Alternative	Length (ft)	Description
FW Alt 1	8,200	• Alignment on SE Dodge Park Boulevard, SE Cottrell Road, SE Lusted Road, and SE Hosner Road • FWP connections to Conduit 3 on SE Cottrell Road, and to conduits 2 and 4 on SE Hosner Road
FW Alt 2	8,400	• Alignment on SE Dodge Park Boulevard, within private property boundaries, and SE Hosner Road • Connections to Conduit 3 within private property boundaries, and to conduits 2 and 4 on SE Hosner Road
FW Alt 3	13,800	• Alignment on SE Dodge Park Boulevard and SE Altman Road • Connections to conduits 2, 3, and 4 on SE Altman Road
FW Alt 4	3,600	• Alignment directly northwest • Connections to conduits 2, 3, and 4 on SE Lusted Road; includes replacement of approximately 2,000 feet of existing downstream conduit for seismic mitigation
FW Alt 5	14,100	• Alignment on SE Dodge Park Boulevard, within private property boundaries, SE Lusted Road, and SE Altman Road • Connections to conduits 2, 3, and 4 on SE Altman Road

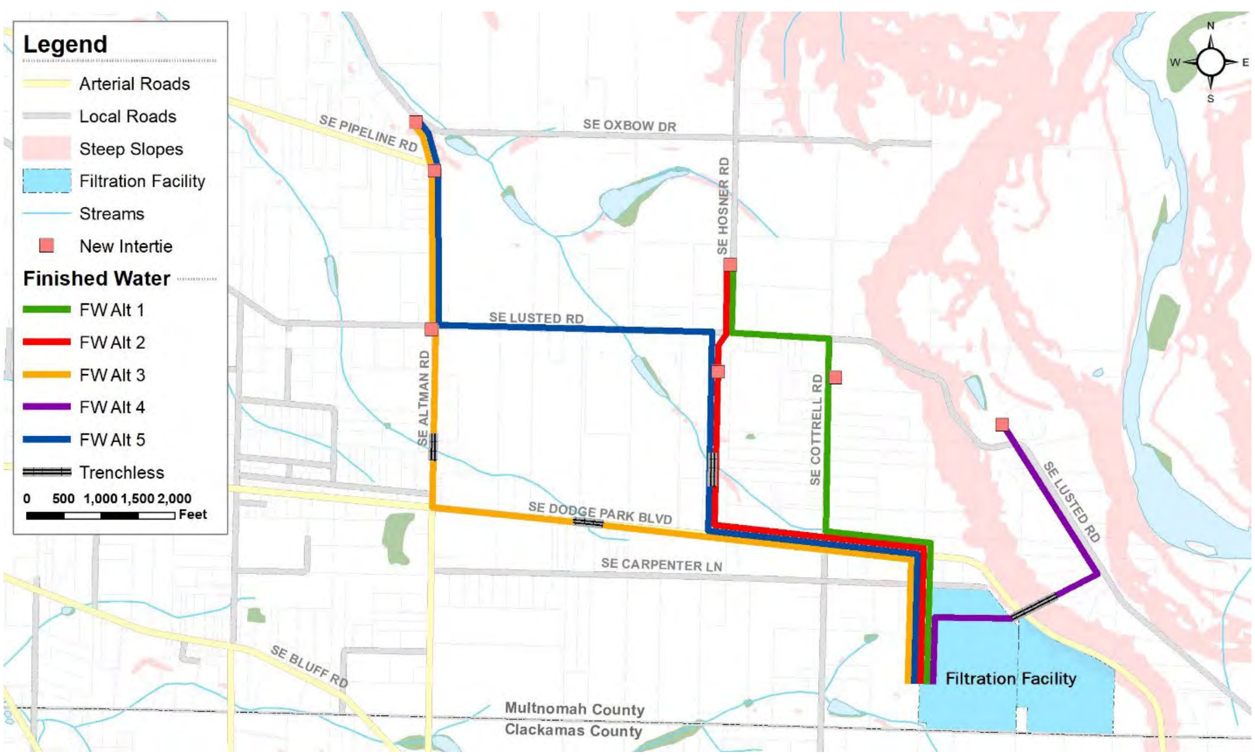


Figure 8-9. Refined FWP alternatives

8.4.3 Refined Pipeline Alternatives Fatal Flaw Screening

A fatal flaw screening analysis was conducted at the refined pipeline alternatives workshop using recommendations provided by the GTAC. The GTAC, as described in Section 8.1.5, is a group of technical experts identified and contracted with the program to provide advice regarding geotechnical and seismic considerations.

RW Alt 2 Eliminated

While not deemed by the GTAC as fatally flawed, RW Alt 2 was recommended to be set aside due to geologic and seismic hazards and challenging constructability requirements along the proposed alignment. RW Alt 2 is located within the narrow roadway of SE Dodge Park Boulevard (approximately 25 to 30 feet shoulder to shoulder, varies by location).

Between SE Proctor Road and the filtration facility site, the elevation of the roadway begins to climb from the lower terrace on SE Lusted Road to the upper terrace at the filtration facility site (Figure 8-10).

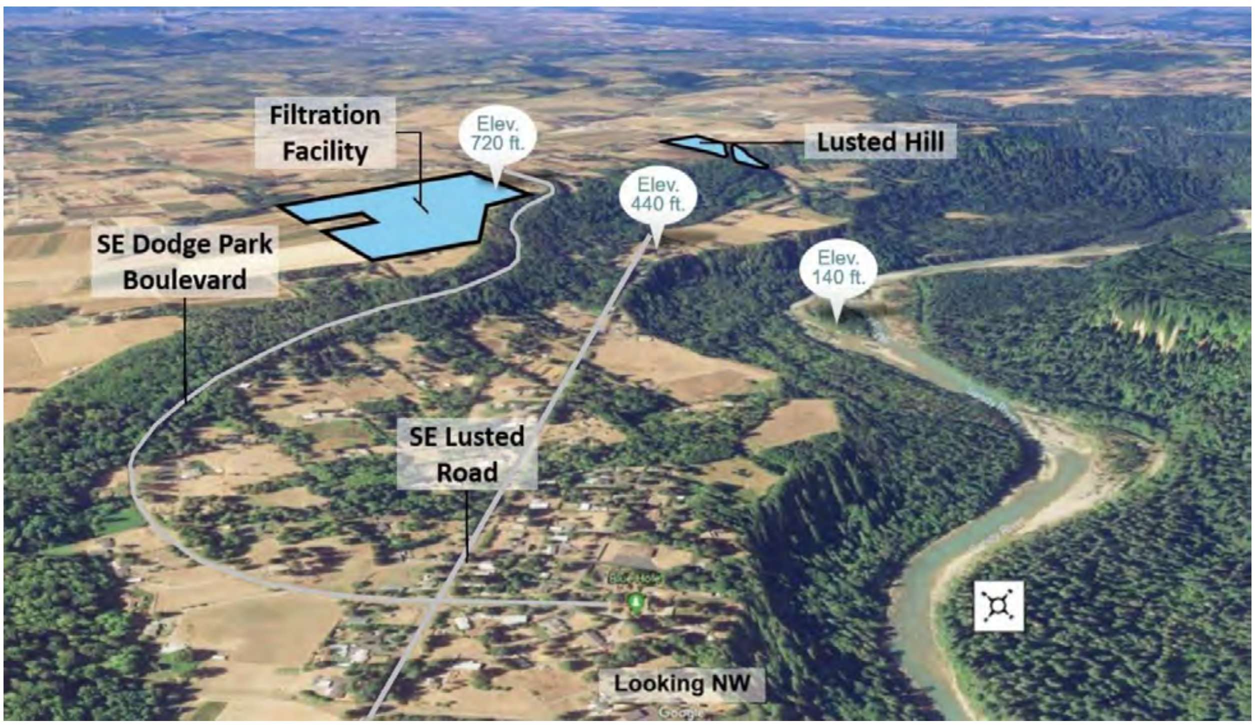


Figure 8-10. Aerial view showing the elevation change from the Sandy River to the filtration facility site

Several shallow/surficial failures (5 to 10 feet deep) were identified in geotechnical explorations on SE Dodge Park Boulevard, as the road begins to gain elevation near the RW Alt 1 and 1A tunnel. Typical open trench pipeline excavations for the RWP are expected on the order of 15 feet (depth to bedding). To install the RWP below these shallow failures, excavations between 20 to 25 feet would be required. Excavations of this magnitude located within the steep slopes of SE Dodge Park Boulevard will require significant and costly shoring systems (e.g., soldier pile walls with tieback or soil nail walls).

Constructability issues are further complicated by the compact work zone and narrow width of SE Dodge Park Boulevard. The preferred method of seismic pipeline design is to avoid hazards, rather than mitigate them. Even if these challenging constructability issues can be surmounted, installing the pipeline transverse to the failure plane of these hazards is to be avoided when less risky alternatives are available. For these reasons, further analysis of RW Alt 2 was tabled; however, the alternative was scored, and these findings were presented at the workshop.

RW Alt 4 Eliminated

Based on field explorations and historical knowledge of the region, the GTAC concluded that RW Alt 4 is fatally flawed, because of “very high” seismic hazard risks where SE Lusted Road approaches the steep slope above the Sandy River. This is approximately 1,500 feet northwest of the Multnomah County line on SE Lusted Road. In addition, the estimated shaft depth required for the trenchless crossing at the filtration facility site is likely too deep to be considered feasible. Furthermore, there is not enough room within the existing right-of-way to provide a reasonable setback to the top of the slope to minimize or avoid the hazard.

Figure 8-11 shows the location of these seismic hazards for the RWP alternatives.

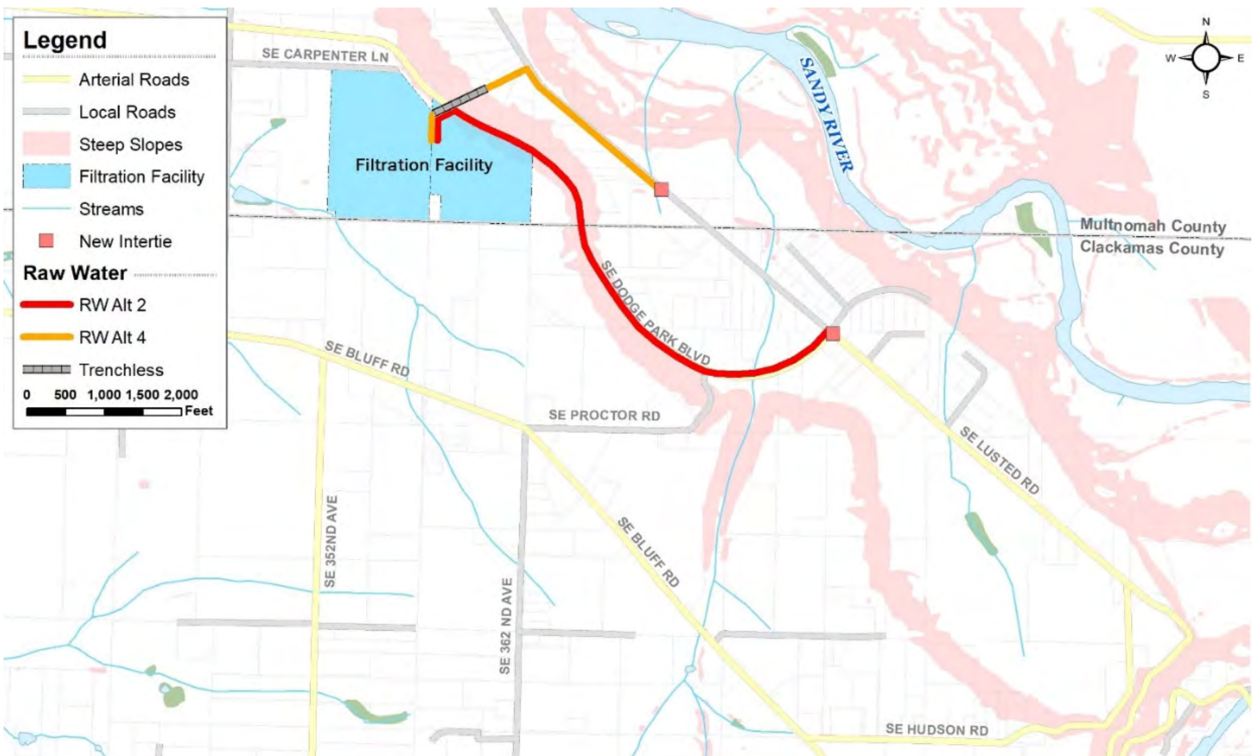


Figure 8-11. Fatally flawed (and not considered further) RWP alternatives

FW Alt 4 Eliminated

Based on the same findings and recommendations provided by the GTAC for RW Alt 4, FW Alt 4 was eliminated. FW Alt 4 is in nearly the same location as RW Alt 4 adjacent to the steep slope above the Sandy River (Figure 8-12). This slope is a "very high" seismic hazard risk and significant seismic mitigation would be required to construct along this alignment. It is best practice to avoid significant seismic hazards.

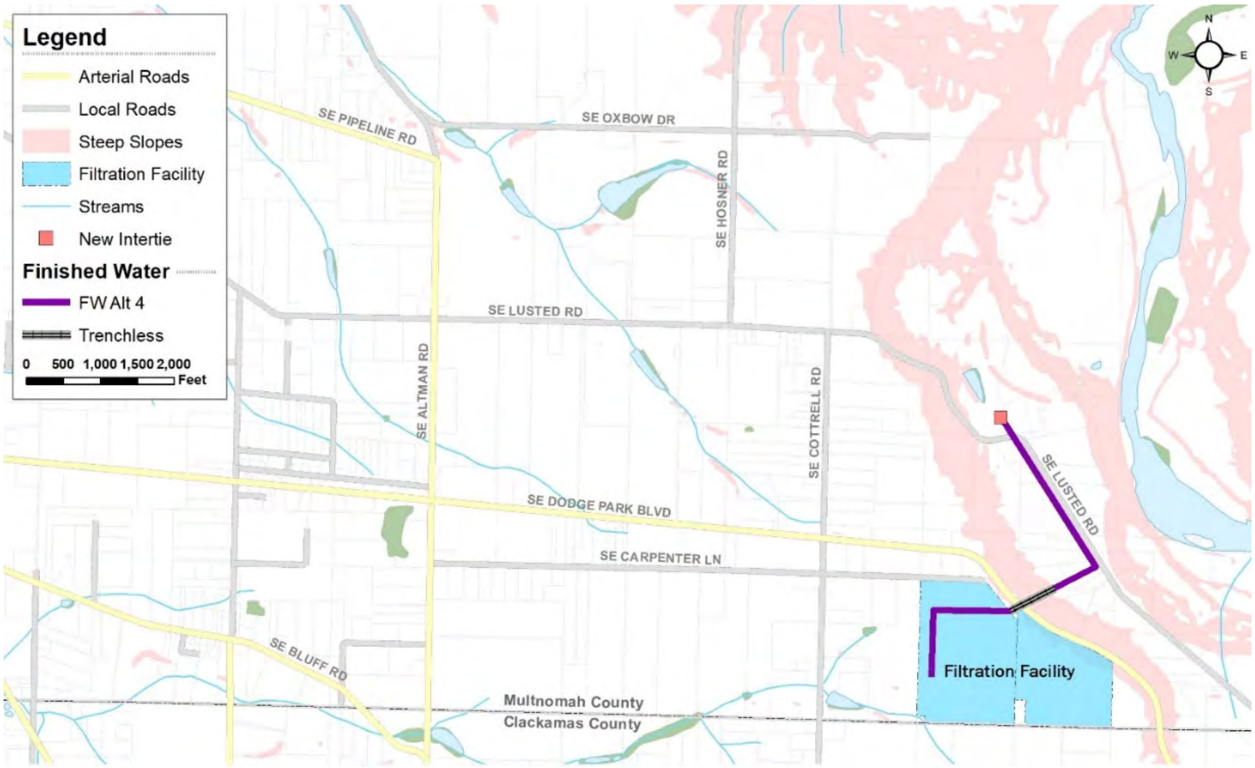


Figure 8-12. Fatally flawed FWP alternative

8.4.4 Refined Pipeline Alternatives Evaluation

In the second phase of the alternatives evaluation, the refined alternatives were comparatively rated on a score of 1 to 5 for each of the project values assuming a single pipeline within the proposed alignment. The consideration of alternatives focused on evaluating feasible pairings to identify two RWP alternatives and two FWP alternatives for further study.

Prior to the refined alternatives workshop, the RWP and FWP alternatives were pre-scored based on prior pipeline-specific meetings. In addition, scoring input was provided by subject matter experts (e.g., permitting, environmental, cultural resources, constructability, and geotechnical). Scores were then validated or adjusted based on discussion and consensus at the workshop. The resulting scoring was used to highlight pipeline route differences, prompt discussion around potential concerns, and help advance alternatives for further consideration.

Comparative Cost Criterion

The costs presented at the workshop and the evaluation tables herein were comparative only, for the purpose of evaluating alternatives (Class 5 estimate). Cost estimates considered only construction costs, not total project costs (engineering, land acquisition, permitting, and program management are excluded), and were determined using a per inch of diameter per foot of alignment unit cost estimate based on other similar, recent pipeline projects in the Portland metropolitan area. Final cost of the RWPs and FWPs will depend on many factors that are currently still under evaluation and will be optimized to meet project budget constraints.

These factors include:

- Pipeline diameter and length
- Design capacity
- Dual or single pipelines
- Pipeline route
- Existing geological conditions
- Existing infrastructure conditions
- Additional infrastructure needs (i.e., new interties)
- Permitting requirements
- Property acquisition costs
- Extent of existing conduit replacement (i.e., limits of Conduit 3 replacement on the FWP)
- Trenchless lengths and selected trenchless method technologies
- Separate or “shared” tunnels for dual pipeline routes

Refined Raw Water Alternatives Evaluation

The final value ratings and rank for the refined RWP alternatives based on the second pipelines workshop are shown on Figure 8-13. The ratings and comparative cost for each alternative are based on a single pipeline.

Raw Water Pipeline Alignment Alternatives			Pipe Size	Public Health and Water Quality	Resiliency/Reliability	Integration	Comparative Cost	Implementation	Meet Future Needs	Community Interests	Environmental Impacts	Sum	Rank
RW Alt 1	Green	"C-5" easement	Single 90"	N/A	4	5	$52M	4	5	5	3	26	1
RW Alt 1A	Dashed Green	Lusted Road direct	Single 90"	N/A	3	3	$37M	4	3	4	4	21	3
RW Alt 2	Red	Dodge Park Blvd	Single 90"	N/A	2	3	$47M	2	3	3	4	17	5
RW Alt 3	Blue	"C-5" + Proctor	Single 96"	N/A	4	5	$59M	3	5	3	3	23	2
RW Alt 4	Orange	Tunnel from Lusted	Single 90"	N/A	1			1	FATAL FLAWS FOR RESILIENCE & IMPLEMENTATION				
RW Alt 5	Purple	Dodge Park + Proctor	Single 96"	N/A	4	3	$52M	3	3	3	4	20	4

Figure 8-13. Refined RWP alternatives value ratings and rank

With the understanding that two RWPs will be constructed, attendees at the workshop evaluated multiple viable pairings. Table 8-7 shows possible RWP paired alternatives, including the benefits, trade-offs, and total comparative costs for each pairing. A total of five RWP pairings were evaluated and discussed. Ultimately, the group determined that pairing Alts 1 and 1A provided the most benefit, fewest trade-offs, and best value. In addition, the combination of Alts 1 and 1A allows the possibility of a combined trenchless tunnel. Two alternatives that share a common tunnel alignment will significantly save costs.

Table 8-7. Refined RWP Paired Alternatives Evaluation Summary

Paired Alternatives	Benefits	Trade-offs	Comparative Cost[a]
RW Alts 1 & 1A	• Combined tunnel possible • Mostly independent alignments	• Some shared alignment • Rely on existing conduits	$89M
RW Alts 1 & 3	• Mostly independent alignments	• Combined tunnel not possible • Some shared alignment	$111M
RW Alts 1A & 3	• Two independent alignments	• Combined tunnel not possible • Rely on existing conduits	$96M
RW Alt 1 (dual)	• Combined tunnel possible	• Shared alignment	$104M
RW Alts 1 & 5	• Two independent alignments	• Combined tunnel not possible • Rely on existing conduits	$104M

a. Class 5 estimate; assumed tunnel costs are subcontractor costs.

Refined Finished Water Alternatives Evaluation

The final value ratings and rank for the refined FWP alternatives based on the second pipelines workshop are shown on Figure 8-14. The ratings and comparative cost for each alternative are based on the assumption of constructing a single pipeline.

Finished Water Pipeline Alignment Alternatives			Description	Public Health and Water Quality	Resiliency/Reliability	Integration	Comparative Cost	Implementation	Meet Future Needs	Community Interests	Environmental Impacts	Sum	Rank
FW Alt 1	Green	Hosner via Cottrell	Single 72"	N/A	4	5	$29M	2	4	2	5	22	4
FW Alt 2	Red	Hosner via farmland	Single 72"	N/A	4	5	$30M	4	4	4	3	24	3
FW Alt 3	Orange	Altman via Dodge Park	Single 66"	N/A	5	4	$43M	5	5	3	4	26	1
FW Alt 4	Purple	Lusted Tunnel	Single 72"	N/A	1			1	FATAL FLAWS FOR RESILIENCE & IMPLEMENTATION				
FW Alt 5	Blue	Altman via farmland and Lusted Road	Single 66"	N/A	5	4	$43M	4	5	4	3	25	2

Figure 8-14. Refined FWP alternatives value ratings and rank

Similar to the raw water alternatives, it was assumed two FWPs will be constructed, and multiple viable paired alternatives were evaluated. Table 8-8 shows FWP paired alternatives, including the benefits, trade-offs, and total comparative costs for each pairing. A total of six FWP paired alternatives were evaluated. FW Alts 3 and 5 were the highest ranked paired alternatives as the alignments provide the most benefits, fewest trade-offs, and best value. Adjusting the alignment of FW Alt 5 to avoid apparent surface water features, if possible, was discussed at the workshop.

Table 8-8. Refined FWP Paired Alternatives Evaluation Summary

Paired Alternatives	Benefits	Trade-offs	Comparative Cost[a]
FW Alts 3 & 5	• Replaces poor condition existing conduit and access easement	• Some shared alignment • EFU easement required • Some farmland easement required	$86M
FW Alts 2 & 3	• Replaces poor condition existing conduit	• Some shared alignment • New easement required • Some farmland easement required • Retains access constraints to existing easement	$73M
FW Alts 2 & 5	• Replaces poor condition existing conduit	• Most shared alignment • Most farmland easement required • Retains access constraints to existing easement	$73M
FW Alts 1 & 2	• Shortest combined route	• Some shared alignment • Relies on poor condition existing conduit • EFU easement required • Some farmland easement required • Requires deep pipeline excavation due to HGL • Retains access constraints to existing easement	$59M
FW Alts 1 & 5	• Replaces poor condition existing conduit • Mostly independent alignments	• Some farmland easement required • Requires deep pipeline excavation due to HGL • Retains access constraints to existing easement	$72M
FW Alts 1 & 3	• Replaces poor condition existing conduit • Mostly right-of-way • Mostly independent alignments	• Requires deep pipeline excavation due to HGL • Retains access constraints to existing easement	$72M

a. Class 5 estimate; assumed tunnel costs are subcontractor costs.

8.4.5 Refined Pipeline Alternatives Summary

This section described the second phase of the pipeline alternatives evaluation process, during which refined RWP and FWP alternatives were evaluated. This evaluation was informed by findings from additional geotechnical explorations, wetland and sensitive species investigations, environmental assessments, cultural resources, and other field investigations. In addition, these alternatives were used to develop the *Basis of Estimate Report* (Appendix B).

Following the refined alternatives workshop, additional geotechnical analyses were completed for RW Alts 1 and 1A to confirm the tunnel location. These additional geotechnical borings included deep sonic drillings (200 or more feet below ground surface) into the geologic formations of the expected trenchless profile, which allowed characterization of the soil properties to better understand the expected tunneling means and methods.

Key considerations for project definition include:

- RW Alts 1 and 1A were ranked highest and advanced for further evaluation (Figure 8-15).
- Although RW Alts 3 and 5 did not score as high, they were also advanced for further evaluation as a fallback if additional explorations along the RW Alts 1 and 1A trenchless crossing deemed the routes infeasible due to problematic tunneling conditions, deep shaft construction, or permitting (environmental or land use) concerns.
- FW Alts 3 and 5 were ranked highest and advanced for further evaluation (Figure 8-16).

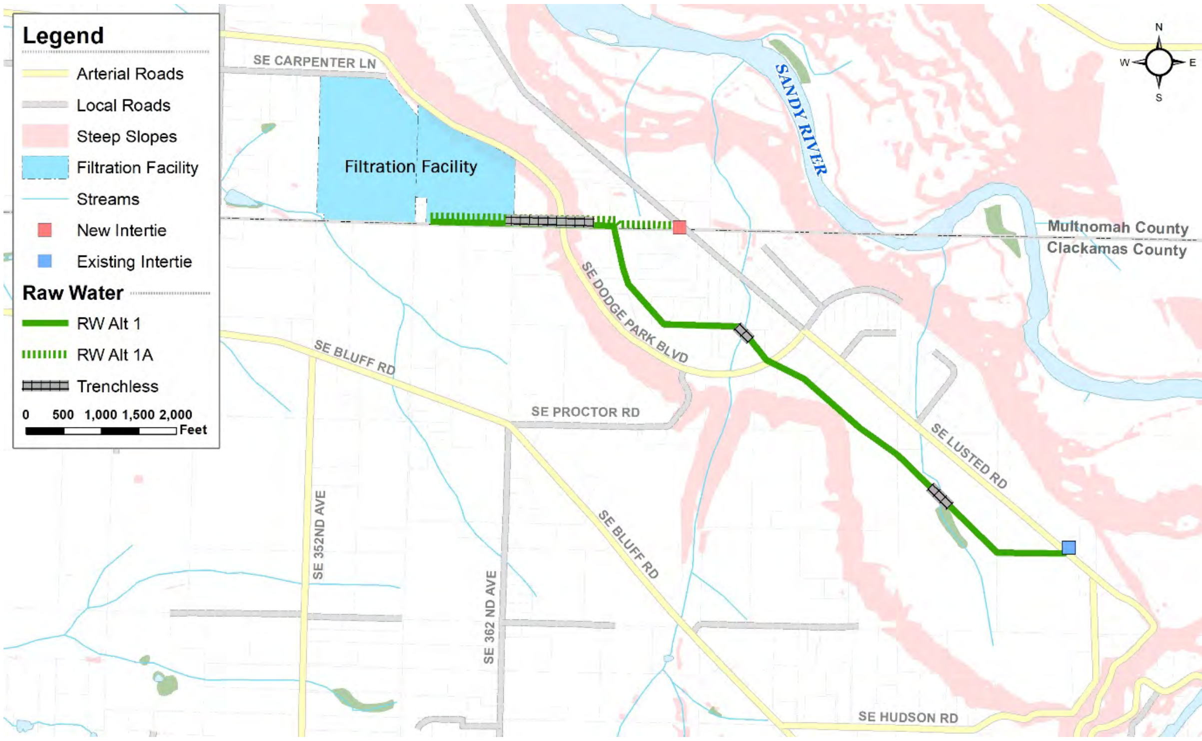


Figure 8-15. Highest ranked refined RWP alternatives

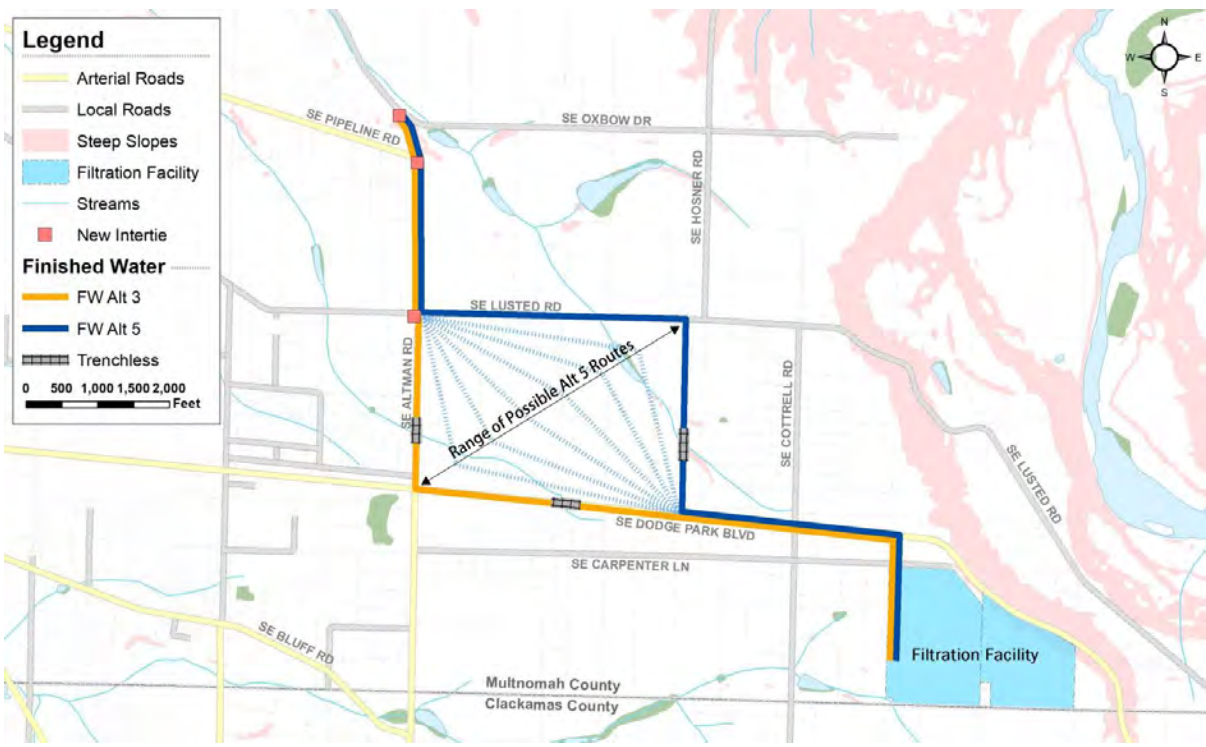


Figure 8-16. Highest ranked refined FWP alternatives

8.5 Optimized Pipeline Alternatives

In the third and final phase of pipeline alternatives evaluation, optimized variants of the refined alternatives advanced from the second workshop were developed and further evaluated.

After the refined alternatives workshop, additional field investigations were performed along the highest ranked alignments, including:

- **Phase II ESA.** Soils were collected and tested for petroleum hydrocarbons, pesticides, herbicides, and metals.
- **Wetland Delineation.** Wetland limits were surveyed, and sensitive species investigations were conducted.
- **Cultural Pedestrian Survey.** Visual surveys of the properties were performed and shallow soil probes were examined for historic or cultural relevance.
- **Geotechnical Explorations.** Additional borings were performed, and groundwater monitoring dataloggers were installed.
- **Hydraulic Modeling.** Preliminary modeling of the alternatives were developed and incorporated with models of existing infrastructure. Final pipe sizing will be based on the final model after calibration is conducted in late 2020.
- **Traffic.** Construction traffic and possible detour routes were investigated and analyzed to determine viable traffic control alternatives.
- **Survey.** Topographic and utility surveys were conducted.

The third phase of evaluation occurred after the City Council direction for two pipes to and from the filtration facility; therefore, paired alternatives were considered. Following the optimized alternatives workshop, the RWP and FWP alternatives were shared with community members to gather feedback. The resulting highest ranked alternatives were advanced as preferred alternatives.

This section includes discussion of the following topics:

- Optimized Raw Water Alternatives
- Optimized Finished Water Alternatives
- Optimized Pipeline Alternatives Evaluation
- Optimized Pipeline Alternatives Summary

8.5.1 Optimized Raw Water Alternatives

After the second pipelines workshop, optimized variants of RW Alts 1 and 1A were identified for further study. Due to the cost of tunneling under the upper terrace to the filtration facility, open cut alternatives for the steep slope were also identified for further evaluation. Additionally, permitting considerations for Multnomah and Clackamas counties were assessed and alternate tunnel locations were evaluated. All alternatives include one pipeline from the Hudson Road Intertie on SE Lusted Road along existing PWB easements to the south of and parallel to SE Lusted Road to SE Dodge Park Boulevard.

Two intertie locations west of SE Lusted Road were considered for variations to Alt 1A: one in Multnomah County and one in Clackamas County.

The five paired RWP alternatives (two open-cut and three tunnel alternatives) considered in the optimized alternatives workshop are shown on Figure 8-17.

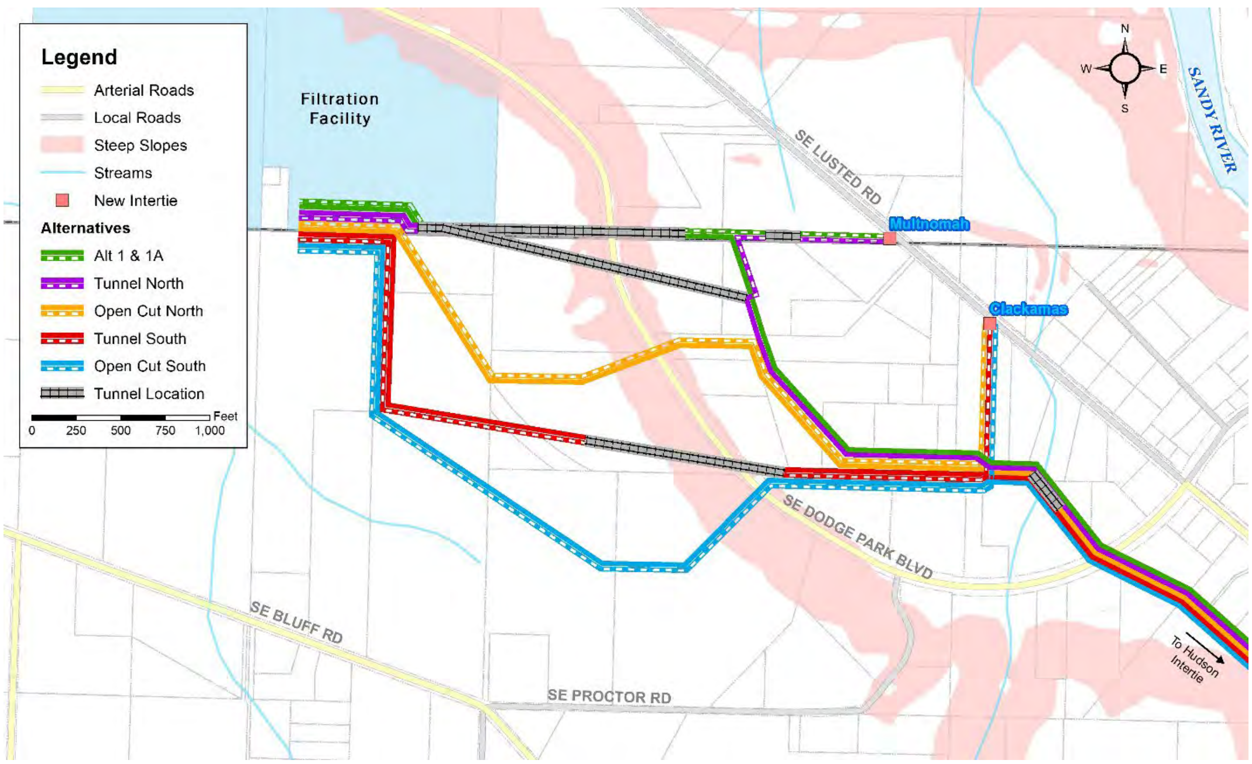


Figure 8-17. Optimized RWP paired alternatives

8.5.2 Optimized Finished Water Alternatives

Consistent with the raw water alternatives, the FWP alternatives were optimized and further studied after the second pipelines workshop.

Optimization of the FWP alternatives resulted in a new alternative along SE Carpenter Lane (Alt 3C) and the adjustment of Alt 5 to avoid multiple stream and wetland crossings between SE Dodge Park Boulevard and SE Lusted Road (Alt 5A). FW Alt 3C provides more resilience, if paired with an alternative along SE Dodge Park Boulevard (separate and independent alignments). FW Alt 3C would also reduce traffic impacts to the greater community (road closures with detours) compared to two pipelines installed along SE Dodge Park Boulevard, which would have a longer construction duration.

Additionally, FW Alt 5A was slightly truncated to connect to the existing conduit at SE Lusted Road and SE Altman Road with an optional segment (Alt 5 Optional) extending north to SE Pipeline Road. FW Alt 3 was unchanged from the refined alternatives workshop.

The FWP paired alternatives considered in the optimized alternatives workshop are shown on Figure 8-18.

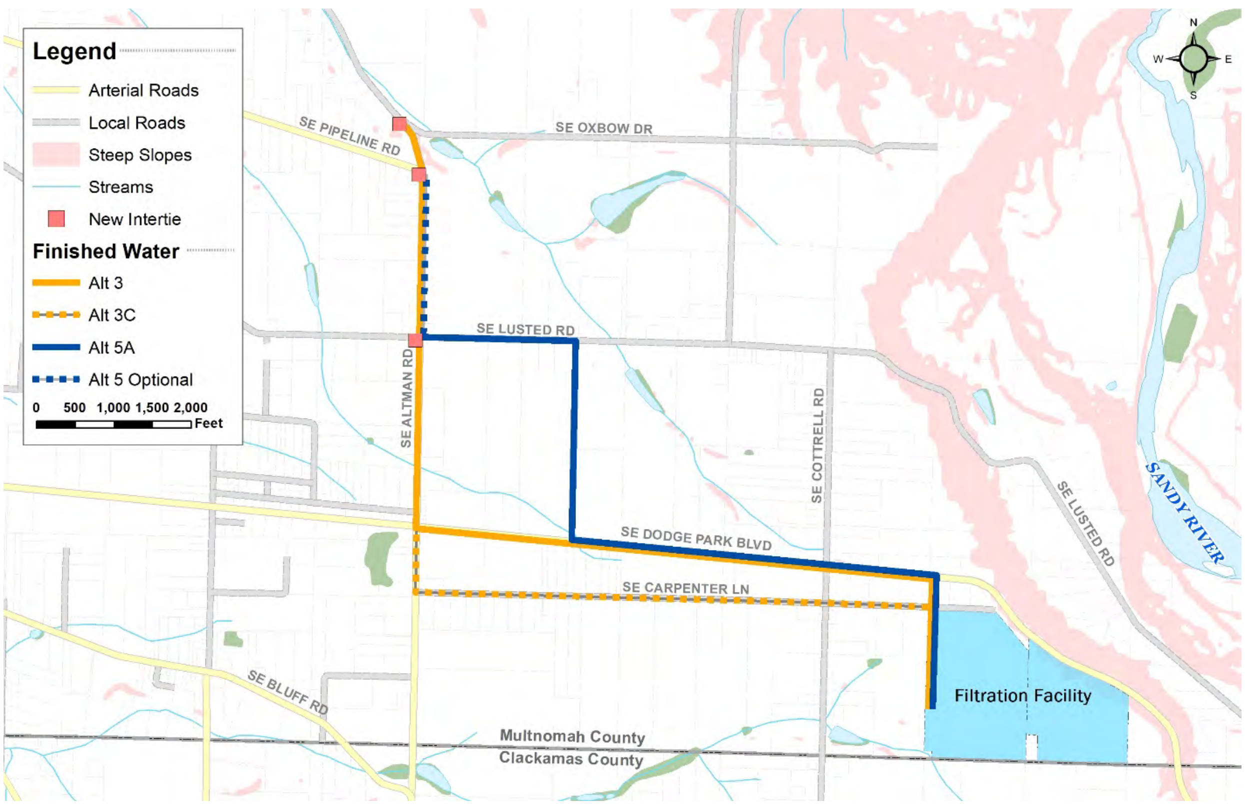


Figure 8-18. Optimized FWP paired alternatives

Additional Considerations

All three optimized FWP alternatives converge at the intersection of SE Lusted Road and SE Altman Road, connect to Conduit 3, then continue north to SE Oxbow Drive to connect to conduits 2 and 4. The need for two FWPs downstream of the Conduit 3 connection along the common alignment north of SE Lusted Road was discussed at the optimized alternatives workshop.

During normal operating conditions, design capacity can be achieved with a single FWP downstream of the Conduit 3 connection. However, if that single FWP is required to be taken out of service for maintenance, capacity to Portland is limited to approximately 65 mgd (existing capacity of Conduit 3). The average daily demand is 85 mgd, so under this scenario a single FWP would not provide sufficient capacity. Although, it should be noted that PWB would not be compromising their preference for dual pipelines, as this single FWP would be constructed downstream of a connection with an existing conduit, which would serve as the second pipeline in the "dual" pipe philosophy.

While construction of a second pipeline north of SE Lusted Road would have additional costs associated and likely require temporary or permanent easements west of the right-of-way, which is zoned EFU land, above ground appurtenances can be installed within the right-of-way and no significant impact to EFU is expected. A conceptual design of the dual FWP alignments is shown on Figure 8-19. With the considerations discussed above at the workshop, a second FWP is preferred due to the benefits to capacity, resilience, and reliability.

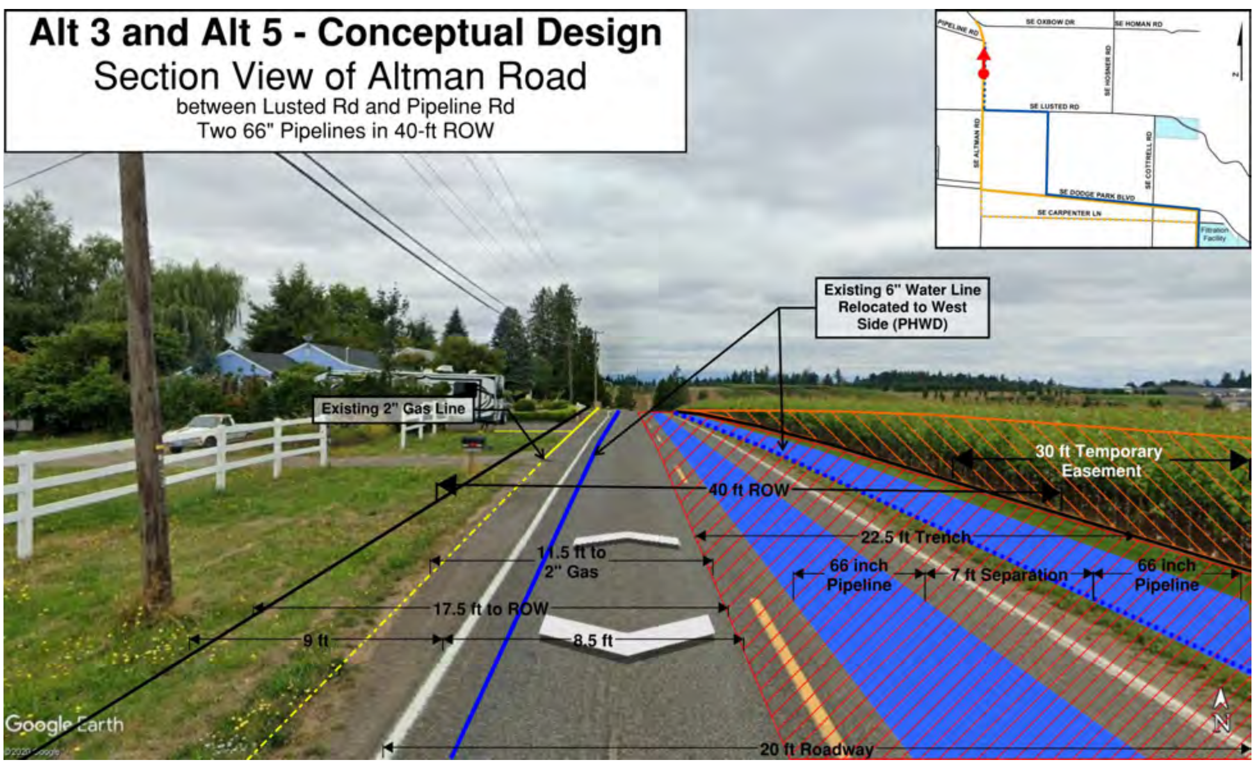


Figure 8-19. Conceptual dual FWP alternative on SE Altman Road north of SE Lusted Road

8.5.3 Optimized Pipeline Alternatives Evaluation

The optimized alternatives were evaluated using the project values applied in previous workshops. Prior to the workshop, the alternatives were pre-rated based on previous pipelines-specific meetings. In addition, rating input was provided by subject matter experts (permitting, environmental, cultural resources, trenchless, constructability, and geotechnical). Ratings were then validated or adjusted based on discussion and consensus at the workshop. The resulting summed scores were used to highlight pipeline route differences, prompt discussion around potential concerns, and help identify preferred pipeline pairings.

Comparative Cost Criterion

The costs presented at the optimized alternatives workshop and the evaluation tables herein were comparative only, for the purpose of evaluating alternatives. Cost estimates considered construction and permanent easement costs, not total project costs (engineering, land ownership acquisition, permitting, and program management costs are excluded). The comparative construction costs used Class 5 estimated unit costs and production rates generally as identified in the *Basis of Estimate Report* (Appendix B). These costs consider pipeline materials, depth of pipeline, trench shoring methods, existing ground conditions, and ground surface restoration. Due to the limitations of these estimated costs, baseline cost comparisons, rather than total costs, were used to highlight the difference in cost for each optimized alternative. Final costs for the RWP and FWP alternatives will depend on many factors currently under evaluation and will be optimized to meet project budget constraints. These factors include:

- Pipeline diameter and length
- Design capacity
- Dual or single pipelines on SE Altman Road
- Pipeline route
- Existing geologic conditions
- Existing infrastructure conditions
- Additional infrastructure needs (i.e., new interties)
- Permitting requirements
- Property acquisition costs
- Extent of existing conduit replacement (i.e., limits of Conduit 3 replacement on the FWP)
- Trenchless lengths and selected trenchless methods

Optimized RWP Alternatives Evaluation

The five RWP alternatives, as shown on Figure 8-16, require a deep tunnel or an open cut crossing of the steep slopes separating the lower terrace and the upper terrace. Design considerations for a tunneled crossing are described above in Section 8.1.11. An open cut crossing would require the hillside to be cleared of trees for an approximately 100-feet-wide corridor. An engineered shoring system approved by a geotechnical engineer would also be required for open cut installation of the RWP. For evaluation purposes, RhinoOne Geotechnical developed a conceptual design for the open cut approach (Figure 8-20 below).

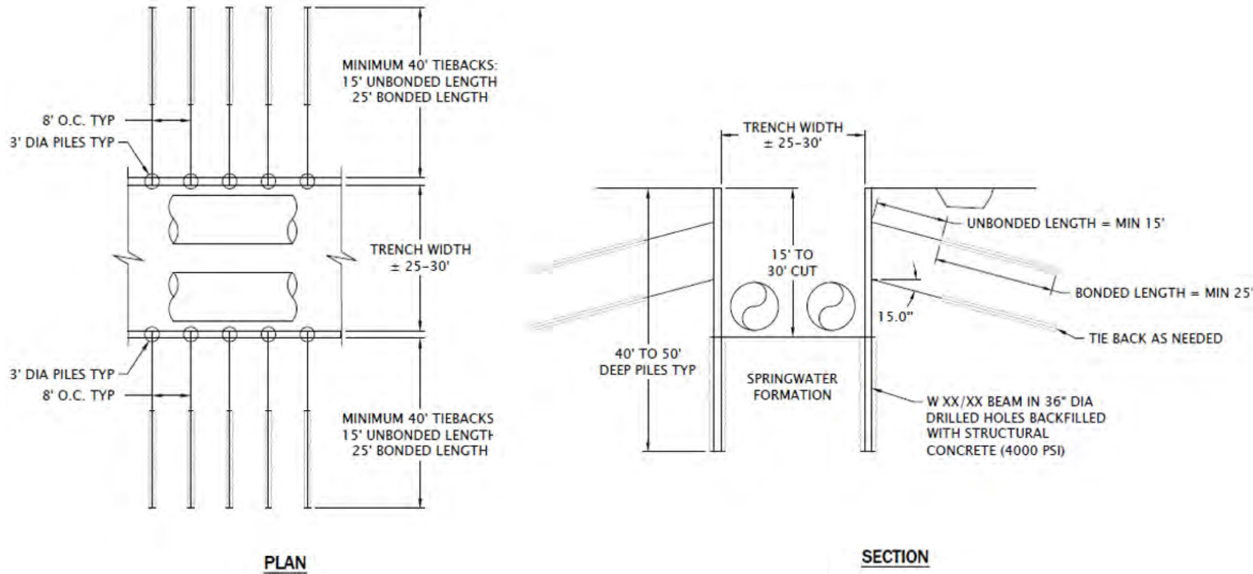


Figure 8-20. Conceptual design for an open cut crossing of the steep slopes

Figure 8-21 presents an example of an open cut pipeline installation along a steep slope.

Figure 8-21. Steep slope pipeline installation at the Allison Creek Hydro Project

Overall, the tunneled RWP alternatives scored significantly higher than the open cut alternatives. A tunneled alternative is more seismically resilient, as the alignment will more reliably avoid the seismic hazard (pipe will be constructed underneath) of the steep slope rather than relying on mitigating the hazard using an engineered open cut approach. Expected seismic mitigation for an open cut alternative could require a combination of strategies, including removal of poor soils, ground improvements or jet grouting, thicker steel pipe wall, and more robust pipe joints (butt welding). The tunneled alternatives are also expected to be less impactful to the community and environment and thus were rated higher for these values. Finally, permitting a tunneled alternative avoids hillside clearing and is expected to be more consistent with county slope hazard requirements than an open cut option.

A summary of value ratings and rank for each RWP paired alternative is shown on Figure 8-22. A more detailed evaluation of each alternative, including a list of benefits and trade-offs is described in Table 8-9 below.

Preferred RWP Alternative

As a result of the optimized alternatives evaluation, the preferred RWP alternative is a pairing of RW Alts 1 and 1A. The key differentiators for each of the values are described below.

- **Resilience/Reliability:** most seismically resilient alternative because the alignments are mostly independent and avoid seismic hazards.
- **Integration:** less frequent maintenance needs for tunnel and deep shaft.
- **Implementation:** requires the fewest easements, has less risk of permitting delays, and includes the least amount of total pipe.
- **Meet Future Needs:** no differentiation from other alternatives.
- **Community Interests:** trenchless construction avoids impacts to hillside forest, alignment limits road closure needs, and trenchless methods reduce noise and vibrations.
- **Environmental:** trenchless construction limits impacts to hillside forest habitat.

Raw Water Pipeline Alignment Alternatives		Intertie Locations	Public Health and Water Quality	Resiliency/Reliability	Integration	Comparative Cost	Implementation	Meet Future Needs	Community Interests	Environmental Impacts	Sum	Rank
RW Alt 1 & Alt 1A	Green	Hudson; Multnomah	N/A	5	4	Base Line	4	4	3	4	24	1
RW Clackamas Tunnel North	Purple	Hudson; Multnomah	N/A	5	4	+$2M	3	4	2	4	22	2
RW Open Cut North	Orange	Hudson; Clackamas	N/A	4	3	-$15M	2	4	1	3	17	4
RW Clackamas Tunnel South	Red	Hudson; Clackamas	N/A	4	4	+$1M	2	4	2	5	21	3
RW Open Cut South	Blue	Hudson; Clackamas	N/A	4	3	-$10M	1	4	1	3	16	5

Figure 8-22. Optimized RWP paired alternatives value ratings and rank

Table 8-9. Optimized RWP Paired Alternatives Evaluation Summary

Paired Alternatives	Benefits	Trade-offs	Comparative Cost[a]
RW Alts 1 & 1A	• Most seismically resilient • Most independent alignments • Least amount of total pipe • Less easement acquisition • Less risk of permitting delays • Favorable intertie location	• More costly than open cut alternatives • Tunnel construction risk	Baseline
Clackamas Tunnel North	• Most seismically resilient • Less risk of permitting delays	• More costly than open cut alternatives • More challenging to implement than RW Alts 1 and 1A tunnel • Tunnel construction risk	+$2M
Clackamas Open-cut North	• Most cost-effective alternative	• More environmental impacts than tunneled options • Least seismically resilient • Clearing of slope (community interests) • Significant easement acquisition • Higher risk of permitting delays	-($15M)
Clackamas Tunnel South	• Most seismically resilient • Less permitting risk	• More costly than open cut alternatives • More challenging to implement than RW Alts 1 and 1A tunnel • Tunnel construction risk	+$1M
Clackamas Open-cut South	• More cost-effective than trenchless	• More environmental impacts than tunneled options • Least seismically resilient • Clearing of slope (community interests) • Longest route (+26% more than RW Alts 1 and 1A) • Significant easement acquisition • Higher risk of permitting delays	-($10M)

a. Class 5 estimate.

Optimized FWP Alternatives Evaluation

The three FWP paired alternatives considered for the final phase of evaluation are shown on Figure 8-20 below. The noteworthy change from the previous workshop was the addition of FW Alt 3C running east from the filtration facility to SE Altman Road.

FW Alt 3C would provide significant resilience and reliability benefits because the two FWPs would be located on independent alignments, as opposed two pipelines running parallel and in close proximity to each other on SE Dodge Park Boulevard (FW Alts 3 and 5). Since two FWPs will be installed, the more separation between those pipelines the better to protect from possible known and unknown issues during the life of the pipeline (i.e., seismic events or possible damage by future utility contractors).

Another benefit of FW Alt 3C would be reduction of the construction duration and traffic impacts on SE Dodge Park Boulevard, which are expected for FW Alts 3 and 5. Dual pipeline construction on SE Dodge Park Boulevard will take significantly longer than single pipeline construction.

A SE Carpenter Lane alignment would provide farther separation from a pipeline on SE Dodge Park Boulevard. However, due to the narrow right-of-way along SE Carpenter Lane, this alternative would require temporary construction easements and would considerably impact immediate residents. Comparatively, the much wider right-of-way along SE Dodge Park Boulevard allows for two FWPs with at least a minimum separation. There would also be fewer residential impacts on SE Dodge Park Boulevard compared to SE Carpenter Lane.

A third alternative was also evaluated to construct three smaller FWPs, one along SE Carpenter Lane (Alt 3C) and two along SE Dodge Park Boulevard (Alt 5A). Each of the new FWPs would connect directly to the three existing Bull Run Conduits downstream. This alternative was evaluated to potentially reduce the footprint for downstream intertie facilities. As shown in the scoring results on Figure 8-23 below, this alternative scored poorly and was more costly. A major drawback to the three-pipe alternative is that it would likely require future upsizing to meet ultimate capacity goals for the filtration facility, which would further increase costs. For these reasons, the three-pipe alternative was eliminated.

A summary of scores for each of the three FWP alternatives is shown on Figure 8-23 below. A more detailed evaluation of each alternative, including a list of benefits and trade-offs is presented in Table 8-10 below.

Preferred FWP Alternative

As a result of the optimized alternatives evaluation, the preferred FWP alternative is a pairing of Alts 3 and 5. The key differentiators for each of the values are described below.

- **Resilience/Reliability:** wide right-of-way on SE Dodge Park Boulevard can allow for some separation of dual pipes with parallel alignments.
- **Integration:** shortest length of deep pipe section from the clearwell outlet at the filtration facility to the location of minimum cover depth downstream on the pipeline.
- **Implementation:** requires the least number of additional easements.
- **Meet Future Needs:** pipeline sized to meet current design capacity and can provide facility ultimate capacity (with future downstream improvements).
- **Community Interests:** less impact to residential streets; longer construction duration on SE Dodge Park Boulevard (feasible detours available).
- **Environmental:** alignments avoid need for trenchless installations for stream crossings.

Finished Water Pipeline Alignment Alternatives - Dodge Park vs. Carpenter Lane	Public Health and Water Quality	Resiliency/Reliability	Integration	Comparative Cost†	Implementation	Meet Future Needs	Community Interests	Environmental Impacts	Sum	Rank
Dodge Park Blvd - separate trenches Alt 3 & Alt 5A	N/A	**3**	**4**	Base Line	**4**	**5**	**4**	**4**	**24**	**1**
Carpenter Lane and Dodge Park - Alt 3C & Alt 5A	N/A	**5**	**4**	+$5M	**2**	**5**	**3**	**4**	**23**	**2**
Three Pipelines (two on Dodge one on Carp) Alt 3C & Alt 5Ax2 & Alt 5 Optional	N/A	**5**	**3**	+$18M	**2**	**2**	**2**	**3**	**17**	**3**

Figure 8-23. Optimized FWP paired alternatives value ratings and rank

Table 8-10. Optimized FWP Paired Alternatives Evaluation Summary

Paired Alternatives	Benefits	Trade-offs	Comparative Cost[a]
SE Dodge Park Boulevard (dual) Alts 3 & 5A	• Wide right-of-way on SE Dodge Park Boulevard • Most cost-effective alternative	• Shared alignment (less resilient) • SE Dodge Park Boulevard road closure • Impacts to filtration facility construction haul routes on SE Dodge Park Boulevard	Baseline
SE Carpenter Lane and SE Dodge Park Boulevard Alts 3C & 5A	• Separate alignments (more resilient) • Reduced impact to SE Dodge Park Boulevard (through traffic and construction traffic)	• Impacts to SE Carpenter Lane residents • Requires temporary construction easements (narrow work zone)	+$5M
Three Pipelines (two on SE Dodge Park Boulevard, one on SE Carpenter Lane) Alts 3C & 5A & 5 Optional	• Flexible operations • Eliminates interties offsite	• Pipelines undersized for ultimate capacity • Same impacts as the other two alternatives	+$18M

a. Class 5 estimate.

Additional Considerations

Three of the optimized RWP alternatives and three of the optimized FWP alternatives were shared with the community to gather feedback. Input was received over 3 months through in-person meetings and online surveys. The community feedback on pipeline route preferences and traffic and safety considerations was used to inform the evaluation process.

8.5.4 Optimized Pipeline Alternatives Summary

This section described the third phase of the pipeline alternatives evaluation process, during which optimized RWP and FWP alternatives were evaluated.

Key considerations for project definition include:

- The preferred RWP alternative is a pairing of RW Alts 1 and 1A.
- The preferred FWP alternative is a pairing of FW Alts 3 and 5.
- Two FWPs are recommended on SE Altman Road north of SE Lusted Road.

8.6 Interties

This section describes the interties required upstream of the RWPs and downstream of the FWPs to connect the filtration facility with existing conduits. The existing Hudson Road Intertie and approximate locations of new interties are shown on Figure 8-24.

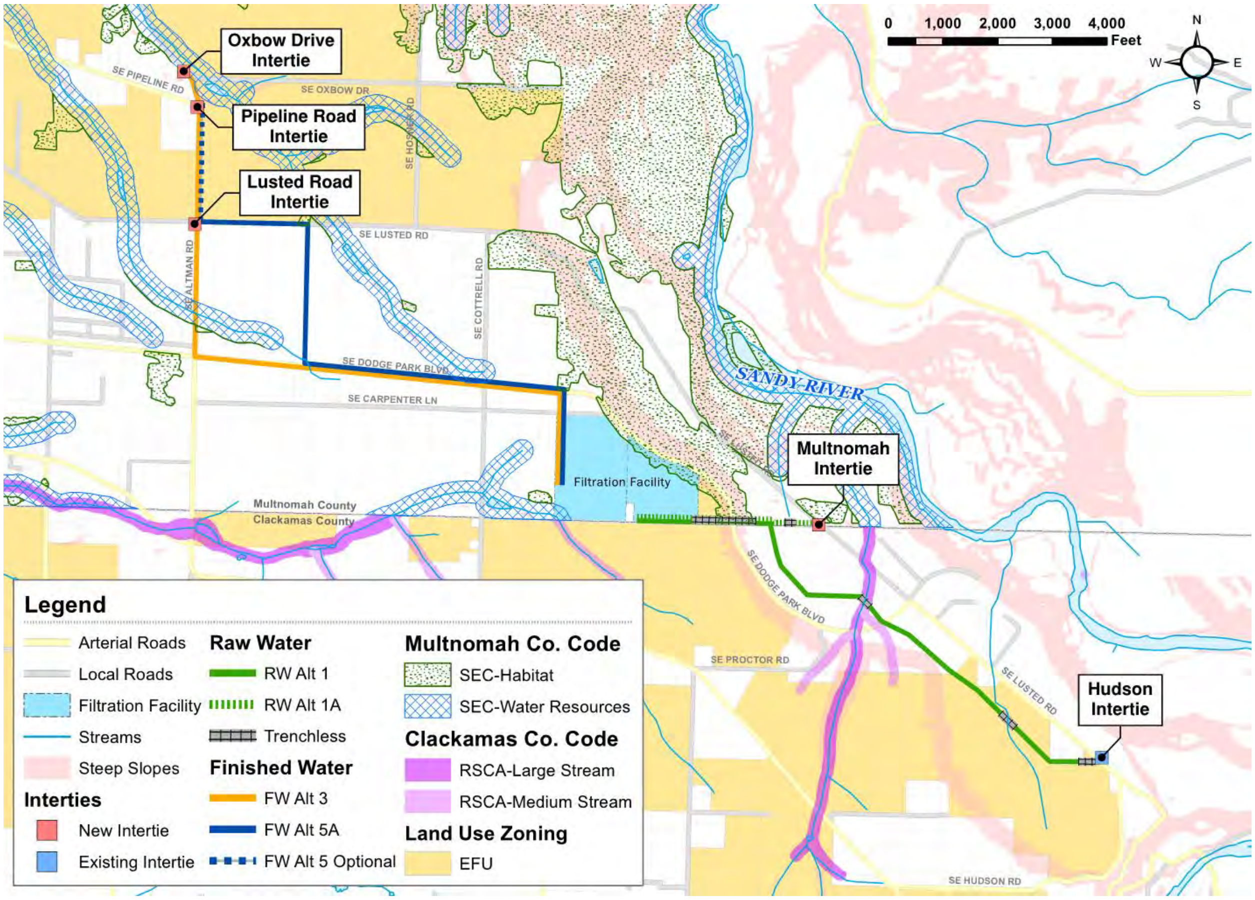


Figure 8-24. Locations for interties to connect the RWPs and FWPs with upstream and downstream conduits

8.6.1 Raw Water Interties

The following new or existing interties are anticipated to connect the Bull Run Conduits to the RWPs.

Hudson Road Intertie

Hudson Road Intertie on SE Lusted Road is where conduits 2, 3, and 4 connect with valves that allow flow in each conduit to be adjusted for demand, maintenance, and hydraulic considerations. When Hudson Road Intertie was constructed in 2004, future needs were considered. The existing 66-inch connection stub out will be the connection point for RW Alt 1.

Multnomah Intertie

RW Alt 1A will connect to the Bull Run Conduits at a new intertie west of SE Lusted Road near the southern Multnomah County boundary. The Multnomah Intertie will connect RW Alt 1A to two or three of the existing conduits and include flow control or isolation valves for conduits downstream of the Hudson Road Intertie. This intertie will be critical for startup and commissioning of the filtration facility. Temporary accommodations will likely be needed to maintain flow to the existing supply system and allow flow through the RWPs to the filtration facility before the facility is fully operational.

8.6.2 Finished Water Interties

The following new interties are anticipated to connect the FWPs to the Bull Run Conduits.

Lusted Road Intertie

Located near the SE Altman Road and SE Lusted Road intersection, the Lusted Road Intertie will connect FW Alts 3 and 5 to Conduit 3. The Lusted Road Intertie will include a flow control valve and metering for Conduit 3 downstream of the filtration facility and require a permanent easement or property acquisition of approximately 7,000 square feet. Additional valves will be installed in the right-of-way for diversion and flow control between FW Alts 3 and 5.

Pipeline Road Intertie

Located west of the SE Altman Road and SE Pipeline Road intersection, the Pipeline Road Intertie will allow FW Alt 3 and the optional segment of FW Alt 5A to connect to conduits 2 and 4. This intertie will include flow control valves and metering for downstream conduits and require a permanent easement or property acquisition of approximately 14,000 square feet.

Oxbow Drive Intertie

Located where SE Altman Road merges with SE Oxbow Drive, the Oxbow Drive Intertie will provide a direct connection of FW Alt 3 with Conduit 4. The connection will be within the right-of-way. Flow control for this intertie will be provided by the Pipeline Road Intertie. This connection is not expected to require a permanent easement.

8.7 Pipeline Alternatives Summary

This chapter described the planning considerations, values criteria, and evaluation process used to assess feasibility of potential pipeline alternatives. The preferred alternatives identified as a result of this evaluation are RW Alts 1 and 1A and FW Alts 3 and 5 (Figure 8-25). These preferred alternatives will be further investigated and validated during design.

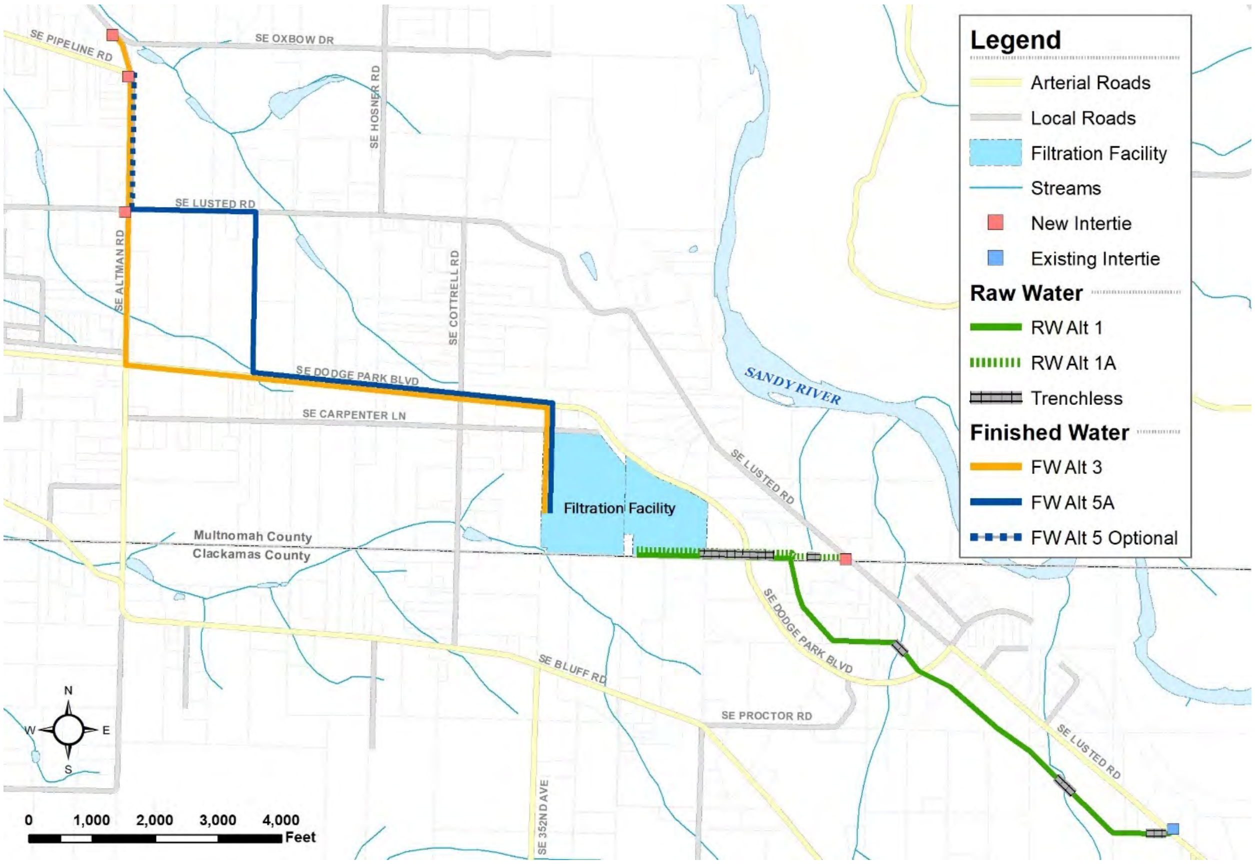


Figure 8-25. Preferred RWP and FWP alternatives

References

Jacobs, *Filtration Plant Key Decision Process TM,* PWB, September 2018.

Pure Technologies, *Conduit 3 Condition Assessment and Improvement Plan,* PWB, June 2018.

Chapter 9
Alternatives Evaluation

This chapter reaches conclusions on key project considerations that significantly impact performance and cost, starting with the feasible treatment process alternatives described in Chapter 6: Treatment Process Alternatives. Feasible alternatives for the filtration facility and pipelines were refined based on the values described in Chapter 1: Introduction, then packaged into project options with initial comparative cost estimates. Portland Water Bureau (PWB) staff recommended a project option to Portland City Council on November 13, 2019, that offered the best value to Portland customers.

This chapter describes the overall evaluation process:

- 9.1 Filtration Facility Alternatives Evaluation
- 9.2 Pipeline Alternatives Evaluation
- 9.3 Project Options
- 9.4 Alternatives Evaluation Summary

9.1 Filtration Facility Alternatives Evaluation

The key considerations for the filtration facility identified in Chapter 6 included:

- **Sedimentation.** Preference to include sedimentation in the treatment design (full-sized sedimentation basins are assumed for project definition; final selection of full-sized or hybrid basins will be validated during design).
- **Ozone**. Preference to include ozonation in the treatment design (pre-ozonation is assumed for project definition).
- **Filtration rate.** Preference to design the filters with a standard filtration rate of 6 gallons per minute/square feet (6 gpm/sf is assumed for project definition; higher rates are to be considered during design based on review of pilot plant study findings).
- **Clearwell sizing.** Suggested clearwell sizing ranges from 10 to 16 million gallons (16 MG is assumed for project definition; the clearwell size will be optimized during design).
- **Facility capacity.** An initial range of 145 to 160 million gallons per day (mgd) facility peak day finished water capacity has been considered (145 mgd is assumed for project definition; the final capacity will be validated during design in conjunction with updated cost estimates as required).

Of these five considerations, the first two (sedimentation and ozone) were subject to a rigorous evaluation by the Technical Advisory Committee (TAC). These two considerations have a significant impact on the ability of the filtration facility to deliver the priority values of water quality and resilience and reliability. The TAC evaluation was used to vet feasible alternatives to be advanced to project options for further evaluation. The alternatives for the remaining considerations (filtration rate, clearwell sizing, and facility capacity) were viewed as sufficiently defined to advance to project options.

This section includes a description of the TAC approach and outcomes, identification of the components incorporated into the project options, and a summary of filtration facility cost estimates.

9.1.1 Technical Advisory Committee Workshop

The TAC workshop was conducted from February 26 to 28, 2019, with a follow up workshop held on March 19, 2019. Participants included TAC members, the program team, and internal PWB stakeholders.

Recognizing the two key considerations (sedimentation and ozone) are interrelated, the TAC focused on the six alternatives identified in Table 9-1. All alternatives include standard filtration treatment process components, as described in Chapter 6.

Table 9-1. Sedimentation and Ozone Alternatives

Alternative	Sedimentation			Ozone	
	Full	Hybrid	Defer	Include	Defer
1–Direct without Ozone	–	–	✓	–	✓
2–Hybrid without Ozone	–	✓	–	–	✓
3–Full Conventional without Ozone	✓	–	–	–	✓
4–Direct with Ozone	–	–	✓	✓	–
5–Hybrid with Ozone	–	✓	–	✓	–
6–Full Conventional with Ozone	✓	–	–	✓	–

The TAC was tasked with developing a treatment report card for each of the six combinations, evaluating the ability of each alternative to address a range of nine treatment goals associated with the project values of 1) water quality and public health, and 2) reliability and resilience to earthquakes and fires. For each treatment goal, the alternatives were assigned a score between "no impact" and "best practice." The treatment goals and the grades for each of the alternatives are shown in Table 9-2. The report cards were developed in four teams. In some cases, consensus among the teams was not reached and a range of grades is shown.

Table 9-2. Initial Qualitative Screening and Evaluation of Alternatives

Treatment Goal	Alternative					
	1	2	3	4	5	6
	Direct	Hybrid	Full Conventional	Direct + Ozone	Hybrid + Ozone	Full Conventional + Ozone
Fire Event	1–2	2–3	3	2	3	4
Algal Toxins	2	2	3	3–4	4	4
Algae Clogging	1	2	3	2	3	4
Operational Flexibility	2	3	3	3	3–4	4
Storm Events	1	3–4	4	1	4	4
Disinfection Byproducts	2	3	3	3	4	4
Taste and Odor	2	2–3	3	4	4	4
Color	2	3	3	4	4	4
Manganese and Iron	3	4	4	3	3–4	3–4
Status	Inadequate	Selected for further evaluation	Selected for further evaluation	Inadequate	Selected for further evaluation	Selected for further evaluation

Ranking Legend: 4: Best Practice | 3: Good, works well | 2: Provides some benefit, but may be partial | 1: No or little impact

Key discussion points from the report card exercise included:

- Sedimentation basins greatly increase the ability of the filtration facility to respond to water quality challenges, primarily because of the ability to dose large amounts of powdered activated carbon (PAC) if ozone is not used.
- PAC has broad functionality in adsorbing moderate levels of contaminants such as algal toxins and compounds that cause tastes and odors, as well as elevated levels of total organic carbon (TOC) that would likely be present in the source water after a fire event.
- Ozone has even broader functionality than PAC, addressing both a wider range of contaminants (e.g., algal toxins) as well as higher concentrations of those contaminants. For many contaminants, sedimentation basins (with PAC but without ozone) graded well. Ozone with sedimentation was graded as a best practice.
- Resilience to treat water after a fire in the Bull Run Watershed should not focus solely on turbidity—other communities have seen elevated organic levels for several years after a fire. The additional organics and turbidity cannot be removed without sedimentation basins

and a solids handling system with the capacity to deal with additional solids from PAC (if used) and increased coagulant and polymer doses.

- Ozone offers both resilience benefits (including treating high consequence, low likelihood events such as algal toxins) as well as everyday benefits of better water quality. Those everyday benefits likely include improved removal of particles, removal of taste and odor-causing compounds (potentially, including those coming from recycle streams), lower levels of disinfection byproducts, and additional disinfection for microorganisms such as *Cryptosporidium*.
- The treatment process selection should consider that the filtration facility has zero liquid discharge, and ozone has additional benefits of inactivating *Cryptosporidium* or oxidizing taste and odor-causing compounds that may be recycled from the solids handling process.

The overall conclusions of the TAC were:

- **Conventional sedimentation** basins should be included in project options.
  - Sedimentation basins are essential to meeting the priority values of water quality and resilience and reliability. This conclusion was based on the ability to continue production of water after a fire and to respond moderately well to a full range of water quality risks.
  - The hybrid sedimentation basin (designed to treat winter flows) and conventional sedimentation basin (designed to treat year-round) options are very similar. When hybrid sedimentations basins are sized to account for reduced performance from colder winter water temperatures, there is little difference in size from the full-sized basins. The specific loading rate becomes a design-level decision.
- **Ozone** should ultimately be included in the filtration facility. Building the filtration facility without ozone initially, then incorporating ozone in a future phase is feasible; however, if the facility does not include ozone, the use of PAC and enlarged solids handling facilities must be considered.

The TAC recommendations regarding sedimentation and ozone and the conclusions drawn from the alternatives evaluations described in Chapter 6 were then packaged into project options for further consideration (Table 9-3).

Table 9-3. Feasible Alternatives for Key Considerations

Unit Process	Minimum Feasible Alternatives	Additional Alternatives (Greater Benefit, Higher Cost)
Sedimentation Basins	"Full" conventional sedimentation	"Full" conventional sedimentation
Ozone	Ozone deferred to future phase[a]	Ozone included in initial construction[a]
Filtration Rate	Increased filtration rate based on pilot study, assumed 8 gpm/sf	Standard filtration rate of 6 gpm/sf
Clearwell Volume	10 MG	16 MG
Facility Capacity	145 mgd	160 mgd

a. Alternatives with deferred ozone implementation were assumed to include PAC systems; alternatives with immediate ozone implementation do not include PAC systems.

9.1.2 Filtration Facility Alternatives Evaluation Summary

Key considerations for project definition include:

- A filtration rate of 6 gpm/sf is assumed for project definition; higher rates are to be considered during design based on review of pilot plant study findings.
- Clearwell sizing of 16 MG is assumed for project definition; the clearwell size will be optimized during design.
- TAC input validated inclusion of conventional sedimentation basins and ozone in the treatment process to enhance supply resilience and provide multiple water quality benefits.
- TAC input allowed for phasing of ozone but noted interim use of PAC prior to installing ozone will require additional solids handling capacity.
- TAC input noted the sizing difference between hybrid and full-sized sedimentation basins is negligible after accounting for reduced performance from colder winter water temperatures.

9.2 Pipeline Alternatives Evaluation

The consideration of pipeline alternatives described in this chapter focuses on evaluating raw and finished water alignments relative to two considerations.

- **Timing:** considering building two raw and two finished water alternatives either initially to improve reliability, or in phases.
- **Length:** considering the length of new pipelines to enable retiring nearby segments of existing, aging infrastructure and help improve overall system resilience.

The evaluation process for pipelines is further described in Chapter 8: Pipeline Alternatives.

9.2.1 Pipeline Alternatives Evaluation Summary

For project definition, two preferred raw water pipeline alignments and two preferred finished water pipeline alignments were advanced for further evaluation. PWB expects to determine final pipe route locations and potential property and easement needs in 2021 once the pipeline designer is on board.

9.3 Project Options

With multiple feasible treatment alternatives available, the next step was to combine the alternatives into project options and seek input beyond the program team. A cost prioritization workshop was conducted with the program team in July 2019 to identify components that could most easily be deferred. Based on that workshop, three project options were created:

- **Full Implementation.** This option best represents the project values by incorporating key investments in water quality and resilience.
- **Phased Implementation**. This option phases full implementation to reduce initial investment.
- **Minimum Compliance**. This option represents the minimum level of investment to achieve regulatory compliance.

PWB staff presented the three project options to Portland City Council at a work session on September 19, 2019. This initial presentation was followed by presentations to the Portland Utility Board and Citizens' Utility Board. The goal was to receive additional feedback on which project option best aligned with project values. The three options are described in more detail in the following sections. Note the third phase of the pipeline alternatives evaluation was completed after Council provided input on the project options.

9.3.1 Full Implementation Option

The Full Implementation project option is to build a filtration facility that will address the greatest number of risks to supply and water quality, gives customers a higher level of public health protection, and replaces more aging infrastructure with two pipes constructed to survive an earthquake (Figure 9-1).

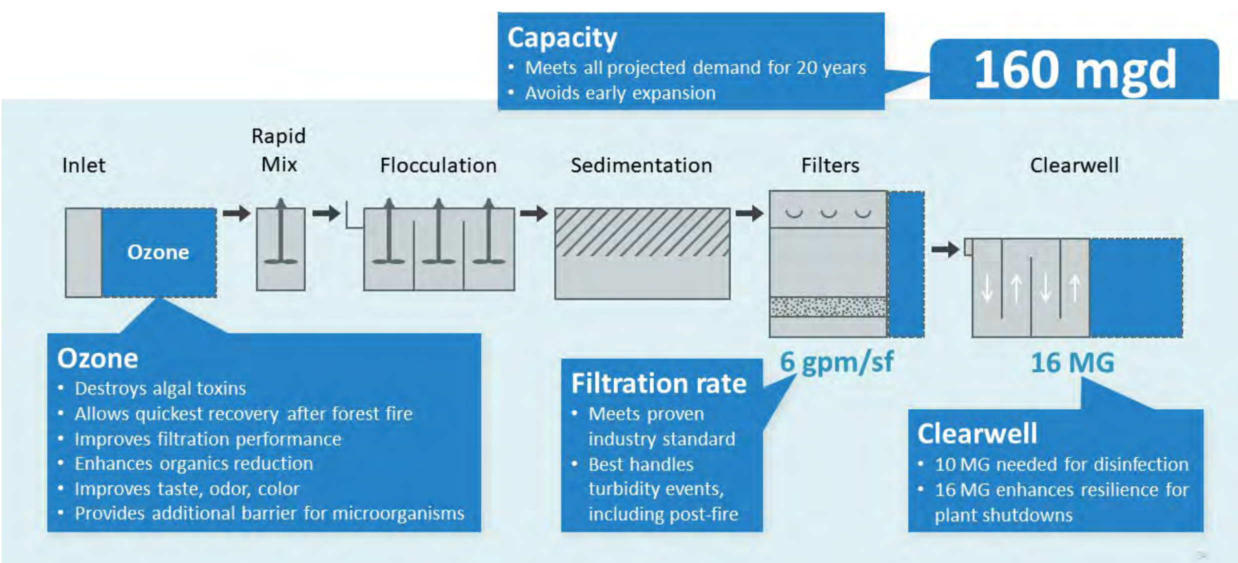


Figure 9-1. Full Implementation project option

9.3.2 Phased Implementation Option

The Phased Implementation project option is to build a filtration facility that addresses more water quality risks but will cost more in the long run for upgrades to meet projected future demand and to construct a second finished water pipe for reliability (Figure 9-2).

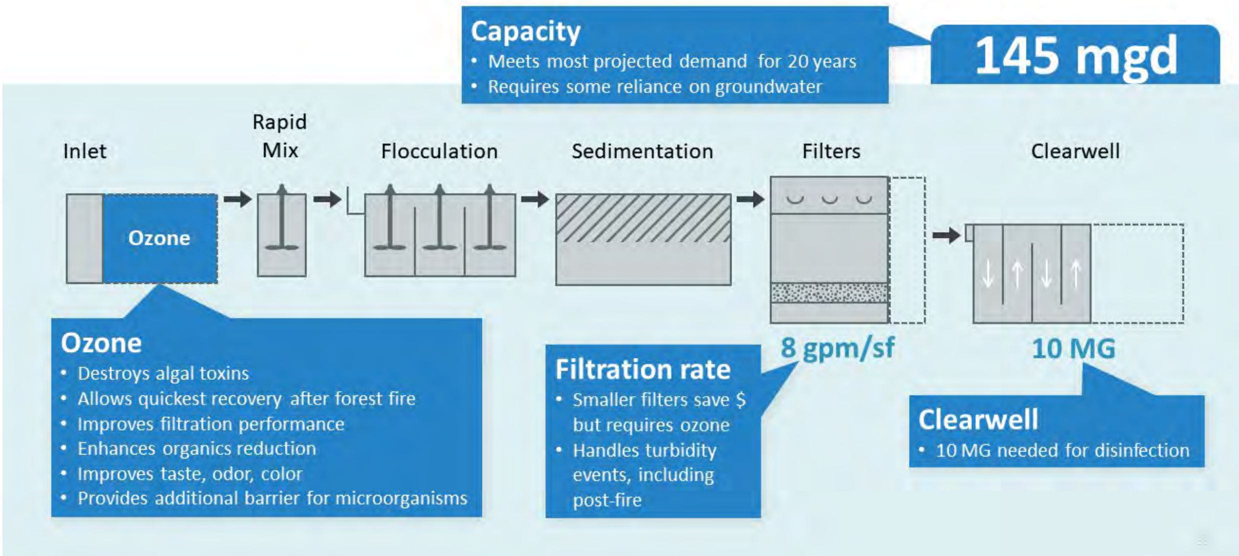


Figure 9-2. Phased Implementation project option

9.3.3 Minimum Compliance Option

The Minimum Compliance project option is to build the minimum treatment to remove *Cryptosporidium* and other microorganisms while reducing turbidity, but would require future investments to address water quality and other risks and would delay construction of a second raw and finished water pipe that would improve reliability (Figure 9-3).

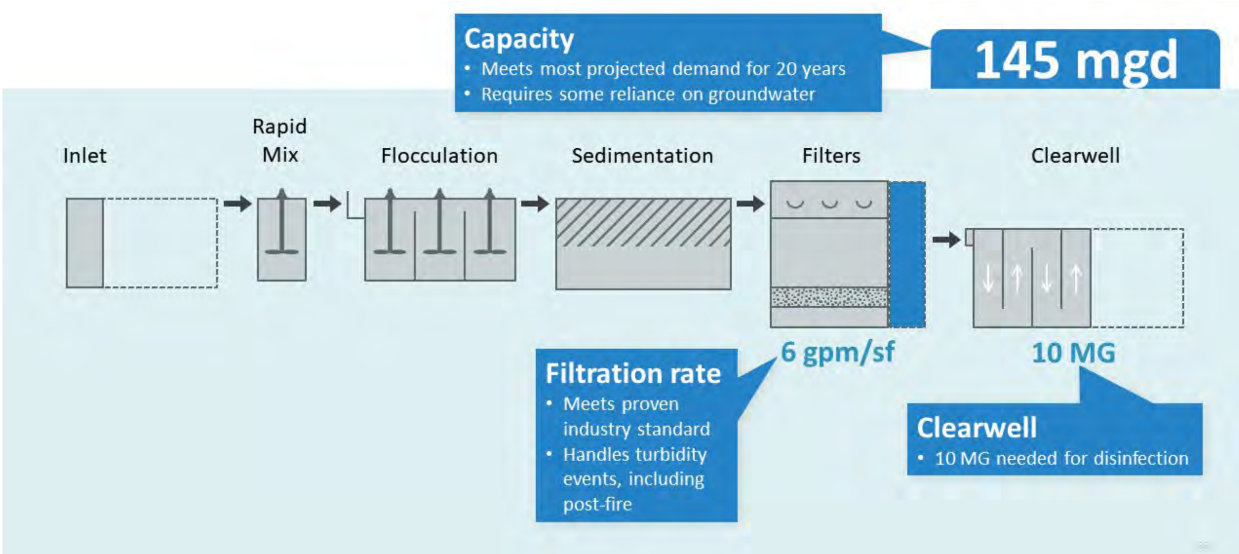


Figure 9-3. Minimum Compliance project option

The specific components of the three project options are summarized in Figure 9-4 below. The options primarily differ by construction phasing. The Full Implementation project option would be completed in one phase. This option produces the highest quality water and best protects against risks such as algal toxins and wildfires, and potential customer considerations (e.g., perceived taste and odor changes) that may be encountered during facility startup. The Full Implementation project option also replaces more nearby pipe at risk of failure due to poor condition or earthquakes.

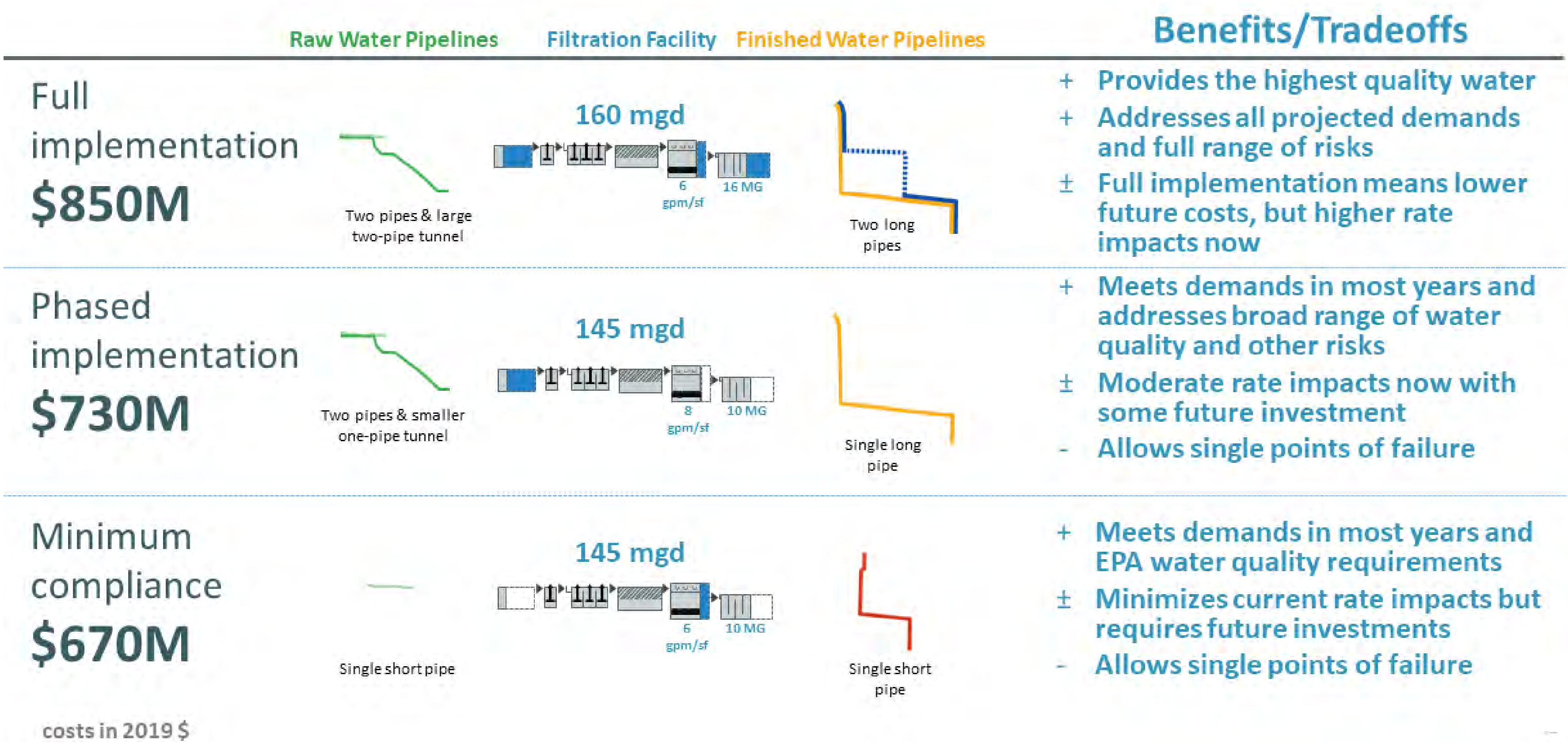


Figure 9-4. Summary of three project options highlighting benefits and trade-offs

The comparison breakdown of initial cost estimates for each project option as presented to City Council in September 2019 is shown in Table 9-4. The components include feasible alternatives for the filtration facility and pipelines identified in Chapter 6 and Chapter 8. Presented costs are total project costs, including planning, design, construction, program management, and PWB staff time.

Table 9-4. Construction Costs for Project Options (September 2019)[a]

Cost Component	2017	2019 Full Implementation	2019 Phased Implementation	2019 Minimum Compliance
Filtration Facility				
Construction		$500	$450	$435
City Services		$19	$19	$19
Program Management and Support Services		$68	$68	$68
Design Consultant		$56	$51	$51
Subtotal	**$500**	**$643**	**$588**	**$573**
Pipelines				
Construction		$178	$120	$71
City Services/Design Consultant		$20	$20	$20
Subtotal		**$198**	**$140**	**$91**
Total[b]		**$850**	**$730**	**$670**

a. Costs shown are in millions.

b. Total costs are rounded up to next highest $10 million. Accuracy range is -30% to +50%.

9.3.4 Recommended Option

Following the September 2019 City Council work session, PWB staff received feedback from the Council, the Portland Utility Board, the Citizens' Utility Board, and the public, and tailored a Recommended project option that reduces overall project cost by optimizing the facility capacity, filtration rate, and storage capacity (Figure 9-5).

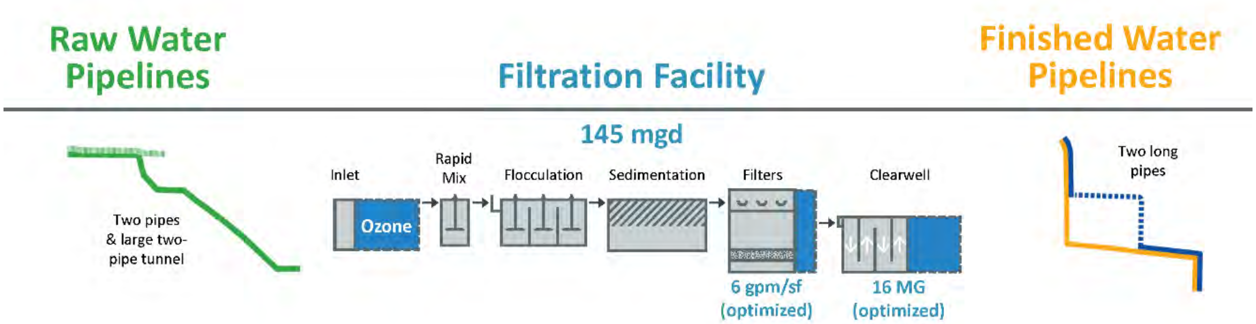


Figure 9-5. Recommended project option

The Recommended project option provides the following benefits:

- Reduces cost by reducing facility capacity to 145 mgd
- Includes ozone to improve filtration resilience to fires, turbidity, algal toxins, and other water quality events
- Replaces more aging infrastructure with longer new finished water pipes
- Provides adequate gravity flow through the facility
- Enhances resilience and operational flexibility with two finished water and raw water pipes
- Reduces project cost and complexity by completing the work at one time
- Reduces impacts to the neighborhood by completing work at one time

9.3.5 Project Options Summary

This section described project options that represent a range of potential investments to provide greater water quality protections and benefits to Portland customers. These project options were presented to City Council in a September 2019 work session. Subsequently, based on input from Council and other stakeholders, PWB developed a Recommended project option that was adopted by Council in November 2019.

Key considerations for project definition include:

- After consideration of a range of potential investments, Portland City Council adopted the Recommended project option on November 27, 2019.
- The Recommended project option is very similar to the Full Implementation project option, but reduces overall project cost by optimizing the facility capacity, filtration rate, and storage capacity.
- The Recommended project option includes ozone in the treatment process to produce the highest quality water and best protect against risks such as algal toxins and fire.
- The Recommended project option assumes two finished water and raw water pipelines and includes longer finished water pipelines to replace nearby segments of aging pipeline.

9.4 Alternatives Evaluation Summary

This chapter described the treatment process and pipeline alternatives that were packaged into project options to identify a Recommended project option that will provide the greatest water quality protection and benefits to Portland customers.

Key considerations for project definition include:

- TAC input validated inclusion of conventional sedimentation basins and ozone in the treatment process to enhance supply resilience and provide multiple water quality benefits.
- For project definition, two raw water and two finished water pipelines are assumed. Longer finished water pipelines will allow for replacement of more aging infrastructure.
- After consideration of a range of potential investments, Portland City Council adopted the Recommended project option on November 27, 2019.
- The Recommended project option reduces overall project cost by optimizing the facility capacity, filtration rate, and storage capacity.

Chapter 10

Selected Option

Chapter 6: Treatment Process Alternatives and Chapter 8: Pipeline Alternatives described a series of viable alternatives for the filtration facility and associated pipelines. Chapter 9: Alternatives Evaluation described how the feasible alternatives were packaged into project options representing a range of potential investments. Chapter 9 also described the development of a recommended project option that Portland Water Bureau (PWB) staff presented to the Portland City Council on November 13, 2019.

This chapter describes the recommended project option that was selected by City Council and adopted in a November 27, 2019, Resolution 37460. The selected project option provides the greatest water quality protection and benefits to Portland customers, while reducing overall project cost by optimizing the facility capacity, filtration rate, and storage capacity. The selected option also right-sizes dual raw water and finished water pipelines to meet capacity needs and improve overall system reliability.

This chapter includes the following sections:

- 10.1 Selected Option Description
- 10.2 Implementation
- 10.3 Selected Option Summary

10.1 Selected Option Description

This section describes the selected project option by providing a basic process flow schematic, design criteria, hydraulic profile, conceptual site layout, and summary of key features used to develop a planning-level cost estimate for the filtration facility and associated pipelines.

The selected project option illustrated in Figure 10-1 includes:

- Target filtration facility capacity of 145 million gallons per day (mgd).
- Conventional treatment including rapid mix, flocculation, sedimentation, and filtration to best handle turbidity events.
- Ozone for enhanced water quality and resilience.
- Clearwell (onsite water storage) sized for disinfection and operational flexibility.
- Two pipelines to and from the filtration facility to maximize gravity flow, reduce future impacts to the filtration facility neighborhood, and provide a more resilient, easier-to-maintain system.

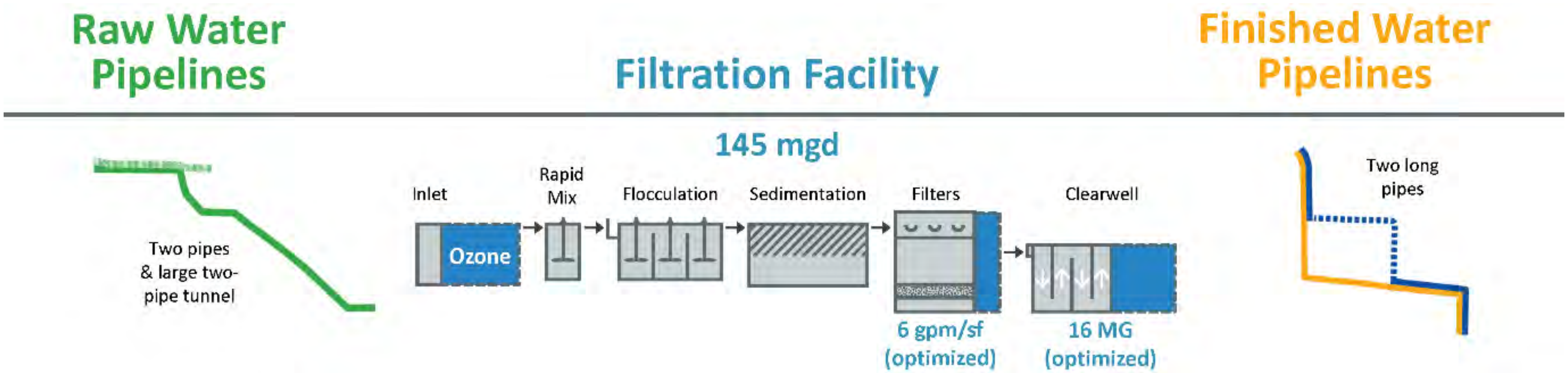


Figure 10-1. Selected project option for the filtration facility and associated pipelines

10.1.1 Filtration Facility

This section provides information describing the selected project option. The level of detail, while general in nature, provides information to understand the nature of the option and to develop a planning-level cost estimate and an associated implementation plan. Key features of the selected option include the following elements:

- Process flow schematic
- Design criteria
- Hydraulic profile
- Site layout
- Support buildings and auxiliary systems
- Design schedule

Process Flow Schematic

A conceptual process schematic for the selected option is shown in Figure 10-2. This schematic includes the features identified in Figure 10-1 and provides additional context relative to connections between these and other associated processes such as solids handling. Many of the conceptual processes associated with solids handling, for example, are assumed for project definition to generate an initial cost estimate.

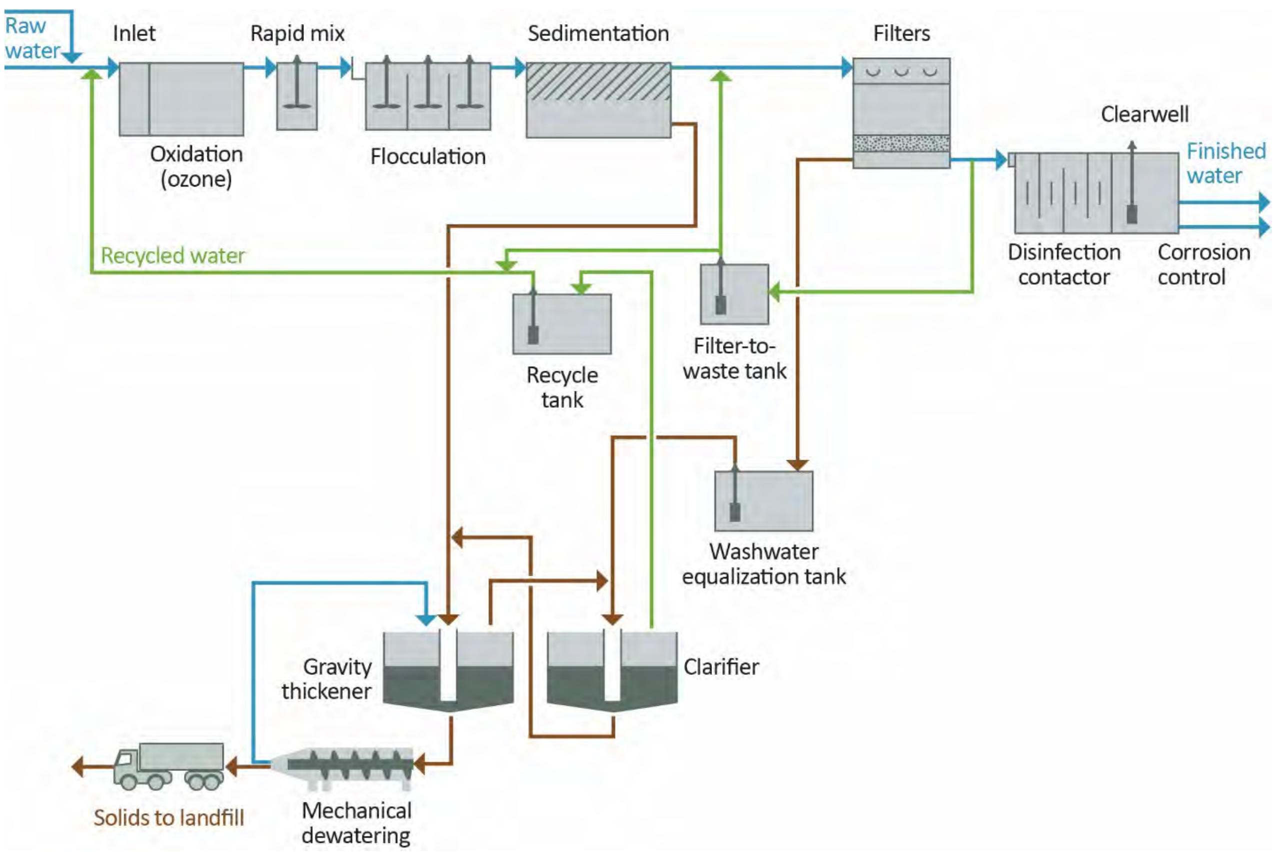


Figure 10-2. Conceptual process flow schematic for the selected project option

Note this figure is very similar to Figure 6-22 with the exception of showing additional detail for the solids separation process.

The process flow schematic identifies multiple flow routing options. For instance, the schematic indicates that the oxidation contactor can be used for pre-oxidation ahead of rapid mix or intermediate oxidation ahead of filtration. For now, this dual process flexibility is maintained to permit the highest degree of treatment flexibility, pending definitive resolution during the pilot study. It is assumed the final configuration through the ozone contactor will be determined during design.

The schematic also shows that filter-to-waste water can be routed to either the head of the facility or directly to the filters if water quality is favorable for direct recycle. Finally, the schematic shows the recycle tank is intended to equalize recycled water flows to the head of the facility per the Filter Backwash Recycling Rule, 40 Code of Federal Regulations 141.76, and Oregon Administrative Rule 333-061-0032-10. Also note that the process flow schematic suggests this will be a zero-liquid discharge facility, with liquid waste streams and process drains assumed to be recycled to appropriate locations within the filtration facility, as discussed in Chapter 6.

Design Criteria

A summary of conceptual design criteria for unit process sizing and cost estimating is provided in the *Basis of Estimate Report* (Appendix B). The design criteria illustrate two scenarios 1) initial design capacity for peak day finished water flow, and 2) ultimate capacity for peak day water flow. The unit process elements described in the design criteria are consistent with the process flow schematic shown above. Generally, the criteria are intentionally conservative for the development of planning-level cost estimates. Additionally, redundant units are included so that no part of the filtration facility is out of service during equipment outages.

Hydraulic Profile

Figure 10-3 illustrates the preliminary hydraulic profile of the selected project option. Flows entering and leaving the filtration facility will be by gravity—no pumping of raw or finished water is anticipated. For this profile, pre-ozonation upstream of the rapid mix facilities is assumed. A fixed weir elevation suitable for controlling water surface elevation entering the post-disinfection contactor is assumed to discourage negative head conditions within the filter media during periods of high unit filtration rates and filter bed head loss.

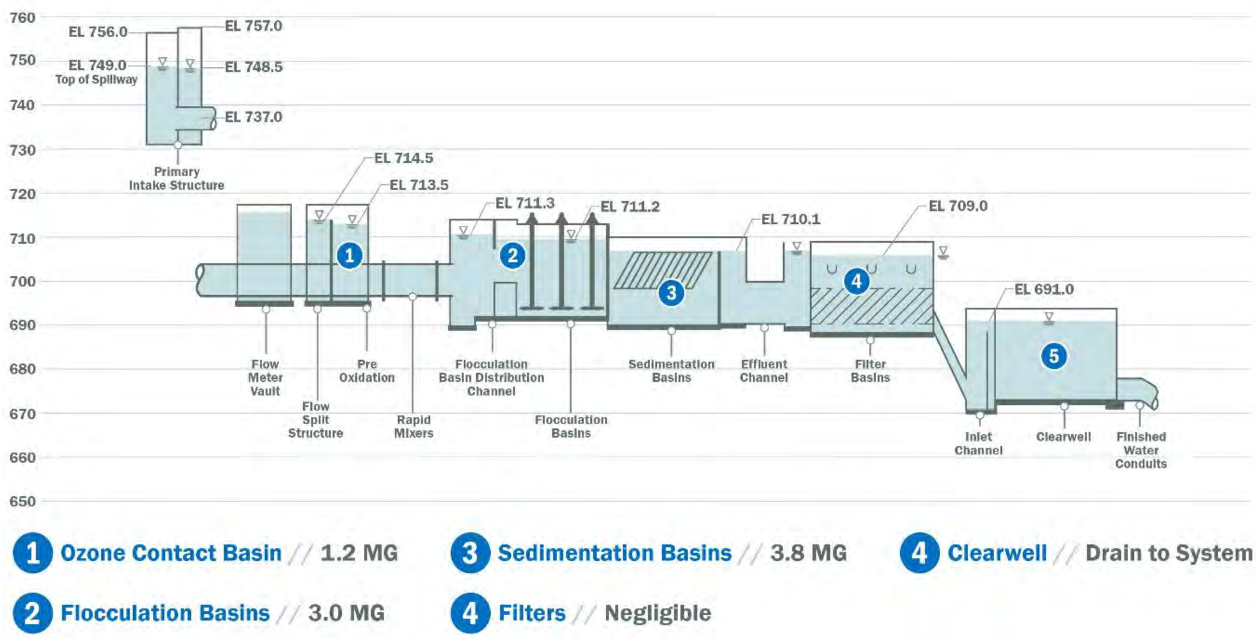


Figure 10-3. Conceptual hydraulic profile for the selected project option

Site Layout

Based on the process flow schematic and the associated design criteria, a general overall conceptual filtration facility layout for the selected project option is shown in Figure 10-4 below.

Note that this conceptual site layout depicts facilities for the 145 mgd filtration facility design capacity along with the additional facilities needed for the facility to be expanded by 50 percent to an ultimate capacity.

For project definition, as described in Chapter 4: Planning Considerations, the overall facility layout is assumed and arranged to follow the existing northeast to southwest topographical "fall line" where water flows off the site by gravity.

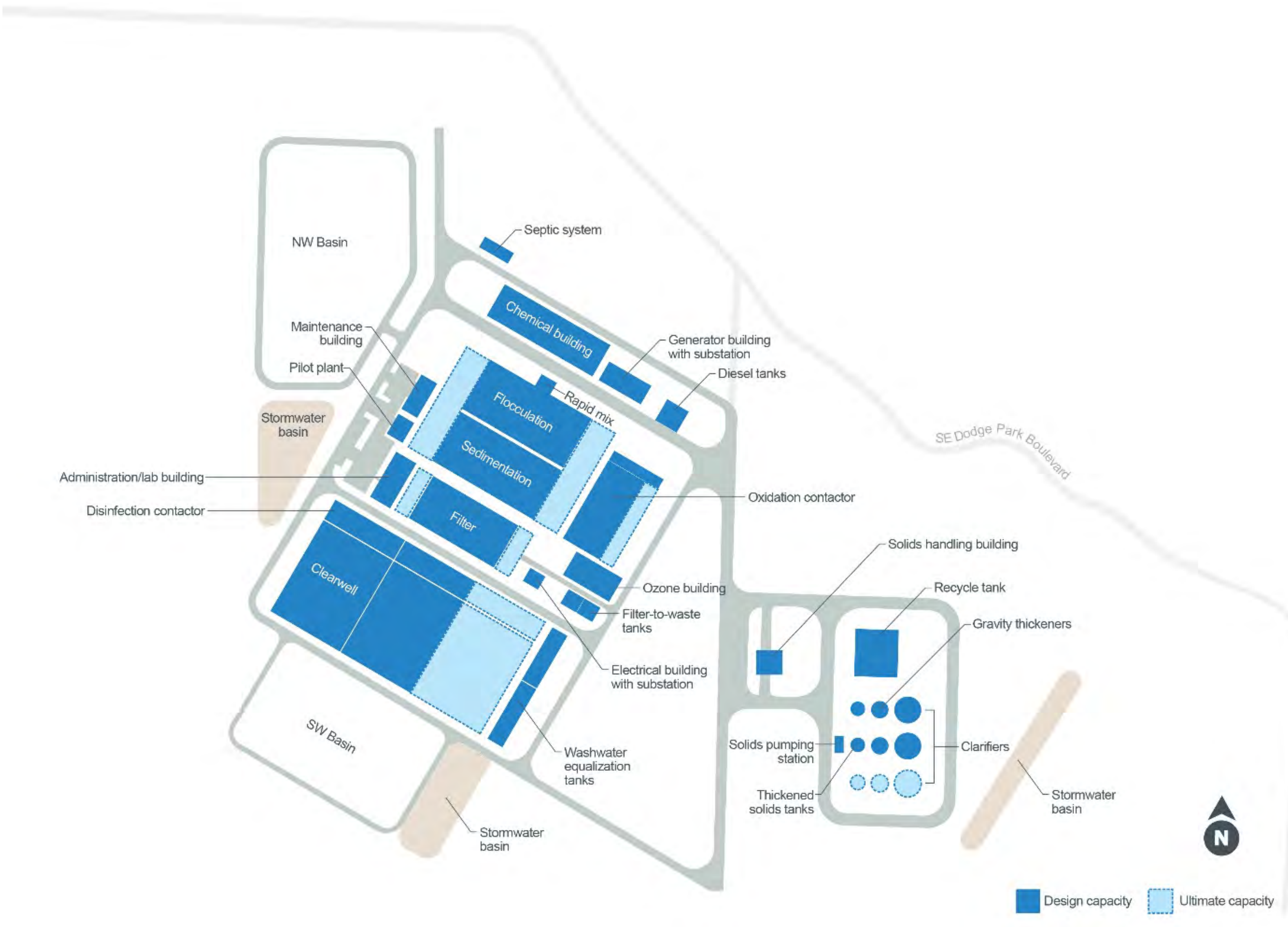


Figure 10-4. Conceptual filtration facility site layout for selected project option

The conceptual site layout shows raw water is conveyed to the facility from the southeast portion of the site to the oxidation contactor, followed by the remaining unit processes identified in the process flow schematic. The site layout illustrates the following:

- Liquid streams located sequentially along the assumed topographic fall-line.
- An open channel distribution system for process flows (sized for the ultimate capacity).
- A tunnel system between the flocculation and sedimentation basins connecting the filter gallery with the rapid mix system. This tunnel system provides a looped route (coupled with topside liquid stream walkways) for operators to traverse the facilities underground, while also providing an accessible corridor for chemical feed piping and other facility utilities.
- Double stacked channels for the inlet and outlet of the ozone contactor for flow-through or bypass modes of operation.
- An oxidation contactor configured to function in a pre- or intermediate oxidation mode. It is assumed that the hydraulic impacts of this arrangement will be evaluated in more detail during design and a singular flow through configuration (pre-oxidation) may be selected.
- Finished water conveyed to the western edge of the site and then routed northwards.
- Solids handling systems located on higher ground east of the main filtration facility.

Support Buildings and Auxiliary Systems

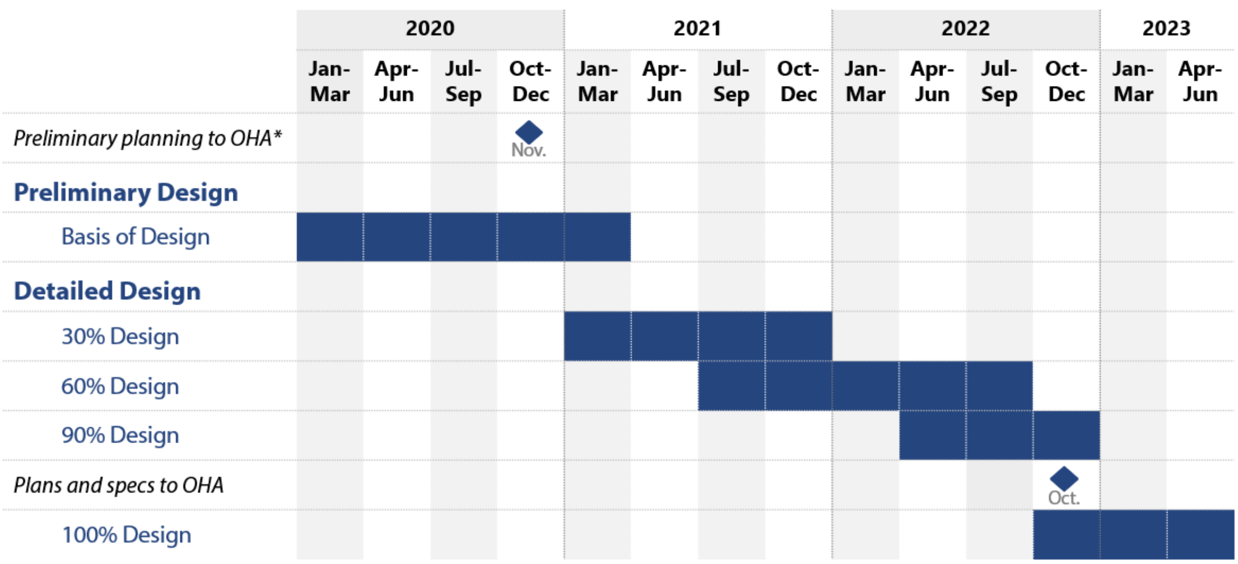
Chapter 7: Filtration Facility Support Systems provides an overview of the major support systems assumed for facility operations. A brief summary of these structures follows:

- **Administration Building.** Provides the main administrative, control, and laboratory functions of the filtration facility. The lower floor of this building provides direct access to the filter gallery, and the upper floor provides direct access to the top of the filters. The Control Center is situated on the upper floor and permits a view of treatment processes.
- **Maintenance Building.** Houses maintenance staff, parts inventory, and workshops to support maintenance and repair activities throughout the filtration facility.
- **Chemical Feed Building.** Houses chemical storage and feed systems with the exception of oxygen storage and ozone generation equipment. Incompatible chemicals are stored in separate rooms and full-scale liquid spill containment is provided for each chemical. The south side of the building has multiple chemical delivery truck unloading systems to fill each storage vessel. A tunnel extension or utilidor concept is suggested to convey process chemicals from the Chemical Feed Building to the main liquid stream tunnel system.
- **Ozone Building.** Liquid oxygen tanks and evaporators are located outside the building with adjacent truck access for unloading. The building houses the ozone generators, nitrogen boost system, and ozone generator cooling systems. Ozone solution is transferred to the ozone contactors for side stream injection.
- **Solids Handling Building.** Houses polymer feed equipment on the first floor with dewatering devices located on the second floor. Drive-through truck access on the first floor permits direct gravity truck loading of dewatered cake.

- **Main Electrical/Generator Building.** Consists of two principal rooms. One room for two 1.75 megawatt diesel generators and associated day tanks. One room for the main facility switchgear distributing 480 volts power to the facility. The bulk fuel storage is located outside the building.
- **Electrical Substation.** Outdoor facility that receives primary power from Portland General Electric and transforms it to 480 volts. Equipment is expected to be rated for 15 kV service.
- **Overflow Basins.** NW Basin to receive overflows from the majority of the liquid stream processes. SW Basin to receive overflows from the clearwell. These two basins are to be lined to prevent groundwater exchange, with overflows recycled to the head of the facility for treatment. During periods when overflows are not present, stored rainfall could be pumped to the stormwater basins for treatment.
- **Stormwater Detention Basins.** Stormwater treatment consists of bioswales followed by equalization storage in unlined basins prior to controlled release to the environment.

Design Schedule

The schedule identifies milestones for the filtration facility design completion (Figure 10-5).



*Oregon Health Authority (OHA)

Figure 10-5. Design schedule showing key milestones for detailed design and regulatory compliance

10.1.2 Pipelines

The selected project option confirmed the project will include two pipelines to and from the filtration facility to maximize gravity flow (to achieve at least 145 mgd), reduce future impacts to the filtration facility neighborhood, and provide a more resilient, easier-to-maintain system.

This section describes the following content:

- Preferred Pipeline Alternatives
- Hydraulic Profile and Pipe Sizing

Preferred Pipeline Alternatives

The preferred pipeline alternatives are RW Alts 1 and 1A and FW Alts 3 and 5 (Figure 10-6).

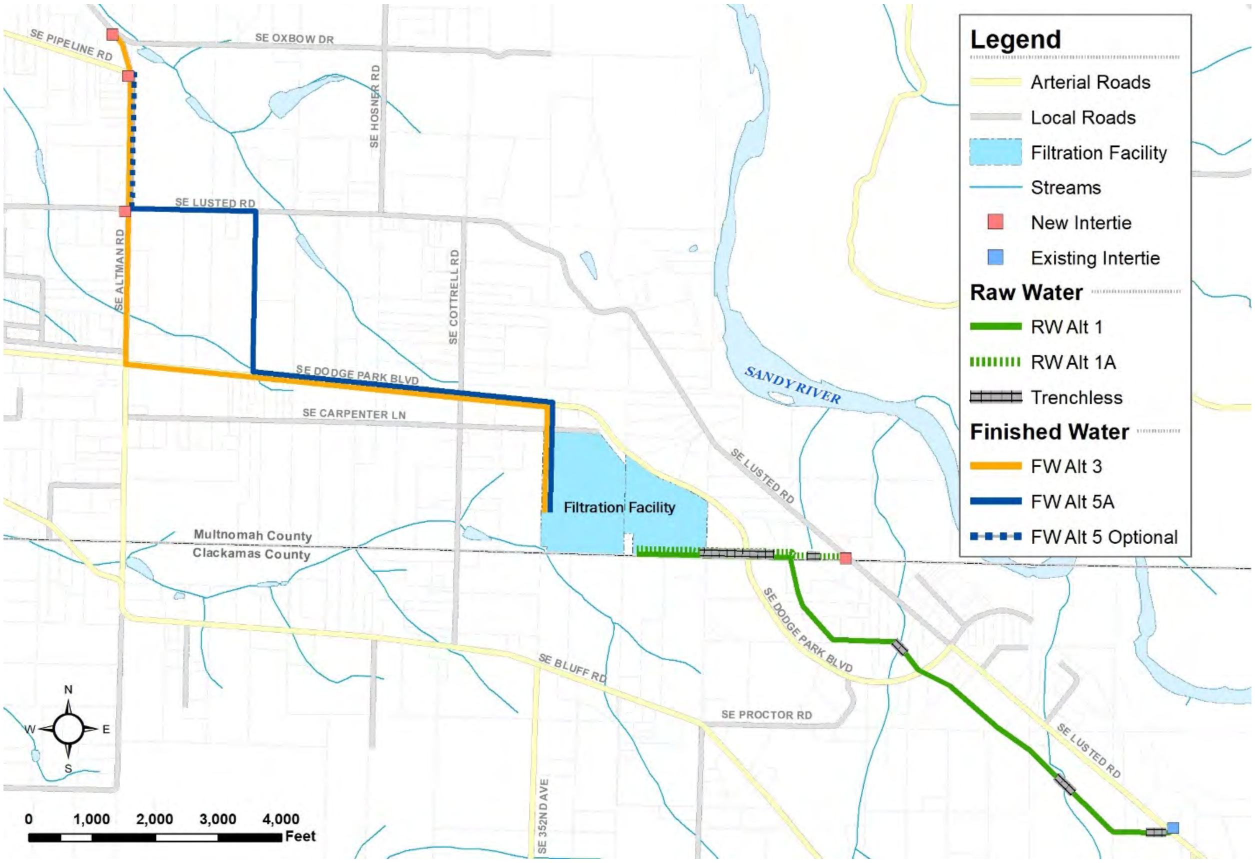


Figure 10-6. Preferred alternatives for raw water and finished water pipelines

Hydraulic Profile and Pipe Sizing

To meet the hydraulic constraints and capacity goals at the filtration facility site while maintaining gravity flow, the raw water pipelines must remain below elevation 715 feet (top of pipe), and the finished water pipelines must remain below the minimum operating level of the clearwell. These hydraulic constraints will result in deep open cut excavations on the filtration facility site and at the beginning of the finished water pipelines east of SE Cottrell Road.

The two raw water pipelines are assumed to each be 72 inches in diameter and the two finished water pipelines 66 inches in diameter. The raw water pipe size must be larger due to the required capacity and the available hydraulic head. Pipe sizes will be confirmed after model calibration in late 2020 and finalized in detailed design.

10.1.3 Cost Estimate

This section presents a construction cost estimate for the filtration facility and associated pipelines. A cost estimate for annual operation and maintenance costs is also provided. A description of cost estimating procedures, assumptions, and the detailed cost estimates are included in the *Basis of Estimate Report* (Appendix B).

Construction Cost Estimate

The estimated capital construction costs for key elements of the selected project option are summarized in Table 10-1. This table presents an un-escalated construction cost estimate based on November 2019 dollars.

Table 10-1. Selected Option Construction Cost Estimate Summary (Class 4 Estimate[a])

Component	Cost (millions)
Filtration Facility (145 mgd peak day capacity)	
Inlet Structure	$4.7
Pre-ozonation	$30.5
Rapid Mix	$1.7
Flocculation	$15.4
Sedimentation	$31.6
Filtration (6 gpm/sf criterion with GAC media)	$71.6
Disinfection Contactor/Clearwells (16 MG)	$47.4
Filter-to-waste Tanks	$2.5
Washwater Equalization Tanks	$6.1
Solids System	$27.1
Chemical Feed	$23.6
Administration/Laboratory Building	$19.3
Maintenance Building	$4.3
Sitework	$75.2
Facility-wide Electrical and Instrumentation	$66.4
Solar Power	$0.7
Major Yard Piping	$16.9
Subtotal Filtration Facility	**$445.0**
Pipelines	
Raw Water Pipelines	$106.5
Finished Water Pipelines	$89.9
Subtotal Pipelines	**$196.4**
Total Construction Cost	**$641.4**

a. *Costs are shown in November 2019 dollars with an accuracy range of -30 to +50 percent based on the AACE International Recommended Practice No. 18R-97.*

Annual Operating Cost Estimate

The estimated annualized operating costs for the selected project option are listed in Table 10-2. The estimates account for annual average flows by year and identified anticipated operations and maintenance costs such as periodic GAC media and equipment replacement. Note that for project definition, the use of GAC media is assumed to provide additional cost conservatism over the use of anthracite media.

Table 10-2. Estimated Selected Option Annualized Operating Costs

Component	Cost[a] (million/year)
Filtration Facility	
Labor	$3.08
Electricity	$0.55
Chemicals	$3.62
Equipment Repair and Maintenance Equipment Replacement and Overhaul	$3.36
Solids disposal	$0.198
Other	$1.89
Subtotal Filtration Facility	**$12.7**
Raw and Finished Water Pipelines[b]	$0.8
Total Annual Operating Cost	**$13.5**

a. November 2019 dollars. Assumes project start of 2027 and a uniform annualized cost series with an annual 6 percent interest rate and 3 percent escalation rate. Costs run to the year 2045.

b. Repair and maintenance only.

10.1.4 Selected Option Description Summary

This section described high-level elements of the selected project option and summarized the associated planning-level costs.

- **Filtration Facility.** The process flow schematic, design criteria, hydraulic profile, conceptual site layout, and summary of support buildings and auxiliary systems are provided in sufficient detail to develop a planning-level cost estimate.
  - Process schematic assumes the facility is zero-liquid discharge and allows flexibility for either pre- or intermediate oxidation.
  - Design criteria and the conceptual site layout include scenarios for the initial design capacity and the ultimate capacity. Design criteria are generally conservative with redundant units provided for reliability.
  - Hydraulic profile assumes flows to and from the facility will be by gravity.
- **Pipelines.** Two raw water and two finished water pipelines are assumed for project definition.
- **Cost Estimate.** The construction cost estimate and annualized operating cost estimate are presented in 2019 dollars.

10.2 Implementation

This section provides a high-level description of implementation for construction of the selected project option.

This section includes discussion of the following topics:

- Delivery Method
- Construction Schedule
- Future Expandability
- Implementation Summary

10.2.1 Delivery Method

Early in the design phase, PWB will select a firm to provide CM/GC services. The CM/GC delivery method combines the scope of work of a general contractor with that of a construction manager under a single contract. The CM/GC contract includes a pre-construction services phase during the design process followed by the construction services phase. The pre-construction services phase will provide:

- Constructability input into the design
- Construction subcontract packaging recommendations
- Development of a Guaranteed Maximum Price (GMP)

The CM/GC will collaborate with PWB and the designer on design development and preparation of construction documents. As design progresses, the CM/GC will initiate pricing the work and assembling a GMP. Once the design has progressed to a level suitable for construction estimating, the CM/GC will submit the project GMP to PWB. After agreement is reached on a GMP, the construction phase will be authorized, and the CM/GC will begin construction and eventual commissioning of the filtration facility. Should PWB be unable to come to terms on the GMP, there remains an option to complete the design and publicly advertise the construction contract as a traditional design-bid-build delivery.

During the construction services phase, the CM/GC will procure subcontracts with trade contractors using multiple bid packages, typically on a low bid approach, to construct the project. The CM/GC will manage the construction contract and may be allowed to self-perform portions of the trade work. Because the CM/GC has the flexibility to package subcontracted work during the pre-construction phase with PWB input, local and Disadvantaged, Minority-owned, Women-owned, Emerging Small Business, and Service Disabled Veteran Enterprise participation can be maximized by focusing subcontractor procurement in those areas to the local market.

A major benefit of CM/GC is the collaboration that can result between PWB, the designer, and the contractor. The contractor becomes, in effect, a member of the project team during design, providing constructability and value-engineering input, thereby improving the quality of the design, potentially reducing the cost of construction, and affording the contractor buy-in to the design documents.

10.2.2 Schedule

Construction of the filtration facility is expected to take 4 to 5 years. Construction of the pipelines is expected to take about 3 years. Construction for both projects is anticipated to begin in 2023 and is required to be substantially complete by September 30, 2027 (Figure 10-7). Details about construction activities and schedules will be developed after designs are complete and contractors are selected.

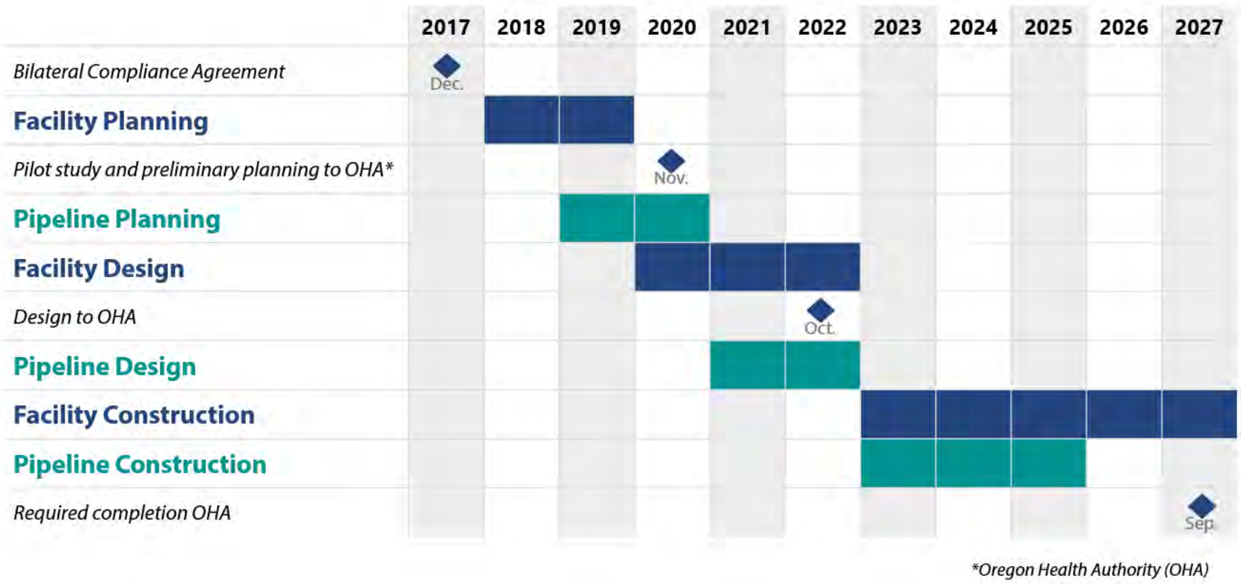


Figure 10-7. Construction is anticipated to begin in 2023 and be substantially complete by September 30, 2027

10.2.3 Future Expandability

As described in Chapter 4: Planning Considerations, the planned ultimate capacity is potential future expansion up to 50 percent of the design capacity. It is suggested that impacts of potential future facility expansion be evaluated during initial design, especially in regard to major unit processes and key support systems such as standby engine generators. Although certain parts of the facility, such as the Administration Building and Maintenance Building, may not need expansion to accommodate potential capacity increases, is important that the design of unit process facilities include sufficient parallel units to promote flexibility and allow for sensible expansion. In addition, pipelines should be designed to perform with higher working pressures and be compatible with future upstream improvements in the Bull Run Watershed and at Headworks (e.g., connection to Dam 2 head). Planned flexibility in the initial design capacity will need to be balanced with consideration of cost, final facility layout complexity, space set aside for expansion, and potential operational impacts.

10.2.4 Implementation Summary

This section outlined key elements of the selected project implementation to meet the compliance deadline of September 30, 2027. These key elements include:

- The filtration facility and associated pipelines will use the CM/GC delivery method. CM/GC contractors will be selected for each project to provide pre-construction services during the design process followed by construction services.
- Construction of the filtration facility is expected to take 4 to 5 years, including time for commissioning and startup. Construction of the pipelines is expected to take about 3 years.
- Construction of the filtration facility and pipelines is anticipated to begin in 2023 and is required to be substantially complete by September 30, 2027.
- The filtration facility design capacity is 145 mgd. The design should also accommodate flexibility for potential future expansion to the ultimate capacity, especially in regard to treatment process facilities.

10.3 Selected Option Summary

This chapter described the selected project option adopted by City Council in Resolution 37460 on November 27, 2019, and the anticipated schedule for project design and construction to substantially complete the filtration facility by the required deadline of September 30, 2027.

- **Selected Option.** Key considerations for project definition include:
  - To meet projected peak day demands, the target filtration facility capacity is 145 mgd.
  - The treatment process will be conventional treatment, including rapid mix, flocculation, sedimentation, and filtration.
  - Ozone will be included in the treatment process.
  - The clearwell will be sized for disinfection and operational flexibility.
  - There will be two pipelines into and out of the filtration facility.
- **Implementation Plan.** Key considerations for project definition include:
  - PWB will select a firm to provide CM/GC services, including pre-construction and construction services. Construction is required to be substantially complete by September 30, 2027.
  - Facility design should consider planned flexibility for potential future expansion up to 50 percent of the design capacity.

Appendices

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PORTLAND WATER BUREAU
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