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City of Longview
Mint Farm Regional Water Treatment Plant
Preliminary Design Report

Part 1
Basis of Design Report

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Prepared for

City of Longview
1525 Broadway Street
Longview, WA 98632

K/J Project No.0997003*00

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Section 1: Introduction

This report is the Basis of Design Report, Part 1 of the Preliminary Design Report for the Mint Farm Regional Water Treatment Plant and Well Field. This report is a deliverable in satisfaction of contractual scope of work requirements and is the first of three parts comprising the full Preliminary Design Report. Part 2 of the Preliminary Design Report will cover Hydrogeology and Water Quality and Part 3 will cover Environmental Permitting and Archaeological Resource Protection.

1.1 Authorization

Kennedy/Jenks Consultants was selected competitively by the City of Longview to perform work on the Design of Groundwater Source and Mint Farm Regional Water Treatment Project (MFRWTP Project). Project work was authorized by contract dated March 20, 2009 issued by the City of Longview. This report is a part of the work of the MFRWTP Project.

1.2 Project Description

The MFRWTP Project is comprised of five major subdivisions of work: Phase A: Project Management and Quality Assurance; Phase B: Preliminary Design Report; Phase C: Hydrogeology; Phase D: Environmental Permitting; and Phase E: Final Design. The overarching project objective is to document the characteristics and suitability of the Mint Farm area deep aquifer to serve as a raw drinking water source in perpetuity for the City of Longview, which would replace the existing Cowlitz River raw water source. In addition, the project objective is to determine the optimum location and treatment technologies for a new regional water treatment plant to treat the deep aquifer groundwater. As such, the project comprises:

- Investigation of groundwater flow and flow characteristics
- Investigation of groundwater quality
- Assessment of environmental risk associated with the site and the use of the groundwater as a drinking water source
- Human Health Risk Assessment of the groundwater source as well as, in parallel, the existing Cowlitz River source and the Columbia River
- Preliminary design of treatment plant and well field facilities, including pilot testing of granular media filtration technologies, and alternative analysis and selection of a preferred groundwater filtration technology
- Assistance with securing a new groundwater rights for the proposed deep aquifer source and preparation of a new Well Head Protection Plan for the new source
- Assistance with environmental permitting, antiquities preservation compliance, Critical Areas ordinance, and other permits and applications related to the design and construction of the new treatment facility and well field
- Final detailed design of selected facilities and preparation of document plans and specifications suitable for public bid and subsequent construction

- All project management, quality assurance and quality control activities to ensure the project work and deliverables are commensurate with the industry standard of care and comply with the contractual requirements and scope of work

1.3 Purpose of Report

As Part 1 of the Preliminary Design Report, this Basis of Design Report shall serve as the guiding document for the final, detailed design process. This report provides discussion of and establishes the water quality and treatment objectives for a new groundwater treatment plant for the City of Longview. This report includes pilot study results for granular media filtration and presents an alternatives analysis selecting the preferred treatment process for filtration. Subsequent chapters of the report discuss other plant treatment processes, well field pumping, distribution system connection and hydraulic analysis, and design elements from all major design disciplines requiring guidance for completion of the detailed design. The report includes a preliminary treatment plant, well field, and distribution system connection cost estimate based on a nominally conceptual, ten percent design. Conceptual, preliminary design drawings have been completed and are included to illustrate and support the discussion and conclusions contained in this report.

1.4 Abbreviations

The following abbreviations are provided for the reader's use in decoding acronyms used in the environmental engineering practice and abbreviations commonly found in engineering reports and drawings.

Abbreviation	Description
ºC	degree Celsius
ºF	degree Fahrenheit
µg/L	microgram per liter
µMhos/cm	micromhos per centimeter
AAA	authentication, authorization, and access
AACE	Association for the Advancement of Cost Engineering
A/C	asphaltic concrete
ACI	American Concrete Institute
ADD	average daily demand
AISI	American Iron and Steel Institute
ANSI	American National Standard Institute
ASCE	American Society of Civil Engineers
ATS	automatic transfer switch
bgs	below ground surface
CAS	Chemical Abstract Service
CCR	Consumer Confidence Report
CDC	Center for Disease Control
CDID #1	Consolidated Diking Improvements District #1
CEC	contaminants of emerging concern
cfm	cubic feet per minute
cfs	cubic feet per second

Abbreviation	Description
City	City of Longview
COPS	Critical Operations Power Systems
Cowlitz PUD	Public Utility District No. 1 of Cowlitz County
dBA	A-weighted decibels
DCOA	Designated Critical Operations Area
DOH	Washington State Department of Health
EPA	United States Environmental Protection Agency
ft	feet
gal	gallon
gm	gram
gpm	gallons per minute
HAA-5	haloacetic acids
HLR	hydraulic loading rate
HMI	human-machine interface
HP	horsepower
HVAC	heating, ventilating, and air conditioning
ICC	International Code Council
ICP	inductively coupled plasma
IFC	International Fire Code
IBC	International Building Code
IMC	International Mechanical Code
in	inches
IO	input/output
IPC	International Plumbing Code
kV	kilovolt
kW	kilowatt
kWh	kilowatt hour
lb	pound (lbs for plural)
m^3	cubic meter
max	maximum
MCC	motor control center
MCL	maximum contaminant level
MDD	maximum daily demand
min	minimum
MG	million gallons
MGD	million gallons per day
mg/L	milligrams per liter
ml	milliliter
mm	millimeter
NA	not applicable
NASPEC	North American Specification
NEC	National Electrical Code

Abbreviation	Description
NFPA	National Fire Protection Association
NIOSH	National Institute for Occupational Safety and Health
No.	number
NSF	National Sanitation Foundation
OADB	outdoor air dry bulb
OAWB	outdoor air wet bulb
ORP	oxygen reduction potential
OSHA	Occupational Safety and Health Administration
PDR	Preliminary Design Report
PLC	programmable logic controller
psi	pounds per square inch
PRV	pressure regulating valve
PW	production well
RWTP	Regional Water Treatment Plant
SBUH	Santa Barbara Urban Hydrograph
SCADA	supervisory control and data acquisition
SCFM	standard cubic feet per minute
SCS	Soil Conservation Service
SEI	Structural Engineering Institute
sf	square feet
SHGC	solar heat gain coefficient
SMCL	secondary maximum contaminant levels (SMCLs for plural)
TDH	total dynamic head
TOC	total organic carbon
TSS	total suspended solids
TTHM	total trihalomethanes
UFC	Uniform Fire Code
UFRV	unit filter run volume
UL	Underwriters Laboratory
UPS	uninterruptible power supply
V	volt
VAV	variable air valve
VFD	variable frequency drive
VOC	volatile organic compound (VOCs for plural)
WAC	Washington Administrative Code
WEP	wire equivalent protection
WTP	water treatment plant

Section 2: Water Quality and Treatment Objectives

In this section, the water quality and treatment objectives are presented, along with the procedure, results, and conclusions from a granular media pilot study performed in late April and May 2009, using Prudential Boulevard Test Well water.

2.1 Water Quality

Available water quality data are presented in Table 2.1 for the Production Well No. 1 (PW1) and the Prudential Boulevard Test Well. The Prudential Boulevard Test Well is of interest because Kennedy/Jenks Consultants performed a pilot study of three alternative granular media filters using water from that well as the raw water source. At the time of the pilot study, the proposed location of the water treatment plant had not been finalized and the Prudential Boulevard Test Well was available for the study. It pumps from the deep aquifer below the Mint Farm.

Table 2.1: Well Water Quality

Parameter		Production Well 1[(a)]	Prudential Boulevard Test Well[(b)]
Target Contaminants			
Iron (mg/L)	Total	0.93	1.2
	Dissolved	NA	< 0.02
Manganese (mg/L)	Total	0.597	0.601
	Dissolved	NA	0.574
Arsenic (μg/L)	Total	5.7	12.7
	Dissolved	NA	11.8
	As-III	6.7	6.54
	As-V	< 2.0	7.64
Other Water Quality			
pH		7.3	7.8
Alkalinity (mg/L as $CaCO_3$)		105	130
Temperature (°C)		11.7	12
Hardness (mg/L as $CaCO_3$)		99	112
Total Dissolved Solids (mg/L)		187	201
Conductivity (μMhos/cm)		240	307
Apparent Color (color units)		25	5
Total organic carbon (mg/L)		1.6	1.8
Dissolved organic carbon (mg/L)		NA	1.6
Total Silica (mg/L)		51.4	25.0
Fluoride (mg/L)		< 0.2	< 0.2
Chloride (mg/L)		8.4	12
Nitrate (mg/L)		< 0.1	0.3
Sulfate (mg/L)		0.45	0.35
Phosphate (mg/L)		0.33	0.55
Ammonia (mg/L)		0.19	NA

Notes:

(a) Based upon water quality samples collected 5 October 2009, except iron, manganese, and arsenic for which the average of four weekly samples collected between 5 October and 21 October 2009, is presented.
(b) Based upon water quality data collected during a pilot study in May 2009.
(c) NA = not available
(d) mg/L = milligrams per liter
(e) μg/L = micrograms per liter
(f) °C = degrees Celsius
(g) CaCO3 = calcium carbonate
(h) μMhos/cm = micromhos per centimeter

Extensive water quality data for PW1 are being collected as part of an extended pump test that ran concurrently with the writing of this report. Data from the baseline testing, performed at the start of the pump test in early October 2009, are included in Table 2.1. The water quality data for PW1 are considered the most representative of the anticipated raw water quality for the treatment plant.

Water quality for PW1 and the Prudential Boulevard Test Well are very similar; although, the levels of iron, manganese, arsenic, and pH are lower in PW1 and color silica are higher in PW1 (Table 2.1).

2.2 Treatment Objectives

The treatment objectives are presented in Table 2.2. The selected treatment goals for iron and manganese are well below the secondary maximum contaminant levels (SMCLs) and are based upon Kennedy/Jenks Consultants' experience. The proposed levels of iron and manganese are anticipated to be more acceptable to consumers by avoiding the staining that these metals can cause. The City of Longview (City) would like to maintain arsenic levels below the Washington State Department of Health (DOH) trigger (5 μg/L) which requires mandatory additional language concerning the level of arsenic in the finished water to be included in the City's Consumer Confidence Report (CCR).

Table 2.2: Proposed Pilot Study Filtered Water Quality Goals

Parameter	Water Quality Goal	MCL	SMCL
Total Iron	≤ 0.05 mg/L	-	0.3 mg/L
Total Manganese	≤ 0.02 mg/L	-	0.05 mg/L
Total Arsenic	≤ 5 μg/L	10 μg/L	-

Notes:

MCL = maximum contaminant level
SMCL = secondary maximum contaminant level

2.3 Pretreatment Assessment

The two objectives of pretreatment for a granular media treatment process designed to remove iron, manganese, and arsenic are: to provide adequate oxidation; and to provide conditions so that arsenic binds to iron to facilitate its removal.

2.3.1 Oxidation

Oxidation is required in order to convert iron and manganese into metal oxides which can be removed by attachment to the granular media. In the case of manganese, complete oxidation to

an insoluble particle occurs after attachment to the granular media. The continuous presence of an oxidant is necessary to complete the oxidation, such that the manganese and iron oxide solids can be removed during backwashing. Oxidation of iron and manganese is readily achieved with free chlorine.

Based upon Kennedy/Jenks Consultants' experience, contact time to facilitate adequate oxidation of iron, manganese, and arsenic can take place in the pipe between the point of oxidant addition and the filter vessels. Therefore, a separate unit process to provide a substantial contact time is not required.

Additional information on the oxidation alternatives is provided in Section 4.3.

2.3.2 Pre-Treatment for Arsenic Removal

The presence of an oxidant is required to convert Arsenic-III to Arsenic-V. Arsenic-V tends to readily form complexes with iron to a much greater extent than Arsenic-III, so this oxidation tends to increase the removal of arsenic in an oxidation/granular media filtration process. This oxidation is readily achieved with free chlorine.

Two key factors for arsenic removal are the iron concentration and the pH of the water. The presence of sufficient iron is required in order to remove arsenic, once oxidized to the Arsenic-V form. A general rule of thumb is that a mass-based ratio of iron to arsenic of 20:1 is required in order to achieve any significant arsenic removal. Arsenic will generally form complexes with iron in the pH range of 5.5 to 8.0; however, arsenic removal is pH sensitive and near complete removal of arsenic has been reported at a pH of less than 7. When the pH is in the range of 7 to 8, higher levels of iron typically increase arsenic removal, in particular as iron to arsenic ratio increase above 20:1.

The iron to arsenic ratio in PW1 and the Prudential Boulevard Test Wells are 163:1 and 100:1, respectively. With these relatively high levels of iron, there appears to be sufficient native iron to achieve significant arsenic removal.

If supplemental iron were considered, ferric chloride is recommended because of its low cost, its higher availability, and its addition to the raw water would tend to suppress pH, which tends to improve arsenic removal efficiency. Because the source of supply is groundwater and the target contaminants for removal are arsenic, iron, and manganese, ferric chloride would not be added as a coagulant as is the case for surface water treatment. The water treatment plant is envisioned for groundwater treatment only, not surface water.

Further discussion on the removal of iron, manganese, and arsenic using oxidation and filtration (including ferric chloride addition) is provided in Section 2.4, Granular Media Pilot Study.

2.4 Granular Media Pilot Study

2.4.1 Pilot Study Protocol

2.4.1.1 Introduction and Purpose

This section presents the protocol used for a pilot study to test the performance of alternative granular media for treating groundwater at the Mint Farm. The three main constituents of concern are iron, manganese, and arsenic. The 5-week pilot study was performed in late April and May 2009. The Prudential Boulevard Test Well at the Mint Farm provided the source water for the pilot equipment.

The purpose of the pilot study was to determine if the selected media could meet the treated water goals for this study (see Section 2.2) and to determine design and operating parameters under which the goals are achieved. The parameters determined in this study were oxidant demand, hydraulic loading rate, filter run time between backwashes, timing of arsenic, iron, and manganese breakthrough during the filter run, and settleability of backwash solids.

A Hazard Appraisal and Recognition Plan (HARP) was completed prior to the pilot study.

2.4.1.2 Preliminary Oxidant Demand Testing

Preliminary oxidant demand tests were performed with chlorine and potassium permanganate. The raw groundwater was dosed with chlorine, and the chlorine residual was measured over a range of times. The chlorine dosage producing a chlorine residual in the range of 0.5 to 1 mg/L after 10 minutes of holding time was selected as the starting point for pilot operations.

In a second test, potassium permanganate was dosed at a concentration sufficient to oxidize the dissolved manganese in the water. The potassium permanganate dosage that produced a slight pink color after 10 minutes of holding time was selected as the starting point for pilot operations. These dosages were modified during operation of the pilot units, as required to maintain a residual in the pilot filter column effluents.

2.4.1.3 Pilot Study Equipment

The pilot filter equipment consisted of skid-mounted filters, each with a diameter of 6 inches. The operator varied the flow rate through the filters, and the selected flow rate was maintained by on-board controls. The filter skids included instrumentation to measure filtered water turbidity and the flow rate through each filter. The skid was designed to operate automatically and, as such, did not require regular attention to maintain normal operations. Chemicals were dosed using chemical feed pumps that fed chemical solutions, prepared at specific concentrations, to provide the required dosages. Operator attention was required to verify that the system was operating properly, to address system alarms, to check and replenish chemicals, and to collect water quality samples. A schematic of the pilot equipment is presented in Figure 2.1 and drawings of the filter skid are provided in Appendix A. The media design for the pilot filters is presented in Table 2.3.

Table 2.3: Pilot Filter Media Design

Parameter	Pilot Filter Greensand	 Greensand Plus	 Silica Sand
Top Media	Anthracite	Anthracite	Anthracite
Depth (inches)	24	24	24
Effective size (mm)	0.95 - 1.05	0.95 - 1.05	0.95 - 1.05
Uniformity coefficient	< 1.4	< 1.4	< 1.4
Specific gravity	1.6	1.6	1.6
Bottom Media	Greensand	Greensand Plus	Silica Sand
Depth (inch)	12	12	12
Effective size (mm)	0.30 - 0.35	0.30 - 0.35	0.52
Uniformity coefficient	< 1.6	< 1.6	< 1.31
Specific gravity	2.4	2.4	2.6
Total L to d ratio	1500	1500	1200

Notes:

(a) mm = millimeters
(b) L to d = ratio of media depth to effective size

Greensand is a media made from the mineral Gluconite. Greensand Plus is a media marketed by Inversand Company that is similar to Greensand but contains a silica sand core with an outer coating of manganese dioxide. Greensand Plus can withstand higher operating pressures than Greensand. Manganese oxide coated silica sand (silica sand) was the third media type tested.

During the first several days of testing, a range of hydraulic loading rates were tested in order to determine a feasible rate for this source water. Hydraulic loading rates for subsequent tests were based upon the initial results. The addition of ferric chloride was tested as a means of improving arsenic removal by lowering the pH and providing additional iron for adsorption of the arsenic, in the last week of the study.

2.4.1.4 Backwash Protocol

Backwashing was performed using a combination of air and water wash. The following sequence was used:

- The filter influent and effluent valves were closed to take the filter offline.
- Air scour was performed at a rate of 2 standard cubic feet per minute per square foot (scfm/sf) with simultaneous water wash at a rate of 6 gallons per minute per square foot (gpm/sf) for 3 to 4 minutes.
- The air valve was gradually closed and the water wash rate was increased to 12 gpm/sf for 7 to 8 minutes to remove the solids from the media.
- No filter-to-waste cycle was used.
- The filter was restarted, after performing any adjustment to operating conditions, and operations monitoring began immediately after the filter was re-started.
- Filter effluent water quality sampling resumed as soon as practical upon restarting the filter.

2.4.1.5 Backwash Water Settling Tests

Samples of the backwash water were collected for settling tests. Settling was performed under quiescent conditions using standard jar testing equipment. The turbidity of the supernatant was measured as a function of time to determine the settling characteristics of the solids.

2.4.1.6 Pilot Study Operations

The pilot equipment vendor provided pilot operation training at the beginning of the study. Kennedy/Jenks Consultants operated the pilot equipment. The pilot operator performed manual backwashes using simultaneous air scour and water wash followed by water only wash to achieve a bed expansion of approximately 40 percent.

2.4.1.7 Raw Water Characterization

The Prudential Boulevard Test Well water quality is described in Section 2.1.

2.4.1.8 Pilot Study Sampling and Analysis

The sampling and analysis plan for the pilot study is presented in Table 2.4. Temperature and pH were measured at the pilot site using standardized probes. Turbidity, alkalinity, iron, and manganese were measured using standard Hach™ test methods.

Figure 2.1: Pilot Equipment Schematic

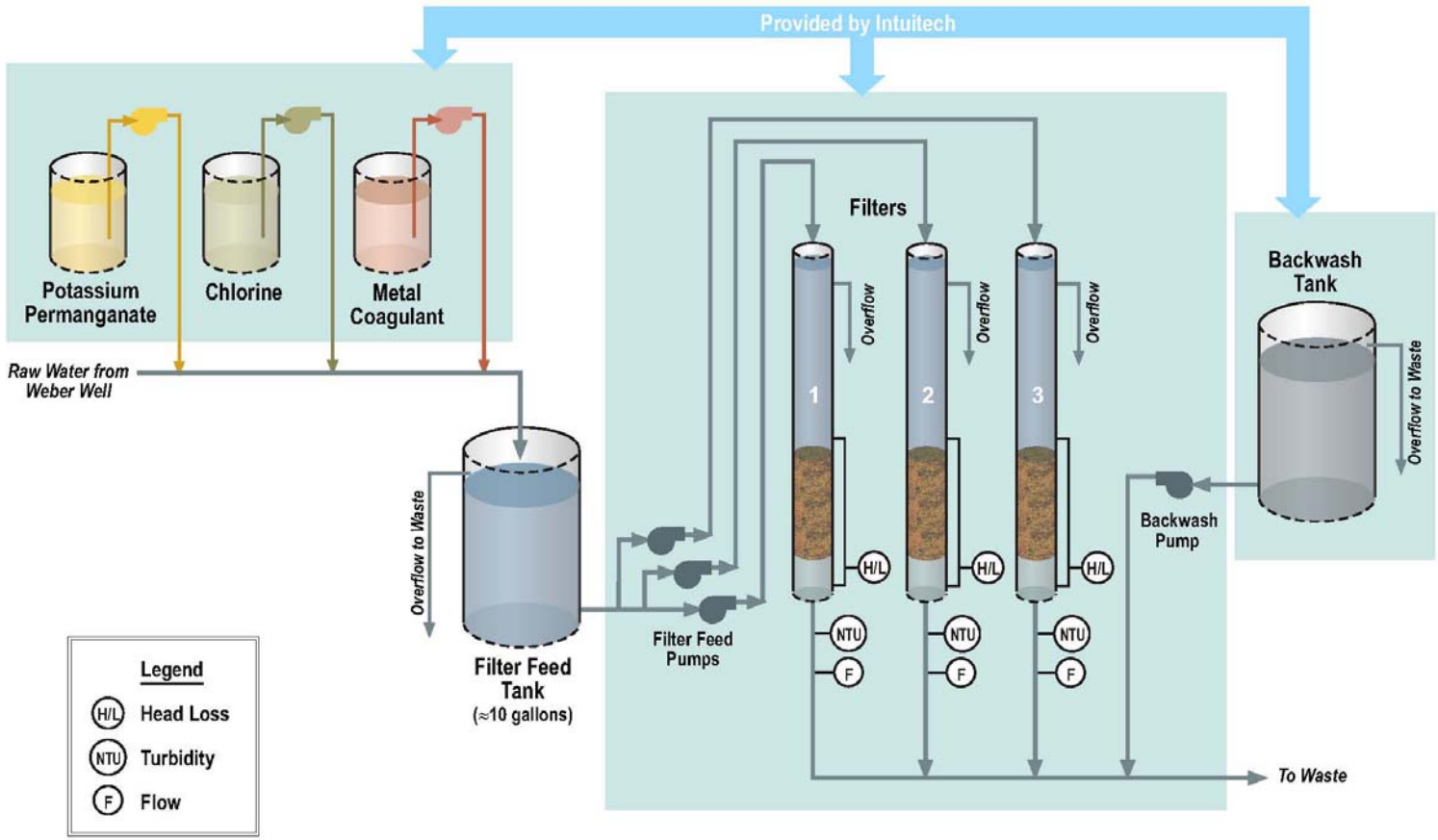


Table 2.4: Sampling and Analysis Plan

Parameter	Raw	Greensand	Greensand Plus	Silica Sand
Hand-Recorded Data:				
Flow rate (gpm)	3 / day	6 / day	6 / day	6 / day
Head loss (ft)		6 / day	6 / day	6 / day
Turbidity (NTU)		6 / day	6 / day	6 / day
Onsite Analyses:				
Temperature (°C)	1 / day	-	-	-
Chlorine Residual		3 / day	3 / day	3 / day
Alkalinity	1 / week	1 / week	1 / week	1 / week
pH[(a)]	2 / day	2 / day	2 / day	2 / day
Oxygen Reduction Potential (ORP)	1 / day	1 / day	1 / day	1 / day
Dissolved Oxygen	1 / day	1 / day	1 / day	1 / day
Iron (Total)	3 / day	6 / day	6 / day	6 / day
Manganese (Total)	3 / day	6 / day	6 / day	6 / day
Samples Shipped to Certified Lab:				
Iron (Total)	1 / day	1 / day	1 / day	1 / day
Iron (Dissolved)	2 / week	2 / week	2 / week	2 / week
Manganese (Total)	1 / day	1 / day	1 / day	1 / day
Manganese (Dissolved)	2 / week	2 / week	2 / week	2 / week
Arsenic (Total)	1 / day	1 / day	1 / day	1 / day
Arsenic Species (As-III and As-V)	1 / week	1 / study	1 / study	1 / study
Silica (Total)	1 / week	1 / week	1 / week	1 / week
TOC and DOC	2 / study	2 / study	2 / study	2 / study
DOH Primary and Secondary Inorganics[(b)]	1 / study	1 / study	1 / study	1 / study
Inorganics for Corrosion Control [(c)]	1 / study	1 / study	1 / study	1 / study

Notes:

(a) pH to be measured before and after addition of chlorine to the raw water.
(b) Includes regulated metals, cyanide, fluoride, nitrate/nitrite, calcium, magnesium, chloride, color, sulfate, total dissolved solids, hardness, foaming agents, conductivity, silica, and phosphorus.
(c) Includes pH, alkalinity, hardness, calcium, total dissolved solids, conductivity, sulfate, and chloride.

Iron, manganese, and arsenic concentrations were confirmed by submitting samples to a certified laboratory for metals analysis using an inductively coupled plasma (ICP) method. Prior to analysis, dissolved metals were determined by filtering the samples through a filter with a nominal pore size of 0.45 microns.

2.4.2 Pilot Study Results and Conclusions

2.4.2.1 Operational Parameters

Besides the type of media to be used to effectively treat a specific raw water, the only significant operational parameter in pilot filtration testing is the hydraulic loading rate (HLR) to the filters. During this pilot test, hydraulic loading rates in the range of 2 to 7 gpm per square foot (sf) were tested. The rate of 4 gpm/sf was found to be maximum reasonable rate that provided a run length of sufficient duration. Operation at higher rates would likely cause an elevated backwash frequency, which would result in an excessive volume of backwash water usage and lower rates would incur a much greater capital expenditure for more filter facilities.

2.4.2.2 Removal of Target Inorganic Contaminants

In general, during each filter run the levels of the target contaminants remained well below the treatment goals for the entirety of the filter operating time. Typical data for manganese removal, at a HLR of 4 gpm/sf, and feeding chlorine (with a small dose of potassium permanganate), and run data when feeding both chlorine and ferric chloride, are presented on Figures 2.2 and 2.3, respectively. When chlorine only was added, filter effluent concentrations of arsenic were consistently greater than 5 μg/L but less than the MCL of 10 μg/L. When ferric chloride was fed at a dose of 2 mg/L, in addition to chlorine, arsenic concentrations in the filter effluent water were consistently well below 5 μg/L, throughout the filter runs.

Also, when ferric chloride was fed, the concentrations of iron and manganese in the filter effluent remained above the treatment goal, and generally below the SMCL, for approximately 30 to 60 minutes of run time immediately following a backwash, before they dropped below the treatment goals.

It was observed in longer duration filter runs that the concentrations of iron, manganese, and to a lesser extent arsenic, began to increase. The higher levels of iron and manganese at the end of filter runs contributed to a visible color in the filter effluent water. By ending the run and initiating a backwash when the iron and manganese were approximately half the SMCL, this undesirable colored water was avoided.

Figure 2.2: Typical Filter Run Manganese Data (Feeding Chlorine)

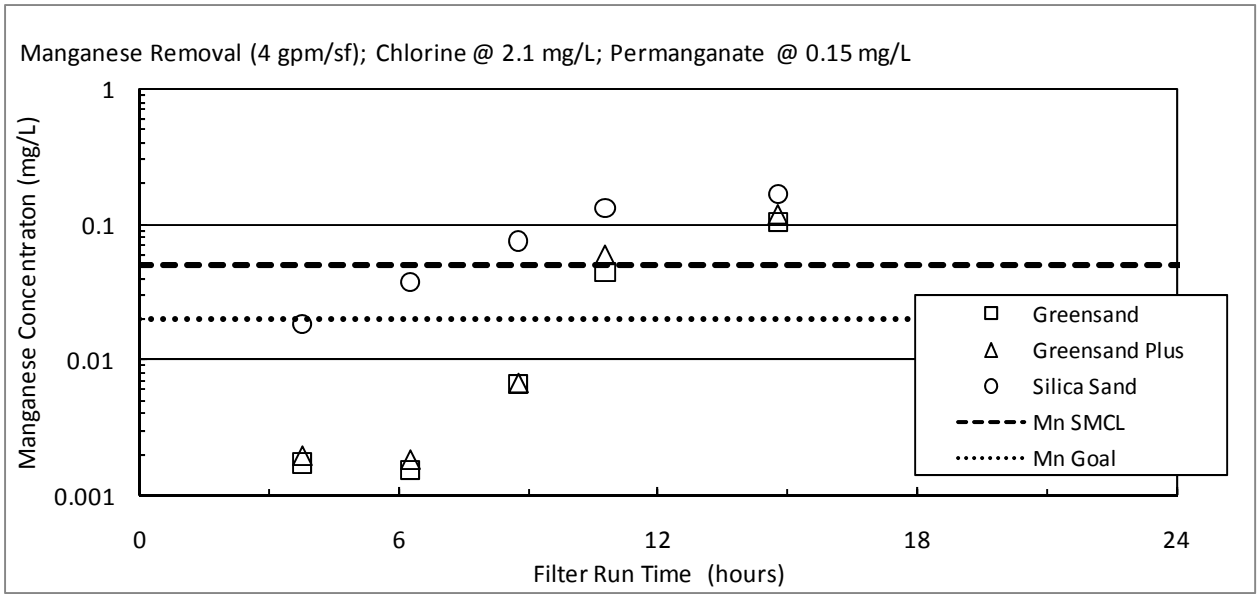
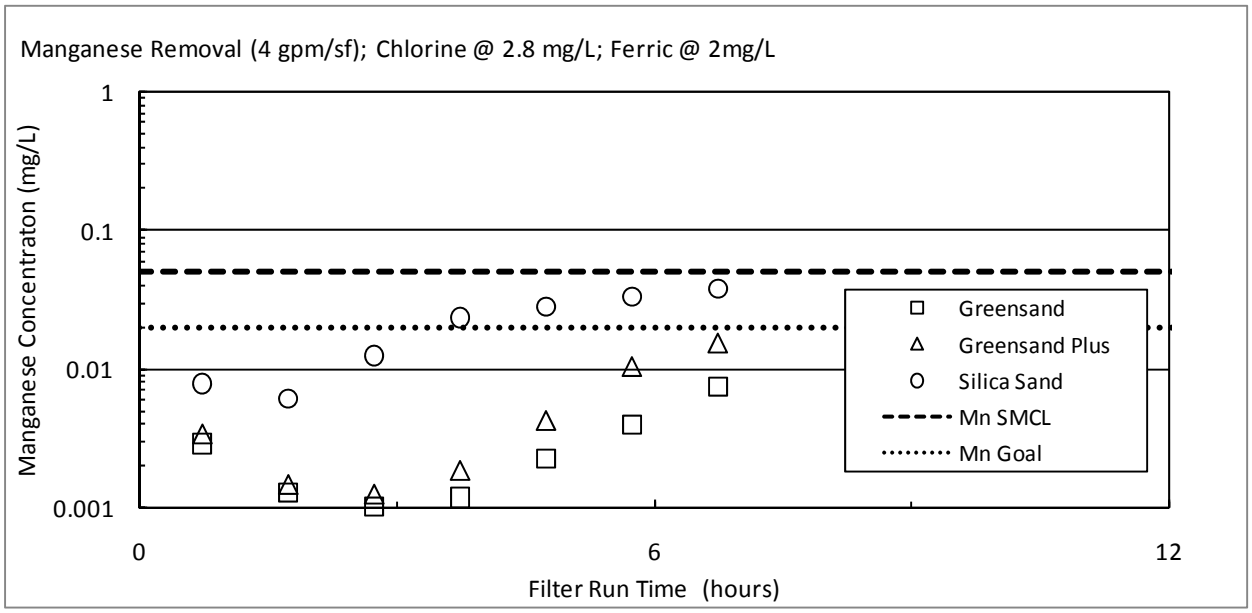


Figure 2.3: Typical Filter Run Manganese Data (Feeding Chlorine and Ferric Chloride)



2.4.2.3 Filter Run Time

A summary of the filter run times for various operating conditions are presented in Table 2.5. The filter runs were defined as the time from the initiation of the run until the time at which the iron or manganese concentration exceeded half of its SMCL.

Table 2.5: Summary of Filter Run Times for Various Operating Conditions

Hydraulic Loading Rate	Chemicals Fed	Filter Run Time (Hours)			UFRV (gal/ft^2)
		Greensand	Greensand+	Silica Sand	
7	Cl_2	5-6	5-6	2-4	2,520
4	PP	8	6-8	2-4	1,920
	Cl_2	10	10	4	2,400
	Cl_2 + ferric	7	6	4	1,680
3	Cl_2	13	13	8-10	2,340
	Cl_2 + ferric	9	9	6-8	1,620
2	Cl_2	20	20	12 (approx.)	2,400
	Cl_2 + ferric	14	14	10 (approx.)	1,680

Notes:

(a) UFRV = unit filter run volume, based upon Greensand media filter run time
(b) Cl_2 = sodium hypochlorite
(c) PP = potassium permanganate
(d) Ferric = ferric chloride

Pilot test results showed that Greensand and Greensand Plus demonstrated similar run longevity, with silica sand performing at significantly shorter durations. The hydraulic loading rate of 4 gpm/sf was selected as a reasonable maximum feasible filtration rate, and several runs

were performed while operating at that rate. Table 2.6 presents average filter run times for several of those runs. Other data collected during the pilot study are presented in Appendix B.

Table 2.6: Filter Run Length Summary (at 4 gpm/ft²)

Media	Average Filter Run Length	
	With Chlorine Only[(a)]	With Chlorine and Ferric[(b)]
Greensand	10 hours	7 hours
Greensand Plus	10 hours	6 hours
Silica Sand	4 hours	4 hours

Notes:

(a) Six filter runs performed over 11 days of operation.
(b) Five filter runs performed over 5 days of operation.

Elevated levels of iron, manganese, and color occurred in some filter runs, although this behavior did not occur at the initiation of every filter run. The silica sand media tended to exhibit higher levels of iron when spiking did occur. It is likely that as the media continues to mature and season, such spiking will reduce in magnitude and may disappear altogether. Figures 2.4 and 2.5 present iron concentrations and color data, respectively, to illustrate typical elevated concentrations of these parameters at the start and end of a filter run.

Figure 2.4: Iron Concentrations for a Filter Run in Which Higher Levels of Iron Occurred Early and Late in the Run

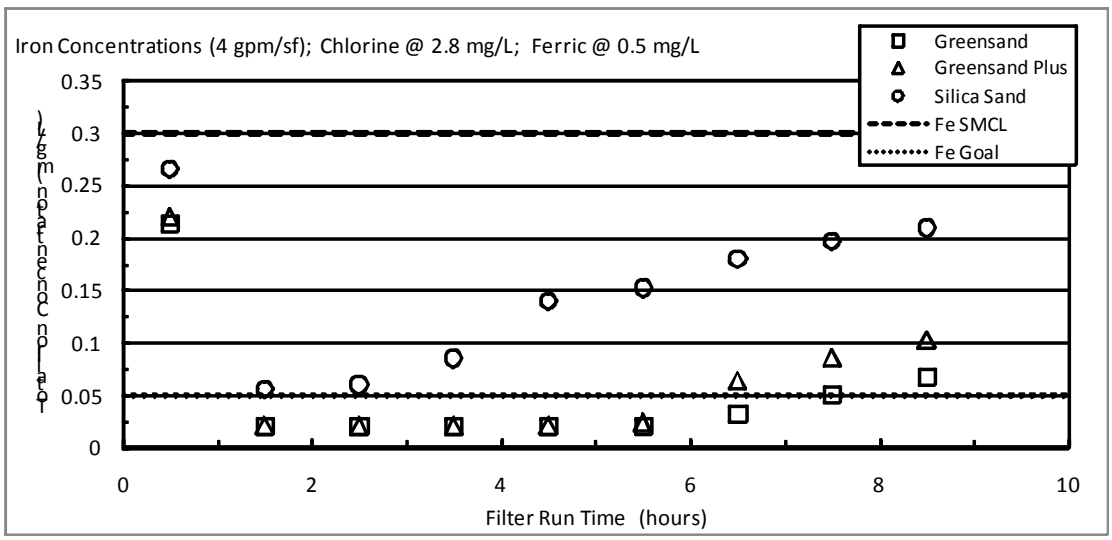
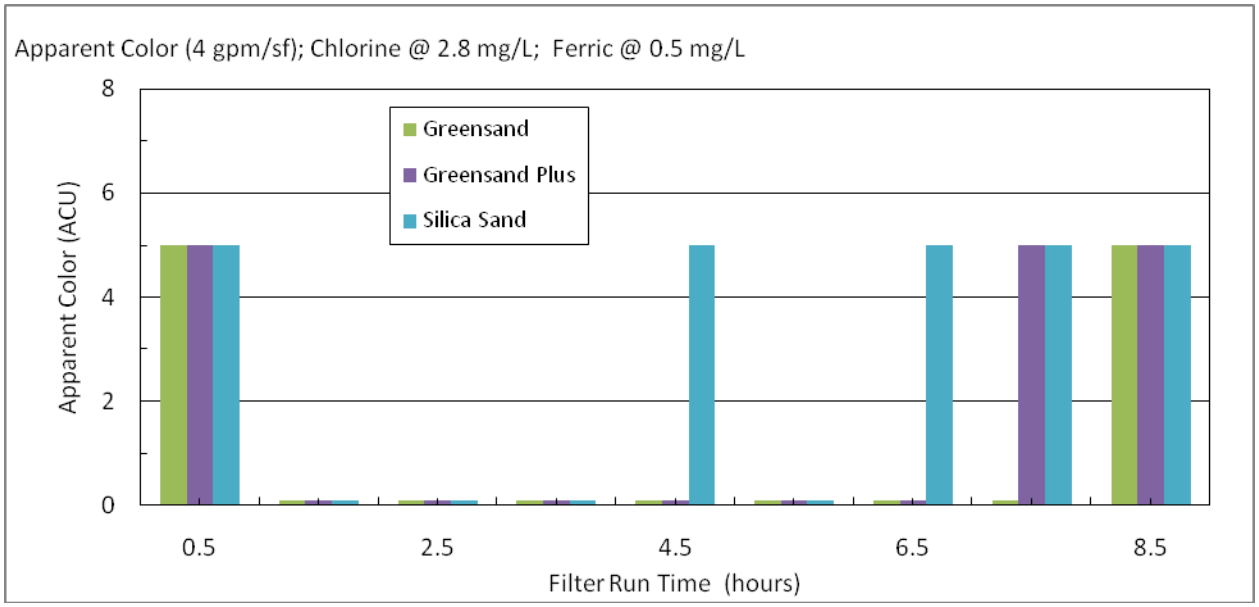


Figure 2.5: Apparent Color Data



2.4.2.4 Disinfection By-Products in the Treated Water

The total organic carbon (TOC) of the well water (1.8 mg/L) was higher than normally expected for a groundwater and within the range of values measured in the City's existing source of supply, the Cowlitz River (TOC in the range of 0.73 to 1.8 mg/L, between years 2004 and 2008). Disinfection by-product tests were performed on treated water from the pilot study to determine the extent of formation of the regulated compounds: Total trihalomethanes (TTHM) and haloacetic acids (HAA-5). Results for samples collected from each of the filter media effluents are as follows:

- Greensand: TTHM = 13.5 μg/L, HAA-5 = 12.7 μg/L
- Greensand Plus: TTHM = 13.4 μg/L, HAA-5 = 12.6 μg/L
- Silica Sand: TTHM = 12.4 μg/L, HAA-5 = 11.2 μg/L.

All three granular media had similar levels of these by-products. Results for a filter effluent sample collected and held for 24 hours are as follows:

- Greensand Plus: TTHM = 31.1μg/L, HAA-5 = 19.4 μg/L.

These results are well below the MCLs for TTHM (80 μg/L) and HAA-5 (60 μg/L).

2.4.2.5 Backwash Water Settling Tests

The settling characteristics of the backwash water solids were determined on a sample of backwash water collected during the pilot study using a standard jar test apparatus. The solids tended to settle readily without the addition of a chemical to aid settling. Gravity settling under quiescent conditions with a loading rate in the range of 0.25 to 0.5 gpm/ft^2 could be used to remove solids from the backwash water and return the supernatant to the head of the plant. Plate or tube settlers could be used to increase the loading rate to 1 to 2 gpm/ft^2.

2.4.2.6 Conclusions and Recommendations

The conclusions and recommendations based upon the results from this pilot study are as follows:

- **The Greensand media is expected to achieve the treated water goals of this study and is preferred for the treatment of this groundwater.** Greensand Plus could be considered as an alternative with similar performance. The silica sand, however, media had shorter filter runs and was more likely to exhibit higher filter effluent concentrations of iron, manganese, and color in the treated water.

- **The process of filtering to waste should be considered in the facilities design.** During filter run time, in the first few minutes after backwashing, higher levels of iron and manganese appeared in the filter effluent. Filtering to waste is the practice of sending the first few minutes of filter flow into a storage basin for settling and rerouting of the decanted supernatant water back to the head of the plant. This practice will keep solids-associated iron and manganese from entering the distribution system. Because effluent from all filters is blended, it may not be critical or practical to filter to waste. The effluent from a filter that has been returned to service following a backwash will likely be sufficiently diluted by the flow from the other filters because filter effluents are combined. Thus, from dilution, the combined plant finished water may likely comply with the finished water goals. Filtering to waste will be further investigated during final design.

- **Confirm distribution system water quality considerations during production test well operation.** The finished water from the new groundwater treatment facility will differ from the treated Cowlitz River water currently in the City's system. At start-up of the new regional water treatment plant (RWTP) when existing water and new treated water will blend in the distribution system, there may be issues with taste, odor, or scaling. Complaints are most likely to cite "water spotting" resulting from higher hardness inherent in the groundwater as opposed to complaints about deposition of a significant amount of calcium carbonate scale within home plumbing. Complaints are more likely during the brief transition period from surface water to groundwater and these will likely decrease over time. Thus, the anticipated finished water quality should be confirmed with samples collected during the pump test of the first production well. Also, at start-up of the new treatment facility, the City should be prepared to perform an extensive flushing program to mitigate problems from sloughing of existing scale within the distribution system pipes and to receive and respond to possible complaints due to the greater hardness of the groundwater.

- **Perform further disinfection by-product testing at longer holding times.** Holding times representative of the maximum water age in the distribution system are recommended to determine if there is a regulatory concern with TTHMs and/or HAA-5. This testing was conducted as part of the water quality testing during the first production well pump test.

- **Backwash water solids settling should be included in the facilities design.** Solids in the backwash wastewater tended to settle readily, under quiescent conditions, without the addition of chemicals. A backwash settling process is recommended for the new RWTP so that backwash supernatant can be recycled to the head of the plant and increase the efficiency of the use of the groundwater supply.

Section 3: Filtration Alternatives Evaluation

As concluded in Section 2 on water quality objectives, some form of filtration technology is necessary to provide full and adequate treatment of the deep aquifer groundwater. Also, as mentioned in Section 2, the City has in the past considered various filtration technologies through work preceding this project. Specifically a previous City consultant conducted an abbreviated microfiltration membrane and dual media sand filter pilot study. Against this background, and with the understanding that filtration is the primary treatment process used to treat the deep aquifer groundwater, the City needed to analyze and select between feasible filtration technologies. This section discusses the process by which Kennedy/Jenks facilitated the City stakeholder's selection of a filtration treatment technology.

3.1 Alternatives Evaluation Process

The filtration technology alternative evaluation process was conducted in five phases: identification and briefing of project stakeholders; discussion and selection of feasible alternatives; discussion and selection of evaluation criteria; preliminary work for stakeholder preparation to evaluate; and final workshop evaluation and selection.

3.1.1 Project Stakeholders

Stakeholders are those who have an interest in or are affected by the consequences of decisions on this project. Stakeholders for a major utilities project such as this one typically are representatives of the municipal and/or utility management, utility financial staff, facility (water treatment plant and distribution system) management, operations and maintenance staff, Washington State Department of Health (DOH) regulatory officials, design engineers and construction managers, and at times, representatives of the general public or specific public interest groups.

For this project the stakeholder representatives were:

- City Public Works Director
- City Project Engineer
- Beacon Hill Utility District Representative
- Cowlitz Public Utility District (PUD) Representative
- DOH Regulatory Engineer
- City Facility Operations Manager
- Cowlitz River Regional Water Treatment Plant (RWTP) Operator
- City Utilities Engineer.

3.1.2 Feasible Filtration Alternatives

From previous work preparing for this project and from the body of knowledge on water treatment filtration, the City and Kennedy/Jenks concluded the two feasible filtration

technologies to be considered for the Mint Farm Regional Water Treatment Plant were: granular media filtration (of which there are several forms) and membrane filtration.

Other filtration methods are: slow sand filtration, diatomaceous earth filtration, conventional filtration, and other processes allowed under WAC 246-290. These treatment methods were screened out from further consideration based on the following:

- Demonstrated inability to meet water quality objectives with a groundwater containing iron, manganese, and arsenic
- Unproven ability to meet water quality objectives with a groundwater containing iron, manganese, and arsenic
- Not compatible with the treated water capacity requirements for this project
- Significant high cost – either capital or operational
- Lack of regulatory acceptance.

As discussed in Section 2, Kennedy/Jenks conducted pilot testing of granular media using several different forms: greensand with anthracite coal, "greensand plus" with anthracite, and dual media manganese oxide – coated silica sand with anthracite. Our conclusions from pilot testing supported the use of greensand granular media as the preferred granular media alternative. Consequently, the filtration technology evaluation for this project compared greensand granular media filtration to membrane filtration.

3.1.3 Evaluation Criteria

To facilitate the evaluation process, Kennedy/Jenks proposed a list of evaluation criteria to the City that could serve to select the best filtration technology for the City's project needs. After review with the City, the list was reduced down to eight distinct evaluation criteria. Keeping the list of evaluation criteria short and distinct allows stakeholders to ultimately compare alternatives more concisely and with a greater level of understanding. The list of selected evaluation criteria is presented in subsection 3.2.

3.1.4 Stakeholder Preparation

The goal of the alternative evaluation process, as stated in the introduction to this section, is to engage and inform the project stakeholders in order that they are prepared to make the best project decision, and then convene the stakeholders to conduct the process of alternative selection in a facilitated environment. In order to prepare stakeholders for the Alternatives Evaluation Workshop, the following materials were prepared and sent to the City for distribution to stakeholders prior to the workshop:

- Workshop agenda and list of activities with brief descriptions
- Final list of evaluation criteria with explanations of each criterion
- Paired comparison weighting matrix form with evaluation criteria and example values
- Alternatives evaluation matrix form.

3.1.5 Alternatives Evaluation Workshop

The Alternatives Evaluation Workshop for stakeholders was held on 14 July 2009. In addition to Kennedy/Jenks staff, the following stakeholder representatives were present and participated in this workshop:

- Jeff Cameron, P.E., Longview Public Works Director
- Amy Blain, P.E., Longview Project Engineer
- Kim Adamson, Beacon Hill Sewer District Manager
- Jacki Masters, Longview Water/Sewer System Superintendent
- Vic Richards, Longview RWTP Chief Operator
- Robert Menzia, City of Longview Utilities
- Teresa Walker, P.E., State of Washington DOH
- Jeff Marti, State of Washington DOE.

At the workshop, participants were briefed on the agenda for the workshop activities, then Kennedy/Jenks staff facilitated weighting of evaluation criteria and final evaluation of alternatives using the evaluation criteria. Stakeholders discussed and evaluated the alternatives by reaching consensus on a score for each alternative considered under each evaluation criteria. A final tabulated scoring matrix, applying weights to scores for each alternative and each evaluation criteria, was completed on a computer and using an overhead projector such that all stakeholders could visualize the process. Sensitivity analysis, explained in more detail below, was used to check any scoring for which it was either difficult to reach consensus or where it was perceived a change in a particular score would change the final outcome.

3.2 Evaluation Criteria and Weighting

Alternative evaluation criteria were developed as discussed above between Kennedy/Jenks and the City. These are the criteria that have greater meaning and encompassed the stakeholders' primary issues and concerns:

1. ***Capital Cost*** – Comparative (to the other alternative) construction cost and City programmatic cost associated with the treatment alternative (not a stand-alone cost).

2. ***Net Present Value --*** Comparative (to the other alternative) net present value (NPV) associated with the alternative: includes capital cost, cost of operations and maintenance labor, chemicals and consumed materials, power consumption, and equipment repair and replacement (not a stand-alone NPV).

3. ***Operability*** – Ability of the treatment alternative to be operated by the current staff with minimal retraining.

4. ***Performance*** - Discoverable record of satisfactory performance of the treatment alternative to meet treatment objectives of removing iron, manganese and arsenic from groundwater.

5. ***Flexibility*** - Ability of the treatment alternative to be modified and perform successfully with fluctuation in raw water quality or with change of the raw water source.

6. ***Complexity*** – Mechanical, controls and hydraulic complexity of the treatment alternative.

7. ***Capacity*** - Relative capacity the treatment alternative provides within a specific treatment plant footprint.

8. ***Regulatory Acceptance*** - Acceptance and trust of treatment alternative viability by regulatory agencies.

3.2.1 Weighting Matrix

Rarely are evaluation criteria of the same relative importance to stakeholders when deciding between alternatives. Establishing relative significance of evaluation criteria using a weight for each improves the effectiveness and outcome of the evaluation process.

In order to generate weighting factors for each evaluation criterion, as part of the Alternatives Evaluation Workshop, a weighting matrix was created using the Paired Comparison Method. In this method, stakeholders compare and discuss each evaluation criterion with respect to each other criterion to establish which of the two is relatively more important to them thus deserving more "weight". For the eight evaluation criteria of the Alternatives Evaluation Workshop, stakeholders considered 29 comparisons. At the end of the exercise the individual weights were added up to produce a total weight for each evaluation criterion. The completed matrix with results is shown below in Table 3.1. It is important to note that a paired comparison weighting is not affected by consideration of the alternatives, thus the treatment alternatives are presented after the weighting matrix. The paired comparison weighting is simply used to establish the stakeholders' consensus characterization of the relative importance of each criterion.

Table 3.1: Paired Comparison Weighting Matrix

Evaluation Criteria	***Capital Cost (CC)***	***Net Present Value (PV)***	***Operability (OP)***	***Performance (PF)***	***Flexibility (FX)***	***Complexity (CX)***	***Capacity (CA)***	***Regulatory Acceptance (RA)***	***Total Weight***
Capital Cost (CC)		-	3[a]	-	3	4	-	-	10
Net Present Value (PV)	3		3	-	3	4[b]	-	-	13
Operability (OP)	-	-		-	-	1	-	-	1
Performance (PF)	3	3	3		4	3	1[c]	2	19
Flexibility (FX)	-	-	2	-		-	-[d]	-	2
Complexity (CX)	-	-	1	-	2		-	-	3
Capacity (CA)	3	3	3	1	2	3		-	15
Regulatory Acceptance (RA)	3	3	2	-	3	2	3		16

[a] Example – Capital Cost is "clearly more important" (3) than Operability
[b] Example – Net Present Value is "much more important" (4) than Complexity.
[c] Example – Performance is no more important than Capacity; neither criterion is more important (1).
d Example – Flexibility is less important that Capacity (-).
Numerical weight interpretations (applies to criteria named in row titles on left side of table):

4 – Much more important
3 – Clearly more important
2 – Slightly more important
1 – Neither criterion is more important
- (dash) – Criterion (named on left) is less important than criterion named

in column titles above.

3.3 Filtration Alternatives

As discussed in the introduction to this section, only granular media filtration and membrane filtration were considered feasible alternatives for this project. At the City's request, and in order to provide perspective to the cost comparison component of this alternatives analysis, Kennedy/Jenks also provides in this section a summary of findings from *the City of Longview RWTP Constructability Review* (Kennedy/Jenks Consultants 2008) *(Constructability Review)* and a summary table of updated costs for the eight-year phased program of capital investments required to repair and rehabilitate the existing water treatment plant. The nature of, advantages of, and disadvantages of greensand filtration and membrane filtration were presented to and discussed with stakeholders at the Alternatives Evaluation Workshop.

Capital and operating costs were of significant interest to the City in these evaluations; a relative cost comparison of both greensand filtration and membrane filtration is presented in subsection 3.3.4 below. This analysis, at City request, includes a comparative cost to repair and rehabilitate the existing Cowlitz River RWTP.

3.3.1 Greensand Granular Media Filtration

Pilot testing results and analysis presented in Section 2 concluded that of all the granular media filtration methods considered, greensand granular media filtration was the best fit and provided the highest quality treated water at the most effective operating conditions for the deep aquifer groundwater. Greensand filtration will require some pretreatment and typically is achieved through the use of pressurized filter vessels. This offers the opportunity to utilize the well pumps as the means to pressurize and send flow through the filters and directly into the City distribution system.

3.3.1.1 Pretreatment

The greensand granular media process would require pretreatment to oxidize and precipitate iron and arsenic. To achieve pretreatment, an oxidant, such as chlorine, will need to be added to the groundwater and allowed adequate contact time, prior to passing through the greensand. Pre-oxidation allows adsorption of non and manganese oxide particles on the greensand media in pressure vessels. The greensand media itself is pre-treated with permanganate after being loaded into the pressure filters but prior to initiating operations, which in turn facilitates adsorption of manganese onto the filter media.

3.3.1.2 Ancillary Equipment and Systems

Other process and ancillary treatment equipment required, besides pressure filters, is as follows:

- Raw water wells
- Raw water well pumps
- Emergency engine generator
- Backwash recovery tanks and backwash return pumps
- Sodium hypochlorite pre-oxidation and distribution system residual system
- Fluoridation system

- Administrative and laboratory building
- Solids dewatering facilities – including solids pumping.

3.3.1.3 Capacity

Longer filter runs will likely be achieved at full-scale since production well PW1 was found to have lower levels of iron, manganese, and arsenic than the Prudential Boulevard Well used for the pilot test. Additionally, if at a future date the City decided to use surface water instead of groundwater, the greensand pressure filtration vessels could serve as a pre-treatment to membrane filtration. In treatment of surface waters, granular media on its own would not suffice, additional treatment would be required.

3.3.1.4 Evaluation

Greensand filtration is a proven treatment process for iron, manganese and arsenic removal from groundwaters. The system is familiar to the water treatment industry and has demonstrated robust and highly reliable performance. Table 3.2, below, presents a summary of advantages and disadvantages of the greensand filtration treatment process as discussed with stakeholders in the Alternatives Evaluation Workshop.

Table 3.2: Advantages and Disadvantages – Greensand Filtration

Advantages	Disadvantages
• Well-established, robust and proven process for removal of iron, manganese, and arsenic • Distribution pumping not needed • Major equipment, such as pressure vessels are provided by numerous vendors and can be designed without pre-selection • Media type can be changed in the future if a new improved media becomes available • Lower capital cost than membranes • Lower operations and maintenance cost than membranes • Operations, maintenance and controls simpler than for membranes • Operations and maintenance simpler than existing Cowlitz River RWTP • Lower power and operations costs • Resilient to influent hydraulic loading • Fewer buildings required • Operator skills required similar to those for existing plant • Significant expertise is available from vendors' to provide start-up and commissioning support, and ongoing operations assistance	• Footprint of process (especially filters and washwater basins) may be larger (but less of the footprint is in buildings) than required for membrane filtration. • Requires additional process facilities to treat surface water should the City choose to add surface water treatment in the future.

3.3.2 Membrane Filtration

3.3.2.1 Pretreatment

Membrane filtration treatment of the deep aquifer groundwater for this project would also require adding an oxidant to oxidize the iron and manganese to form filterable particles. A key difference between the two processes is that microfiltration/ultrafiltration hollow fiber membranes use a physical separation process to remove (strain out) particles and solid contaminants in the raw water where greensand granular media filtration relies on absorption (and less rigorous straining) processes. As a consequence, any pretreatment prior to membrane filtration must ensure complete precipitation of all iron, manganese and arsenic species such that these constituents are not still dissolved and able to pass through the membranes when being filtered. In greensand filtration, precipitation and absorption occur within the filtration media. As a consequence, membrane filtration will require addition of a stronger oxidant than chlorine, typically potassium permanganate, and a dedicated contact basin upstream of the membranes to enhance the formation of manganese oxides prior to filtration.

3.3.2.2 Ancillary Equipment and Systems

Other process and ancillary treatment equipment required, besides membrane filtration modules, is as follows:

- Raw water wells
- Raw water well pumps
- Emergency engine generator
- Membrane building
- Membrane cleaning system and equipment
- Complex membrane instrumentation and controls package
- Sodium hypochlorite and potassium permanganate pre-oxidation system
- Sodium hypochlorite distribution system residual system
- Fluoridation system
- Equalization tank for chemical contact and constant membrane feed
- Administrative and laboratory building
- Solids dewatering facilities – including solids pumping
- Distribution system booster pumping station.

3.3.2.3 Capacity

From previous studies and industry performance data the capacity (flux rate) through the membrane filters themselves, as part of a membrane filtration process to treat the deep aquifer groundwater, would be greater than that for the greensand pressure filters. The anticipated flux rate for membrane filters is 40 to 60 gpm/sq ft. The membrane modules' footprint in a membrane filtration plant would likely be smaller than that required for greensand pressure filters. However, because the pre-treatment process requirements for a membrane filtration process is more rigorous than that for a greensand filtration process, the overall plant footprint

for the two processes would be very similar in size. The footprint may not be similar for structures, however, since membrane filters would require more building and tankage space.

3.3.2.4 Evaluation

Membrane filtration is a proven *surface* water treatment process, but has not been used extensively for groundwater treatment of iron and manganese. An abbreviated membrane pilot study conducted in 2008 by the City suggested it might be difficult for membranes to consistently meet the treatment objective, specifically for removal of manganese; removal of arsenic was not tested in that study. Furthermore, the presence of any organics in the groundwater could contribute to increased fouling and chemical cleanings of a membrane system. DOH has expressed concern over the use of membranes in this project primarily due to the unanswered questions in previous membrane pilot studies and in general the lack of proven performance of membranes in treating groundwater for the removal of iron, manganese, and arsenic.

The footprint of the filtration process would likely be smaller than that for greensand, but would require additional building and tank structures, such as a raw water contact tank, a membrane building or additional floor space in other plant buildings for membrane filtration racks, and a stand-alone finished water pumping station. Membranes are inherently a proprietary product and would require a long-term relationship with a specific vendor. Membrane systems have higher operations and maintenance cost than granular media because of the need to purchase new membranes on a regular basis (typically five years for drinking water applications) from the chosen vendor and because of wear during operation and the aggressive chemicals used in routine membrane cleaning. Treatment oxidants might also impact the service life of membranes. The membrane system could be easily expanded because of the modular nature of the system.

Table 3.3 presents the summary of advantages and disadvantages of the membrane filtration treatment process as discussed with stakeholders in the Alternatives Evaluation Workshop.

Table 3.3: Advantages and Disadvantages – Membrane Filtration

Advantages	Disadvantages
• Modular process that can be reasonably easily expanded by adding additional membrane racks or modules • Efficiency of filtration (capacity per square foot of footprint); Footprint of filtration process will be smaller • Flexibility to treat surface water, if the City chooses to add surface water treatment in the future • Positive barrier to	• Footprint of structures and tanks (raw water contact tank, membrane rack building space, and stand-alone finished water pump station) will be greater (though overall footprint may be similar) • Higher capital cost than greensand • Higher operations and maintenance cost than greensand media (membranes require replacement approximately every 5 to 7 years • Finished water booster pumping required • Complexity of mechanical, instrumentation and controls, and controls interface • Proprietary vendor package – pre--purchase required prior to design • Unproven in iron and manganese removal – did not meet treatment goal for manganese or arsenic removal in pilot testing by others • Unfamiliar operations requirements/operator skills

Advantages	Disadvantages
particulate solids and most pathogens	different than for existing treatment plant • DOH has expressed concern with use of membranes for this project. • Potassium permanganate is required to adequately oxidize iron and manganese upstream from membrane filtration – increased operational complexity and number of chemicals and chemical systems on site • Membranes are vulnerable to exposure to strong oxidizers such as chlorine and permanganate – potential to reduce membrane life, increase replacement costs.

3.3.3 Rehabilitate the Existing Treatment Plant

Though not considered a viable alternative due to City concerns with continued vulnerability of the Cowlitz River source water to silt-laden flow which plugs the intake system, consequent higher maintenance costs, limitations on capacity, risk of flood-induced water quality degradation, and risk of contamination with contaminants of emerging concern (CECs), a summary of the phased repair and rehabilitation program is provided here in conjunction with the summary of costs for this program updated from the *Constructability Review*.

3.3.3.1 Key Elements in Repair and Rehabilitation

As discussed in the *Constructability Review*, the Cowlitz River RWTP must remain in operation during its repair and an eight-year phased construction effort is anticipated. Rehabilitating the existing RWTP would also require costly upgrades to meet current codes, significant repairs to the existing intake structure on the Cowlitz River, and design and construction of new river training structures. To better characterize these elements the executive summary of the *Constructability Review* is included in Appendix C of this report.

3.3.4 Alternatives Cost Comparison

Cost is of significant importance to the City in all decisions made on the MFRWTP Project. Capital cost and net present value costs are key factors in the selection of the preferred filtration alternative. In order to more specifically characterize the difference in cost of alternatives, albeit at a very early date in the design process where there was not much detail known of final design requirements, the City requested Kennedy/Jenks perform a relative comparison of both capital construction costs and net present value (including capital construction costs) for both filtration alternatives as best as can be expressed at this time. In addition, the City requested the present value costs for rehabilitation and repair of the existing RWTP be presented with the alternatives' cost comparison to give cost perspective to the MFRWTP Project.

3.3.4.1 Comparative Capital Construction Costs

Tables 3.4 and 3.5 present the comparative construction costs for Membrane Filtration and Greensand Filtration in year 2009 dollars escalated to midpoint of construction in year 2012. The primary source for unit costs and some quantities for these estimates was the *Value Engineering Workshop Report (Smith-Culp, September 4, 2007).* In addition, many quantity estimates were updated by the Kennedy/Jenks conceptual design as of May 2009. In some cases, unit costs, where more specifically known at the time of the alternatives analysis, were

used in place of Value Engineering costs. In a few instances, in the preparation of this report and based on final site layout drawings and calculations, quantities and costs were updated in both membrane and greensand alternative estimates to ensure fair and accurate cost comparison. These comparative estimates do not include the following common costs shared by both alternatives: site work, well development, well pump, and well building costs; distribution piping costs; landscaping and other general work; sludge dewatering and drying facility costs; and mobilization costs.

Table 3.4: Greensand Granular Media Filtration – Comparative Construction Cost Estimate

Item	Units	Unit Cost	Quantity	Cost (2007 $)
Site Work:				
cleaing and grubbing	LS	$25,000	1	$25,000
grading and paving	LS	$135,000.0	1	$135,000
site dewatering	LS	$150,000	1	$150,000
site fencing	FT	$20	1,000	$20,000
site piping	LS	$970,000	1	$970,000
preloading	CY	$13	12,000	$155,466
Excavation:				
backwash recovery tanks (footprint: 85' x 130', 8' depth)	CY	$25	3,274	$81,850
filter gallery	CY	$25	1,200	$30,000
office/treatment building	CY	$25	1,200	$30,000
Foundations:				
Base (3' thick gravel)				
filter gallery and filters	CY	$19	450	$8,329
office/treatment building	CY	$19	692	$12,807
backwash recovery tanks	CY	$19	1,228	$22,728
Concrete foundation				
filters	CY	$463	78	$36,090
office/treatment building	CY	$463		$
backwash recovery tank	CY	$463	76	$35,165
Structural Concrete:				
Backwash recovery tanks:				
bottom slab	CY	$463	765	$353,962
basin walls	CY	$833	222	$185,078
basin covers (metal)	SF	$51	1,296	$65,977
Buildings and Floor Slabs:				
Office/treatment building – masonry				
building shell and slab	SF	$185	3,813	$705,703
Filter gallery building – masonry				
building shell and slab	SF	$185	3,578	$662,209
Mechanical				
Mechanical equipment for process and building mechanical	LS	$1,200,000	1	$1,200,000
Additional cost for larger well pumps	LS	$250,000	1	$250,000
Chemical feed piping/fittings	LS	$35,000	1	$35,000
Pressure vessels w/ small reactors	Each	$412,724	9	$3,714,518
Surge tank	Each	$86,061	1	$86,061
Equipment				
Chem feed (fluoride, polymer) and other equipment	LS	$1,400,000	1	$1,400,000
On-site hypochlorite generation	LS	$116,000	1	$116,000
Subtotal				$10,486,942
Electrical	LS	PERC	12%	$1,258,433
Instrumentation and Controls	LS	PERC	6%	$629,217
Subtotal				**$12,374,592**
Overhead, profit, contingency and taxes factor		61%		$7,548,501
Total				**$19,923,093**
	Sept 2007 ENR 7938		Adjust to $ 2009 =	$21,529,389
	May 2009 ENR 8578		**Adjust to midpoint of const. =**	**$22,400,000**
			(Rounded)	

Table 3.5: Membrane Filtration – Comparative Construction Cost Estimate

Item	Units	Unit Cost	Quantity	Cost (2007 $)
Site Work:				
cleaing and grubbing	LS	$25,000	1	$25,000
grading and paving	LS	$135,000	1	$135,000
site dewatering	LS	$150,000	1	$150,000
site fencing	FT	$20	1,000	$20,000
site piping	LS	$970,000	1	$970,000
preloading	CY	$13	12,000	$155,466
Excavation:				
Steel backwash recovery tanks (2 at 60 ft dia)	CY	$25	667	$16,675
Contact tank	CY	$25	267	$6,667
Distibution pump station	CY	$25	111	$2,778
Office/treatment building (includes membrane area)	CY	$25	1,500	$37,500
Foundations:				
Base (3' thick gravel)				
office/treatment building	CY	$19	865	$16,009
backwash recovery tanks	CY	$19	667	$12,339
contact tank – 60' x 40'	CY	$19	267	$4,935
distribution system pumping station	CY	$19	111	$2,056
Concrete foundation				
contact tank	CY	$463	33	$15,423
office/treatment building	CY	$463		$
backwash recovery tanks	CY	$463	63	$29,057
Structural Concrete:				
Contact tank				
bottom slab	CY	$463	89	$41,128
basin walls	CY	$833	600	$499,711
metal covers	SF	$51	6,000	$305,379
Buildings and Floor Slabs:				
Office/treatment building – masonry				
building shell and slab	SF	$185	4,766	$882,129
Distribution Pump Station – masonry				
building shell and slab	SF	$150	1,000	$150,000
Mechanical				
Mechanical equipment for process and building mechanical	LS	$1,200,000	1	$1,200,000
Distribution pumps	LS	$90,000	1	$90,000
Process skid piping	Each	$35,000	7	$245,000
Process skid valves	Each	$8,000	7	$56,000
Chemical feed piping/fittings	LS	$35,000	1	$35,000
Bolted steel backwash tanks (2 at 60 ft dia)/ incl covers	LS	$660,000	1	$660,000
Equipment				
Membrane vendor package – membranes, appurtenances, controls	LS	$7,500,000	1	$7,500,000
Chem feed (fluoride, permanganate, polymer) and other equipment	LS	$1,610,000	1	$1,610,000
On-site hypochlorite generation	LS	$116,000	1	$116,000
Subtotal				**$14,989,252**
Electrical	LS	PERC	8%	$1,199,140
Instrumentation and Controls	LS	PERC	4%	$599,570
Subtotal				**$16,787,962**
Overhead, profit, contingency and taxes factor		61%		$10,240,657
Total				**$27,028,619**

Sept 2007 ENR 7938		Adjust to $ 2009 =	$29,207,797
May 2009 ENR 8578	**Adjust to midpoint of const. =**	**$**	**30,400,000**
	(Rounded)		

From the comparative cost estimates it is clear that the greensand media alternative has a relatively lower capital cost.

3.3.4.2 Capital and Net Present Value Costs for Repair and Rehabilitation of the Existing RWTP

Table 3.6 is a summary of the phased construction costs required to repair and rehabilitate the existing Cowlitz River RWTP. This summary table is taken from the *2008 Constructability Review* of the existing RWTP facility. Table 3.6 shows NPV costs which include capital construction and City programmatic costs.

Table 3.6: Summary of Phased Construction Costs

Table 12: Existing RWTP Rehabilitation Programmatic Costs

	Project	2008	2009	2010	2011	2012	2013	2014	2015	Net Present Value (2008$)
Escalated Cost:	**$55,449,219**	**$250,000**	**$803,745**	**$4,706,379**	**$8,243,841**	**$14,317,508**	**$14,888,639**	**$8,025,644**	**$4,213,463**	**$45,934,607**
Project Phases:										
Phase 1: Intake Rehabiltation and River Training Structures										**$6,187,500**
Study		$250,000	$250,000							$500,000
Design			$284,375	$284,375						$568,750
Construction Management				$284,375	$284,375					$568,750
Construction				$2,275,000	$2,275,000					$4,550,000
Escalation			$13,196	$215,925	$332,018					
Phase 2: West Treatment Train										**$26,109,876**
Preliminary Design (All RWTP Phases)			$250,000	$250,000						$500,000
Design				$1,280,494	$1,280,494					$2,560,988
Construction Management					$384,148	$1,280,494	$896,346			$2,560,988
Construction					$3,073,185	$10,243,950	$7,170,765			$20,487,901
Escalation			$6,174	$116,210	$614,621	$2,145,997	$1,980,663			
Phase 3: Treatment Train 2 and 3 Replacement										**$13,637,232**
Design						$545,489	$818,234			$1,363,723
Construction Management							$340,931	$681,862	$340,931	$1,363,723
Construction							$2,727,446	$5,454,893	$2,727,446	$10,909,785
Escalation						$101,577	$954,254	$1,888,890	$1,145,086	
Phase 4: Repair, Replace, or Abandon Train 1										
Design										$0
Construction Management										$0
Construction										$0
Escalation										$0

Source: 2008 Longview Regional Water Treatment Plant Constructability Study

Note: Phase 4 is only required if more than 20 mgd of capacity is desired in the future.
Train 1 will reach the end of its useful life by year 2016, as determined in the 2008 Constructability Study.

3.3.4.3 Comparative Net Present Value Costs

Table 3.7 presents operations, maintenance, chemical replacement, power and other recurring costs, expressed as the net present value of a 20-year stream of annual costs. Table 3.7 also includes the comparative capital costs developed in Tables 3.4 and 3.5, for both alternatives, but with costs added to represent facilities common to both alternatives mentioned above. Finally Table 3.7 gives, for comparison, a summary of the phased program costs and NPV operation and maintenance costs from City records for this existing RWTP.

Again, Table 3.7 shows that the greensand media alternative has a decided advantage in relative NPV cost over both the membrane alternative and repair and rehabilitation of the existing RWTR.

Table 3.7: Comparative NPV Costs for Alternatives – 2009 Dollars

Cost Elements	Greensand Alternative	Membrane Alternative	Rehabilitate Existing Surface Water Plant
Comparative Construction Cost	$ 30,800,000	$ 38,800,000	$ 38,100,000
Comparative O&M NPV[1]	$ 9,075,260	$ 22,316,212	$ 12,645,854
City Allied Costs[2]	$ 687,000	$ 687,000	$ 1,840,000
Engineering Costs[3]	$ 8,574,000	$ 8,574,000	$ 10,200,000
Property Acquisition[4]	$ 2,000,000	$ 2,000,000	$ -
Other City Expenses[5]	$ 235,000	$ 235,000	$ -
Total Comparative NPV[6]	**$ 51,400,000**	**$ 72,600,000**	**$ 62,800,000**

Notes

[1]20-year stream of annual costs
[2]Includes administrative, legal, staff CM, and other staff costs
[3]Includes preliminary design, design, services during construction, and construction management costs
[4]Total cost including taxes
[5]Includes water quality analyis (lab costs), hydrogeology and water resource investigation, and public relations
[6]Total NPV costs are rounded

Assumptions

* Comparative construction costs for membranes and greensand include $5 million ($8.4 escalated) to account for common costs (wells, pipelines, solids drying, mobilization)
* Comparative capital construction costs iare escalated to midpoint of construction
* Mint Farm is a single investment program - built and online in 4th quarter 2012
* Cowlitz River is an 8-year phased investment program - final phase constructed and online by 4th quarter 2017
* No annual O&M costs included - this analysis strictly based on programmatic capital costs (NPV) in 2009 dollars

From Table 3.6, the NPV cost in 2009 dollars, even without representative operations and maintenance recurring costs, is significantly higher for the repair and rehabilitation of the existing RWTP facility.

Note that demolition costs for the existing facility will be an additive component of the programmatic cost for construction of a new MFRWTP facility. However, the exact timing of demolition, or whether the City will choose to entirely demolish the existing facility, has not been

decided and thus this cost is not included in the present cost comparison. For use in future considerations a cost estimate of full demolition was prepared for the City and can be found in Appendix G of this report. The estimated demolition cost in 2009 dollars is $1.17 million.

3.4 Evaluation of Filtration Alternatives

Prior to the 14 July 2009, Alternatives Evaluation Workshop, an evaluation matrix form was prepared which provided a place for the weighting factors from the weighting matrix and a place for stakeholders' evaluation scores. During the workshop, using the alternatives information from Kennedy/Jenks Consultants, summarized previously in this section, and with input from the City staff and other stakeholders and existing data available concerning the project, stakeholders discussed alternatives with respect to evaluation criteria and came to a consensus scoring of each alternative for each criterion. Alternatives scoring and the evaluation matrix were then completed during the workshop.

3.4.1 Evaluation Matrix

Table 3.8 shows the completed evaluation matrix for the proposed treatment alternatives of greensand granular media and membrane filtration. In every category except capacity, greensand granular media was ranked as the superior process. The analysis presented in Table 3.8 represents the scoring concluded by the stakeholders.

Table 3.8: Filtration Technology Evaluation Matrix

Evaluation Criteria	Weight	Greensand Media Score	Membrane Score	Greensand Media Weighted Score	Membrane Weighted Score
Capital Cost (CC)	***10***	5	4	50	40
Net Present Value (PV)	***13***	8	2	104	26
Operability (OP)	***1***	7	3	7	3
Performance (PF)	***19***	9	5	171	95
Flexibility (FX)	***2***	5	5	10	10
Complexity (CX)	***3***	8	5	24	15
Capacity (CA)	***15***	3	8	45	120
Regulatory Acceptance (RA)	***16***	8	5	128	80
Total (Weighted) Score				**539**	**389**

Numerical score interpretations:

1 – Low
10 -- High

Where there were doubts and discussion (and at times changed opinions) on disadvantages of membrane filtration for groundwater, the effects of such changes in scoring on the overall scoring were tested. In no case did any of these “sensitivity analyses” change the overall conclusion of the evaluation.

3.5 Selected Filtration Alternative

Based on the evaluation matrix shown in Table 3.8, granular media filtration is the preferred alternative for treatment of the deep aquifer groundwater of this project. Better and more reliable performance, lower net present value, more confident regulatory acceptance, and lower capital cost were the major drivers in the selection of greensand media filtration as the preferred treatment technology. Filtration design objectives are presented as part of Section 2; Section 4 and the design data table drawing show filtration operating and design parameters.

Section 4: Water Treatment Plant Process Design Elements

4.1 Water Treatment Plant Basis of Design

Section 4 documents the basis for preliminary design of the treatment and mechanical processes at the proposed water treatment plant.

4.1.1 Water System Demand Projections

Water system demand projections were prepared in the Water System Plan for the Longview-Kelso Urban Area (Water System Plan) by Kennedy/Jenks Consultants in October 2005. The Water System Plan projects future water demands through 2025. Based on those projections, the water demand within the service area for the Longview RWTP will be 7.91 million gallons per day (MGD) average daily demand and 16.61 MGD maximum daily demand in 2025.

The Water System Plan water demand estimates were updated to project water demand through 2059 using a similar methodology. The updated estimates are provided in Appendix D, and a summary of the updated water demand projections for the service area are provided in Table 4.1. It should be noted that these updated projections are preliminary estimates only, and are subject to approval by the DOH.

Table 4.1: Water System Demand Projections

Year	Average Day Demand (ADD) (MGD)	Maximum Day Demand (MDD) (MGD)
2025	7.91[(a)]	16.61[(a)]
2029	8.31[(b)]	17.43[(b)]
2059	12.05[(b)]	25.29[(b)]

Notes:

(a) Based on Water System Plan (Kennedy/Jenks Consultants 2005).
(b) Preliminary projections based on methodology similar to Water System Plan.

4.1.2 Design Criteria Summary

Based on the Water System Plan and updated projections, the Longview RWTP should provide a maximum day firm production capacity of at least 16.6 MGD for projected maximum day water demands in 2025. The City also requested that consideration be given to future plant expansions to provide for the projected water demands in 2059. The proposed plant capacity is summarized in Table 4.2.

Table 4.2: Proposed Capacity for Longview RWTP

Proposed Total Capacity (24-hour operation)	22.8 MGD
Proposed Firm[(a)] Capacity (24-hour operation)	17 MGD
Proposed Firm Capacity (12-hour/day operation)	8.5 MGD

Note:

(a) Firm capacity is the capacity with the largest mechanical unit out of service.

As seen in the preliminary drawings, space is allocated for the future construction and installation of equipment, processes, and structures as required for expansion to a firm capacity of 25 MGD. It is not anticipated that the Longview RWTP will be staffed and operating 24 hours each day; therefore, the proposed capacity for the plant will allow for intermittent plant operation for ADDs.

Water quality objectives for the treatment processes are described in Section 2.2, and the design basis for the treatment processes and equipment are discussed in the sections below.

4.2 Greensand Filters

Design criteria for the filtration system and air scour blowers are summarized on Drawing G-7 in Appendix E.

4.2.1 Filter Design Summary

The details of the filters are provided in the design criteria table presented in Section 4.9. In brief, a total of nine filter vessels are provided in order to achieve the design filtration capacity. The media includes anthracite and greensand (or greensand plus) to provide adequate solids storage capacity and a reasonable head loss accumulation rate.

Sufficient filters are provided so that the design hydraulic loading rate of 4 gpm/sf will be maintained with one filter out of service (in backwash) at the design capacity.

The filters are horizontal pressure vessels, 12 feet in diameter and 34 feet long. The nominal surface area in each filter is 408 sf (not including the dished ends of the vessel).

4.2.2 Air Scour Blowers

Air scour will be provided to help clean the media by increasing abrasion among media particles and removal of metal oxides from the media during backwashing. The design air scour rate is 2 scfm/sf. Two blowers are provided, each with a capacity of 1,000 scfm. It is anticipated that the filters will be backwashed one at a time, so the blowers will operate in a duty/standby arrangement.

4.3 Oxidation Alternatives

4.3.1 Constituents Exerting Oxidant Demand

Dissolved iron and manganese are normally present in the ferrous iron (Fe^{2+}) and manganous manganese (Mn^{2+}) states. In the presence of an oxidizing agent, these ions are oxidized to the ferric (Fe^{3+}) and manganic (Mn^{4+}) states which form insoluble precipitates that can be removed by filtration.

Oxidation of iron and manganese is dependent upon the following factors:

- Oxidant concentration
- Temperature
- pH
- Contact time

- Organic material such as humic, tannic, and fulvic acids
- Silicates.

As the oxidant concentration is increased, the driving force to complete the oxidation-reduction reaction is increased. This in turn decreases the required contact time. Hydrogen sulfide (H_2S) and ammonia (NH_3) exert oxidant demands that compete with the oxidation of iron and manganese. Ammonia exerts an oxidant demand on chlorine by combining with chlorine to form chloramines; whereas hydrogen sulfide exerts an oxidant demand on chlorine, potassium permanganate, and ozone. The hydrogen sulfide and ammonia demands must be satisfied before the iron and manganese can be oxidized. Higher temperatures tend to increase the reaction kinetics, increasing the iron and manganese oxidation rates. However, temperature is not as important as pH.

Iron and manganese are normally oxidized much more rapidly in solutions with elevated pH. As a general rule, iron and manganese oxidation rates increase approximately 100 fold for each unit of pH increase. For municipal water supplies, the optimum pH for iron and manganese oxidation is frequently in the range of 8.0 to 8.5.

When organic material is present in water sources, organically complexed iron and manganese may be present. Organically bound metals are more difficult to oxidize and remove from the water. Significant quantities of organic materials were not present in the groundwater samples taken from the site.

Silicates are used in water treatment to sequester iron and manganese to prevent the oxidized particles from agglomerating and forming visible precipitates. Silicates occur naturally in water supplies in a number of forms including soluble, amorphous, and particulate. Naturally occurring silicate has the potential to form complexes with iron and manganese, making them more difficult to remove from the water supply.

4.3.2 Alternative Oxidation Chemicals

Alternatives oxidants are described in this section.

4.3.2.1 Aeration

Aeration has been used for many years as a means of oxidizing iron prior to filtration. However, aeration takes much too long to oxidize manganese to be the primary means of removing manganese in a water treatment plant. Aeration performs several functions, which include:

- Providing oxygen, which serves as the oxidant
- Stripping carbon dioxide from solution, which raising the pH, increasing the oxidation reaction kinetics
- Stripping hydrogen sulfide from the water, which reduces the chlorine demand
- Stripping volatile organic compounds (VOCs) and radon from the water (if present).

Due to the space and facility requirements for aeration, detention, and repumping, it is not considered further for this project.

4.3.2.2 Chlorine

Chlorine is used to oxidize both iron and manganese, as shown in the following reactions:

$2Fe^{2+} + Cl_2 + 6H_2O \rightarrow 2Fe(OH)_{3(s)} + 2Cl + 6H^+$
$Mn^{2+} + Cl_2 + 2H_2O \rightarrow MnO_{2(s)} + 2Cl + 4H^+$

Chlorine is a strong oxidant that is significantly less expensive than potassium permanganate. As a general rule, chlorine should be fed at a dose to meet the oxidant demand and provide a free chlorine residual of at least 0.5 mg/L. The excess free chlorine residual helps drive the oxidation reaction to completion.

At pH 7 and higher, the oxidation of ferrous iron by chlorine is rapid if the iron is not complexed. Oxidation of manganous manganese by chlorine is quite slow at the raw water pH of 7.3. Raising the pH to 8.0 or higher significantly increases the reaction rate. However, if it is partially oxidized upstream of granular media filters, it readily attaches to the media surface. The continuous presence of a chlorine residual facilitates complete oxidation of the manganese, while it is attached to the media.

4.3.2.2.1 Chlorine Gas

Chlorine gas has a long record of historical use for oxidation and disinfection of drinking water. It is currently used at the City's RWTP. Its main advantages are low operating cost and proven record of performance.

The main drawback to chlorine gas is numerous safety precautions must be taken to avoid gas leaks and EPA has specific risk management requirements related to gaseous chlorine. The federal government is also currently considering regulations that may further limit the use of chlorine gas at municipal water treatment facilities. The main driver for the new legislation is to limit chlorine gas use due to its inherent security risk when the resident population is nearby a water treatment plant using chlorine gas.

4.3.2.2.2 Delivered Hypochlorite

Delivered hypochlorite is considered much safer than chlorine gas and is subject to fewer regulations. Another advantage is the chemical feed system is simple to operate and inexpensive. A disadvantage is high strength solutions are more cost effective to ship. In addition, these solutions have a limited shelf life. Storage requirements are often tied to supply availability and in some cases the product strength significantly degrades prior to use. The chlorine feed systems can be designed so that the bulk solution is used within a reasonable time to preserve the solution strength.

4.3.2.2.3 Onsite Generation of Hypochlorite

Onsite generation of hypochlorite has gained significant popularity in the United States over the past several years. Typically, the generators produce a 0.8 % solution in a two-stage process, which converts salt, water, and power (electricity) into sodium hypochlorite and hydrogen gas.

Newer onsite hypochlorite generation equipment uses a membrane-separated cell. The membrane in the electrolyzer separates the two chemical reactions into two distinct steps, producing three potential products:

1. Chlorine gas

2. 15 % caustic soda, which can be used for other process needs

3. 12 % hypochlorite solution (by mixing the chlorine gas and caustic) which is then diluted to 0.8 % solution

The main benefits of onsite hypochlorite generation are it is safer because large quantities of high strength hypochlorite or chlorine gas storage are not required, the hypochlorite is produced as needed, and it offers protection against potential rapid increases in future chlorine costs. However, onsite generation is more capital intensive compared to bulk delivery of hypochlorite.

4.3.2.3 Potassium Permanganate

Potassium permanganate is used to oxidize iron and manganese as shown in the following reactions:

$3Fe^{2+} + KMnO_4 + 7H_2O \rightarrow 3Fe(OH)_{3(s)} + MnO_{2(s)} + K^+ + 5H^+$
$3Mn^{2+} + 2KMnO_4 + 2H_2O \rightarrow 5MnO_{2(s)} + 2K^+ + 4H^+$

Potassium permanganate is a stronger oxidant than chlorine. It traditionally has been used for manganese oxidation due to its faster reaction kinetics, compared to chlorine. Potassium permanganate has the advantage of not producing disinfection by-products such as TTHMs. However, potassium permanganate costs several times more than chlorine. In addition, if potassium permanganate is overfed, the treated water turns pink, which is aesthetically objectionable and can cause accelerated pitting corrosion in copper pipe.

In many cases, the actual potassium permanganate dosage used is less than the stoichiometric requirement, because the dissolved manganese is adsorbed onto manganese dioxide particles on the filter media. The adsorbed manganese is oxidized by the catalytic effect of the manganese dioxide as shown below, reducing the potassium permanganate requirement.

$Mn^{2+} + MnO_{2(s)} + H_2O \rightarrow MN_2O_{3(s)} + 2H^+$

4.3.2.4 Ozone

Ozone has been used on occasion to oxidize iron and manganese. Ozone is a more powerful oxidant than either chlorine or potassium permanganate; however, the capital and maintenance costs are much higher for ozone. Ozone has been used as the primary oxidant when the iron and manganese are extensively complexed with organic material and other oxidants are not effective. Although organics are present in the raw water, complexation did not appear to be problematic during the pilot study (see Section 2.4). The high capital cost to construct an ozone facility would be prohibitive for the City and, as such, ozone is not considered further for this project.

4.3.3 Oxidation Alternative Recommendation

Based upon the successful use of chlorine during the pilot study and the higher cost of other alternatives, chlorine is recommended as the sole oxidant for the City. Given the proximity of the proposed plant location to residential areas and nearby occupied industrial facilities, chlorine gas is not recommended. Onsite generation has many benefits, although given its high capital cost compared to bulk delivery, it is not recommended. Bulk delivery of hypochlorite bleach solutions is recommended as the preferred form of chlorine for the City.

4.3.4 Hypochlorite Storage and Metering System Design Summary

Engineering guidance suggests that storage of hypochlorite be limited to a maximum of about 30 days to minimize the amount chemical degradation over time. Two 3,650-gallon storage tanks will be provided and hold a 13-day supply based on the maximum daily usage of 558 gallons/day and up to a 36-day supply based on the average daily usage of 200 gallons/day. The City may opt to only partially fill the tanks and reduce the stored hypochlorite supply below 30 days during low demand periods.

The hypochlorite tanks will be located outside to reduce building costs. Because hypochlorite can crystallize at low temperatures and because the half-life of hypochlorite is substantially reduced at high temperatures, the tanks will be insulated, heat-traced, and shaded from the sun with a sloped roof. A secondary containment wall surrounding the tanks is also required to contain chemical spills. A sump will be integrated into the containment area, and a temporary pump can be used to pump out chemical spills or any water that collects inside the containment area.

Two diaphragm metering pumps (one as standby) will be used to pump hypochlorite from the storage tanks to the dosing point upstream of the filters. The diaphragm pumps will be located inside the Office/Treatment building to facilitate operator oversight and maintenance. In order to provide flow proportional feed, the metering pump stroke length and motor speed are paced with the treatment plant flow rate.

Design criteria for the proposed oxidation system are summarized on Drawing G-7 in Appendix E.

4.4 Fluoridation

Fluoride is frequently present in groundwater in concentrations ranging from 0.1 mg/L to more than 12 mg/L. According to the Center for Disease Control (CDC), the optimum fluoride concentration in potable water for preventing tooth decay is considered to a range from 0.7 mg/L to 1.2 mg/L. Higher concentrations of fluoride can be detrimental or even toxic.

At fluoride concentrations around 2 mg/L, mottling of the teeth (dental fluorosis) is likely to occur. For this reason, the EPA has set a non-enforceable SMCL for fluoride at 2.0 mg/L. The primary maximum contaminant level (MCL) is set at 4.0 mg/L to provide an adequate safety factor against the occurrence of crippling skeletal fluorosis and other chronic adverse health effects. Large doses of fluoride are toxic. The minimum lethal fluoride dose for humans is approximately 2 grams (gm).

Preliminary raw water quality data from a 5-week pilot study indicates that background fluoride concentration in the raw source water from the Prudential Boulevard Well is approximately 0.2 mg/L. DOH requires that the fluoride level be maintained between 0.8 and 1.3 mg/L by Chapter 246-290-460 Washington Administrative Code (WAC 246-290-460) when the water supply is fluoridated. In some cases, the fluoride concentration is varied seasonally because water consumption tends to vary with the air temperature.

Fluorosilicic acid (H_2SiF_6) and sodium fluoride (NaF) are the most commonly used chemicals for fluoridation of water supplies. Other fluoride compounds have been used for fluoridation in limited cases; however, they will not be considered in this evaluation.

DOH requires that drinking water additives be certified as complying with American National Standard Institute/National Sanitation Foundation (ANSI/NSF) 60 Drinking Water Treatment

Chemicals – Health Effects. Either the NSF or Underwriters Laboratories (UL) will provide certification of additives to ANSI/NSF 60.

4.4.1 Sodium Fluoride

Sodium fluoride is available in a white powder or crystal form, although it is occasionally tinted blue for identification. Typically, sodium fluoride is available at 97 to 99 % purity, which is approximately 44 % fluoride ion. Sodium fluoride has a molecular weight of 42.0 and specific gravity of 2.79. At typical water temperatures, it has a solubility of 4.03 gm/100 ml (18,200 mg/L of fluoride). Sodium fluoride is covered by AWWA Standard B701.

The pH of a saturated solution is 7.6 and, therefore, has a minimal effect on a water's pH and alkalinity. Sodium fluoride is less corrosive than fluorosilicic acid; however, glass exposed to sodium fluoride dust will etch.

Sodium fluoride dust is toxic, both with respect to respiration and skin contact. The crystalline form is preferred for manual handling, as it produces much less dust than the powder. The maximum allowable concentration of fluoride dust in the air is 2.5 mg per cubic meter (m^3) of air. When handling sodium fluoride, workers should wear a National Institute for Occupational Safety and Health (NIOSH) approved respirator with a rubber face to mask seal and replaceable cartridges. Gauntlet length neoprene gloves and aprons are also necessary.

Under Article 80 of the Uniform Fire Code (UFC), sodium fluoride is classified as an irritant and a toxic. Secondary containment, mechanical exhaust ventilation with standby power, and a fire-extinguishing system would be required.

Sodium fluoride bags must be handled carefully. When opening the bags, they should be cut rather than torn to minimize dust. The contents of the bags should be poured slowly and not bellowed to empty the bag. Empty bags should not be burned but should be rinsed and disposed of properly.

Both the powder and crystal form are stocked locally. The crystalline form is available in 50-pound bags, 40 bags per pallet. Assuming a background fluoride concentration in the water of 0.2 mg/L and a target fluoride concentration of 1.0 mg/L, the City would require 45,600 pounds (lbs) of sodium fluoride to provide for 55.4 lbs/day of average fluoride usage annually. This equivalent to 913 50-pound bags or 23 pallets annually.

Either dry feeders or saturators can be used for a dry, crystalline, or powder form of sodium fluoride. Dry feeders are considerably more complex and expensive than either direct solution feed or saturators, and are not considered in this evaluation.

In a saturator, water flowing at a slow rate through a bed of sodium fluoride will form a nearly constant 4 percent strength saturated solution for normal water temperatures. Therefore, a precise control over the amount of dry, bulk sodium fluoride added to generate the solution is not needed. If the make-up water has a hardness of 50 mg/L or more, it should be softened to prevent fouling and plugging of the inlet to the saturator, pumps and injectors with calcium fluoride (CaF_2) and magnesium fluoride (MgF_2).

Commercially available 50-gallon saturators utilize polyethylene tanks. These saturators hold 300 lbs of sodium fluoride crystals and require refilling when the sodium fluoride levels drops to 100 lbs. Either a larger custom saturator or multiple saturators will be needed, given that the water treatment plant will require 125 lbs of sodium fluoride on an average day or 262 lbs on a peak day. The saturators are manually filled by carefully slitting the sodium fluoride sacks and

pouring contents into the saturator. A dust collector, ventilation hood, and exhaust fan are desirable to reduce the fluoride dust in the air.

A level switch and a solenoid valve are used to control the solution level in the saturator. An anti-siphon valve is located on the make-up water line, as this is a cross-connection. A water meter is commonly installed on the make-up water line to indicate the amount of fluoride solution fed. A metering pump is used to pump the sodium fluoride solution to dosing points.

4.4.2 Fluorosilicic Acid

Fluorosilicic acid (H_2SiF_6) is commonly known as hydrofluosilicic acid. It is also known as fluosilicic acid, silicofluoric acid, and hexafluosilicic acid. Fluorosilicic acid is available in a liquid form at 20 to 35 percent strength. It is available locally at 23.5 to 25 percent strength. The 25 percent strength fluorosilicic acid corresponds to 244,000 mg/L fluoride. In a pure form, fluorosilicic acid is a colorless, transparent, corrosive liquid. Commercial fluorosilicic acid occasionally contains dissolved iron impurities causing a straw color. Fluorosilicic acid has a molecular weight of 144.1. A 23 percent solution will freeze at 4 degrees Fahrenheit (°F). Fluorosilicic acid is covered by AWWA Standard B703.

The pH of fluorosilicic acid is quite low (1.2 for a 23 percent solution). When added to poorly buffered waters, it can lower the pH and alkalinity of the water. Due to the acid's corrosivity, rubber-lined or plastic storage tanks are used. Fluorosilicic acid will irritate the skin necessitating the use of protective clothing and safety equipment such as rubber gloves, goggles/face shields, rubber apron and boots, and safety showers/eye wash stations. Secondary containment is required to contain spills, and considering the size proposed facility, the volume of fluorosilicic acid stored on site would likely require an automatic fire-extinguishing system. Fluorosilicic acid is slightly volatile and the fumes have a pungent odor which can irritate the respiratory system. The fumes tend to etch glass and corrode equipment. Continuous ventilation with a standby power supply is typically required by building codes.

Assuming a background fluoride concentration in the water of 0.2 mg/L and a target fluoride concentration of 1.0 mg/L, 9,900 gallons of 25 percent fluorosilicic acid would be required to provide 55.4 lbs/day of average fluoride usage annually.

Fluorosilicic acid feed system is relatively simple compared to a sodium fluoride saturator system from a mechanical perspective. The fluorosilicic feed system would be comprised of an acid storage tank, a transfer pump, a day tank, and two metering pumps (one standby).

4.4.3 Fluoridation Alternative Recommendation

Two chemical alternatives, fluorosilicic acid and sodium fluoride, were considered for fluoridation. Table 4.3 summarizes the typical advantages and disadvantages of each chemical.

Fluorosilicic acid is recommended for the proposed facilities because it is required less routine maintenance, is a relatively simple system, and does not have toxic dust issues associated with chemical handling. In addition, the cost for the acid is expected to be lower than sodium fluoride with bulk purchase.

Table 4.3: Fluoride Chemical Alternative Comparison

Alternative	Typical Advantages	Typical Disadvantages
Fluorosilicic Acid	Less frequent chemical handling Less routine maintenance Relatively simple mechanical feed system No requirement for water softening	Strong acid (pH 1.2) Lower pH of poorly buffered waters Protective gear for handling of acidic liquid and corrosive fumes Less expensive
Sodium Fluoride	Relatively constant solubility for normal water temperatures Neutral pH of the solution	More frequent chemical handling More routine maintenance More complex mechanical feed system Water softening required for high hardness Protective gear for toxic dust More expensive

4.4.4 Fluoridation Storage and Metering System Design Summary

The Center for Disease Control (CDC) recommends that storage be provided for a minimum of a 3-month fluoride supply due to occasional chemical shortages. A single 6,000-gallon storage tank will be provided and hold more than a 6-month supply based on the average daily usage of 27.2 gallons/day.

A single transfer pump will be provided to pump solution from the storage tank into a day tank. A magnetically-driven, non-metallic centrifugal pump will be used as the transfer pump.

The CDC recommends that no more than a 7-day supply of fluorosilicic acid be connected to the suction side of the metering pump. A 100-gallon day tank will provide 1 to 4 days of storage for the maximum and average daily usage. The day tank typically sits on a platform scale. The amount of fluoride fed is either recorded manually or with a loss-of-weight recorder.

Two diaphragm metering pumps (one as standby) will be used to pump fluorosilicic acid from the day tank to the dosing point in the treated waterline. In order to provide flow proportional feed, the metering pump stroke length and motor speed are paced with the treatment plant flow rate. An anti-siphon valve will be located on the metering pump discharge line to reduce the potential for accidentally overfeeding fluoride. The metering pump needs to be located above the day tank to avoid siphoning. A low-level cutoff should be provided to shut the metering pump off at a preset liquid level in the day tank.

Design criteria for the proposed fluoridation system are summarized on Drawing G-7 in Appendix E.

4.5 Corrosion Control and pH Adjustment

The groundwater at the Mint Farm differs in quality from the surface water that the City currently uses as a source of supply. Table 4.4 presents calculated values of the Langelier Saturation

Index and the Calcium Carbonate Precipitation Potential for the two sources of supply. The effect of pH adjustment, of the groundwater supply, from 7.3 to 7.5 using caustic soda is presented. These indices indicate that the Cowlitz River water is somewhat aggressive (i.e. tends to dissolve calcium carbonate scale). The pH-adjusted groundwater has similar values for these two indices and is expected to behave similarly with respect to calcium carbonate chemistry.

Table 4.4: Comparison of Cowlitz River and Well Water Quality

Parameter	Treated Cowlitz River Water	Untreated Groundwater (pH = 7.3)	Treated Groundwater (pH = 7.5)
Langelier Saturation Index	- 0.8	- 0.72	-0.52
Calcium Carbonate Precipitation Potential (mg/L as $CaCO_3$)	- 5.4	- 18	-10

The treated groundwater will have higher alkalinity than the finished surface water and as a consequence, the pH of the water in the distribution system will be more consistent, when using the groundwater source.

Note that the stability of existing pipe scale in the distribution system and the extent of possible scale sloughing cannot be readily predicted from the calculation of these indices. A flushing program will be recommended based upon the anticipated mixing patterns as the new groundwater supply is introduced into the distribution system. The system hydraulic model will be used to help identify areas of the distribution system that should be targeted for flushing. This flushing program is anticipated to mitigate the impact of any scale sloughing that may occur.

4.5.1 Sodium Hydroxide Storage and Metering System Design Summary

Using sodium hydroxide, a dosing rate of between 3.0 to 6.0 mg/L is required to adjust pH from 7.2 or 7.3 to 7.5. One 6,000-gallon storage tank will be provided to hold a 18-day supply based on the maximum daily usage of 329 gallons/day and up to a 48-day supply based on the average daily usage of 125 gallons/day. Due to the chemical's sensitivity to cold weather, the sodium hydroxide storage tanks will be located inside the Office/Treatment Building.

Two diaphragm metering pumps (one as standby) will be used to pump sodium hydroxide solution from the storage tanks to the dosing point upstream of the filters. In order to provide flow proportional feed, the metering pump stroke length and motor speed are paced with the treatment plant flow rate.

Design criteria for the proposed chemical system are summarized on Drawing G-7 in Appendix E.

4.6 Backwash Water Recovery Alternatives

4.6.1 Backwash Quantities

It will be necessary to periodically backwash the filters to remove solids trapped in the filters. Backwash is initiated based on filter run time, pressure loss across the filter, or turbidity breakthrough; or backwash may be manually initiated by the operator at any time.

The backwash sequence begins by closing the influent water valve and the filter effluent valve to take the filter offline. The backwash cycle includes the following operations:

- Drain the water in the filter to just above the media surface
- Air scour the media and begin a low-rate water backwash
- Continue low rate backwash to overflow
- High-rate water backwash
- Filter-to-waste.

The volume of water purged during drain-down is estimated to be 8,600 gallons per filter. During the air scour cycle, no water is wasted from the filter, as it is refilling.

Backwash rates are based on the media's specific gravity and diameter, and the water's temperature. Greensand and greensand plus require a backwash rate of 12 gpm/sf at 12°C to expand the media 40 percent. Assuming an 11-minute backwash period (3 minute low-rate, 8 minute high-rate) and a filter area of 408 sf per filter, the backwash volume generated is estimated to be 39,200 gallons per filter assuming only high-rate backwash is discharged to the backwash tank. Assuming a filter-to-waste duration of 5 minutes at 4 gpm/sf, 8,400 gallons of water would be produced during filter-to-waste. The total volume of waste wash water per filter is estimated to be approximately 56,200 gallons.

Based upon the pilot study results and assuming that no ferric chloride is fed, the design filter run length is 12 hours. If a small amount of ferric chloride is required for arsenic removal (0.25 to 0.5 mg/L as Fe), the design filter run length would be closer to 9 hours.

Wash water holding tanks are required to contain the backwash and filter-to-waste wash water. Two tanks are provided, each with sufficient volume to hold the waste wash water from five filter backwashes. The working volume of each holding tank is required to be 281,000 gallons minimum.

Alternatives for handling this backwash water, once collected, are described in the next section.

4.6.2 Backwash Water Handling

There are several alternative methods of wash water handling and disposal. One method is to discharge the wash water to infiltration basins on the plant site and allow the water to infiltrate into the ground. A second method involves discharging the entire backwash volume to the sewer system. A third method is to provide some means of removing the solids from the wash water so that the clean supernatant can be recovered and fed back to the head of the plant. For the third method, three alternative processes for removing solids from the wash water are considered: multi-stage membrane thickening, gravity settling, and geomembrane tubes.

4.6.2.1 Backwash Water Infiltration Basin Disposal System

In this alternative, the backwash water would be pumped to an initial settling basin where it would be discharged through a modulating valve or control orifice to infiltration basins. The water in the settling basin would be released at a very slow rate to minimize carryover of solids to the infiltration basis. After the filters were backwashed and the decant drained to the infiltration basins, the waste pumps would pump the sludge to the sewer.

This method has many disadvantages:

- This method does not allow for the backwash water to be recovered and therefore, is an inherently inefficient use of the groundwater resource as a drinking water source of supply.
- This alternative would require a large land area.
- There could be a local mounding of the groundwater.
- There will be a carryover of solids from the initial settling basin. Over time, these solids will cause the surface of the infiltration basins to clog, and rehabilitation earthwork will be required to restore proper performance.

For the above reasons, the use of infiltration basins was discarded as a viable alternative and was not considered further for this project.

4.6.2.2 Discharge to the Sewer

The backwash holding basin would be the same size and cost whether the wash water is collected and then disposed to the sewer or recovered. Recovering most of the water conserves the chemical conditioning and pump energy already expended, and reduces the impact on the sewer system and the Three Rivers Regional Wastewater Treatment Plant. The existing sewer limits on arsenic are 0.1 mg/L and for Total Suspended Solids (TSS) the current limit is 250 mg/L. The arsenic limit would not likely be exceeded in the waste wash water; however, the TSS limit would be exceeded. It is anticipated that discharging all of the waste wash water to the sewer would result in fees to address the impact to the Three Rivers Regional Wastewater Treatment Plant. Therefore, discharging all of the backwash water to the sewer will not be considered further for this project.

4.6.2.3 Backwash Water Recovery System

Alternatives for removing the solids from the backwash water and recovering that water by recycling it to the head of the plant are presented below.

4.6.2.3.1 Multi-Stage Membrane Thickening

Multi-stage membrane thickening offers the benefit of efficient removal of essentially all of the particles from the wash water and consistent permeate water quality for recycling to the head of the plant. However, the membrane systems would be much more mechanically and electronically complex than the main treatment processes, capital costs would be very high, and operational expenses would be elevated due to the need for chemical cleaning, disposal of spent cleaning chemicals, greater operator attention, and membrane replacement costs. Because this alternative is complex and costly it is not considered further for this project.

4.6.2.3.2 Gravity Settling

Using this alternative, the backwash water would be stored in an above- or below-ground holding tank. The backwash water would be captured, and approximately 90 to 95 percent of the backwash water would be recycled to the main treatment process after settling. Each wash water holding tank would serve as both a holding tank and settling basin. The settled sludge will be collected as described in Section 4.7.

All tank alternatives would be configured in a similar manner. The main differences between the alternatives are the tank construction method, the tank's depth, and surface area. The floor over the backwash storage tank would be a flat slab with support beams across the width of the tank and columns at regular intervals to reduce the moment in the floor support beams. A small backwash recycle chamber would be constructed in a corner of the tank. Two or three sluice gates would be installed in one wall of the backwash recycle chamber. The sluice gates would be used to decant the settled water from the backwash holding tank into the backwash recycle chamber. Vertical turbine backwash return pumps would recycle decanted water back through the filters.

Two submersible, non-clogging waste pumps (one as standby) would be used to pump the settled sludge as described in Section 4.7. Instrumentation inside the backwash holding tank and recycle chambers would include level transmitters and redundant level switches to stop and start the backwash return and waste sludge pumps.

When the plant is operating at full capacity, the operation of each holding tank would be as follows:

Filling time = 6 hours
Settling time = 2 hours
Decant time = 3.5 hours
Idle time prior to initiating filling = 0.5 hours

When the plant is operated at less than its capacity, the filling and decanting times will increase proportionately. Two tanks are provided, such that, while one tank is filling the other tank is in settling/decanting mode. The cumulative backwash recycle is not expected to exceed 10 percent of the total influent flow at full capacity.

4.6.2.3.3 Aboveground Tank

An above-ground tilt-up concrete, steel, or cast-in-place concrete tank could be used to store the backwash water. This type of construction is expected to be more cost-effective at the site because a below-grade structure would require a special support structure, like piles, whereas the soil underneath an above-grade structure could be pre-loaded with imported fill (provided that the soil bearing pressure of the structure is below the threshold limits identified in the geotechnical report). To evenly distribute the weight of an above-grade tank, a mat foundation would be used.

Water from backwashing cycled would drain to the blow-off/plant drain manhole under gravity (for tank drain-down) or would flow to the above-ground backwash storage tank under pressure (for low- and high-rate backwashes).

4.6.2.3.4 Conventional Concrete Tank Constructed Below Grade

In this configuration, the tank would be located below grade so that waste wash waters could flow by gravity into the tanks. Because of the poor condition of the soils for construction, below-grade tanks would require special supports, such as piles. Such piles add significantly to the

cost of construction making below-grade construction of large tanks less cost-effective than aboveground alternatives.

4.6.3 Backwash Water Recovery System Recommendation

An aboveground tank designed to hold the waste wash water and also allows it to settle out the solids is preferred because of its simplicity, low operating cost, and low construction cost. The tank will generally be designed and operated as presented in Section 4.6.2.3.2. Further details on the specific components of the tank, materials of construction, and construction method will be developed in the design phase of this project. Design criteria for the proposed backwash storage tank is summarized on Drawing G-7 in Appendix E.

4.7 Waste Solids Handling and Dewatering Alternatives

Periodically, solids accumulated in the backwash storage tanks will need to be wasted from the tanks. At smaller water treatment plants, this backwash waste may be discharged to the sanitary sewer with minimal downstream impacts. For larger water treatment plants, backwash waste quantities have the potential to exceed the capacity of downstream sanitary collection and treatment facilities. Depending on the level of removal of certain compounds from the raw water at the treatment plant, backwash waste may also contain concentrations of regulated compounds that exceed the limits of what is allowable to discharge to a sanitary sewer.

As described previously, it is expected that TSS concentrations in the backwash waste at the proposed treatment facilities will exceed the maximum concentration of 250 mg/L for discharge to the sewer. For this reason, this preliminary design assumes that direct discharge of backwash waste to the sewer will not be feasible and solids handling and dewatering will be required.

4.7.1 Backwash Waste Solids Quantities

The majority of solids generated at the treatment plant are expected to be metal solids, from the removal of iron and manganese, and silica solids. Although it is not anticipated that ferric chloride or another coagulant will be required for removal of arsenic, the use of a coagulant would generate additional solids.

Based on the raw water quality data from the pilot testing, the raw water is expected to have total iron and manganese concentrations of 1.2 mg/L and 0.6 mg/L, respectively. Assuming complete removal of the iron and manganese and assuming that 33 percent of the solids generated will be silica solids, the proposed facilities will generate about 200 lbs of dry solids per day when producing 8.3 MGD of water (average daily demand in design year).

As seen in the drawings in Appendix E, the invert of the lower decant gate in the proposed backwash storage tanks is approximately 4 feet above the floor at the gate. The floor is sloped away from the wall that the decant gate is mounted on, down to a backwash waste pump on the opposite side of the tank. This volume between the invert of the gate and the floor of the tank is intended for storage of backwash waste within the tank. Storage of backwash waste within the backwash tank will allow the waste solids to consolidate and thicken, which may reduce backwash waste volume when waste solids are pumped out of the tanks.

Preliminarily, the backwash waste is expected to thicken to a concentration of 0.5 percent to 1 percent solids in the backwash storage tanks. When the treatment plant is producing 8.3 MGD of water, the backwash waste solids volume is expected to be approximately 33,000 to 67,000 gallons if backwash waste solids are pumped from the tanks every two weeks.

4.7.2 Waste Solids Handling and Dewatering Alternatives

Three alternatives were considered for waste solids handling and dewatering:

- Thickening Lagoons/Solids Drying Beds – discharge waste solids to a lined basin. Periodically decant water and then allow time for solids to dry by evaporation.

- Mechanical Dewatering – discharge waste solids to a waste solids holding tank. Add polymer and dewater solids using a centrifuge, screw press, plate-and-frame press, belt filter press, or another mechanical device.

- Geotube™ Dewatering – Add polymer and discharge waste solids to a geotextile dewatering bag in a lined basin.

Each alternative is briefly described in the following sections.

4.7.2.1 Thickening Lagoons/Drying Beds

If land is readily available and inexpensive and the climate conducive to air drying, the use of sludge lagoons/drying beds can be a cost-effective means of thickening and drying waste solids. Lagoons can be earthen, asphalt, or gunnite lined basins equipped with inlet control devices and overflow structures. Wastes with settleable solids are discharged into the lagoons, where the solids are separated by gravity sedimentation. Water is periodically decanted off the surface of the thickening lagoon to aid in consolidation and drying of the solids. Decanted water would be discharged to the sewer or to the plant drain manhole.

A common approach used at many water treatment plants is to use lagoons not only as thickeners (with continuous decanting), but also as drying beds after a predetermined filling period. Three months of filling and an average drying cycle time of 3 months is a common design parameter that is used. A ramp is typically provided into the lagoon/drying bed to remove dried solids with a front-end or skid-steer loader. Dried solids would be hauled to the landfill. Solids drying beds are capable of dewatering the solids to 25 percent to 40 percent, depending on drying conditions. Higher solids content would result in lower removal and disposal costs. Of all of the evaluated alternatives, sludge lagoons/drying beds are the least mechanically complex. Operator oversight of this process is typically limited to adjustment of the decanting mechanism and removal of solids. Performance of this alternative is susceptible to weather conditions, and covers may be considered if this alternative is selected for detailed design.

In wet regions such as the Pacific Northwest, a maximum sludge loading rate of 8 lbs/ft^2 per filling cycle is typically used to size the lagoons/drying beds. The actual footprint of the lagoon is usually at least 1.5 times the area computed, because of the additional area required for berms, access roads, and ramps. Applying these conceptual design criteria at the proposed water treatment plant, one lagoon with a conservative 6-month filling/6-month drying cycle would require approximately 6,750 ft^2 of area. A minimum of three cells are typically provided, allowing for one lagoon to be drying, one lagoon to be filling, and one lagoon to be idle. Assuming three cells would be provided, the total required area for this alternative would be approximately 20,250 ft^2. Increasing the number of cells may increase process flexibility considering the limited drying season, but increasing the number of cells would also increase the amount of space required. If this alternative is selected for detailed design, increasing the number of cells may be considered in greater detail.

There appears to be space for creation of thickening lagoons/solids drying beds across the road from the Office/Treatment Building to the southeast.

4.7.2.2 Mechanical Dewatering and Solids Storage Tank

There are several mechanical dewatering technologies available, including plate-and-frame filter press, belt filter press, screw press, centrifuge, vacuum filters, and others. As compared to the other evaluated alternatives, mechanical dewatering is a relatively expensive alternative and is typically applied at larger treatment plants or plants with space constraints.

For application at the proposed Longview RWTP, the mechanical dewatering process would likely require a separate solids storage tank, prior to the dewatering process, to serve as flow equalization and an additional thickening step. To achieve an "optimal" solids concentration, polymer is typically added to the sludge, which increases costs associated with operations of a mechanical process. A mechanical dewatering alternative would likely require a greater amount of operator oversight, resulting in additional operating labor costs. Due to the increased amount of operator oversight required, mechanical dewatering processes are typically located in a building, resulting in additional building cost. The solids handling building could be located across the street from the Office/Treatment Building to the southeast.

Filtrate from the mechanical dewatering process would be collected in a sump inside the building and drained to the sewer or plant drain manhole. Solids from mechanical dewatering would be conveyed into a wheeled dumpster or directly into a dump truck. Dewatered solids concentration can vary depending on the type of technology used and the properties of the feed solids – ranging from 12 percent to 25 percent cake solids for centrifuges to upwards of 40 percent cake solids for plate-and-frame filter presses.

4.7.2.3 Geotube™ Dewatering

Geotubes™ are formed from a woven geotextile that has been stitched together in the shape of a large bag. The woven geotextile material has a nominal pore size of approximately 250 microns, allowing the fabric to pass water and strain solids. The tube is equipped with one or more filling ports, with the number of ports depending on the size of the bag. A flexible hose would be provided to connect the waste solids discharge to the fill port, and a strap is used to hold the filling nozzle in place.

Before waste solids are pumped into the tube, polymer is added to bind the solids together and aid in filtration. Filtered water drains from the Geotube™ container and solids remain inside the bag. High solids capture efficiency is possible if the polymer system is appropriately designed and operated.

Once the tube has reached its solids storage capacity, the tube is typically left to sit idle for a period of several months – allowing for further solids consolidation and more dewatering. Once the solids inside the bag have reached the desired dewatered solids concentration, the bag is cut open and the solids are removed with a front-end loader and hauled to the dump along with the spent tube.

Case studies have indicated that other water treatment plants utilizing this technology for dewatering alum sludge have achieved dewatered solids concentrations of up to 40 percent. However, based on conversations with other facilities using this technology in the Pacific Northwest, a dewatered solids concentration of 15 percent to 25 percent may be more typical at Longview.

This alternative is expected to be the most cost-effective of the alternatives evaluated, as the Geotubes™ are relatively inexpensive and they require significantly less space and concrete than thickening lagoons/drying beds. Operator oversight of this alternative mostly involves management of the polymer system, which is required to achieve high solids capture efficiency and good dewatering. Performance of this alternative is expected to be less sensitive to rain and

inclement weather than the thickening lagoons/drying beds, because the Geotube™ should not absorb water when it has been filled.

Of all the options available, the Geotube™ is the least proven alternative because there is limited experience with this dewatering method at other western United States groundwater treatment plants that remove iron and manganese. Geotube™ technology has been successfully applied in dewatering of metal solids generated during the reclamation of mining wastewater and also in the dewatering of alum sludges. Locally, the City of Everett and the City of Grants Pass utilize Geotubes™ at their surface water treatment plants. Because solids generated at surface water treatment plants typically consist of more silt and sand than metal solids, Geotube™ performance at these plants cannot be assumed for the proposed treatment facilities. If this alternative is selected for detailed design, further investigation will be conducted to determine the feasibility of this technology with metal solids and pilot testing may be recommended.

4.7.3 Waste Solids Handling and Dewatering Recommendation

Three waste solids handling and dewatering alternatives were considered – sludge lagoons and solids drying beds, mechanical dewatering, and Geotube™ dewatering. Table 4.5 summarizes the typical advantages and disadvantages of each alternative.

Table 4.5: Waste Solids Handling and Dewatering Alternative Comparison

Alternative	Typical Advantages	Typical Disadvantages
Thickening Lagoons and Solids Drying Beds	Low capital investment Lowest O&M costs Mechanically simple Least operations oversight Proven technology	Largest footprint Performance in wet climate may not be ideal and covers may be required
Mechanical Dewatering	Performance in wet climate not anticipated to be an issue Smallest footprint Proven technology	High capital investment Highest O&M costs Most operations oversight Building space is required Requires careful management of polymer system
Geotube™ Dewatering	Least capital investment Low O&M Costs Mechanically simple Smaller footprint Performance in wet climate not anticipated to be an issue Minimal operations oversight	Requires careful management of polymer system Technology is less proven in this specific application

Geotubes™ are recommended for consideration because the substantial cost savings offered by this technology warrant further investigation in detailed design. Additionally, this alternative is expected to perform better in wet climates than thickening lagoons and solids drying beds. If it is determined in detailed design that this alternative is not appropriate to carry forward, thickening lagoons and solids drying beds will be recommended.

4.7.4 Waste Solids Handling and Dewatering Design Summary

Three Geotubes™ are proposed, with each tube having a storage capacity of approximately 160 tons or 5,500 cubic feet of solids. Each tube would measure approximately 23 feet across by 57 feet long. At a dry solids production rate of 200 lbs/day, each tube would hold about 240 days of 15 percent dewatered solids.

The tubes will be located on an asphalt pad that is sloped to a trench drain, and the trench drain will drain the blow-off/drain manhole or to the sewer. A rolled curb around the outside of the pad will help contain the water that drains from the tube, as well as rainwater.

4.8 Major Pumping Systems

The following summary lists the major pumping systems and provides a brief description of each.

- Well Pumps (4) – The well pumps will pump through the treatment plant to the distribution system. It appears that the site will be able to accommodate two additional well pumps in the future.

- Backwash Return Pumps (2) – The backwash return pumps will pump decanted water from the backwash storage tanks into the filter influent header.

- Backwash Waste Pumps (2) – The backwash waste pumps will pump waste solids out of the backwash storage tanks to the dewatering process.

- Blow-Off Manhole/Plant Drain Pumps (2) – The blow-off/plant drain pumps will pump blow-off from the wells, miscellaneous plant drainage, and dewatering filtrate to the sewer.

Design criteria for the proposed major pumping systems are summarized on Drawing G-7 in Appendix E, and described in greater detail below.

4.8.1 Well Pumps

One existing water supply well with a production rate of 3,800 gpm and three new water supply wells with a production rate of 4,000 gpm/each will provide 22.8 MGD of total pumping capacity and 17 MGD of firm pumping capacity. Raw water from the wells will be pumped through a 30-inch raw water main to the filters, through the water treatment plant, and into a 30-inch treated water main to distribution system.

Based on hydraulic modeling of the distribution system, pressures are anticipated to be approximately 90 to 110 pounds per square inch gauge (psig) at the connection to the Weber Avenue 12-inch water main. Preliminary well testing data suggests that the well water level is close to the ground surface and there will be very little drawdown. Based on these preliminary findings, the total dynamic head (TDH) required for each pump would be approximately 300 feet.

Three pumps types were initially considered for the well pumps: split case pumps, submersible pumps, and vertical line shaft turbine pumps. Split case pumps were screened from further consideration because of potential priming issues. Although the well water level is near the ground surface, a split case pump mounted at grade will not always have a flooded suction. Although an automatic priming system could be utilized, priming systems are not always failsafe and they add an element of complexity that the other alternatives avoid. Typical advantages and disadvantages for the other two pump types are summarized in Table 4.6 below.

Table 4.6: Well Pump Alternative Comparison

Alternative	Typical Advantages	Typical Disadvantages
Submersible Well Pump	No well building	Maintenance of motor is difficult and more expensive Higher pump cost Lower efficiency
Vertical Turbine Pump	Lower pump cost Higher efficiency (than submersible) Ease of motor maintenance Bowls are adjustable from surface to improve pump efficiency over time	Requires well building

For this application, vertical turbine pumps are recommended because this pump type typically offers higher pumping efficiency and features that facilitate maintenance. Based on other clients experience and conversations with pump suppliers, vertical turbine pumps are also preferred because submersible pumps of this size might be a maintenance issue in the future. The additional cost of the above-grade well buildings is likely to be partially offset by the savings in pump cost.

A vertical line shaft turbine pump, powered by a 450 horsepower (hp) motor, is recommended and will deliver 3,800 to 4,000 gpm at a TDH of 300 feet with an operating efficiency of approximately 80 percent.

4.8.2 Backwash Return Pumps

The filter backwash and filter-to-waste water is discharged to a backwash recovery basin where the solids will be allowed to settle. After the solids have settled, the decanted backwash water will be sent to the filter influent line and recycled through the pressure filters. Typically the backwash recycle rate is 5 to 10 percent of the raw water feed rate for a groundwater treatment plant.

It is anticipated that one backwash cycle will backwash up to six filters into one backwash recovery basin, and that one backwash cycle may generate about 340,000 gallons of water. Assuming that 3.5 hours are allowed for decanting the backwash tank, backwash recycle will be returned to the filter influent piping at a rate of approximately 1,650 gpm.

A vertical turbine pump, powered by 150 hp motor, is proposed to recycle the decanted backwash water at a rate of 1,350 gpm at a TDH of 300 feet with an operating efficiency of approximately 80 percent. Two backwash return pumps are proposed to operate as duty/standby pumps. The pumps will be located in the filter gallery building, and will be connected to the backwash storage tanks via underground piping. The cans for the pumps will extend below the floor of the filter gallery building.

4.8.3 Backwash Waste Pumps

Settled solids will remain in the backwash storage tanks until the tank is cleaned or until solids are pumped out using the backwash waste pumps. The backwash waste pumps will pump solids from the backwash basins to the Geotube™ dewatering system.

A submersible pump is proposed for the backwash waste pumps. The pumps will be connected to a rail system on the side of the backwash storage tanks, and will be retrievable from the top of tanks using a davit crane. One 300 gpm pump operating at 20 feet TDH will be provided in each tank. A standby backwash waste pump should be shelved onsite in the event that a duty pump fails or needs to be removed from service.

4.8.4 Blow-Off Manhole/Plant Drain Pumps

A blow-off/plant drain manhole will be provided to receive water from the well house blow-off manholes, drain down from the pressure filters, and filtrate from the Geotubes™. Two submersible pumps will be placed in the manhole and will pump the water to the existing sewer line along Weber Avenue. The pumps will operate in a lead/lag configuration. Two 400 gpm pumps are proposed.

4.9 Preliminary Design Summary

A process flow diagram, which shows a schematic interconnection the primary components of the proposed facility, is shown on Drawing G-5 in Appendix E. The design criteria for the proposed facility is summarizes on Drawing G-7 in Appendix E.

4.10 Hydraulic Profile

Hydraulic profiles are developed to assist designers with setting structure elevations and identifying hydraulic bottlenecks, and are intended to reduce the possibility of structures overflowing in systems with gravity flow. A hydraulic profile is not needed for this project and was not developed because all flow is pumped through the facility.

Section 5: Building Design Elements

This section presents a preliminary building layout and identifies key criteria used in the design of the buildings.

5.1 Introduction

The following summary lists the primary buildings and structures to be located on site and provides a brief description of each.

- Well Houses (4) – Each well house will contain electrical equipment for its associated well.
- Filter Gallery Building – The filter gallery building will contain the face piping and valves for the greensand filters, air scour blowers, backwash return pumps, and associated controls.
- Backwash Storage Tanks (2) – The backwash storage tanks are partially buried, reinforced concrete tanks that store and settled backwash from the greensand filters. The backwash waste pumps will be located inside the tanks. The tanks will be constructed with aluminum covers, and will have walkways on top of the tanks to facilitate operator access and maintenance.
- Office/Treatment Building – The office/treatment building will contain an office and meeting room, a lab, staff locker rooms, a control room, a mechanical room, an electrical room, and chemical storage areas for fluorosilicic acid, and sodium hydroxide. Storage for hypochlorite will be provided outside the building.
- Sludge Drying Beds (3) – The Geotube™ dewatering beds will receive settled solids from the backwash storage tanks. The dewatering beds will be paved with asphalt or lined with concrete.

5.2 Building Design Criteria

A preliminary Building Code Review has been performed and is summarized below.

5.2.1 Applicable Codes, Standards, and Planning Requirements

This Code Review is developed using the 2006 International Code Council (ICC) Codes with 2007 Washington State Amendments. The International Building Code (IBC) is used for all building design requirements [including referenced International Fire Code (IFC) sections], and supersedes National Fire Protection Association (NFPA) requirements in all areas. As a general rule, the NFPA requirements are used for electrical design and for items that are not addressed in the IBC.

- International Building Code (IBC), 2006
- International Fire Code (IFC), 2006
- International Plumbing Code (IPC), 2006

- International Mechanical Code (IMC), 2006
- National Electrical Code (NEC), 2005
- State of Washington Energy Code, 2007
- WAC 296-46 Electrical
- Washington State Ventilation & Indoor Air Quality, current edition
- Declaration of Protective Covenants, Conditions, Restrictions, Easements, and Agreements for Mint Farm Industrial Park, 2006.

In addition, the standards listed below may be utilized in the design of buildings or elements of these structures.

- Minimum Design Loads for Buildings and Other Structures, ASCE Standard ASCE/SEI 7-05, American Society of Civil Engineers, 2005.
- Standard Specifications for Highway Bridges, Fifteenth Edition, American Association of State Highway and Transportation Officials, Inc.
- Building Code Requirements for Reinforced Concrete (ACI 318-05) and Commentary (ACI 318R-05), American Concrete Institute.
- Manual of Steel Construction, Specification for Structural Steel Buildings, American Institute of Steel Construction, Inc., Thirteenth Edition, 9 March 2005.
- North American Specification for the Design of Cold-Formed Steel Structural Members, AISI/COS/NASPEC 2001, American Iron and Steel Institute, 2001 Edition, with 2004 revisions.
- Building Code Requirements for Masonry Structures, ACI 530, American Concrete Institute, 2002.
- American Society of Heating, Refrigerating, and Air Conditioning Engineers Standards (ASHRAE).
- Sheet Metal and Air Conditioning Contractors National Association (SMACNA).
- National Fire Protection Association (NFPA).

5.2.2 Code Review

The Well Houses and Filter Gallery Building are considered F-1 occupancy, industrial moderate hazard per IBC 306.

The Office/Treatment Building is considered mixed occupancy containing B office, F-1 industrial moderate hazard, and H-4 hazardous occupancies per IBC 304, 306, and 307.6.

The buildings will be Type II-B, non-rated construction and may be constructed of any materials approved by the IBC (concrete, masonry, or steel). Buildings will be within the maximum allowable area of 17,500 sf and maximum allowable height of 55 feet per the IBC.

If a building is more than 30 feet from any property line or building separation line it will not require fire-rated exterior wall construction. If a building is less than 30 feet but more than 10 feet away, the building will require 1-hour rated construction on that side.

Proposed physical and health hazards include:

- Sodium Hydroxide, 25 percent solution: 6,000 gallons in a single tank. CAS No. 1310-73-2. Classified by the IFC as a Corrosive with a 1,000-gallon exempted quantity in a sprinklered building.

- Hydrofluosilicic Acid, 25 percent solution: One 6,000-gallon tank and one 100-gallon day tank. CAS No. 16961-83-4. Classified by the IFC as a Corrosive with a 1,000-gallon exempted quantity in a sprinklered building.

- Sodium Hypochlorite, 12.5 percent solution: 7,000 gallons stored in two 3,500 gallon outdoor tanks. CAS No. 7681-52-9. Classified by the IFC as a Corrosive with a 2,000-gallon maximum allowable quantity per outdoor control area. No combustible materials or vegetation within 15 feet of the tanks. The outdoor storage area may only be 25 percent enclosed by walls.

- Diesel Fuel: 12,000 gallons stored in an outdoor tank; CAS No. 000126-00-0. Classified as a Combustible Liquid (Class 2). There are no limits on the allowable quantity of outdoor storage. Per the IFC, the diesel tank is required to be a minimum 15 feet from any property lines or 5 feet away from buildings. No combustible materials or vegetation within 15 feet of the tank.

Code requirements include the following:

- Spill Control and Secondary Containment: The IFC 2704.2 establishes the requirements for spill control and secondary containment for hazardous material liquids. Spill control and secondary containment is required for all hazardous chemicals with aggregate quantities over 1,000 gallons. Floors shall be liquid-tight, sloped, recessed, curbed, diked, or designed with sumps or collection systems to contain spillage. Indoor containment curbs shall be at a height to contain the maximum liquid stored plus a 20-minute sprinkler flow at 0.02 gpm/sf. Outdoor containment curbs shall be designed to include the volume of a 24-hour rainfall as determined by a 25-year storm. Individual spill containment areas should be provided for each chemical.

- Liquid level Limit Controls: Tanks containing hazardous liquids in excess of 500 gallons shall be equipped with a liquid level limit control to prevent tank overfilling.

- Chemical Separation: Hydrofluosilicic Acid is not compatible with Sodium Hydroxide. The tanks should be separated by a solid, non-combustible partition extending no less than 18 inches above and to the sides of the taller tank or by a distance of no less than 20 feet. The exterior wall separating indoor and outdoor chemical storage shall be a 2-hour fire barrier.

- Handicap Access: Required for hallways, offices, restrooms, and office support spaces. No handicap access is required in process or mechanical areas.

- Automatic Fire Protection: The B and F-1 occupancies within the building do not require a fire extinguishing system. The H-4 occupancy must have an automatic fire extinguishing system per IFC 2704.5 and NFPA 13. Unless the building is divided by a 2-hour firewall, the entire facility must be sprinkled, regardless of occupancy per

NFPA 13 (Note: There are a few exceptions to the requirement for sprinkling the entire building mentioned in the 2006 IBC. The 2006 IBC conflicts with the common interpretation of NFPA 13, thus any deviation from NFPA 13 should be discussed with the local building official and fire marshal). If a 2-hour firewall is provided between the H-4 occupancy area (chemical storage) and the rest of the Office/Treatment Building, any doors in the firewall must also be 2-hour rated.

- Fire Hydrant: Required within 250 feet of the Office/Treatment Building.
- Exits: Required exit doors shall be 36 inches wide by 80 inches high at a minimum, swinging type (roll-up doors are not legal exits). Panic hardware is required for H occupancies. Panic hardware and two exits are required for electrical rooms with equipment rated over 1,200 amps. Two exits are required for H occupancies over 3,000 sf.
- Insulation: Required in heated and air-conditioned areas.
- Fire Alarm and Smoke Detection System: Required.
- Exit Illumination: Required at 1-foot candle.
- Fire Extinguishers: Required with a maximum travel distance of 75 feet to any extinguisher.
- Emergency Eyewash/Shower Station: Required in areas containing liquids that constitute a health hazard per Occupational Safety and Health Administration (OSHA).
- Ventilation: Buildings will be ventilated both naturally and by mechanical means.

5.2.3 Structural Criteria and Geotechnical Considerations

5.2.3.1 Structural Loads

Dead Loads: Dead loads shall consist of the weight of all materials and fixed equipment incorporated into the building or other structure.

Live Loads: Live loads are those loads produced by the use and occupancy of the building or structure and do not include dead load, construction load, or environmental loads such as wind load, snow load, rain load, earthquake load or flood load. Floors shall be designed for the unit loads in the Table 5.1 below.

For additional loads not indicated in Table 5.1, the unit loads will be as set forth in ASCE 7-05, Table 4-1. Concrete floor slabs-on-grade should not be less than those given for heavy manufacturing or storage warehouse; 250 pounds per square foot (psf) uniform load and 3,000-lb concentrated load except in areas where vehicles may access and then the concentrated load should be increased to 8,000 lbs. Floor live loads in equipment rooms, pump rooms, electrical rooms, and areas where equipment may be moved to various locations should be not less than those given for light manufacturing or storage warehouse; 125 psf uniform load and 2,000-lb concentrated load. Live loads for grated and plated areas should equal or exceed the corresponding floor live load for the given area. Access hatches should equal or exceed the corresponding floor live load for the given area.

Vehicle loads shall be in accordance with the latest edition of the AASHTO Standard Specifications for Highway Bridges. There are four standard classes of highway loading: H 20, H 15, HS 20, and HS 15. Loadings H 15 and HS 15 are 75 percent of loadings H 20 and HS 20, respectively. The H loadings are for two axle trucks and the HS loadings are for a tractor truck with a semi-trailer. The maximum axle loading for an H 20 or HS 20 vehicle is 32,000 lbs and the maximum axle loading for an H 15 or HS 15 vehicle is 24,000 lbs.

Snow Loads: A ground snow load of 25 psf will be utilized for roof design. Snow loads will be adjusted for buildings in accordance with ASCE 7-05 Chapter 7.

Table 5.1: Minimum Uniformly Distributed Live Loads and Minimum Concentrated Live Loads

Occupancy or Use	Uniform (psf)	Concentrated (lbs)
Balconies (exterior)	100	
Catwalks for maintenance access	40	300
Fixed Ladders		300
Garages (passenger vehicles only)	40	3,000
Handrails, guardrails, and grab bars	50	200
File and computer rooms	100	2,000
Offices	50	2,000
Roofs (uniform load subject to reduction for area and pitch)	20	2,000
Sidewalks, vehicular driveways	250	8,000
Stairs and exit-ways	100	300/tread
Light manufacturing or storage warehouse	125	2,000
Heavy manufacturing or storage warehouse	250	3,000
Walkways and elevated platforms	60	-

Wind Loads: Buildings and treatment structures less than 60 feet in height should be designed in accordance with ASCE 7-05 Chapter 6 for wind effects based on the factors listed in Table 5.2.

Table 5.2: Wind Load Design Requirements and Criteria

Description of Coefficient	Coefficient
Exposure (flat and generally open terrain)	C
Basic Wind Speed, mph	85
Topographic Factor	$K_{ZT} = 1.0$
Wind Importance Factor	$I_W = 1.15$ (for occupancy category III)

Earthquake Loads: Buildings and treatment structures will be designed to resist the effects of earthquake ground motions in accordance with adopted building codes and national standards for non-building structures. The purpose of the earthquake provisions in building codes is primarily to safeguard against major structural failures and loss of life, not to limit damage or maintain function. The design basis ground motion utilized in the design of the buildings and treatment structures is that ground motion that has a 10 percent chance of being exceeded in 50 years. The design of new buildings and the earthquake forces shall be determined considering seismic zoning, site characteristics, occupancy, configuration, structural system, and building height. The existing buildings and new buildings are primarily bearing wall

structural systems with concrete masonry unit (CMU) shear walls utilized as the lateral force resisting system. Existing and new buildings typically have flexible roof diaphragms of either plywood or structural metal materials. Seismic load design requirements and criteria are summarized in the Table 5.3 below:

Table 5.3: Seismic Load Design Requirements and Criteria

Description of Coefficient	Coefficient
Occupancy Category	III (Water Treatment, but not fire flow)
Spectral Coefficient, S_S	0.611
Spectral Coefficient, S_1	0.288
Soil Profile	S_E
Soil Coefficient F_A	1.48
Soil Coefficient Fv	2.85
Seismic Design Coefficient, S_{DS}	0.603
Seismic Design Coefficient, S_{D1}	0.547
Seismic Design Category	D
Overstrength and Ductility Coefficient, R	5.0 (Special Reinf. CMU Shear Wall)
Seismic Amplification Factor, Ω	2.5 (Special Reinf. CMU Shear Wall)
Overstrength and Ductility Coefficient, R	3.0 (partially buried tankd

Hydraulic structures are considered special structures by the building codes and require special consideration for their response characteristics and environment that is not covered by most building codes. Earthquake loads for non-building liquid containing concrete structures should be determined utilizing ACI 350.3-06 Standard. It should be noted that the ACI standard provides results at allowable stress levels, which are lower than the seismic forces obtained from the 2006 IBC, which are at strength levels. In addition to calculating the seismic loads on rectangular and circular liquid containing concrete structures, the ACI 350.3 standard will be utilized for determining the freeboard associated with the maximum wave oscillation generated by earthquake acceleration.

Other Minimum Loads: In addition to the loads listed above, buildings and non-building structures will be designed to resist other loads including fluid pressures, hydrostatic uplift, lateral soil pressures, ponding loads, and self-straining forces, as applicable.

5.2.3.2 Structural Tests and Inspections

Structural tests and inspections shall be provided for certain types of work. The drawings and the technical specifications will provide detailed information on the quality assurance and testing and inspection requirements for different materials in the shop and in the field.

Special Inspections: Special inspections for certain materials of construction or procedures will be provided as noted on the Special Inspection and Testing Schedule on the Structural Drawings.

Structural Observation: Structural observation will be required in accordance with Chapter 17 of the 2006 IBC.

5.2.3.3 Foundations and Retaining Walls

While final geotechnical investigations have not been completed, preliminary investigations indicate several design issues which will require special mitigation. The site is underlain by at

least 100 feet of moderately loose silty sand and silt. The depth and compaction of the soil make both consolidation settlement and liquefaction settlement probable at the site.

While deep foundations were considered, it is understood from the preliminary geotechnical investigation that a pre-load on the site of around 2,000 psf for around 6 months will help to mitigate the primary consolidation and limit the liquefaction potential of the upper layers of the soil.

After a pre-load, we anticipate placing structures on a layer of compacted structural fill, layered with geotextile fabric to control differential settlement. With this site preparation we expect that consolidation differential settlement and potential liquefaction settlement to less than 1 inch of differential settlement, given a maximum load of 1,500 psf. We anticipate that the all areas under structures will be preloaded, and that either spread (for buildings) or mat (for tanks) foundations will be provided with bearing pressures not exceeding 1,500 psf.

Final recommendations will be provided shortly, including the necessary requirements for duration of preload, the required depth of the reinforced earth mat and an inset required for tanks to achieve 1,500 psf.

For all excavations and foundations, high groundwater is anticipated for much of the year. Dewatering will be required for most major excavations, but may be alleviated for short duration shallow excavations during the summer. All tanks will be designed to resist buoyant uplift based on a water surface near grade.

5.2.4 Building Mechanical (HVAC, Plumbing, Fire Protection)

5.2.4.1 HVAC

The purpose of the heating, ventilating, and air conditioning (HVAC) systems will be to provide safe and comfortable conditions for people and stable temperatures for electrical equipment and instrumentation. This section summarizes the design criteria for HVAC systems that will be provided for the Office/Treatment Building, Well Houses, and the Filter Gallery Building.

The HVAC system for each building/space will be designed in accordance with the IMC and the Energy Code. The Energy Code designates spaces as one of the following: Conditioned Space, Semi-Heated Space, or Unconditioned Space. A Conditioned Spaces is one that is either cooled space, fully heated, semi-heated, or indirectly conditioned space. A space is considered Semi-Heated if the each heating system is controlled by a thermostat with a maximum temperature setpoint of 45°F with an output capacity not exceeding 8 BTU/hr-sf. If the heating system capacity exceeds 8 BTU/hr-sf or the maximum temperature setpoint exceeds 45°F, then the space is considered Conditioned. All Conditioned Spaces must meet the prescriptive envelope requirements listed in Tables 5.4 and 5.5.

Table 5.4 summarizes the prescriptive envelope requirements for electric resistance and other fuels (including heat pump) heating systems. Wall insulation may be omitted in semi-heated spaces that are heated by means other than electric resistance heat, such as natural gas, oil and propane, in addition to heat pumps and terminal electric resistance heating used in variable air volume (VAV) systems. All other envelope insulation is required, including floor perimeter insulation and roof insulation. However, it is assumed that natural gas is not available at the facility and that all heating systems will be electric resistance or heat pump.

Table 5.4: Energy Code Prescriptive Building Envelope Requirements

	Door	Perimeter	Roof		Wall (Masonry) Metal Furring		Wall (Non-Masonry)
			Over Attic	All Other			
Heating Type	U-Factor	R-Value	R-Value	R-Value	U-Factor	R-Value	R-Value
Electric Resistance	0.6	10	38	30	0.15	11	19
Other Fuels or Heat Pump	0.6	10	30	21	0.15	11	19

Table 5.5 summarized the prescriptive glazing requirements of the Energy Code, including the maximum allowable U-factor and solar heat gain coefficient (SHGC) applicable to glazing. Doors are also required to meet the requirements.

Table 5.5: Energy Code Prescriptive Glazing Requirements

	0%-30%		30%-45%	
	Maximum U-Factor	**Maximum SHGC**	**Maximum U-Factor**	**Maximum SHGC**
Windows				
Electric Resistance	0.4	0.4	NA	NA
All Other Fuels or Heat Pump	0.55	0.45	0.45	0.4
Skylights				
Electric Resistance	0.6	0.4	NA	NA
All Other Fuels or Heat Pump	0.7	0.45	0.6	0.45

Table 5.6 summarizes the outdoor air dry bulb and wet bulb (OADB, OAWB) for Longview, Washington, as well as the Energy Code space designation and indoor design temperatures for each space. The OADB temperatures were taken from the Recommended Outdoor Design Temperatures for Washington State published by the Puget Sound Chapter of ASHRAE for Longview, Washington.

Table 5.6: HVAC Design Temperatures

	Heating Design Temperature (^oF)	Cooling Design Temperature (^oF)	Space Type
OADB/OAWB	24/-	87/67	-
Well Houses	45	90	Conditioned
Filter Gallery Building	45	90	Unconditioned
Electrical Room –Office/Treatment Building	60	80	Conditioned
Office, Meeting Room, Control Room, Lab, Locker Rooms – Office/Treatment Building	70	78	Conditioned
Chemical Treatment Area – Office/Treatment Building	45	90	Conditioned

The IMC requires that spaces classified as Hazardous occupancies (such as the chemical treatment area) be ventilated at 1 cubic feet per minute per square foot (cfm/sf). All other areas will be ventilated in accordance with the Washington Ventilation and Indoor Air Quality Code.

5.2.4.2 Plumbing

Potable water will be provided at the facility for use in the restroom sinks, water closets and showers, emergency eyewash/showers, and any other sinks around the treatment facility requiring potable water. The closest potable water connection to the facility will be along Weber Avenue. Tepid water will be provided for all emergency eyewash/showers in accordance with ANSI Z358.1-2004. Sanitary sewer will be required at the Office/Treatment Building. Floor drains will be required at the Well Houses and the Filter Gallery Building. Plumbing will be provided in accordance with the Uniform Plumbing Code (UPC).

5.2.4.3 Fire Protection

Fire sprinklers will be provided for the Chemical Treatment Area in the Office/Treatment Building. A fire separation wall will be provided between the Chemical Treatment Area and the remainder of the Office/Treatment Building to allow the fire sprinkler system to be omitted from the office area. The fire sprinkler system shall be provided in accordance with the IFC and NFPA 13 requirements for Ordinary Hazard Group 1 occupancy. The required flow rate for the fire sprinkler system shall be 0.15 gpm/sf of sprinkler operation in accordance with NFPA 13. All other buildings and areas at the facility do not require fire sprinklers. Fire extinguishers will be provided as required by the IFC.

5.3 Building Descriptions and Layouts

A plan of the proposed facility are included as Appendix E.

5.3.1 Structural and Architectural Design Approach

The overall architectural goal is to create a design that accommodates all operational requirements, meets both budget and regulatory requirements, and blends aesthetically into its surroundings.

5.3.1.1 Operational Requirements

The primary objective of the building design will be to provide an envelope for the process that takes place inside. The design will follow close coordination with the other design disciplines involved on the project. Operational issues include the following:

Efficiency. Provide an efficient workspace to allow productivity and safety. The layout of the facility should allow easy supervision and monitoring. This includes construction of indoor pipe galleries and buildings for the wells to facilitate supervision and maintenance activities.

Energy and Maintenance. The selection of materials will reflect the fact that this is a hard working, utilitarian facility. Finishes must be damage resistant and easily maintained. The use of natural light with light reflective interiors will increase energy efficiency.

Function. Provide adequate clearances allowing ease of operation and access to all mechanical equipment.

5.3.1.2 Materials

The selection of both interior and exterior materials must meet several criteria. They must be durable, easy to maintain, and readily available; but, they must also meets functional needs. They must also conform to the visual and aesthetic requirements of the surrounding built environment. Specifically, the external design, color, and appearance of the building must be in harmony in relation to other development within the business park.

Interior: Occupied spaces such as labs and administrative spaces will be treated with a higher level of finish such as carpet or resilient flooring, painted gypsum wallboard walls and acoustical ceilings to create a comfortable, professional environment. Interior partitions will be of metal stud construction for durability and dimensional precision. The more mechanical/process spaces will employ concrete or CMU walls with appropriate coatings as necessary for the environment. The floors in the utilitarian spaces will be sealed concrete. Interior finishes will be light in color to promote daylighting and maximize indoor light reflectivity.

Exterior: Exterior finishes and materials will be similar to the interior mechanical/process spaces with an emphasis on durability, low maintenance, and an industrial aesthetic. Exterior walls will be constructed primarily of concrete and CMU with possible accents of ribbed or corrugated metal panels.

5.3.1.3 Roof Systems

Roofs will be framed using wide flange or tube steel members with structural steel decking. Sloping roof forms will be preferred and will be clad in standing seam metal with a high performance coating. Overhangs will be used to protect exterior wall surfaces and openings. Flat (low slope) roofs, if employed, will be a single ply membrane roof system such as Hypalon or EPDM. If a walking surface is desired on any portion of the roof, a pedestal type or inverted roof membrane assembly roof system can be specified. Electronic bird repellent devises have proven effective at nearby buildings and may be installed on the roof.

5.3.1.4 Wall Systems

Exterior walls will be load bearing construction on concrete spread footing foundations. Proposed wall materials are as follows:

- Concrete: Very durable and low maintenance. Detail can be achieved with the use of form liners, reveal strips and color, either via stain, dyes, or paint.

- CMU: Can add additional texture and detail and can be used as accents with concrete. CMU can be natural, painted, split face, ground face, or scored and comes in a variety of colors.

- Metal panels: Available with high performance coatings suitable for this location and are available in many profiles and colors. Can be used as an exterior wall finish or for entrance or window canopies.

We anticipate the use of a combination of these materials to achieve the proposed aesthetic and operational requirements.

5.3.1.5 Openings

Special care will be given to the exterior envelope regarding water intrusion, particularly at openings. Doors will be hollow metal with insulated hollow metal frames. Overhead service doors will be steel insulated roll-up doors with motor driven operators.

Windows will be anodized aluminum with insulated, low E glazing. Operable windows, if any, will be screened casement or sliding.

Louvers will be anodized aluminum and may be part of a door or window system where appropriate.

5.3.1.6 Lighting

Lighting includes general interior, task lighting, exterior, exit, emergency, and site lighting. Interior lighting will typically be fluorescent with halogen, high-bay lighting where appropriate. Exterior lighting will be coordinated with the site lighting to create a consistent appearance. Directional, cut-off or concealed lighting may be used to minimize lighting impacts to neighboring areas.

5.3.1.7 Site

Site design issues that will be addressed in detailed design include:

- Vehicle access and parking
- Noise control
- Landscaping.

5.3.2 Mechanical Design Approach

5.3.2.1 Office/Treatment Building

The Office/Treatment Building will be separated into two separate areas. The northern side, including the electrical room, offices, operations room, lab, mechanical room, meeting room, and restrooms will be a conditioned space that will be heated and cooled at the design temperatures in Table 5.6. The Electrical Room will be conditioned by a split system heat pump, with the remaining office area, operations room, lab, and restrooms conditioned by a separate split system heat pump. The supply air to the restrooms and lab will be directly exhausted and will not be returned to the air handling unit. The air handling units that serve the office area and the Electrical Room will be located in the mechanical room and the outdoor heat pumps will be located on the eastern side of the building. Envelope insulation will be provided as required for Conditioned Spaces with heat pump heating systems in Table 5.4 and Table 5.5.

The southern side of the building (chemical treatment area) will be ventilated at 1 cfm/sf as required by the IBC for hazardous occupancies. The ventilation system will consist of exhaust fans and intake louvers for supplying outside makeup air to the Chemical Treatment Area. The Chemical Treatment Area will be insulated in accordance with the requirements in Tables 5.4 and 5.5 for conditioned spaces and will be heated using electric resistance heaters to maintain the temperatures listed in Table 5.6.

Plumbing at the Office/Treatment Building is expected to consist of potable water use in the restrooms and lab as well as any emergency eyewash/showers that are located in the Chemical Treatment Area. Sanitary sewer will also be required for the restrooms for the water closets, sinks and showers. Floor drains will be located as needed in the Chemical Treatment Area as well as the restrooms and Mechanical Room.

The fire sprinkler system shall be a wet pipe design and shall serve the Chemical Treatment Area. An independent fire water connection with backflow preventer will be provided to the Weber Avenue City water line. The fire sprinkler riser will be located in the Chemical Treatment

Area with a fire department connection located outside of the building. Fire extinguishers shall be provided in accordance with the IFC for the entire Office/Treatment Building.

5.3.2.2 Well Houses

Each Well House will contain one vertical turbine pump with a 450 hp motor (approximate) that will produce large amounts of heat while in operation. The cooling design temperature listed in Table 5.6 is a typical design temperature for this type of space. However, because a ventilation only system will be utilized, the actual temperature inside of the Well Houses will be higher than the OADB during the summer while the pumps are in operation. The Well Houses will be insulated in accordance with the requirements in Tables 5.4 and 5.5 for Conditioned Spaces and will be heated using electric resistance heaters to provide freeze protection at the temperatures listed in Table 5.6. Heating requirements for the Well Houses will be investigated further in detailed design, and if feasible, motor heaters may be installed in lieu of providing an electric heater and wall insulation.

Each Well House will be provided with floor drains that will connect to the plant drain or sanitary sewer system. Fire extinguishing systems are not required at Well Houses.

5.3.2.3 Filter Gallery Building

The Filter Gallery Building contains a blower room and a pressure filter gallery. The blower room will contain two 60 Hp blowers (1 duty, 1 standby) that will require 1,000 scfm each during operation. Additional ventilation will be provided for the blower room if the process air does not maintain the space within the acceptable cooling design temperature listed in Table 5.6. A ventilation only system will be utilized; consequently, the actual temperature inside of the blower room will typically be higher than the OADB during blower operation under summer outdoor design conditions. Makeup air for the process air and the ventilation air will be provided by intake louvers. The blower room will be an unheated space and will not be insulated.

The pressure filter gallery will contain process piping and valves for the filters located outside of the building. There will also be two vertical turbine-type backwash return pumps with 200 hp motors located inside of the space. Ventilation will be provided to maintain the cooling design temperature listed in Table 5.4. Makeup air will be provided by intake louvers. The pressure filter gallery will be an unheated space and envelope insulation will not be provided. Strip heaters on the pump motors will be provided to reduce condensation.

The Filter Gallery Building will be provided non-potable water and hose bibbs for washdown and maintenance purposes. The building will be provided with floor drains connected to the plant drain or sanitary sewer system. Fire protection is not expected to be required in the Filter Gallery Building.

5.3.3 Security Considerations

Building security is incorporated into the design as indicated below:

- Door switches (entry alarms) will be provided. Door switches will be activated by a key card.
- Security cameras providing views of the Office/Treatment Building and Filter Gallery Building will be provided.

Section 6: Site Design Elements

6.1 Site Layout

The preliminary facility layout includes an Office/Treatment Building, Backwash Storage Tanks, a Filter Gallery Building, greensand filters, surge tank, standby generator and fuel tank, access roads and parking, and sludge drying beds located on the northern/northeastern side of the site. The wells are located on the southern/southwestern side of the site, opposite the main structures, to minimize activity within the wellhead protection areas.

A site plan showing a preliminary layout of the proposed facility is included in Appendix E.

6.2 Zoning and Land Use

The site is within the Mint Farm Industrial Park limits and is governed by the Protective Covenants, Conditions, Restrictions, Easements, and Agreements for the Mint Farm Industrial Park (Mint Farm Covenants). The Mint Farm Covenants provide guidelines which define such items as: allowed land use, land coverage, setbacks, access, building and architectural standards, outside storage, parking, landscaping, and outside lighting.

Some of the Mint Farm Covenants from Article III, Restriction and Use of Property, are applicable to the site design for the water treatment plant and are listed below. These covenants will be integrated into the design of the site, with the noted exceptions. The full Mint Farm Covenants document is available from the City of Longview Building and Planning Department.

3.5 LAND COVERAGE. The minimum floor area of the principal building on a Lot shall cover at least 20 percent of the total buildable Lot area, exclusive of setbacks. The minimum floor area of the principal building may be established at a lesser percent of the total buildable Lot area, subject to a variance approved by the Declarant. Covered parking and storage apart from and not connected to the principal building shall not be considered to fulfill part of this requirement. ***Note: A variance may be needed for this requirement. This issue will be explored in detailed design.***

3.6 BUILDING STANDARDS. Pre-engineered metal buildings and metal clad buildings will be permitted; however, "tilt-up" concrete buildings are preferred. Roof overhangs may extend a maximum of 24 inches into the interior setbacks. ***Note: A CMU building is proposed. CMU construction does not appear to be precluded by the Mint Farm Covenants, however the architectural committee has the discretion to determine which building materials are in harmony with the surrounding developments within the park. This issue will be explored in detailed design.***

3.7 STREET FRONTING REQUIRED. Every principal structure shall front on or be located on property which fronts upon a street which is dedicated to and maintained by a municipal corporation.

3.8 SETBACKS. For every principal building allowed in The Park, there shall be a front yard having a depth of not less than 60 feet and a side yard on each side of the building of not less than 20 feet in width, except that a side yard of a corner Lot shall be not less than 50 feet in width along the side street property line and a rear yard of not less than 20 feet provided, however, that there shall be no side yard setback required between buildings having common ownership at the time of construction of such buildings or thereafter. A strip of land at least 10 feet in width abutting the street right-of-way shall be

landscaped with evergreen plants in accordance with the provisions of section 3.15(a) hereof. The set-back areas between the building lines and the property lines are to be used either for landscaped areas or for off-street surfaced parking and loading operations. A gatehouse shall be permitted in the set-back area provided the structure is limited to one story and 100 square feet in floor area. All setback areas shall be fully planted in a manner compatible and complementary to the architectural style of the building. A maximum slope of 3:1 is allowed in the setback areas. A transition shall be provided at the top and bottom of all slopes. The construction of streets outside the boundaries of the Plat shall not have the effect of converting any lot within the Plat to a corner lot.

3.9 OUTSIDE STORAGE. Outside storage shall not be permitted within the required setback areas that are between any public right of way and any building fronting thereon but shall be allowed within the buildable Lot area and within the setback areas along interior property lines; Stored materials within the buildable Lot area should be screened from view from the public right of way upon which any building fronts by means of a fence or wall or landscaping no less than five and not more than 15 feet in height. Outside storage shall be permitted only for materials which are associated with the principle use conducted on the premises. Within the required setback areas lying between a public right of way and any building fronting thereon, the height of the stockpile should not exceed the height of the obscuring fence and such storage should be screened on all sides visible from adjacent public streets using materials designed to harmonize with the architectural design. The provisions of this section are subject to variances granted by the Declarant or by the Architectural Committee.

3.10 FENCES AND WALLS. No fence or wall shall be constructed closer than 10 feet from the right-of-way of a public street, and no fence or wall shall be constructed within a drainage easement. All fences and walls shall be designed and constructed in a manner that is complimentary to the architectural design of the building. Except for walls or fences constructed to screen areas as required herein, no fence or wall shall exceed 10 feet in height, plus appropriate barbed wire or other security devices.

3.11 OFF-STREET PARKING. Every permitted land use within The Park shall provide off-street parking facilities for automobiles, as required by Chapter 19.78 of the Longview, Washington, Municipal Code. No parking shall be permitted to be closer than 7 feet from an interior property line. Adjacent to dedicated open space, no parking or pavement shall be permitted closer than 15 feet. No parking is permitted on public streets.

3.12 LOADING. Any establishment where the loading and unloading of trucks is a normal part of its operation shall provide off-street truck loading facilities as follows:

(a) Loading berths shall be permitted in the building setback areas, except that portion of a corner Lot formed by connecting two points 60 feet from the intersection of the street property lines.
(b) No berth shall be so located as to require direct entry and exit to the street.

6.3 Site Civil Improvements

6.3.1 Utilities Serving the Site

The following utilities are anticipated necessary at the facility: sanitary sewer, potable water, stormwater management, electrical power, telephone, and communications. Natural gas service will not be provided to the site but is available in the area. All of these utilities are anticipated to

be available in the Weber Avenue right-of-way. The design of the extension of Weber Avenue to the site is complete, and it is anticipated that the construction of the extension of this public street will be completed by August 2010 and before construction of the proposed facility. A complete discussion of electrical and communications issues and availability is provided in other sections of this report.

6.3.2 Stormwater Improvements

All development activity within the City is subject to the requirements of the City's Stormwater Manual (Manual) dated September 2009, which presents the minimum design standards for erosion and stormwater control.

Figure 2.1.1 of the Manual indicates that if more than 5,000 sf of land is disturbed, then a site plan should be submitted and a local Excavation and Grading Permit should be obtained before the project begins (Building, Public Improvement, and/or Right-of-Way permits satisfy this requirement). If more than 5,000 sf of new impervious area is created, then a storm drainage plan showing compliance with water quantity (detention), quality, and amenity criteria is required.

6.3.2.1 Water Quantity Control

Section 2.2.5 of the Manual indicates that projects creating more than 5,000 sf of new impervious surfaces must provide on-site stormwater flow control. Stormwater facilities must be sized such that the peak release rate from the development does not exceed the peak release rate from:

- Half of the pre-developed 2-year, 24-hour design storm,
- The pre-developed 25-year, 24-hour design storm, and
- The pre-developed 100-year, 24-hour design storm.

Table 6.1: 24-Hour Design Storms for the Longview-Kelso Urbanized Area

2-Year	2.54 inches	25-Year	4.37 inches
5-Year	3.10 inches	50-Year	5.17 inches
10-Year	3.60 inches	100-Year	6.17 inches

Section 1.4 of the Manual indicates that projects located within the drainage boundary of the Consolidated Diking Improvements District #1 (CDID #1) are exempt from flow control requirements. After discussing this project with CDID #1, the Mint Farm development is within the drainage boundary of CDID #1. Runoff from the Mint Farm is conveyed via Ditch 12 or Ditch 10 to a regional stormwater facility on Industrial Way, west of the Mint Farm. CDID #1 requires a fee in lieu of the onsite detention facility that would otherwise be required. CDID #1 has indicated that this fee is typically 1 dollar per cubic foot of storage required.

In accordance with the Draft Conceptual Geotechnical Report (Geotechnical Report) dated 13 October 2009, and prepared by Landau Associates, the near-surface soil is really fine-grained and groundwater is very close to the ground surface, especially during the winter and early spring months. Consequently, infiltration at the site will be minimal.

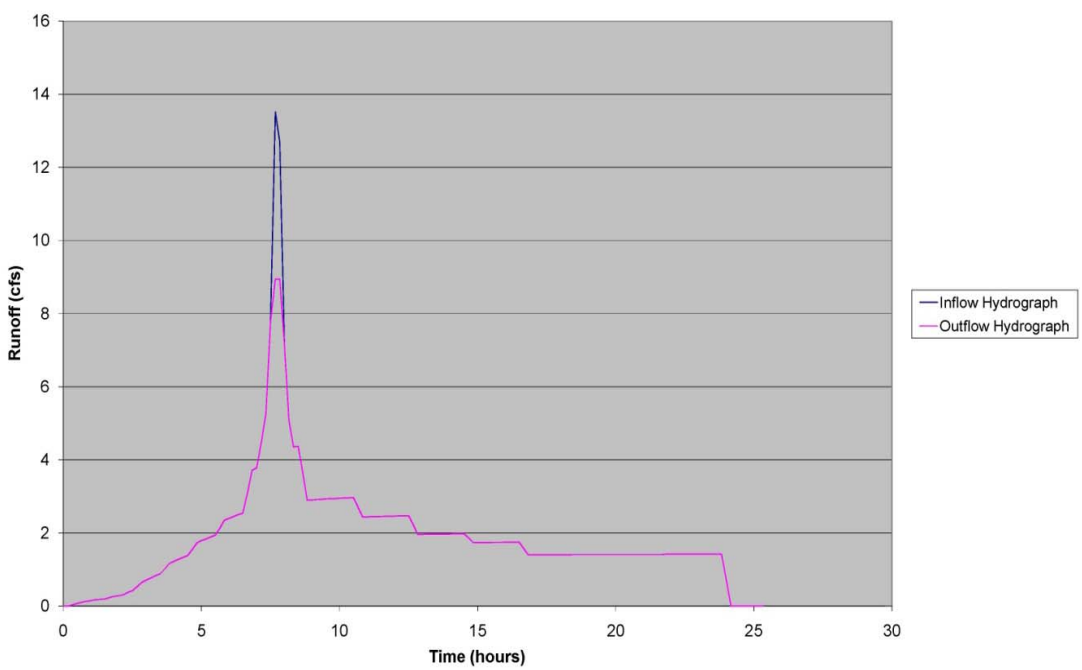
In accordance with the requirements of the Manual, the allowable release rate from the site is the runoff generated from the pre-developed 100-year, 24-hour design storm. The Santa Barbara Urban Hydrograph (SBUH) methodology is a common software used to estimate runoff generated from storm events. The computer program “HYD”, developed by King County, Washington Department of Public Works, was used for the SBUH calculations for this project.

In accordance with the surface infiltration design requirements of the Manual, a 100-year storm (6.17 inches of rainfall over 24 hours, NRCS Type 1A rainfall distribution) was used to generate the runoff flow rates presented. The pre-developed conditions consist of 9.9 acres of pervious surface. A conservative pervious area Curve Number of 86 was used, based on local soil conditions. The pre-developed time of concentration, Tc, is estimated to be 27 minutes.

The post-developed conditions consist of 1.8 acres of impervious surface and 8.1 acres of pervious surface. A conservative estimate of 5 minutes was used for post-developed conditions. A conservative pervious area Curve Number of 86 was used, based on local soil conditions, and an impervious area Curve Number of 98 was used, which is generally the industry standard for impervious surfaces.

Preliminary calculations indicate that this allowable release rate is 8.94 cfs. Figure 7.1 presents the inflow and outflow hydrographs for the 100-year, 24-hour storm event. The inflow hydrograph, generated from the “HYD” program, represents the inflow into a stormwater facility from the new storm drainage piping. The outflow hydrograph represents the release rate. At the early stages of the inflow hydrograph, the incoming runoff is released until the incoming runoff exceeds the allowable release rate of 8.94 cfs. After the incoming stormwater exceeds this amount, runoff accumulates in a stormwater facility until shortly after the peak inflow, 13.52 cfs, is reached. After this point, the inflow is less than the allowable release rate, and the runoff stored is gradually released.

Figure 6.1: Inflow and Outflow



The area between the inflow and outflow hydrographs represents the volume that must be contained within a stormwater facility. Using the "HYD" program, it was determined that 6,380 cubic feet of storage would be needed.

6.3.2.2 Water Quality

Section 2.2.6 of the Manual indicates that projects creating more than 5,000 sf of new impervious surfaces must provide onsite water quality control. The water quality design storm for the Longview-Kelso urban area is defined as two-thirds of the 2-year, 24-hour storm, with a Soil Conservation Service (SCS) 1A type rainfall distribution; therefore, the water quality design storm is 1.69 inches in 24 hours (2.54 inches x 0.667).

6.3.2.3 Amenity Criteria

Section 2.2.7 of the Manual identifies three goals that projects creating 5,000 sf of new impervious surface shall comply with. At least one of these goals shall be satisfied. However, Section 1.4 of the Manual indicates that projects located within the Manufacturing Districts, the Commercial Warehouse District, and the Redevelopment Project District are exempt from meeting these amenity goals. Although a District Map is not currently available online, discussions with City staff have indicated that the Mint Farm is located in the Manufacturing District. As such, this project is exempt from meeting the amenity goals. However, the stormwater facility will still be required to satisfy the "Facility Requirements," such as aesthetics, labeling and signage, and sustainable maintenance, which are described in greater detail in this section of the Manual.

6.3.2.4 Summary of Stormwater Requirements

As previously discussed, this project is exempt from stormwater quantity control and amenity criteria identified in the Manual. Stormwater quality requirements must still be satisfied. The intent to address the water quality requirements will be to install a stormwater swale in a landscaped strip along the site's frontage with Weber Avenue, as shown in the site plan in Appendix E. The swale will be designed in accordance with the applicable requirements of Section 2.2.7 of the Manual, regarding aesthetics, labeling and signage, and sustainable maintenance.

6.3.3 Fire Hydrants

Fire hydrants are required within 250 feet of the Office/Treatment Building. Fire hydrants will be designed in accordance with City Standard Plan W-150. The design of the extension of Weber Avenue indicates that fire hydrants, spaced approximately every 500 feet, will be provided along the site's frontage with Weber Avenue. Fire hydrant location for any hydrants needed on the treatment plant site will be developed in detailed design.

6.3.4 Vehicular Access

Vehicular access will be provided by driveway approaches connecting the site to Weber Avenue. It is anticipated that two driveway approaches will be provided, at the western and eastern sides of the site. Driveway approaches will be designed in accordance with City Standard Plan ST-160. It is anticipated that the site will accommodate pickup trucks, automobile traffic, commercial refuse trucks, chemical delivery trucks, and firefighting trucks. The larger trucks control design of vehicular access from the street and within the site. A 50-foot minimum curb and paving radius should be used for curb cuts and driveway design based on the truck traffic requirements. During preliminary site layout, drive and apron paved areas will be studied

to confirm the ability of vehicles to turn around, make deliveries, and not impact parked vehicles or run off into landscaped areas.

6.3.5 Parking

Section 3.11 of the Mint Farm Covenants indicates that off-street parking shall be provided in accordance with Chapter 19.78 of the City's Municipal Code (Code). Section 19.78.0200 of the Code indicates that off-street automobile parking spaces shall not be less than 9 feet wide and 20 feet deep. As such, the required parking spaces are proposed to be 9 feet by 20 feet. Sections 19.78.080 and 19.78.100 of the Code identify the number of parking spaces based on specific uses such as commercial retail, professional offices, bowling alleys, churches, etc. The proposed use of the site does not appear to fall within the categories indicated in these two sections. However, Section 19.78.090 of the Code provides guidance for other uses. This section of the Code indicates that one space shall be provided for every 1,000 sf of floor area of all buildings or one parking space for each three full-time employees during the maximum working shift. Because the Office/Treatment Building has a floor area of 6,650 sf, it is anticipated that seven parking spaces will be required.

Section 19.78.025 of the Code indicates that handicap parking stalls shall be provided in accordance with the requirements of 42 U.S.C. Chapter 126 (Equal Opportunity for Individuals with Disabilities) and with the laws of the State of Washington relating to parking facilities for persons with disabilities or handicaps (Ord. 2619 § 3, 1996). These requirements indicate that for one to 25 parking spaces, one space must be an accessible space with an adjacent 5-foot wide striped access aisle and marked with the international symbol of accessibility. Further, at least one of every eight accessible spaces must be van accessible (requiring an 8-foot wide access aisle) and marked accordingly as being van accessible. Striping of accessible spaces will be installed in accordance with City Standard Plan ST-140.

6.3.6 Site Security

As an industrial facility storing hazardous chemicals and a water treatment plant for potable water supply, careful consideration needs to be given to site security. The visual aspects of such security and coordination with the landscaping are important design guidelines. The Building Design Elements section of this report address features to be provided for building security. For site security, chain link fence around the entire site is provided. At the access roads to Weber Avenue, card-activated automatic locking gates will be provided.

6.3.7 Site Signage and Lighting

The Building and Electrical Design Elements sections of this report discuss signage and lighting design issues and concepts.

6.3.8 Landscaping

Landscaping features will be developed in detailed design, and will conform to the Mint Farm Covenants and other applicable codes. The landscape architecture will be designed within the context of the surrounding industrial park.

6.3.9 Site Work Construction Materials

In accordance with the recommendations in the Geotechnical Report, paving on site and for the access drives will consist of 3 inches of asphaltic concrete (A/C) and 6 inches of aggregate

base over a properly prepared subgrade, in accordance with City Standard Plan ST-080. On access drives that will not be subject to traffic from chemical storage vehicles or other vehicles containing potentially hazardous material, permeable pavement will be considered. Access roads to the well houses will be graveled to minimize impacts within the wellhead protection zone. Paved areas may receive striping to designate parking stalls, handicap designations, direction arrows and fire lane as required. Paved access roads will be designed with Type A curb and gutter in accordance with City Standard Plan ST-110.

Sidewalks and doorway aprons will be designed using un-reinforced concrete over a properly prepared sub-base and gravel base, in accordance with City Standard Plan ST-150. Walkway widths shall conform to Code, but will not be less than 5 feet. Pedestrian ramps will be designed in accordance with City Standard Plans ST-130 and ST-140.

Sanitary sewer and storm drain manholes will be designed in accordance with City Standard Plans SS-020 and SD-130, respectively. Sanitary sewer and storm drain manhole frames will be designed in accordance with City Standard Plans SS-100 and SS-110 or SD-150 and SD-160, respectively. Sanitary sewer and storm drain manhole covers will be designed in accordance with City Standard Plans SS-120 and SD-170, respectively. Cleanouts will be designed in accordance with City Standard Plan SS-160.

Catch basins will be designed in accordance with City Standard Plan SD-010, with inlet frames and grates designed in accordance with City Standard Plans SD-100 and SD-110, respectively.

Sanitary sewer pipe will be designed with PVC pipe conforming to ASTM D-3034, in accordance with City Standard Plan SS-000. Storm drain pipe will be designed with high-density polyethylene (HDPE) pipe, in accordance with City Standard Plan SD-000.

Water pipe will conform to the following pipe specifications:

- Pipe (exposed and buried): 2 ½ inches and larger; ductile iron AWWA C151, mechanical joint ANSI A21.11 cement mortar lined for buried pipe.
- Fittings (exposed): Flanged, ASTM A 536, cement mortar lined.
- Fittings (buried): ANSI A21.10, mechanical joint ANSI A21.11, cement mortar lined.
- Valves (exposed and buried): Isolating and throttling – butterfly, gate, or ball valve.
- Corrosion Protection: Polyethylene encased.
- Anchorage: All pressure pipe shall be anchored with Megalug or TR-Flex mechanical joints; all pipe shall have secondary restraint with thrust blocks or approved equal unless otherwise agreed by City of Longview.

All yard piping will be designed with pipe bedding in accordance with City Standard Plan SS-240, and trenches will be excavated and backfilled in accordance with the applicable City Standard Plans ST-010, ST-020, and ST-040.

Interior ladders and ventilation will be provided for personnel access where required in vaults, etc.

Section 7: Electrical and Instrumentation &Control Design Elements

7.1 Introduction

The electrical distribution for the water treatment plant will be designed to meet requirements of the 2008 NEC, NFPA 70. A standby generator will be used as the plant alternate source of power and it will be sized to operate all vital components at peak water production, including critical lighting and ventilation.

7.1.1 Incoming Power Service

The local provider of electrical power is Public Utility District No. 1 of Cowlitz County (Cowlitz PUD). Power will be provided to the plant at 12.47 kilovolt (kV), 3-phase. The largest transformer that the Cowlitz PUD can provide is 1,500 KVA. The facility loads will exceed the capacity of the largest transformer that Cowlitz PUD can provide; therefore, the City will purchase its own service transformer.

The capacity of the utility power distribution lines that cover this location should be verified with respect to the projected load for this facility. It is anticipated that an underground service drop can be obtained from the existing power lines and be routed to a pad-mount transformer located outside the Electrical Room.

The layout and space allocation is currently based on standard Puget Sound Energy guidelines until more specific direction can be obtained from Cowlitz PUD, who will be the power provider for this facility.

It is expected that the main transformer will be purchased by the City; thus, revenue metering for the plant will be performed at the 12.47 KV level (primary metering). The location of the utility revenue meter is anticipated to be on a pedestal near the transformer, or on the power pole from which the primary power cables to the plant originate. This decision will be at the discretion of Cowlitz PUD.

7.1.2 Electrical Distribution

Current load estimates indicate that the initial plant loads will require an electrical service with an approximate rating of 3,000 amps. Expansion to a future maximum day production capacity of 25 MGD will require an electrical service with an approximate rating of 4,000 amps.

A 4,160 V (4 KV) power distribution system was explored when considering the large electrical load from the well pumps. If 4 KV power was provided for the well pumps, the cables from the Electrical Room to the motor starters and from the starters running down to the well pumps could be smaller than that required if the well pumps were 480 V rated. However, at least two separate transformers (480 V and 4 KV) and two switchgear sets would be required. Having two transformers would add complexity to the transfer of power from the main electrical service to standby power.

Considering the additional cost and complexity that a 4 KV system would likely add to the project, power distribution will be provided using 480 V, 3- phase power from the centrally located Electrical Room.

A 480 V distribution system could consist of one or more transformers. If multiple transformers were installed, each would serve a dedicated area of the plant. Each transformer would require its own automatic transfer switch (ATS) to provide for connection of the standby generator set should the transformer or its associated service switchgear be inoperable. The added complexity and cost of this approach, given the budget and space limitations, were considered and set aside in favor of a single transformer approach. A single transformer will provide the benefit of simplified power distribution and less equipment to maintain in the facility. If the City has a strong preference for two smaller transformers, the idea can be explored during detailed design.

In order to meet the initial electrical load, a service size of 3,000 amps could be met with a 2,500 kilovolt-ampere (kVA) transformer. The future 25 MGD facility electrical load of 4,000 amps could be met with a 3,500 KVA transformer. A typical service transformer of these sizes can be expected to remain in service for 30 to 40 years.

It is recommended that space and electrical capacity be designed as follows:

1. Design the electrical service entrance for the plant to accommodate the future electrical capacity of 4,000 amps. Electrical switchgear buses should be sized for the future load capacity and any underground raceways for major equipment should have capacity for future build out. Leave space in the Electrical Room for future electrical distribution equipment that matches the allocation of space for future process equipment.

2. Provide a single12 kV-480 V distribution transformer rated at 2,500 kVA. Space will be allocated on the site for the eventual replacement, if required, with a 3,500 kVA transformer.

3. Provide a single ATS in the Electrical Room. The standby generator will be located as near as feasible to the Electrical Room so that it can provide power the ATS. Operation of the ATS from ‘utility power’ to ‘standby power’ will be automatic; return to ‘utility power’ upon restoration of power will be a manual operation.

It is recommended that the incoming service switchgear be a power switchgear with programmed transfer from utility to standby power to serve as both a service entrance rated disconnect and ATS. Suppliers such as Eaton and Square D offer this with mechanical interlocking between the two main breakers. This arrangement allows service to be provided either from the utility power service or from the standby generator. Rather than using transfer switches, the power circuit breakers are switched between the two sources. This serves the purpose of the incoming circuit breakers and transfer switch; therefore, it saves the added space of an ATS.

7.1.3 Electrical Room

The Electrical Room will contain the main plant service entrance switchgear plus the motor control center for the plant. The NEC has language that generally calls for any building on a campus type facility to have its own power disconnect and for the number of power disconnects to be limited to six or less. Because this plant will likely have the distribution system radiating from the single electrical room, buildings on this site may have more than one electrical power circuits feeding them. A general interpretation by the State of Washington code authorities is a facility can be considered contiguous if it has structural continuity. That is, if it is connected by tunnel or gallery or by interconnecting roof, it can be viewed as a single building. It is not expected that buildings on the proposed water treatment plant site will be viewed this way.

The NEC has exceptions to this requirement for industrial facilities where “documented safe switching procedures” are in place. Generally speaking, this is probably intended for facilities where electricians are on staff. Applicability of this exception will be verified in detailed design.

The Electrical Room will be designed with a 2-hour fire rating, and the Chemical Storage Area with H-4 occupancy will by physically separated from the rest of the Office/Treatment Building with a 2-hour firewall. It is anticipated that the Electrical Room will not be sprinklered, but this will be confirmed with the City Fire Marshal. This issue should be verified with the local fire jurisdiction.

7.2 Single Line Diagram

A single-line diagram for the proposed site is shown in Appendix E. The new transformer for the water treatment plant will be a 2,500 kVA transformer, which will support the electrical loads for the proposed facility. Installation of additional wells and future capacity expansions will require a transformer replacement.

7.2.1 Electrical Loads

A preliminary determination of the electrical load for the proposed facility is shown in Table 7.1.

Table 7.1: Water Treatment Plant Electrical Load

Equipment (New)	Load, each[(a)] (Amps)	Duty Quantity	Max Load (Amps)
Well Pumps	515	4	2060
Backwash Return Pumps	240	1	240
Air Scour Blowers	77	1	77
Other Pumps		LOT	152
120/208 Transformer	118	1	118
25% of Largest Motor	129	1	129
TOTAL CONNECTED LOAD (AMPS)			2776
TOTAL CONNECTED LOAD (kVA)			2,308

Note:

(a) Estimate only

7.3 Standby Power System

Article 708, Critical Operations Power Systems (COPS) was added to the NFPA 70-2008 code to provide facilities with a higher level of protection from disasters. Article 708 provides an additional level of protection for the electrical equipment and wiring such that in the event of a natural or human created disaster, the COPS facility will still function while other facilities may fail.

COPS systems are generally installed in vital facilities that, if destroyed or incapacitated, would disrupt national security, the economy, public health, and/or safety. The local authority having jurisdiction has the authority to require a designated area or all of the facility to meet these new

requirements. If only a designated area in the facility is required to meet this section of the code, it is referred to as the Designated Critical Operations Area (DCOA). It is anticipated that parts of the proposed facility could be designated as DCOA and subject to requirements of COPS. Some of the COPS requirements applicable to the standby power system are indicated below:

- A COPS alternate power source shall have the capacity and rating to run full load continuously for a minimum period of 72 hours.
- The alternate power source can be a generator, uninterruptible power supply (UPS), or a fuel cell system.
- The alternate power source must have selective load pickup and shedding capabilities as needed to ensure power to (1) the COPS and emergency circuits, (2) the legally required standby circuits, and (3) the optional standby circuits, in that order of priority. If the DCOA is fed by a single generator source, a means for connecting a portable generator shall also be provided.

A dual-feed power supply is not acceptable for a COPS facility; therefore, a standby generator is proposed. The standby generator will be sized, at the minimum, to carry the critical plant load for well pumping and disinfection. The best way to distribute standby power is via the normal power distribution system. Therefore, the connection of the standby generator at the main service switchgear is the most efficient means to accomplish standby power distribution.

One of the new requirements of the 2008 NEC is, for any legally required standby power system, the electrical distribution must be selectively coordinated. It is assumed for this discussion that the standby generator will be a legally required system. The ramifications of this new code provision may be that the plant may need to use fused overcurrent protection rather than more conventional circuit breakers. Circuit breakers may be able to meet the selective coordination requirements but it may demand the use of larger and more expensive solid state trip type breakers, as well as more sophisticated fault logic schemes that are normally not seen in this size facility. For now, it will be assumed that fused protection will be utilized.

Another aspect of the standby power distribution will be the need for generator load control. The plant will be under the control of an overall plant control system. That plant control system will need to have control of the plant loads down to the individual equipment level and will need to be programmed to sequence the plant loads on and off such that the loading on the generator is managed while it is in operation.

7.3.1 Generator Size

It is estimated that a 2,000 kW emergency generator will be required to power the equipment summarized in Section 7.2.1 to provide for 20 MGD of water production capacity. This sizing should be validated via a generator sizing program once better load information is available during detailed design. Any plant motor loads that are across the line started and that are larger than 30 hp should be considered for soft start.

7.3.2 Generator Load Bank

In order to allow the City to test the generator regularly without connecting it to the distribution system, the generator will be equipped with an automatic load bank rated at approximately 50 percent of rated generator capacity, (i.e., 1,000 kW). This is the maximum capacity load bank available in a duct or radiator mounted configuration. A stand-alone, pad-mounted load bank provided in a NEMA 3R outdoor enclosure with cooling fans may be considered.

7.3.3 Generator Manufacturer

Standby generators of this capacity are currently required to meet EPA Tier 2 emissions requirements. Caterpillar, Cummins, and Kohler all provide generators meeting these requirements and are the recommended manufacturers for the size of generators being considered. Standby generators will be required to meet EPA Tier 4 emissions requirements beginning on 1 January 2011, whereas emergency generators will only be required to meet the EPA Tier 2 requirements. If the standby generator is designated part of a COPS, it will only be required to meet EPA Tier 2 requirements regardless of the purchase and installation date. However, if the standby generator is not designated part of a COPS and is purchased and installed after 1 January 2011 it will need to meet the required Tier 4 emissions which may impact availability. Generator availability and manufacturer preferences will be explored during detailed design. Delivery time for this size generator meeting EPA Tier 2 emissions is typically around 24 weeks.

7.3.4 Diesel Fuel Tank

A stand-alone, aboveground, double-wall diesel fuel storage tank will be provided as shown on the site plan in Appendix E. The storage tank will be sized to provide enough fuel for 72 hours of generator operation at the anticipated load. A 2,000 kW generator consumes approximately 145 gallons per hour of fuel at 100 percent load and 110 gallons per hour at 75 percent load. Seventy-two hours of operation at full load would therefore require approximately 10,500 gallons of fuel storage. Because the generator is unlikely to be fully loaded for long periods of time, fuel consumption over a normal 72-hour period is unlikely to exceed approximately 8,000 gallons. A 10,000-gallon storage tank is proposed for preliminary design. If the facility is not a DCOA, the stand-alone fuel tank could be removed from the project and the fuel tank could be located underneath the generator.

Diesel fuel has a limited shelf life, typically one year. Addition of a fuel stabilizer typically extends the life of diesel fuel to two years. One-half-hour of testing per month at maximum load will consume approximately 900 gallons of fuel per year, so care must be taken to ensure that the fuel remains fresh. Dispensing fuel from this location would require extensive modifications to the project to comply with applicable codes and is not recommended.

7.3.5 Noise Considerations

The standby generator will be installed outside in a sound attenuating, weather-protected enclosure. The enclosure may be either a skin-tight or walk-in type enclosure, depending on the level of attenuation required. The exhaust system will be configured with a minimum critical grade silencer providing 25 dBA of noise attenuation. Attenuation requirements and enclosure preferences will be developed during detailed design.

7.4 Other Issues

The 2008 NEC calls for power disconnects at motors unless documented safe switching procedures are practiced at the facility. Documented and enforced lock out-tag out procedures are considered as compliance with this. For the City, it will be assumed for this project that there will not be full time electrical staff at the plant and that the usual lock out – tag out procedures that are practiced at larger industrial facilities may not be in effect for this site. Unless directed otherwise, the plant design will be based around providing safety switches at all motorized equipment per the NEC. Note that the NEC does not require disconnects for all equipment. There are exceptions for variable frequency drive (VFD) driven equipment and for large motors over 100 hp and for equipment in classified locations.

Exterior lighting will be kept to a minimum on this site in deference to the growing demand for dark sky compliance. Lights will be provided over exterior exit doors and manually controlled lights for process areas where maintenance may be needed after dark. Roadways will have minimal lighting, mostly at intersections and at the plant entrances. If enhanced after hours security is a concern, then this may warrant further attention to strike a balance between these requirements.

It is anticipated that the plant fire alarm panel and central phone equipment and network communications equipment will be in the Office/Treatment Building.

Power factor correction is generally provided based on the utility power company charges for lower power factor. The rate tariff for this plant is not known at this time. The implementation of power factor correction will need to be addressed when this information is available.

7.5 Instrumentation and Control

The instrumentation and controls system is designed based on the following criteria:

- It must provide a safe process for the occupants, the operators, the facilities, and the environment.
- It must be secured against unauthorized access to physical properties such as buildings and equipment. It must also be able to protect against network intrusion as well as unauthorized data access.
- It must provide a reliable, continuous process with minimum staffing. Provision must be made for unattended operation during portions of the operations work schedule. This will include alerting on-call personnel to handle emergency situations during non-attended periods.
- It must provide controls for manual operation close at hand to processes in case telemetry goes down.
- It must guarantee system responsiveness. The system must be fast enough to respond to any change in process. Data exchange between systems must be efficient.
- It must be feasible in term of costs. When selecting system, the entire plant must be considered to ensure the overall performance. Criteria for selecting such systems include performance, cost of operation, maintenance, training, and inventories.
- The system must be compatible with existing equipment and systems and be flexible for future expansion.
- The system must have UPS feed from both main power and standby power.

In addition to the above criteria, the instrumentation and controls system at the plant must also incorporate the following functionalities into the design:

- Process automation.
- Local and remote process monitoring.
- Local and remote control with responsiveness.

- A flexible and reliable alarm management system.

- Data acquisition and automatic database synchronization.

- A system integration that allows different systems with different functions to be able to exchange data smoothly, at the same time preserving their functionalities. This will ensure minimum downtime. In the case of a failed system, it will not bring down the whole process, and it will allow independent system maintenance.

- A system integrity that provides authentication, authorization, and access. Some of the examples include password, IP cameras, tamper switches, fire alarm system, and security system.

7.5.1 Process Automation

The following paragraphs discuss the design philosophy and the instrument and control system selection criteria.

- The degree of process automation for the facilities will depend on expected staffing levels, operator skill sets, facility operating hours, and shift coverage. Unattended operation will require a high degree of automation. The level of automation will be explored with the City in detailed design.

- Some of the instruments and control equipment must be fail-safe in case of a system failure. Design should address physical safety first, equipment protections second, and continued system service, third. The City will participate in addressing these criteria.

- The new instruments and control equipment are to be latest generation. Instruments and controls repeated throughout the system will be of a common model and manufacturer where possible. An upgrade path must be considered for existing systems that will not be updated immediately. City maintenance and operations personnel will participate in specifying these items. Instruments and control equipment should also have self-diagnostic capability, have communication capability, and allow flexible configuration. Common communication protocols for subsystems or wider network use, such as Hart, DeviceNet, or Modbus, will be determined with City input.

- The selection of instruments and control equipment must be standardized. It may result in higher initial costs, but in the long run it will benefit the plant in reduced inventory, maintenance, and training costs.

- Besides the traditional hardwire input/outputs (I/Os), the application of an industrial network will be explored in detailed design. Instrument networks such as ModBus and DeviceNet allow information to be exchanged at lower cost per I/O than traditional hardwire I/Os. Hardwire I/Os will be used for critical applications.

- Where applicable, the instruments and control equipment must be supplied with redundant power supply. Besides plant backup generator, the use of solar power or UPS may be required. For example, communication systems, programmable logic controllers (PLCs), and chlorine residual analyzers will be powered by a UPS because chlorine residual monitoring should not be interrupted even if the whole plant were to lose its main power supply.

7.5.2 Central Versus Local Monitoring

Local human-machine interface (HMI) devices will allow the plant operators to monitor and control the plant while they are at an equipment system site. Local operation will be coordinated with central control from an operation center in the Office/Treatment Building. The local stations will consist of panel mounted display interfaces, switches, lights, and other control hardware. The central monitoring station will interface with various systems such as process automation systems, security system, fire alarm system, and any IP camera or other monitoring system. The central HMI stations will include an industrial class (PC) with two separate monitors. Each monitor will be able to display process information and alarm information, and support information such as vendor support documents, IP camera images, engineering documents such as loop diagrams and motor control schematics, and O&M manuals.

7.5.3 Alarm Management

The following paragraphs describe the proposed alarm system, as well as alarm handling strategies.

- The programmer configures the instruments or equipment with preset set points that will trigger an alarm; for example, Hi Tank Level Alarm, Lo Pressure Level, Fire Detected Alarm. The instruments or control equipment can also monitor its status and generate alarms such as equipment failures alarm, communication failure alarm, etc. The programmer can also program the system so that certain events will also trigger an alarm. For example, if an intruder tries to access the network, after a set number of unsuccessful tries, the system will trigger an alarm.

- Preliminarily, the alarms can be categorized into a minimum of four classes. The following describes these classes, and can be modified and implemented according to plant's alarm management standards such as color, flashing, alarm acknowledgement, alarm-printing, and reports. Class I alarm is the most critical alarm. It includes Fire Alarm, plant emergency shutdown, etc. This is a facility level alarm. Class II is a process area alarm such as a filter process shutdown. Generally, this could cause plant system normal shutdown. Class III is a process system level alarm. This alarm will not cause the plant to shut down. For example, if a primary system fails, the standby system will be switched on. Class IV is an instrument/equipment level alarm; for example, warning of equipment failure or Hi/Lo process level.

- All alarms will be reported to central HMIs. Alarms can also be reported to the local subsystem HMIs via wireless communication or fiber optics connection. Priority alarms can also be routed to email, pagers, or cellular phones via the control system alarm dialer.

- Operations management of alarms will be discussed in detailed design. Typically an operator must be at the plant to acknowledge priority alarms. Non-priority alarms can be acknowledged at remote HMIs. Depending on the criticality of the alarm, the City may determine which classifications of alarms an operator can and cannot bypass.

- All alarms will be logged to a server and available for viewing through a browser or sent to a printer if desired. The information includes the type of alarm, date and time occurred, process value, alarm description, and alarm class. Alarm acknowledgement also can be printed with the above alarm information and the operator's name.

- Care must be taken to avoid excessive alarming of information only type messages.

7.5.4 Data Acquisition

One of the most important requirements of a supervisory control and data acquisition (SCADA) system is its ability to acquire process information and archive data. The central HMI collects data and distributes data available for offsite monitoring and reporting. System synchronization, hardware preferences, and other SCADA features will be developed in detailed design and coordinated with the City's SCADA engineer - Harris Group.

7.5.5 System Integration

The design of the instrumentation and controls system at the plant includes process automation, and addresses process continuity and operator safety in a secured environment. It must protect the facilities, the environment, and the occupants. This requires an integration of different systems coordinating with each other; however, this does not mean that each system cannot function by itself. It is imperative that the following systems communicate with each other efficiently.

- Process automation systems such as well pump PLCs, filtration PLCs, and chemical dosing PLCs.
- Electrical System: Motor control center (MCC), switchgear protection, monitor, and control network.
- Security system including video cameras. The building access system includes card readers.
- Fire protection system and sprinkler.
- Wireless Ethernet communication system or fiber optics.
- Network router and/or switches.
- Alarm auto-dialer.
- Telephone system.
- Support Systems: HVAC control, lighting control.

7.5.6 System Integrity

The following paragraphs discuss the protection schemes available against accidents as well as illegal intrusions.

- The plant protection schemes include fire protection and intrusion detection. The plant access should be logged and protected by the card reader.
- Video cameras can also be used with pan/tilt/zoom features. Each camera could be individually controlled and monitored either from local HMI or offsite. Cameras could be located at the plant's entrance gates, inside the control room, and other locations the City would like to monitor.
- The process equipment protection schemes include password. Equipment panel should be equipped with tamper switches.

- System access is protected and controlled by authentication, authorization, and access (AAA) control. This allows flexibility in system management and resource management. PLC, switches, and controllers are protected by password. The HMI system security scheme controls user access to data, application, and communication equipment.

- Network access is controlled by filtering methods such as MAC filter (Layer 2), IP Subnet filter (Layer 3), or Type of Service protocol filter (Layer 4), which includes HTTP, FTP, SMP, and SNMP.

- Data integrity should be protected by 128-bit encryption. Wireless communication system data are protected by wire equivalent protection (WEP). The database has its own encryption and protection.

7.6 Process & Instrumentation Diagrams (P&IDs)

Preliminary P&IDs for the proposed facility are included in Appendix E. At this time, the process portion of the drawings is partially complete, but the PLC and SCADA levels remain to be developed in detailed design.

Section 8: Distribution System Design Elements

8.1 Distribution System Hydraulic Modeling

The Longview Water Distribution System Hydraulic Model (model), developed and calibrated as part of the 2005 Water System Plant Update (2005 Plan) was used to analyze the impact of the new Mint Farm RWTP) on the existing distribution system. The original model was updated and calibrated using available geographical information system (GIS) information and through discussions with City staff as to changes that occurred in the distribution system and operational conditions since the 2005 Plan. Some of these updates included the addition of new pipes, revisions to pump station curves, pressure regulating valve (PRV) settings, and other water system modeling elements. The model was calibrated as part of the 2005 Plan and few changes were found as a result of the updates to the model made for the Mint Farm RWTP analysis; therefore, calibration of the model was considered current.

In order to update the demands of the new model, the City's 2005 Plan projected demands for maximum day demand (MDD) were extrapolated out to year 2029. In the 2005 Water System Plan, the forecast MDD ended at 2025. For the purposes of this study, the City decided to extrapolate those demands to a 20-year projection (year 2029). Based upon the projections developed in the 2005 Plan, the combined MDD in year 2029 is 17.43 mgd.

Table 8.1 provides the combined totals for both Beacon Hill (formerly Cowlitz County PUD) and the City RWTP demands through year 2029. Table 8.1 also shows the average day demand (ADD) for year 2029 as 8.31 mgd.

Figure 8.1 illustrates the revised configuration of the existing distribution system for the City with the new Mint Farm RWTP included. As can be seen on this figure, the new Mint Farm water system will be more centrally located within the existing distribution system. The existing regional water treatment plant, also identified on this figure, will be taken off line and abandoned.

8.1.1 Hydraulic Impacts

A number of hydraulic modeling scenarios were conducted to establish the correct transmission main diameter and path. The 2029 MDD of 17.4 MGD was used as the target demand. Figure 8.1 identifies the optimum pipeline path and diameter for the new system. As can be seen from Figure 8.2, a 30-inch proposed transmission main, east to the existing Weyerhaeuser railroad right-of-way and then north to a connection with the existing 20-inch transmission main, appears to provide the best hydraulic conditions for the new treatment plant. Table 8.2 identifies the projected velocities of the new pipeline as well as the increase in pressures in the local area as a result of the increased hydraulic grade line.

Table 8.1: Projected Water Demands

	Longview			Beacon Hill			Combined Totals		
Year	ADD (mgd)	MDD (mgd)	Total Projected ERU	ADD (mgd)	MDD (mgd)	Total Projected ERU	ADD (mgd)	MDD (mgd)	Total Projected ERU
2005	5.28	11.09	27,185	0.96	2.01	4,659	6.24	13.10	31,843
2006	5.34	11.20	27,458	0.98	2.06	4,758	6.32	13.26	32,216
2007	5.39	11.31	27,734	1.00	2.10	4,859	6.39	13.41	32,593
2008	5.45	11.43	28,013	1.02	2.14	4,962	6.47	13.57	32,974
2009	5.50	11.54	28,295	1.04	2.19	5,067	6.54	13.73	33,362
2010	5.56	11.66	28,579	1.07	2.24	5,175	6.62	13.89	33,754
2011	5.61	11.78	28,866	1.09	2.28	5,285	6.70	14.06	34,151
2012	5.67	11.89	29,156	1.11	2.33	5,397	6.78	14.23	34,553
2013	5.72	12.01	29,449	1.13	2.38	5,511	6.86	14.39	34,960
2014	5.78	12.13	29,745	1.16	2.43	5,628	6.94	14.57	35,373
2015	5.84	12.26	30,044	1.18	2.48	5,747	7.02	14.74	35,792
2016	5.90	12.38	30,346	1.21	2.54	5,870	7.11	14.91	36,216
2017	5.96	12.50	30,651	1.23	2.59	5,994	7.19	15.09	36,645
2018	6.02	12.63	30,959	1.26	2.64	6,121	7.28	15.27	37,080
2019	6.08	12.76	31,270	1.29	2.70	6,251	7.37	15.46	37,522
2020	6.14	12.88	31,584	1.31	2.76	6,384	7.45	15.64	37,968
2021	6.20	13.01	31,902	1.34	2.82	6,519	7.54	15.83	38,421
2022	6.26	13.14	32,223	1.37	2.88	6,658	7.64	16.02	38,880
2023	6.33	13.28	32,546	1.40	2.94	6,799	7.73	16.21	39,345
2024	6.39	13.41	32,873	1.43	3.00	6,943	7.82	16.41	39,816
2025	6.45	13.54	33,204	1.46	3.06	7,091	7.91	16.61	40,294
2026	6.52	13.68	33,537	1.49	3.13	7,241	8.01	16.81	40,778
2027	6.59	13.82	33,875	1.52	3.19	7,395	8.11	17.01	41,269
2028	6.65	13.96	34,215	1.56	3.26	7,552	8.21	17.22	41,767
2029	6.72	14.10	34,558	1.59	3.33	7,712	8.31	17.43	42,271

Note:

(a) ERU – Equivalent Residential Unit

Figure 8.1: Proposed Mint Farm RWTP Project Site

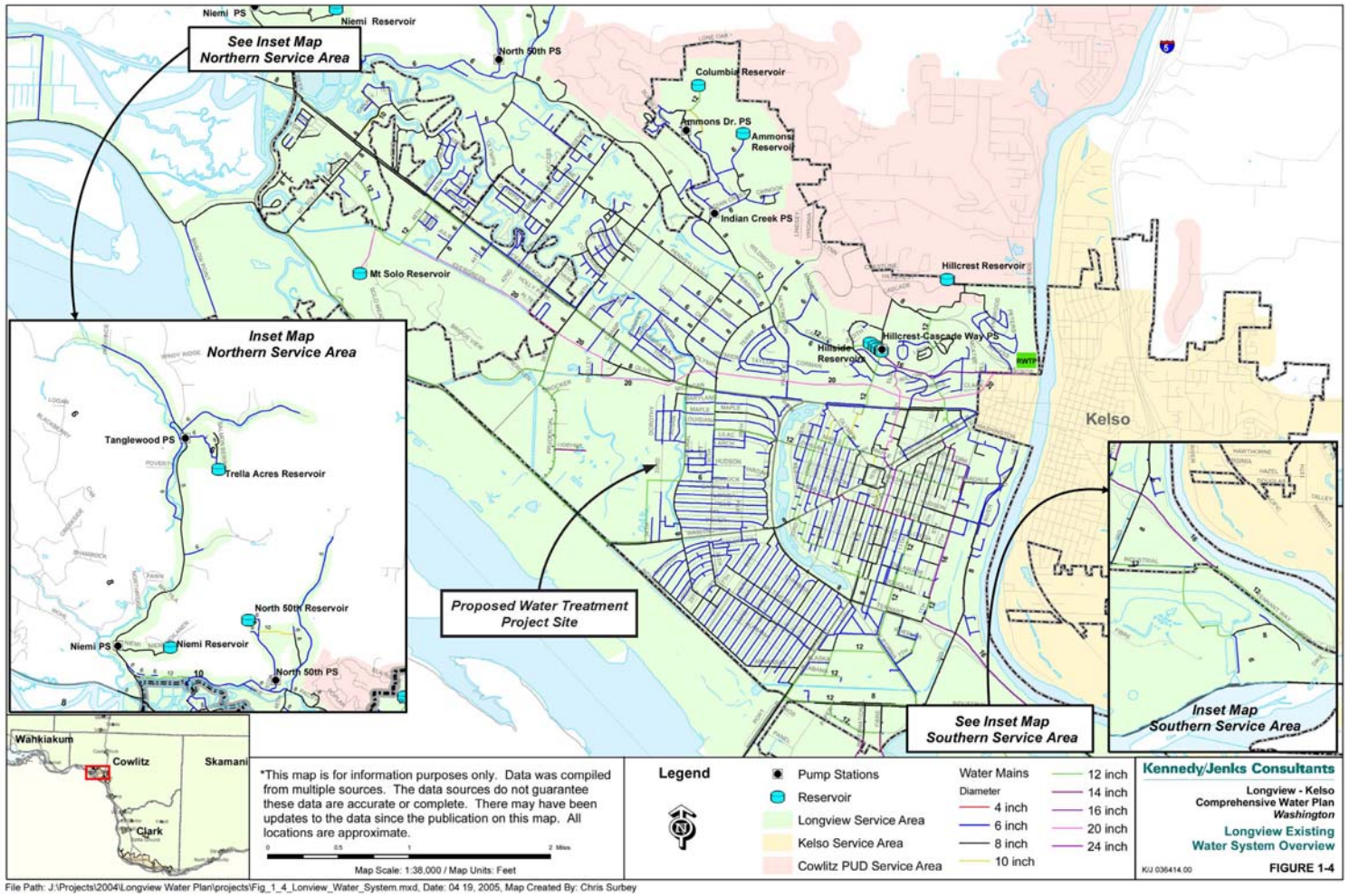


Figure 8.2: WTP Connection to Existing System

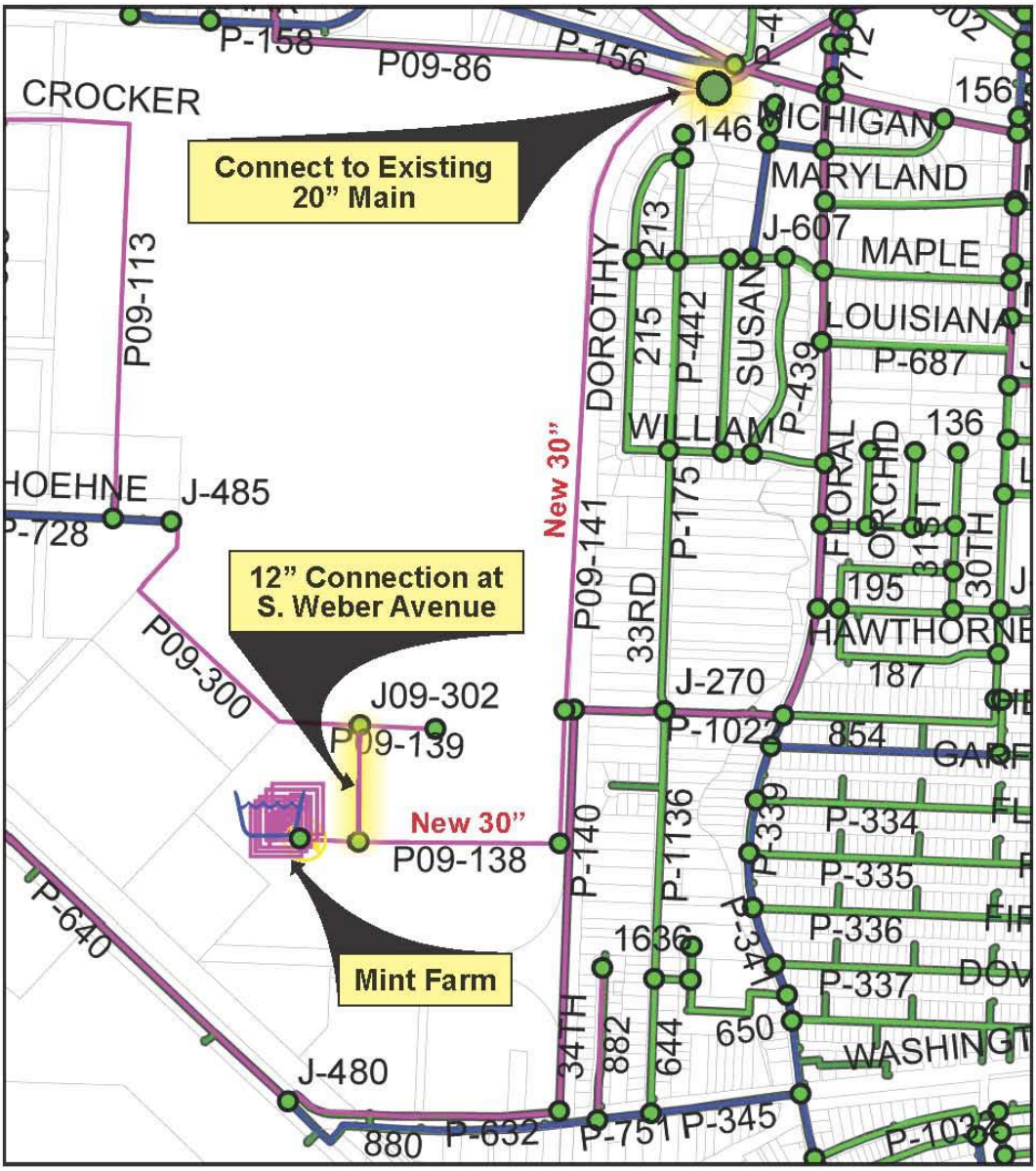


Table 8.2: New Distribution Main Characteristics Year 2029

Key Pipeline	Diameter	Velocity ft/sec	Junction I.D.	Location	Junction Pressures Year 2029, MDD (psi)	Junction Pressures Year 2009 (psi)
P-09-138	30”	5.3	5-270	33rd and Hemlock	110	98
P-09-141	30”	4.0	509-302	S. Weber	112	100
P-09-300	12”	3.7	J-116	34th and Washington	108	98
			J09-201	Connerview to existing 20”	108	102

Currently the pressures in the local area east of the new Mint Farm RWTP are about 98 to 100 psi. The new treatment plant will increase pressures in this area to a projected maximum pressure of about 109 psi at year 2029. This represents about a 10 psi increase over the current pressure. It should be remembered that this is the MDD in year 2029. Pressures in this local area will be substantially less during periods of ADD up until the year 2029. Conversely, the pressures around the existing RTWP will decrease significantly due to the absence of the large pumps required to push water from the RWTP to the Hillside Reservoirs. As reported in the 2005 Water Master Plan, pressures around the existing regional water treatment plant exceed 110 psi on a regular basis. With the new Mint Farm RWTP online, pressures around the existing water treatment plant will decrease from over 116 psi to under 90 psi. Table 8.2 summarizes the pertinent modeling results.

An added benefit of the location of the Mint Farm RWTP is that it is more centrally located between the Hillside and Mt. Solo reservoirs. The production from the new RWTP will enter the distribution systems primary 20-inch main at about the middle of its run rather than at the far easterly location as is currently provided by the existing RWTP. This has the effect of more uniformly providing distribution to both the Hillside and Mt. Solo reservoirs. After year 2029, additional flows will be realized from the Mint Farm RWTP. However, incidental information gathered as part of the hydraulic model suggests that improving the distribution system east of the Weyerhaeuser railroad tracks would accommodate future flows.

8.1.2 Surge Analysis

The potential for damaging surges is possible wherever large flows are transmitted through a single transmission main. To prevent this, surge protection will be included in the design of the new treatment plant. It is the policy of Kennedy/Jenks Consultants to use surge protection tanks rather than surge valves in the design of new systems. It is also our experience that potentially damaging surges are not properly anticipated by the use of valving facilities. Therefore, a large pressure tank will be used to absorb any surges within the system. A stand alone surge analysis will be prepared during the design phase of this project in order to properly size the surge protection tank. In the interim, the Basis of Design Report will include a surge facility as part of the overall plan and will be included in the cost estimate.

8.1.3 Water Quality Impacts

It is anticipated that the City’s current flushing program will need to be revised to accommodate start-up of the Mint Farm RWTP. During system start-up, reverse flows and minor changes in

water quality may result in some deposition of materials or removal of materials from the pipe linings. During the design phase of this project, the existing hydraulic model will be updated in order to analyze water quality or water age parameters based upon additional information to be provided by the City along with chlorine concentration and decay rates from which a water age analysis will be conducted to facilitate start up. As mentioned previously, at the start up, flushing will be managed as a result of this modeling effort through the incorporation of a start-up plan. The water quality start-up and transition plan will be prepared well in advance of the final construction of the treatment plant. The water quality transition plan is expected to be ready for review by the City and the State a few months prior to completion of the treatment plant. This is expected sometime in year 2011 or early 2012. By then, additional distribution system parameters and data regarding water temperature, water quality, and actual demand patterns will be more firmly established.

8.2 Transmission Pipeline

8.2.1 Preliminary Pipe Alignment

In general, the pipeline alignment will be routed east from the cul-de-sac on Weber Avenue approximately 1,500 feet, towards the railroad tracks owned by Weyerhaeuser. From this location, the transmission main will be installed parallel to and west of the railroad tracks, in an easement within the Weyerhaeuser right-of-way. The approximate length of the alignment parallel to these tracks is 6,000 feet. Therefore, the total length of the transmission pipeline is approximately 7,500 feet from the new RWTP site to the existing transmission system.

8.2.2 Hydraulic and Geotechnical Considerations

Open-trench construction is the fastest and least expensive method of installing pipelines. In addition, it presents very little risk of additional costs due to unforeseen circumstances. A hydraulic excavator with sufficient reach should be adequate to excavate the proposed trenches. Wood debris and logs are often encountered in the alluvial deposits anticipated, and the contractor should be prepared to handle and dispose of such oversized material. Upon reaching the trench bottom, a smooth-bladed bucket should be used to clean the trench bottom of loose and/or disturbed soil. Trench excavations must be sloped or provided with temporary shoring.

The Geotechnical Report (Landau 2009) indicates that excavations for test pits conducted by others in the Mint Farm Industrial Park encountered groundwater seepage at depths ranging from 4 to 6 feet below ground surface (bgs). Further, the groundwater level could rise to near the existing ground surface during the winter and early spring months. As such, dewatering will likely be necessary during installation of the transmission pipeline. The Geotechnical Report indicates that maximum drawdowns of 6 to 8 feet bgs will be required to maintain groundwater levels at least 2 feet below the trench bottom during summer construction. During the winter and early spring months, maximum drawdowns may be as much as 12 feet.

In accordance with City Standard Plan W-000, the transmission pipe will have a minimum cover of 36 inches over the top of the pipe. With a diameter of 30 inches, a trench approximately 5 feet wide by 6 to 7 feet deep is anticipated. Trenches will be excavated and backfilled in accordance with the applicable City Standard Plans ST-010, ST-020, and ST-040. The transmission main will be designed with air release valves located at the high points (in accordance with City Standard Plan W-190) and blow-offs at the low points (in accordance with City Standard Plan W-170).

Section 9: Preliminary Project Cost Estimate

9.1 Estimate of Probable Construction Cost

Conceptual estimates of probable construction cost for the proposed MFRWTP are summarized below. The estimate of probable construction cost includes the following factors:

- 6 percent (approximately) for project-specific overhead costs, including mobilization, supervision, bonds, insurance, etc.
- 15 percent for contractor overhead and profit.
- 7.9 percent for sales tax on materials.
- 10 percent allowance for design contingency on vendor quotations received for major equipment items (i.e. pressure filters, well pumps, etc).
- 30 percent allowance for design contingency for all other project work. This includes work items not specifically detailed on the individual line items in the cost estimate (i.e. electrical wiring, small diameter pipe, valves, and appurtenances, etc).

Probable project cost may be determined by adding the following factors to the estimate of probable construction cost:

- 30 percent for engineering, construction management, legal/administrative costs, etc.
- 10 percent owner contingency to cover unforeseen costs occurring during construction.

However, more specific estimates of City programmatic costs are given in Section 3.

The estimate of probably construction and probably project costs do not include a deduction for the very competitive bid climate that is likely to continue for some time.

The accuracy of this estimate of probable cost should be considered to be within -15 percent to +30 percent of the total cost shown, consistent with a preliminary estimate (Estimate Class 4) as defined by the Association for the Advancement of Cost Engineering (AACE).

Table 9.1: Estimate of Probable Construction Cost and Probable Project Cost

Cost Breakdown	Estimate of Probable Cost
Incidentals (Mobilization, Bonds, Insurance, etc)	$1,500,000
General Site Work and Yard Piping	$1,786,900
Groundwater Wells and Well Buildings	$1,902,000
Pressure Filters and Filter Gallery Building	$5,565,329
Backwash Storage Tanks	$2,295,760
Office/Treatment Building	$1,101,228
Solids Dewatering and Drying System	$85,966
Transmission Main	$1,272,218
Electrical, Instrumentation, and Controls	$1,308,763
Subtotal	$16,818,965
Contractor Overhead and Profit at 15%	$2,522,845
Subtotal	$19,341,809
Taxes at 7.9% (Materials and Labor)	$1,528,003
Subtotal	$20,869,812
Engineering Allowance (on all except major equipment) at 30%	$4,419,877
Engineering Allowance (on major equipment) at 10%	$613,689
Subtotal	$25,903,378
Escalation to Midpoint of Construction (Jan 2012) at 4.0%	$1,036,135
Estimate of Probable Construction Cost	**$26,940,000**
Engineering Fees, Admin/Legal, Construction Management at 30%	$8,100,000
Recommended Owner Contingency at 10%	$2,700,000
	($37,740,000)
Estimated Range of Probable Project Cost	**$31,000,000 to $49,000,000**

Notes:

(a) Range of probable project cost based on AACE estimate accuracy of -15% to +30%.
(b) Does not include property acquisition costs

Refer to Appendix F for additional detail on this cost estimate.

9.2 Construction Schedule, Site Access and Staging

Construction of the proposed facility is expected to take between 22 and 26 months. Construction of the transmission main is expected to take between 4 and 8 months.

Preloading of site will need to commence at least 6 months before excavation begins. Connections between the new transmission main and the existing distribution system could be made using a tapping saddle to reduce service interruptions.

Staging of contractor's facilities, laydown areas, and equipment storage should be limited to spaces outside of the wellhead protection areas. Construction trailers and onsite equipment storage could be staged at the proposed location of the dewatering beds during construction of most of the facilities. The S. Weber Avenue extension road construction is expected to be completed prior to the start of construction of the proposed facilities and thus we anticipate the site will be accessible from paved public roads. Designated gravel construction entrances and wheel washers should be considered for the site to reduce soil deposition on public roads.

Access and staging for construction of the transmission main will be explored in detailed design.

9.3 Water Supply Transitioning

A complete start-up plan for introduction of finished water from the new MFRWTP facility into the distribution system concurrent with decreasing output from the existing Cowlitz River RWTP will be prepared as part of the construction activities. The project specifications will require the Contractor to submit a start-up plan for transitioning the finished MFRWTP water into the distribution system and will be required to show careful planning and coordination with City staff, DOH, design consultants, and construction management. No transitioning activites will be allowed to start until the start-up plan is complete and approved. As a minimum, City management, DOH representatives, Beacon Hill, and Cowlitz PUD staff, and the design consultant should review the start-up plan and submit comments to the City's construction manager for the project.

9.4 Alternative Delivery Assessment

Alternative project delivery methods were considered when developing the project scope. Subsequently, in discussion with the City, the merits of alternative delivery were found to be not favorable nor compelling for this project and the current municipal construction industry market. Alternative project delivery will not be considered further in preliminary design activities for this project, but may be revisited again should the construction market or national financial outlook change before this project goes out for public bid. Given this constraint, it is unlikely that design-build alternatives will be considered at any time, it is possible that GCCM delivery could be considered should market changes manifest relatively soon.