City of Snohomish

General Sewer Plan and Wastewater Facilities Plan

DRAFT

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Executive Summary

This *General Sewer Plan and Wastewater Facilities Plan* (Plan) for the City of Snohomish (City) updates the *2010 City of Snohomish General Sewer Plan and Wastewater Facilities Plan Update* (2010 Plan) and the 2005 *General Sewer Plan and Wastewater Facilities Plan* (2005 Plan). This Plan addresses the City's sewer system and wastewater treatment needs for the projected sewer flows and loads through 2036.

The Washington State Growth Management Act (GMA) requires the City to provide sanitary services for the population growth that will occur over the next several decades. The City's rate of population growth over the past decade has been significantly lower than the rate forecast in 2005 for the current GMA comprehensive planning cycle. Reflective of the slow growth rate countywide over the first half of the current planning cycle and of more recent regional policies on population allocation, the Snohomish County Council adopted amendments to the Countywide Planning Policies (CPPs) in 2013 that revised the current growth target for the City and its urban growth area from a population of 14,535 in 2025 to one of 14,494 in 2035[1, 2]. Expansion of the current urban growth area (UGA) boundary, previously considered a near-term goal that would have the effect of increasing the City's population and employment capacity, has been deferred due to policy decisions at the County level. As a result, the currently permitted capacity of the wastewater treatment plant (WWTP) is projected to be sufficient through 2022. A capacity analysis of the WWTP showed that the plant itself has the capacity to treat all the flows throughout the planning horizon with just a few improvements. The permitted capacity will need to be increased to reflect the larger capacity available. If the City Council adopts a higher population target than that formalized in the CPPs or if opportunities to achieve the previously considered UGA expansion are realized within the planning horizon of this Plan, potential increases in required treatment and conveyance capacities must be evaluated and addressed.

ES.1 Introduction (Chapter 1)

This Plan provides a summary of the City's current sewage capacities; an analysis of the impact of projected growth on the City's sewage collection, conveyance, and treatment systems; and proposes a Capital Improvement Program (CIP) to alleviate system deficiencies. It also documents the City's financial plan to implement the CIP.

The City's current wastewater system service area is approximately 1,900 acres. The City owns, operates, and manages wastewater collection and treatment facilities. The City's wastewater collection facilities include gravity sewers, sewer force mains, and pump stations that convey wastewater to the City's WWTP. The WWTP is a four-stage lagoon treatment system, which was retrofitted with a submerged fixed-film (SFF) media system in 2012. The WWTP also includes influent pumping and flow measurement, screening, addition of supplemental alkalinity, effluent filtration, and chlorine disinfection followed by dechlorination. Treated wastewater is discharged to the Snohomish River. Biosolids that collect in the lagoons are periodically dredged, dewatered and land applied at a beneficial use facility (BUF).

Over the next 20 years, the City's sewered residential population is expected to grow to over 14,500 people. The City's sewer service area is expected to grow by approximately 700 acres.

[1] Snohomish County Ordinance No. 13-032, effective 30 June 2013.
[2] Snohomish County Ordinance No. 16-078, effective 12 October 2016

This Plan evaluates the collection and treatment facilities ability to accommodate both existing and future wastewater collection and treatment needs.

This Plan complies with the Washington State Department of Ecology (Ecology) requirements for General Sewer Plans (Washington Administrative Code [WAC] 173-240-050) and Wastewater Facilities Plans (WAC 173-240-060) as shown in Table ES-1 and ES-2, respectively.

Table ES-1 General Sewer Plan Requirements per WAC 173-240-050

WAC 173-240-050 Reference Paragraph	Description of Requirement	Location in Plan
3a	Purpose and need for proposed plan	Section 1.2
3b	Who owns, operates, and maintains the wastewater system	Section 1.4
3c	Existing and proposed service boundaries	Chapter 3
3d	Layout map showing boundaries; existing sewer facilities; proposed sewers; topography and elevations; streams, lakes; and other water bodies; water systems	Figure 2-2, Figure 2-3, Figure 2-4, Figure 6-1, Figure 12-1
3e	Population trends	Section 3.2.3
3f	Existing domestic and/or industrial wastewater facilities within 20 miles	Figure 1-1
3g	Infiltration and inflow problems	Section 5.3
3h	Treatment systems and adequacy of such treatment	Chapter 9
3i	Identify industrial wastewater sources	Section 5.1.1
3j	Discussion of water systems	Section 2.7
3k	Discussion of collection alternatives	Chapter 7
3l	Define construction cost and O&M costs	Chapters 9 and 12
3m	Compliance with water quality management plan	Sections 2.4.1 and 4.2.1
3n	SEPA compliance	Appendix A

Table ES-2 Wastewater Facilities Plan Requirements per WAC 173-240-060

WAC 173-240-060 Reference Paragraph	Description of Requirement	Location in Plan
3a	Who owns, operates, and maintains the wastewater system	Section 1.4
3b	Location map and existing and proposed service boundaries	Figure 1-1, Chapter 3
3c	Expected future wastewater quantity and quality	Chapter 8
3d	Degree of treatment required	Chapters 4 and 9
3e	Description of receiving water and water quality standards	Section 4.2
3f	Proposed treatment processes and discussion of alternatives evaluated	Chapter 11
3g	Design data and sizing calculations of unit processes	Chapters 9 and 11
3h	Discussion of various treatment plant sites	Not applicable. Per Chapter 9, there is an existing treatment plant.
3i	WWTP flow diagram, layout and hydraulic profile	Figures 9-1, 9-2 and 9-3
3j	Discussion of infiltration/inflow and overflows	Chapter 7
3k	Discussion of special provisions for treating industrial waste	Not applicable. Per Section 5.1.1, there are no significant industrial sources in the service area.
3l	Outfall analysis	Section 9.4.9
3m	Discussion of sludge disposal	Section 9.6
3n	Provisions for future needs	Chapters 9 and 12
3o	Staffing and testing requirements	Section 9.7
3p	Estimated capital, operation and maintenance costs associated with proposed improvements	Chapters 11 and 12
3q	Compliance with applicable water quality management plans	Sections 2.4.1 and 4.2.1
3r	SEPA compliance	Appendix A

ES.2 Background (Chapter 2)

The City is located approximately 25 miles northeast of Seattle, Washington and 8 miles southeast of Everett, Washington, in the south-central portion of Snohomish County. The City was founded in 1858 and was originally known as Cadyville. The name was changed to Snohomish in 1871. The City is primarily residential with some commercial areas.
Water resources in the Snohomish area include the Snohomish and Pilchuck Rivers in the south and east, Cemetery and Harkins Fork Creeks in the west, Bunk Foss Creek in the north and Swifty Creek, Blackmans Lake, and Blackmans Lake Creek in the north and central portions. Flooding in the City by the Snohomish and Pilchuck Rivers is confined primarily to the floodplains in the southeastern part of the City. The majority of the City lies above the Snohomish and Pilchuck River 100-year flood plains as defined by Federal Emergency Management Agency (FEMA); therefore, it is generally not impacted by river flooding. The City's WWTP and Combined Sewer Overflow (CSO) Pump Station are located in the flood plain and have occasionally been isolated by flooding events during storms (2006 and 2009), though the WWTP site is built at an elevation above the 100-year flood level. The predominant soil group in the study area is classified by the U.S. Soil Conservation Service as Tokul gravely loam overlaying glacial till. The groundwater table is generally four feet or more below the surface during the summer.

The City operates a potable water system for the majority of its residents. The water system consists primarily of a storage and distribution system. The distribution system is separated into three service area zones defined by elevation. Until recently, water to the lower zone had been fed from a water treatment plant on the Pilchuck River that was owned and operated by the City. The City has discontinued use of the plant and plans to decommission it. Now, all zones are served from four connections to the City of Everett transmission line.

ES.3 Land Use and Population (Chapter 3)

The projected sewered population for the City over the planning horizon of this Plan is presented in Table ES-3. The build-out scenario is for modeling purposes only and does not reflect population growth goals or constraints.

Table ES-3 Service Area Population and Employment Estimates

Year	Residential Population	Employment Population
2016	9,569	4,677
2022	10,348	5,027
2036 (Current City Limits)[(a)]	12,358	6,111
2036 (UGA and Portion of North Planning Area)[(b)]	14,505	6,223
Buildout	17,995	7,236

Note:
(a) These 2036 projections only consider serving the current City limits.
(b) These 2036 projections include serving the current City limits, the entire UGA (excluding the portion south of the Snohomish River) and the portion of the North Planning Area south of State Route 2.

ES.4 Permits, Requirements, and Regulations (Chapter 4)

The City manages and operates their sewer system in accordance with state, local, and federal regulations. The permitting requirements and regulations described in Chapter 4 of this Plan provide a framework for the planning, design, operation, and management of the system to maintain the desired level of service to sewer utility customers. Requirements and regulations discussed in Chapter 4 that are applicable to the proposed design and operation of the sewer system include the following:

- Federal regulations
  - Clean Water Act
  - Water Pollution Control Act
  - Standards for Use or Disposal of Sewage Sludge
  - U.S. EPA Reliability Criteria
  - National Environmental Policy Act
  - Floodplains, Wetlands, and Flood Insurance
  - Coastal Zone Management
  - Magnuson-Stevens Fishery Conservation and Management Act
  - Public Participation
- State Policies and Regulations
  - Water Quality Standards for Surface Waters
  - Combined-Sewer Overflow Regulations
  - National Pollutant Discharge Elimination System (NPDES) Permit
  - Washington Department of Ecology Criteria for Sewage Works Design
  - Joint Aquatics Resources Permit
  - State Environmental Policy Act (SEPA)
- Local Policies
  - Fire Code
  - City Service Area Policies
  - Shoreline Management Program and Substantial Development Permit
  - Building Codes
  - County Requirements for Archeological Sites
  - SEPA Review
  - Critical Areas Review

ES.5 Collection System Design Criteria and Flows (Chapter 5)

The unit and projected flows used to model the City's collection system are presented in Table ES-4.

Table ES-4 Unit and Projected Wastewater Flows

Parameter	2016	2022	2036 (Current City Limits)	2036 (UGA and Portion North Planning Area)	Buildout
Sewered Residential Population	9,569	10,348	12,358	14,505	17,995
Residential Flow (gpcd)[a]	60.0	60.0	60.0	60.0	60.0
Employment Population	4,677	5,027	6,111	6,223	7,236
Employment Flow (gped)[b]	14.3	14.3	14.3	14.3	14.3
Average Dry Weather Flow (MGD)[c]	0.64	0.69	0.83	0.96	1.18
Average Annual Flow (MGD)	1.30	1.40	1.68	1.95	2.40
Maximum Month Flow (MGD)	2.73	2.80	2.95	3.22	3.44
Peak Day Flow (MGD)	8.30	8.10	8.50	9.42	9.58
Peak Hour Flow (MGD)	14.38	13.67	14.36	14.74	16.88

Note:
(a) gpcd is gallons per capita per day.
(b) gped is gallons per employee per day.
(c) MGD is million gallons per day.

ES.6 Collection System Description (Chapter 6)

The City's collection system consists of gravity conveyance, force mains, and pump stations. The gravity sewer system can be divided into two categories: the combined sewer system (CSS) (stormwater and sanitary flow) and the separated collection system (sanitary flow only). The CSS serves most of the downtown and much of it is over 100 years old. There is currently approximately 46,500 lineal feet of combined sewer pipe ranging in diameter from 6 to 30 inches. The City is currently working to separate portions of the CSS by installing stormwater pipes which will discharge to the Snohomish River near the WWTP. The separated system consists of approximately 140,000 linear feet of PVC and concrete sewer pipe ranging in diameter from 6 to 18 inches. Most of the separated system pipes were installed prior to 1950 except for the west trunk that was installed around 2011. The collection system includes 15 pump stations and 2.5 miles of force mains (ranging from 3-inch to 18-inch). The City's pump stations are equipped with Cornell, Flygt, Gorman Rupp, Gardner Triangle or Zoeller submersible pumps.

ES.7 Collection System Analysis (Chapter 7)

The existing wastewater conveyance system was analyzed using the xpSWMM modeling platform. The projected populations and their distributions are the basis for establishing future system requirements. ArcGIS files were imported into the model with the rim elevations, invert elevations, and pipe diameters that were available. Where necessary, gaps in the data were filled using as-built drawings, field surveys, topographic maps, and the City's most recent Wastewater System Map.

The City's conveyance system was analyzed using a somewhat truncated model, simulating all of the main trunks, most of the pipes in the separated sewer system area, inputs for each street in the CSS, and the larger pump stations. Only the main trunks in the CSS were modeled since it had been recently modeled in EPA SWMM 5 for the CSO Management Plan with no capacity constraints found. The model can be updated and expanded in the future, as needed.

Deficiencies in the collection system are defined as:

- Gravity Pipe Segments: Surcharging depth within five feet of the rim of the manhole, or equal to the pipe diameter (d/D of 1) for manholes less than 5 feet deep.
- Force Mains: Flow velocity exceeds 8 feet per second (fps).
- Pump Stations: Firm capacity is exceeded causing the backup pump to turn on.

When model simulation results exceed these criteria in piping or in lift stations, they are identified as deficient and system improvements are identified to resolve them. Where pipe sections were identified as requiring an upgrade, the proposed upgrade was sized to provide capacity equal to, or greater than, the estimated 2036 flows according to the criteria above. Under buildout conditions, there did not appear to be additional capacity deficiencies with the implementation of the recommended west trunk diversion (CIP C-6A). This is the case even with conservative flow projections that did not consider separation in the CSS area beyond Avenue F. It is expected that prior to buildout all of the CSS area would be separated.

ES.8 Treatment Plant Flow and Load Evaluation (Chapter 8)

The WWTP capacity was evaluated at projections for 2022 and 2036 to determine its ability to treat the incoming wastewater at the predicted loadings while meeting the effluent limitations. Flow projections from the xpSWMM model were used as inputs in the WWTP capacity evaluation. Waste load projections were calculated using estimated loading factors and population projections.

Waste load factors (in terms of pounds per capita per day [ppcd]) for influent 5-day biochemical oxygen demand (BOD_5), total suspended solids (TSS), ammonia-nitrogen (ammonia-N) and total Kjeldahl nitrogen (TKN) were estimated based on discharge monitoring report (DMR) data from 2013 through 2016 and current residential population. These factors were applied to the projected residential population to develop waste load projections for BOD_5, TSS, ammonia-N and TKN in terms of ppd at the WWTP. The loading from the employment population is incorporated into these estimated loading factors, since the factors are based on the total historical loadings into the WWTP. Because the current and projected future ratios of residential to employment populations remain nearly the same, as shown in Table ES-3, only the residential population was used to determine the loading factors and project future loads for simplicity. A summary of the design criteria, current, and projected flows and loadings at the WWTP is shown in Table ES-5.

Table ES-5 Design and Projected Influent Loads

Component	Design	2016	Projected	
			2022	2036
Sewered Residential Population[(a)]	NA	9,569	10,348	14,505
Flow, MGD				
Average Annual	2.04	1.30	1.40	1.95
Maximum Month	2.80	2.73	2.80	3.22
Maximum Day	9.90	8.30	8.10	9.42
Peak Hour	22.80	14.38	13.67	14.74
BOD, ppd				
Average Annual	2,892	1,996	2,160	3,030
Maximum Month	3,960	2,724	2,950	4,130
TSS, ppd				
Average Annual	3,233	1,772	1,920	2,690
Maximum Month	4,400	2,333	2,520	3,540
Total Kjeldahl Nitrogen (TKN), ppd				
Average annual	348	276	300	420
Maximum Month	467	362	390	550
Ammonia-N, ppd				
Average annual	226	161	170	240
Maximum Month	290	194	210	290

Note:
(a) The loading from the employment population is incorporated into the residential loading since the residential loading factors are based on the total historical load and employment population is projected to grow at approximately the same rate as the residential population.

ES.9 Existing Wastewater Treatment Facility (Chapter 9)

The process capacity and hydraulic capacity of the WWTP were assessed using the values in Table ES-5. The process capacity was assessed by comparing typical operating ranges for various treatment processes to determine if the facility operates within the typical range. A hydraulic profile of the WWTP was also developed to assess the hydraulic impacts of the projected flows. The WWTP evaluation resulted in the identification of improvements that were categorized either as being O&M, redundancy, or capacity related. Major improvements include upgrade and expansion of effluent filtration, replacing the existing chlorine gas disinfection system, replacing and upgrading the existing floating surface aerators in Lagoon 1, retrofitting the influent screw pumps with variable frequency drives, and replacing three of the four existing motor control centers (one having already been replaced recently).

ES.10 Regional Wastewater Treatment Alternatives (Chapter 10)

The City had previously pursued regional treatment with the City of Everett. The City retained CH_2M HILL to prepare the 2010 Plan, which determined that the total capital cost of regionalization would be approximately $43 million, including an estimated $20 million to purchase capacity at the Everett WPCF. The 2010 Plan projected a City population of

approximately 17,554 at the end of the previous planning horizon (2024) in the 2005 Plan, which included population growth from UGA expansion. However, the 2010 Plan noted that the economic recession might slow the expected population growth.

Because of the extensive time needed to plan, design, and construct the regionalization project, the City to proceeded with near-term improvements of the existing WWTP to ensure compliance with its National Pollutant Discharge Elimination System (NPDES) permit. Due to the success of the near-term improvements, plans for a large UGA expansion being abandoned, and the City's growth rate being much flatter than estimated in the 2010 Plan, the plan for regionalization was postponed indefinitely. The City has no intention of pursuing regionalization further.

ES.11 Wastewater Treatment Facility Improvements (Chapter 11)

Multiple viable alternatives were identified and evaluated for upgrading and expanding effluent filtration, replacing the existing chlorine gas disinfection system, and replacing and upgrading the existing floating surface aerators in Lagoon 1 to determine preferred alternatives for implementation.

Alternatives evaluated for upgrading and expanding effluent filtration include:

- Expansion of Existing DynaSand® Sand Filters
- EcoWash™ Sand Filters
- Compressible Media Filters
- Disk Filters

Alternatives evaluated for replacing the chlorine gas disinfection system include:

- Delivered Sodium Hypochlorite Trade Solution
- Onsite Sodium Hypochlorite Generation System
- UV Disinfection
- Delivered Peracetic Acid Solution System

Alternatives evaluated for replacing the floating surface aerators in Lagoon 1 include:

- Floating Surface Aerators
- Floating Aspirating Aerators
- Diffused Aeration
- Venturi Injector

Based on life-cycle cost evaluations and consideration of non-cost factors (e.g., performance, safety, footprint, complexity, expandability and pretreatment requirements), the recommended alternatives are as follows:

- Replacing the existing DynaSand® sand filters with two new compressible media filters.
- Replacing the chlorine gas disinfection system with a peracetic acid system
- Replacing the floating surface aerators in Lagoon 1 with new floating surface aerators

ES.12 Recommended Improvements (Chapter 12)

The primary goal of this Plan is the preparation of the 6-year (2020 thru 2025) and Long-Term (2026 thru 2036) CIP. The 6-year and 20-year CIPs are provided in Table ES-6 and Table ES-7, respectively. Actual costs can and will differ from the opinions of probable project costs. Changes in scope during design, volatility in the bidding climate, the number of contractors bidding on a project, and their approach to bidding and completing the work will all impact actual project costs.

Table ES-6 Opinion of Probable Project Costs, 6-Year CIP

CIP No.	Project	Opinion of Probable Project Cost
C-1	Capital Reserve	$2,000,000
C-2	General Conveyance System Repair and Replacements (2020-2025)	$2,365,000
C-3	CSO 2 Separation Program	$2,315,000
C-4	Emerson Street Utility Improvement	$100,000
C-5	Upsize Gravity Main at 13th Street and Avenue A	$1,050,000
C-6	Western Gravity Flow Diversion (Planning)	$119,000
P-1	Champagne Pump Station (PS 7) Upgrade and New Force Main	$1,460,000
P-2	Kla-Ha-Ya Pump Station (PS 11) Decommissioning	$42,000
P-3	Mobile 80 kW Generator	$64,000
P-4	Rainier Pump Station (PS 2) Generator (100kW)	$580,000
P-5	Rainier Pump Station (PS 2) Upgrades	$760,000
W-1	WWTP NPDES Re-Rating	$75,000
W-2	Upgrade/Expand Effluent Filtration	$2,320,000
W-3	General WWTP Improvements (2020-2025)	$2,365,000
W-4	Replace the Chlorine Gas Disinfection System	$1,290,000
W-5	Add Non-Potable Water Air Gaps	$500,000
	Total Opinion of Probable Project Cost	**$17,405,000**

Table ES-7 Opinion of Probable Project Costs, Long-Term CIP

CIP No.	Project	Opinion of Probable Project Cost
C-6	Western Gravity Flow Diversion	$2,981,000
C-7	Rainier Pump Station (PS 2) Force Main Replacement	$760,000
C-8	General Conveyance System Repair and Replacements (2026-2037)	$8,115,000
C-9	Interurban Trail Conveyance System Upgrade	$1,019,000
C-10	CSO 1 Separation Program	$4,500,000
C-11	CIPP Pipe Restoration North of Blackmans Lake	$451,000
C-12	Pipe Extension North of the Casino Pump Station (PS 14)	$1,479,000
C-13	New Gravity Line and Decommissioning the Ferguson Pump Station (PS 8)	$2,088,000
P-6	Lincoln Pump Station (PS 3) Replacement	$390,000
P-7	Rainier Pump Station (PS 2) Wet Well Replacement/Expansion	$230,000
W-6	Replace MCCs and Retrofit the Influent Screw Pumps with VFDs	$3,490,000
W-7	Biosolids Removal	$800,000
W-8	General WWTP Improvements (2026-2037)	$8,115,000
	Total Opinion of Probable Project Cost	**$34,418,000**

Implementation of the projects identified in Table ES-6 and Table ES-7 will allow the City to address the capacity, obsolescence, O&M, and redundancy limitations identified within the City's wastewater system based on the projected flows and loads over the planning horizon.

The opinions of probable cost herein are based on perception of current conditions at the project location. This opinion reflects a professional opinion of construction costs at this time and is subject to change as the project design matures.

ES.13 Financial Plan (Chapter 13)

The financial plan prepared by FCS Group summarizes the finances of the sewer utility, including three-year history, outstanding debt, capital funding sources and a financing plan for the capital improvements identified in Chapter 12. The impact on monthly residential sewer rates is estimated to be $$$$ over the next six years, based on the 6-year CIP schedule included in this Plan. This impact will increase the current monthly rate of $$$$ to $$$$ from 2020 through 2025.

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Chapter 1 Introduction

1.1 Introduction

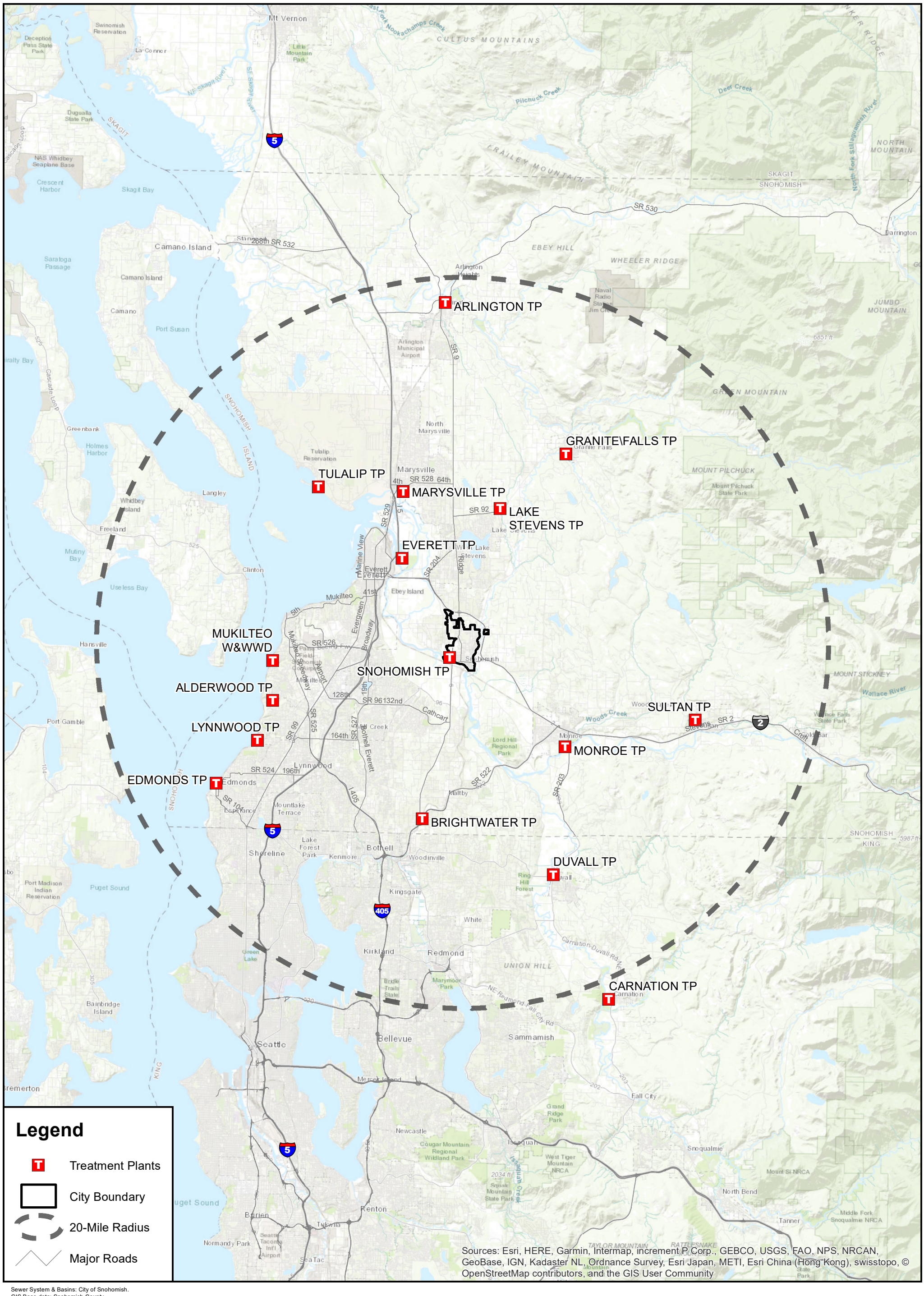
The City of Snohomish (City) is located in western Snohomish County and is bounded to the south by the Snohomish River. The location is shown in Figure 1-1. The surrounding area is a combination of rural and suburban lands in unincorporated Snohomish County.

The City was founded in 1858 and was originally known as Cadyville. The name was changed to Snohomish in 1871. The City is primarily residential with some commercial areas and light industrial activity. The 2016 residential population in the City and urban growth area (UGA) was estimated to be 11,095 based on County land use data, the 2010 Census population by block, and other data is discussed in Chapter 3.

The City's current wastewater system service area is approximately 1,900 acres. The City owns, operates, and manages the wastewater collection and treatment facilities. The City's wastewater collection facilities include gravity sewers, sewer force mains, and pump stations that convey wastewater to the City's wastewater treatment plant (WWTP). The heart of the WWTP is a four-stage lagoon treatment system, which was retrofitted with a submerged fixed-film (SFF) media system in 2012. The WWTP also includes influent pumping and flow measurement, screening, addition of supplemental alkalinity, effluent filtration, and chlorine disinfection followed by dechlorination. Treated wastewater is discharged to the Snohomish River. Biosolids from the wastewater treatment process are occasionally dredged from the lagoons and land applied at a beneficial use facility (BUF).

The City's sewered residential population is expected to grow from 9,569 people in 2016 to 12,358 people in 2036 if there is no sewer expansion into the North Planning Area north of State Route 2 (but including the small section of the North Planning Area south of State Route 2). However, if there is expansion into the entire North Planning Area, the sewered residential population is expected to grow to 14,505 residents. The City's sewer service area is expected to grow by approximately 700 acres if there is expansion into the North Planning area north of State Route 2 for a total of about 2,600 acres. This General Sewer Plan and Wastewater Facilities Plan (Plan) evaluates future collection and treatment facilities required to accommodate both existing and future wastewater collection and treatment needs based on current and future service populations.

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Sewer System & Basins: City of Snohomish.
GIS Base data: Snohomish County.
Data sources supplied may not reflect current or actual conditions. This map is a geographic representation based on information available. It does not represent survey data. No warranty is made concerning the accuracy, currency, or completeness of data depicted on this map.
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City of Snohomish Washington

Vicinity Map

General Sewer Plan and
Wastewater Facilities Plan
City of Snohomish, Washington
February 2019

Figure
1-1

P:\Mapping\Maps_Generated\Snohomish_City\19-10615.00\003\maps\Fig 1-1 Vicinity Map - 11x17.mxd 2/19/2019 ctolentino

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1.2 Purpose

This Plan replaces the *2010 City of Snohomish General Sewer Plan and Wastewater Facilities Plan Update* (2010 Plan) and the *2005 General Sewer Plan and Wastewater Facilities Plan* (2005 Plan), which was amended by the 2010 Plan. This Plan is prepared for the City to fulfill the requirements of Chapters 173-240-050 and 173-240-060 of the Washington Administrative Code (WAC), Chapter 90.48 of the Revised Code of Washington (RCW), and RCW 36.70A (Growth Management Act). The City can use this Plan as a guide for managing and operating the sewer system and coordinating expansions and upgrades to the infrastructure through the planning horizon. It also serves as a guide for policy development and decision-making processes for the City. The WAC requirements for a general sewer plan and wastewater facilities plan are outlined in Table 1-1 and Table 1-2, respectively. The Plan provides the public and regulatory agencies with information on the City's plans for sewer system extensions to areas designated as urban.

Table 1-1 General Sewer Plan Requirements per WAC 173-240-050

WAC 173-240-050 Reference Paragraph	Description of Requirement	Location in Plan
3a	Purpose and need for proposed plan	Section 1.2
3b	Who owns, operates, and maintains the wastewater system	Section 1.4
3c	Existing and proposed service boundaries	Chapter 3
3d	Layout map showing boundaries; existing sewer facilities; proposed sewers; topography and elevations; streams, lakes; and other water bodies; water systems	Figure 2-2, Figure 2-3, Figure 2-4, Figure 6-1, Figure 12-1
3e	Population trends	Section 3.2.3
3f	Existing domestic and/or industrial wastewater facilities within 20 miles	Figure 1-1
3g	Infiltration and inflow problems	Section 5.3
3h	Treatment systems and adequacy of such treatment	Chapter 9
3i	Identify industrial wastewater sources	Section 5.1.1
3j	Discussion of water systems	Section 2.7
3k	Discussion of collection alternatives	Chapter 7
3l	Define construction cost and O&M costs	Chapters 9 and 12
3m	Compliance with water quality management plan	Sections 2.4.1 and 4.2.1
3n	SEPA compliance	Appendix A

Table 1-2 Wastewater Facilities Plan Requirements per WAC 173-240-060

WAC 173-240-060 Reference Paragraph	Description of Requirement	Location in Plan
3a	Who owns, operates, and maintains the wastewater system	Section 1.4
3b	Location map and existing and proposed service boundaries	Figure 1-1, Chapter 3
3c	Expected future wastewater quantity and quality	Chapter 8
3d	Degree of treatment required	Chapters 4 and 9
3e	Description of receiving water and water quality standards	Section 4.2
3f	Proposed treatment processes and discussion of alternatives evaluated	Chapter 11
3g	Design data and sizing calculations of unit processes	Chapters 9 and 11
3h	Discussion of various treatment plant sites	Not applicable. Per Chapter 9, there is an existing treatment plant.
3i	WWTP flow diagram, layout and hydraulic profile	Figures 9-1, 9-2 and 9-3
3j	Discussion of infiltration/inflow and overflows	Chapter 7
3k	Discussion of special provisions for treating industrial waste	Not applicable. Per Section 5.1.1, there are no significant industrial sources in the service area.
3l	Outfall analysis	Section 9.4.9
3m	Discussion of sludge disposal	Section 9.6
3n	Provisions for future needs	Chapters 9 and 12
3o	Staffing and testing requirements	Section 9.7
3p	Estimated capital, operation and maintenance costs associated with proposed improvements	Chapters 11 and 12
3q	Compliance with applicable water quality management plans	Sections 2.4.1 and 4.2.1
3r	SEPA compliance	Appendix A

This Plan is based on a planning horizon through 2036, given current population targets do not extend past 2035, with projections for 2022, 2036, and a full build-out capacity scenario.

The existing and future capacities of the sewer system were evaluated based on current and anticipated future wastewater flow rates. Future wastewater flow rates are estimated from existing flow data and population and employment growth projected within the sewer service area by Snohomish County's adopted 2035 Population and Employment Growth Targets for the City of Snohomish UGA for target year 2035, and uses the adopted targets as an interpolation point for 2022 and extrapolation to 2036 and build-out.

A capital improvement plan is provided that prioritizes improvements, and provides opinions of probable project costs. A Wastewater Financial Gap Analysis has been completed by FCS Group. This analysis is summarized in Chapter 13 of this Plan and included in Appendix C.

1.3 Authorization and Scope

In 2007, the City began preparation of an amendment to the 2005 Plan to re-evaluate upgrade and expansion alternatives for the WWTP. A draft of the Wastewater Treatment Plant Facility Plan Amendment provided to the City in 2009 estimated the cost of the comprehensive upgrade and expansion to be up to $44.7 million, which included accommodating a potentially large UGA expansion.

Due to the high cost, the City decided to pursue regional treatment with the City of Everett, an alternative with a comparable cost estimate. The City entered into an Agreed Order (Agreed Order No. 7974) with the Washington State Department of Ecology (Ecology) that set a timeline for progress on the regionalization project. The City retained CH_2M HILL to prepare the 2010 Plan, which determined that the cost of regionalization would be approximately $43 million, including an estimated $20 million to purchase capacity at the City of Everett WWTP. The 2010 Plan projected a City population of approximately 17,554 at the end of the previous planning horizon (2024) in the 2005 Plan, which included population growth from UGA expansion. However, the 2010 Plan noted that the economic recession might slow the expected population growth.

Because of the extensive time needed to plan, design, and construct the regionalization project, a second Agreed Order (Agreed Order No. 7973) from Ecology required the City to undertake near-term improvements of the existing WWTP to comply with its National Pollutant Discharge Elimination System (NPDES) permit. Kennedy/Jenks Consultants was retained to prepare an Engineering Report for the near-term improvements and related plans and specifications and perform construction management and post-construction services. The near-term WWTP improvements project included installing a SFF media system in three of the four WWTP lagoons. The SFF media system was started up at the end of September 2012, and construction was completed in February 2013. Based on the initial success of these near-term improvements, Ecology issued Agreed Order No. 10467, which amended initial Agreed Order No. 7974, that allowed plans for regionalization to be delayed or postponed indefinitely pending further demonstrated success of the near-term improvements.

The City submitted their Notice of Compliance (Appendix B) to Ecology in 2015. On March 10, 2015, Ecology determined that the City had satisfied the conditions in Agreed Order No. 7973 and Agreed Order No. 10467, and was thus no longer required to connect their wastewater system to the City of Everett's conveyance system.

BHC Consultants was retained to prepare this Plan to focus on continued improvements at the City WWTP and to the conveyance system. These improvements focus on further increasing the treatment reliability and efficiency that began with completing the near-term improvements

project and providing additional treatment and conveyance capacity as required throughout the planning horizon. Because there have been significant changes in population projections and associated wastewater flows, loads and plans for treatment compared to the 2005 Plan and 2010 Plan, this Plan completely replaces those previous plans.

The City has a Biosolids Plan (2015) that addresses biosolids currently stored in the existing lagoons and a separate CSO Management Plan (2014) that evaluates the CSO improvements already implemented and suggests additional enhancements to the system to provide further control. Those additional enhancements are reflected in the proposed capital improvement projects discussed herein.

1.4 Wastewater System Ownership and Operation

The City owns, operates, and maintains the collection system, including a number of pump stations, within the City limits. The City also owns, operates, and maintains the WWTP site and facilities.

Contact information for the City of Snohomish wastewater system is as follows:

Steve Schuller, P.E., City Administrator / Utility General Manager
City of Snohomish
116 Union Avenue
Snohomish, WA 98290
360.568.3115
schuller@SnohomishWA.gov

1.5 Related Studies

The following studies and planning documents were used in preparing this report. Other City of Snohomish, City of Everett, and Snohomish County plans and regulations; other federal, state, and local laws and regulations; and Ecology guidance on sewer planning and design are not listed, as they are widely available on the Web and public domain.

- Wastewater Staffing Evaluation Report. September 2006. Kennedy/Jenks.
- City of Snohomish General Sewer Plan and Wastewater Facilities Plan. May 2005. Tetra Tech/KCM.
- 2010 City of Snohomish General Sewer Plan and Wastewater Facilities Plan Update. February 2011. CH2M HILL.
- Flow Management Plan. October 2013. City of Snohomish.
- Biosolids Sampling Plan. May 2014. Kennedy/Jenks.
- Disinfection System Upgrade Plan. August 2014. BHC Consultants.
- Combined Sewer Overflow Management Plan. October 2014. BHC Consultants.
- Lagoon Lining Evaluation Report. 2014. Leak Location Services, Inc.
- Biosolids Management Plan. January 2015. Kennedy/Jenks.
- Biosolids Removal and Reuse Project. February 2015. BHC Consultants and Kennedy/Jenks.
- Combined Sewer Overflow Post-Construction Monitoring Plan. July 2015. BHC Consultants.
- City of Snohomish Comprehensive Plan. March 2016. City of Snohomish.
- Wastewater Financial Gap Analysis

Chapter 2 Background

2.1 Study Area

The City of Snohomish is approximately 25 miles northeast of Seattle, Washington and 8 miles southeast of Everett, Washington, in the south-central portion of Snohomish County (see Figure 1-1). The City is on the north side of the Snohomish River at its confluence with the Pilchuck River. The study area for this Plan encompasses approximately 5.9 square miles (3,770 acres) of land roughly bordered by State Route 2 on the north, the Pilchuck River on the east, the Snohomish River on the south, and 85th/83rd Avenue SE on the west. This study area includes the 2,467-acre area within the current city limits, the 887-acre portion of the urban growth area (UGA) outside the city limits and north of the Snohomish River, and the portion of the North Planning Area south of State Route 2 that is approximately 150 acres. The study area excludes the 265-acre portion of the UGA south of the Snohomish River and the North Planning area north of State Route 2. Figure 2-1 shows the city limits, the UGA boundary, the North Planning Area, and the overall study area boundary.

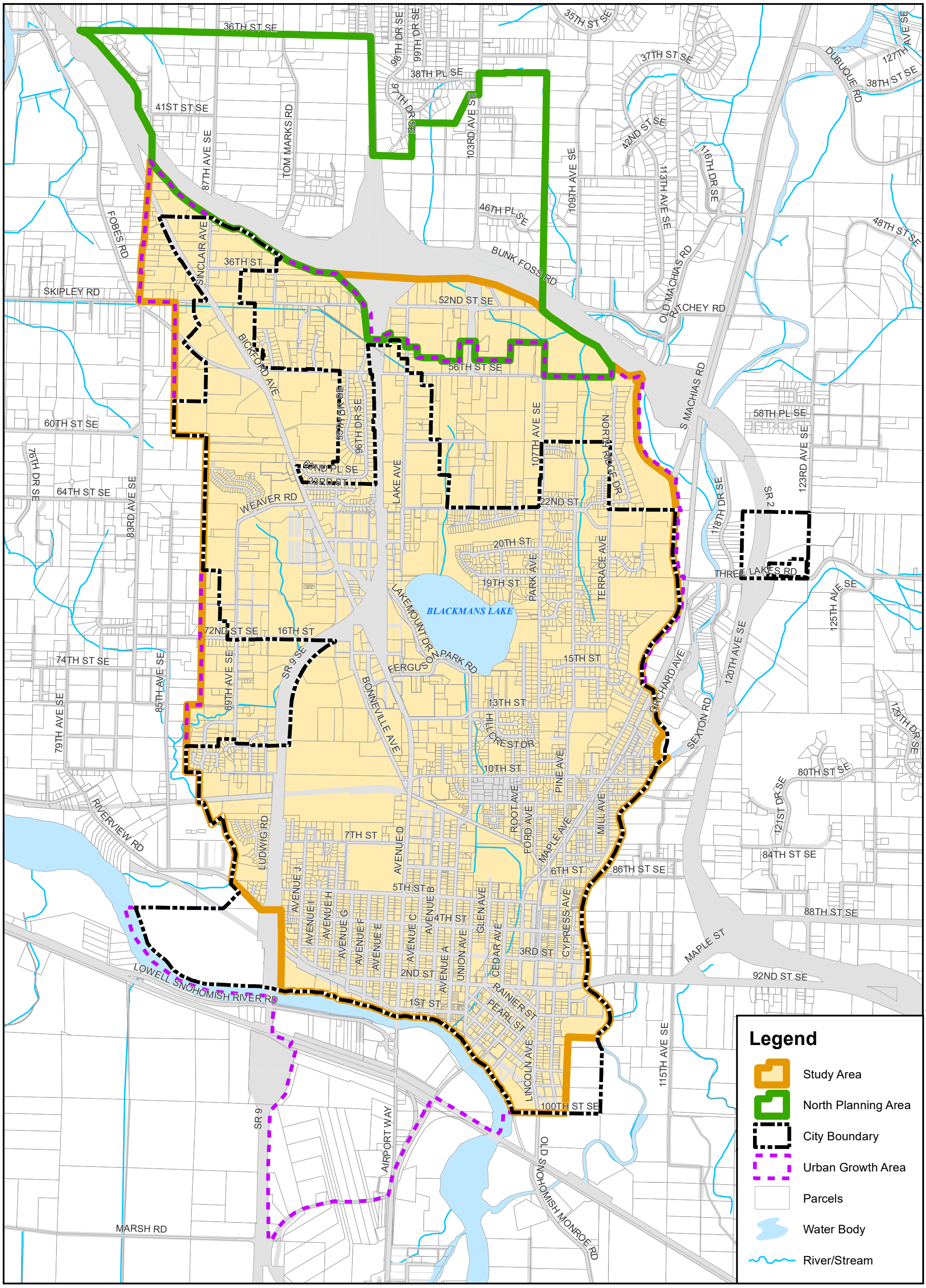
As described in the City's most recent comprehensive plan, the total North Planning Area is approximately 683 acres in size. The area is currently designated Rural Urban Transition Area (RUTA) by Snohomish County. This area has been identified by the City as a logical and beneficial expansion of the UGA, should additional capacity be required. Following a lengthy public outreach process to residents of the area, many of whom indicated a strong orientation to Snohomish, the City Council passed Resolution 1224 in February 2009 designating it as an area of interest for future municipal expansion. The City acknowledges that the North Planning Area is currently designated a rural area and most of it will likely remain so throughout the current planning horizon and beyond. However, the City believes there is potential for expansion of the UGA and/or sewer service into the portion of the North Planning Area south of State Route 2 within the planning horizon, such that this is given consideration in this Plan. Therefore, that portion of the North Planning Area is included in the study area, as shown in Figure 2-1.

The City's existing sewers serve most of the area within the city limits. The unincorporated portion of the UGA and the North Planning Area are unsewered at this time. Although the UGA includes a portion of land south of the Snohomish River, the City has indicated there are no plans to incorporate or extend sewer service into that area within the planning horizon. It is assumed that within the planning horizon sewer service will be extended to the UGA areas north of the Snohomish River and the portion of the North Planning Area south of State Route 2.

2.2 Topography

Topography within the study area generally slopes from north to south, with the southeastern portions draining to the Pilchuck River. Elevations range from about 15 feet in the southeastern portion of the City to more than 290 feet in the northeastern portion of the City, with slopes varying from virtually flat to more than 25 percent. Figure 2-2 shows the study area topography.

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City of Snohomish Washington

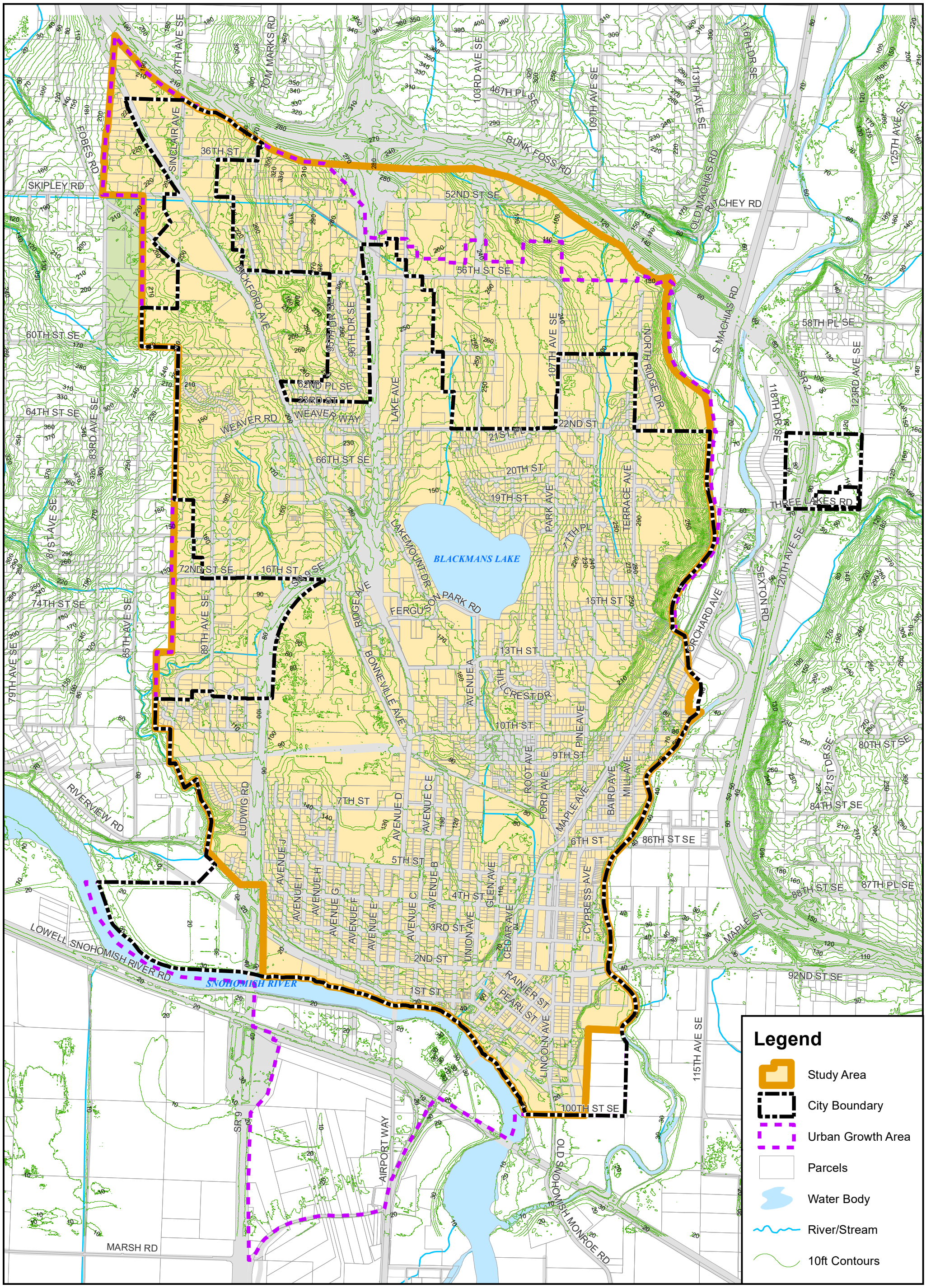
Study Area

General Sewer Plan and
Wastewater Facilities Plan
City of Snohomish, Washington
February 2019

Figure
2-1

P:\Mapping\Maps_Generated\Snohomish_City\19-10615.00\003\maps\Fig 2-1 Study Area - 11x17.mxd 2/19/2019 ctolentino

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Legend

- Study Area
- City Boundary
- Urban Growth Area
- Parcels
- Water Body
- River/Stream
- 10ft Contours

Contours: City of Snohomish.
GIS Base data: Snohomish County.
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City of Snohomish Washington

Study Area Topography

General Sewer Plan and
Wastewater Facilities Plan
City of Snohomish, Washington
February 2019

Figure
2-2

P:\Mapping\Maps_Generated\Snohomish_City\19-10615.00\003\maps\Fig 2-2 Study Area Topography - 11x17.mxd 2/19/2019 ctolentino

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2.3 Climate

The climate in the study area is typical for the Puget Sound region and is characterized by cool to warm summers with generally wet fall, winter, and spring seasons, during which the majority of the 49 inches of annual precipitation is likely to occur. The annual average temperature range is between a high of 61°F and a low of 43°F.

2.4 Surface Waters

2.4.1 Water Bodies

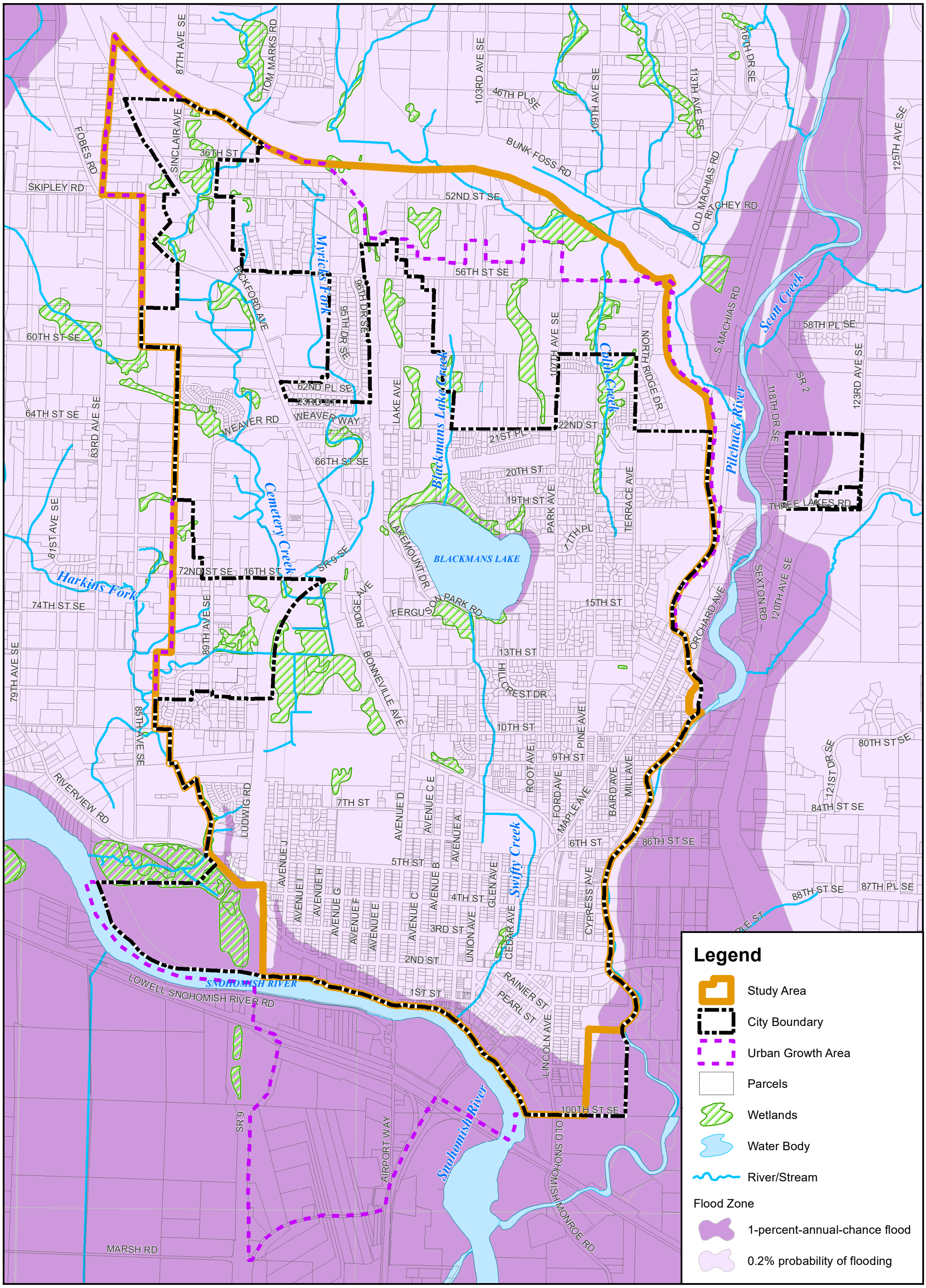
The major surface water bodies in the study area are the Snohomish and Pilchuck Rivers in the south and east, Cemetery and Harkins Fork Creeks in the west, Bunk Foss Creek in the north and Swifty Creek, Blackmans Lake, and Blackmans Lake Creek in the north and central portions of the city. Figure 2-3 shows these major surface waters in the study area. Interspersed throughout much of the study area are large designated critical areas. Critical areas, as defined in the City's Comprehensive Plan, include wetlands, areas with a critical recharging effect on aquifers used for potable water, fish and wildlife habitat conservation areas, frequently flooded areas, and geologically hazardous areas. The critical areas located within the study area are also shown in Figure 2-3.

The Snohomish River drains an area of approximately 1,780 square miles and begins at the confluence of the Skykomish and Snoqualmie Rivers near Monroe. For approximately 5 miles downstream of Monroe, the river's gradient is moderate (2 to 3 feet per mile); it flattens abruptly as it reaches Snohomish. Below Snohomish, the river is characterized by an increasingly braided and meandering series of tidally influenced channels, eventually discharging into Port Gardner immediately north of Everett. The lower 3 miles of the main river channel are dredged to maintain shallow-draft navigation access.

The average annual discharge of the river at Snohomish (River Mile 12.7) is estimated to be 11,000 cubic feet per second (cfs); the median discharge at this site is roughly 8,800 cfs (USGS, 1985). These discharges rank the Snohomish as the second largest river (behind the Skagit) in the Puget Sound/Hood Canal region. Like many rivers in Western Washington, the Snohomish River exhibits two seasonal flow peaks coinciding with high precipitation and snowmelt periods. Minimum seasonal flows occur during the late summer to early fall; the estimated 7-day/10-year low flow at Snohomish is 1,330 cfs.

The Snohomish River is listed as an impaired waterway for fecal coliform bacteria and temperature, based on Ecology's 2016 Water Quality Assessment 305(b) report and 303(d) list approved by the U.S. Environmental Protection Agency (EPA). The section of the river listed for temperature is right at the City's boundary with the river and encompasses the area where the City's WWTP outfall discharges. The section of the river listed for fecal coliform bacteria is approximately 3.5 miles upstream of the City. The river had previously also been listed for dissolved oxygen, which led Ecology to develop total maximum daily load (TMDL) limits that impacted the WWTP discharge requirements and necessitated the recent near-term improvements at the WWTP. Additionally, Snohomish County has a program to monitor the Snohomish River tributaries, described in the 2015 *Snohomish County Total Maximum Daily Load Monitoring Quality Assurance Project Plan*. This program was triggered by the Federal Water Pollution Control Act (Clean Water Act) Title 33 United States Code, Section 1251 et seq. and Appendix 2 (Total Maximum Daily Loads) of the 2013 – 2018 NPDES Systems Phase 1 Municipal Stormwater Permit issued to Snohomish County by Ecology.

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Wetlands: Snohomish County. Flood Hazard Area: FEMA/WA Dept. of Ecology 2010
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City of Snohomish Washington

Study Area Surface Waters and Critical Areas

General Sewer Plan and
Wastewater Facilities Plan
City of Snohomish, Washington
February 2019

Figure
2-3

P:\Mapping\Maps_Generated\Snohomish_City\19-10615.00\003\maps\Fig 2-3 Study Area Surface Waters & Critical Areas - 11x17.mxd 2/19/2019 ctolentino

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Surface water that does not enter one of the City's creeks moves via overland flow, underground storm drainage pipes or the combined wastewater system to the Pilchuck or Snohomish River. Because of the area's topography, the surface water drainage system works well. Biofiltration is used to the extent possible for stormwater that does not go to the WWTP. The City's 2013 Stormwater Comprehensive Plan Update outlines improvements to further enhance stormwater quality and control runoff.

2.4.2 Floodplains

Flooding in the City of Snohomish by the Snohomish and Pilchuck Rivers is confined primarily to the floodplains in the southeastern part of the City. Considerable development and platting has occurred in both floodplains, though much more development has occurred in the Harvey Airfield area in the Snohomish River floodplain. Both the City and the County have granted plats in these two floodplains, as early as the 1890s. The City and County floodplain regulations do not prohibit development but do require that any allowed development be floodproofed to 1 foot above the base flood elevation (100-year flood elevation). Both the City and the County have adopted ordinances that meet Federal Emergency Management Agency (FEMA) requirements for floodplains.

The descriptions of the floodplains were most recently revised in FEMA's 2005 *Flood Rate Insurance Study and Flood Insurance Rate Maps* (FIRMs). Based on these most recent descriptions, the WWTP has a 0.2 percent annual chance of flooding (because the dikes around the WWTP are at the 500-year flood elevation) and Pump Station No. 1 (CSO) has a 1 percent annual chance of flooding (it is in the 100-year floodplain). Pump Station No. 11 (Kla-Ha-Ya) is also in the 100-year floodplain, but it floods when the Snohomish River stage reaches approximately 25 feet, which occurs most years.

2.5 Groundwater

Groundwater hydrology in the study area is typical of an area predominated by glacial till. Movement of groundwater tends to be in the direction of either the Pilchuck or the Snohomish River, with the predominant flow being to the south and southwest. No known uses of groundwater are recorded for the study area. During the summer, the groundwater is generally 4 feet or more below the surface.

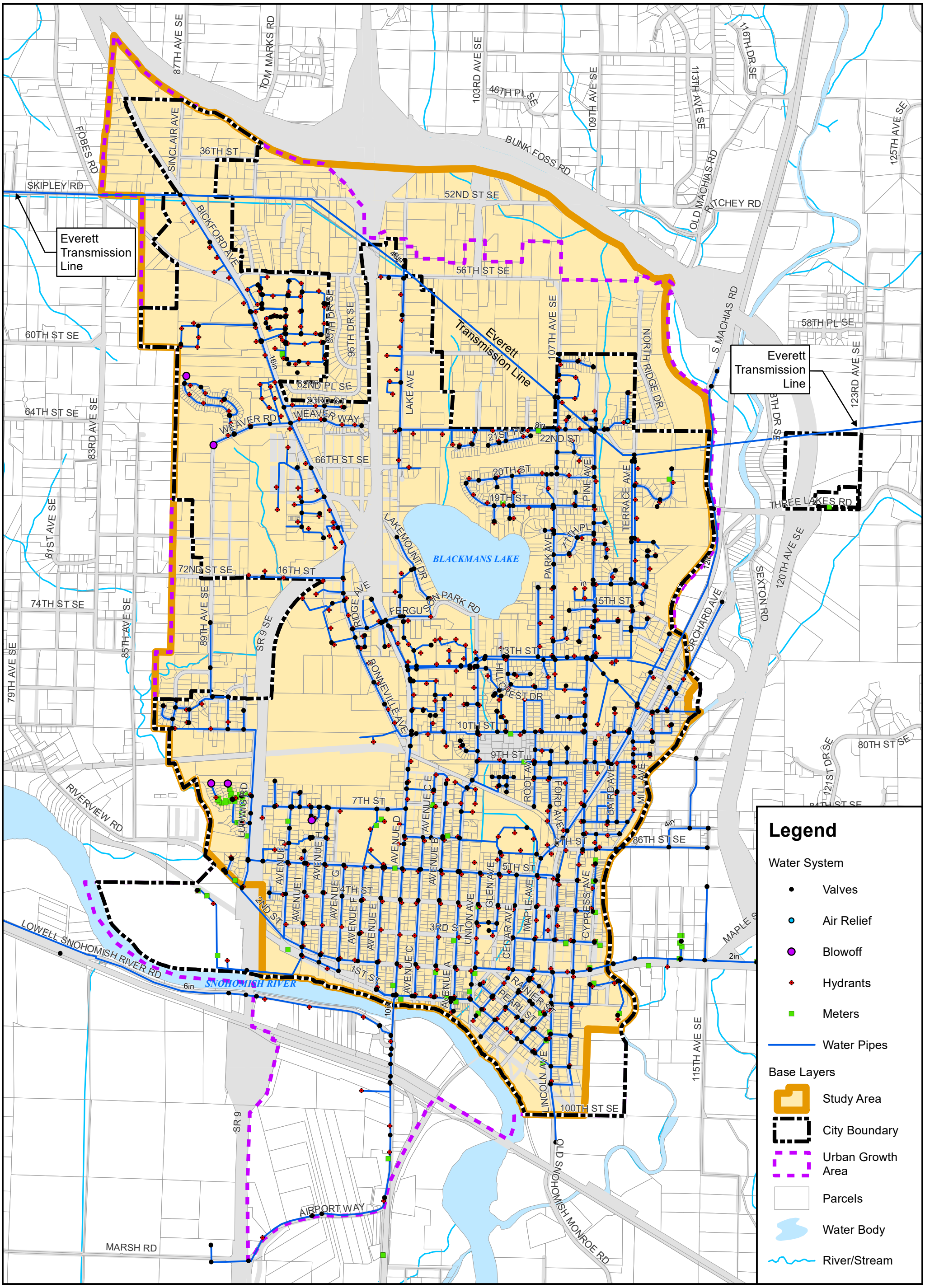
2.6 Geology

The predominant soils group in the study area is classified by the U.S. Soil Conservation Service as Tokul gravelly loam overlaying glacial till. This group is characterized as a moderately well drained soil, dark brown to dark yellowish brown in color. Soils in and around the City's WWTP are classified as Pilchuck loamy sand and Puyallup sandy loam.

2.7 Water Supply

The City's existing water system is comprised of two reservoirs with a total capacity of 7.7 million gallons; a distribution system of 66 miles of metal and plastic piping of varying sizes, ages, and materials; and four active pressure reducing stations that divide the water system into six pressure zones (see Figure 2-4). The City purchases water wholesale from the City of Everett via five connections to Everett's Transmission Line No. 5. No additional treatment is required for Everett water supply.

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Water System: City of Snohomish.
GIS Base data: Snohomish County.
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City of Snohomish Water System

General Sewer Plan and
Wastewater Facilities Plan
City of Snohomish, Washington
February 2019

Figure
2-4

P:\Mapping\Maps_Generated\Snohomish_City\19-10615.00\003\maps\Fig 2-4 City of Snohomish Water System - 11x17.mxd 2/19/2019 ctolentino

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Until recently, water to the lower zone had been fed from the diversion dam and water treatment plant located 14 miles northeast on the Pilchuck River. The City has since discontinued use of that plant and plans to decommission it. Now, all zones are served from the connections to the City of Everett transmission line. Additionally, Reservoir No. 1 was decommissioned in 2014 due to its poor structural condition. An analysis showed that the City's water system has sufficient supply and storage without a replacement reservoir.

Private water districts and associations supply water in the unincorporated UGA and to some areas in the City. The City will incorporate these systems into the City water utility as water and sewer are extended to new development in that area. The City is gradually reconnecting portions of the south pressure zone at the higher elevations to the north pressure zone to increase domestic water pressure and fire flow to acceptable levels.

2.8 Other Wastewater Facilities

Under the requirements of WAC 173-240-050 for General Sewer Plans, Snohomish must assess the feasibility of developing regional wastewater facilities with neighboring communities and industries within 20 miles. The following treatment facilities are within 20 miles of the City:

- Marysville
- Everett
- Lake Stevens Sewer District
- Mukilteo Water and Wastewater District
- Edmonds
- Arlington
- Tulalip Tribes of Washington
- Duvall
- Lynwood
- Alderwood Water and Sewer District
- Granite Falls
- Sultan
- Monroe
- King County's Brightwater Treatment Plant

As discussed in Chapter 10, the City considered regional facilities with the Brightwater Wastewater Treatment Plant in Woodinville and the City of Everett Water Pollution Control Facility, but is no longer pursuing regionalization due to the high cost.

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Chapter 3 Land Use and Population

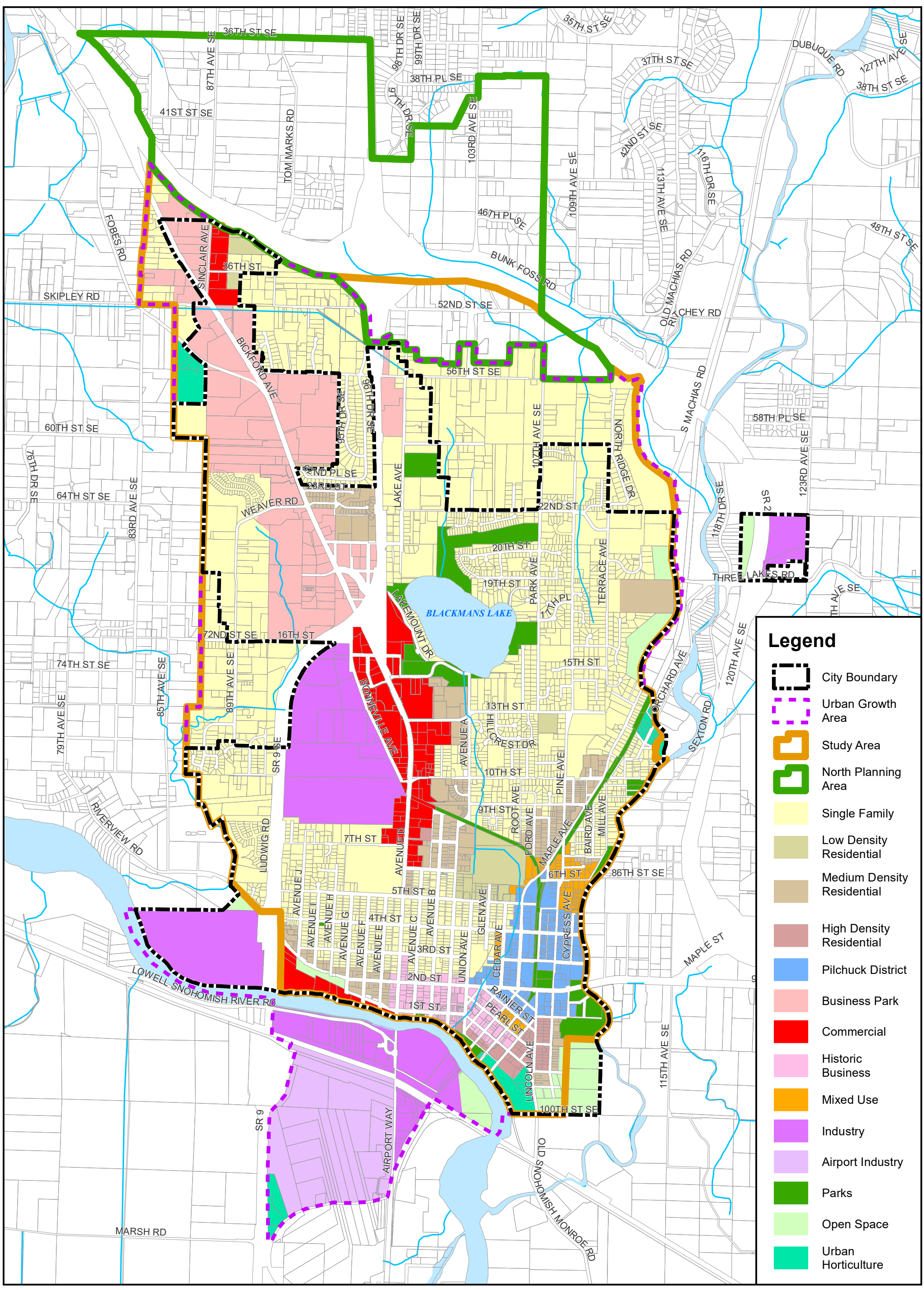
3.1 Land Use

The land use within the City and UGA as of March 2016 is shown on Figure 3-1. Land use designations shown on this figure were provided by the City of Snohomish Planning and Development Services Department. Designations include residential classifications of Single Family, Low Density Residential, Medium Density Residential, High Density Residential, and Mixed Use. Commercial Classifications include Commercial, Business Park, Historical Business, Industry, and Airport Industry. There are also designations for Open Space, Urban Horticulture, and Parks.

Within the City limits, about 51.5 percent of the area excluding right-of-way is designated residential. There is not much vacant buildable space remaining within the City limits, which is expected to dampen construction of additional housing units. As a result, population is expected to grow slowly, at least until annexations occur and sewer service can be extended to the annexed areas. It is anticipated that the City boundaries will expand within the UGA, to accommodate anticipated growth. None of the approximately 740 acres in the study area outside the current City limits are presently served by the City's sanitary sewer system. About 150 acres of the study area outside the current City limits is associated with the portion of the North Planning Area south of State Route 2. Additionally, the City has indicated that a significant portion of the study area that is northeast of the current City limits has relatively little space for development due to the topography and location of creeks and critical areas.

According to the City of Snohomish Comprehensive Plan (March 2016), the City is estimated to have a marginal capacity deficit of about five percent of the allocated population increase within the current City limits. However, when also considering capacity within the City's UGA, there is over 10 percent more capacity than needed to accommodate allocated population growth. Per the comprehensive plan, the City supports moving 150 persons from the 2035 population target for the City limits to the target for the City's UGA through the Snohomish County Tomorrow target reconciliation process. If resolution of the deficit cannot be achieved through the reconciliation process, the City will consider other options, which may include amending the Land Use Designation Map to provide the necessary additional residential capacity within the current City limits. This would in turn impact current zoning.

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Land Use: City of Snohomish.
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City of Snohomish Washington

Land Use Map

General Sewer Plan and
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City of Snohomish, Washington
February 2019

Figure
3-1

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3.2 Planning Areas

3.2.1 General

Sewer system planning for the City relies on land use plans and population forecasts that conform to the GMA, Snohomish County Comprehensive Plan, and the City of Snohomish Comprehensive Plan. Residential and employment population estimates for the City's service area have been developed for each sub-basin for the 2022, 2036, and build-out planning horizons. The City's proposed sewer service area includes the current City limits, its UGA (except for the portion south of the Snohomish River) and the portion of the North Planning Area south of State Route 2. The service area is divided into 54 sub-basins which are shown on Figure 3-2. These basins were delineated based on the current sewer system and approved by the City.

3.2.2 Existing Population

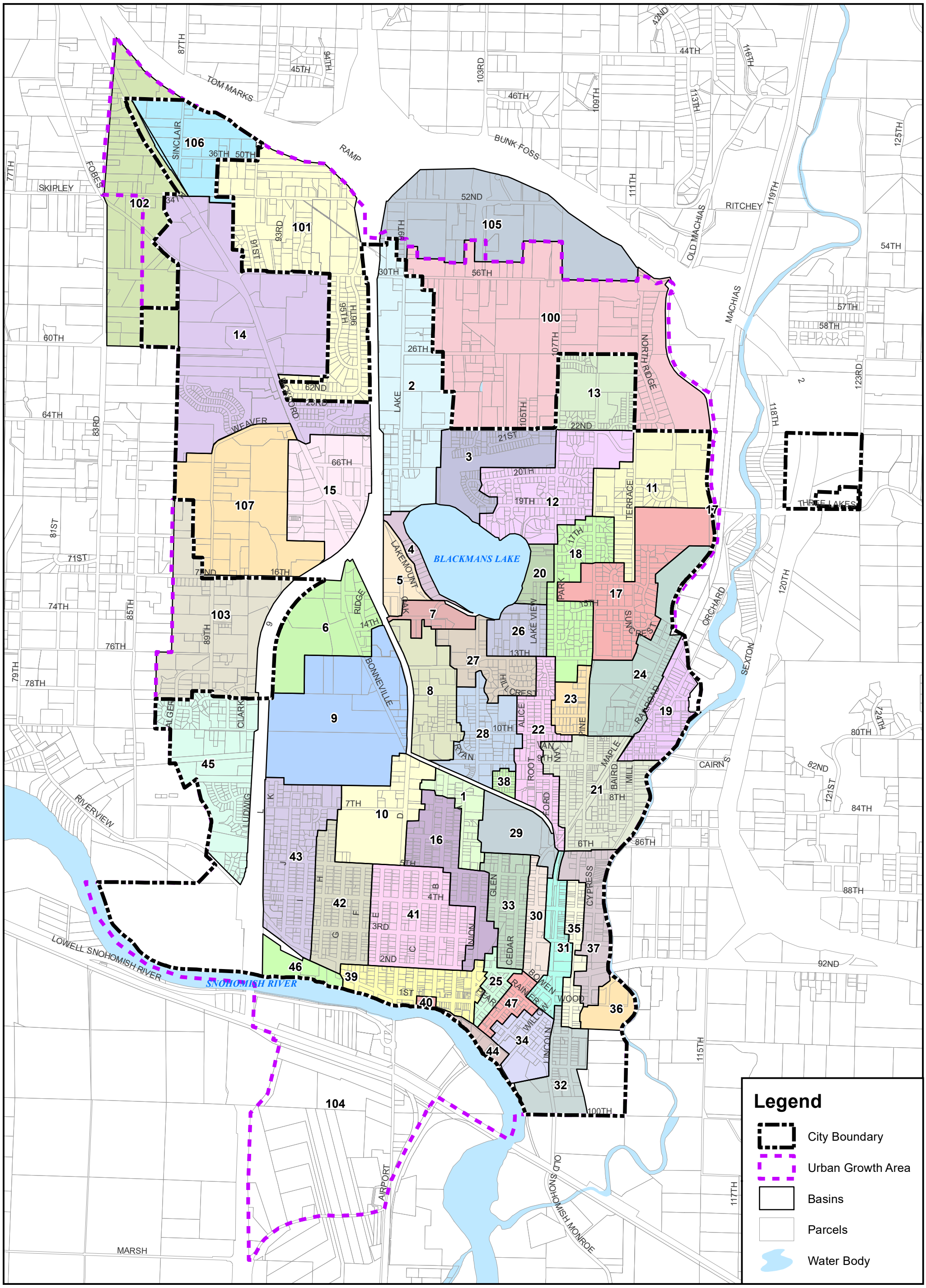
The two different populations contributing sewage are residential, and employment. The methodology used combined various available resources to establish the most accurate population estimates and projections for the purpose of sewer modeling and capital improvement project (CIP) identification. A detailed methodology and list of resources can be found in Appendix D.

Year 2015 served as the residential population baseline, however the baseline year varied for each contributing population. Baseline residential population estimates were calculated using County land use data, 2010 Census population by block, and OFM 2015 SAEP population growth estimates by block-group, and adjusted based on 2015 TAZ housing unit data. Census block population data was distributed to parcels based on population density and residential acreage.

Year 2014 served as the baseline employment population year. The employment population includes employees from commercial, schools, and government facilities. Individual Students are not included in the employment or the residential values. Covered employment estimates for 2014 were obtained from Puget Sound Regional Council (PSRC) staff by custom data request. Covered employment refers to positions covered by the Washington State Unemployment Insurance Act, and accounts for approximately 85-90% of all employment. Covered employment basin estimates were adjusted based on 2015 TAZ employment data. Baseline population estimates were aggregated per sub-basin and used as the first known data point to interpolate existing (2016) and future populations.

Existing residential and employment population estimates were calculated by interpolating between baseline data and Snohomish County 2035 Population and Employment Growth Targets for the UGA.

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Sewer System & Basins: City of Snohomish.
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City of Snohomish Washington

Sub-Basins Map

General Sewer Plan and
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City of Snohomish, Washington
February 2019

Figure
3-2

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3.2.3 Future Population

Future population figures were interpolated for 2016 and 2022, and extrapolated for 2036, between 2015 baseline data and 2035 projections. Residential and employment populations were forecasted for the existing (2016), 2022, 2036, and build-out planning horizons. Residential and employment population projections for 2035 were derived from a combined analysis of Snohomish County's adopted 2035 Growth Targets and the 2012 Buildable Lands Report (BLR) for Snohomish County. For the City of Snohomish UGA, CPP for Snohomish County adopted a 2035 Population Growth Target of 10,657 and a 2035 Employment Growth Target of 6,941. These adopted targets were distributed throughout the UGA based on development capacity and aggregated by sub-basin.

Residential build-out calculations also utilized the BLR. Prior to market reductions, the BLR housing unit capacity equates to the build-out capacity. The parcel-level housing unit capacity was aggregated by basin and increased by a factor of 15% to account for previous market-factor reductions. The build-out population was calculated for each basin as a function of adjusted housing unit capacity and average household size.

Similar to residential growth, employment build-out calculations also utilized the BLR. The BLR GIS data provides the additional employment capacity for each parcel. The BLR parcel-level employment capacity was aggregated by basin and increased by a factor of 15-percent to account for prior market-reductions. The revised employment capacity is added to the baseline to establish the build-out employment population.

The populations listed in Table 3-1 below represent total City and UGA populations, while Table 3-2 lists the sewered populations. Because the City does not know for sure if the sewer system will be expanded into the northern sub-basins within the horizon, two different scenarios were developed for 2036 population projections, but only the 2036 population projection with expansion into the northernmost sub-basins of the study area was evaluated to be more conservative.

Table 3-1 Population Forecasts for the City of Snohomish and UGA

Year	Residential Population	Employment Population
2016	11,095	5,480
2022	12,246	5,913
2036	14,932	6,900
Buildout	18,033	7,899

Table 3-2 Sewered Population Forecasts for the City of Snohomish and UGA

Year	Residential Population	Employment Population
2016	9,569	4,677
2022	10,348	5,027
2036 (Current City Limits Only)[(a)]	12,358	6,111
2036 (Including UGA and Portion of North Planning Area)[(b)]	14,505	6,223
Buildout	17,995	7,236

Note:
(a) These 2036 projections only consider serving the current City limits.
(b) These 2036 projections include serving the current City limits, the entire UGA (excluding the portion south of the Snohomish River) and the portion of the North Planning Area south of State Route 2.

Chapter 4 Permits, Requirements, and Regulations

Wastewater must be collected, treated, and disposed of or reused in a way that protects public health and receiving water quality, generates no objectionable off-site odors or aesthetic nuisances, and complies with all applicable regulations. Wastewater treatment facilities must meet the regulations and requirements of many federal, state, and local regulatory agencies. This chapter summarizes applicable rules and regulations that typically apply to wastewater projects.

4.1 Federal Regulations

4.1.1 Federal Water Quality Acts

Programs and policies designed to protect water quality were first initiated on a nationwide scale by the Federal Water Pollution Control Act of 1956. This act was amended by the Water Quality Act of 1965, the Clean Water Restoration Act of 1966, and the Water Quality Improvement Act of 1970. The Federal Water Pollution Act Amendment of 1972 (Public Law 92-500) replaced the previous language of the Act entirely. This Act requires states to establish water quality standards for all of their water bodies. The standard must consist of two parts: a designation of the use of the water body; and the water quality criteria that water body must maintain to protect the designated uses from pollution. The State of Washington complies with this regulation through WAC 173-201A, which is described later.

The Clean Water Act of 1977, in further amending the Act, required any agency conducting an activity that may result in a discharge into navigable waters to obtain certification from the appropriate water pollution control agency, verifying that the discharge complies with applicable effluent limitations and water quality standards. Further, these amendments established the National Pollutant Discharge Elimination System (NPDES) permits, which regulate point discharges into water, and required various types of water quality planning by states. Grants for facilities and training were also authorized under these amendments.

With increased environmental awareness of the extent and effects of nonpoint pollution, including stormwater, additional amendments to the Federal Clean Water Act were passed by Congress in early 1987. These amendments, referred to as the Water Quality Act of 1987, and especially Section 319, direct the states in developing programs designed to reduce nonpoint source pollution. These sources of pollution have become increasingly evident as abatement of point source pollution has occurred. The amendments required each state to do the following:

- Submit a report identifying navigable waters that cannot meet water quality standards without action to control pollution.
- Identify the categories of pollution sources.
- Describe processes for identifying best management practices and control strategies.
- Identify state and local programs for controlling pollution from both point and nonpoint sources.

4.1.2 Federal Effluent Limitations

Section 301 of the federal Water Pollution Control Act requires all publicly owned wastewater treatment facilities to provide a minimum of secondary treatment unless a special waiver is obtained. This act defines secondary treatment as follows:

- The monthly average of biochemical oxygen demand (BOD) and total suspended solids (TSS) concentrations shall not exceed 30 milligrams per liter (mg/L).
- The weekly average of BOD and TSS concentrations shall not exceed 45 mg/L.
- The monthly average removal of BOD and TSS shall be at least 85 percent (except 72.2% removal of TSS November through March).
- The pH of the effluent shall be between 6.4 and 9.0.

There can be exceptions to these regulations when treatment works receive combined sewer flows or certain industrial wastes. However, in general, these are the minimum federal requirements for effluent quality. Ecology administers these regulations under the NPDES as discussed later in this chapter.

4.1.3 Federal Standards for Use or Disposal of Sludge

The federal document that regulates the use and disposal of sewage sludge is the Code of Federal Regulations, Part 503 (40 CFR 503, EPA 1993). These regulations, published in February 1993, address three main sludge disposal options:

- Land application
- Surface disposal
- Incineration

Land-applied sludge must meet requirements in the 503 regulations for pathogen and vector attraction reduction. Two basic classes for pathogen reduction are established in the regulations. In general, sludge distributed in bagged form must meet Class A requirements. Sludge applied to the land in bulk form must meet Class B requirements. The discussion below focuses on the regulations applicable to Class B requirements for bulk land application because that is the disposal option utilized by the City.

Pathogen Reduction

Class B sludge must have levels of fecal coliform organisms less than 2 million per gram of total solids, or meet other requirements, or the sludge must have been treated with an EPA-defined "process to significantly reduce pathogens (PSRP)." These processes include aerobic digestion for a mean cell residence time greater than 40 days at 20ºC or 60 days at 15ºC, air drying, anaerobic digestion, composting, or lime stabilization.

Sludge in the City's existing WWTP is collected in the lagoons, where they are anaerobically and aerobically digested over time until they are dredged, dewatered and land applied. Because the lagoons are not an EPA-defined PSRP, the City tests for density of fecal coliform organisms prior to dredging to be sure that the sludge meets the requirements for Class B pathogen reduction.

Vector Attraction Reduction

The regulations require that land-applied sludge be processed to reduce its "vector attraction." This means that the sludge should be stabilized sufficiently to not be an attraction to rodents or birds that could spread pathogens contained in the sludge and thereby increase the risk of human exposure. The most common of the available options for demonstrating vector attraction reduction is that the volatile solids concentration in the sludge be reduced through processing by at least 38 percent. However, because the City cannot measure the volatile solids in the sludge as it settles out, they instead utilize a different option for demonstrating vector attraction reduction wherein a sample of the stabilized sludge from the lagoons is anaerobically digested

for 40 days at a temperature between 30 and 37 degrees Celsius. If the reduction in volatile solids of the stabilized sludge is less than 17 percent, sufficient vector attraction reduction has occurred. A series of other options are provided for reducing vector attraction, including injection below the ground surface and soil incorporation within 6 hours of land application.

Metals

Limits are specified for the concentration of various metals in the sludge and for the cumulative loading of these metals on the land used for its application. Table 4-1 lists the concentration limits for any sludge that is land applied. Table 4-2 lists further guidelines for sludge that is land applied in bulk. Either the monthly average concentration criteria or the cumulative pollutant loading rate criteria must be met.

Table 4-1 Ceiling Concentrations for Metals in Land-Applied Sludge

Parameter	Ceiling Concentration Limit (mg/kg)
Arsenic	75
Cadmium	85
Copper	4,300
Lead	840
Mercury	57
Molybdenum	75
Nickel	420
Selenium	100
Zinc	7,500

Table 4-2 Metal Concentration Limits for Bulk Sewage Sludge Land Application

Parameter	Monthly Avg. Concentration Limit (mg/kg)	Cumulative Pollutant Loading Rate (kg/hectare)
Arsenic	41	41
Cadmium	39	39
Copper	1,500	1,500
Lead	300	300
Mercury	17	17
Nickel	420	420
Selenium	100	100
Zinc	2,800	2,800

Other Measures

In addition to regulating the quality of biosolids, the regulations require specific management measures, including the following:

- Record-Keeping and Reporting – Records must be kept by the owner describing the quantity and quality of the biosolids that have been applied to specific sites for up to five years. Even if the owner has a contract for biosolids disposal with a private contractor, the owner is ultimately responsible for the record-keeping and reporting.
- Monitoring – The owner is responsible for monitoring the biosolids for metals and specific pathogens on a regular basis.
- Management Practices – Biosolids should not be applied to flooded, frozen, or snow-covered ground, so that biosolids do not enter surface waters.

4.1.4 EPA Reliability Criteria

An important reference for wastewater treatment plant reliability is the EPA's *Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability* (EPA 1974), which is also referenced in Ecology's *Criteria for Sewage Works Design* (2008). This document outlines requirements in three reliability classes, with specific provisions for each unit process. Table 4-3 summarizes its requirements for component reliability.

The City's WWTP is currently permitted with a reliability classification of Class II. WWTP improvements proposed in this Plan will be consistent with the reliability criteria for a Class II facility.

Table 4-3 Summary of EPA Design Criteria for System and Component Reliability

Component	Class I	Class II	Class III
Reliability classification	Works discharging into navigable waters that could be permanently or unacceptably damaged by effluent that was degraded in quality for only a few hours. Examples of Reliability Class I works might be those discharging near drinking water reservoirs, into shellfish waters, or in proximity to areas used for water contact sports.	Works discharging into navigable waters that would not be permanently or unacceptably damaged by short-term effluent quality degradation, but could be damaged by continued (on the order of several days) effluent degradation.	Works not otherwise classified as Reliability Class I or II
Trash removal	Required	Same as Class I	Same as Class I
Grit removal	Required if sludge is handled	Same as Class I	Same as Class I
Clean-out of solids	Provisions for cleaning of solids required for components prior to degritting or sedimentation	Same as Class I	Same as Class I
Controlled diversion	Screened, gravity overflow required with alarm, annunciation, and measurement of flow discharged. Holding basin required	Same as Class I, but no holding basin required	Same, as Class I but no holding basin required
Unit operation bypassing	Required except for unit operations with two or more open basins	Same as Class I	Same as Class I
Mechanically cleaned bar screens	Backup manual screen required	Same as Class I	Same as Class I
Pumps	Capacity to handle peak flow with any one pump out of service must be provided	Same as Class I	Same as Class I
Comminution	Overflow bypass must be provided with manual bar screen	Same as Class I	Same as Class I
Primary sedimentation basins	With largest unit out, remaining units shall have design flow of at least 50 percent of the total design flow to that unit operation	Same as Class I	At least two basins

Table 4-3 Summary of EPA Design Criteria for System and Component Reliability

Component	Class I	Class II	Class III
Final and chemical sedimentation basins, trickling filters, filters, and activated carbon columns	With largest unit out, remaining units shall have design flow of at least 75 percent of the total design flow to that unit operation	With largest unit out, remaining units shall have design flow of at least 50 percent of the total design flow to that unit operation; backup not required for chemical sedimentation basins, filters, and activated carbon columns	At least two basins; backup not required for chemical sedimentation basins, filters, and activated carbon columns
Aeration basin	At least two equal volumes shall be provided	Same as Class I	Single basin permissible
Aeration blowers or aerators	Sufficient to provide for peak oxygen demands with the largest capacity unit out of service	Same as Class I	At least two units
Diffusers	Designed so that isolation of the largest section of diffusers does not measurably impair oxygen transfer capability	Same as Class I	Same as Class I
Chemical flash mixer	At least two basins or a backup means of adding chemicals	Backup not required	Backup not required
Flocculation basins	At least two basins	Backup not required	Backup not required
Disinfectant contact basins	With largest unit out, remaining units shall have design flow of at least 50 percent of the total design flow to that unit operation	Same as Class I	Same as Class I
Sludge handling	Alternate methods of sludge disposal and/or treatment shall be provided for each sludge treatment unit operation without installed backup capability. No recycles permitted that will compromise liquid treatment.	Same as Class I	Same as Class I
Sludge holding tanks	May be used to back up downstream tanks	Same as Class I	Same as Class I
Sludge pumps	A backup pump shall be provided for each set of pumps that performs the same function. The capacity of the pumps shall be such that with any one pump out of service, the remaining pumps will have capacity to handle the peak flow.	Same as Class I	Same as Class I

Table 4-3 Summary of EPA Design Criteria for System and Component Reliability

Component	Class I	Class II	Class III
Anaerobic sludge digestion	At least two digestion tanks shall be provided. At least two of the digestion tanks provided shall be designed to permit processing all types of sludge normally digested. Tanks shall have sufficient flexibility or backup equipment to ensure that mixing is not lost when any one piece of equipment is out of service. Uninstalled backup is acceptable for mixing equipment	Same as Class I	Same as Class I
Aerobic sludge digestion	Backup aeration basin not required. At least two blowers shall be provided. Uninstalled backup is permissible. Largest section of diffusers can be isolated.	Same as Class I	Same as Class I
Sludge holding tanks	May be used to back up downstream tanks	Same as Class I	Same as Class I
Vacuum filter	There shall be sufficient number of vacuum filters to enable the design flow to be dewatered with largest capacity unit out of service. Two vacuum pumps and two filtrate pumps shall service each vacuum filter. These may be uninstalled.	Same as Class I	Same as Class I
Centrifuges	There shall be sufficient number of units to enable the design flow to be dewatered with largest capacity unit out of service. The backup unit may be uninstalled.	Same as Class I	Same as Class I
Incinerators	A backup incinerator is not required. Auxiliary equipment shall be provided with backup.	Same as Class I	Same as Class I
Electric power source	Two separate and independent sources of electric power shall be provided from two separate utility substations or from a single substation and a facility generator. Capacity of backup power shall be sufficient to operate all vital components, during peak wastewater flow conditions, together with critical lighting and ventilation.	Same as Class I except vital components to support the secondary processes need not be included if treatment equivalent to sedimentation and disinfection is provided.	Sufficient to operate screening or comminution facilities, main wastewater pumps, primary sedimentation, and disinfection during peak flow together with critical lighting and ventilation.

Table 4-3 Summary of EPA Design Criteria for System and Component Reliability

Component	Class I	Class II	Class III
Power distribution external to the works	The independent sources of power shall be distributed to the works transformers in a way to minimize common mode failures from affecting both sources.	Same as Class I	Same as Class I
Power distribution within the works	See Referenced EPA document	Same as Class I	Same as Class I
Instrumentation and control systems	Automatic control systems whose failures could result in a controlled diversion or a violation of the effluent limitations shall be provided with a manual override. Instrumentation whose failure could result in a controlled diversion or a violation of the effluent limitations shall be provided with an installed backup sensor and readout. Alarms shall be provided to monitor the condition of equipment whose failure could result in a controlled diversion or a violation of the effluent limitations. Vital instrumentation and control equipment shall be designed to permit alignment and calibration without requiring a controlled diversion or a violation of the effluent limitations	Same as Class I	Same as Class I
Auxiliary systems	If a malfunction of the system can result in controlled diversion or a violation of the effluent limitations and the required function cannot be done by any other means, then the system shall have backup capability.	Same as Class I	Same as Class I

Reference: U. S. Environmental Protection Agency. Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability. MCD-05, EPA-430-99-74-001. Office of Water Program Operations. Washington, D. C.,

4.1.5 National Environmental Policy Act

The National Environmental Policy Act (NEPA) requires environmental review of proposed federal actions that could significantly affect the quality of the human environment. This environmental review takes the form of an environmental impact statement if the proposal would have significant impacts, or an environmental assessment, if it would not. Federal actions that typically could not, individually or cumulatively, have a significant environmental impact might be categorically excluded from environmental review under an agency's NEPA implementing procedures. Generally, EPA is the NEPA lead agency for proposals for federal funding of local wastewater treatment facilities. Ecology, EPA, and other agencies try to coordinate early environmental impact review in conjunction with review under the State Environmental Policy Act (SEPA) to facilitate timely funding decisions. In regard to this Plan, no environmental reviews other than review under SEPA are expected to be required.

4.1.6 Clean Air Act

The Federal Clean Air Act of 1992 requires that all federally funded projects be in compliance with state and regional air quality plans. The local air-quality authority for Snohomish County is the Puget Sound Clean Air Agency (PSCAA); agency requirements are discussed later in this chapter, though actions proposed in this Plan are not expected to require review by the PSCAA.

4.1.7 Historical and Archaeological Sites

Both federal and state laws require agencies to assess the effects of their proposed projects on significant archeological and historic properties. If facility improvement projects impact identified historical or archaeological sites, a more detailed evaluation of the site and potential impact of the proposed actions will be required, but are not expected to disturb any historical or archaeological sites.

4.1.8 Floodplains, Wetlands, and Flood Insurance

Various federal laws and requirements, including the Clean Water Act and Executive Orders 11988 and 11990 on floodplains and wetlands, limit federal agency approval of development of wetlands and floodplains. Under the GMA, cities and counties in Washington State are required to limit development in "critical areas," which include wetlands and floodplains. These local critical area regulations implement federal, state, and local protections for wetlands and floodplains. Actions proposed in this Plan are not expected to impact such critical areas.

4.1.9 Agricultural Lands

It is EPA policy under the Farmland Protection Policy Act (PL 97-98) to protect agricultural lands from "irreversible loss as an environmental or essential food production resource." Actions proposed in this Plan are not expected to impact agricultural lands.

4.1.10 Coastal Zone Management

The Coastal Zone Management Act requires that all federal activities be consistent with approved state coastal zone management programs to the maximum extent possible. This project is located in a coastal zone county and is consistent with Washington's Coastal Zone Management Program and enforceable regulatory policies (SEPA, Water Quality, Air Quality and the Shoreline Master Program). A shoreline development permit would be needed prior to construction if construction is planned within 200 feet of the ordinary high-water mark. For Snohomish County and the City of Snohomish, the local Shoreline Master Programs (SMPs) are

the applicable Coastal Zone Management programs and contain the applicable development standards. It is possible that actions in this Plan could require a shoreline development permit.

4.1.11 Wild and Scenic Rivers

To comply with the Wild and Scenic Rivers Act, proposed projects should not directly and adversely impact any wild, scenic, or recreational river area. Actions proposed in this Plan are not expected to yield any such impacts.

4.1.12 Fish and Wildlife Protection

Projects with a federal "nexus," including federal permits, approvals, or funding, require compliance with the Endangered Species Act. Listed fish species include the following:

- Bull trout – federally threatened and state candidate
- Chinook Salmon – federally threatened and state candidate
- Coho salmon – federally threatened
- Steelhead trout – federally threatened and a state candidate

In addition, the Bald Eagle has been reduced from a federally threatened species to a federal species of concern and a state sensitive species. Adverse impacts to endangered species are not expected from the actions proposed in this Plan.

4.1.13 Magnuson-Stevens Fishery Conservation and Management Act

In December 1998, the National Marine Fisheries Service (which has since been renamed as NOAA Fisheries) issued interim final regulations to implement the Essential Fish Habitat (EFH) requirements of the 1996 Sustainable Fisheries Act. This act significantly amended the Magnuson-Stevens Fishery Conservation and Management Act of 1976. Another amendment was made in 2006 to amend the High Seas Driftnet Fishing Moratorium Protection Act.

The amended Magnuson-Stevens Act requires the following: for federal actions that may adversely affect EFH, except activities covered by a General Concurrence, federal agencies, must provide a written assessment of the effects of that action on EFH. EFH is defined as "waters and substrate necessary to fish for spawning, breeding, feeding, or growth to maturity." EFH must always include the critical habitat of endangered and threatened species.

If a project affects an endangered species of plant or wildlife, it should include mitigating measures to reduce the impact. Actions proposed in this plan could require completion of a biological assessment to demonstrate no adverse impacts to critical habitat.

4.1.14 Public Participation

The City of Snohomish has implemented a proactive and extensive public outreach and participation program as part of the development and approval of this General Sewer Plan and Wastewater Facilities Plan.

The City has conducted a SEPA review for this Plan. Public notice was initially mailed and published on February ??, 2019. The public written comment period for this notice extended until March ??, 2019. The City followed up with interagency and tribal consultations and outreach to environmental groups.

Subsequent to the SEPA public notice, the proposed draft Plan was circulated for a 30-day public and agency comment period on February ??, 2019. Following public hearings and additional opportunities to comment before the Snohomish Planning Commission and City Council, the City Council is to make its decision on adoption of the Plan in mid 2019.

4.2 State Policies and Regulations

The Clean Water Act allows states to establish more stringent water quality requirements than are required by federal law. Like most other states, Washington State has developed requirements pertaining to surface water quality more stringent than those developed by the federal government. Ecology administers the NPDES wastewater and stormwater permits and has requirements relating to protection of ground and surface waters.

Agencies other than Ecology can also have involvement in construction and operation of facilities located in critical areas. The Washington State Department of Fish and Wildlife (WDFW) has involvement in cases involving fish-bearing streams. In addition, the Washington State Department of Natural Resources (DNR) has authority for facilities to be constructed on tidelands or along shorelines. To promote efficiency and reduce overlap, state agencies and the U.S. Army Corps of Engineers developed a Joint Aquatic Resource Permit Application (JARPA), which can be submitted for the following permits:

- WDFW's Hydraulic Project Approval (HPA)
- Local agency shoreline management permits
- Department of Ecology Water Quality Certification and Approval for Exceedance of Water Quality Standards
- Corps of Engineers Section 404 and Section 10 Permits
- Marine and aquatic lease.

For actions proposed in this Plan, a JARPA may be needed for a shoreline management permit. Depending on final alignment and design considerations relating to wetlands and streams, an HPA could also be required.

4.2.1 Water Quality Standards for Surface Waters

The applicable water quality standards for construction in or near streams or the shoreline are those adopted by Ecology pursuant to Section 303 of the Federal Water Pollution Act Amendments. *Water Quality Standards for Surface Waters of the State of Washington* was last promulgated by Ecology in 1999 (WAC 173-201A). These standards describe general water quality conditions and classifications for specific surface waters and the water quality desired for each class. General conditions listed under the water quality standards are as follows:

- Existing beneficial uses shall be maintained and protected and no further degradation that could interfere with or become injurious to existing beneficial uses shall be allowed.

- Whenever the natural conditions of waters are of a lower quality than the criteria assigned, the natural conditions shall constitute the water quality criteria.

- Water quality shall be maintained and protected in waters designated as outstanding resource waters in WAC 173-201A-080. These waters are the following:

    - Waters in national parks, national monuments, national preserves, national wildlife refuges, national wilderness areas, federal wild and scenic rivers, national

seashores, national marine sanctuaries, national recreation areas, national scenic areas, and national estuarine research reserves.
- Waters in state parks, state natural areas, state wildlife management areas, and state scenic rivers.
- Documented aquatic habitat of priority species as determined by the Department of Fish and Wildlife.
- Documented critical habitat for populations of threatened or endangered species of native anadromous fish.
- Waters of exceptional recreational or ecological significance.

- Whenever waters are of a higher quality than the criteria assigned for them, the existing water quality shall be protected and pollution of said waters that will reduce the existing quality shall not be allowed, except in instances where:

  - It is clear, after satisfactory public participation and intergovernmental coordination, that overriding considerations of the public interest will be served.
  - All wastes and other materials and substances discharged into said waters shall be provided with all known, available, and reasonable methods of prevention, control, and treatment by new and existing point sources before discharge. All activities that result in the pollution of waters from nonpoint sources shall be provided with all known, available, and reasonable best management practices.
  - When the lowering of water quality in high quality waters is authorized, the lower water quality shall still be of high enough quality to fully support all existing beneficial uses.

General classifications applying to various surface water bodies not specifically classified under WAC 173-201A-130 or 173-201A-140 are as follows (applicable items only):

1. All surface waters lying within national parks, national forests, and/or wilderness areas are classified Class AA or Lake Class.

2. All lakes and their feeder streams within the state are classified Lake Class and Class AA respectively, except for those feeder streams specifically classified otherwise*.*

6. (*Items 3 through 5 not repeated herein)* All unclassified surface waters that are tributaries to Class AA waters are classified Class AA. All other unclassified surface waters in the state are hereby classified Class A.

Ecology classifies the Snohomish River as a Class A water body from the southern tip of Ebey Island (River Mile 8.1), along the City waterfront and upstream to the confluence of the Skykomish and Snoqualmie Rivers (River Mile 20.5), per WAC 173-201A-130. No other water bodies in the City are specifically listed and classified in the WAC.

Based on the 1999 Snohomish River Estuary Total Maximum Daily Load study performed by Ecology, mass load limits were implemented for discharge to the Snohomish River. The City's WWTP discharge was allocated specific mass limits, which are reflected in the NPDES permit. The City is in compliance with these limits, which is contributing to the success of the current cleanup plan for the Snohomish River Estuary.

4.2.2 Combined-Sewer Overflow Regulations

The regulation that governs facilities for reducing combined-sewer overflows (CSOs) is WAC 173-245. WAC Section 173-245-015 (1) states that "All CSO sites shall achieve and at least maintain the greatest reasonable reduction, and neither cause violations of applicable water quality standards, nor restrictions to the characteristic use of the receiving water, nor accumulation of deposits which: (a) Exceed sediment criteria or standards; or (b) have an adverse biological effect." According to WAC Section 173-245-020 (22), the term "The greatest reasonable reduction" is defined as "control of each CSO in such a way that an average of one untreated discharge may occur per year." Ecology has indicated that their interpretation of this means a total of no more than five overflow events should occur over a 5-year period.

4.2.3 NPDES Wastewater Permit

Effluent limits from the new NPDES permit, expected to become effective in the second half of 2018, are summarized in Table 4-4 below. The NPDES permit allows a monthly average influent flow of up to 2.80 MGD and influent waste loads of 3,960 pounds per day (ppd) of 5-day biochemical oxygen demand (BOD_5) and 4,400 ppd of total suspended solids (TSS). When the actual flow or waste load reaches 85 percent of any one of the design criteria for three consecutive months, or the projected plant flow or loading will reach design capacity within five years, a plan and schedule for continuing to maintain capacity must be submitted to Ecology.

The permit specifies total maximum daily load (TMDL) mass-based limits for TSS year-round; 5-day carbonaceous biochemical oxygen demand ($CBOD_5$) from November through June; and nitrogenous BOD (NBOD)+$CBOD_5$ from July through October (low river flow period). The combined mass limit of NBOD+$CBOD_5$ allows flexibility in meeting aggressive seasonal mass-based $CBOD_5$ and ammonia-N discharge limits, but still satisfies the TMDL water quality objectives for maintaining adequate dissolved oxygen (DO) in the downstream estuary. The permit also specifies effluent limits for fecal coliform bacteria, pH, total residual chlorine, and total residual peracetic acid. The limit for residual peracetic acid is included with the expectation that the City will implement a permanent installation of peracetic acid as an alternate disinfectant to chlorine, as discussed in Chapter 11.

Table 4-4 Current NPDES Permit Effluent Limits

Parameter	Effluent Limits
$CBOD_5$[(a)]	
Monthly Average	25 mg/L[(b)], 85% removal of influent $CBOD_5$
Monthly Average (Nov-Jun)	584 ppd[(c)]
Weekly Average	40 mg/L
Weekly Average (Nov-Jun)	934 ppd
NBOD[(d)]+$CBOD_5$[(e)]	(effective July – October only)
Monthly Average	134 ppd
Daily Maximum	301 ppd

Table 4-4 Current NPDES Permit Effluent Limits

Parameter	Effluent Limits
TSS[f]	
Monthly Average (Apr-Oct)	30 mg/L, 701 ppd, 85% removal of influent TSS
Monthly Average (Nov-Mar)	30 mg/L, 701 ppd, 72.2% removal of influent TSS
Weekly Average	45 mg/L, 1,051 ppd
Fecal Coliform Bacteria	
Monthly Average (geometric mean)	200 cfu/100 ml[g]
Weekly Average (geometric mean)	400 cfu/100 ml
pH	
Daily Minimum	6.4
Daily Maximum	9.0
Total Residual Chlorine	
Monthly Average	83 µg/l[h]
Daily Maximum	209 µg/l
Total Residual Peracetic Acid	
Daily Maximum	1.0 mg/L

Notes: (Effective from November 1, 2012 through October 30, 2017)
(a) $CBOD_5$ = 5-day carbonaceous biochemical oxygen demand
(b) mg/L = milligrams per liter
(c) ppd = pounds per day
(d) NBOD = nitrogenous biochemical oxygen demand
(e) NBOD+CBOD is calculated using the following equation: NBOD+CBOD (ppd) = (2.1 * total ammonia-N (ppd)) + $CBOD_5$(ppd). Calculate total ammonia-N and $CBOD_5$ using measurements from the same composite sample.
(f) TSS = total suspended solids
(g) cfu/100 ml = colony forming units per 100 milliliters
(h) µg/l = micrograms per liter

Fish Consumption Rule

Ecology recently approved a new rule that revises water quality standards under WAC 173-201A to more accurately reflect fish consumption and associated cancer risk. For most municipalities, metals such as copper, lead, mercury and zinc are the primary toxics of concern. More exotic chemical and metals are rarely measured in significant quantities in effluent from municipal wastewater treatment plants unless they are generated from a specific industry. Although conventional wastewater treatment processes typically do not directly affect the presence of metals, there can be incidental uptake/removal of metals through the treatment processes. Often times, if metals exceed the water quality standards, they often are best controlled at the source (e.g., pH control in potable water systems to avoid leaching of lead). However, some municipal wastewater treatment plants do attempt to remove certain metals (e.g., using alum to precipitate copper) or increase hardness so that there is a smaller dissolved fraction.

Because the new rule did not change the water quality standards for copper, lead, mercury or zinc that were in effect during the most recent mixing zone study (Cosmopolitan Engineering, 2001), it is unlikely that the reasonable potential for exceeding the water quality standards would be any different. Therefore, this new rule should not require any action by the City to make improvements to the WWTP. However, the proposed re-rating of the WWTP will require a new mixing zone study and reasonable potential analysis to confirm that the increased volume of discharge will still comply with the water quality standards. Given that the proposed re-rating is for a modest increase in capacity, it is anticipated that there still would not be reasonable potential to exceed the water quality standards for metals.

Future Treatment Requirements

Section 305(b) of the Clean Water Act requires each state to submit a biennial report to Congress evaluating the quality of its streams, rivers, and lakes. Starting with the 2010 Water Quality Assessment, Ecology, with U.S. Environmental Protection Agency (EPA) approval, began using a rotating system to complete the assessment. The 2010 cycle focuses on marine waters and the 2014 cycle focused on fresh waters.

The most recent assessment of fresh waters by Washington State, published in 2016, lists the segment of the Snohomish River to which the WWTP discharges as Category 5 for temperature, Category 2 for pH, and Category 1 for bacteria and ammonia-N (Table 4-5). Swifty (Ferguson) Creek, Blackmans Lake, and Cemetery Creek are also on the 303 (d) Water Body Listing for bacteria (Table 4-5).

Table 4-5 Water Quality Assessment in City of Snohomish

303 (d) / 305 (b) Water Body Listing	Parameter	Previous (2012)	Current (2016)	Change in Water Quality
Snohomish River				
- Downstream of WWTP	Temperature	Category 2	Category 5	Worsened
- Downstream of WWTP	pH	Category 1	Category 2	Worsened
- Downstream of WWTP	Bacteria	Category 5	Category 1	Improved
- Downstream of WWTP	Ammonia-N	Category 1	Category 1	No Change
Swifty (Ferguson) Creek	Bacteria	Category 3	Category 5	Worsened
Blackmans Lake	Bacteria	Category 5	Category 5	No Change
Cemetery Creek	Bacteria	Category 3	Category 5	Worsened

A Category 5 listing means data show that established water quality standards for the receiving water have been violated for one or more pollutants and that no TMDLs are currently in place to address those pollutants. A Category 5 listing requires action to develop TMDLs for the constituents of concern.

A Category 3 listing means that water there is insufficient data to meet minimum requirements according to Ecology's Policy 1-11.

A Category 2 listing means evidence indicates that a water quality problem may exist, but the data is insufficient to clearly establish the existence of a problem. Perhaps collected data shows water quality is close to violating the standards or, where collected data implies violation of standards, proper scientific methods were not used to collect the data. In either case, Ecology typically continues to monitor these constituents of concern.

A Category 1 listing means that the water body met standards for the tested pollutants.

Based on the most recent water quality assessment, it appears that Ecology may in the future conduct a TMDL for temperature in the Snohomish River. If Ecology initiates a TMDL study for temperature, it is recommended that the City participate in the study by assisting with data collection and providing input on the process. This will ensure that impacts from the WWTP are accurately represented in the study. Furthermore, the City should consider conducting a dynamic temperature analysis as part of the next mixing zone study, which would be performed as part of the WWTP re-rating effort recommended in Chapter 9, to more accurately predict actual conditions and subsequent impact from the WWTP based on historical data.

In addition to constituents listed in the 2016 Water Quality Assessment, future water quality requirements also depend on the continued success of the current cleanup plan for the Snohomish Estuary. Additional TMDLs could be developed for nutrients if the current TMDLs for $CBOD_5$ and Ammonia-N do not return the Snohomish Estuary to compliance with the water quality standards for DO. However, this result seems unlikely, given that the 2016 Water Quality Assessment does not list dissolved oxygen for the Snohomish River or Snohomish River Estuary and the listing for ammonia-N has remained unchanged at Category 1. The current listings suggest that the TMDL limits currently in place have achieved the desired improvement in water quality.

4.2.4 NPDES Stormwater Permit

Construction projects that disturb one or more acres of land require a construction general permit for stormwater discharge under NPDES requirements. Mitigation measures are required, including preparation of a Storm Water Pollution Prevention Plan. During construction, temporary erosion and sediment control measures must be implemented as identified in the Storm Water Pollution Prevention Plan.

4.2.5 Washington State Standards for Use and Disposal of Sludge

WAC 173-308, *Biosolids Management,* establishes guidelines for treatment and land application of biosolids generated by municipal wastewater treatment facilities. These mirror the federal guidelines in 40 CFR 503 that are described earlier in this chapter. Ecology has authority to enforce these rules and may, if it chooses, delegate some of the authority to local health departments.

4.2.6 Washington Department of Ecology Criteria for Sewage Works Design

The *Criteria for Sewage Works Design* (Ecology, 2008), also known as the “Orange Book”, is a guide for design of sewage collection and treatment systems. The primary goals of the manual are as follows:

- To ensure that the design of sewage collection and treatment systems is consistent with state public health and water quality objectives

- To establish a basis for the design and review of plans and specifications for sewage treatment works and sewerage systems
- To establish the minimum requirements and limiting factors for review of sewage treatment work and sewerage system plans and specifications
- To assist the owner or the owner's authorized engineer in the preparation of plans, specifications, reports, and other data
- To guide departments in their determination of whether to issue approvals, permits, or certificates for sewage treatment works or sewer systems.

Ecology uses the Orange Book design guidelines to review and approve reports, plans, and specifications. Design guidelines presented in the Orange Book have been referenced to evaluate the capacity of the City's collection system and WWTP and help establish criteria for proposed improvements. The Orange Book also presents guidelines for reliability criteria. In general, state requirements for reliability criteria follow the federal requirements outlined in Table 4-3. The state reliability classification scheme is shown in Table 4-6. The wastewater facilities proposed in this Plan will comply with the EPA and Ecology Class II reliability criteria.

Table 4-6 Reliability Classifications in the Orange Book

Reliability Class	Applicable Facilities
I	Works whose discharge, or potential discharge, (1) is into public water supply, shellfish, or primary contact recreation waters, or (2) as a result of its volume and/or character, could permanently or unacceptably damage or affect the receiving waters or public health if normal operations were interrupted.
II	Works whose discharge, or potential discharge, as a result of its volume and/or character, would not permanently or unacceptably damage or affect the receiving waters or public health during periods of short-term operations interruptions, but could be damaging if continued interruption of normal operations were to occur (on the order of several days).
III	Works not otherwise classified as Reliability Class I or II.

4.2.7 Standards for Water Reclamation

In September 1997, the Washington State Departments of Health and Ecology jointly released a set of standards for wastewater reclamation projects, governed by Chapter 90.46 of the Revised Code of Washington (RCW). The *Water Reclamation and Reuse Standards* described the treatment and quality requirements for a variety of end uses and defined four basic classes of reuse quality, along with their suitability for various end uses. The four classes varied from Class A (highest quality) to Class D (lowest quality).

Ecology recently enacted a new rule codified as Chapter 173-219 of the WAC. This rule is an implementation of Chapter 90.46 of the RCW and establishes requirements for the generation, distribution, storage and use of reclaimed water. Although this rule has many similarities to the previous *Water Reclamation and Reuse Standards*, it also includes some significant differences. For example, the new rule includes only two classifications of reclaimed water, Class A and B.

4.2.8 State Dam Safety Regulations

Impoundments that store more than 10 acre-feet of water, such as the lagoons at the City's WWTP, are subject to the state dam safety regulations outlined in WAC 173-175. Exemptions may be obtained from the Dam Safety Unit at the Department of Ecology if embankment heights are less than 6 feet. No changes to the existing lagoon embankments are proposed in this Plan.

4.2.9 Joint Aquatics Resources Permit

If construction will be performed in or near state waterways, a joint aquatic resources permit application (JARPA) may need to be prepared. To promote efficiency and reduce overlap, state agencies and the U.S. Army Corps of Engineers developed the JARPA, which can be submitted for the following permits:

- WDFW's Hydraulic Project Approval (HPA)
- Local agency shoreline management permits
- Department of Ecology Water Quality Certification and Approval for Exceedance of Water Quality Standards
- Corps of Engineers Section 404 and Section 10 Permits
- Marine and aquatic lease.

As mentioned previously, JARPA may be necessary for certain actions proposed in this Plan.

4.2.10 State Environmental Policy Act

The State Environmental Policy Act (SEPA) is Washington State's parallel statute to NEPA. Similar to the federal law and White House Council on Environmental Quality NEPA Rules, Ecology issues statewide rules for SEPA compliance, and each agency of the state, including municipalities such as cities and counties, has a set of implementing procedures and carries out SEPA for its own proposed actions. Ecology's process for coordinating environmental review for grant funding is discussed below. The local SEPA process is discussed in the Local Policies section of this chapter.

4.2.11 State Environmental Review Process

To be eligible for financial assistance from state water quality grants and loans administered by Ecology, this Plan must comply with the State Environmental Review Process (SERP; WAC 173-98-100). The SERP was established to help ensure that environmentally sound alternatives are selected and to satisfy the state's responsibility to help ensure that recipients comply with NEPA and other applicable environmental laws, regulations, and executive orders. At this time, the City has not identified any proposed actions in this Plan for potential state funding. However, should that change, this Plan may be amended to include additional documentation as may be required to satisfy SERP requirements for a particular project action.

4.2.12 Department of Archaeology and Historic Preservation Approval

Cultural resources are addressed in over 100 federal laws, regulations, and guidelines, including NEPA and the National Historic Preservation Act of 1966, amended in 1992 (NHPA). Section 106 of the NHPA requires federally assisted undertakings to account for the effects of those undertakings on historic properties that are included in or may be eligible to be included in the National Register of Historic Places. "Historic properties" refers to prehistoric archaeological sites as well as buildings, structures, and other historic sites.

Applicable state laws include the Indian Graves and Records Act (RCW 27.44), which prohibits knowingly disturbing a Native American or historic grave, and the Archaeological Sites and Resources Act (RCW 27.53), which requires that anyone proposing to excavate into, disturb, or remove artifacts from an archaeological site on public or private lands obtain a permit from the Department of Archaeology and Historic Preservation.

Three elements are involved in cultural resources studies following Section 106 procedures:

1. The identification and evaluation of historic properties.
2. Assessment of effects of the proposed undertaking on historic properties.
3. Consultation among principal parties to consider ways to avoid, reduce, or mitigate adverse effects.

The first element, identification and evaluation, is of most concern at the beginning stages of projects. Methods for identification of historic properties consist of archival research, field survey, and consultation.

Archival research, including a check of the Washington state site inventory and records at the Department of Archaeology and Historic Preservation (DAHP), is conducted prior to any field activity in order to determine if sites are already recorded in the project area or its vicinity. Other information is collected from ethnographic and historic accounts, previous regional cultural resource investigations, informants, maps, photographs, and environmental information. Research to determine the age of landforms involved and the extent of modern disturbance are especially important. Locations of archaeological sites may be identified by this process. The potential for buried and hence undiscovered sites, or uplifted former shorelines favorable for habitation, may also be determined. Field visits are made after completion of the background research to verify field conditions, discuss construction locations and methods, and to identify historic properties. The results of these investigations are presented in a report for submittal to appropriate agencies, the DAHP, and, in this case, to the Tulalip Tribes. The report includes recommendations for dealing with any sites discovered, additional discovery measures, if necessary, monitoring high-potential locations, and a Discovery Plan to be enacted in the event archaeological material is encountered during construction.

At this point, impact to cultural resources are not expected as a result of the actions proposed in this Plan, but this will need to be confirmed as specific project actions are taken. If potential impacts are identified, the procedure outlined above will need to be implemented.

4.3 Local Policies

4.3.1 Puget Sound Clean Air Agency

The Puget Sound Clean Air Agency (PSCAA) is a local regulatory agency with jurisdiction over air emissions in the Puget Sound region (King, Kitsap, Pierce and Snohomish Counties). The agency's primary concern with wastewater treatment facilities is from odor generation. If odor-producing facilities are designed, PSCAA should be consulted for input and comments. At least 60 days prior to the construction of such facilities, a notice of construction (NOC) must be filed with PSCAA. An NOC permit application must be approved in order to construct, erect, install, alter, reconstruct, or relocate any stationary or portable device capable of releasing contaminants in the atmosphere.

The proposed actions in this Plan are not expected to involve modifications to an existing process or addition of a new process that produces air contaminants requiring an NOC.

4.3.2 Fire Code

County fire officials have authority to enforce the International Fire Code adopted by the State of Washington under RCW 19.27.

4.3.3 City Service Area Policies

Developers are required to abide by City standards. All developers should consult with City staff to ensure compliance with City regulations and ordinances.

4.3.4 Shoreline Substantial Development Permit

If construction will be performed within 200 feet of ordinary high-water mark of any significant water body, a shoreline substantial development permit will be required. It is possible that actions proposed in this Plan will involve construction activity within 200 feet of a significant water body, including portions of the WWTP near the Snohomish River.

4.3.5 Local Building Codes and Permits

All new construction must abide by City and County building codes and required permits. Project documents must be reviewed by the City building department, as applicable.

4.3.6 County Requirements for Archeological Sites

The Snohomish County Department of Planning and Development Services requires protection of state-registered archeological sites and mitigation of impacts to such sites in accordance with RCW 27.53. For projects near registered sites, the County requires that a professional archaeologist evaluate the site to determine potential impacts and recommend mitigation. An EZ-1 Historical and Cultural Resources Review form should be submitted to the Department of Archaeology and Historic Preservation if it is not clear whether or not the project site is near an archeological or historic site. They will evaluate the site and decide if RCW 27.53 applies to the project. If historical or archeological resources are encountered during construction, the County requires all work to halt and state, county and the local tribal authorities to be notified immediately. Local tribal addresses and telephone numbers include the following:

Tulalip Tribes of Washington
6406 Marine Drive
Tulalip, WA 98271
1-800-869-8287

4.3.7 SEPA Review

The City issued a Determination of Non-Significance on ?????????, 2019 for the SEPA review associated with this Plan, which was publicly noticed, as well as circulated to agencies with jurisdiction. As this SEPA review is for non-project action, additional SEPA review will be required for specific project actions.

4.3.8 Critical Areas Review

In noting the importance of sensitive habitats and wildlife species, and in complying with the Washington State Growth Management Act of 1990, the Environmental Protection Element of the City of Snohomish's Comprehensive Plan provides the policy framework for critical

protection regulations. Snohomish County Planning and Development Services have developed interactive maps using a GIS database to identify critical areas, land use designations, etc. Critical areas located on these maps include wetlands, streams, floodplains, and geologic hazard areas. Critical areas are also addressed in the City of Snohomish Municipal Code (SMC) Chapter 14.51 Drainage Basin Protection.

4.3.9 Shoreline Management Program

The City of Snohomish has adopted a Shoreline Management Program as required by the Shoreline Management Act of 1971 (RCW 90.58). Shorelines covered by each Shoreline Management Program generally include all water areas of the state, including marine and fresh waters and their associated wetlands, together with the underlying lands, except for the following:

- Shorelines along streams and their associated wetlands where the mean annual flow is less than 20 cubic feet per second
- Shorelines of lakes less than 20 acres in area.

Shoreline jurisdiction includes lands extending landward for 200 feet in all directions or measured on a horizontal plane from the ordinary high-water mark.

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Chapter 5 Collection System Design Criteria and Flows

5.1 Existing Wastewater Flows

The City's WWTP Discharge Monitoring Reports (DMRs) from 2011 through 2015, hourly flow data from the WWTP flow meter, and flow data from meters installed at Avenue A and 8th Street, the Rainier Pump Station and on 8th Street by Goodwill were used to determine existing wastewater flows in the City. The following is a description of each calculated flow parameter:

- **Average Annual Flow** – This flow condition is defined as the average of daily flows during the year.
- **Average Dry Weather Flow** – This flow condition is defined as the average daily flow for a period during the months of June through September when little rainfall was recorded. The intent of presenting this data is to capture the base domestic flow conditions with the minimum impact from infiltration and inflow (I/I).
- **Average Wet Weather Flow** – This flow condition is defined as the average daily flow from the months of November through March. All flows during this period were analyzed regardless of the amount of precipitation.
- **Maximum Month Flow** – This flow condition is defined as the highest monthly average flow. This flow condition is of particular interest for the WWTP because the NPDES permit is written with monthly discharge limitations based on this flow.
- **Peak Day Flow** – This flow condition is defined as the maximum daily flow in a given year.
- **Peak Hour Flow** – This flow condition is defined as the peak sustained flow rate occurring during a one-hour period. It is used to size the collection and interceptor sewers, pump stations, flow meters, and WWTP hydraulic processes.

5.1.1 Average Annual Flow

Figure 5-1 graphically shows the average annual flow 2006 through 2015. Since the system is partially combined, with some areas combining the stormwater system with the wastewater system, the average annual flow is greatly affected by the rainfall during the year. This relationship can be seen in Figure 5-1 where the total annual precipitation for each year is plotted along with the average annual flow for each year. Due to the significant variation in rainfall over the last couple years, the overall average annual flow was calculated as the average of the last five years of annual flow, thus capturing some wet years and some dry years, with an overall average annual flow of 1.30 MGD.

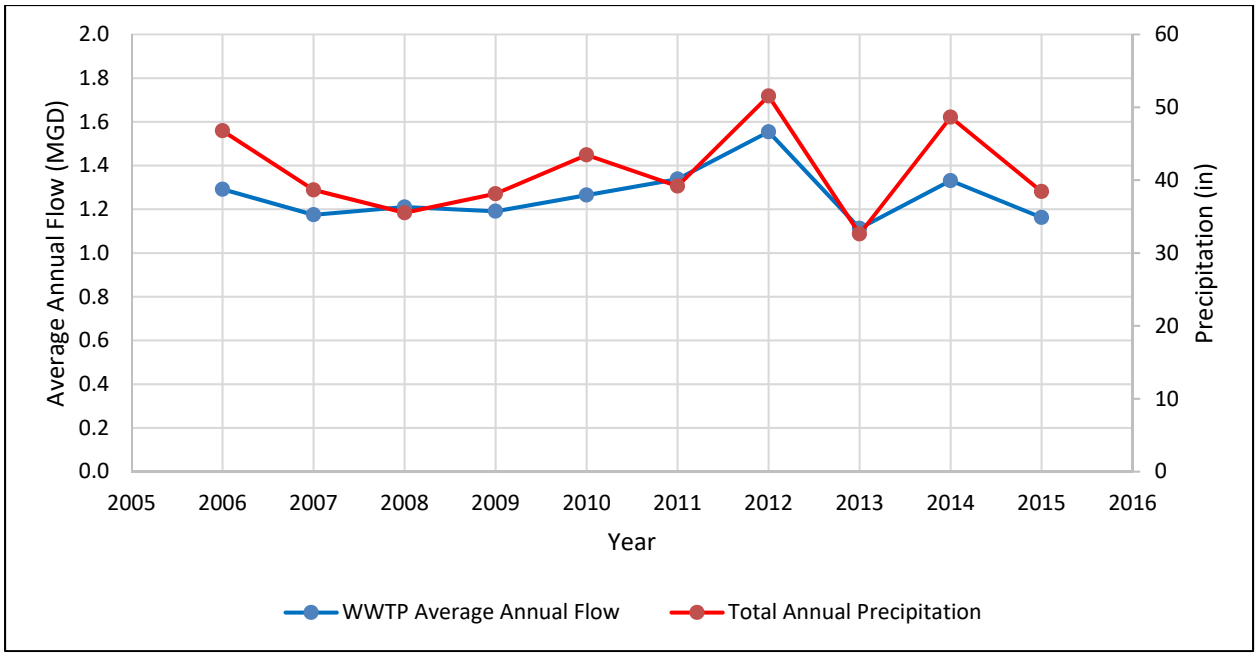


Figure 5-1 Average Annual Flow and Total Annual Rainfall

The average dry weather flow (DWF) is the flow during the months when the groundwater is at its lowest and there is minimal rain, which is often June through September. Figure 5-2 shows the average flow for each of the dry weather months between 2013 and 2015. Since the City of Snohomish has several months with little to no rain, the average DWF was calculated based on those months. Months during the dry season which had significant rain were not included, resulting in an average DWF of 0.68 MGD.

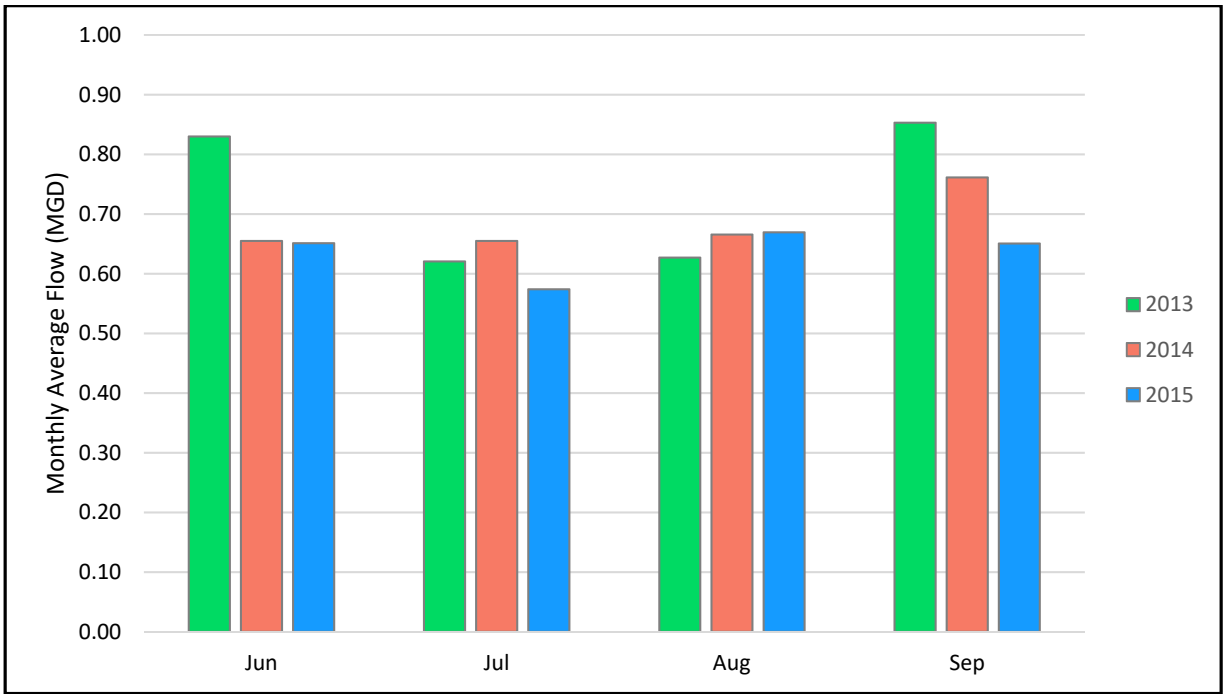


Figure 5-2 Average Flow During Dry Weather Months

The development of the population distribution and projections is described in Chapter 3. The wastewater contributions are broken into residential and employment components. While there is an employment sewage component, there are no significant industrial discharges to the City's sewer system.

The per capita residential and employment flow rates were calculated using population breakdowns and the average DWF from the flow meters at Ave A and 8th Street and the WWTP. The known average DWF at the Avenue A and 8th Street flow meter with the known residential and employment populations contributing to that meter were used in conjunction with the known average DWF at the WWTP and its contributing populations to estimate residential and employment per capita flows. The calculated per capita residential and employment values are shown in Table 5-1.

Table 5-1 Sewer Unit Flow Rates

Residential, gallons per capita per day (gpcd)	Employment, gallons per employee per day (gped)
60.0	14.3

5.1.2 Monthly Average Day Flow

Monthly average flows for the last 10 years are provided with the DMR data in Appendix E. The five-year monthly average is provided in Table 5-2. Table 5-3 shows the average daily flow, average DWF, and average wet weather flow (WWF) between 2011 and 2015.

Table 5-2 Five-Year Monthly Average Daily Flow at the WWTP (2011 – 2015)

Average Monthly Flow (MGD)											
Jan	**Feb**	**March**	**April**	**May**	**June**	**July**	**Aug**	**Sept**	**Oct**	**Nov**	**Dec**
1.84	1.64	1.98	1.40	1.12	0.95	0.72	0.71	0.75	0.98	1.60	1.92

Table 5-3 Five-Year Average Flows at the WWTP (2011 – 2015)

Average Daily Flow (MGD)	Average Dry Weather Flow (MGD)	Average Wet Weather Flow (MGD)
1.30	0.68	1.79

5.1.3 Maximum Month and Peak Day Flows

The maximum month and peak day flows recorded at the City of Snohomish WWTP from 2011 through 2015 are summarized in Table 5-4. The data shows that the wettest months are generally November through January.

Table 5-4 Maximum Month and Peak Day Flows (2011 – 2015)

Year	Maximum Month Flow (MGD)	Month	Peak Day Flow (MGD)	Day
2011	2.27	**March**	6.09	13-Mar-11
2012	2.73	**December**	8.24	19-Nov-12
2013	2.24	**January**	7.64	29-Jan-13
2014	2.56	**March**	6.47	16-Mar-14
2015	2.67	**December**	8.30	8-Dec-15
Average	2.49		7.35	

5.1.4 Peak Hour Flows

There was very little hourly flow data available for the WWTP, which did not provide sufficient data to calculate peak hour dry weather flow and peak hour wet weather flow and the associated peaking factors.

The peak hour dry weather flow was estimated using data from the flow meter at Avenue A and 8th Street (Avenue A meter). A diurnal pattern was calculated for residential flow from data at that meter as discussed below in Section 5.2. The peak hour factor from the diurnal pattern was used to calculate the peak hour flow at the WWTP. A factor of 1.55 was applied to the average DWF at the WWTP to get the peak hour DWF of 0.99 MGD.

The peak hour WWF was determined using a computer model (discussed in Chapter 7) of the existing system for a derived 25-year storm. The resulting current peak hour WWF predicted by the model is 11.94 MGD.

5.1.5 Peaking Factors

The peaking factors shown in Table 5-5 were calculated by dividing the highest maximum month and highest peak day flows for 2011 through 2015 and the model predicted peak hour flow of 11.94 MGD by the calculated average annual flow of 1.30 MGD between 2011 and 2105. The resulting peaking factors are high, since this is a partially combined system as discussed in Chapter 6.

Table 5-5 Wastewater Flow Peaking Factors (2011 – 2015)

Maximum Month to Annual Average Flow	2.10
Peak Day to Annual Average Flow	6.38
Peak Hour to Annual Average Flow	9.19

5.2 Diurnal Curve

Typically, sewer flows for residential areas are lowest at night and highest during the morning and evening. In commercial areas the peak is during the middle of the day around lunch time. This distribution of flow throughout the day is described by diurnal curves. The sanitary sewer diurnal curve was calculated by normalizing the average annual flow on an hourly basis. A normalized curve is unitless and has an average of 1 and can be multiplied by the daily flow to determine the flow based on the time of day. The residential average diurnal curve was

calculated using flow data from the Avenue A meter and the commercial average diurnal curve was calculated using flow data from the meter at 8th Street and Goodwill. The curves are presented as Figure 5-3. These curves are used by the computer model described in Chapter 7 to simulate flow variations throughout the modeled time period.

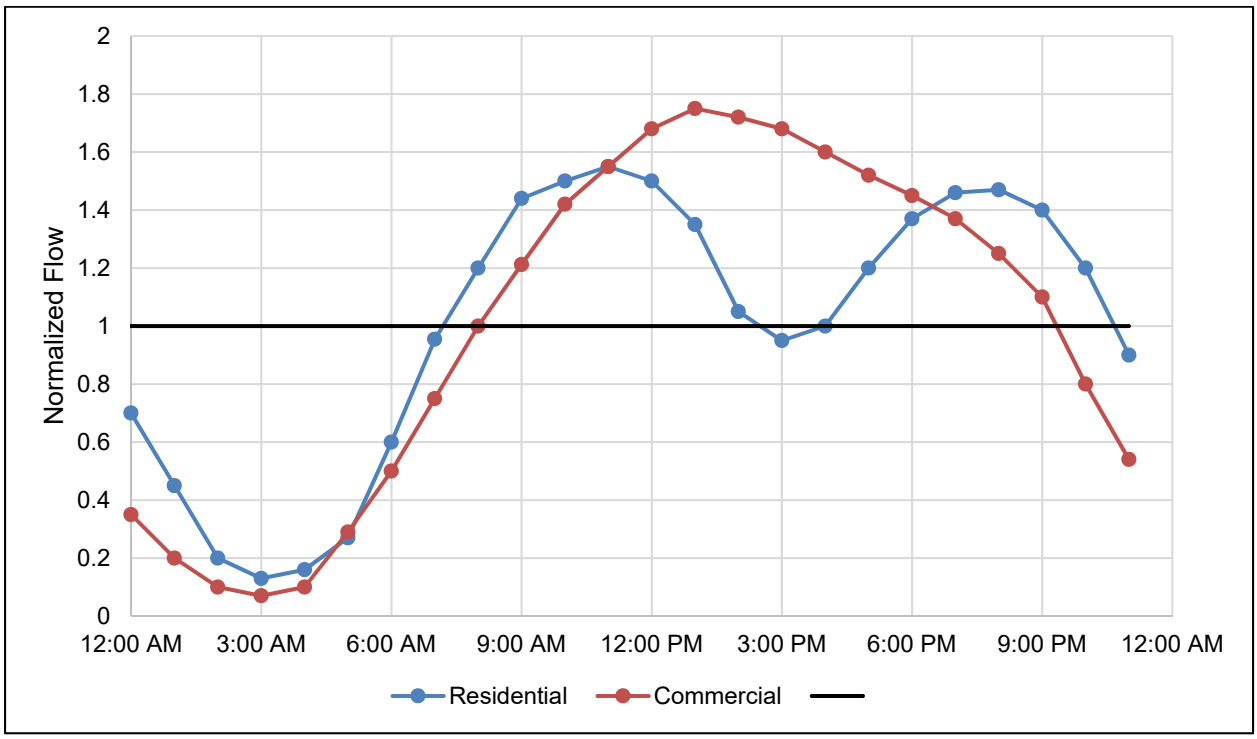


Figure 5-3 Residential and Commercial Sewer Diurnal Curves

5.3 Infiltration and Inflow Analysis

Infiltration and inflow (I/I) is the wastewater component consisting of stormwater surface runoff entering the sewer system as inflow and infiltration of groundwater into the sewer system. Inflow typically enters the sewer system from storm sewer connections, basement sump pumps, roof drains, and submerged manholes. Infiltration occurs as groundwater leaks into the sewer system through cracked or broken pipes and manholes or through loose joints and connections. I/I is important in determining the peak day and peak hour flows throughout the system. I/I can vary significantly due to changes in groundwater tables, rainfall intensity, rainfall duration, and rain event peak timing during the day.

The DWF for the calculation of I/I is defined as the dry weather flow (no significant inflow) while there is high ground water. The DWF was calculated using flow data from the Avenue A meter between 12/5/13 and 12/11/13 when there was no rain with three dry days prior. December is part of the wet season when the ground water would be high, such that the effects of infiltration should be fully represented by this time period.

WWF is the average daily flow during a period of significant rainfall. For this system, the flow from the storm on 11/14/2015 through 11/18/2015 was averaged to get a WWF.
The base I/I or groundwater infiltration (GWI) was quantified by averaging the nighttime flows over several days, during dry weather conditions. The nighttime flows were assumed to be mostly groundwater.

The base sanitary flow (BSF) is the difference between the DWF and the GWI. The average inflow is the difference between the WWF and the DWF.

The Avenue A meter was used to calculate the per-person inflow and infiltration. All of the area contributing to the meter consists of separated sewers and does not include any combined sewers that would otherwise inflate the contributions from inflow. The I/I analysis is presented in Table 5-6.

Table 5-6 I/I Characteristics at Ave A Flow Meter

Dry Weather Flow (DWF) (gpd) [(a)]	146,831
DWF / Person (gpcd)[(b)]	78
Wet Weather Flow (WWF) (gpd)[(c)]	720,130
WWF / Person (gpcd)[(b)]	384
Base I/I or Groundwater Infiltration (GWI) (gpd)	78,419
Base Sanitary Flow (BSF) (gpd)	68,412
Average Inflow (gpd)	573,299
Population Contributing to the Avenue A Meter	1,874
GWI / Person (gpcd)[(b)]	42
Average WW Inflow / Person (gpcd)[(b)]	306

Note:
(a) The DWF for the calculation of I/I is defined as the dry weather flow while there is high ground water. The DWF was calculated using flow from 12/5/13 through 12/11/13 when there was no rain with three dry days prior.
(b) gpcd is the gallons per capita per day.
(c) WWF was calculated using flow data from the Avenue A meter during the storm on 11/14/15 through 11/18/15.

Assessing the degree of infiltration was based on the EPA publication “Infiltration/Inflow – I/I Analysis and Project Certification” dated May 1985, which was reissued by Ecology as Publication No. 97-03. This publication establishes the following thresholds for excessive infiltration and inflow:

- If DWF during a period of seasonal high groundwater is less than 120 gpcd, infiltration is non-excessive.
- If WWF is less than 275 gpcd and the WWTP does not experience hydraulic overloads during storm events, inflow is non-excessive.

Based on the above criteria and the values presented in Table 5-6, the estimated infiltration rate of 42 gpcd into the system is not excessive. However, the estimated inflow rate of 306 gpcd into the separated system tributary to the Avenue A meter is over the threshold of 275 gpcd and is deemed to be excessive, even though the WWTP is capable of handling the hydraulic load. It should be noted that the area tributary to the Avenue A meter is known to have the highest I/I in the system. Pump station run times and anecdotal evidence noted by the City indicate that most of the collection system has lower I/I than the area contributing to the Avenue A meter. Therefore, the I/I applied to the rest of the system in the model was 20% less than the area

contributing to the Avenue A flow meter, based on consideration and comparison of pump station run times.

The degree of inflow suggests that there are many sources that have stormwater directly connected to the sewer system, even beyond the known storm drain connections. The City could conduct a detailed survey to determine the sources of the stormwater inflow. As identified in Chapter 12, the City plans to separate the storm and sanitary sewers along Avenue F, which is tributary to CSO #1, as part of the 6-year capital improvement plan. This is estimated to reduce total inflow by about 6%. As discussed in Chapter 12, there will be additional money allocated towards further separation of the storm and sanitary sewers in the drainage areas tributary to CSO #1 and #2. Because the timing and degree of separation are dependent on a number of factors, such as ability to combine these separation projects with other work (e.g., street overlays and other utility pipe replacements) and availability of funding, the flow projections conservatively assume no further storm and sanitary sewer separations beyond Avenue F. However, when further separation is completed, it will have a significant impact on I/I. For example, separating storm and sanitary sewers tributary to CSO #1 between Avenue E and Riverview Lane would yield a total estimated inflow reduction of approximately 30%.

I/I was last evaluated in detail as part of the 2005 Plan. Unlike for this Plan, where flow monitors were located at strategic points throughout the collection system during both dry and wet weather flow conditions, the 2005 Plan utilized pump runtime data from the two largest pump stations to estimate I/I. The 2005 Plan acknowledges that the I/I estimates were “very approximate” because of uncertainties in the actual pumping rates and at the time issues with backflow into the wet wells due to leaking check valves and inaccurate runtime meters. The 2005 Plan estimated I/I rates for the maximum month of 850 gallons per acre per day in the separated sewer areas. Multiplying that I/I rate by the separated sewer service area of 1,175 acres at that time and dividing by the estimated population of 6,570 within that area (proportioned based on a total service population of 8,390 and a total service area of about 1,500 acres) yields an I/I factor of 152 gpcd for the maximum month. This is less than half the estimated 348 gpcd average wet weather I/I for the area tributary to the Avenue A meter noted in Table 5-6 above, which would be comparable to the maximum month I/I. Although the 2005 Plan estimated a much lower I/I on average, the peak day and peak hour I/I rates estimated in the 2005 Plan resulted in similar or substantially higher peak flow projections for similar service populations. For example, as shown in Table 5-7 below, the projected peak day and peak hour flows are 9.42 MGD and 14.74 MGD, respectively, for 2036 (including the UGA and a portion of the North Planning Area) with a population of 14,505 and separation of storm and sanitary sewers along Avenue F only. Comparatively, the 2005 Plan projected peak day and peak hour flows of 9.4 MGD and 28.9 MGD for a slightly smaller population of 14,133 and no separation of storm and sanitary sewers. Given known issues with the data and the high variability of results, a meaningful comparison cannot be made with the 2005 Plan I/I analysis to determine if I/I has since increased, decreased or not changed significantly. Even a simple comparison of flows to the WWTP would not be accurate because prior to completion of the CSO Pump Station in 2012 there were a number of CSO events during peak flows that would reduce the measured influent flow to the WWTP.

5.4 Projected Flows

Wastewater flows were projected to future conditions based on designated land uses and Snohomish County population allocations for the current and future service areas. Unit flows developed above are assumed to remain constant throughout the planning horizon. Much of the City's existing sewer collection system was built 70 years ago or more and is assumed to have reached a state where further significant increases in I/I is unlikely. Therefore, a constant I/I rate was assumed for future I/I projections, as discussed in Chapter 7. However, as noted previously, the flow projections conservatively assume no separation of storm and sanitary sewers beyond separation along Avenue F. A summary of unit flows and projected flows for the planning horizon are shown in Table 5-7.

Although the model accurately calculated the average and maximum month flows, it predicted peak day flows that were a little lower than historically recorded. Therefore, the model predicted peak day flows were adjusted proportionally based on the difference between the 2016 model predicted peak day flow of 6.89 MGD and the actual peak day flow of 8.30 MGD. This increased the projected peak day flows by approximately 20% compared to what the model predicted. Similarly, the proportional increase in the model predicted and adjusted peak day flow was applied to the model calculated peak hour flow, since it was assumed the predicted peak hour flow may also be lower than the actual peak hour flow. These adjustments provided more conservative peak flow projections than what the model predicted, which should be more representative of measured peak flows.

Table 5-7 Unit and Projected Wastewater Flows

Parameter	2016	2022	2036 (Current City Limits Only)	2036 (UGA and Portion North Planning Area)	Buildout
Served Residential Population	9,569	10,348	12,358	14,505	17,995
Residential Flow (gpcd)	60.0	60.0	60.0	60.0	60.0
Employment Population	4,677	5,027	6,111	6,223	7,236
Employment Flow (gped)	14.3	14.3	14.3	14.3	14.3
Average Dry Weather Flow (MGD)	0.64	0.69	0.83	0.96	1.18
Average Annual Flow (MGD)	1.30	1.40	1.68	1.95	2.40
Maximum Month Flow (MGD)	2.73	2.80	2.95	3.22	3.44
Model Predicted Peak Day Flow (MGD)	6.89	6.72	7.06	7.82	7.95
Adjusted Peak Day Flow (MGD)	8.30	8.10	8.50	9.42	9.58
Model Predicted Peak Hour Flow (MGD)	11.94	11.34	11.93	12.24	14.01
Adjusted Peak Hour Flow (MGD)	14.38	13.67	14.36	14.74	16.88

The projected wastewater flows are based on both an increasing population as well as an expanding sewer system service area. However, it should be noted that flows, and peak flows in particular, could be reduced significantly from the projected values when the City implements further sewer separation beyond the separation along Avenue F that is already factored into the above flow projections. Table 5-8 lists the progressive expansion of the sewer system by the percentage of each sub-basins' population that is projected to be served by that year and Table 5-9 provides the total population served by basin. Sub-basins are shown on Figure 3-2.

Table 5-8 Percent of Sub-Basin Population Served

Basin	2016	2022	2036 without North Planning Area	2036 with North Planning Area	Buildout
100	0%	0%	0%	100%	100%
101	0%	0%	0%	100%	100%
102	0%	0%	0%	100%	100%
103	0%	20%	50%	50%	100%
105	0%	0%	0%	100%	100%
106	0%	0%	100%	100%	100%
107	0%	20%	50%	50%	100%

Table 5-9 Total Sub-Basin Population Served

Basin	2016	2022	2036 without North Planning Area	2036 with North Planning Area	Buildout
1	421	424	433	433	443
2	155	208	330	330	501
3	293	294	298	298	301
4	20	20	20	20	20
5	146	146	146	146	146
6	249	272	327	327	368
7	33	46	75	75	100
8	627	677	794	794	900
9	190	271	461	461	581
10	361	407	515	515	606
11	232	266	344	344	434
12	426	442	480	480	525
13	300	306	318	318	333
14	1,271	1,427	1,791	1,791	2,207
15	312	371	507	507	600
16	543	551	570	570	583
17	395	400	411	411	435
18	326	327	329	329	331
19	227	228	230	230	232

Table 5-9 Total Sub-Basin Population Served					
20	14	14	15	15	16
21	768	795	834	834	893
22	460	463	469	469	477
23	213	213	213	213	213
24	228	233	244	244	258
25	105	108	116	116	123
26	65	69	78	78	89
27	406	407	409	409	411
28	339	343	352	352	363
29	189	193	201	201	217
30	265	318	443	443	560
31	169	194	251	251	305
32	163	177	209	209	246
33	377	390	422	422	446
34	313	334	384	384	422
35	216	260	362	362	451
36	0	0	0	0	0
37	342	440	668	668	865
38	149	149	149	149	149
39	621	655	735	735	796
40	20	20	20	20	20
41	535	549	584	584	607
42	436	437	439	439	441
43	518	536	579	579	628
44	14	14	14	14	18
45	375	431	563	563	714
46	210	210	210	210	210
47	212	240	304	304	346
100	0	0	0	1,025	1,558
101	0	0	0	857	1,047
102	0	0	0	314	429
103	0	61	222	222	604
104	0	0	0	0	0
105	0	0	0	62	62
106	0	0	387	387	469
107	0	41	215	215	1,131

Chapter 6 Collection System Description

6.1 History of System Development

The City of Snohomish was incorporated in 1890, with logging and shipping on the Northern Pacific Railroad being the primary commercial activity. The first sewage collection systems were installed in the early 1900s and operated as a combined sanitary/storm sewer system with various outfalls to the Snohomish River. The combined sewer system (CSS) was expanded to meet the needs of the population growth until the 1950s, when the new sewers constructed were sanitary only.

The remainder of the City's sewer system, lying outside of the CSS area, consists of PVC and concrete sewer pipe, most of which have been installed since the late 1950s. There are 15 sewage pump stations serving the sanitary system, three of which pump flows are generated from the CSS.

The City's wastewater treatment plant is located in the southwest corner of the City, just north of the Snohomish River and west of State Route 9. The City's first treatment plant, a facultative stabilizing lagoon, was constructed in 1958 to eliminate the discharge of raw sewage into the Snohomish River. In 1995, the plant was upgraded to a multi-lagoon aerated system. The plant was upgraded again in 2012 to include submerged fixed-film media in the lagoons. The City's sewer system is shown in Figure 6-1.

6.2 On-site Sewer Systems

Based on water, sewer and garbage billing account information provided by City staff, it is estimated that 37 households within the current city limits are not connected to the City's sewer system and still rely on on-site sewer systems. This equates to approximately one percent of the City population, which is negligible in terms of planning. Therefore, this Plan assumes that the full City population is currently served by the sewer system and any growth within city limits will also receive sewer service. Outside the city limits, but within the City's urban growth boundary, all of the existing residences and businesses are served by on-site sewer systems.

6.3 Gravity Sewers

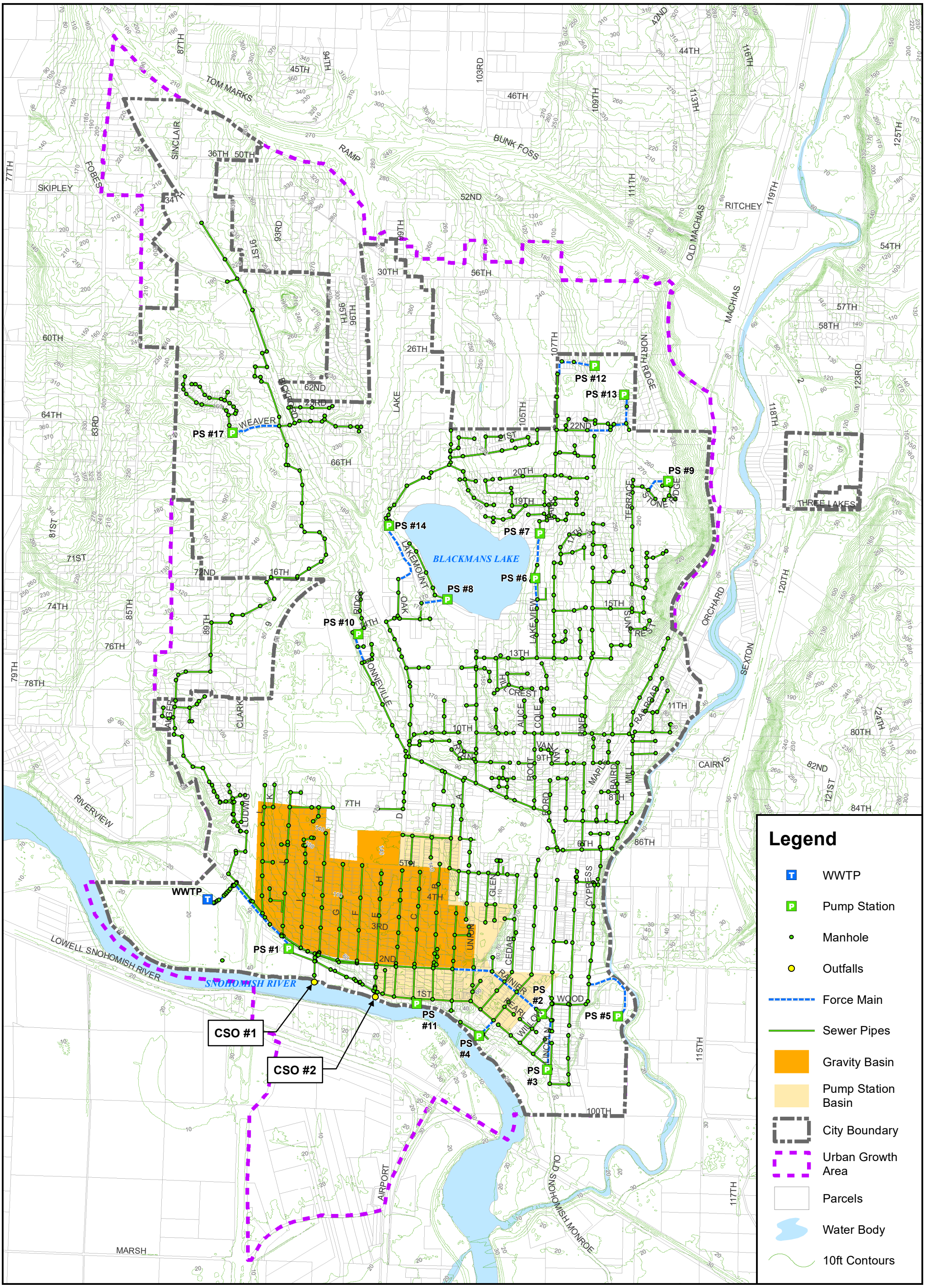
The City's gravity sewer system can be divided into two categories, the CSS and the separated collection system.

6.3.1 Separated Sewers

The separated (sanitary sewer only) portion of the City's sewer collection system lies within the current city limits and outside of the combined-sewer area. It consists of approximately 140,000 linear feet of PVC and concrete sewer pipe ranging in diameter from 6 to 18 inches.

The west side of the separated system flows by gravity to the WWTP. Flow from this area is conveyed through the west trunk (i.e., Cemetery Creek Trunk) that is intended to also serve the area north of Blackman's Lake and east of State Route 9, as well as the UGA west of State Route 9. The rest of the separated system either flows by gravity or is pumped by one of the 10 pump stations in the separated system to the Rainier Pump Station (Pump Station No. 2). From the Rainier Pump Station, the flow is pumped into the Second Street Trunk to the WWTP. The west trunk and the Second Street Trunk combine at the top of the driveway to the WWTP on 2nd Street and flow to the plant by gravity through a 30-inch PVC pipe. Most of the separated system pipes were installed prior to 1950 except for the west trunk that was installed around 2011.

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Sewer System & Basins: City of Snohomish.
GIS Base data: Snohomish County.
Data sources supplied may not reflect current or actual conditions. This map is a geographic representation based on information available. It does not represent survey data. No warranty is made concerning the accuracy, currency, or completeness of data depicted on this map.
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City of Snohomish Washington

Existing Wastewater Collection System

General Sewer Plan and
Wastewater Facilities Plan
City of Snohomish, Washington
February 2019

Figure
6-1

P:\Mapping\Maps_Generated\Snohomish_City\19-10615.00\003\maps\Fig 6-1 Existing Wastewater Collection System - 11x17.mxd 2/19/2019 ctolentino

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6.3.1 Combined Sewers

The downtown core area is served by a combined sanitary and storm sewer drainage system. This portion of the collection system is over 100 years old. There is currently approximately 46,500 lineal feet of combined pipe ranging in diameter from 6 to 30 inches. Flow from the western portion of the CSS reaches the WWTP by gravity flow through the Second Street Trunk. Flow from the eastern portion of the CSS either flows by gravity or is pumped to the CSO Pump Station (Pump Station No. 1). The CSO Pump Station pumps directly to the WWTP.

The City is currently working to separate storm and sanitary sewers in the western portion of the CSS (tributary to CSO 1) by completing installation of a stormwater main that will discharge into a stormwater utility lagoon near the WWTP and infiltrate on-site. A new stormwater pipe has already been installed along Avenue F and is awaiting extension of the stormwater main to the stormwater utility lagoon for final separation, to be completed in 2018.

The CSS still has two outfall (overflow) lines that discharge directly into the Snohomish River when the sewer system's capacity is exceeded. These overflow locations are labeled on Figure 6-1. CSO #1 is connected to the gravity CSS and CSO #2 is connected to the pumped CSS and Rainier PS (PS No. 2) prior to CSO Pump Station (PS No. 1). From 2012 through 2018 there have been four CSOs at CSO #1 (8/11/2018, 5/5/2017, 9/5/2013 and 10/16/2012) and six CSOs at CSO #2 (8/11/2018, 5/5/2017, 9/2/2016, 9/3/2014, 9/5/2013, and 10/16/2012). Per the NPDES permit, the City is allowed up to one CSO per year per outfall on a 5-year average. The City has been in compliance with the permit with a 5-year average of 0.57 CSOs per year at CSO #1 and 0.86 CSOs per year at CSO #2. In 2015, the City implemented some of the operational changes at the CSO Pump Station that were recommended in the *Combined Sewer Overflow Management Plan* (2014). These changes are also discussed in detail in the *Combined Sewer Overflow Post-Construction Monitoring Plan* (2015).

6.4 Pump Stations and Force Mains

Because of the City's varied topography, numerous pump stations are required to convey sewage to the WWTP. The 15 wastewater pump stations presently operated and maintained by the City are shown in Figure 6-1 and summarized in Table 6-1. Pump Stations No. 1 (CSO Pump Station), No. 4 (Commercial), and No. 11 (Kla-Ha-Ya) serve the downtown CSS. There are three types of pump stations: submersible, wet well/dry well, and suction lift (Gorman-Rupp). All have been constructed since 1957 and most have undergone major modifications and renovations over the years. Most recently, in 2015, all pump stations received telemetry upgrades. There are 2.5 miles of force mains (ranging from 3-inch to 18-inch) constructed of asbestos cement, cast-iron or PVC pipe.

6.4.1 Standby Power

Currently, nine of the 15 pump stations have on-site emergency generators. These pump stations are listed in Table 6-1.

Table 6-1 Pump Station Summary

Pump Station No.	Pump Station Name	Year Online	Number of Pumps	Pump Manufacturer	Pump Capacity (GPM)	Force Main Material	Force Main Diameter (inch)	TDH (ft)	Electrical Equipment Updated	Drainage Service Area	Generator
PS 1	CSO	2010	4	Gorman Rupp	3,500	PVC	18	37	2010	Combined	Yes
PS 2	Rainier	1987	3	Cornell	1,500	AC	18	50	2014/2015	Separate*	No
PS 3	Lincoln	1958, upgraded 1998	2	Gorman Rupp	280	AC	6	37	2010	Separate*	Yes
PS 4	Commercial	1997	2	Gorman Rupp	150	Steel	4	44	2010	Combined	Yes
PS 5	Pilchuck	1995	2	Gorman Rupp	75	PVC	3	60	2014/2016	Separate*	No
PS 6	Hill Park	1996	2	Gorman Rupp	225	PVC	8	35	2008	Separate	Yes
PS 7	Champagne	1997	2	Gorman Rupp	240	AC	4	58	2008	Separate	Yes
PS 8	Ferguson Park	1977	2	Gardner Triangle	75	PVC	4	34	2014/2015	Separate	Yes
PS 9	Stone Ridge	1992	2	Gorman Rupp	125	PVC	4	47	2014/2015	Separate	No
PS 10	Bonneville	1994	2	Gorman Rupp	140	PVC	4	48	2014/2015	Separate	No

Table 6-1 Pump Station Summary

Pump Station No.	Pump Station Name	Year Online	Number of Pumps	Pump Manufacturer	Pump Capacity (GPM)	Force Main Material	Force Main Diameter (inch)	TDH (ft)	Electrical Equipment Updated	Drainage Service Area	Generator
PS 11	Kla-Ha-Ya	1995	1	Zoeller	50	PVC	4	35	Unknown	Combined	No
PS 12	Rainbow	1997	2	Gorman Rupp	120	PVC	4	62	2014/2015	Separate	No
PS 13	Clark's Pond	2007	2	Gorman Rupp	200	PVC	4	61	2008	Separate	Yes
PS 14	Casino	1998	2	Gorman Rupp	160	PVC	4	60	2014/2015	Separate	Yes
PS 17	Shadowood	2013	2	Gorman Rupp	266	Ductile	6	40.6	2013	Separate	Yes

Note:
* Though unconfirmed, it is believed there are some stormwater inflows to these pump stations

6.4.2 Overflows

When the original pump stations were built, many of them had overflow pipes constructed in their wet wells. When water levels in the wet wells rose too high, sewage would flow through an overflow pipe and discharge into a nearby surface water. All of these overflows, with the exception of the two combined-sewer overflows at 2nd Street and Avenue H (CSO #1) and at 1st Street and Avenue E (CSO #2), have been plugged.

6.4.3 Telemetry

Pump Stations

At each of the pump stations, except Pump Station No. 11 (Kla-Ha-Ya), a radio wave telemetry system is used for transmitting a status signal from the pump station to the application server (APPSVR02) located at the CSO pump station. Communication fails, high-level, low-level, wet well not changing, pump not running status and other critical alarms are transmitted to APPSVR02. A workstation (WWCS workstation) has been installed in the Wastewater Collections Department office and all pump stations, except Kla-Ha-Ya, can be monitored and controlled from the WWCS workstation. Alarms can also be viewed and acknowledged from the WWCS workstation (see Figure 6-2 for example screen shot of the Rainier Pump Station) and are retained in the APPSVR02 historical database. Remote access to the telemetry system is also available with the use of an iPad or iPhone giving the operators the same system access while in the field. If telemetry fails, there is an autodialer at the CSO Pump Station that will notify staff the telemetry system is down. Telemetry features are summarized in Table 6-2.

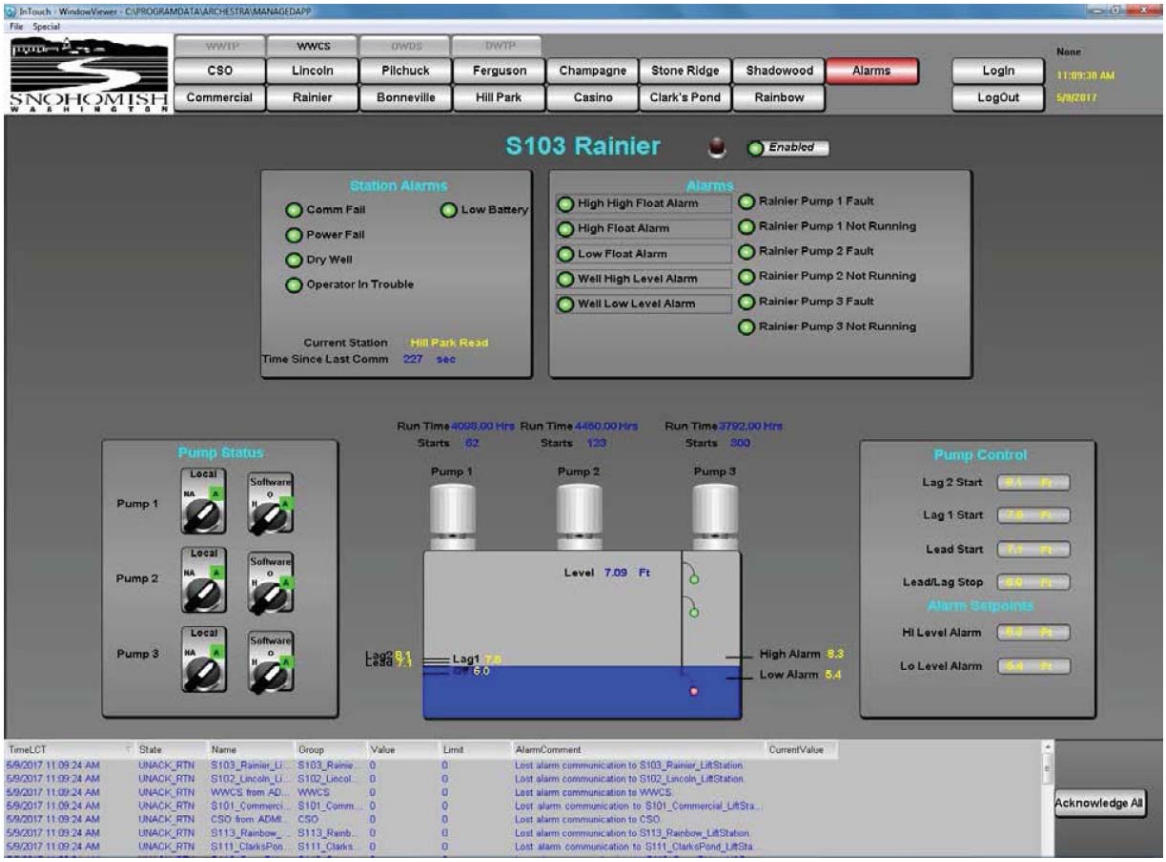


Figure 6-2 APPSVR02 Sample Screen Shot – Rainier Pump Station

Table 6-2 Telemetry Summary

Alarms and Sensors	No. 1 CSO	No. 2 Rainier	No. 3 Lincoln	No. 4 Commercial	No. 5 Pilchuck	No. 6 Hill Park	No. 7 Champagne	No. 8 Ferguson Park	No. 9 Stone Ridge	No. 10 Bonneville	No. 11 Kla-Ha-Ya	No. 12 Rainbow	No. 13 Clark's Pond	No. 14 Casino	No. 17 Shadowood
High Level	E	E	E	E	E	E	E	E	E	E	–	E	E	E	E
Low Level	E	E	E	E	E	E	E	E	E	E	–	E	E	E	E
Pump No. 1 Run	E	E	E	E	E	E	E	E	E	E	–	E	E	E	E
Pump No. 1 Fail	E	E	E	E	E	E	E	E	E	E	–	E	E	E	E
Pump No. 2 Run	E	E	E	E	E	E	E	E	E	E	N/A	E	E	E	E
Pump No. 2 Fail	E	E	E	E	E	E	E	E	E	E	N/A	E	E	E	E
Power Failure	E	E	E	E	E	E	E	E	E	E	–	E	E	E	E
Water in Drywell	–	–	N/A	N/A	N/A	N/A	N/A	–	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Operator in Trouble	–	E	–	–	–	–	–	–	–	–	–	–	–	–	–
WW Level	E	E	E	E	E	E	E	E	E	E	–	E	E	E	E
Legend: E = Existing and Functional; – = None; N/A = Not Applicable for that Station Type															

Wastewater Treatment Plant

Alarms for the WWTP are sent from the WWTP Application Server (APPSVR01) located at the WWTP to APPSVR02 through a fiber optic cable connection between the two locations. A workstation (WWTPLAB) has been installed in the WWTP office and a second in the Chlorine building (WWTPCHEM) located at the south east corner of the WWTP. The WWTP system operations can be monitored and controlled from either the WWTPLAB or the WWTPCHEM. Alarms can also be viewed and acknowledged from these two locations and are retained in the APPSVR01 historical database. Remote access to the telemetry system is also available with the use of an iPad or iPhone giving the operators the same system access while in the field. Figure 6-3 shows an example screen shot of the APPSVR01 workstation interface for the WWTP.

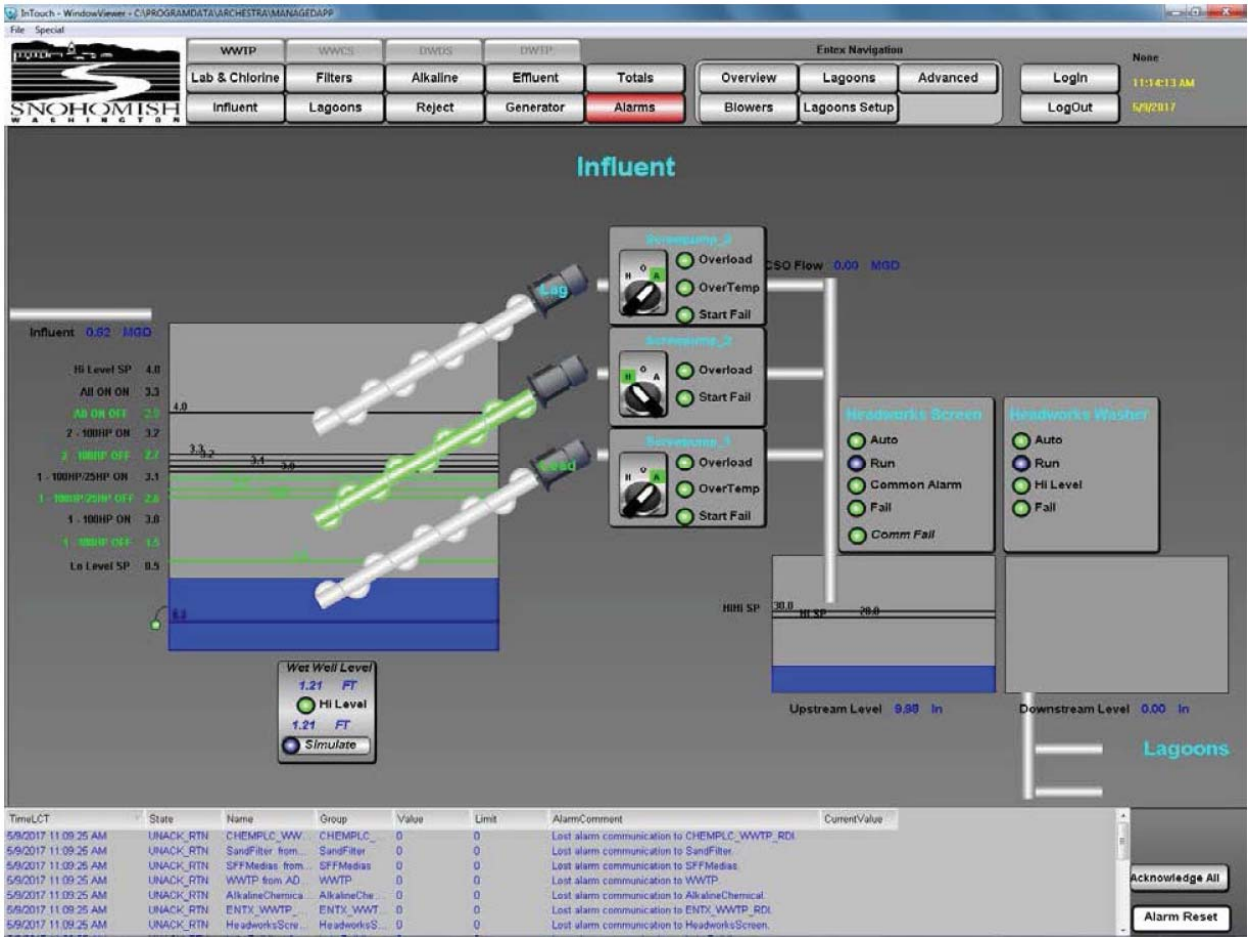


Figure 6-3 APPSVR01 Sample Screen Shot – WWTP Influent

6.4.4 Pump Station No. 1, CSO Pump Station

Pump Station No. 1 is located off of 1st Street, approximately 200 feet southeast of the intersection of 1st Street and 2nd Street (Figure 6-4). The station was built on vacant land owned by the City and can be accessed from 1st Street.



Figure 6-4 CSO Pump Station Location

The CSO Pump Station began operation in September 2011. It was built to replace the Ironworks Pump Station and provide the improvements necessary to reduce the number of combined sewer overflows (CSOs). Prior to startup of the station, millions of gallons of combined sewage overflowed into the Snohomish River every year through the two CSO outfalls (CSO #1 and CSO #2). Both CSO #1 and CSO #2 outfalls still exist. Control structures are in place to convey combined sewage to the CSO Pump Station. Only when flow of combined sewage reaches extreme levels does a portion of the flow get diverted to one or both of the CSO outfalls. Since start-up of the CSO Pump Station, combined sewage has on average been diverted to the CSO outfalls less than once per year.

A layout of the CSO Pump Station is shown in Figure 6-5. The CSO Pump Station includes a 12-foot diameter and approximately 18.5-foot deep wet well; four above-grade, self-priming, centrifugal pumps; interconnecting piping between the pumps and wet well; an 18-inch discharge force main; a tributary 30-inch gravity pipeline approximately 1,750 feet long; a control building; and a standby generator. Each pump is connected to a variable frequency drive (VFD). The VFDs can be used to vary pump output with wet well level, but are normally used to allow soft start of the pumps, after which the pumps run at a set speed. There is a second wet well of equal size with space planned for two above-grade, self-priming, centrifugal pumps that will be used in the future to pump storm water, once the tributary combined sewers are separated.

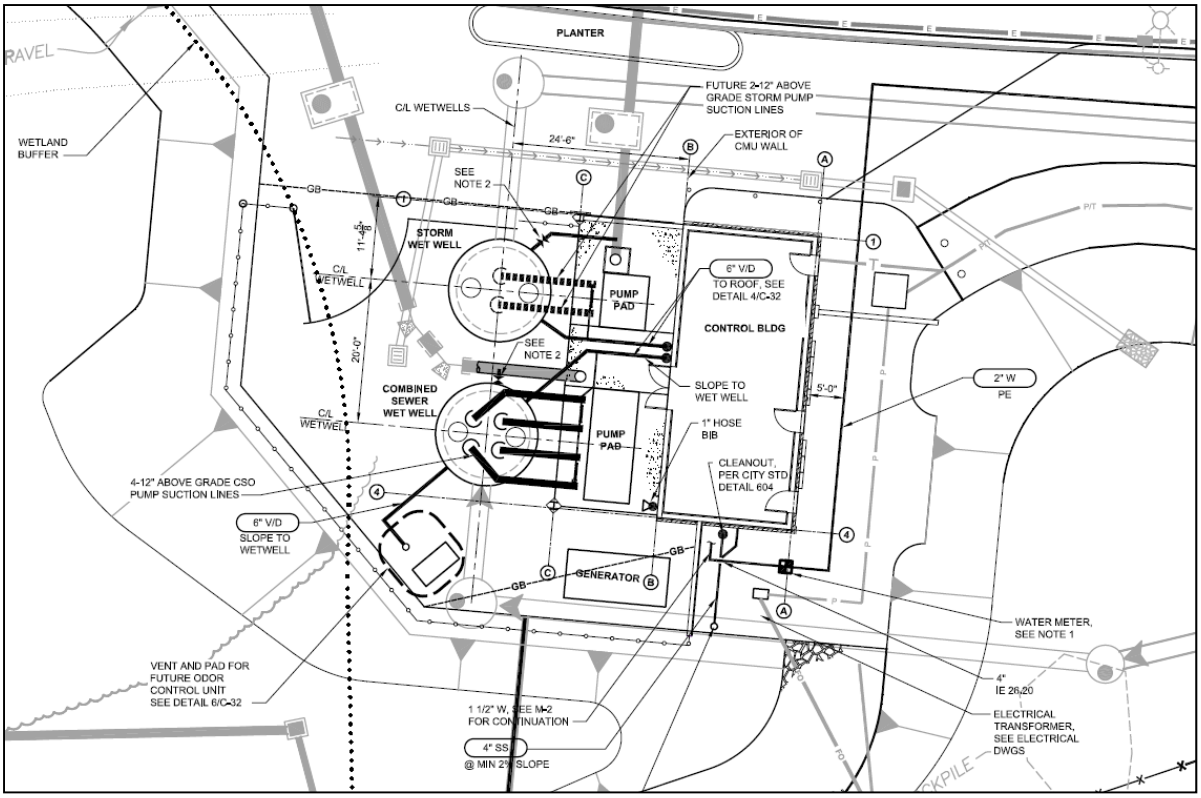


Figure 6-5 CSO Pump Station Layout

The CSO Pump Station is operated based on level in the wet well. The four centrifugal pumps each have a capacity of about 3,500 gpm at 37 feet of total dynamic head, which provides a firm capacity of about 7,200 gpm with three of four pumps running. The four pumps are designated as lead, lag, lag-lag and standby. These designations are automatically rotated to distribute runtime of the pumps. When the level in the wet well reaches the Lead Pump Start elevation, the lead pump is started. When the lead pump is running at or above the Lag Pump Start elevation for more than 30 seconds, the lag pump will be started. If the lead and lag pumps are running at or above the Lag-Lag Pump Start elevation for more than 30 seconds, the lag-lag pump will be started. If the level in the wet well reaches the High Level, then all four pumps are called to run via a hardwired interlock with a high-level float switch and will remain running until the level drops to the Low Level and all pumps are called to stop via a hardwired interlock with a low-level float switch.

6.4.5 Pump Station No. 2, Rainier

The Rainier Pump Station is a wet well/dry well facility located at the intersection of Rainier Avenue and Willow Street. A diagram of the Rainier Pump Station and adjacent and connecting piping is shown in Figure 6-6. This pump station serves the majority of the City's separated system. The original Rainier Pump Station was constructed in 1959 and it remained in continuous service until approximately March 1988, when completion of extensive pump station modifications occurred. These included the following:

- Replacement of all interior piping, valves, pumps, and motors
- Replacement of automatic controls, electric service and wiring, motor control center and lighting, and addition of accessories for use of new portable emergency generator
- Replacement of ladders and grating

- Addition of dry well ventilation and dehumidifier
- Sealing the existing wet well
- Installation of a clean-out valve on the existing 16-inch diameter force main and providing new bypass piping to allow future dewatering of the existing force main.

The Rainier Pump Station was originally equipped with two pumps, each capable of pumping 1,500 gpm against a total dynamic head of 50 feet. In March 2004, City staff conducted drawdown tests to determine the performance of the pumps and concluded that the pumps were pumping about half the design pumping rate. As a result, the City conducted a rebuild of the two existing pumps and added a third pump. Additionally, a dry well blower has been provided that, when placed on automatic operation, will run whenever the access hatch door is open or whenever the inside dry well temperature exceeds the value set on the heat sensing thermostat.

6.4.6 Pump Station No. 3, Lincoln

This pump station is located in the southeast corner of the City and serves a primarily residential area in the separated system. It is a self-contained wet well type station containing two 6.2-horsepower submersible Gorman Rupp pumps, each capable of pumping at 280 gpm. The previous Flygt pumps were replaced in 1998 with Gorman Rupp pumps of a similar design. The pumps can be removed for service by means of a guide rail and chain.

The wet well is ventilated by a draft tube to atmosphere and the pumps are controlled by a float-type level switch. The lead and lag pumps are automatically alternated at each start. All electrical controls and switches are located in weatherproof panels outside the station.

This pump station has experienced significant grease problems. City staff has added proprietary cultures of grease-eating bacteria to the wet well to alleviate the grease buildup. Past observations have reported all valves in the valve box being submerged under several feet of water, with the exception of the tall air-release valve.

6.4.7 Pump Station No. 4, Commercial (a.k.a. Cady Park)

This pump station is located at the south end of the City in Cady Park along the Snohomish River and serves a small portion of the City's CSS area. The station was last upgraded in 1998 and is a suction lift type, Gorman-Rupp, 6-foot by 6-foot, above-ground, fiberglass, wet well mount, package-type station. It contains two pumps with 5-horsepower motors and a capacity of 150 gpm each. The pumps and motors are housed above ground, directly above the wet well. This pump station has had issues with root intrusion and clogged check valves.

6.4.8 Pump Station No. 5, Pilchuck

This pump station is located in the southeast corner of the City in Pilchuck Park and serves two bathrooms at the park, as well as the floor drain to the park's storage room, which also collects surface runoff from the parking lot. It is a suction lift type, Gorman-Rupp, 6-foot by 6-foot, above-ground, fiberglass, wet well mount, package-type station that was upgraded in 1995. The station contains two 7.5-horsepower motors with a capacity of 75 gpm each. The pumps and motors are housed above ground, directly above the wet well.

The 3-inch diameter force main exiting the pump station does not meet Ecology size guidelines (diameter) for handling solids; however, the relatively low 3.4 foot-per-second velocity indicates that the force main pipe size is appropriate to promote scouring and prevent settling of solids.

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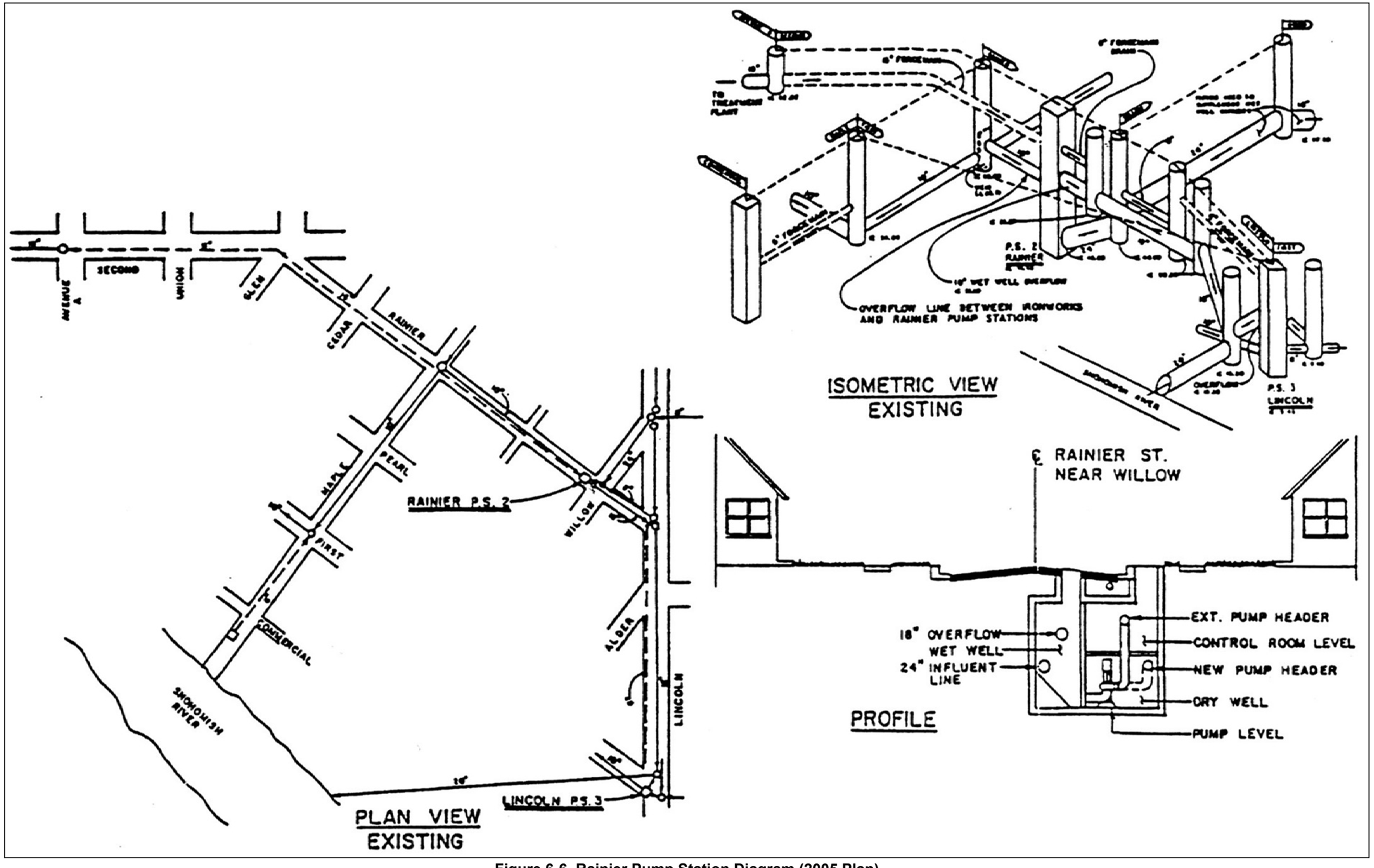


Figure 6-6 Rainier Pump Station Diagram (2005 Plan)

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6.4.9 Pump Station No. 6, Hill Park

This pump station is located on the east edge of Blackmans Lake in Hill Park and serves approximately six houses, as well as the park restrooms. The pump station is a suction lift type, Gorman-Rupp, 6-foot by 6-foot, above-ground, fiberglass, wet well mount, package-type station that was upgraded in 1997. The station contains two 5-horsepower motors with a capacity of 225 gpm each. The pumps and motors are housed above ground, directly above the wet well.

6.4.10 Pump Station No. 7, Champagne Lane

This pump station is located at the northeast edge of Blackmans Lake and serves a small number of residences in the Champagne Lane development. The pump station is a suction lift type, Gorman-Rupp, 6-foot by 6-foot, above-ground, fiberglass, wet well mount, package-type station that was upgraded in 1997. The station contains two 10-horsepower motors with a capacity of 240 gpm each. The pumps and motors are housed above ground, directly above the wet well. The force main exiting from this station is a very old asbestos cement pipe in poor condition, located in a very wet marshy area.

6.4.11 Pump Station No. 8, Ferguson Park

This pump station is located on the southwest edge of Blackmans Lake in Ferguson Park. It is a wet well mount, package-type station, consisting of two pumps with 3-horsepower motors, each with a capacity of 75 gpm. The pumps and motors are housed above ground directly above the wet well.

6.4.12 Pump Station No. 9, Stoneridge

This pump station is located in the northeast corner of the City and serves the Stoneridge neighborhood. It is a suction lift type, Gorman-Rupp, 6-foot by 6-foot, above-ground, fiberglass, wet well mount, package-type station that was constructed in 1993. The station contains two 7.5-horsepower motors with a capacity of 125 gpm each. The pumps and motors are housed above ground, directly above the wet well.

6.4.13 Pump Station No. 10, Bonneville (a.k.a. 14th Street)

The Bonneville Pump Station is located at the intersection of Bonneville Avenue and 14th Street. This suction lift type, Gorman-Rupp, 6-foot by 6-foot, above-ground, fiberglass, wet well mount, package-type station was constructed in 1993. The station contains two pumps with 7.5-horsepower motors and a capacity of 140 gpm each. The pumps and motor are housed above ground, directly above the wet well.

6.4.14 Pump Station No. 11, Kla-Ha-Ya

The Kla-Ha-Ya Pump Station is located in the south end of the City on the shores of the Snohomish River in Kla-Ha-Ya Park. The pump station is subject to flooding when the Snohomish River rises to an elevation of 25 feet. City staff turn off the high alarm and keep the pumps running in automatic mode when the wet well is below flood waters. It currently serves four commercial properties and a couple of storm drains. In August 1995, it was discovered that these four commercial properties had been connected into a storm drain that drained directly into the Snohomish River. To correct this violation as quickly as possible, the City installed a temporary pump station. The original station contained a 1,000-gallon storage tank and one 2-horsepower submersible pump with a capacity of 50 gpm.

In the first quarter of 1996, the City improved the pump station. It was determined that the existing pump had the needed capacity to handle the current inflow rate, so the pump was reused. However, the single pump does not meet reliability requirements. Since this site was subject to flooding, a watertight concrete wet well was installed, but the station still experiences high infiltration and inflow. This pump station has no emergency generator hookup or telemetry system, but does have an audio and visual alarm on site. Additionally, the galvanized corrugated metal pipe conveying wastewater to the station is corroded.

6.4.15 Pump Station No. 12, Rainbow

The Rainbow Pump Station is located in the north end of the City and serves the homes located along Rainbow Place. This pump station is a suction lift type, Gorman-Rupp, 6-foot by 6-foot, above-ground, fiberglass, wet well mount, package-type station that was constructed in 1997. High wetwell levels are indicated with a red flashing light, but there is no audible alarm. The station contains two pumps with 7.5-horsepower motors and a capacity of 120 gpm each. The pumps and motor are housed above ground, directly above the wet well.

This pump station has at times experienced significant grease problems. City staff has added proprietary cultures of grease-eating bacteria to the wet well to alleviate grease buildup as needed.

6.4.16 Pump Station No. 13, Eden Farms

The Eden Farms Pump Station was taken off line February 1, 2007, after construction of the existing west trunk (i.e., Cemetery Creek Trunk) was completed. The station was located near the intersection of State Route 9 and 72nd Street SE and served the Snohomish Business Park.

6.4.17 Pump Station No. 13, Clark's Pond

The Clark's Pond Pump Station is located at the end of Baird Avenue near Clark's Pond and serves approximately 30 single family residences. The pump station is a suction lift type, Gorman-Rupp, package type pump station that was constructed in 2007. The station contains two pumps with 10-horsepower motors and a capacity of 200 gpm each. The pumps and motors are housed above ground, directly above the wet well.

6.4.18 Pump Station No. 14, Casino (a.k.a. Lakemount Drive)

The Casino Pump Station is located on Lakemount Drive, just east of Avenue D and serves the Casino Royale neighborhood. It is a suction lift type, Gorman-Rupp, 6-foot by 6-foot, above-ground, fiberglass, wet well mount, package-type station that was constructed in 1998. The station contains two pumps with 7.5-horsepower motors and a capacity of 160 gpm each. The pumps and motor are housed above ground, directly above the wet well.

This pump station has at times experienced significant grease problems. City staff has added proprietary cultures of grease-eating bacteria to the wet well to alleviate grease buildup as needed. Several manholes directly upstream of this pump station are not sealed, and during wet weather, significant infiltration occurs.

6.4.19 Pump Station No. 17, Shadowood

The Shadowood Pump Station is located at the intersection of Cady Drive and Weaver Road. This pump station was built in anticipation of future housing developments; much of the surrounding area is currently undeveloped. The pump station is a suction lift type, Gorman-Rupp, package type pump station that was constructed in 2013. The station contains two pumps with 10-horsepower motors and a capacity of 266 gpm each. The pumps and motor are housed above ground, directly above the wet well.

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Chapter 7 Collection System Analysis

7.1 Introduction

Analysis of the City's wastewater conveyance system is a critical component in determining the ability of the existing infrastructure to accommodate future growth. The City's conveyance system was analyzed using a truncated model. Only the main trunks in the combined sewer system (CSS) were modeled since that portion of the collection system had been recently modeled in EPA SWMM 5 for the CSO Management Plan with no capacity constraints found. Most of the rest of the collection system was modeled except for some small branches of pipe for which pipe and manhole data was not available. The I/I and base flows entering the omitted pipes were added to the nearest modeled downstream node. Small pump stations with less than 8 acres of contributing area were excluded from the model. The Pilchuck Pump Station (PS 5) was also excluded since the 15 acres of contributing area is a park with minimal flow contribution. Table 7-1 lists all of the pump stations and their contributing area.

Table 7-1 Pump Station Contributing Area

PS Number	PS Name	Contributing Area (ac)	Included in Model
PS 1	CSO	142.5	✓
PS 2	Rainier	740.9	✓
PS 3	Lincoln	36.9	✓
PS 4	Commercial	4	
PS 5[(a)]	Pilchuck	15	
PS 6	Hill Park	10.9	✓
PS 7	Champagne	68.5	✓
PS 8	Ferguson Park	8.7	✓
PS 9	Stone Ridge	6.3	
PS 10	Bonneville	8.8	✓
PS 11	Kla-Ha-Ya	1.5	
PS 12	Rainbow	4.4	
PS 13	Clark's Pond	7.3	
PS 14	Casino	44.3	✓
PS 17	Shadowood	7.9	

Note:
a) The contributing area is a park with minimal flow contribution

The truncated model allows for an accurate representation of the most critical components and the majority of the City's conveyance system, and the simultaneous analysis of both gravity and pressure systems. The system was analyzed for existing conditions (2016), 2022 conditions, two different conditions for 2036 (2036 population without system expansion outside the City limits and 2036 population with expansion into the UGA and a portion of the North Planning Area south of State Route 2), and the theoretical buildout population with expansion into the UGA and a portion of the North Planning Area south of State Route 2.

7.2 Model Software

XPSWMM 2016.01 by XP Solutions was the hydraulic modeling software used to model the City's collection system. XPSWMM 2016.01 is a dynamic hydraulic model that combines 1D calculations for upstream to downstream flow with 2D overland flow calculations. The model is designed for modeling various scenarios including urban sanitary and combined sewer systems. The model can import data from ArcGIS (ArcMap) to assist in building the model.

7.3 Model Development

ArcGIS files were imported into the model with the rim elevations, invert elevations, and pipe diameters that were included. Where available, holes in the data were filled using as-built drawings, field surveys, topographic maps, and the City's Wastewater Collection System Map from (Figure 6-1). Missing invert elevations of the remaining manholes were linearly interpolated between known elevations.

The City's conveyance system was analyzed using a truncated model, simulating most of the pipes in the separated sewer system area, inputs for each street in the CSS area, and the larger pump stations as listed in Table 7-1. The model can be expanded in the future, as needed.

The following information was used in developing the hydraulic model of the existing sewer collection system. Additional details on the existing sewer system are included in Chapter 5.

7.3.1 Gravity Sewers

Gravity sewer rim and invert elevations were obtained primarily through a GIS database kept by the City. As-built drawings, field surveys, the City's Wastewater Collection System Map (Figure 6-1) and topographical maps were used to fill in missing or inconsistent data.

7.3.2 Pump Stations

Pump station wet well dimensions, invert levels, set points and pump data were mostly obtained from Pump Station Data Sheets provided by the City with the information for the CSO Pump Station coming from as-built drawings. Wet well rim elevations were found using as-built drawings, the City's Wastewater Collection System Map (Figure 6-1) and topographical maps. The design flow and dynamic head was used to model most of the pump stations as design points. The CSO Pump Station was modeled using data from recent drawdown pump tests.

7.4 Model Loading

Meaningful modeling results can only be obtained if the quantity of flows and the location where they enter the system in the model reflect actual conditions. Wastewater flow consists of two separate elements: sanitary sewer flow and infiltration and inflow (I/I). Sanitary sewer flow is typically referred to as Dry Weather Flow (DWF) in the model (actual DWF in the collection system usually includes a minor amount of base I/I that is accounted for in the modeled I/I). I/I is simulated in the model using the RTK method in a rainfall dependent inflow and infiltration (RDII) module. All flow is loaded to model "nodes", which are typically manholes along gravity pipelines. The CSS area had the additional component of overland stormwater flow entering the modeled system.

7.4.1 Sanitary Sewer Flows

Existing and projected sanitary sewer flow rates were developed for each sub-basin using the following information:

- Population and employment data and projections (described in Chapter 3)
- Existing measured flow rates (described in Chapter 5)
- Diurnal curves (described in Chapter 5)

Model loading was assigned as the calculated base flow rate in the sub-basin, based on residential and employment populations for the sub-basin and associated per capita flow rates, divided evenly between each of the modeled nodes in the sub-basin.

7.4.2 Combined Sewer Flows

Sub-basins for the CSS area had an additional model component of overland stormwater flow into the system. Data from a previous EPA SWMM 5 model of the combined system was used for calculating the contributing land area and overland flow variables. The amount of contributing area was reduced to account for the higher than previously expected I/I from the upstream separated sewer system. The areas were calibrated such that during the 5/5/2017, 9/5/2013 and 10/16/2012 storms, during which CSOs were recorded, the model showed a CSO occurring and that during the 11/14/2015 storm, during which no CSO was recorded, the model did not show a CSO. For the 5/5/2017 storm event, the model correctly predicted an overflow at CSO 2, but did not predict an overflow at CSO 1. However, the reported overflow at CSO 1 was only 50 gallons. After the modeling was completed there was another CSO on 8/11/2018. The storm's characteristics of high intensity and short duration align with the other storms that caused CSOs.

7.4.3 Infiltration and Inflow (I/I)

The RTK Method was used to model RDII in the model. This method uses three triangular hydrographs to estimate response times associated with fast inflow and the slower ground water infiltration (GWI). The three hydrographs have their own R, T and K values to simulate fast, medium and slow response times.

- R: the fraction of rainfall volume entering the sewer system as RDII
- T: the time to peak flow
- K: the ratio of time of recession to time of peak (T)

These three hydrographs are combined to form one overall system response hydrograph. Figure 7-1 shows an example of how the overall RDII hydrograph is compiled.

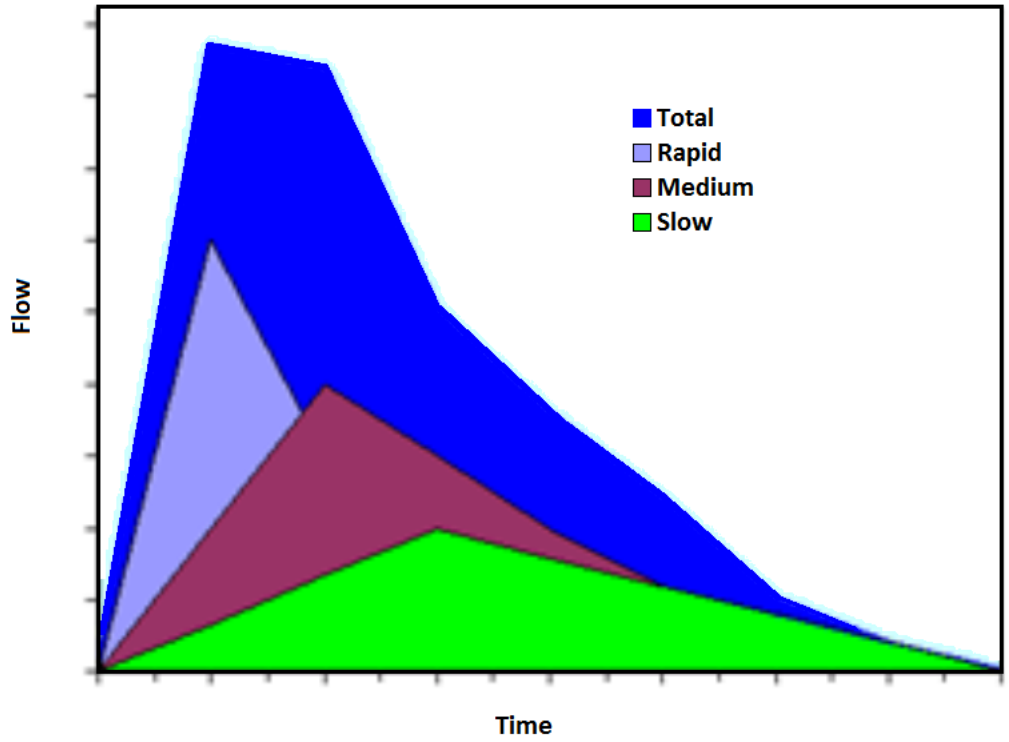


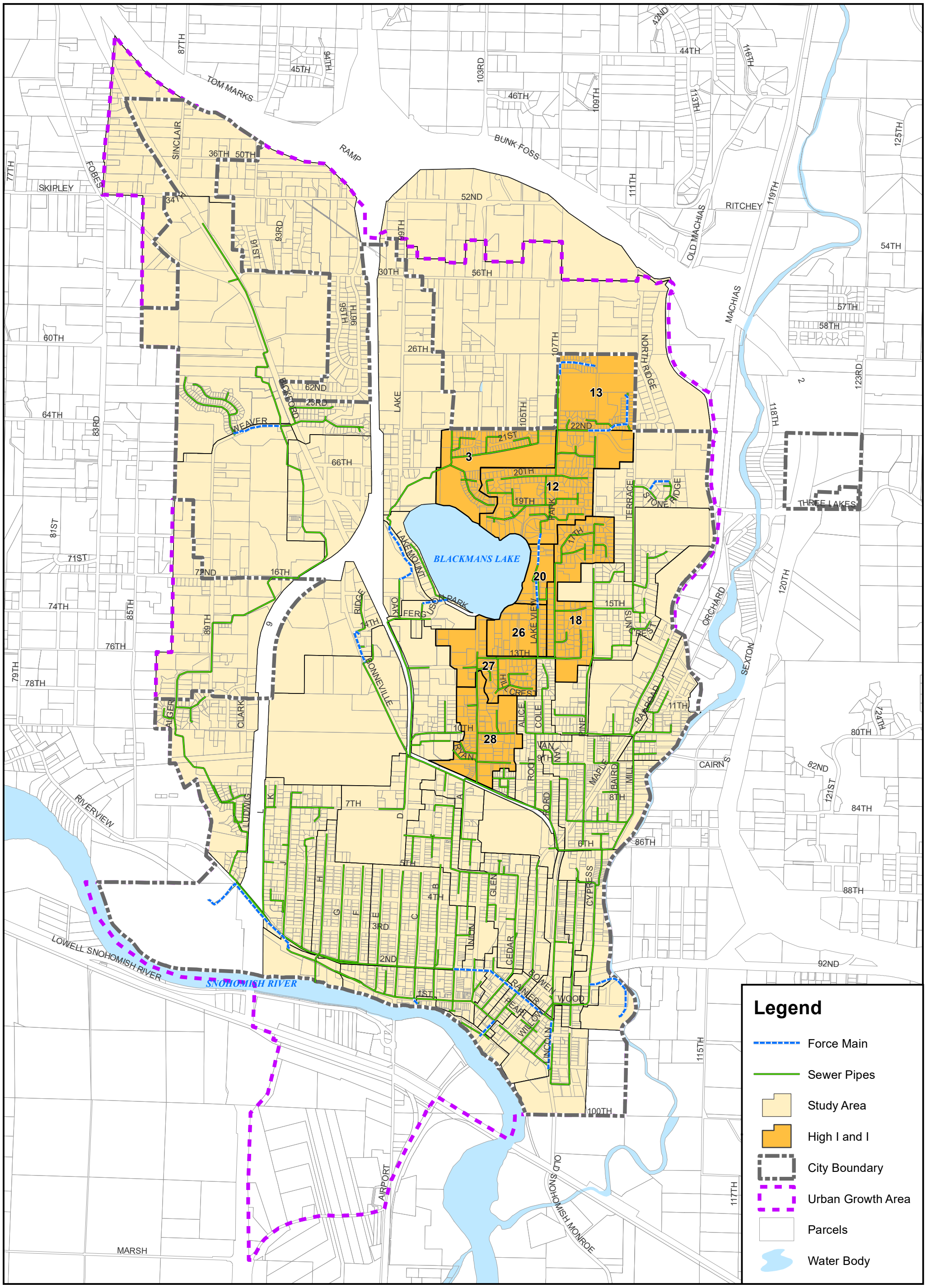
Figure 7-1 Example RTK Method Hydrographs for Modeling RDII

The RDII was calibrated based on the Avenue A flow meter. Two separate RTK hydrographs were compiled based on the City's working knowledge about the system. They have observed that the middle area of the City north of Avenue A and 8th Street (shown in Figure 7-2) appears to have approximately 20 percent more I/I than the rest of the system (not including overland stormwater flow into the CSS). This portion of the system has separate sewer and stormwater systems, but the flows in the sewer pipes during and after storms indicate that stormwater is being piped directly into the sewer system. For this reason, a second RDII hydrograph was compiled with a 20 percent lower peak flow and 20 percent less volume, which was applied to the rest of the system in the computer model.

Total I/I for each basin was calculated using the RDII hydrograph and the approximate area contributing to the sewer system. The contributing area was derived from the area of parcels and rights-of-way adjacent to existing sewers, and was adjusted based on engineering judgment and knowledge of the sewer system.

Model I/I loading was assigned to nodes based on the length of pipe between the target node and the upstream node. Pipes branching off the main lines that were not modeled were included in the total pipe length contributing to the I/I in that node.

Determining how I/I should be projected into the future as the collection system expands and ages is a key issue. Based on the King County Regional Infiltration/Inflow Control Program, a widely accepted assumption in Western Washington is to increase the I/I component of sewer flow by 7 percent per decade, up to a maximum of 28 percent. Much of the City's existing sewer collection system was built 70 years ago or more, and thus has likely reached the maximum I/I rate. Therefore, a constant I/I rate was assumed for current and future I/I projections. The City agreed with this assumption based on their knowledge and observations of the collection system.



Legend

- Force Main
- Sewer Pipes
- Study Area
- High I and I
- City Boundary
- Urban Growth Area
- Parcels
- Water Body

Sewer System & Basins: City of Snohomish.
GIS Base data: Snohomish County.
Data sources supplied may not reflect current or actual conditions. This map is a geographic representation based on information available. It does not represent survey data. No warranty is made concerning the accuracy, currency, or completeness of data depicted on this map.
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City of Snohomish Washington

Sub-Basins with High I/I

General Sewer Plan and
Wastewater Facilities Plan
City of Snohomish, Washington
February 2019

Figure
7-2

P:\Mapping\Maps_Generated\Snohomish_City\19-10615.00\003\maps\Fig 7-2 Sub-Basins with High I_I - 11x17.mxd 2/19/2019 ctolentino

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When sewers are separated in the CSS portion of the service area, the City plans to install both new storm and sanitary sewers. Therefore, it is assumed that the separated areas will not experience I/I higher than the majority of the rest of the service area.

7.5 Model Calibration

The model was calibrated using data from the following sources:

- Flow Meter at Avenue A and 8th Street (Avenue A flow meter) with:
  - Three months of wet weather data
  - One month of dry weather data
  - One large storm
- Flow meter at the Rainier Pump Station with:
  - Three months of wet weather data
  - One month of dry weather data
  - One large storm
- WWTP Discharge Monitoring Reports (DMRs)
- CSO Records

The Avenue A flow meter was used to calibrate the RDII. The Rainier Pump Station flow meter was used to determine the average area contributing to RDII per linear foot of pipe and verify calibration of the RDII variables. Both flow meters only caught one significant storm from which to calibrate the model. The DMRs were used to calibrate the flow coming from the west trunk and total daily flows. The per capita rates were calculated using the Avenue A flow meter, Rainier Pump Station flow meter and the DMRs. The CSO records were used to confirm the stormwater runoff contributing area in the CSS area.

Key factors examined were total volume of wastewater, peak flow, and rate of flow recession after the storm. Figure 7-3 shows the modeled flow data compared to the measured flow data at the Rainier Pump Station for the one large storm. The figure shows how closely the model flow followed the actual measured flow pattern. The total volume difference from 11/7/2015 to 11/18/2015 between the meter at the Rainier Pump Station and the model was less than 1%. The modeled peak of the storm was within 3% of the actual measured peak and the recession tracks well with the model results, as shown in Figure 7-3. The peak flow measured at the Avenue A flow meter was approximately 5.5% greater than what was modeled. At the WWTP, the volume difference is less than 6% with a peak daily volume during the storm having just a 4% difference between what was modeled and measured. These values indicate that the model was calibrated well to the storm and provides more confidence in the results for predicted future flows.

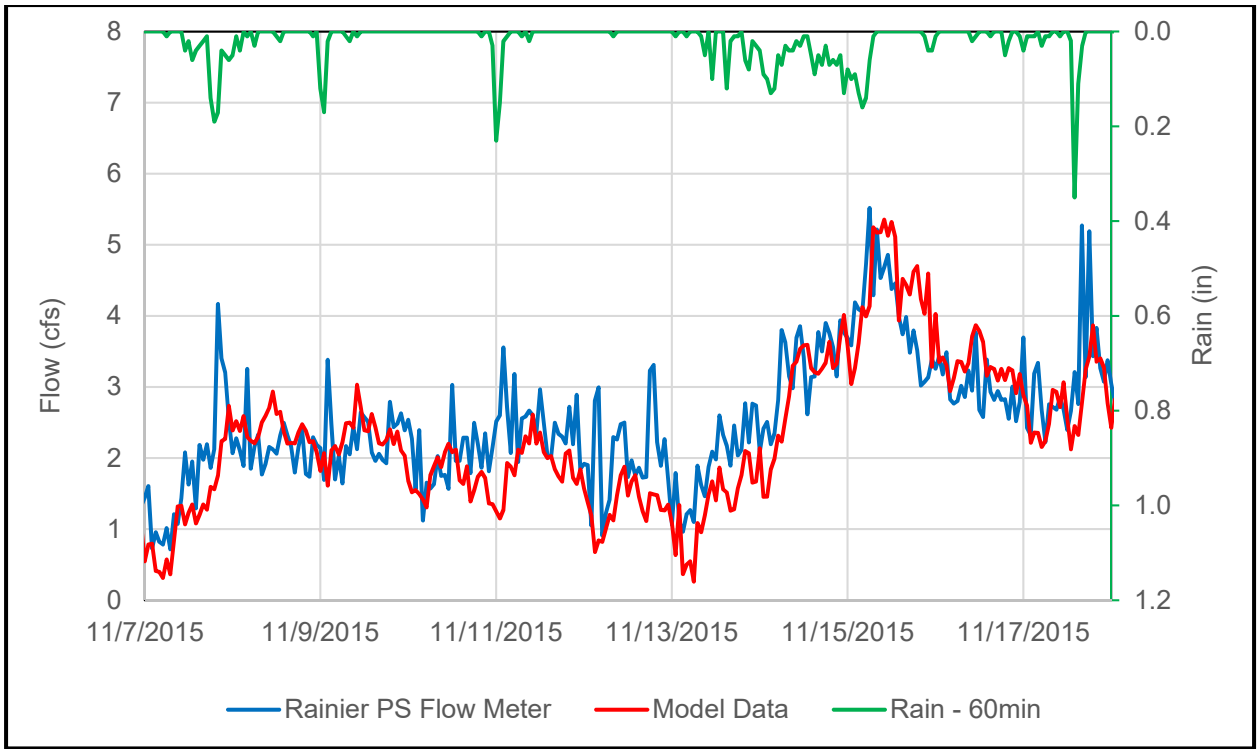


Figure 7-3 Flow At Rainier Pump Station

7.6 Future Collection System Expansion

Future collection system expansion was modeled by adding flow from all future population growth into the existing system model. Sewer extensions were not sized, but as the need arises, the model can be updated to ensure that new collection system components are constructed with adequate capacity for future growth.

7.7 Modeling Scenarios

Five main scenarios were developed to analyze the City's wastewater conveyance system. The scenarios listed in Table 7-2 incorporated planned changes to the system over time. Other scenarios were looked at for potential expansion of the collection system north of State Route 2, though that concept is not part of the current planning period. There are two 2036 scenarios: one excluding expansion of the sewer service area beyond the current City limits and the other including system expansion into the northern UGA and a portion of the North Planning Area south of State Route 2 (northern expansion). Though the City plans to eventually make this expansion into the north, the exact timeline has not been determined. The timing of the development will depend on when developers want to start and how long it will take for the system to be upgraded to accommodate the additional flow.

The City has considered two options for modifying the existing system to accommodate the northern expansion:

1. Divert flow from sub-basins 2, 3, 6, 100 and 105 to the West Trunk and decommission the Bonneville (PS 10) and Casino (PS 14) Pump Stations.
2. Upgrade the existing infrastructure to accommodate flow from sub-basins 2, 3, 6, 100 and 105.

The first option would add approximately 3,000 linear feet of 18-inch and 12-inch gravity pipe to route flow from these future developments to the West Trunk just west of State Route 9. The second option would upgrade the Casino Pump Station (PS 14) and replace approximately 1,700 feet of existing gravity pipe between Avenue A and Root Avenue along the interurban trail to increase capacity within the existing collection system. It is estimated that either option would have an opinion of probable project cost around $3 million.

Although the second option would have the benefit of replacing existing aging pipes and could possibly be phased in a manner that would allow the City to assign some of the upgrades to developers, there would likely be limited opportunities to phase the proposed improvements and such phasing would defer identified needs, which would not be favorable for O&M and reliability. The first option would likely need to be constructed in a single phase, but it also defers some future replacement and O&M costs by allowing both the Casino and Bonneville Pump Stations (PS 14 and 10) to be decommissioned. Additionally, the first option allows greater flexibility for accommodating buildout conditions. By routing flow to the West Trunk, the hydraulic model predicts that there will not be additional capacity deficiencies under buildout conditions, beyond those already planned to be addressed within the planning horizon. For these reasons, the City is planning to implement the first option of diverting flow to the West Trunk.

The Buildout scenario was modeled using this planned western diversion, which is favored by the City. Basins 101, 102 and 103, which are currently outside the City limits, are already west of State Route 9 and so would naturally be served by the West Trunk.

Table 7-2 Model Scenarios

Scenario	2016	2022	2036 (Current City Limits Only)	2036 (UGA and Portion North Planning Area)	Buildout
Population	2016	2022	2036 within current City limits	2036 with expansion to sub-basins 100, 101, 102, 103, 105, 106, and 107	Buildout including sub-basins 100, 101, 102, 103, 105, 106, and 107
Streets Disconnected from CSS	None	Avenue F	Avenue F	Avenue F	Avenue F
Collection System Changes	Existing as of 1/1/2016	Same as 2016	Same as 2016	Removal of PS 10 and 14 Re-direct flow from northern basins 2, 3, 6, 100 and 105 to West Trunk	Removal of PS 10 and 14 Re-direct flow from northern basins 2, 3, 6, 100 and 105 to West Trunk

The City has begun implementing part of the CSO 1 Separation Program (CIP C-10), with separation along Avenue F in 2018, and is in the initial planning phase for the CSO 2 Separation Program (CIP C-3). These programs are to remove stormwater connections to the CSS and re-direct the stormwater into a designated stormwater conveyance system. Removing stormwater from the CSS reduces the amount of water to be treated at the WWTP and further decreases the risk of having a CSO event. There is currently a gravity stormwater main in 2nd Street that was built in 2009 in preparation for the CSO 1 Separation Program, which includes separation along Avenues E, F, G, H, I, J and Riverview Lane within the CSS area. However, the City has recently placed a higher priority on the CSO 2 Separation Program, which includes separation along Avenues A, B, C and D within the CSS area, because CSO 2 is closer to the limit of averaging 1 CSO event per year over a 5-year period. Due to the uncertainty as to exactly what separations will occur in what order and when, conservatively only separation of Avenue F was included in the modeling. In general, these separations will be coordinated with other planned roadway and utility improvements along the same streets.

The models exploring sewer system expansion north of State Route 2 were used to test the system's capacity. The results show that additional flow, up to perhaps 1.5 MGD, could be routed through the West Trunk without significant conveyance system upgrades. However, a 12-inch diameter portion of the West trunk north of 16th Street would need to be upsized.

7.8 Hydraulic Modeling Analysis

7.8.1 Modeling Deficiency Criteria

The purpose of the modeling work is to determine deficiencies in the City's collection system. A 25-year, 24-hour storm was modeled to find if and where there are deficiencies in the system. Deficiencies in the collection system are defined as:

- Gravity Pipe Segments: Surcharging depth within five feet of the rim of the manhole, or equal to the pipe diameter (d/D of 1) for manholes less than 5 feet deep.
- Force Mains: Flow velocity exceeds 8 feet per second (fps).
- Pump Stations: Firm capacity is exceeded causing the backup pump to turn on.

7.8.2 Design Storm

A 25-year, 24-hour design storm was modeled to determine system deficiencies. Based on an isopluvial map, a 25-year storm for the City produces about 3 inches of accumulated rain in 24-hours (Figure 7-4). Since part of the system is combined, storms with sharp peaks after steady rain puts the greatest strain on the conveyance system because of the sharp influx of water into a system that is already partially filled. The 3/15/2015 storm was chosen as the 24-hour storm pattern and uniformly escalated to total 3 inches of rain (Figure 7-5). The resulting design storm fit the duration and volume criteria while still mimicking the storm patterns that strain the system.

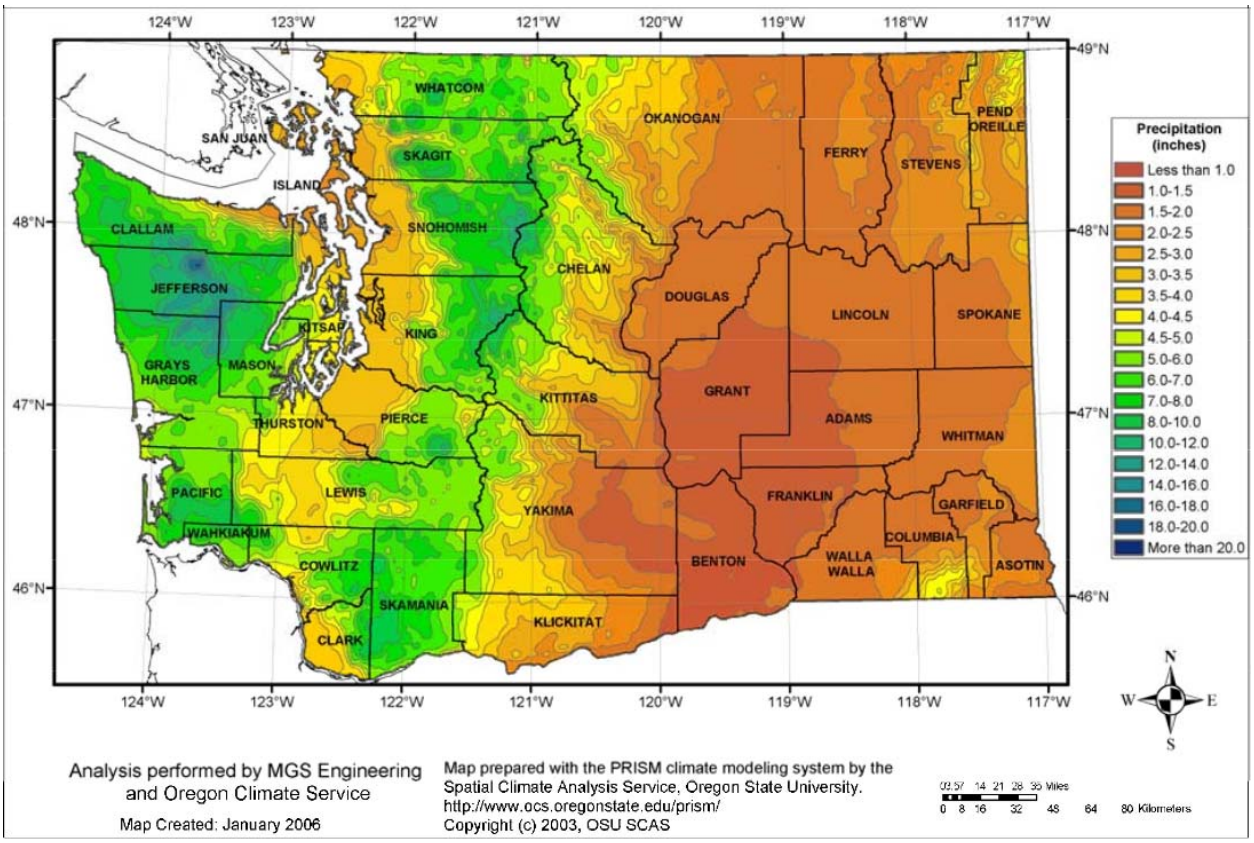


Figure 7-4 Washington State 25-Year, 24-Hour Isopluvial Map

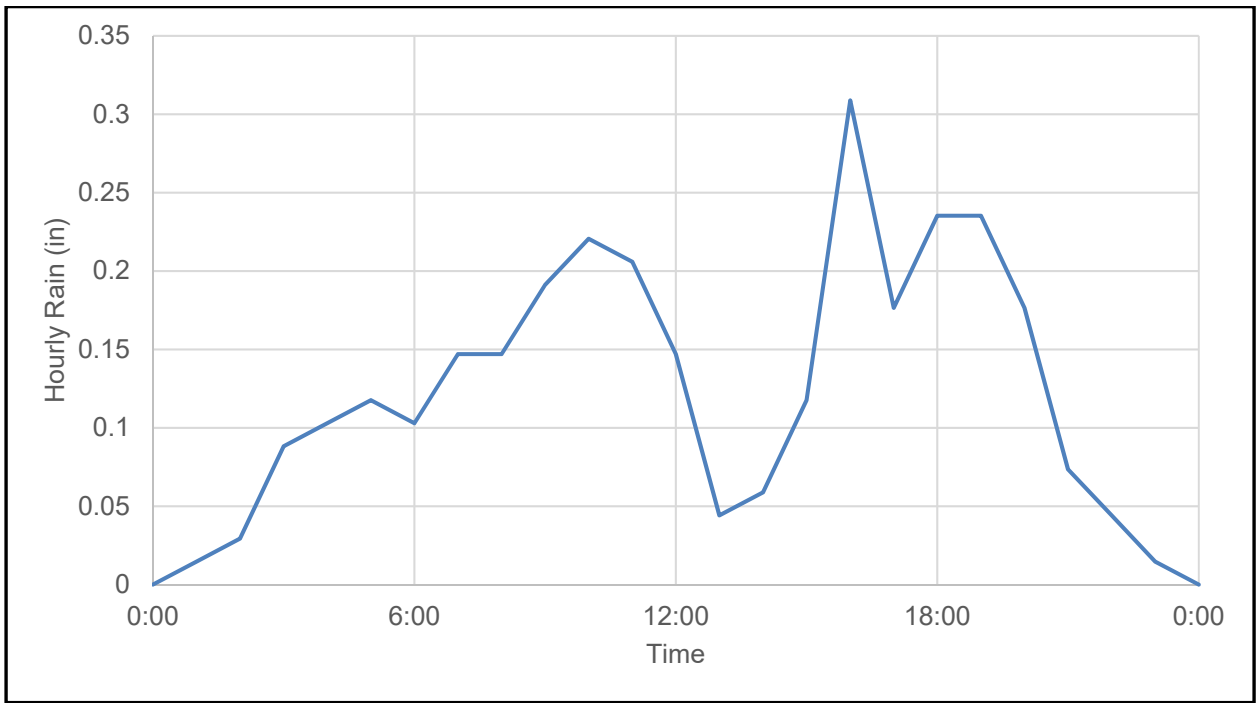


Figure 7-5 25-Year Design Storm

7.8.3 Existing (2016) Collection System Performance

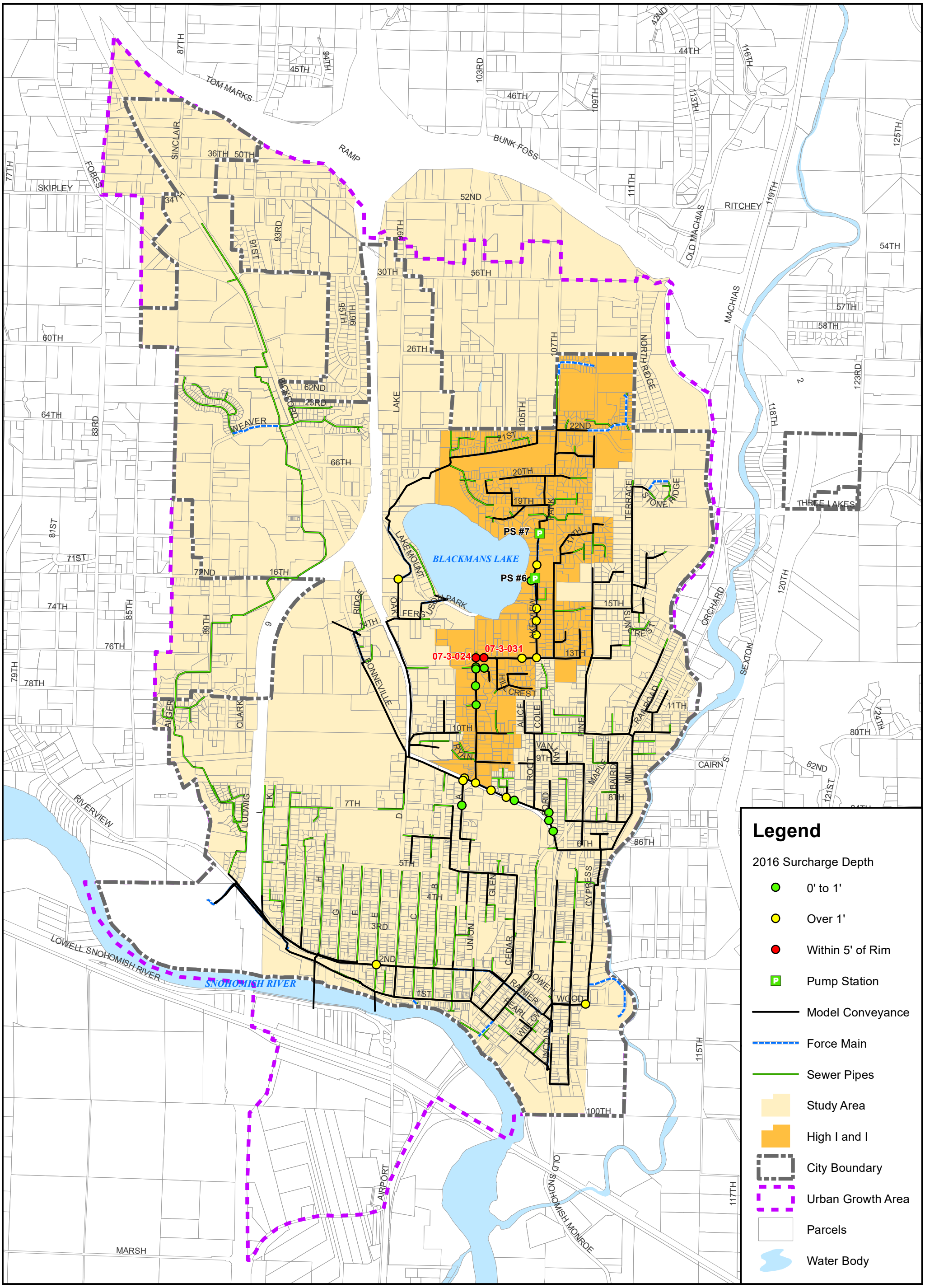
The existing collection system was evaluated to calibrate the model to 2016 flows and populations and determine if there are current deficiencies. Table 7-3 provides a summary of the existing (2016) gravity piping deficiencies. The current pump station deficiencies are listed in Table 7-4 showing the design flow of each pump compared to the predicted flow rates into the pump station. Figure 7-6 shows the location of these deficiencies as well as locations with any manhole surcharging. All of the deficiencies identified are in the portion of the conveyance system which the City indicated has the highest I/I.

Table 7-3 Existing (2016) Gravity Piping Deficiencies

Location	Manhole Number	Depth of Water to Manhole Rim (ft)
13th Street between Avenue A and Lakeview Rd	07-3-031	2.57
Intersection of Avenue A and 13th Street	07-3-024	0.95

Table 7-4 Existing (2016) Pump Station Deficiencies

Pump Station	Pump Design Flow (gpm)	Max Flow into PS (gpm)
PS #7, Champagne Lane	240	365
PS #6, Hill Park	225	491



Legend

2016 Surcharge Depth

- 0' to 1'
- Over 1'
- Within 5' of Rim
- Pump Station
- Model Conveyance
- Force Main
- Sewer Pipes
- Study Area
- High I and I
- City Boundary
- Urban Growth Area
- Parcels
- Water Body

Sewer System & Basins: City of Snohomish.
GIS Base data: Snohomish County.
Data sources supplied may not reflect current or actual conditions. This map is a geographic representation based on information available. It does not represent survey data. No warranty is made concerning the accuracy, currency, or completeness of data depicted on this map.
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City of Snohomish Washington

Existing (2016) Conveyance System Surcharging & Deficiencies

General Sewer Plan and
Wastewater Facilities Plan
City of Snohomish, Washington
February 2019

Figure
7-6

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7.8.4 2022 Model Results

Following calibration of the model, 2022 projected populations and collection system changes (Table 7-2) were modeled to determine if there are deficiencies projected for 2022. For this scenario, there are no additional deficient gravity sewers, pump stations, or force mains compared to the 2016 scenario.

7.8.5 2036 Model Results

The next models included 2036 projected populations and the collection system changes listed in Table 7-2 to determine if there are projected deficiencies in 2036 with and without expanding the system into the UGA and the portion of the North Planning Area south of State Route 2. For both 2036 scenarios, there are no additional deficient gravity sewers, pump stations, or force mains compared to the 2016 scenario.

The majority of flow in the system during storms comes from I/I, which was not modeled to increase over time as discussed earlier. For this reason, the manhole surcharging and the flow rate do not increase significantly despite the 20 years of increasing population shown in Table 3-2.

7.8.6 Buildout Model Results

The Buildout projected populations were modeled with the upgrades needed for the current system capacity deficiencies to determine if there are additional projected capacity deficiencies if the City reaches buildout capacity. The conveyance system was configured as described in Table 7-2 with flow from expansion of service to the north diverted to the West Trunk. The model showed no additional capacity deficiencies in the system, though there was more surcharging in manholes within acceptable limits.

7.8.7 Combined Sewer Overflow Compliance

The NPDES permit has a Performance Standard for Controlled CSOs, defined as no more than one untreated discharge per outfall per year on a 5-year rolling average. The City's current NPDES permit required submittal of a "Combined Sewer Overflow Reduction Plan Amendment" identifying which of the two CSO outfalls can be categorized as being in compliance with this portion of the permit. Alternatives must be identified and evaluated to reach the performance standard for each CSO outfall that does not meet the performance standard and an expected compliance date must be provided.

As documented in the October 2014 *Combined Sewer Overflow Management Plan* (CSO Management Plan) (BHC, 2014), both CSO outfalls were determined to have met the performance standard based on review of historical data and were projected to continue to be in compliance into the future, based on hydraulic modeling of the CSS using 20-years of historical rainfall data. As a result, neither CSO outfall required identification, evaluation or implementation of additional controls.

Despite meeting the compliance criteria for the NPDES permit CSO performance standard, the City is intent on further reducing CSOs. The City has already implemented some of the operational adjustments and system improvements recommended in the CSO Management Plan, including:

- Increased WWTP Lagoon Attenuation Volume – Since biosolids were dredged from the lagoons in 2015, the City has been able to lower the normal operating levels in the lagoons during the wet weather months to provide more volume for flow attenuation.

- Modified CSO Pump Station Speed Control – Modified pump controls to allow operation at 100 percent speed, which increased the throughput of the pump station and reduced potential for upstream surcharging and a resulting CSO discharge.

- Automated Lagoon Effluent Flow Control/Level Adjustment – The City has recently installed electric actuators to allow automated control of effluent gates at the chlorine contact tank. These can be used to control lagoon level, effluent discharge and attenuate flow.

- CSO Sewer Separation Projects – The City is currently working complete separation of storm and sanitary sewers along Avenue F (tributary to CSO 1) by completing the extension of an existing stormwater main that will discharge into a stormwater utility lagoon near the WWTP and infiltrate on-site. A new stormwater pipe has already been installed along Avenue F and is awaiting extension of the stormwater main to the stormwater utility lagoon for final separation, to be completed in 2018. Subsequently, the City will make additional connections to the stormwater main under the CSO 1 Separation Program (CIP C-10) with separation of Avenues E through Riverview Lane between 2nd and 5th Streets.

The capital improvement program in Chapter 12 also includes funding for the CSO 2 Separation Program (CIP C-3). The CSO 2 Separation Program involves separation of storm and sanitary sewers along 1st Street and Avenues A, B, C and D between 1st Street and 2nd Street and conveying stormwater from these areas to an existing stormwater wet well at the CSO pump Station. An existing force main would transfer stormwater from the wet well to the existing stormwater main that would then convey it to the stormwater utility lagoon. These separation programs will disconnect stormwater pipes from the CSS and re-direct the flow to a designated stormwater main that conveys it to the stormwater utility lagoon instead of the WWTP. These projects are discussed further in Chapter 12. The City is also considering raising the weir at the CSO 2 outfall to further reduce the potential for very small, short duration overflows have occurred in the past during very intense summer showers.

Since completion of the CSO Management Plan, the City has collected additional data to support hydraulic modeling for this Plan. This additional data and the subsequent modeling has not changed the results, conclusions, or recommendations pertaining to the CSO outfalls and further mitigation of potential CSO events. Both CSO outfalls are still considered as meeting the performance standard and no additional improvements or controls are deemed necessary.

Chapter 8 Treatment Plant Flow and Load Evaluation

8.1 Sewered Population

Reflective of the slow growth rate countywide and of more recent regional policies on population allocation, the Snohomish County Council adopted amendments to the Countywide Planning Policies (CPPs) in 2013 that revised the current growth target for the City and its urban growth area from a population of 14,535 in 2025 to one of 14,494 in 2035[1]. In 2016 another amendment was adopted that shifted some of the growth from the City limits to the unincorporated areas but kept the total UGA population target the same as the 2013 amendment[2]. The CPPs now project a 2035 population of 12,139 within the current City limits and 2,354 within the unincorporated areas of the City's current UGA.

Sewered population growth was estimated using the 2035 CPPs population allocation figures. The City plans to pursue expansion of the UGA at some point after 2022 but does not know if it will happen in the planning horizon. For this reason, two different populations were considered for the 2036 projections, although the UGA expansion was included in the buildout scenario. For the purpose of being conservative, the wastewater flow and load projections for 2036 include the sewered population expanding into parts of the UGA and north planning area. Table 8-1 provides a summary of the projected sewered population through buildout, based on these assumptions.

Table 8-1 Sewered Population Projections

Sewered Population	2016	2022	2036[(a)]	Build Out
Residential	9,569	10,348	14,505	17,995
Employment	4,677	5,027	6,223	7,236

Note:
(a) The City does not know if the sewer system will be expanded in the northern basins in the planning horizon so both scenarios were modeled but only the 2036 population with northern expansion was evaluated to be more conservative.

8.2 Historical Wastewater Flows and Loads

8.2.1 Data Sources

Data from WWTP discharge monitoring reports (DMRs) from January 2006 through December 2015 was reviewed. Table 8-2 summarizes the DMR data for this period. The WWTP began to receive CSOs from a new CSO pump station in October 2011. Therefore, the data is split into two periods: one from 2006 through 2010 and the other from 2011 through 2015.

[1] Snohomish County Ordinance No. 13-032, Effective 30 June 2013
[2] Snohomish County Ordinance No. 16-078, Effective 12 October 2016

Table 8-2 Summary of Plant Influent Data

	Flow (MGD)	BOD_5[(a)] (mg/L)	BOD_5 (ppd)	$CBOD_5$[(b)] (mg/L)	$CBOD_5$ (ppd)	TSS (mg/L)	TSS (ppd)
2006 through 2010							
Overall							
Average	1.23	202	1,729	183	1,581	159	1,404
Minimum Monthly Average	0.66	86	1,235	85	1,262	75	1,081
Maximum Monthly Average	2.51	330	2,147	301	2,050	232	2,031
November through June							
Average	1.44	169	1,753	155	1,608	137	1,460
Maximum Monthly Average	2.51	277	2,147	268	2,050	197	2,031
July through October							
Average	0.81	267	1,681	240	1,525	202	1,290
Maximum Monthly Average	1.24	330	2,143	301	1,745	232	1,506
2011 through 2015							
Overall							
Average	1.30	225	2,070	198	2,141	165	1,666
Minimum Monthly Average	0.57	103	1,448	94	448	90	818
Maximum Monthly Average	2.73	401	2,808	394	8,957	362	2,333
November through June							
Average	1.56	174	2,127	158	2,054	146	1,738
Maximum Monthly Average	2.73	323	2,808	324	7,375	269	2,333
July through October							
Average	0.79	297	1,991	252	1,660	191	1,567
Maximum Monthly Average	1.40	401	2,725	394	4,598	362	2,103

Note:
(a) BOD_5 = 5-day biochemical oxygen demand
(b) $CBOD_5$ = 5-day carbonaceous biochemical oxygen demand

Figure 8-1 contains charts that show the annual flow and loading trending from 2006 through 2015. The annual average flow was fairly consistent from 2006 through 2009. Between 2009 and 2012 there was a noticeable increase in annual flow. After a peak in annual average flow in 2012 (1.55 MGD), 2013 had the lowest annual average flow during that ten-year period (1.11 MGD). 2014 and 2015 flows went back up to the level of flow seen in 2006 through 2009. No distinct trend exists for the maximum month flow because it can be impacted significantly by weather conditions. Over the last 10 years the $CBOD_5$, BOD_5 and TSS loads have continued an upward trend, shown in Figure 8-2. There was a large spike in BOD_5 loads in 2011, but they went back down in 2012, but appears to continue a gradual upwards trend.

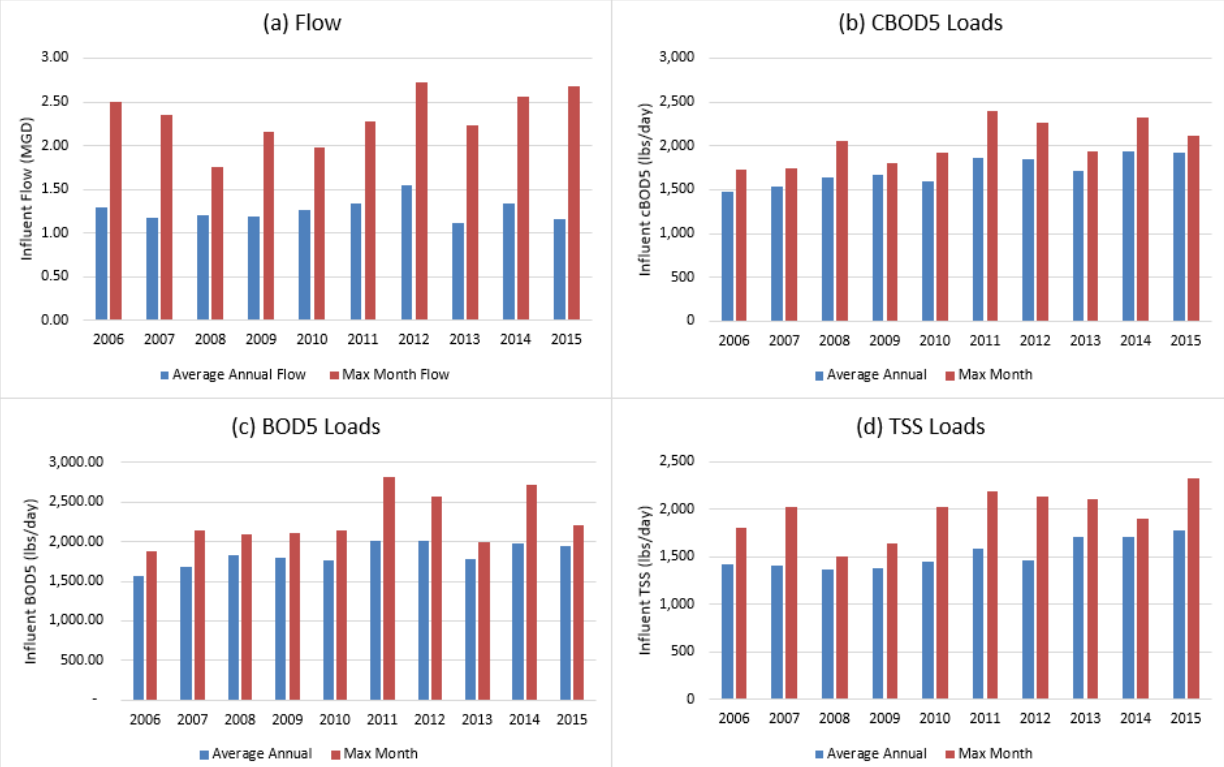


Figure 8-1 WWTP Annual Flow And Loading Trending from 2006 Through 2015

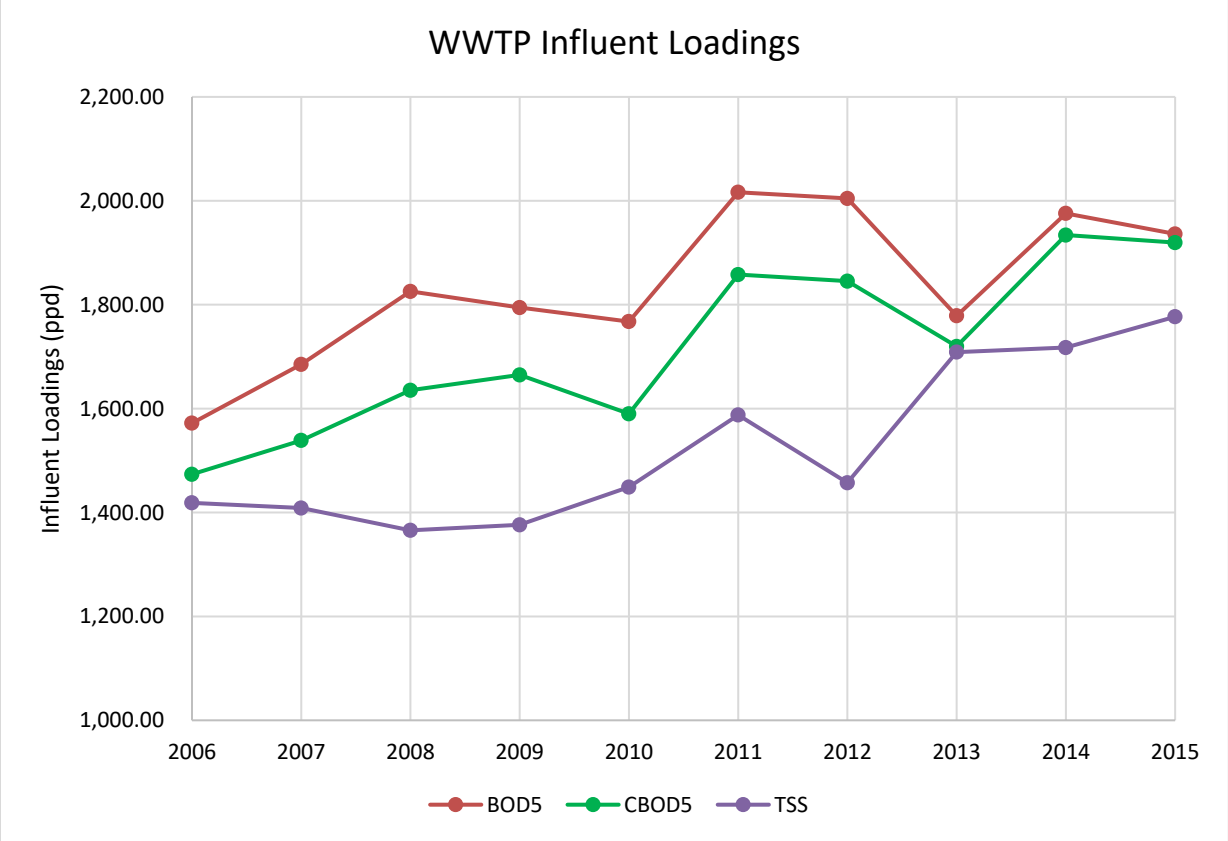


Figure 8-2 WWTP Annual Influent Loadings from 2006 Through 2015

8.3 Existing Wastewater BOD_5 Loading

Influent wastewater quality samples are collected weekly. The collected wastewater represents flow from all sources, including residential, commercial, and infiltration and inflow. The design criteria for BOD_5 at the WWTP are expressed in terms of annual average and maximum month BOD_5 loading in pounds per day (ppd). The permitted monthly maximum BOD_5 is 3,960 ppd.

DMRs were used to determine influent BOD_5 loading in pounds per day based on influent flow. Daily data was then used to generate monthly loading data for 2006 through 2015. This same data was also used to determine loading factors for establishing maximum month and average annual loadings for future service area populations.

8.3.1 Annual Average BOD_5 Loading

The annual average loading is the average of the monthly loadings over the course of one calendar year. The monthly BOD_5 loadings and associated annual average loadings and peak month ratios are provided in Table 8-3.

Table 8-3 Monthly BOD_5 Loading Summary (2006 – 2015)

Month	BOD_5 (ppd)									
	2006	**2007**	**2008**	**2009**	**2010**	**2011**	**2012**	**2013**	**2014**	**2015**
January	1,256	1,865	2,075	1,764	1,824	2,808	1,673	1,921	2,216	1,703
February	1,754	1,713	1,813	1,677	1,925	1,653	2,256	1,715	2,153	1,882
March	1,667	1,563	1,864	2,030	1,617	2,581	2,570	1,769	2,449	2,200
April	1,545	1,554	1,529	1,660	2,147	1,662	1,801	1,598	1,826	1,834
May	1,596	1,235	1,882	2,116	1,728	1,706	2,484	1,817	1,620	2,082
June	1,571	1,732	2,091	1,656	2,005	2,357	2,046	1,830	1,556	1,905
July	1,442	1,606	1,556	1,772	1,452	1,937	1,563	1,624	1,556	1,887
August	1,878	1,785	1,759	1,600	1,694	1,798	1,803	1,700	1,559	1,691
September	1,419	2,143	1,872	1,956	1,508	1,919	1,448	1,941	2,724	1,963
October	1,243	1,828	1,881	1,608	1,621	1,829	1,698	1,988	2,081	1,784
November	1,797	1,695	1,885	1,749	1,717	1,986	2,560	1,664	1,964	2,089
December	1,695	1,504	1,700	1,949	1,974	1,960	2,153	1,776	2,007	2,212
Annual Average	**1,572**	**1,685**	**1,826**	**1,795**	**1,768**	**2,016**	**2,005**	**1,779**	**1,976**	**1,936**
Maximum Month	**1,878**	**2,143**	**2,091**	**2,116**	**2,147**	**2,808**	**2,570**	**1,988**	**2,724**	**2,212**
Maximum Month Ratio (Peaking Factor)	**1.19**	**1.27**	**1.15**	**1.18**	**1.21**	**1.39**	**1.28**	**1.12**	**1.38**	**1.14**

The monthly average BOD_5 loadings for 2006-2015 vary between a low of 1,235 and a high of 2,808 ppd. The maximum monthly BOD_5 loadings for each year ranged from 47 to 71 percent of the design and permitted capacity. The ratio of peak month to annual average BOD_5 loading ranged from 1.12 up to 1.39 with a five-year average (2011 – 2015) of 1.26. This ratio is an indicator of variability over the course of the year. A ratio close to 1 indicates lower fluctuations in wastewater quality. Peak day BOD_5 loadings are not shown in Table 8-3 as they are not typically

used in the design of treatment facilities. However, influent loads recorded by the City do occasionally have significant spikes.

The 2011-2015 annual average BOD_5 concentration was 225 milligrams per liter (mg/L). Typical domestic wastewater has a BOD_5 concentration in the 200-250 mg/L range, indicating the City's wastewater is considered typical for BOD_5.

8.4 Existing Wastewater TSS Loading

The design criteria for TSS at the WWTP are expressed in terms of annual average and maximum month TSS loading in ppd. The permitted monthly maximum TSS is 4,400 ppd.

DMRs were used to determine influent TSS loading in pounds per day based on influent flow. Daily data was then used to generate monthly loading data for 2006 through 2015. This same data was also used to determine loading factors for establishing maximum month and average day loadings for future service area populations.

8.4.1 Annual Average TSS Loading

The annual average loading is the average of the monthly loadings over the course of one calendar year. The monthly TSS loadings and associated annual average loadings and peak month ratios are provided in Table 8-4.

Table 8-4 Monthly TSS Loading Summary (2006-2015)

Month	TSS (ppd)									
	2006	**2007**	**2008**	**2009**	**2010**	**2011**	**2012**	**2013**	**2014**	**2015**
January	1,402	2,031	1,477	1,639	1,801	1,791	1,402	2,112	1,872	1,685
February	1,194	1,459	1,278	1,424	1,389	1,279	1,462	1,826	1,724	1,938
March	1,180	1,533	1,422	1,584	1,239	2,190	1,728	1,494	1,832	2,333
April	1,506	1,366	1,237	1,252	1,470	1,412	1,481	1,441	1,736	1,716
May	1,289	1,377	1,379	1,507	1,409	1,498	1,583	1,614	1,652	1,698
June	1,256	1,308	1,501	1,141	1,583	1,679	1,481	1,804	1,422	1,848
July	1,258	1,155	1,166	1,268	1,179	1,362	1,234	2,103	1,422	1,532
August	1,399	1,081	1,478	1,313	1,188	1,373	972	1,794	1,846	1,830
September	1,426	1,274	1,139	1,247	1,324	1,493	818	1,521	1,879	1,506
October	1,506	1,275	1,421	1,440	1,264	1,379	1,246	1,451	1,606	1,507
November	1,802	1,608	1,416	1,254	1,516	1,964	2,137	1,558	1,718	1,587
December	1,806	1,435	1,475	1,445	2,025	1,634	1,944	1,788	1,901	2,140
Average Annual	**1,419**	**1,409**	**1,366**	**1,376**	**1,449**	**1,588**	**1,457**	**1,709**	**1,717**	**1,777**
Maximum Month	**1,806**	**2,031**	**1,501**	**1,639**	**2,025**	**2,190**	**2,137**	**2,112**	**1,901**	**2,333**
Maximum Month Ratio (Peaking Factor)	**1.27**	**1.44**	**1.10**	**1.19**	**1.40**	**1.38**	**1.47**	**1.24**	**1.11**	**1.31**

The monthly average TSS loadings for 2006-2015 vary between a low of 818 and a high of 2,333 ppd. The maximum monthly TSS loadings for each year ranged from 34 to 53 percent of the WWTP design capacity and permit limit. The ratio of peak month to annual average TSS load ranged from 1.07 to 1.47 with a five-year average ratio (2011 – 2015) of 1.30. This ratio is an indicator of variability over the course of the year. The TSS average maximum month ratio of 1.30 indicates variability in TSS loading over the course of the 2011-2015 period to be on the low side of the expected range of 1.3-1.4. Peak day TSS loadings are not shown in Table 8-4 as they are not typically used in the design of treatment facilities. However, influent loads recorded by the City do occasionally have significant spikes. The 2011-2015 annual average TSS concentration was approximately 178 mg/L. Medium strength domestic wastewater has a TSS concentration around 200 mg/L, indicating the City's wastewater is considered on the weaker side for TSS.

8.5 Projected Wastewater Flows and Loads

Wastewater flows were projected to future conditions based on the land uses established by current zoning for the future service area. Unit flows developed in Chapter 5 were assumed to remain constant throughout the planning horizon.

8.5.1 Wastewater Flows

Per capita residential wastewater flow rates and population projections developed in previous chapters were used to estimate future flows. Table 8-5 provides a summary of the flow projections discussed in Chapter 5. This analysis assumes that there will be expansion into the UGA and North Planning Area south of State Route 2 by 2036.

Table 8-5 Flow Projections

	2016	2022	2036[(a)]
Sewered Residential Population	9,569	10,348	14,505
Maximum Month Flow (MGD)	2.73	2.80	3.22
Peak Day Flow (MGD)	8.30	8.10[(b)]	9.42
Peak Hour Flow (MGD)	14.38	13.67[(b)]	14.74

Note:
(a) The 2036 scenario in this analysis assumes expansion into the UGA and North Planning Area south of State Route 2.
(b) Peak flows are projected to drop between 2016 and 2022 due to the completion of storm and sanitary sewer separation along Avenue F during this timeframe.

8.5.2 Loading Factors

Loading factors are commonly used to project the total wastewater loadings at the WWTP for treatment process sizing, upgrades, and improvements. The loading factors are calculated by dividing the maximum month loading or annual average loading by the residential population served at that time. The loading from the employment population is incorporated into these loading factors since residential population is divided into the total historical loadings. Because the current and projected future ratios of residential to employment populations remain nearly the same, as seen in Chapter 3, only the residential population was used to determine the loading factors and project future loads for simplicity. Annual average and maximum month loading factors were calculated for BOD_5, TSS, Ammonia-N and total Kjeldahl nitrogen (TKN) loadings and are shown in Table 8-6. These loading factors represent the highest historical values.

Table 8-6 Waste Loading Factors

	BOD_5[(b)]	TSS[(b)]	Ammonia[(c)]	TKN[(c)]
Annual Average Load (ppcd)[(a)]	0.21	0.19	0.02	0.03
Maximum Month Factor (ppcd)[(a)]	0.28	0.24	0.02	0.04

Notes:
(a) 2016 loading factors are calculated using current sewered residential population. Employment is incorporated into the loading factor as discussed above.
(b) BOD_5 and TSS Loading factors are based on comparison with 2013-2016 data, considered to be most representative of current conditions.
(c) Ammonia and TKN Loading factors are based on 2013-2014 Ammonia and TKN data from DMRs.

8.5.3 Wastewater Loading

Waste load projections are based on population projections. Although I/I represents significant hydraulic loading at the WWTP, it is assumed to have negligible waste load. Waste load factors [in terms of pounds per capita per day (ppcd)] were estimated based on influent BOD_5, TSS, Ammonia-N and TKN DMR data from 2013 through 2016. These factors were applied to the updated population projections to develop waste load projections for BOD_5, TSS, Ammonia-N and TKN in terms of ppd. The waste load projections for the planning period are provided in Table 8-7.

Table 8-7 Waste Load Projections

	2016	2022	2036
Average Annual Load[(a)]			
Sewered Residential Population	9,569	10,255	14,272
BOD_5 (ppd)	1,996	2,160	3,030
TSS (ppd)	1,772	1,920	2,690
Ammonia-N (ppd)	161	170	240
TKN (ppd)	276	300	420
Maximum Month Load[(a)]			
BOD_5 (ppd)	2,724	2,950	4,130
TSS (ppd)	2,333	2,520	3,540
Ammonia-N (ppd)	194	210	290
TKN (ppd)	362	390	550

Notes:
(a) Projected loadings are based on residential service population projections and calculated 2016 loading factors, and are rounded to the nearest 10 ppd.

8.6 Design Criteria

Updated flow and load projections were compared with actual historical data from the WWTP discharge monitoring reports, as presented on Figure 8-3, Figure 8-4, and Figure 8-5. As shown, the updated projections indicate that the permitted capacity for influent flow will be reached before influent wastewater reaches permitted capacities for BOD_5 or TSS loading. The rate change in the increased loading rate starting in 2022 is in response to the population increase from the assumed northern expansion between 2022 and 2036.

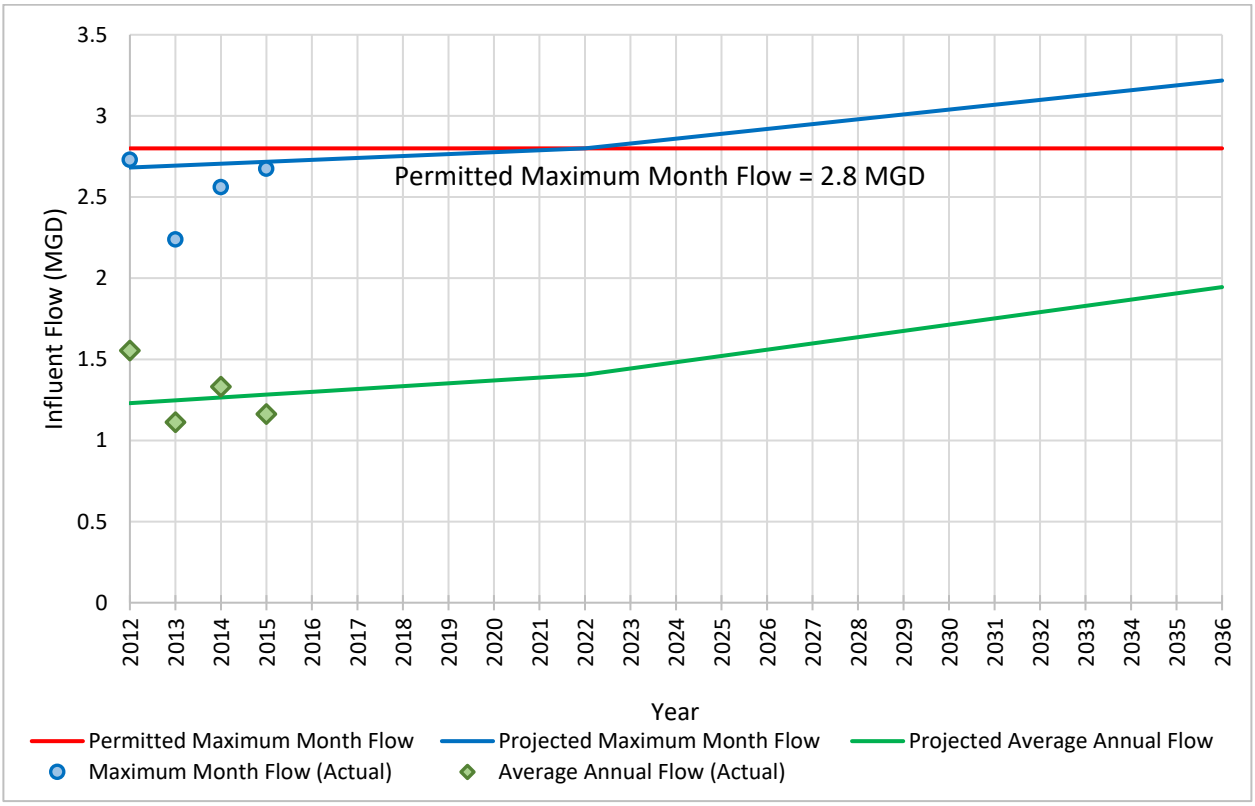


Figure 8-3 Maximum Month Influent Flow Projection with Historical Data

As shown on Figure 8-3, the maximum month flow during 2012 was near the permitted capacity of 2.8 MGD due to the occurrence of an extreme storm event in December 2012 (accumulative rainfall of 8.7 inches) and the prevention of any CSO events throughout 2012. As shown for following years, the maximum month flow is below the projected value in the absence of such extremes. The WWTP is projected to surpass its permitted flow of 2.8 MGD in 2023, which falls within the planning horizon.

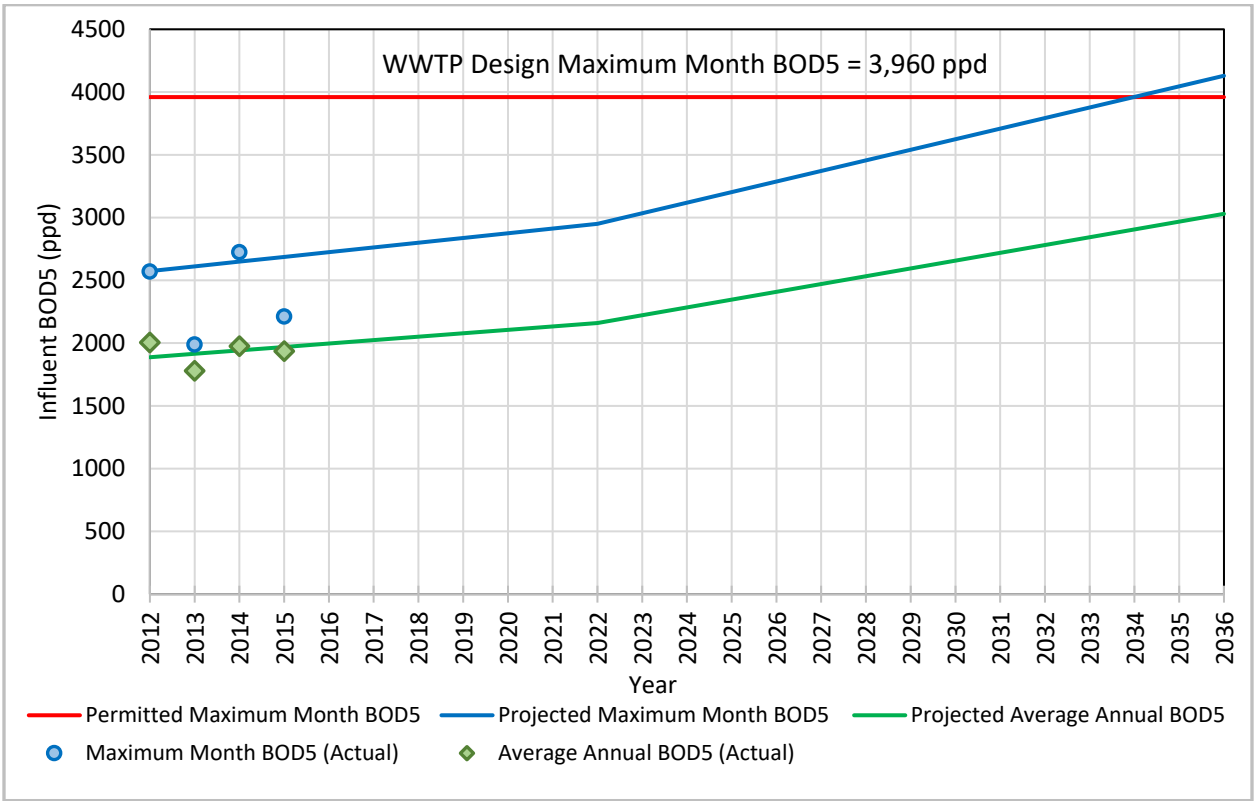


Figure 8-4 Maximum Month Influent BOD5 Load Projection with Historical Data

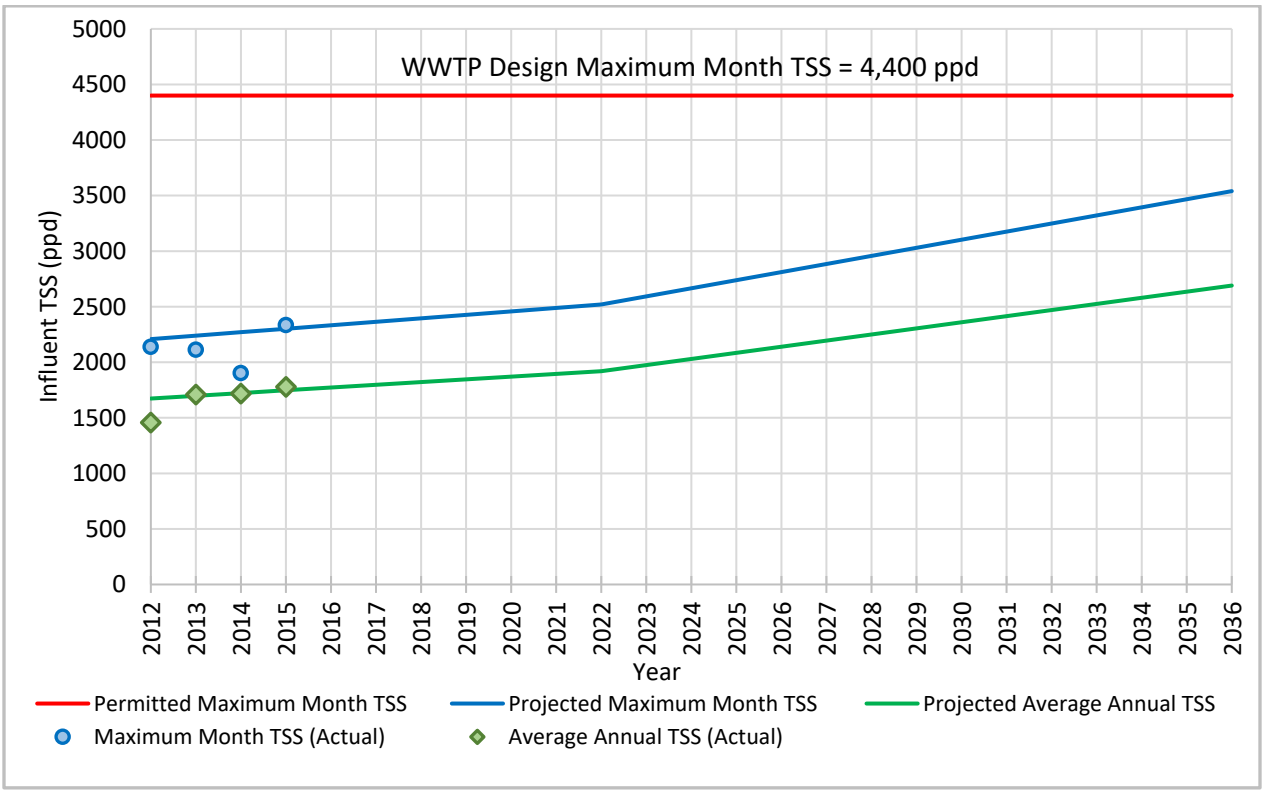


Figure 8-5 Maximum Month Influent TSS Load Projection with Historical Data

The planning horizon for this Plan extends through 2036. As indicated by the projections, the WWTP appears to have sufficient flow capacity through 2022, based on the current permitted capacity. After 2022 flows are projected to surpass the permitted capacity for maximum month flow. After 2034, the maximum monthly BOD_5 load is projected to surpass the permitted influent BOD_5 load for the WWTP, but the maximum monthly TSS load remains within the permitted influent TSS load throughout the planning horizon. The projected 2036 flows and loadings were used as the basis for identifying deficiencies and recommending improvements to the existing WWTP.

Chapter 9 Existing Wastewater Treatment Facility

9.1 History

The original WWTP was constructed in 1958 and consisted of a 40-acre stabilization pond with chlorine disinfection facilities and an outfall to the Snohomish River. New effluent chlorination facilities consisting of a chlorine contact tank and chlorine feed and mixing equipment were added in 1989, along with a new outfall.

In 1995, the City implemented several improvements to enhance performance. A new headworks was constructed upstream of a new lagoon system. The headworks consisted of three influent screw pumps, a rotary screen, and a manually cleaned bar screen. The lagoon system consists of four aerated lagoons on 10 acres of the old stabilization pond, functioning as a dual-powered, multi-cellular (DPMC) aerated lagoon system. The DPMC system includes a large completely mixed aerated lagoon cell followed by three smaller partially mixed lagoon cells of equal volume. The lagoons are equipped with floating high-speed surface aerators for aeration and mixing. The remaining 30 acres of the old lagoon was used for treatment during construction, but has been decommissioned. Other major improvements implemented in 1995 include addition of an effluent filtration system, laboratory/office building and chlorination building and modifications to the chlorine contact tank to facilitate dechlorination. Dechlorination facilities were added to comply with the residual chlorine limits specified in the NPDES permit.

Not long after completion of WWTP improvements in 1995, Ecology issued NPDES permit limits for ammonia as a result of a TMDL study for the Snohomish River Estuary. Because the WWTP improvements completed in 1995 were not designed to remove ammonia, the WWTP struggled to comply with those new limits. Consequently, the City entered into a Consent Decree in 2003. The 2005 Plan was prepared to address requirements of the Consent Decree. However, the 2005 Plan was not implemented due to significant changes in the public works staff, the priority to first address CSOs and initial plans for expansion of the City's UGA boundary. In 2009, the City decided to pursue regionalization of wastewater treatment in conjunction with the City of Everett. The City prepared the 2010 Plan and subsequent design documents for a conveyance system that would transport all wastewater collected at the WWTP to the City of Everett (Everett) collection system, as required under Agreed Order No. 7973 with Ecology.

In 2012, the City completed near-term improvements to the existing WWTP to comply with separate Agreed Order No. 7974 issued by Ecology. Ecology established the near-term Agreed Order because of the extensive amount of time required to plan and design the conveyance system project for regionalization and secure funds to finance it. Primary improvements included in the near-term project included: (1) replacing the old rotary screen with a new multi-rake bar screen to expand the screening flow capacity and meet Ecology's 2007 biosolids rule, (2) installing SFF media modules in the lagoons to enhance nitrification and soluble $CBOD_5$ removal, and (3) installing a supplemental alkalinity feed system for the lagoons to provide sufficient alkalinity to support nitrification and maintain a neutral pH for growth of nitrifying bacteria.

Based on the initial success of these near-term improvements, Ecology issued Agreed Order No. 10467, which amended initial Agreed Order No. 7974, that allowed plans for regionalization to be delayed or postponed indefinitely pending further demonstrated success of the near-term

improvements. The City submitted their Notice of Compliance (Appendix B) with all of the Agreed Orders to Ecology in 2015. On March 10, 2015, Ecology determined that the City had satisfied the conditions in Agreed Order No. 7973 and Agreed Order No. 10467 (amending Agreed Order No. 7974) and was thus no longer required to connect their wastewater system to the City of Everett's conveyance system. The City has indefinitely postponed the conveyance project and is focusing on improvements to the current system.

A site plan and process flow diagram of the current WWTP are shown in Figure 9-1 and Figure 9-2, respectively.

NORTH

0 50 100

Scale in Feet

WOODEN FENCE

GREASE INTERCEPTOR

LAGOON ACCESS RAMP

CONC PAD W/ POWER TRANS

COMPRESSED GAS BLDG

REJECT PUMP STATION

GATE

CHLORINATION BUILDING

CONTROL PANEL FOR FILTERS

CONTROL PANEL FOR PUMP

ACCESS LID

EFFLUENT FILTER

CHLORINE CONTACT TANK

BLOWER BLDG

EFFLUENT CONTROL STRUCTURE

VAULT

FILTER PUMP STATION

DECANT STRUCTURE

GRAVEL PATH

OUTLET STRUCTURE, TYP

INFLUENT LIFT STATION

CHAINLINK FENCE

CHEMICAL BLDG

LAGOON 1

LAGOON NO. 2

LAGOON NO. 3

LAGOON NO. 4

LAGOON DRAIN STRUC

SNOHOMISH RIVER

GENERATOR BUILDING

WATER ACCESS BOX

LUMINARE ON CONC PAD

SURFACE AERATOR, TYP

LAGOON DRAIN STRUCTURE

CONC PAD WITH POWER TRANS

PARKING

LAB/ OFFICE

DIESEL STORAGE TANK

MOORING CABLE, TYP

MOORING POST, TYP

CARGO CONTAINER

OUTLET STRUC, TYP

GRAVEL

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Existing WWTP Site Plan

General Sewer Plan and Wastewater Facilities Plan

SNOHOMISH WASHINGTON

February 2019

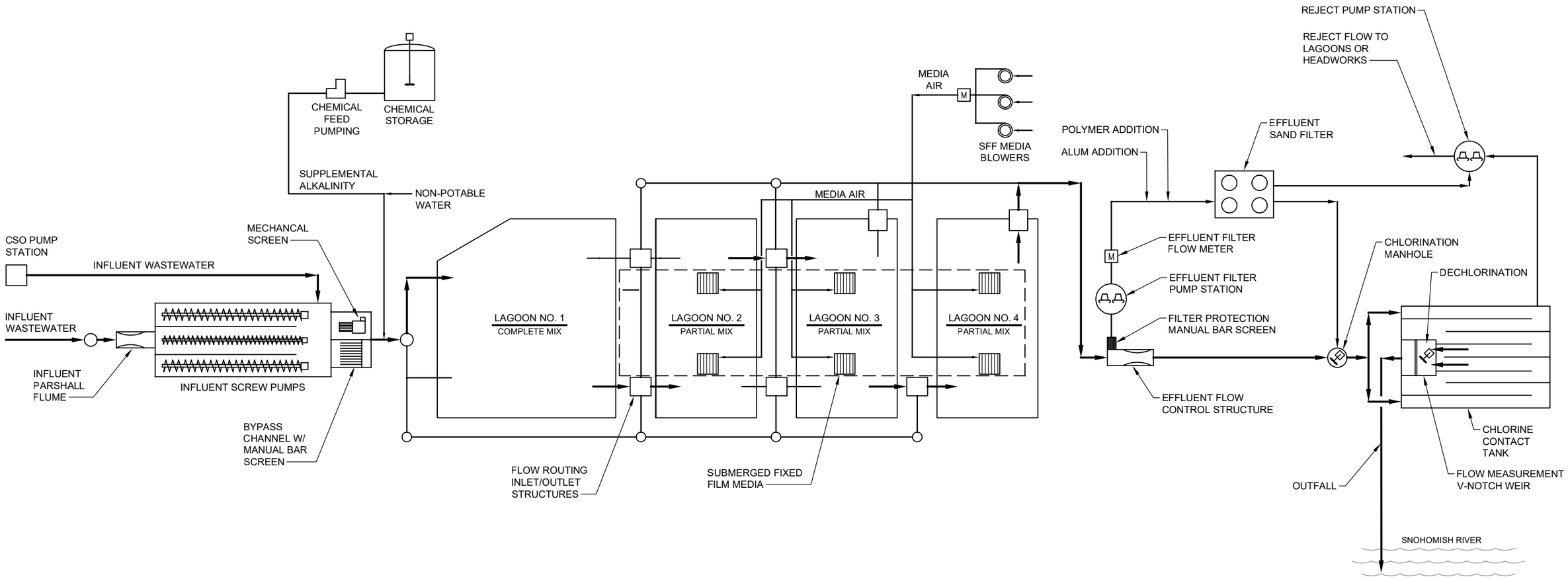
Figure

9-1

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NOTE:

FLOW DIRECTION ARROWS REPRESENT NORMAL OPERATIONAL FLOW ROUTING.

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WWTP
Process Flow Diagram
General Sewer Plan and
Wastewater Facilities Plan
February 2019

Figure
9-2

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9.2 Treatment Plant Permit Compliance

Data from plant DMRs from January 2006 through December 2015 were reviewed for this update. Table 9-1 (Revised 2016) summarizes the effluent data from this period. As evident from comparing the 2006 through 2009 data, the 2010 through 2013, and 2014 through 2015 data in Table 9-1, the City's actions to enhance treatment has improved effluent quality, particularly during the critical months of July through October. Further improvements in effluent quality have been achieved since the startup of the SFF media system and other near-term improvements in September 2012 which can be seen in the 2014 through 2015 data in Table 9-1

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Table 9-1 Summary of Plant Effluent Data (January 2006 through December 2015)

	Daily Flow (MGD)	Daily $CBOD_5$ (mg/L)	Daily $CBOD_5$ (ppd)	Monthly $CBOD_5$ (ppd)	Monthly $CBOD_5$ (percent removal)	Daily TSS (mg/L)	Monthly TSS (ppd)	Monthly TSS (percent removal)	Daily Ammonia (mg/L)	Daily Ammonia (ppd)	Monthly Ammonia (ppd)	Fecal Coliform (#/100 mL)	pH	Total Residual Chlorine (µg/L)
2006 through 2009														
Overall														
Average	1.36	11	116	119	92	13	133	90	13	151	149	44	7	2
Minimum	0.00	3	5	22	77	2	24	79	0	0	3	1	6	0
Maximum	8.42	32	1103	364	99	37	301	98	39	910	358	1333	9	500
November through June														
Average	1.59	10	134	139	91	11	145	90	16	206	198	50	7	1
Maximum	8.42	26	1103	364	96	31	301	97	39	910	358	1333	9	375
July through October														
Average	0.89	11	80	79	95	16	110	92	9	55	51	30	6.9	4
Maximum	4.69	32	295	148	99	37	225	98	34	278	175	1160	8.1	500
2010 through 2013														
Overall														
Average	1.59	6	84	80	96	11	121	92	9	128	118	20	6.9	8
Minimum	0	3	10	16	90	3	32	83	0	0	1	1	6	0
Maximum	10.8	26	512	185	99	48	384	97	26	779	313	1130	7.7	8400
November through June														
Average	1.9	7	105	101	95	11	146	92	12	178	171	26	6.9	11
Maximum	10.8	26	512	185	97	48	384	97	26	779	313	1130	7.6	8400
July through October														
Average	0.86	5	35	32	98	12	73	94	2	13	13	4	6.9	1
Maximum	3.58	9	133	55	99	36	181	97	12	86	30	150	7.7	26
2014 through 2015														
Overall														
Average	1.38	5	68	58	97	12	180	92	6	89	57	13	7.0	6
Minimum	0.00	2	9	13	92	3	28	78	0	0	1	1	6.0	0
Maximum	9.61	26	608	191	99	48	539	98	26	779	188	1130	7.7	8400
November through June														
Average	1.66	5	78	77	97	15	239	90	6	79	84	3	6.7	2.7
Maximum	8.10	9	562	191	99	38	539	96	19	608	188	21	7.3	91
July through October														
Average	0.77	3	21	21	99	8	61	96	0.3	2	2	1	7.0	2.6
Maximum	2.73	6	84	36	99	28	192	98	5	27	9	12	7.7	66

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9.3 Hydraulic Capacity Evaluation

A hydraulic evaluation was performed to determine if there are areas where the wastewater cannot flow through or be contained during peak flows or high river levels. If this were to happen, the water level upstream of the problem area may rise, which can cause other hydraulic and/or process problems. In worst-case situations, water levels may rise high enough to flow over walls or berms. In addition to avoiding adverse impacts to treatment processes and overflows, it is also important to protect processes and equipment from damage due to flooding. Ecology's Criteria for Sewage Works Design (Orange Book) states that "plant unit processes shall be located at an elevation which is not subject to the 100-year flood/wave action".

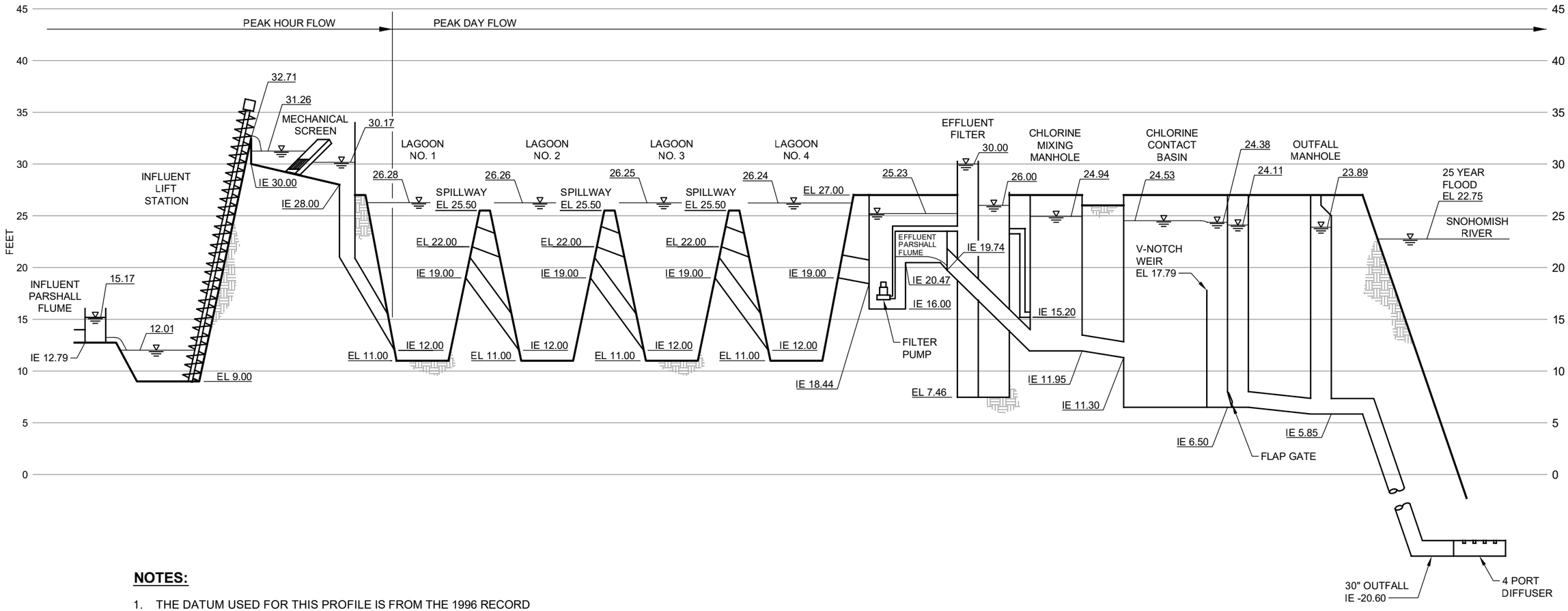
9.3.1 General

A hydraulic profile of the WWTP was developed to assess the hydraulic impacts of the projected flows (see Figure 9-3). The water surface elevations shown on Figure 9-3 are based on the projected 2036 peak hour flow through the headworks and into the first lagoon cell and the projected 2036 peak day flow from the first lagoon through the outfall, due to flow attenuation in the lagoon cells. The water surface elevations in the hydraulic profile are also based on the 25-year flood condition in the Snohomish River, which is based on a river elevation of 22.75 ft using the datum in the hydraulic profile from the 1996 record drawings. That datum is approximately 3 feet lower than the NAVD 88 datum. So, the 25-year flood elevation in the river would be about 25.75 feet converted to the NAVD 88 datum. The hydraulic analysis (see Appendix F) did not identify any hydraulic restrictions in the structures or piping for the projected flows.

A 2-foot tall downward opening weir gate is provided at the upstream end of each lagoon outlet structure. These weir gates can be manually positioned to control flow into the outlet structures and water level in the lagoons. Downstream canal gates can be used to isolate lagoons or throttle flow from outlet structures. When lagoon levels are higher than normal, such that the weir gate is submerged, the WWTP staff manually adjusts the canal gate on the discharge from the Lagoon No. 4 outlet structure and the isolation gates on the outlet from each of the chlorine contact chambers to manage flow through the disinfection system and attenuation in the lagoons. Every foot of additional operating depth above the normal operating depth of 10 feet translates to approximately 2 million gallons of additional volume total in all four lagoons. The City has recently added electric actuators to the gates on the chlorine contact chamber outlets that will enable remote/automated isolation and control of water levels in the lagoons.

The existing 30-inch outfall extending from the chlorine contact tank terminates in the Snohomish River with a four-port diffuser at a depth of approximately -20.60 feet using the hydraulic profile datum from the 1996 record drawings (approximately -17.6 feet NAVD 88). The latest mixing zone study was performed in 2001 by Cosmopolitan Engineering using a maximum month flow (MMF) of 2.8 MGD, which is consistent with the current NPDES permit. Because the 2001 mixing zone study reflects the current permitted flow and load capacities and these capacities are projected to be exceeded by 2036, a new mixing study will need to be performed for higher flows before the WWTP can be re-rated for a higher flow. Further detail on what is required for a re-rating is provided in the discussion of flows and loads below.

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NOTES:

1. THE DATUM USED FOR THIS PROFILE IS FROM THE 1996 RECORD DRAWINGS, WHICH IS ABOUT 3 FEET BELOW THE NAVD 88 DATUM.
2. WATER SURFACE ELEVATIONS BASED ON PROJECTED 2036 PEAK FLOWS, PEAK HOUR FLOW APPLIED THROUGH HEADWORKS AND PEAK DAY FLOW APPLIED DOWNSTREAM OF HEADWORKS.

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WWTP
Hydraulic Profile
General Sewer Plan and
Wastewater Facilities Plan
February 2019

Figure
9-3

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9.3.2 Flood Conditions

The current discharge arrangement allows sufficient flow by gravity from the chlorine contact tank through the outfall under most flood conditions. Only under extreme flood conditions is gravity flow through the outfall no longer possible. Over the last 10 years, there have only been two instances during which gravity flow through the outfall into the river was not possible for a period of time. One instance occurred during November 2006 when the river stage crested at about 33.5 feet, equivalent to about 27.4 feet on the NAVD 88 datum. In this instance, there was sufficient storage available in the lagoons to hold the influent flow, such that if gravity discharge ceased for a short period of time until the river level subsided, there was not risk of the lagoons or process structures overflowing. The second instance occurred in January 2009 when the river stage again crested at about 33.5 feet (27.4 feet on the NAVD 88 datum). This time, there was too much influent entering the WWTP from continued rainfall to hold all of the flow in the lagoons until the river level subsided enough to make gravity flow through the outfall possible. Therefore, the City rented a portable pump and for a period of about 24 hours the pump was used to convey effluent to the river until the following day when gravity flow was possible again. In both instances, the elevation at which the river crested (about 27.4 feet NAVD 88) was very close to the previously established 100-year flood elevation at the time (about 27.7 feet NAVD 88), which has been recently revised to about 29.0 feet NAVD 88.

The City of Snohomish General Sewer Plan and Wastewater Facilities Plan (Tetra Tech/KCM, May 2005) recommended the addition of an effluent pump station. However, given that such infrastructure would be utilized very infrequently (over the last 10 years it would have been required on only two occasions at most for periods of 24 hours or less) and the cost of constructing and maintaining an effluent pump station would be substantial, a permanent effluent pumping installation is impractical. By monitoring real-time flood warning information on the Snohomish County website, the WWTP staff has plenty of advanced warning to maximize available storage in the lagoons and setup and implement temporary pumping as a last resort.

The City had been maintaining higher lagoon levels to provide a greater buffer between the sediment layer and the water surface in an effort to minimize the impact of periodic lagoon turnover events. The City removed biosolids from the lagoons in 2015, after which the normal operating level in the lagoons could be reduced from a depth of 11.5 feet to the original design depth of 10.0 feet. This increased the storage volume available for attenuation from 9.2 MG to 13.5 MG, a 32 percent increase. The greater storage volume allows the City to maintain a lower effluent flow during storm events. This allows the City to better utilize the lagoons for extra storage during extreme flooding conditions.

The City of Snohomish also purchased a portable gas-powered trash pump that is capable of pumping up to 1,000 gallons per minute. This pump can be used if the lagoons get overwhelmed during extreme flood conditions. However, with the increased volume available in the lagoons for attenuation of flow and the fact that river stage often crests after the more intense rainfall and associated influent flow, it is anticipated that the need for pumping would be very rare.

9.4 Process Capacity Analysis

A process review and capacity evaluation were performed for each unit process at the treatment plant. Each unit process treatment capacity was evaluated to identify deficiencies under the existing and projected flows and loads.

9.4.1 Flows and Loads

Table 9-2 compares existing and projected flows and loads to the WWTP's rated capacity in the NPDES permit and the 1996 and 2013 record drawings. As discussed in Chapter 8, the proportional difference between the modeled and actual peak day flow for 2016 was used to increase the projected peak day flows for 2022 and 2036. This same proportional difference was used to increase projected peak hour flows as well. These adjustments represent more conservative peak flow projections than what the modeled predicted.

Table 9-2 Revised Projections for 2036 Planning Horizon

Component	Design	2016	Projected	
			2022	2036
Sewered Residential Population	N/A	9,569	10,348	14,505
Flow, MGD				
Average Annual	2.04	1.30	1.40	1.95
Maximum Month	2.8	2.73	2.80	3.22
Maximum Day	9.9	8.30	8.10	9.42
Peak Hour	30.9	14.38	13.67	14.74
BOD_5, ppd				
Average Annual	2,892	1,996	2,160	3,030
Maximum Month	3,960	2,724	2,950	4,130
TSS, ppd				
Average Annual	3,233	1,772	1,920	2,690
Maximum Month	4,400	2,333	2,520	3,540
Total Kjeldahl Nitrogen (TKN), ppd (Actual Design includes 30% Safety Factor)				
Avg. Annual (452 ppd w/ S.F.)	348	276	300	420
Max. Month (607 ppd w/ S.F.)	467	362	390	550
Ammonia-N, ppd				
Avg. Annual (294 ppd w/ S.F.)	226	161	170	240
Max. Month (377 ppd w/ S.F.)	290	194	210	290

The projected maximum month flow in 2036 of 3.22 MGD is higher than the WWTP's rated capacity of 2.8 MGD. As discussed above, the hydraulic analysis demonstrates that the WWTP is capable of hydraulically handling the projected 2036 flows. The BOD_5 loading is also projected to surpass the permitted maximum month BOD_5 loading before the end of the planning horizon. The Ammonia-N is projected to stay below or at the design loading for the planning horizon. Projected TKN loads for 2036 exceed design values referenced in the 2013 record drawings. However, the SFF media system was sized with a safety factor not represented in the design values that allows it to treat 30% more TKN and ammonia-N than is represented by the design values in the 2013 record drawings. This safety factor was added because of limited ammonia and TKN available at the time of design and the fact the that SFF media system was considered a new and developmental technology.

The current (2016) peak flows to the treatment plant is higher than projected for 2022 due to planned separation of storm and sanitary sewers along Avenue F, as discussed in Chapter 7 and summarized in Table 7-2, but then are projected to increase again for 2036 as more growth

occurs. Although additional sewer separation is planned to occur during the planning horizon, the projections are conservative in that they only account for sewer separation along Avenue F.

9.4.2 Influent Flow Measurement

Description

Wastewater enters the headworks through a 30-inch gravity sewer line and an 18-inch sewer force main extending from the CSO pump station. The 30-inch sewer line feeds into an open channel that directs the flow through a fiberglass Parshall flume with an 18-inch throat, which produces a differential head that can be related to discharge flow rate. An ultrasonic level sensor measures the liquid level created by the Parshall flume and converts the measurement to a corresponding flow. The influent flow through the 18-inch force main is measured by a 16-inch magnetic flow meter located near the CSO pump station and transmitted to the WWTP supervisory control and data acquisition (SCADA) system for use in tracking and monitoring total influent flow. Flow from the 18-inch force main is introduced downstream of the influent pumps, such that it is not measured by the Parshall flume.

Capacity

The 18-inch flume has adequate capacity to measure influent flows up to 25 MGD. A summary of the design, existing and projected influent flows is provided in Appendix G. The analysis of the current systems shows that there are no projected deficiencies with the influent Parshall flume in the planning horizon.

9.4.3 Influent Pumps

Description

Downstream of the Parshall flume, wastewater flow enters a wet well at the upstream end of three Archimedes-type screw pumps that lift the flow to the screening unit. Two screw pumps have an outer diameter of 72 inches with a capacity of 15.5 MGD each and are powered by 100 horsepower (hp) electric motors. The center screw pump is 42 inches in diameter with a capacity of 3.5 MGD and is powered by a 25 hp electric motor. The pumps provide a lift of over 20 feet.

Capacity

The highest projected PHF to the treatment plant during the planning horizon is 14.74 MGD, of which approximately two-thirds would enter through the 30-inch gravity sewer main and the remaining one-third through the 18-inch force main.

If one of the largest screw pumps is out of service, one small and one large pump remain available for operation with a combined capacity of 19 MGD. This is more than sufficient to convey the projected PHF, particularly considering that a portion of the PHF is pumped from the CSO pump station to the headworks downstream of the screw pumps through the 18-inch force main. A summary of the design, existing and projected future parameters of the influent pumps is provided in Appendix G.

As discussed below, the MCCs that power the influent pumps are planned to be replaced during the planning horizon. When EMCC1 powering the influent pumps is replaced it is recommended that variable frequency drives for the influent pumps be installed to improve energy efficiency of the pumps.

9.4.4 Screening Systems

Description

Wastewater from the screw pumps and the CSO pump station normally passes through a multi-rake bar screen. Wastewater can be diverted to a manual bar screen or an emergency bypass channel if the bar screen is out of service.

The multi-rake bar screen with ¼-inch openings and associated washer/compactor were installed in 2012 during the near-term improvements to replace the old mechanical screen with ½-inch openings. The new screen provides increased treatment capacity and rag removal and conforms to the new biosolids rule adopted by Ecology in 2007, which requires screening through equipment with a maximum aperture of 3/8-inch prior to beneficial use of biosolids. Wastewater flows through the screen by gravity, and a mat of screenings accumulates on the screen bars. When the differential between the upstream and downstream water levels reaches a set point, a rake mechanism is activated automatically and cleans the bar rack. Screenings scraped by the rakes are deposited into the washer/compactor and washed and dewatered. The compacted screenings are then discharged to a waste container and disposed of as municipal solid waste.

A manual bar screen with ¾-inch openings is provided as a standby unit in a channel parallel to the multi-rake bar screen. When the wastewater passes the screening process, flow is conveyed to a flow control structure, which provides directional flow routing to different areas of Lagoon 1.

Capacity

The multi-rake bar screen has a rated capacity of 23.5 MGD, which exceeds the projected PHF. The manual bar screen serves as a backup. There are no hydraulic limitations caused by the screens. A capacity overview is provided in Appendix G.

9.4.5 DPMC Aerated Lagoon System with SFF Media

Description

The DPMC aerated lagoon system consists of a single, completely mixed cell followed by three equal-volume partially mixed cells. The system was designed for a total hydraulic retention time (HRT) of 7.35 days at the permitted MMF of 2.8 MGD. The completely mixed lagoon (Lagoon 1) has eighteen 15-hp floating surface aerators, and each of the partially mixed lagoons (Lagoons 2, 3, and 4) is equipped with three 7.5-hp floating surface aerators.

The aerated lagoons at the WWTP were originally designed to remove organic material through all four lagoon cells and retain settleable solids in the three partially mixed lagoon cells, with the primary goal of achieving removal of $CBOD_5$. After a TMDL for the Snohomish Estuary was established in 2000, the NPDES permit was modified, and the WWTP had difficulty complying with the discharge requirements, especially TMDL-based limits on Ammonia-N and $CBOD_5$ during July through October. To comply with the permit limits, in 2012 a total of 54 SFF media modules were added to Lagoons 2, 3, and 4 to enhance $CBOD_5$ and Ammonia-N removal, along with a new aeration system to provide air to the SFF media and a supplemental alkalinity dosing system to support nitrification. The SFF media system was placed into operation at the end of September 2012, in accordance with the near-term Agreed Order, and has been operating successfully since.

Flow routing among the lagoons is controlled by inlet-outlet structures, lagoon piping, and distribution manholes. Five sets of inlet/outlet chambers control the direction of the flow among the four lagoons. Each set consists of two separate chambers connected by a 24-inch pipe and a 24-inch overflow bypass line. The inlet chamber houses one 72-by-24-inch weir gate, which controls the flow and elevation of wastewater among the lagoons, and a canal gate that controls flow through the overflow bypass line. The outlet chamber houses four canal gates to route flows to various inlet locations in the lagoons or bypass flow to a lagoon further downstream. Flow may bypass a lagoon during maintenance, such as dredging for removal of accumulated biosolids.

The four lagoons are lined with a 30-mil polyvinyl chloride (PVC) impermeable liner to prevent leakage from entering the groundwater. A leak detection survey was performed on all of the lagoon liners in 2014 and no leaks were found. Two lagoon drain structures can be used to dewater the lagoons. Drainage from a lagoon is conveyed to the drain structures through an 8-inch pipe. Additionally, a 6-inch perforated PVC pipe lagoon underdrain located in the fine sand layer under the lagoon PVC liner is plumbed to the drainage structures to relieve groundwater pressure on the liner when the lagoon is drained.

The typical operating depth of the lagoons is 10 feet, which corresponds to a water surface elevation of about 24 feet based on NAVD 88 (21 feet based on the datum used in the 1996 hydraulic profile). Emergency spillways in the dikes between lagoons are at an elevation of about 28.5 feet NAVD 88 (25.5 feet based on the datum used in the 1996 hydraulic profile). The upstream invert elevation of the headworks overflow channels that were added during the near-term improvements project in 2012 is about 29.2 feet NAVD 88. Therefore, flow would pass over the spillways before backing up into the headworks overflow channels. The headworks overflow channels were added to allow flow to pass directly into Lagoon 1 without overflowing the headworks structure if the screw pumps fail.

At the effluent end of partially mixed Lagoons 3 and 4, outlet chambers can convey flow from the lagoons into an effluent control structure. Each outlet chamber houses one 72-by-24-inch weir gate and two canal gates. Typically, the outlet structure in Lagoon 4 serves as the final outlet, but the outlet in Lagoon 3 can be used, if necessary, by manipulating the canal gates in the inlet/outlet chambers of Lagoons 3 and 4.

The essential function of the DPMC treatment process is to biologically remove the biodegradable organic matter ($CBOD_5$) from the wastewater. The aeration power levels in the completely mixed lagoon are such that settleable solids are generally maintained in suspension and adequate DO concentrations are provided to the bacteria to sustain their metabolic activities. As the wastewater is mixed, bacteria that are in suspension within the mixture decompose the biodegradable organic matter in the wastewater and convert it into either low-energy compounds necessary for growth (respiration) or new cells (growth), with byproducts of Ammonia-N (NH_3), water (H_2O), and carbon dioxide (CO_2). This process removes most of the soluble $CBOD_5$ concentration.

The aeration power intensities in the partially mixed lagoons (Lagoons 2, 3, and 4) are such that only the non-settleable fraction of the suspended solids is maintained in suspension. These settling lagoons are dedicated to (1) gravity sedimentation of the settleable suspended solids, (2) biological stabilization of the settled solids (i.e., digestion), (3) sludge storage, and (4) additional removal of $CBOD_5$ remaining in the effluent from the first lagoon. Dividing the settling volume into three equally sized lagoons in series further enhances the removal of $CBOD_5$ and capture of settleable solids.

The main purpose of the SFF media is to enhance ammonia-N removal in the WWTP by increasing the solids retention time (SRT) of microorganisms in the process to allow nitrifying bacteria to proliferate. Although nitrifying bacteria can be free-swimming, they are typically more successful at establishing a stable population and achieving the desired level of treatment when attached to a medium. The population of free-swimming nitrifying bacteria generally declines under environmental stress, such as abnormal environmental conditions, limited substrate availability, and presence of predators. Limited substrate is a common stressor, since ammonia-N concentrations are low when the process is achieving complete or near complete nitrification. Also, ammonia-N concentrations are very dilute in the lagoons, given the large volume and high I/I during storm events. Unlike a conventional activated sludge facility, lagoons do not contain a large mass of solids to which the nitrifying bacteria can attach. Nor can the SRT in the process be controlled with return of settled activated sludge. The knitted polyester web attached on the SFF module provides surfaces where nitrifying and heterotrophic bacteria can attach and form a biofilm and thus increase the overall SRT.

Organic material, ammonia-N, and other nutrients are removed as wastewater flows past the biofilm, and microorganisms use these substances for growth. By attaching to the media, a relatively high inventory of microorganisms is maintained within the process, rather than most microorganisms washing out with the effluent or settling in the lagoons. The higher concentration of biomass not only benefits nitrification, but also increases biological treatment capacity to improve $CBOD_5$ removal, particularly during storm events when the HRT in the lagoons is reduced significantly. Excess biological solids that accumulate on the media are sloughed off during routine air scouring and settle in the lagoons, where they are digested.

The SFF media blowers provide supplemental air for bacterial growth on the SFF media and scouring of the media to remove excess biomass and strip away trash and debris that may collect on the media.

The supplemental alkalinity dosing system is designed to maintain optimal pH through the system for nitrifying bacteria and reduce the potential for nitrite lock in the chlorine disinfection process. Nitrifying bacteria are more sensitive to pH than typical heterotrophic bacteria used for $CBOD_5$ oxidation. Alkalinity consumption that occurs during nitrification can cause the pH to drop and inhibit nitrification if insufficient alkalinity is present for buffering. Nitrifying bacteria achieve optimal performance at a pH between 7.5 and 8, but may function adequately at a pH as low as 6.5. Typically, nitrification is completely inhibited at a pH below 6.

Capacity

An overview of the design and projected capacities of the lagoons is show in Appendix G. Additionally, detailed calculations of lagoon capacities are provided in Appendix H. Table 9-3 provides a capacity summary for complete-mixed cell Lagoon 1. The HRT in Lagoon 1 varies over time but stays in an acceptable range. The aeration energy required for mixing and oxidation of $CBOD_5$ in Lagoon 1 is projected to be sufficient with the existing aerators through 2036. Although the floating surface aerators in Lagoons 2, 3 and 4 are typically not used currently, it is conservatively estimated that there will be insufficient aeration energy in Lagoons 2, 3 and 4 to meet the remaining demand for nitrification by 2036 even with those aerators running, as described below. Planned upgrades and replacement of the existing surface aerators will resolve this issue.

The estimated aeration requirements assume an operating dissolved oxygen (DO) level of 2.0 mg/L in the lagoons under worst-case conditions and an existing standard oxygen transfer efficiency (SOTE) for the existing aerators of 1.75 pounds of oxygen per horsepower per hour (lbs/hp/hr). Although the normal target DO level is 3.0 mg/L, allowing a reduced DO level under rare worst-case conditions still provides sufficient residual so as not to be an impediment to effective treatment. Normally, surface aerators of the vintage currently installed would have an SOTE of about 2.0 lbs/hp/hr. A SOTE of 1.75 lbs/hp/hr is used to reflect degradation in performance due to age.

Table 9-3 WWTP Dual-Power Multi-Cellular Aerated Lagoon 1 - Complete-Mixed Cell Sizing Overview

Component	Design	Projected		
		2016	2022	2036
Type	**Complete Mix**	**Complete Mix**	**Complete Mix**	**Complete Mix**
Total Volume, MG	10	10	10	10
HRT Max Month Flow, days	3.6	3.7	3.6	3.1
Minimum HRT @ Max Month Flow	2.5	2.5	2.5	2.5
Complete-Mix Cell Volume Required	7.00	6.83	7.00	8.05
Aeration Sizing for Winter (without nitrification)				
S_0 (Influent max mo. BOD at peak day flow), mg/L	170	120	126	154
Ro_2 (max month oxygen consumption rate), kg/hr	112	77	83	116
Ro_2 (max month oxygen consumption rate), lbs/hr	246	169	183	256
Aeration Sizing for Summer (with nitrification)				
Influent TKN load, kg/h (based on max month load)	8.82	6.84	7.37	10.39
Oxygen demand for nitrification, kg O2/kg N	4.6	4.6	4.6	4.6
Operating oxygen concentration, mg/L	2.0	2.0	2.0	2.0
Existing surface aerator SOTE, lbs/hp/hr	1.75	1.75	1.75	1.75
Achievable % nitrification in complete-mix cell	23%	100%	100%	10%
Ro_2 (max month oxygen consumption rate), kg/hr	112	99	99	114
Ro_2 (max month oxygen consumption rate), lbs/hr	246	218	217	252
Total Required Aeration Energy, HP	270	270	270	270

Table 9-4 provides a capacity summary for partial-mixed cell Lagoons 2, 3 and 4. Details of the design and projected capacities of Lagoons 1, 2, 3, and 4 are provided in Appendix H.

Table 9-4 WWTP Dual-Power Multi-Cellular Aerated Lagoon 2 / 3 / 4 - Sizing Overview

Component	Design	Projected		
		2016	2022	2036
Type	**Partial Mix**	**Partial Mix**	**Partial Mix**	**Partial Mix**
Total Volume, MG	10.5	10.5	10.5	10.5
HRT Max Month Flow, days	3.8	3.9	3.8	3.3
Min. Surface Area Required for Overflow, sf	132,000	110,700	108,000	125,600
Aeration Sizing				
Ro2 (max month oxygen consumption rate), kg/hr each	11.44	11.44	11.44	11.44
Ro2 (max month oxygen consumption rate), lbs/hr each	25.22	25.22	25.22	25.22
Total Benthal Oxygen Demand, lbs/hr	76	76	76	76
Remaining Nitrification Oxygen Demand from Lagoon 1, lbs/hr	90	21	39	104
Total Required Oxygen Demand, lbs/hr	145	76	76	171
Actual Oxygen Transfer from Aerators, lbs/hr	67	67	67	67
Actual Oxygen Transfer from Blowers, lbs/hr	84	84	84	84
Total Available Oxygen Transfer for Partial-Mixed Cells, lbs/hr	151	151	151	151
Oxygen Supply Deficiency for Partial-Mixed Cells, lbs/hr	0	0	0	20
SFF Media				
Unused Surface Area Capacity @ Max Month Load, sf	58,217	88,217	80,217	34,503
Available Add'l BOD Removal Capacity, ppd	1,455	2,205	2,005	863

As mentioned above, the remaining nitrification oxygen demand and benthal oxygen demand in Lagoons 2, 3 and 4 exceed the available oxygen transfer rate of the current system by 2036. This deficiency will be addressed by upgrading and replacing the existing surface aerators in Lagoon 1, which will provide more air in Lagoon 1 to reduce the remaining nitrification demand carried over into Lagoons 2, 3 and 4. The HRT in Lagoons 2, 3 and 4 varies over time but stays in an acceptable range.

9.4.6 Effluent Filters

Description

The existing filtration system at the WWTP consists of continuous backwash, upflow, deep bed sand filters, a filter feed pump station containing two filter feed pumps, chemical feed system for polymer, and a reject pump station containing two reject pumps. Currently, only polymer is dosed for flocculation prior to filtration.

The original purpose of the filtration system was to assist with TSS removal to achieve NPDES permit limits, especially for flows above 2.0 MGD. Historically, the only violations of TSS concentration in the last 5+ years occurred in November 2012 during the construction period for the near-term improvements and in December 2013 when lower than normal temperatures caused significant lagoon turnover events and resuspension of settled solids caused prior to the lagoon biosolids having been dredged and removed. Currently, the filters treat only a portion of the effluent flow to provide additional TSS and $CBOD_5$ removal year-round by improving removal of solids, including algae.

The filtration system was not operated immediately after installation due to electrical issues preventing proper operation. The electrical problems were corrected and the filters placed in service in the autumn of 2010. Because usage hours on the various components are relatively low for the age of the system, it remains in generally good condition.

The existing sand filters are designed with 80-inch-deep sand beds instead of the typical 60-inch depth. The additional depth was likely intended to compensate for algal blooms that are common in lagoon wastewater plants. Currently, the WWTP utilizes ultrasonic transducers to help control algal blooms, and that practice has been largely successful.

Capacity

The effluent filters are currently run to provide additional $CBOD_5$ removal to comply with the stringent TMDL limits for NBOD+CBOD during the critical summer months and the TMDL limits on TSS and $CBOD_5$ for the rest of the year. According to the original design documents, the filters were designed for a hydraulic loading of 2.8 gallons per minute per square foot (gpm/sf) of filter surface area, which yields a design capacity of 0.8 MGD. These filters can typically be loaded at rates of up to 5 gpm/sf. With the recent replacement of a high headloss static mixer, the output of the filter feed pumps has increased to about 1.4 MGD, which allows for a loading rate of about 4.8 gpm/sf. Appendix G presents the design, existing and projected capacity of the effluent sand filter and pumps.

Because of higher influent flows during the winter months, lagoon turnover events that resuspend settled solids during very cold weather, slower chemical reaction rates and higher viscosity at lower temperatures, the performance of the existing effluent filters declines during the winter months. To improve TSS removal during the winter months, the existing filters need to be upgraded and expanded to enhance performance and allow filtration of more of the effluent flow. See Chapter 11 for discussion and evaluation of alternatives to upgrade and expand the effluent filtration process.

9.4.7 Chlorination

Description

Disinfection is a process that deactivates or destroys microorganisms remaining in the effluent. The current NPDES permit uses fecal coliform as an indicator organism to verify that sufficient disinfection has occurred. Chlorination using a chlorine gas injection system is the existing disinfection process. When chlorine gas dissolves in water, it produces hypochlorous acid. If ammonia-N is present in the water, the chlorine reacts with the ammonia-N and produces chloramines. Both hypochlorous acid and chloramines are oxidants that can deactivate or destroy bacteria in the water.

The existing chlorination process consists of two pairs of 150-pound chlorine gas cylinders with a scale, a flow-paced chlorinator with a maximum dose capacity of 100 ppd, a manually operated chlorinator with a maximum dose capacity of 100 ppd, a chlorine mixer, a chlorine contact tank, a wet scrubber to treat chlorine gas leaks, chlorine gas leak detectors, a chlorine residual analyzer, and associated piping, tubing and valves. The chlorine feed system is located in the control room of the Chlorination Building, and the gas cylinders are stored in the storage room adjacent to the control room.

The chlorinator withdraws chlorine gas from gas cylinders under vacuum conditions, which is then conveyed to a mixer located in the chlorination manhole. The effluent is mixed rapidly with the chlorine gas in the manhole and then enters the chlorine contact tank. The chlorine contact tank allows the chlorinated effluent to pass through plug-flow channels, providing sufficient contact time with the chlorine to reduce the fecal coliform count.

The chlorine contact tank is a two-chambered tank equipped with isolation valves and drain valves and a maximum volume of 138,125 gallons. The actual volume varies based on flow (the effluent V-notch weir controls the water surface elevation in the contact chambers based on flow) and river elevation (at very high river levels the V-notch weir will become submerged and the water level in the contact chambers will rise and increase overall contact time). Each chamber contains baffling to promote plug-flow and minimize short circuiting to provide the hydraulic retention time (HRT) needed for adequate disinfection. Normally, flow is divided equally among the chambers before recombining prior to discharge. A V-notch weir measures the effluent flow at the end of the chlorine contact tank prior to discharge to the outfall.

Capacity

The disinfection process must have sufficient capacity to handle peak flows, because all wastewater must be disinfected year-round to comply with NPDES permit limits. The chlorination system was originally designed to use chloramines for disinfection, with no need for nitrification. Since the treatment plant began to provide nitrification during the critical dry weather months, some significant increases in chlorine dosage had occurred, particularly during seasonal periods when the lagoons are transitioning from nearly full nitrification in the critical dry weather months to little or no nitrification during the wet weather months. However, with addition of supplemental alkalinity to avoid nitrite-lock and addition of automated dose control following the near-term improvements, the chlorine dose requirements have stabilized. A summary of the chlorine disinfection capacity evaluation is provided in Table 9-5 with details provided in Appendix G.

Table 9-5 WWTP Chlorine Contact Basin Capacity Overview

Component	Design	Projected		
		Existing (2016)	2022	2036
Chlorine Contact Basin				
Number of Chambers	2	2	2	2
Number of Chambers in Use	2	2	2	2
Total Contact Volume, gal	126,400	125,300	125,300	126,100
HRT (contact time), min				
at annual avg. flow	90	139	129	94
at max. month flow	66	67	65	57
at peak day flow	19	22	23	20

Though the WWTP normally uses both of the chlorine contact basin chambers, calculations show that the plant would only need to utilize one in order to attain the necessary HRT for the average annual flow until after 2022. However, both chambers are needed to meet the requirements for max month and peak day flows. Although the contact time at the projected 2036 maximum month flow is just below the target of 60 minutes, the dose can be increased slightly to provide the same effective disinfection, which is a product of the contact time and chlorine concentration. Additionally, the effluent V-notch weir could be raised to increase the depth and resulting contact time. Furthermore, if the City implements use of peracetic acid as discussed in Chapter 11, the required contact time for this method of disinfection is significantly less. As a result, the capacity of the existing chlorine contact tank would be substantially increased. For these reasons, there is not a capacity deficiency with the existing chlorine contact tank.

The seasonal difference in chlorine dose with and without nitrification can be explained by breakpoint chlorination. When sufficient ammonia-N is present in the effluent, chlorine combines with ammonia-N and produces monochloramine. While monochloramine is not as strong an oxidant as free chlorine, it persists throughout the contact chamber. When chlorine is combined with a highly nitrified effluent, chlorine destroys any small amount of remaining ammonia-N through breakpoint chlorination and then combines with organic nitrogen and forms organo-chloramines which have essentially no disinfecting power. The free chlorine that remains tends to be consumed by other organics in the water as it bleaches the water. Thus, more chlorine must be dosed to achieve a measurable free chlorine residual. The less reactive chloramine that is formed when higher effluent ammonia-N concentrations exist is not consumed in this manner and thus requires a lower chlorine dose, because there are sufficient chloramines for disinfection and a measurable free chlorine residual is not required. The different chlorine doses and demands are listed in Table 9-6.

Table 9-6 WWTP Chlorination Capacity Overview

Component	Design	Projected		
		Existing	2022	2036
Chlorination				
Type	**Chlorine (Cl_2) Gas**	**Chlorine (Cl_2) Gas**	**Chlorine (Cl_2) Gas**	**Chlorine (Cl_2) Gas**
Chlorine Cylinder Capacity, ppd Cl_2	160	160	160	160
Total Chlorinator Capacity, ppd Cl_2	200	200	200	200
Peak Chlorine Dosage with Nitrification (Jul - Oct), mg Cl_2/L	3.0	3.0	3.0	3.0
Peak Chlorine Dosage w/o Nitrification (Nov - Jun), mg/Cl2/L	1.5	1.5	1.5	1.5
Avg. Annual Chlorine Demand with Nitrification, ppd	51	33	35	49
Max Month Demand with Nitrification, ppd	70	68	70	81
Peak Day Demand w/o Nitrification, ppd	124	104	101	118

Deficiencies

The existing chlorine contact tank can provide a contact time of at least 20 minutes, as recommended by the Orange Book, at peak day flows up to about 9.4 MGD, which is approximately equal to the projected 2036 peak day flow at the end of the planning horizon. Additionally, the length-to-width ratio exceeds the minimum recommended ratio of 40:1. As mentioned previously, actuated gates have been installed at the outlets from the chlorine contact chambers, which can help to maximize flow attenuation in the lagoons and minimize flow to the chlorine contact tank to improve overall disinfection efficiency.

Although the chlorine disinfection system has adequate capacity for the planning horizon, the City wants to change the method of disinfection to improve safety, replace aging equipment and comply with current codes. See Chapter 11 for discussion and evaluation of alternatives to replace the chlorine disinfection process. Additionally, the chlorine contact tank requires minor structural repairs and recoating of the interior concrete surfaces.

9.4.8 Dechlorination

Description

Because the chlorinated effluent may contain residual chlorine in excess of the permitted discharge limit, the effluent is dechlorinated with sulfur dioxide (SO_2) gas prior to discharge to the Snohomish River. Without dechlorination, the chlorine residual could be toxic to aquatic life in the receiving water body. The SO_2 gas dissolves in water and produces sulfite ions. The sulfite ion is a strong reducing agent and reduces the hypochlorous acid or chloramines to chloride. Theoretically, 1 milligram per liter (mg/L) of SO_2 is required to reduce 1 mg/L of residual chlorine.

The existing dechlorination process consists of two pairs of 150-pound SO_2 gas cylinders with a scale, two flow-paced sulfonators with a maximum dose capacity of 100 ppd each (one for backup), SO_2 mixer, SO_2 gas leak detectors, chlorine residual analyzer, and associated piping, tubing, and valves. The SO_2 feed system is located in the control room where the chlorine gas feed system is located. The SO_2 gas cylinders are located in the storage room with the chlorine gas cylinders, next to the control room.

The sulfonator withdraws SO_2 gas from gas cylinders under vacuum conditions, which is then conveyed to a mixer located in the dechlorination zone at the end of the chlorine contact tank. The chlorinated effluent reacts with the SO_2 almost instantly to dechlorinate the effluent. The final effluent is then discharged to the Snohomish River through the outfall. A 30-inch flap gate in the nearby outfall manhole prevents backflow from the river to the WWTP.

The sulfonator withdraws SO_2 gas from a gas cylinder and delivers it to the mixer located in the dechlorination chamber at the end of the chlorine contact tank under vacuum. The chlorinated effluent is mixed with the SO_2 solution almost instantly. The final effluent is then discharged to the Snohomish River through the outfall. A 30-inch flap gate at the chamber prevents backflow from the river to the WWTP.

Capacity

Automatic controls based on residual chlorine concentration were added in 2014. Two chlorine residual analyzers are used for automatic control. One residual analyzer measures the residual chlorine near the beginning of the chlorine contact tank to control chlorine dosing, and the other measures the residual chlorine in the dechlorination zone to control SO_2 dosing. This method of automatic control is very similar to the method the WWTP staff used to adjust the dose manually, but the adjustments are now made automatically every 30 seconds.

The total of four online SO_2 cylinders meet the required demand through the planning horizon with the SO_2 dose adjusted in accordance with the residual chlorine, conservatively assuming the residual chlorine is maintained at less than 2 mg/L. No significant improvements to the dechlorination system are necessary or planned during the planning horizon. Should the City implement peracetic acid for disinfection in the future, the SO_2 dosing system could be retained to quench residual peracetic acid in excess of the 1 mg/L discharge limit. However, considering the consistently low peracetic acid residuals observed during testing, the City will also consider removal of the sulfur dioxide system and instead including provisions for dosing liquid sodium bisulfite.

Table 9-7 WWTP Dechlorination Capacity Overview

Component	Design	Projected		
		Existing	2022	2036
Dechlorination				
Type	**Sulfur Dioxide Gas**	**Sulfur Dioxide Gas**	**Sulfur Dioxide Gas**	**Sulfur Dioxide Gas**
Sulfur Dioxide (SO_2) Cylinder Capacity, ppd SO_2	160	160	160	160
Total Sulfonator Capacity, ppd SO_2	200	200	200	200
Typical SO_2 Dosage, mg SO_2/L	1	1	1	1
Avg. Annual SO_2 Demand, ppd	17	11	12	16
Max Month SO_2 Demand, ppd	23	23	23	27
Peak Day SO_2 Demand, ppd	83	69	68	79

9.4.9 Outfall

The 30-inch outfall terminates in the Snohomish River with a four-port diffuser at a depth of approximately -20.60 feet using the hydraulic profile datum from the 1996 record drawings (approximately -17.6 feet NAVD 88). The latest mixing zone study was performed in 2001 by Cosmopolitan Engineering using a MMF of 2.8 MGD, which is consistent with the current NPDES permit. Because projected flows in 2036 are greater than 2.8 MGD, a new mixing zone study will need to be performed prior to increasing the rated capacity of the WWTP.

As identified in Section 9.3 above, there are no hydraulic capacity issues with the current outfall. At very high river levels, gravity discharge to the outfall may be reduced, but the City now has controls in place to hold back flow in the lagoons if necessary due to high river levels.

Based on the mixing zone parameters developed in the 2001 mixing zone study, the recent reasonable potential analysis conducted by Ecology to develop the new NPDES permit did not identify reasonable potential to exceed water quality requirements for any constituents that are were not already limited under the previous permit. Ecology is requiring a new mixing zone study to be completed by August 1, 2021. This will also be required as part of the re-rating study discussed below. This study will either confirm or redefine the mixing zone and reconsider the reasonable potential analysis based on any changes to the mixing zone parameters and assumptions for ambient and discharge water quality and quantity. Because the proposed re-rated capacity is not substantially higher than the current rated capacity, it is not anticipated that there will be reasonable potential for any new constituents to exceed the water quality parameters.

9.5 Electrical Service

9.5.1 Electrical Supply

Supply of electricity to the Snohomish WWTP is provided from a 3-phase, 12,470-volt (V) overhead line extending onto the site from River View Road. There are two services, one is at the northern end of the site at the Laboratory/Office Building and the other is toward the southern end of the site near the Chlorination Building. The primary conductors for each service are routed underground in PVC conduit from the last overhead power pole to a pad-mounted transformer, and then from the transformer to a current transformer compartment and service meter at each building.

There are two sets of motor control center (MCC) line-ups, one (MCC1) in the Laboratory/Office Building and the other (MCC2) in the Chlorination Building. The normal MCCs (NMCCs) serve loads that do not require standby power, and the emergency MCCs (EMCCs) serve loads that are connected to standby power. Power to the Laboratory/Office Building enters a switchboard that feeds both NMCC1 and EMCC1. EMCC1 is also connected to an existing emergency generator located in a separate building near the Laboratory/Office Building. Power to the Chlorination Building is routed from the service meter to Normal Motor Control Center 2 (NMCC2), which feeds power to Emergency Motor Control Center 2 (EMCC2). Currently, EMCC2 can also receive power from the existing emergency generator through Emergency Motor Control Center 1 (EMCC1), located in the Laboratory/Office Building. However, the City is in the process of adding a second emergency generator at the south end of the plant that will be capable of powering both EMCC2 and NMCC2. The new generator will be located on a concrete pad in a skintight sound attenuating enclosure near the Chlorination Building. The connection of EMCC2 to EMCC1 and the existing generator will remain as a backup.

NMCC2 was recently replaced during construction of the near-term improvements to provide power to the new Blower Building associated with the SFF media system and to make the entire NMCC2 service-entrance rated. NMCC2 has a 600-amp main bus and powers the filter feed pumps, the filter air compressor, the filter reject pumps, and the Blower Building main distribution panel board, which feeds power to the 75-horsepower blowers and motorized valves for aeration of the SFF media modules. Additionally, the two main programmable logic controllers, one at the Chlorination Building and one at the Laboratory/Office Building were replaced and the SCADA system upgraded in 2014.

There are no deficiencies with the electrical or control system. However, EMCC1, NMCC1 and EMCC2 are old and will need to be replaced within the planning horizon.

9.5.2 Existing Emergency Generators

The WWTP has two existing emergency generators powered by diesel fuel. One is located in a small building directly east of the Laboratory/Office Building. This emergency generator serves the emergency side of the automatic transfer switch (ATS) in the Laboratory/Office Building Electrical Room and will power EMCC1 in the event of a loss of utility power. The other generator is located in an enclosure outdoors in between the Blower Building and Chlorination Building. This emergency generator was recently installed in 2018 and serves the emergency side of the ATS at the Chlorination Building, which will power both EMCC2 and NMCC2 in the Chlorination Building during a loss of utility power. Additionally, EMCC2 has an emergency feeder from EMCC1 in the Laboratory/Office Building as an additional backup.

Diesel fuel to the emergency generator near the Laboratory/Office Building is supplied from an outdoor aboveground dual-containment 1,000-gallon fuel tank. This amount of fuel should provide approximately 48 hours of continuous operation of the generator at full load. The emergency generator near the Chlorination Building has a sub-base fuel tank that stores fuel for about 24 hours of continuous operation at full load. Both fuel tanks have a low fuel indication which alerts the WWTP staff when the tank should be refilled. There is also a critical low fuel alarm which shuts down the generator prior to running out of fuel, so that the fuel piping does not have to be re-primed.

The emergency generator located near the Chlorination Building is rated for 275 kilowatts (kW) and capable of powering all loads connected to both EMCC2 and NMCC2. The emergency generator near the Laboratory/Office Building is rated for 350 kW at 0.8 power factor and 438 kVA, based on drawing up to 526 amperes at 480V, 3-phase. This capacity is sufficient to power the following loads connected to EMCC1:

- One 25-horsepower screw pump
- One 100-horsepower screw pump
- Influent screen and associated washer/compactor
- Twelve of eighteen 15-horsepower surface aerators in Lagoon No. 1
- All three 7.5-horsepower surface aerators in Lagoon No. 2

All other equipment not powered by the two emergency generators is considered non-essential and can remain idle during a loss of power without serious detrimental impact to the operation or performance of the WWTP. However, if the flow rate is so high that two 100-horsepower screw pumps need to run together, the control system will shut down surface aerators in Lagoon 1 as necessary to avoid overloading the generator. A power monitoring unit on the main bus communicates with the programmable logic controller (PLC). If the load exceeds the output capacity of the generator, the PLC will shut down individual aerators in Lagoon 1, until the load is below the rated output of the generator. A 30-second delay is provided between the shutdowns of each aerator. As a result, the control system will always preferentially power the systems necessary to convey, screen and disinfect the wastewater. This is consistent with the emergency power requirements for a Reliability Class II rated WWTP as identified in the Orange Book Reliability and Redundancy Design Guidelines. The generators will provide emergency power to all vital components (i.e., pumping, screening and disinfection) and critical lighting and ventilation, as well as powering enough of the surface aerators to maintain the biota.

When equipment is restarting after the source of power is transferred, either to generator power or utility power, the starting of equipment is staggered to keep the starting load to a minimum and avoid overwhelming the generators. The screw pumps and chlorine and SO_2 mixers are started first to avoid flooding the headworks structure and ensure proper disinfection and dechlorination. Subsequently, the surface aerators are started in sequenced groups. Enough of the aerators need to be in service to maintain adequate aeration and mixing to keep the lagoon process viable during an extended loss of power.

When the WWTP is operating on generator power, the operator has the ability to control the return to utility power to ensure that utility power is stable before transferring back to utility power. There are indication lights on the front of each ATS for indication of which source is available and to which source the switch is connected. A generator exercise clock provided in each ATS automatically starts the generators and runs them weekly for routine exercising. A switch is provided to choose whether to exercise the generator with or without load.

9.6 Water Service

The City has identified the need to add two air gaps at the WWTP for the purpose of providing cross connection control by physically separating their non-potable water system from the potable water system. This protects the public potable water system from possible contamination and protects the onsite domestic potable water system from possible contamination. According to the Orange Book, "Facility designs must include a pressurized water system supplied through an air gap to facilitate cleaning or flushing of wet wells, dry wells, tanks, basins and equipment." Currently, only reduced pressure backflow preventers are separating the potable and non-potable water systems.

9.7 Biosolids Disposal

The City currently stabilizes and stores sludge in the lagoons. Much of the sludge that is generated is digested in the lagoons, and the rate of sludge accumulation is low. In fact, since the four-stage lagoon process began operation in 1995, sludge removal has only been necessary once and was performed in 2015. The lagoons will likely need to have the biosolids removed again during the planning horizon. The City has a Biosolids Plan separate from this Plan that address biosolids stored in the existing lagoons.

9.8 WWTP Staffing and Testing

A wastewater staffing evaluation report was completed in 2006. The report recommended that City hire an additional full-time employee (FTE) to primarily assist with maintenance activities at the WWTP. Shortly after that recommendation was made, the City hired an additional FTE for the WWTP. As a result, the City has been able to keep up with preventative maintenance activities to keep the plant running efficiently and reliably.

As discussed previously, improvements were made in 2012 that upgraded and replaced the headworks screens, added a supplemental alkalinity dosing system, added a SFF media system with associated aeration blowers, and updated the SCADA system. The City also upgraded its PLCs and programmed control strategies in 2015 to increase reliability and level of control. Although some of these improvements added more equipment to be operated and maintained, they also simplified or eliminated the need for other operational activities. For example, prior to addition a supplemental alkalinity and the SFF media system, the WWTP would periodically experience issues with nitrite-lock due to incomplete nitrification in the lagoons, which required greater monitoring and attention to the chlorine disinfection and dechlorination processes. Additionally, the City routinely seeded the lagoons with nitrifying bacteria to try and maintain a higher level of nitrification. Because these improvements made in 2012 resolve those and other issues and increased the level of automation and control, the City is able to successfully operate and maintain the WWTP with the same level of staffing.

Improvements to the WWTP identified herein and evaluated in Chapter 11 are not anticipated to increase the required staffing level or require additional testing and laboratory work. Replacement of the surface aerators in Lagoon 1 will improve their reliability and reduce maintenance. Upgrade and expansion of the effluent filters will involve a small increase in the amount of equipment to be maintained, but it will also improve the overall performance and efficiency of the process and replace some aging equipment. Upgrades to the existing disinfection system will simplify the process, enhance safety, increase reliability and also replace some aging equipment. For these reasons, there are no recommendations for the City to change the level of staffing at the WWTP.

9.9 Water Reclamation

In accordance with RCW 90.48.112, engineering reports for the construction of new sewer systems and facilities, or for improvements to existing sewer systems and facilities, submitted under RCW 90.48.110 must include consideration of opportunities for the use of reclaimed water.

As mentioned previously, the City has no significant industrial users in its service area, so there are no opportunities for industrial uses of reclaimed water. Furthermore, areas of parks, open space and urban horticulture in the City are clustered in the north central portion of the City around Blackmans Lake about 2 miles from the WWTP and in the southwest corner of the City about 1.5 miles from the WWTP. Both locations would require an extensive conveyance and distribution system. Applications to parks and open spaces with public access would require Class A reclaimed water. Application to urban horticulture could utilize Class B reclaimed water, assuming public access is controlled.

Class A reclaimed water requires removal of total nitrogen to less than 10 mg/L on a monthly average, filtration to less than 2 Nephelometric Turbidity Units (NTU) on a monthly average, and disinfection to total coliform counts less than 2.2 cfu/100 ml on a 7-day median and 4-log virus removal. The WWTP does not have processes in place to remove nitrogen. The current effluent filtration process and the proposed filtration improvements discussed in Chapter 11 cannot provide the level of treatment necessary for Class A reclaimed water, particularly given the presence of algae. Similarly, the current disinfection process and the proposed improvements to the disinfection process discussed in Chapter 11 are not capable of disinfecting to the level required for producing Class A reclaimed water. A much greater capital investment in improvements to the WWTP would be required to produce Class A reclaimed water.

Class B reclaimed water requires disinfection to total coliform counts less than 23 cfu/100 ml on a 7-day median. Even this less stringent level of disinfection could be difficult to reliably achieve without providing more contact time or substantially increasing the required dose. Just the cost of increasing the dose for disinfection is likely to exceed any potential revenue or offset cost if reclaimed water were used in the areas zoned for urban horticulture, which does not take into account the multiple millions of dollars required to convey it to those potential users.

Because the City has sources of supply already available to meet its projected water demands and the cost of providing reclaimed water would far exceed the small potential economic benefit, the City has no plans to produce and distribute reclaimed water.

9.10 WWTP Re-Rating

9.10.1 Required WWTP Capacity Improvements

Based on the hydraulic capacity evaluation presented herein, there are no hydraulic limitations for conveyance of projected flows through the WWTP. Based on the process capacity analysis presented herein, there are two process capacity limitations that need to be addressed to provide adequate treatment of the projected flows and loads. These capacity limitations will be addressed by upgrading and replacing the floating surface aerators in Lagoon 1 and upgrading and expanding the effluent filters. Other identified WWTP improvements to replace the existing chlorine disinfection system, add a second emergency generator, replace MCCs, retrofit VFDs for the influent screw pumps and future removal of biosolids are for improved or continued

operations and maintenance (O&M) and replacement of obsolete equipment and do not impact process capacities.

The City is requesting that Ecology consider a re-rating of the WWTP pending the City completing the upgrade and replacement of the floating surface aerators in Lagoon 1 and upgrade and expansion of the effluent filters, as recommended in Chapter 11. The City is pursuing re-rating of the WWTP because the projected maximum month flow and maximum month BOD_5 load for 2036 exceeds the current permitted capacities. The projected maximum month TSS load for 2036 does not exceed the current permitted capacity.

9.10.2 Determination of Re-Rated WWTP Capacity

A comparison of the existing process capacity and improved process capacity for the lagoons is included at the end of Appendix G. These calculations show that with replacement of the existing floating surface aerators in Lagoon 1 the capacity of the lagoons is increased to 3.30 MGD at an estimated correlating maximum month BOD_5 load of 4,233 ppd and maximum month TKN load of 564 ppd. These maximum month loads were estimated by increasing the projected 2036 loads proportional to the amount that the proposed re-rated flow capacity of 3.30 MGD exceeds the projected 2036 maximum month flow of 3.22 MGD. Detailed calculations of lagoon capacity under these flow and load conditions are included in Appendix H. This capacity was determined by increasing the maximum month flow and proportionally increasing the maximum month BOD_5 and TKN loads until the total oxygen demand (inclusive of oxidation, nitrification and benthal demands) equaled the total oxygen supplied by the new aerators in Lagoon 1, the existing aerators in Lagoons 2, 3 and 4 and the SFF media blowers. The calculated oxygen demand includes the following conservative assumptions:

- Simultaneous occurrence of maximum month flow and loads
- Nitrification of all influent TKN
- Oxidation of all influent BOD5
- Diurnal peaking factor of 1.5
- Highest historical monthly average temperature for reduced oxygen transfer efficiency
- Target dissolved oxygen residual of 2.0 mg/L under these worst-case conditions
- Degradation of standard oxygen transfer efficiency (SOTE) for the new aerators to 2.0 lbs/hp/hr during the planning horizon, compared to the manufacturer's reported value of 3.0 lbs/hp/hr
- Further reduction in the SOTE of the existing aerators in Lagoons 2, 3 and 4 from 1.75 to 1.50 lbs/hp/hr during the planning horizon

Although the lagoons have sufficient volume to treat a higher BOD_5 load, not to mention SFF media capacity not used for nitrification that would be available for treatment of BOD_5, oxygen supply is the limiting factor for BOD_5 and TKN loading.

As described in Chapter 11, the recommended upgrades to the effluent filters would increase the filtration capacity from 1.4 MGD to at least 2.8 MGD. It is not necessary for the effluent filters to filter the entire maximum month flow, but it is necessary to filter a greater proportion of the effluent flow. At the current filtration capacity, a little over half of the current maximum month flow can be filtered at most. This has resulted in the WWTP not meeting the current NPDES permit limit for 85% removal of TSS, also due in part to the dilute influent concentrations from the combined sewer system during high flows. Although the new NPDES permit will have a lower threshold for percent removal during the wet weather months due to the dilute nature of influent wastewater, it is still recommended that additional effluent filtration

capacity be provided to ensure adequate removal. With a capacity of 2.8 MGD, the effluent filters would filter 85% of a re-rated maximum month flow of 3.30 MGD. This should provide sufficient removal to maintain permit compliance, such that effluent filter capacity beyond 2.8 MGD is not a necessity for achieving adequate treatment. Furthermore, the existing layout of the effluent filters and accommodations for flow distribution and expansion of the filters would require replacing the existing filters or constructing new filter cells to gain capacity beyond 2.8 MGD. As this would be prohibitively expensive and is not deemed necessary to maintain compliance with the NPDES permit, it is recommended that the capacity of the effluent filters be doubled by adding two more filters on the opposite side of the filter influent channel, as recommended in Chapter 11.

9.10.3 WWTP Re-Rating Procedure

Ecology has an established procedure for approving a WWTP re-rating, which is documented in Section G1-5.4.2 of the "Orange Book". A successful re-rating must progress along the following basic steps as outlined below.

Scope of Proposed Re-Rating

Requirement: A scope for the proposed re-rating effort must be developed in consultation with Ecology. The scope must define the theoretical basis for the re-rating effort and whether the request qualifies as a standard or non-standard re-rating. A project scope for a standard re-rating request must identify the reasons the entity believes the process/component under review is substantially similar to units described in Ecology's "Orange Book" or other recognized design guidance documents. The scope must also identify methods to be used to validate the claim of similarity. A non-standard re-rating must demonstrate that the facility or unit process will reliably and consistently operate at the new higher design capacity. Ecology will only consider non-standard facility re-rating requests for facilities that can provide full-scale performance data and stress testing.

Proposed: The City proposes the scope as defined herein for the re-rating effort. The City proposes to use the calculations and recommendations provided in this Plan as the basis for re-rating the WWTP. The City acknowledges that re-rating of the WWTP would be pending completion of the recommended upgrade and replacement of the surface aerators in Lagoon 1 and recommended upgrade and expansion of the effluent filters, as identified in this Plan. The City also acknowledges that they would need to complete an updated mixing zone study and anti-degradation analysis to demonstrate no adverse impacts to the receiving water. The City requests Ecology identify any additional requirements that are necessary components of the scope for the re-rating effort.

The City has identified this re-rating effort as a standard re-rating. Ecology's "Orange Book" recognizes aerated lagoon treatment systems as a standard treatment process. Although the "Orange Book" does not specifically reference the DPMC aerated lagoon system, it acknowledges that "Other design models are in use for design of wastewater stabilization lagoons. These models may increase the level of treatment if improvements such as increased mixing intensity, baffling, lagoons in series, mechanical aeration, and effluent filtration and polishing are used." The two most common wastewater treatment references and design guides do specifically acknowledge the DPMC aerated lagoon system. The Water Environment Federations Manual of Practice No. 8 "Design of Municipal Wastewater Treatment Plants" (2010, 5th Ed.) discusses layout and sizing of DPMC aerated lagoons (also referred to as high-performance aerated pond systems) on pages 18-14 and 18-15 of Volume 2. Metcalf & Eddy's "Wastewater Engineering: Treatment and Reuse" (2002, 4th Ed.) discusses DPMC aerated

lagoon systems (also referred to as dual-powered flow-through lagoon systems) in Section 8-8 (pg. 853). Although there is a more recent 5th Edition of this reference published in 2013, this later edition does not include any significant discussion of any lagoon treatment systems. Both of these references cite work by Linvil Rich as the basis for sizing DPMC aerated lagoon systems. It is the equations developed and modified by Rich that are used to assess the capacity of the City's DPMC aerated lagoon system, as documented in Appendices G and H. Additionally, the 2011 U.S. EPA publication "Principles of Design and Operations of Wastewater Treatment Pond Systems for Plant Operators, Engineers, and Managers" discusses operation and performance of high-performance aerated pond systems (or DPMC systems) and specifically identifies this system as the "Rich design". Because the DPMC aerated lagoon system and associated sizing method used in this Plan to assess its capacity are referenced in industry standard references and by leading entities in the industry, this re-rating effort is considered a standard re-rating.

Engineering Report

Requirement: An engineering report must be prepared that evaluates the technical feasibility of re-rating the facility. The engineering report must include the technical basis for the proposed re-rating, an evaluation of the proposed re-rating on each treatment process, and the evaluation and monitoring proposed to demonstrate performance and reliability of the facility at the re-rated capacity. The engineering report will also need to include a new mixing zone study and anti-degradation analysis for the receiving water.

Proposed: The City proposes that this Plan serve to fulfill the requirements of an engineering report for the proposed re-rating effort. As discussed previously, this report has established the technical basis and proposed re-rating for the WWTP. Regarding evaluation and monitoring to demonstrate performance, the City proposes to estimate the actual oxygen transfer rate of the new surface aerators in Lagoon 1 and confirm hydraulic capacity of the upgraded and expanded effluent filters. Once all aerators in Lagoon 1 are replaced, the City will use influent flow, $CBOD_5$, ammonia and TKN measurements; Lagoon 1 dissolved oxygen measurements; number of Lagoon 1 surface aerators in service; and Lagoon 1 effluent CBOD5, ammonia and TKN measurements to estimate oxygen supply in Lagoon 1, which will be used to estimate the actual oxygen transfer rate. The estimated actual oxygen transfer rate will be compared to the value used for design to confirm the aerators are meeting or exceeding the design value. These measurements will be made twice weekly for at least 6 weeks to collect sufficient data confirm performance of the aerators.

After the effluent filters have been upgraded and expanded, the City will test the filters collectively or individually at their rated capacity for a period of at least 1 week each to confirm the filters can operate at the design hydraulic loading rate. The City will measure total effluent flow, filtered flow, number of filter cells in service, lagoon effluent TSS and filtered effluent TSS for 5 days each week. This data will be used to verify that the measured effluent TSS removal is sufficient such that a weighted average of the TSS for 2.80 MGD of filtered effluent and 0.50 MGD of non-filtered effluent (total flow of 3.30 MGD) is within the NPDES permit limits.

Provisional Approval

Requirement: Once the engineering report is approved and any necessary improvements are completed, Ecology will issue provisional approval, which allows the performance of the re-rated facility to be evaluated. This evaluation period may require additional sampling and testing, as mentioned above. Reports on performance are submitted during the evaluation period.

Proposed: Normally, Ecology would issue a provisional approval after the engineering report is approved and any necessary improvements are constructed. However, in this case, the necessary improvements to achieve the re-rating are planned to be completed before additional capacity is required. The existing surface aerators in Lagoon 1 are old and need to be replaced, even before additional aeration capacity is required. Also, upgrading and expanding the effluent filters is planned to be completed before additional capacity is required because the City would like to benefit earlier from additional filtration capacity and improved filter performance. Therefore, the City may be able to demonstrate performance of these improvements without requiring provisional approval of the re-rated capacity.

Approval to Operate

Requirement: Once the re-rated facility demonstrates consistent and reliable performance and compliance, Ecology will issue approval to operate.

Proposed: The City proposes to demonstrate and document performance of the improvements as indicated herein for review and approval by Ecology, prior to Ecology issuing an approval to operate.

Chapter 10 Regional Wastewater Treatment Alternatives

10.1 History

Under the requirements of WAC 173-240-050 for General Sewer Plans, the City of must assess the feasibility of developing regional wastewater facilities with neighboring communities and industries within 20 miles rather than providing its own treatment facilities. Under a regional alternative, the City would no longer treat its wastewater for discharge through its existing outfall to the Snohomish River, but would convey its wastewater in a pipe (approximately 20 inches in diameter) to a treatment plant operated by another municipality for treatment and discharge. The following municipalities and utility districts are within 20 miles of the City of Snohomish:

- Marysville
- Everett
- Lake Stevens Sewer District
- Mukilteo Water and Wastewater District
- Edmonds
- Arlington
- Tulalip Tribes of Washington
- Duvall
- Lynwood
- Alderwood Water and Sewer District
- Granite Falls
- Sultan
- Monroe
- King County's Brightwater Treatment Plant

To be feasible, the wastewater treatment facilities, including their outfalls, need to have adequate capacity and meet applicable water quality standards now and in the planning horizon described in this Plan. The facilities also need to be cost-effective alternatives (e.g., facilities need to be close enough that the conveyance facilities costs are not substantially higher than other alternatives).

The two closest wastewater treatment facilities that could accept the quantity of flow produced by the City of Snohomish are the Brightwater WWTP in Woodinville and the City of Everett Water Pollution Control Facility (WPCF). Conveyance to nearly all of the other facilities would be substantially more difficult and costly due to distance from the City of Snohomish. Many of the other facilities discharge into the Snohomish River or associated water bodies and would not provide the water quality, habitat, and environmental benefits of a deep-water outfall. Only the Everett and Brightwater facilities could be feasible due to capacity, cost, and other relevant factors.

Alternatives to convey wastewater from the existing Snohomish WWTP to Brightwater and the City of Everett were developed and evaluated in the 2010 Plan (CH_2M Hill, February 2011). The 2010 Plan analyzed the costs and other key considerations of these alternatives. This chapter summarizes the results of that analysis and discusses how through compliance with the agreed orders with Ecology that the City determined it was more cost-effective to continue treatment of their wastewater at their existing WWTP.

10.2 Brightwater Alternative

The Brightwater alternative would involve conveying wastewater from the existing City of Snohomish wastewater lagoons to the Brightwater WWTP in Woodinville.

10.2.1 Plant Capacity and Expansion

The Brightwater WWTP has a current design capacity of 40.9 MGD. King County's Amendment No. 1 to the Facilities Plan for the Brightwater Regional Wastewater Treatment System assumes that this capacity will be needed by the County to serve its projected growth. The Brightwater WWTP is designed to allow future expansions, however, the current design would not accommodate current and projected flows from the City of Snohomish. The Brightwater WWTP would need to be expanded for this alternative to be implemented, which would involve additional design, permitting, and construction activities and their associated costs.

10.2.2 Conveyance

Wastewater would still be conveyed through the City's collection system to the existing City WWTP site. The treatment lagoons would be used as equalization basins to allow flow to be pumped to the Brightwater WWTP at a set rate of about 5 MGD.

The conceptual design for a conveyance system to the Brightwater WWTP would include eight pump stations, a 20-inch-diameter force main, and a 24-inch-diameter gravity sewer pipe. The first pump station would be at the existing City WWTP. The conveyance route would be approximately 55,700 feet long (10.5 miles). This would include approximately 45,000 feet of force main and approximately 10,700 feet of gravity sewer.

Figure 10-1 shows a possible conveyance route to the Brightwater WWTP. The force main would leave the City's WWTP site and would be horizontal directionally drilled beneath the Snohomish River before heading south along State Route 9 (SR 9) to Rees Corner and the intersection of SR 9 and Broadway. The route would turn east at this intersection and continue east and south along Broadway until it turned west where Broadway becomes Maltby Road. The route would continue along Maltby Road to the intersection of Maltby Road and SR 9 at Turner Corner. The route would then turn south along SR 9 and continue to the proposed Brightwater WWTP. Facilities would need to be constructed at the Brightwater WWTP to make the connection to the conveyance line and regulate the incoming flows.

Pump stations would be required to overcome elevation increases over the 10.5-mile route. The system would transition from a 20-inch diameter force main to a 24-inch diameter gravity sewer at the high point at elevation 481 feet. This is along Maltby Road approximately 5,500 feet east of the intersection of State Route 9 and Maltby Road. In addition to crossing the Snohomish River, this route also crosses two streams. The pipe would likely be attached to the bridges crossing the streams.



Figure 10-1 Snohomish to Brightwater Conveyance Route (2010 Plan)

10.2.3 Hydraulic Analysis

Total system head was calculated along the conveyance route considering the proposed flow rate, ground profile, and pipe diameter. System head calculations were developed for several pipe diameters; these resulted in different total system head values, which affected the required number of pump stations. One pump station was included for every 100 feet of total system head. It was determined that the optimal configuration is a 20-inch diameter force main and eight pump stations, which resulted in lower overall pipeline and pump station costs.

10.2.4 Planning-Level Costs

The planning-level opinion of probable construction cost developed in the 2010 Plan for the horizontal directional drilling, pump stations, force main, Snohomish WWTP improvements for flow equalization and gravity sewer to convey wastewater from the existing City of Snohomish WWTP to the Brightwater WWTP site in Woodinville is approximately $39 million. This cost does not include land acquisition, easement costs for pump stations and piping, or costs due to delays associated with permit acquisition. This cost is a Class 5 estimate as defined by the American Association of Cost Engineering with an expected accuracy of +50 percent/-30 percent and is in 2009 dollars. At this planning level, the analysis assumes that the construction and environmental impacts would be similar to the alternative for conveyance to the Everett WPCF, except that the energy impacts and carbon footprint would be greater since pumping would be needed along a route that is twice as long as the Everett alternative.

The above opinion of probable construction cost does not include connection fees to the Brightwater WWTP facility. For the estimated number of equivalent residential units in the City of Snohomish service area at the time of connection, this could amount to an additional cost of about $64 million. Other costs to be considered for this alternative are the operation and maintenance costs associated with the eight pump stations and wastewater treatment and disposal charges from the Brightwater WWTP. These are likely to be higher than conveyance to Everett, because the conveyance line is twice as long as the conveyance to Everett (approximately10 miles vs. 5 miles). The first year of O&M costs are estimated at approximately $1.2 million. In addition, it is assumed the annual service charge levied by King County would be approximately $2.5 million for the first year.

10.2.5 Summary

Conveyance to the Brightwater WWTP would be approximately twice as long as the conveyance to Everett, with greater construction and environmental impacts (such as higher energy use for pumping) and capital and long-term O&M costs.

The total capitalized cost of conveyance to the Brightwater WWTP would likely to exceed $100 million, including approximately $39 million in capital costs to construct the conveyance system and an assumed $64 million in connection and capacity charges. Additionally, the City would incur approximately $3.7 million in annual O&M costs and service charges throughout the planning horizon. Due to the high capital and O&M costs, this alternative is not considered feasible.

10.3 Everett Alternative

This flow transfer alternative involves conveying wastewater from the existing Snohomish WWTP site to Everett for treatment at the Everett WPCF. This alternative would eliminate the Snohomish WWTP discharge to the Snohomish River. Evaluating this alternative includes considering new facilities that would be required at the Snohomish WWTP and Everett WPCF, in addition to the impact of adding Snohomish wastewater flow to the Everett collection system.

10.3.1 Plant Capacity and Expansion

Projected influent flow and loading information was provided to the City of Everett to determine if the Everett WPCF has the capacity to treat wastewater from Snohomish. Based on the evaluation documented in the City of Everett Engineering Report (Carollo, 2009), the Everett WPCF has initial capacity to treat raw Snohomish flows and wasteloads. However, current population estimates require that the trickling filter/solids contact (TF/SC) process at the Everett WPCF be expanded to meet longer term Everett and Snohomish requirements. If Snohomish did not transfer flows to Everett, an expansion of the TF/SC process would not be needed until 2027. If the Everett WPCF treats flows from Snohomish, the expansion would likely be required by 2024.

10.3.2 Conveyance

Conveyance to the Everett WPCF would include a new pump station located at the Snohomish WWTP, because gravity flow from Snohomish to the Everett sewer system is not possible. Locating the pump station at the Snohomish WWTP avoids having to modify the existing Snohomish collection system and also allows CSO reduction projects to proceed as planned.

The Everett WPCF is located on Smith Island on the east side of Interstate 5, adjacent to the Snohomish River and Union Slough, and about 10 miles from the Snohomish WWTP. The closest reasonable tie-in location to the Everett sewer system is the South End Interceptor at Lenora Street, which is approximately 5 miles from the Snohomish WWTP. The tie-in to the Everett sewer system would be a “hot tap,” occurring while the South End Interceptor is in service during the dry weather season when there are lower flows. While the Snohomish force main would not cross the Burlington Northern Santa Fe railroad, it would tie in to the South End Interceptor on railroad property.

Four force main alignments were considered for conveying City flows to the Everett South End Interceptor: one south of the Snohomish River and three north of the Snohomish River. Several factors differ for each alignment, including operational complexity and energy requirements, Snohomish River crossing risk, erosion risk, traffic impact, environmental permits, residential property impact, and the need for odor control. All alignments would cross the Snohomish River and a Marshland Flood Control District structure.

The alignment south of the Snohomish River was identified as the most favorable with the fewest environmental impacts and lowest cost. It has the lowest operational complexity with fewer low and high spots resulting in fewer air release valves (and less associated odor concerns) and lower energy requirements for pumping. This alignment would also have the simplest right-of-way acquisition. Alignments north of the Snohomish River would entail additional smaller stream crossings and have greater impact to traffic in residential areas. If permitting issues were to arise with the alignment south of the Snohomish River, the second most favorable route north of the Snohomish River is along Riverview Road. These two alignments are shown in Figure 10-2.

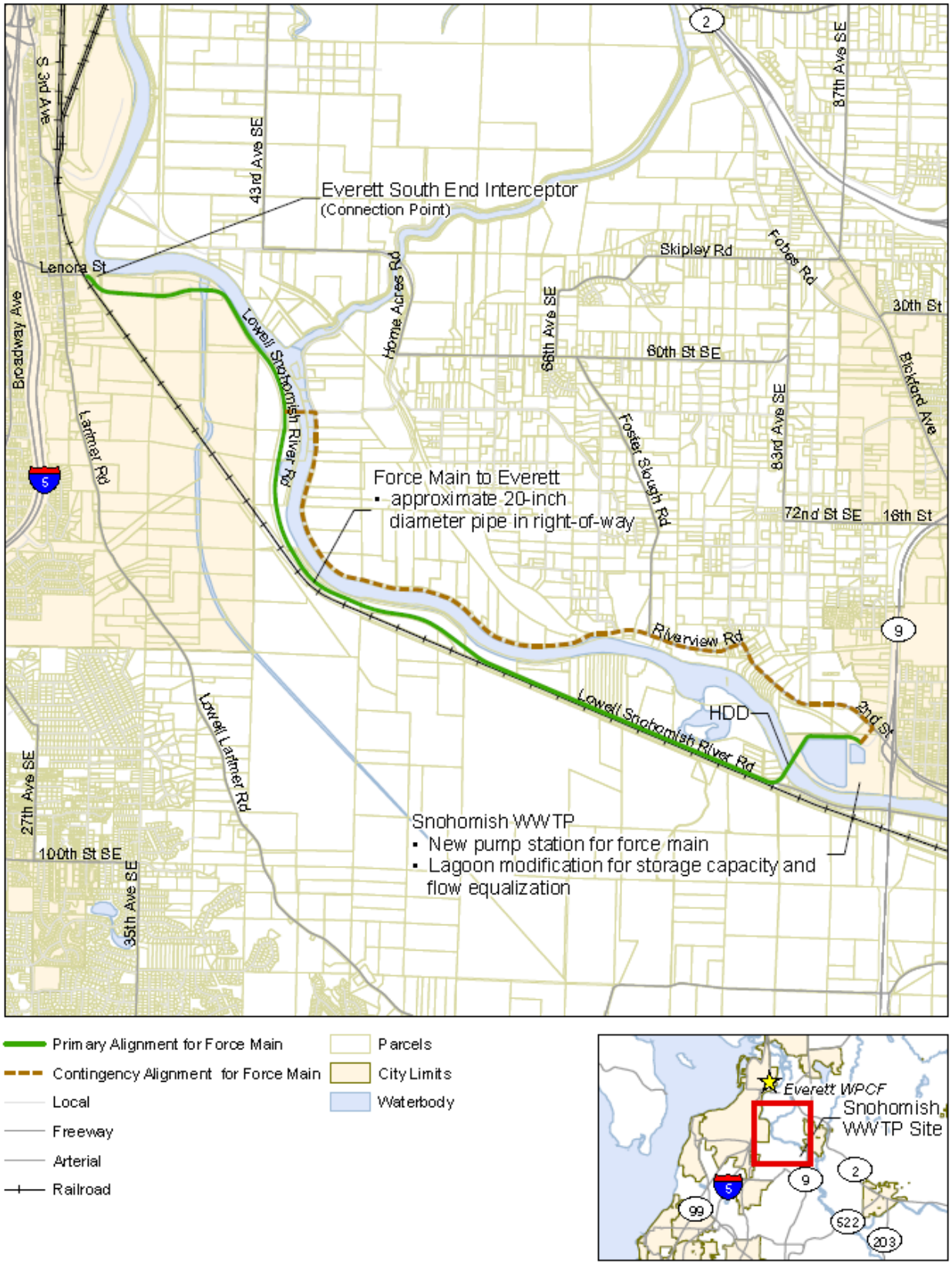


Figure 10-2 Snohomish to Everett Conveyance Route (2010 Plan)

10.3.3 Hydraulic Analysis

As mentioned previously, a new pump station located at the Snohomish WWTP will be needed to transfer flows to Everett. The pump station would be composed of two main chambers: a dry well and a wet well. Wastewater would enter the wet well, whereas the pumps and motors would be located in the dry well chamber. Chemical addition at the pump station might be necessary to reduce odors generated by the wastewater in the lengthy force main.

Two duty sets of two pumps in series would be used, and another set of pumps would be used for standby, for a total of six pumps. The pumps would be non-clog centrifugal pumps, which are well-suited to handle raw sewage and high flows and heads. Based on available and projected plant flow data, the pumps would need to handle a wide range of flow and pressure. The Everett South End Interceptor has a peak capacity of 4,500 gallons per minute (gpm) for flows from Snohomish. Everett anticipates the maximum head at the tie-in location is 85 psi, or about 200 feet. Total dynamic head is expected to range from 30 to 350 feet. The pumps would be sized to handle flows from 500 to 4,500 gpm, the maximum flow that could be accepted by the City of Everett's South End Interceptor. The pump station would have a footprint of approximately 3,000 square feet and would be located west of the existing WWTP facilities, in the old lagoon. This location would take advantage of gravity flow from the existing headworks and lagoons to the pump station wet well. Some filling of the lagoons would be needed to support the pump station. In addition, the pump station would be supported by auger driven piles to stabilize against settling and hydraulic uplift. The force main would be about 20 inches in diameter so the maximum velocity would be approximately 5 feet per second at 4,500 gpm. This ensures that friction losses in the pipe will not be excessive. The force main would be constructed of high-density polyethylene pipe and be installed in a steel casing at the Snohomish River crossing.

10.3.4 Planning-Level Costs

The planning-level opinion of probable construction cost developed in the 2010 Plan for the horizontal directionally drilled river crossing, pump station, Snohomish WWTP improvements for flow equalization and force main to convey wastewater from the existing City of Snohomish WWTP to the Everett WPCF is approximately $23 million. This cost is a Class 5 estimate as defined by the American Association of Cost Engineering with an expected accuracy of +50 percent/-30 percent and is in 2009 dollars. It is assumed the initial Everett connection charge (capitalized cost) will be approximately $20 million, for a total capitalized cost of about $43 million. Additionally, the City would incur an annual service charges of approximately $1 million and annual O&M costs estimated at approximately $412,000 for the first year of operation.

10.3.5 Summary

Because the Everett alternative has substantially lower capital and O&M costs compared to the Brightwater alternative, it was chosen as the favored alternative for regional wastewater treatment.

10.4 Consideration of Regionalization

The City had previously pursued regional treatment with the City of Everett. The City retained CH_2M HILL to prepare the 2010 Plan, which determined that the total capital cost of regionalization would be approximately $43 million, including an estimated $20 million to purchase capacity at the Everett WPCF. The 2010 Plan projected a City population of approximately 17,554 at the end of the previous planning horizon (2024) in the 2005 Plan, which

included population growth from UGA expansion. However, the 2010 Plan noted that the economic recession might slow the expected population growth.

Because of the extensive time needed to plan, design, and construct the regionalization project, the City to proceeded with near-term improvements of the existing WWTP to ensure compliance with its National Pollutant Discharge Elimination System (NPDES) permit. Due to the success of the near-term improvements, plans for a large UGA expansion being abandoned, and the City's growth rate being much flatter than estimated in the 2010 Plan, the plan for regionalization was postponed indefinitely. The City has no intention of pursuing regionalization further.

Chapter 11 Wastewater Treatment Facility Improvement Alternatives

11.1 Overview

The near-term WWTP improvements project was completed in 2012 to bring the WWTP into compliance with Agreed Order No. 7974 – Near-Term Improvements, and a new permit was issued in November 2012. The near-term improvements project included installation of a SFF media system in the lagoons, replacement of an old mechanical screen with a new multi-rake bar screen, installation of a supplemental alkalinity feed system, and installation of instruments for monitoring DO and pH. Improved $CBOD_5$ and ammonia-N removal resulting from the near-term upgrade was validated during the official demonstration period required by Ecology between July 2013 and June 2014. In 2015, Ecology determined that the City had satisfied the conditions in Agreed Order No. 7973 and Agreed Order No. 10467 (amending Agreed Order No. 7974) with their near-term improvements. Therefore, Ecology no longer required the City to connect their wastewater system to the City of Everett's conveyance system and that project was postponed indefinitely.

The 2010 Plan compared the Everett conveyance project with an extensive upgrade and expansion alternative for the WWTP that included, among other items, a new selector-activated sludge process for biological treatment, new cloth disk filters for filtration, and a new ultraviolet (UV) disinfection system. This upgrade and expansion alternative was developed based on projections of substantial growth in the sewer service area and population, due in part to a large expansion of the UGA planned at the time. The growth projections and planned UGA expansion have subsequently been reduced for the current planning horizon.

Because the revised projections predict much lower population growth, and the WWTP upgrades from the near-term improvements project satisfied the requirements in Ecology's Agreed Order No. 7973 and Agreed Order No. 10467, the City is no longer looking at the option of utilizing Everett's Water Pollution Control Facility (WPCF). Therefore, the City has continued to invest in improvements to their WWTP after successful implementation of the near-term improvements by completing the following:

- Initiated regular use of the effluent filters, which had been previously idle.
- Completed pipe repairs within the effluent filters.
- Replaced an existing static mixer to reduce headloss and increase capacity of the filter feed pumps from 0.8 MGD to 1.4 MGD to utilize more of the existing filtration capacity.
- Installed a pressure sensor within the influent channel of the effluent sand filter so level can be monitored in SCADA.
- Added ultrasonic transducers in Lagoons 3 and 4 to help control algal growth and reduce solids loading on the effluent filters. The ultrasonic waves inhibit growth of the algae.
- Installed chlorine residual analyzers and implementing automated control for dosing chlorine gas for disinfection and sulfur dioxide gas for dechlorination.
- Added a redundant sulfonator.
- Installed equipment allowing two more chlorine and SO2 cylinders each, which increased dosing capacity from 80 pounds per day to 160 pounds per day.

- Installed vacuum alarm switches for the chlorine gas and sulfur dioxide gas systems.
- Replaced old failing automatic transfer switches and installed a second generator to improve reliability and increase capacity of backup power.
- Replaced broken inlet and outlet valves in the chlorine contact tank.
- Retrofitted the chlorine contact tank outlet gates along with Lagoon #4 outlet gate with electric actuators for improved control of flow attenuation and ability to isolate discharge to the outfall.
- Removed accumulated biosolids from the lagoons and disposed of them at a designated beneficial use facility.
- Replaced the two main control panels and PLCs, upgraded the SCADA system, and revised the control strategies to improve overall performance.

All of the above recent improvements have helped to further improve safety, reliability, performance and efficiency of the WWTP, though they have not significantly impacted capacity. As discussed in Chapter 9, capacity of the effluent filters and lagoon aeration system needs to be increased to address current deficiencies. Furthermore, the City wants to replace the existing chlorine gas disinfection system to improve safety, replace aging equipment and comply with current codes. The following sections identify and evaluate alternatives to make these improvements and recommend an alternative for implementation. Other improvement needs identified in Chapter 9 do not involve analysis of different alternatives. The recommendations for those improvement needs and the selected alternatives discussed in this Chapter are summarized together in Chapter 12.

11.2 Filtration Alternatives

As discussed in Chapter 9, the existing effluent filters need to be upgraded and expanded to enhance performance and allow filtration of more of the effluent flow, particularly during the winter months when higher flows and lagoon turnover events increase solids and hydraulic loading on the existing filters. Doubling the capacity of the effluent filters from 1.4 MGD to 2.8 MGD would allow for filtration of nearly all effluent flow throughout the planning horizon. It is not practical or economical to filter peak flows, which tend to have low concentration of solids. Although the capacity of the WWTP may be re-rated above 2.8 MGD, doubling the filtration capacity to 2.8 MGD would be a much simpler retrofit, as it would require no changes to the existing filter structure, would allow for uniform modularity among all filter cells, and would still provide substantial improvement in effluent quality. Additionally, compliance with the NPDES permit does not require that all effluent flow is filtered. The filtration alternatives discussed below were identified as reasonable solutions to upgrade and expand the effluent filters.

Applicable design and redundancy requirements from the Orange Book that were considered in this evaluation are as follows:

- The filter system should consist of multiple units so that at least one unit can be backwashed or removed from service without overloading the remaining units (T4-2.7.1, Orange Book, Ecology 2008).

For consistency with this requirement, the filtration alternatives discussed were designed with multiple filter units that can treat part of the WWTP effluent with one unit out of service. Other assumptions for an expanded filtration system with 2.8 MGD capacity include:

- Additional filtration units will be built below grade.

- The filter feed pumps will be replaced to convey a range of flows up to 2.8 MGD.
- Polymer and alum will be used for pretreatment. Although the City currently uses polymer only, in the past both polymer and alum have been used and it is not uncommon to use both in this type of application. Therefore, it is conservatively assumed for this analysis that both chemicals would be utilized.
- Algal control will be continued.

11.2.1 Evaluation Criteria

The evaluation and comparison of filtration alternatives considered the following criteria:

- Capital Construction Cost – opinion of probable construction cost.
- 20-Year O&M Cost – present worth of the annual O&M cost (chemical use, energy, labor, and maintenance).
- Footprint – estimate of the space required and an assessment of how easily the system may fit within the available space.
- Pretreatment – requirement and extent of pretreatment necessary for optimal performance of filtration media.
- Performance – ability to comply with NPDES permit limits; the potential for system failure, based on operational experience; assessment of reliability and longevity.
- Complexity – assessment of system O&M complexity in normal and failure modes. Consideration was also given to the number of physical and mechanical processes involved.
- Expandability – ease with which the process could be expanded or modified to accommodate additional flow and load or comply with more stringent effluent limitations.

11.2.2 Expansion of DynaSand®

Sand filters use a granular media (sand) to remove suspended solids from the effluent stream. These mono-medium filters can pass treated water upward or downward through the sand bed. The medium is available in various grain sizes and is specified based on the constituents of the influent to be filtered.

The existing filters at the WWTP are continuous backwash upflow sand filters (Parkson DynaSand®). A distinctive characteristic of the continuous backwash filter is that backwash and filtration cycles occur simultaneously. The wastewater to be filtered flows upward through the sand bed. At the same time, the sand medium moves in a counter-current direction and is being cleaned continuously. The dirty portion of the sand at the bottom of the filter is pumped by airlift through a central pipe up to a sand washer assembly located at the top of the filter. Compressed air is introduced at the bottom of the airlift to help lift the dirty sand and increase backwash efficiency by removing unwanted particles with increased turbulence, abrasion and shear forces. At the top of the filter, the sand washer assembly separates the dirty particles from the sand. The clean sand is redistributed on top of the filter bed through channels located at the bottom of the sand washer.

Doubling the capacity of the existing continuous backwash DynaSand® system to 2.8 MGD would be accomplished by building two additional filter cells (i.e., four filter modules total) below grade. This alternative also includes new air compressors to provide sufficient air supply for the

larger filtration system. The expanded system would operate in the same manner as the current system, but the size would double. The WWTP staff is familiar with this system, although they have indicated it requires significant effort and can be difficult to maintain.

The continuous backwash filters generate the highest volume of backwash. The reject pumps would need to be replaced to accommodate the increased backwash flow; however, the amount of head generated by the 3-inch discharge pipe would create a bottleneck. Therefore, the 3-inch reject discharge pipeline would also need to be upsized under this alternative. The specific size and routing of the new discharge pipe would be determined during design.

Addtionally, it is recommended that inclusion of a pretreatment flocculation tank be assumed for planning purposes to help combine smaller algae particles into larger floc particles that can be more easily filtered and backwashed from the media. Therefore, construction of a pre-treatment flocculation tank is included in the cost of this alternative.

11.2.3 EcoWash™

A new version of the continuous backwash sand filter has the same basic design but includes additional controls and monitoring devices to trigger intermittent backwashing cycles (Parkson EcoWash™). Intermittent backwash cycles offer several benefits over continuous backwashing:

- Power savings from reduced compressor run time (60 to 90 percent reduction per Parkson).
- Reduced backwash reject flow (60 to 90 percent reduction per Parkson).
- Less cost to re-process backwash flow (60 to 90 percent reduction per Parkson).
- Lower operation and maintenance (O&M) costs.
- Improved filtrate quality.

Installation of the intermittent backwash system is similar to the continuous backwash system, except for additional controls. The backwash cycles are controlled by one of two modes, differential pressure or timer. In differential pressure mode, the system acknowledges the inlet/outlet levels and the airlift/reject starts at a programmed point until the differential is reduced (or timer overrides and starts washing). In timer-controlled mode, the operator programs the timer to initiate sand washing at a certain interval (or differential overrides and starts washing). The EcoWash™ filter has a sand movement detection system tied to an alarm and monitored by the WWTP control system, eliminating the need to physically check sand movement. Other modifications have been made to the airlift design and operation to ensure consistent sand movement. When not in the backwash cycle, the reject line closes automatically, reducing the amount of reject water. It is also important to note the filter uses rotary screw air compressors instead of reciprocating air compressors as used by continuous backwash filters, because the demand for air is not constant.

The EcoWash™ filters are equipped with a central control panel that includes a programmable logic controller (PLC) and a touch screen human-machine interface (HMI). The PLC can communicate via Ethernet with the WWTP SCADA system and other PLCs.

This alternative would involve building two additional filter cells below grade, equipping them with the EcoWash™ system, and retrofitting the existing filters with the same system. The sand

media in all filters would be replaced with 1.4 mm grain size sand, and the air compressors would be changed to accommodate the intermittent requirement for compressed air.

The amount of backwash is lower with EcoWash™, but it does vary over a relatively wide range, and the existing reject pumps will likely require replacement. For this preliminary evaluation, it is assumed a 60 percent reduction in reject flow is achieved and only the reject pumps will be replaced.

11.2.4 Compressible Media Filters

Compressible media filters use a synthetic fiber porous material instead of granular material. The compressible media filter offers several unique benefits:

- The filter can operate at very high filtration rates (10 to 40 gpm/sf compared to 3 to 5 gpm/sf for conventional filters) because of the media's high porosity.
- The filter media and filter bed properties can be modified because the media are compressible.
- The process water that is being filtered is used to backwash the filter.

The filter bed consists of a large number of compressible balls that are formed by shrinking synthetic fibers into a quasi-spherical shape, resulting in a complex woven structure. An individual compressible ball is approximately 1.25 inches in diameter. The density of the media is slightly heavier than water. Because of its low density, the filter media is retained in a confined cell.

Properties of the filter bed (i.e., collector size, porosity, and depth) are altered by adjusting the position of a moveable plate or the hydraulic loading on the cell (depending on the manufacturer). The direction of flow can be upward or downward through the media bed. With the compressible media filter, the properties of the filter can be adjusted by altering the degree of compression of the media in response to changing conditions and the desired effluent quality.

Currently, only two manufacturers provide filtration systems with compressible media, the Schreiber Fuzzy Filter and the WWetco FlexFilter™. The two systems use a similar synthetic compressible ball-shaped medium, but their systems are configured differently. Both filters are considered as potential alternatives. Figure 11-1 shows schematic diagrams for the Fuzzy Filter during its operation, which has an upward flow configuration. The WWetco FlexFilter has a downward flow configuration.

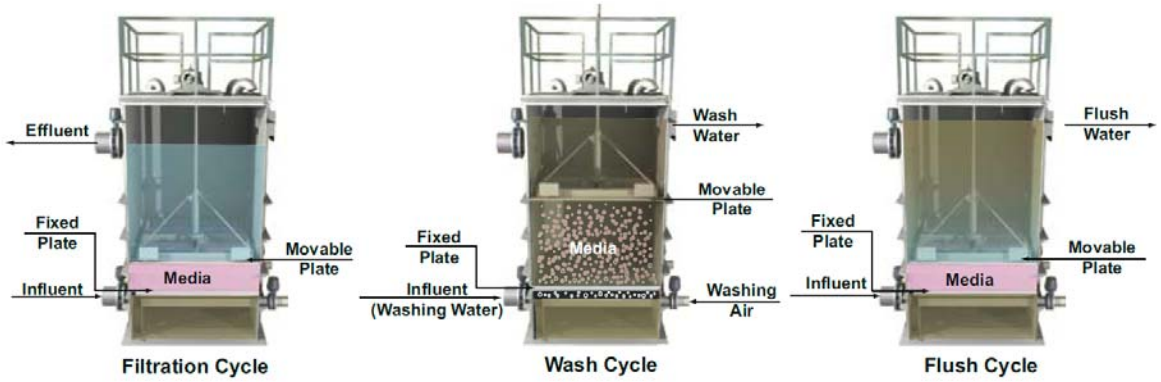


Figure 11-1 Schreiber Fuzzy Filter

Schreiber Fuzzy Filters

The existing sand filter equipment would be replaced with two 7-foot by 7-foot Fuzzy Filter units in fabricated steel tanks installed atop a slab on grade. Consideration was given to installing the filter units within the existing sand filter in-ground concrete tanks, but installing them in new tanks above grade would significantly improve ease of installation and access. The standby blower for the SFF media system could be utilized to provide the necessary scouring air.

Unique to this filter, the size of the media bed can be adjusted by varying the position of two movable plates and thus varying the compression of the synthetic media. This ability to modify the filter in response to influent quality provides the highest loading rate among the four alternatives. The direction of filtration is upward flow, with the filtrate removed from the top of the filter cell.

Although the Fuzzy Filter has no history of use in lagoon treatment plants, the City conducted a 4-week pilot test using a Fuzzy Filter during the months of August and September 2018 when algae concentrations are generally highest. With the same effluent quality and chemical dose, the Fuzzy Filter appeared to perform better than the existing sand filters. The Fuzzy Filter averaged 77% removal of TSS at the design hydraulic loading rate of 30 gpm/sf versus 63% removal of TSS for the existing sand filters. Additionally, the algae did not appear to cause significant fouling of the media and the manufacturer did not indicate the algae would substantially impact the media replacement interval.

It is recommended that inclusion of a pretreatment flocculation tank be assumed for planning purposes to help combine small algae particles into larger floc particles that can be more easily filtered and backwashed from the media. Therefore, construction of a pre-treatment flocculation tank is included in the cost of the Fuzzy Filter alternative.

WWetco FlexFilter™

The existing sand filter equipment would be removed from both filter cells and replaced with a FlexFilter™ compressible media filter system. Because the FlexFilter™ requires three filter cells, construction of a third cell would be necessary in addition to using the two existing filter cells. Construction of a pre-treatment flocculation tank per the manufacturer's recommendation is included in the cost of this alternative.

The FlexFilter™ is a compressible media filter, but it uses downward flow and has no movable plates. The FlexFilter™ uses the hydrostatic force of the influent to compress the media within a V-shaped bladder. The filtrate is removed from the bottom of the filter cell.

This system is not complex, but it has a larger footprint than the other compressible media option, and it provides less flexibility in loading. In addition, this option requires the highest initial capital outlay because of its large footprint. The FlexFilter™ has been installed in at least three similar applications.

11.2.5 Disk Filters

The most common type of disk filters use a cloth membrane as the filter medium. The cloth membrane is a randomly woven fabric used to separate the particles from the fluid to be filtered. The density and thickness of the filter cloth is selected according to the characteristics of the influent wastewater and desired effluent quality. Other filters use membranes woven from stainless steel or synthetic media.

In the fully submerged configuration, several disks covered by the membrane are mounted vertically to a common hollow tube. In the most common configuration, wastewater passes through the membrane by gravity and enters the interior of the filter disks. The hollow tube conveys filtered effluent from the filter. During filtration, the filter is quiescent, which allows larger particles to settle to the bottom of the tank and decreases the amount of solids to be filtered. The settled backwashed solids are pumped intermittently back into the process stream for treatment.

This potential application was discussed with several vendors. One declined to propose due to the propensity for lagoon plants to have algae present. A second vendor suggested compressible media as a better option for this application. The explicit concern of the disk manufacturers is that algae will "blind" the filter media. Therefore, disk filtration was eliminated from further consideration, because it is not possible to eliminate algae in the lagoons.

11.2.6 Summary of Filtration Alternatives

A summary of the development of the filtration alternatives is provided in Table 11-1 below.

Table 11-1 Filtration Alternatives Design Summary[a]

Parameter	Units	DynaSand	EcoWash	Fuzzy Filter	FlexFilter
Design Flows					
Filtration Capacity[b]	MGD	2.8	2.8	4.2	2.8
2036 Annual Avg. Flow[c]	MGD	1.9	1.9	1.9	1.9
Filtration Rates					
at Filtration Capacity	gpm/sf[d]	4.86	4.86	30	10.8
at 2036 Annual Avg. Flow	gpm/sf	3.30	3.30	13.5	7.3
Physical Information					
Number of Existing Cells	each	2	2[e]	0	0
Number of New Cells	each	2	2	2[f]	3[g]
Filtration Area per Cell	sf[h]/cell	100	100	50	90
Existing Filter Footprint	sf	200	200[e]	0	180[g]
New Filter Footprint[i]	sf	200	200	100	90[g]
Flocculation Tank Area	sf	200[j, k]	200[j, k]	200[j, k]	200[j, k]
Total Process Footprint	sf	600	600	300	470
Filter Feed Pump Station					
Number of Pumps[l, m]	each	2(2)	2(2)	2(2)	2(2)
Pump Status		New	New	New	New
Capacity, per Pump	gpm	1,300	1,300	1,300	1,300
Firm Capacity[n]	gpm	1,300	1,300	1,300	1,300
Reject Pump Station					
Max Instantaneous Reject Flow	gpm	292	292	490	75
Reject Flow Frequency	cycles/day	continuous	24	3 to 5	2 to 5
Reject Flow Duration	minutes/cycle	continuous	8 to 22	15	75

Table 11-1 Filtration Alternatives Design Summary[a]

Parameter	Units	DynaSand	EcoWash	Fuzzy Filter	FlexFilter
Estimated Reject Volume per Day	gallons/day	420,500	56,100 to 154,200	58,800	23,000
Number of Pumps[l]	each	2(1)	2(1)	2(1)	2(1)
Pump Status		New	New[o]	New	Existing
Capacity per Pump	gpm	300	300	300	100
Air Compressor or Blower					
Type		Reciprocating	Rotary Screw	Blower, NP[p]	Positive Displacement
Motor	hp	5	5	40	40
Rated Pressure	psig[q]	150	125	NP	3.5
Capacity	scfm[r]	20.8	17.5	NP	900
Total Air Required/Unit	scfm	20.8	10.4	750	900
Other Improvements					
Alum Pumps Upgrade		Required	Required	Required	Required
Alum Storage Tank Upsize		Required	Required	Required	Required
Sand Media Size	mm	1.7	1.4[s]	NA[t]	NA[t]
Algae Control[u]		Required	Required	Required	Required

Notes:
(a) Summary of parameters used to develop opinions of probable cost.
(b) Based on doubling current capacity of 1.4 MGD, except Fuzzy filter would include larger units to capture more peak flow and potential increase in max month flow with proposed plant re-rating.
(c) Flow to the filtration under 2036 average annual flow conditions.
(d) gpm/sf – gallons per minute per square feet.
(e) Existing DynaSand® continuous backwash filters retrofitted to EcoWash™.
(f) Two new filter tanks installed atop a new slab on grade.
(g) Two new filter cells constructed in two existing filter cells and one new filter cell constructed.
(h) sf – square feet.
(i) Filter footprint area outside of the existing filter tank footprint.
(j) Inclusion of flocculation tank assumed for planning purposes.
(k) 14 ft x14 ft x13 ft tank to provide 10-minute retention time at 2.8 MGD.
(l) Total number of units, including backup or spare, with the number of units in service at full capacity in parentheses.
(m) Space is available for only two pumps in the existing wet well. Sized for one lead and one lag pump.
(n) Firm capacity is defined as the capacity of the system with the largest unit out of service.
(o) New reject pumps included, but further analysis may show that existing reject pumps are sufficient.
(p) Not provided by vendor. Use spare blower for SFF media.
(q) psig – pounds per square inch gauge.
(r) scfm – standard cubic feet per minute.
(s) Vendor recommends replacing existing 1.7 mm sand media to improve filtrate quality as long as algal growth is controlled.
(t) NA – not applicable.
(u) Algae control recommended by all vendors.

11.2.7 Filtration Probable Construction and O&M Costs

An opinion of probable construction cost was prepared for each alternative by using a combination of commercial cost estimating software, published cost indexes, cost estimating guides, vendor quotations, and bids or estimates from similar facilities at other treatment plants. The estimates provided in Table 11-2 include markups for site work, mechanical/yard piping, electrical, instrumentation and control, mobilization, contractor overhead and profit, contingency, and sales tax.

An estimate of the annual O&M costs associated with each alternative was prepared using published O&M cost information, desktop estimates of energy and chemical requirements based on typical operation, maintenance estimates related to labor rates, vendor-supplied information on parts cost and replacement interval, and labor estimates from similar facilities at other treatment plants. Although the WWTP currently no longer uses alum with the existing effluent filters, it is included in the estimates of O&M cost based on previous historical use. Estimates of annual O&M costs apply to the evaluated alternative only. Administration, laboratory work, energy, chemical usage, labor, and maintenance associated with other processes and facilities at the WWTP are not included in these estimates.

These estimates are conceptual and should therefore be considered to have an accuracy of +50 to -30 percent, consistent with order-of-magnitude estimates as defined by the American Association of Cost Engineers (AACE).

Table 11-2 Filtration Alternatives Cost Summary[(a)]

Parameter	Units	Parkson DynaSand® (Sand)	Parkson EcoWash™ (Sand)	Schreiber Fuzzy Filter (Compressible)	WWetco Flex Filter™ (Compressible)
Average Energy Use					
Yearly Energy Consumption	kWh/year	81,016	29,706	17,652	57,930
Energy Unit Cost	$/kWh	0.075	0.075	0.075	0.075
Yearly Energy Cost[(b)]	$/year	6,100	2,200	1,300	4,300
Labor					
Monthly Labor Requirements	hrs/month	24	22	24	24
Labor Unit Cost	$/hour	50	50	50	50
Yearly Labor Cost	$/year	14,400	13,200	14,400	14,400
Chemical					
Alum Consumption	lbs/year	82,490	65,992	65,992	65,992
Yearly Alum Cost	$/year	19,800	15,800	15,800	15,800
Polymer Consumption	gals/year	1,935	1,548	1,548	1,548
Yearly Polymer Cost	$/year	44,000	35,400	35,400	35,400
Annual Sand Lost	%/year	1	1	NA	NA
Yearly Sand Replacement Cost	$/year	840	840	NA	NA

Table 11-2 Filtration Alternatives Cost Summary[a]

Parameter	Units	Parkson DynaSand® (Sand)	Parkson EcoWash™ (Sand)	Schreiber Fuzzy Filter (Compressible)	WWetco Flex Filter™ (Compressible)
Yearly Chemical Cost[b]	$/year	64,600	52,100	51,200	51,200
Replacement					
Yearly Replacement Cost[c]	$/year	3,980	2,280	6,710	3,150
Total Yearly O&M Cost[d,e]	**$/year**	**89,100**	**69,800**	**73,600**	**73,100**
Construction Cost[f]					
Process and Equipment	$	543,900	585,400	660,500	708,600
Structures	$	176,800	176,800	69,100	144,600
Other Related Work[g]	$	295,700	310,300	255,400	342,100
Mobilization and OH&P	$	152,500	160,900	147,800	179,300
Contingency and Taxes	$	516,700	545,400	500,800	607,800
Probable Construction Cost (2013)[f]	**$**	**1,687,000**	**1,780,000**	**1,635,000**	**1,984,000**
Probable Project Cost (2013)[f,k]	**$**	**2,194,000**	**2,314,000**	**2,126,000**	**2,580,000**
Probable Project Cost (2018)[f,h,k]	**$**	**2,390,000**	**2,520,000**	**2,316,000**	**2,810,000**
20-Year O&M Cost (2017)[f,i]	**$**	**1,326,000**	**1,038,000**	**1,095,000**	**1,088,000**
20-Year NPV[f,j]	**$**	**3,716,000**	**3,558,000**	**3,411,000**	**3,898,000**

Notes:

(a) All costs are in 2013 dollars unless specified. See Appendix I for further breakdown of construction and O&M estimates.
(b) Yearly energy and chemical use based on operation at rated capacity of 2.8 MGD.
(c) Not all replacements occur every year, but replacement costs are converted to a yearly cost for comparison. Replacement interval varies between media or equipment, but replacement costs are included if the replacement occurs within the planning horizon. See Appendix I for details.
(d) Sum of energy, labor, chemical and replacement cost.
(e) Cost rounded to nearest hundred dollars. The totals presented may not appear to add up to the components listed due to rounding.
(f) Cost rounded to nearest thousand dollars. The totals presented may not appear to add up to the components listed due to rounding.
(g) Other related work includes site work, foundations, electrical, mechanical and controls.
(h) RSMeans Cost Index used to adjust costs to 2018 dollars.
(i) Assumes 3 percent effective interest (interest less inflation) per year for present value of future annuity calculation.
(j) NPV – net present value. Sum of 2018 project cost and O&M costs.
(k) The project cost includes a 30% markup added to the construction cost to cover planning, design and construction services during the project.

The Fuzzy Filter alternative is estimated to have the lowest probable construction cost, but is only slightly less than the probable construction cost estimated for the Parkson DynaSand® alternative. Similarly, the 20-year net present value of the Fuzzy Filter alternative is only slightly lower than that of the Parkson EcoWash™ alternative. These costs are considered comparable, meaning the differences are within the accuracy of the estimates. The WWetco FlexFilter has both the highest capital outlay and highest O&M cost. The DynaSand® continuous backwash filters (same as current filter system) has the highest annual O&M cost. The current filter system at the WWTP has been the highest maintenance item and is in need of further maintenance, such as replacing the airlifts (typically replaced on a 3- to 5-year interval).

11.2.8 Comparison of Filtration Alternatives

An overview of advantages and disadvantages associated with each filtration alternative is provided in Table 11-3. Comparative scoring is provided in Table 11-4.

Each alternative was scored for each criterion in comparison with the other evaluated alternatives. The most favorable, neutral, and least favorable alternatives were scored as (+), 0, and (-), respectively. Justifications for scoring qualitative criteria are based on the advantages and disadvantages summarized in Table 11-3.

The total score in Table 11-4 is provided only as a quick overview of the scoring. The scoring includes some weighting of criteria such that each one does not contribute equally to the overall score. For example, the score for capital cost is multiplied by a factor of 3, whereas the score for footprint is multiplied by a factor of only 1.

Table 11-3 Filtration Alternative Advantages and Disadvantages

Parameter	Parkson DynaSand® Continuous Backwash (Sand)	Parkson EcoWash™ (Sand)	Schreiber Fuzzy Filter (Compressible)	WWetco Flex Filter™ (Compressible)
Footprint	• Expansion doubles existing footprint • Requires construction of 2 below grade tanks • Flocculation tank recommended	• Expansion doubles existing footprint • Requires construction of two below grade tanks • Flocculation tank recommended	• Above grade installation on slab with pre-packaged tanks • Flocculation tank recommended	• Requires three cells (modify 2 existing filter cells; build 1 new) • Requires flocculation tank • Above or below grade installation available
Pretreatment	• Pre-treatment flocculation tank recommended	• Pre-treatment flocculation tank recommended • Estimated to save as much as 20% on chemicals compared to conventional DynaSand	• Pre-treatment flocculation tank recommended • Requires nearly complete elimination of algae	• Pre-treatment flocculation tank required • Requires nearly complete elimination of algae
Performance	• Increased opportunity to trap flocculated colloidal material due to depth of filtration media • Full loading may be limited during peaks of high TSS • Technology has a history of performance at lagoon plants	• Increased opportunity to trap flocculated colloidal material due to depth of filtration media • Provides higher level of filtration than Parkson DynaSand® • Full loading may be limited during peaks of high TSS	• Significantly higher loading capacity (30-40 gpm/sf) • Pilot testing demonstrated somewhat improved performance versus sand filters • Vendor has limited experience with lagoon plants • Cannot be retrofitted for denitrification	• Higher loading (10.8 gpm/sf) • Product is only about 10 years old, but there are installations at lagoon plants • Cannot be retrofitted for denitrification
Complexity	• Complexity during installation	• Complexity during installation • Some additional complexity with automated controls for backwash cycles	• More complex with actuated moving plates to adjust filtration area and blower for scouring	• Complexity during installation • Bladder system for hydrostatic gravity compression has no moving parts
Expandability	• Below grade location requires construction of additional filter cells	• Below grade location requires construction of additional filter cells	• Above grade installations are self-contained and easier to expand	• Below grade location requires construction of additional filter cells • Above grade installations are self-contained, but larger than the Fuzzy Filter

Table 11-4 Filtration Alternatives Comparison

Parameter	Parkson DynaSand® (Sand)	Parkson EcoWash™ (Sand)	Schreiber Fuzzy Filter (Compressible)	WWetco Flex Filter™ (Compressible)	Weighting Factor
Capital Cost	+	0	+	-	x3
O&M Cost	-	+	0	0	x2
Footprint	0	0	0	+	x1
Pretreatment	0	0	0	-	x1
Performance	0	+	+	+	x1
Complexity	+	0	0	+	x1
Expandability	0	0	+	0	x1
Total Score[(a)]	**+2**	**+3**	**+5**	**-1**	

Note:
(a) Total score calculated by summing the products of the weighting factor and score for each parameter.

11.2.9 Filtration Recommendation

Scoring of the filtration alternatives indicates that the Fuzzy Filter alternative is the most favorable. Although the WWTP staff are familiar with the operation of sand filters, they are a relatively simple technology, and the EcoWash™ system should provide better quality effluent and reduce O&M costs compared to the existing sand filters, the City prefers the Fuzzy Filter alternative due to issues staff have experienced with the sand filters and the Fuzzy Filter pilot test demonstrating improved performance. The City believes the Fuzzy Filter has greater operational flexibility and is not convinced the EcoWash™ system would resolve their concerns with the current sand filters. Most importantly, the Fuzzy Filter alternative provides greater capacity, allowing more effluent to be filtered during peak flows and for potential future plant re-rating. For these reasons, it is recommended that the effluent filtration process be upgraded and expanded using Fuzzy Filters.

Using Fuzzy filters for effluent filtration would eliminate use of the existing sand filters. However, it is recommended that the sand filter be decommissioned but not demolished. If the City should face future nitrogen limits, the sand filters could be potentially be utilized as denitrification filters to achieve some level of nitrogen removal prior to discharge. Although nitrogen removal is expected to be required in the future for many facilities discharging directly into Puget Sound, it is not known how this will impact facilities discharging into tributaries to Puget Sound and how far upstream discharges could potentially be regulated. Furthermore, the timeline for compliance with such regulations is unknown, though could be as far out as 2040.

11.3 Disinfection Alternatives

As discussed earlier in this section and in Chapter 9, the City wants to replace the existing chlorine gas disinfection system to improve safety, replace aging equipment and comply with current codes. Viable alternatives to chlorine gas include disinfection with delivered 12.5 percent sodium hypochlorite trade solution, disinfection with a 0.8 percent sodium hypochlorite solution generated onsite, ultraviolet (UV) disinfection, and disinfection with peracetic acid.

Applicable design and redundancy requirements from the Orange Book that were considered in this evaluation are as follows:

- Disinfection units shall be sufficient in number and size so that, with the largest-flow-capacity unit out of service, the remaining units shall have a design flow capacity of at least 50 percent of the total design flow (G2-8.2, Orange Book, Ecology 2008).
- The Orange Book recommends applying all design considerations for chlorine feed systems to SO2 feed systems (T5-6.1.1, Orange Book, Ecology 2008). It also states that the Uniform Fire Code (since replaced by the IFC) does not provide specific guidelines for SO2 gas and suggests consulting with the local Fire Marshal about the SO2 feed system. Although it is possible that the local Fire Marshal may require a sprinkler system for the SO2 storage and control rooms, it is presumed to be unlikely and so has not been included in the cost estimates.
- Because hypochlorite and gas chlorine behave similarly after injection into the wastewater stream, the information presented in Section T5-4 of the Orange Book related to design considerations, design details, and safety is also applicable to sodium hypochlorite use as a trade solution or generated on site.
- Sizing of the UV disinfection alternative is consistent with criteria outlined in Section T5-2 of the Orange Book.

Other assumptions for an upgraded disinfection system include:

- Dechlorination is assumed to continue with SO2 gas, rather than converting to a new chemical such as liquid sodium bisulfite, except perhaps for peracetic acid as noted below. Although SO2 has a National Fire Protection Association (NFPA) health rating of 3 (compared to a NFPA health rating of 2 for sodium bisulfite), it is not toxic like chlorine gas (NFPA health rating of 4), is more stable than sodium bisulfite (which can crystallize at room temperatures in some cases), and allows existing equipment and building spaces to be utilized and not entirely replaced. It would be cost-prohibitive to abandon the existing gas storage and control rooms and associated equipment and construct new space suitable for storing and metering liquid sodium bisulfite.
- The average SO2 dosage is assumed to be approximately 1 mg/L with automated control based on chlorine residual. SO2 is assumed not to be required normally with peracetic acid. The City could retain the SO2 system or include provisions for dosing sodium bisulfite with a peracetic acid system to quench potentially high residual concentrations of peracetic acid. Costs presented herein are based on the City retaining the existing SO2 system.
- The chlorination alternatives (sodium hypochlorite and onsite generation) are sized based on the chlorine dosages presented in Chapter 9 and to provide sufficient supply for the chemical demand under projected 2036 conditions.
- It is assumed that an average peracetic acid dose of 1 mg/L would be required, based on the results of recent full-scale testing.
- The chlorine contact tank would receive minor repairs (e.g., fill minor non-structural cracks) and the interior surfaces would be recoated to enhance structural longevity. Consideration could also be given to raising the effluent weir, since there is significant freeboard available, to increase the volume and capacity of the chlorine contact tank.

- The existing Water Champ® chemical induction mixers for both chlorine and SO2, including all associated appurtenances (i.e., control panel, lifting device, and power supply), would be replaced with new units.
- The existing scrubber would be rehabilitated and reused for SO2.
- The existing chlorine residual analyzers would continue to be used for automated control and continuous monitoring of chlorine residual. A redundant chlorine analyzer would also be provided.
- Automatic valve shutoff system and vacuum alarm switches for the existing chlorine gas system would be repurposed for the SO2 system.

11.3.1 Evaluation Criteria

The evaluation and comparison of disinfection alternatives considered the following criteria:

- Capital Construction Cost – opinion of probable construction cost.
- 20-Year Operation and Maintenance (O&M) Cost – present worth of the annual O&M cost (chemical use, energy, labor, and maintenance).
- Footprint – estimate of the space required and assessment of how easily the system may fit within the available space.
- Operational and Environmental Safety – consideration of potential hazards to WWTP staff and the environment associated with process operation and the storage and handling of hazardous chemicals.
- Performance – ability to comply with NPDES permit limits; assessment of the overall effluent quality at normal and peak flow conditions; assessment of reliability and longevity.
- Complexity – assessment of system O&M complexity in normal and failure modes and consideration for the number of physical and mechanical components involved.
- Expandability/Flexibility – ease with which the process could be expanded or modified to accommodate additional capacity or to comply with more stringent effluent limitations.

11.3.2 Delivered Sodium Hypochlorite Trade Solution

Sodium hypochlorite solution could be delivered to the site as trade solution (i.e., 12.5 percent sodium hypochlorite) and stored in a tank before use. A fill station would provide connections for pumping the chemical from the delivery truck into the storage tank with compressed air. Secondary containment would be provided around the storage tank and fill station to control spills or leaks.

The chemical solution is metered out of the storage tank using metering pumps and can be delivered to the point of use as a neat solution (i.e., no carrier water) or diluted using carrier water. When delivered as a neat solution, a flash mixing device, such as the existing mixers used to mix the chlorine gas solution, is required at the point of delivery to ensure adequate distribution of the chemical.

Although the bulk solution is classified as a corrosive material under the IFC, it is considered to be less hazardous than chlorine gas and therefore, requires less emergency response training and safety systems than for chlorine gas. O&M is relatively simple, although caution is needed when handling and working around the chemical. However, the chemical cost is higher than

chlorine gas. In addition, the stored bulk solution concentration decreases over time which leads to off-gassing from the solution and potential air-binding for pumps. These must be considered in the design and operation of such a system.

A 1,500-gallon storage tank would be large enough to provide more than a 30-day supply of sodium hypochlorite solution (at 12.5 percent) under projected 2036 demands. Two metering pumps, one duty and one standby, would be used to feed the solution to the dosing point. The metering pumps and appurtenances are assumed to be installed within a contained area in a new concrete masonry building that would include an automatic sprinkler system and fire-rated walls. The storage tank would be installed on a concrete pad adjacent to the building. The storage tank would be heat-traced and insulated for protection from freezing.

For dechlorination, the existing gas storage room and control room would continue to be used for dosing SO_2 gas. The existing chlorine gas valve shutoff system and vacuum alarm switches would be repurposed for the SO_2 system. Although the installation of an automatic valve shutoff system may negate the need for a gas scrubber, it is assumed that the existing scrubber would be rehabilitated and reused for scrubbing SO_2 gas.

As discussed in Chapter 9, some minor structural repairs and recoating the interior surfaces of the existing contact chambers would be made as part of this alternative. Structural repairs would include sealing small cracks in the structure and repairing minor leaks that may exist in the interior wall that divides the two contact chambers (to ensure the chambers can be isolated for maintenance).

Changes to the electrical load would include replacement of the existing 2-horsepower chlorine mixer and 3-horsepower SO_2 mixer with new 5-horsepower mixers, installation of two fractional horsepower metering pumps, and installation of a new panel board to serve lighting and HVAC loads associated with the new building. This would likely increase the total horsepower draw an equivalent of about 15 horsepower. This would necessitate some modifications to EMCC2 such as replacement of the existing motor starter and breaker for the chlorine and SO_2 mixers and potentially a larger main breaker. A new panelboard powered from a spare bucket in EMCC2 would provide power to the metering pumps and HVAC and lighting for the new building.

11.3.3 Onsite Sodium Hypochlorite Generation System

Onsite generation uses salt, water, and electricity to produce a 0.8 percent sodium hypochlorite solution. This process does not require the transportation or handling of hazardous chemicals. Softened water is fed into a brine dissolver. The salt dissolves to form a 30 percent brine solution, which is then diluted to achieve a 2.6 percent salt solution. The salt solution is reacted with low voltage DC current in an electrolytic cell to form the dilute hypochlorite solution. The solution is stored in a tank and replenished when the level in the tank drops to a low-level set point. A PLC monitors each aspect of system operation. System controls are commonly customized for each installation.

The solution has a longer shelf life than the delivered sodium hypochlorite trade solution, because the degradation rate of sodium hypochlorite decreases as the solution becomes more dilute. It is less corrosive and exhibits less off-gassing compared to the more concentrated trade solution. Because it is a weak solution, the 0.8 percent solution does not require double containment or special handling and storage procedures or requirements. The process does generate hydrogen gas as a byproduct, which must be vented from the system with blowers,

and a hydrogen leak detection system is required to ensure that this explosive gas does not accumulate.

Given the high energy requirements for this process, the cost effectiveness of onsite generation typically depends on the local cost of power. Because the capital cost of onsite generation is almost always higher than either gas chlorination or delivered sodium hypochlorite trade solution, energy must be relatively inexpensive compared to the cost of chemicals for the life cycle cost of onsite generation to be favorable. An onsite generation system contains more components than the other alternatives and requires salt storage and bag handling and additional softening for the dilution water.

The onsite generation system would produce a maximum of 200 pounds of chlorine per day. A 6,500-gallon storage tank would provide about a 2.4-day supply of sodium hypochlorite solution (0.8 percent) at the estimated 2036 demand. A 360-gallon brine tank would be included to provide about a 7-day supply of salt at the maximum chlorine production rate. Two metering pumps, one duty and one standby, would be used to convey the chlorine solution to the dosing point. The storage tank, brine tank and metering pumps are assumed to be installed within a new building. Since the 0.8 percent sodium hypochlorite solution is not hazardous, an automatic sprinkler system and secondary containment are not required.

The design assumptions for dechlorination are essentially the same as those used for the delivered sodium hypochlorite trade solution alternative, including repairs to the chlorine contact tank, except that the concrete masonry will be larger to house the additional equipment involved with this system and the storage tank will be larger as noted above.

To avoid requiring a larger emergency generator, emergency power would not be provided to the onsite generation system, with the exception of the metering pumps. The volume of stored sodium hypochlorite solution should be sufficient to continue disinfection during loss of power. However, if there is a loss of power for an extended period of time and there is insufficient stored solution, concentrated sodium hypochlorite solution delivered in drums could be diluted and used to fill the storage tank.

The onsite generation system would increase the power draw by approximately 60 amps. This would require a larger main breaker for EMCC2 and modifications to the ATS integral to EMCC2. Additionally, the existing motor starters and breakers for the chlorine and SO_2 mixers would need to be replaced to accommodate the new slightly larger mixers. New breakers would be installed in two spare buckets to power the onsite generation system and a new panel board supplying power to the metering pumps and HVAC and lighting for the new building.

11.3.4 UV Disinfection

UV disinfection has become the disinfection technology of choice for many new installations, primarily because UV disinfection does not leave any residuals or create by-products in the effluent and does not involve the significant purchase or handling of chemicals. Additionally, UV disinfection is not susceptible to nitrite lock. During UV disinfection, effluent passes through an open channel or confined chamber containing a collection of UV lamps. Disinfection occurs when UV light from the lamps contacts the microorganism's DNA, inducing photobiochemical changes that prevent normal replication and inactivate the microorganism.

The effectiveness of UV disinfection depends on the amount of UV light absorbed by the microorganisms (i.e., UV dose), which is directly related to the clarity and transmittance of the

wastewater and the time of exposure. Disinfection typically occurs in less than 1 minute, so it requires a much smaller process volume compared to a chlorine contact tank used for chlorination. The systems are typically equipped with automated cleaning systems for the UV lamps. In addition, UV systems monitor UV intensity and lamp status and notify the operator when one or more lamps require replacement.

UV systems can be installed in an open-channel, gravity-flow configuration or in a closed-conduit (i.e., in-pipe) configuration. Selection of the UV system configuration depends on upstream and downstream hydraulic conditions.

An open-channel system is most common when preceded by gravity flow. Typically, level control weirs are installed in an open-channel flow configuration to ensure that the UV lamps stay submerged during low flows to prevent them from overheating and to prevent organic and inorganic residue from drying on the lamp sleeves. A closed-conduit UV system is most often used when preceded by pumped flow. The pumps provide pressure to convey the effluent through the closed-conduit UV system.

As with onsite generation, the cost effectiveness of UV disinfection depends significantly on the local cost of power, given the high energy requirements for this process. Because the capital cost of UV disinfection is almost always higher than chlorine disinfection when a chlorine contact tank already exists, energy must be relatively inexpensive compared to the cost of chemicals for the life cycle cost of UV disinfection to be favorable. However, the cost effectiveness of UV disinfection may vary significantly, depending on the clarity of the effluent, which affects the transmittance of UV radiation. Higher UV transmittance improves the efficiency of the UV system and reduces the UV dose, number of lamps, and energy required to achieve the disinfection requirements.

Because UV disinfection does not leave any residual or generate by-products, it can simplify compliance with permit limits by eliminating the need for dechlorination. However, facilities that use chlorine in other processes must maintain a small chlorine system to serve those needs.

A new open channel concrete structure would be constructed below grade to hold the new UV banks, and power distribution centers for each UV bank. The entire system would be covered by a steel canopy to protect personnel from the elements during maintenance activities. UV disinfection is assumed to deliver at least the minimum required UV dosage at the projected 2036 peak day flow with one UV bank out of service, after applying a conservative value for UV transmittance for the lagoon effluent and third-party validated lamp fouling and end-of-lamp life factors. The conservative UV transmittance value of 50 percent is reflective of the variability of lagoon effluent turbidity due to the seasonal presence of algae.

The size of the UV disinfection system used for this evaluation is conservative in that a simultaneous occurrence of worst-case conditions is assumed for the design parameters. Design parameters include the projected peak day flow, minimum UV transmittance, maximum suspended solids concentration, maximum indicator organism log reduction, maximum quartz sleeve fouling, minimum UV lamp output, and allowances for photo reactivation.

The UV disinfection system must operate without interruption during power outages by using dual power feeds or placing it on essential circuitry powered by an emergency generator. Although the construction cost estimate for this alternative does not include a larger emergency generator, it is possible that programmed load shedding may enable the existing generator to

power a new UV system. However, as discussed in Chapter 9, the City plans to install a second generator at the south end of the WWTP that could be sized for this new load.

11.3.5 Delivered Peracetic Acid Solution System

Peracetic acid (PAA) is a strong oxidant that has been growing in acceptance as a more potent, yet environmentally friendly, alternative to chlorine. Because PAA is a stronger oxidant than chlorine, it requires a significantly lower dose and less contact time to achieve the same level of disinfection. In general, residual PAA does not affect effluent toxicity, so it does not need to be removed as with chlorine, except if there is an unusually high residual (typically greater than 1 mg/L). In the event of a high residual, the effluent can be quenched with sulfur dioxide or sodium bisulfite to reduce the residual, similar to dechlorination.

PAA has been approved by the United States Environmental Protection Agency (EPA) specifically as a wastewater disinfectant. PAA is a clear, colorless liquid with a very low freezing point. The addition of stabilizers to prevent degradation means that it can be stored for a year and experience less than 1 percent decrease in activity. The primary advantages of PAA versus disinfection with various forms of chlorine include:

- No formation of disinfection byproducts
- Quenching typically not required unless an unusually high dose is required, thereby yielding a relatively high residual
- Longer shelf life compared to sodium hypochlorite
- Less contact time required (typically one-half to one-third compared to chlorine)
- Residual diminishes quickly
- Minimal impact on effluent pH and CBOD5.

PAA solution is typically delivered as a 15 or 22 percent solution that is stored in totes. A bulk storage tank and fill station are typically not required due to the relatively small amount of PAA used for disinfection. Because PAA is a hazardous chemical similar to sodium hypochlorite, secondary containment would be provided around the tote storage area to control spills or leaks. Although required PAA doses are much lower compared to chlorine, the cost of PAA is higher than sodium hypochlorite, which offsets the reduced chemical consumption.

The chemical solution is metered directly out of the totes using metering pumps and is typically diluted using carrier water prior to injection. A diffuser or flash mixing device, such as the existing mixers used to mix the chlorine gas solution, is required at the point of delivery to ensure adequate distribution of the chemical. Because PAA is not compatible with plastics, materials used to convey the neat solution must be stainless steel or Teflon.

Two metering pumps, one duty and one standby, would be used to feed the solution to the dosing point. A small prefabricated building would hose the metering pumps and associated equipment. The totes would be stored in an adjacent are outdoors that includes containment and a canopy to shade the totes from sunlight. Throughout the full-scale testing in 2017 and 2018, no quenching of PAA was needed. However, if required, the existing SO2 system could be retained or provisions for dosing sodium bisulfite included for quenching. Costs for this alternative are based on retaining the existing SO2 system, if quenching facilities are required, in which case the existing gas storage room and control room would continue to be used for dosing SO_2 gas. Additionally, the existing chlorine gas valve shutoff system and vacuum alarm switches would be repurposed for the SO_2 system. Although the installation of an automatic

valve shutoff system may negate the need for a gas scrubber, it is assumed that the existing scrubber would be rehabilitated and reused for scrubbing SO_2 gas.

The City conducted full-scale testing of PAA between July 10, 2017 and April 9, 2018, which successfully demonstrated its efficacy and confirmed that the required dose is in line with expectations from previous bench-scale testing. Following an initial period during which the dose was set conservatively high to ensure adequate disinfection until the effectiveness of PAA was demonstrated, the average dose for the remainder of the testing period was about 0.9 mg/L. The temporary PAA system used for testing consisted of a skid mounted dosing system complete with duty/standby metering pumps and appurtenances, a control panel, two containment pallets for storage of PAA totes, PAA residual monitors and chemical diffusers. The new dosing equipment and tote storage would be housed in a pre-engineered metal building on a concrete slab. Because the PAA system would use static chemical diffusers at various locations to seasonally alter the contact time the existing induction mixers would not need to be replaced. However, repairs to the chlorine contact tank are still included.

Changes to the electrical load would include installation of two fractional horsepower metering pumps, and installation of a new panelboard to serve lighting and HVAC loads associated with the new building. This would likely increase the total horsepower draw an equivalent of about 5 horsepower. This would necessitate some modifications to EMCC2 such as adding a breaker to a spare bucket in EMCC2 for service to a new panelboard that would provide power to the metering pumps and HVAC and lighting for the new building.

11.3.6 Summary of Disinfection Alternatives

A summary of the development of the disinfection alternatives is provided in Table 11-5 below.

Table 11-5 Disinfection Alternatives Design Summary[a]

Parameter	Units	Delivered Hypochlorite	Onsite Generation	UV Disinfection	Delivered PAA
Design Flows					
2036 Average Annual Flow	MGD	1.95	1.95	1.95	1.95
2036 Maximum Month Flow	MGD	3.22	3.22	3.22	3.22
2036 Peak Day Flow	MGD	9.42	9.42	9.42	9.42
NPDES Permit					
Fecal Coliform Bacteria					
Monthly Average	cfu/100 mL[b]	200	200	200	200
Weekly Average	cfu/100 mL	400	400	400	400
Total Residual Chlorine					
Monthly Average	µg/L[c]	83	83	NA[d]	NA
Daily Maximum	µg/L	209	209	NA	NA
Design Criteria					
Chlorine Dose					
July - October	mg/L[e]	3.0	3.0	NA	NA
November - June	mg/L	1.5	1.5	NA	NA
SO2 Dose	mg/L	1.0	1.0	NA	NA
PAA Dose	mg/L	NA	NA	NA	1.0
Chlorine Demand					
2036 Peak Day Flow[f]	ppd[g]	113	113	NA	NA
2036 Average Annual Flow[h]	ppd	32	32	NA	NA
SO_2 Demand					
2036 Peak Day Flow	ppd	75	75	NA	NA
2036 Average Annual Flow	ppd	16	16	NA	NA
PAA Demand[i]					
2036 Peak Day Flow	ppd	NA	NA	NA	75
2036 Average Annual Flow	ppd	NA	NA	NA	16
Contact Time at Peak Day Flow	minutes	22	22	NA	22
UV Transmittance	%	NA	NA	50[j]	NA
Minimum UV Dosage at PDF	mJ/cm^2[k]	NA	NA	40	NA
Lamp Life Factor	-	NA	NA	0.9[l]	NA
Sleeve Fouling Factor	-	NA	NA	0.9[l]	NA

Table 11-5 Disinfection Alternatives Design Summary[a]

Parameter	Units	Delivered Hypochlorite	Onsite Generation	UV Disinfection	Delivered PAA
Physical Information					
New Building Footprint	sf	300	374	None	80
Covered Area Footprint	sf	None	None	600[m]	80[m]
# of Chambers/Channels	Each	2[n]	2[n]	1[o]	2[n]
New Channel Footprint	sf	None	None	260	None
Number of Mixing Units[p]	Each	2	2	NA	2
Number of Metering Pumps	Each	2(1)[q]	2(1)	NA	2(1)
Number of Sulfonators	Each	2(1)	2(1)	NA	2(1)
# of UV Banks/Channel	Each	NA	NA	4(3)	NA
Number of SO_2 Cylinders[r]	Each	4(2)	4(2)	NA	NA
Onsite Cl_2 production rate	ppd	NA	200	NA	NA
Storage Tank Volume	gallons	1,500[s]	6,500[t]	NA	300[u]

Notes:

(a) Summary of parameters used to develop opinions of probable cost.
(b) cfu/100 mL – colony forming units per 100 milliliters.
(c) µg/L – micrograms per liter.
(d) NA – not applicable.
(e) mg/L - milligrams per liter.
(f) Based on weighted average of seasonal chlorine doses.
(g) ppd – pounds per day.
(h) Based on chlorine dose of 1.5 mg/L during wet weather months.
(i) Assumed average PAA dose of 1.0 mg/L based on results of full-scale testing.
(j) Conservative transmittance provided for lagoon effluent considering potential greater variations in TSS comparing to conventional activated sludge system with secondary clarifiers. Higher UV transmittance could result in a significant reduction (about 30 percent) in the equipment cost.
(k) mJ/cm^2 – millijoules per square centimeter.
(l) Third-party validated factors provided by one UV vendor.
(m) Covered area with canopy.
(n) Existing chlorine contact chambers.
(o) New UV disinfection channel.
(p) One for chlorine injection and the other for SO_2 injection.
(q) Total number of units including backup or spare with number of units normally in service or on-line in parentheses.
(r) Maximum withdrawal from two on-line cylinders = 80 ppd.
(s) 33 days of supply at demand under 2036 MMF conditions.
(t) 3.8 days of supply at peak demand under 2036 peak day flow conditions.
(u) Assume PAA delivered in 300 gallon totes, each estimated to last about 1 month.

11.3.7 Disinfection Probable Construction and O&M Costs

An opinion of probable construction cost was prepared for each alternative by using a combination of published cost indexes, cost estimating guides, vendor quotations, and bids or estimates from similar facilities at other treatment plants. The estimates provided in Table 11-6 include markups for site work, mechanical/yard piping, electrical, instrumentation and control, mobilization, contractor overhead and profit, contingency, and sales tax.

An estimate of the annual O&M costs associated with each alternative was prepared using published O&M cost information, desktop estimates of energy and chemical requirements based on typical operation, vendor estimates of labor to perform routine maintenance activities, vendor-supplied information on parts cost and replacement interval, and labor estimates for operations from similar facilities at other treatment plants. Replacement parts cost only include major items that are expected to be replaced within the planning horizon. Items such as pumps are expected to last past the planning horizon. Minor replacements, such as hoses and valves, are not included. O&M cost estimates apply to the evaluated alternative only. Administration, laboratory work, energy, chemical usage, labor, and maintenance associated with other processes and facilities at the WWTP are not included in these estimates.

These estimates are conceptual and should therefore be considered to have an accuracy of +50 to -30 percent, consistent with order-of-magnitude estimates as defined by the American Association of Cost Engineers (AACE).

Table 11-6 Disinfection Alternatives Cost Summary[(a)]

Parameter	Units	Delivered Hypochlorite	Onsite Generation	UV Disinfection	Delivered PAA
Average Energy Use					
Yearly Energy Consumption	kWh/year	65,350	153,872	272,436	65,350
Energy Unit Cost	$/kWh	0.075	0.075	0.075	0.075
Yearly Energy Cost[(b)]	$/year	4,900	11,500	20,400	4,900
Labor					
Monthly Labor Requirements	hrs/month	20	20	30	12
Labor Unit Cost	$/hour	50	50	50	50
Yearly Labor Cost	$/year	12,000	12,000	18,000	7,200
Average Chemical Use					
Sodium Hypochlorite Use	gpd	30	476	NA	NA
PAA Use	gpd	NA	NA	NA	11
Salt Use	ppd	NA	95	NA	NA
SO_2 Use	ppd	16	16	NA	NA
Chlorine/Salt Cost	$/year	21,200	4,400	NA	NA
SO_2 Cost	$/year	12,700	12,700	NA	NA
PAA Cost	$/year	NA	NA	NA	41,000
Yearly Chemical Cost[(b)]	$/year	33,900	17,100	0	41,000

Table 11-6 Disinfection Alternatives Cost Summary[a]

Parameter	Units	Delivered Hypochlorite	Onsite Generation	UV Disinfection	Delivered PAA
Replacement					
Annual Replacement Parts Cost	$/year	1,400[c]	5,800[c]	20,200	0
Annual O&M Cost[d,e]	**$/year**	**52,200**	**46,400**	**58,600**	**49,000**
Process and Equipment	$	202,000	359,300	605,400	224,400
Structures	$	512,700	394,300	112,900	286,200
Other related work[f]	$	250,100	263,800	251,400	178,700
Mobilization and OH&P	$	144,700	152,600	145,500	103,400
Contingency and Taxes	$	526,700	554,900	529,300	376,100
Probable Construction Cost[g,h]	**$**	**1,638,000**	**1,726,000**	**1,646,000**	**1,170,000[i]**
Probable Project Cost (2013) [g,l]	**$**	**2,130,000**	**2,244,000**	**2,140,000**	**N/A[i]**
Probable Project Cost (2018) [g,l]	**$**	**2,320,000**	**2,444,000**	**2,331,000**	**1,463,000**
20-Year O&M Cost (2018) [g,j]	**$**	**777,000**	**690,000**	**872,000**	**729,000**
20-Year NPV[g,k]	**$**	**3,097,000**	**3,134,000**	**3,203,000**	**2,192,000**

Notes:

(a) All costs are in 2013 dollars unless specified. See Appendix I for further breakdown of construction and O&M estimates.
(b) Yearly energy and chemical consumption estimates are based on operation at average annual flow.
(c) Electrolytic cells and storage tanks are assumed to be replaced every 10 years and replacement cost is converted to a yearly cost for comparison.
(d) Sum of energy, labor, chemical and replacement costs.
(e) Cost rounded to nearest hundred dollars.
(f) Other related work includes site work, special foundations, yard piping, electrical, and instrumentation
(g) Cost rounded to nearest thousand dollars. The totals presented may not appear to add up to the components listed due to rounding.
(h) Original estimate of probable construction cost is in 2013 dollars.
(i) Already in 2018 dollars and excludes markup for planning, as there has already been an engineering report prepared based on full-scale testing.
(j) Assumes 3 percent effective interest (interest less inflation) per year for present value of future annuity calculation for the 20-year life cycle period.
(k) Sum of project and 20-year O&M cost.
(l) The project cost includes a 30% markup added to the construction cost to cover planning, design and construction services during the project.

Onsite generation of sodium hypochlorite has the lowest O&M cost, but PAA has the lowest probable construction cost and net present value. This is due partly to the fact that the PAA alternative involves a simpler and smaller building. UV disinfection has both the highest probable construction cost and O&M cost.

11.3.8 Comparison of Disinfection Alternatives

The advantages and disadvantages of each evaluated alternative are listed in Table 11-7. Comparative scoring is provided in Table 11-8. Each alternative was scored for each criterion in comparison with the other evaluated alternatives. The most favorable, neutral, and least favorable alternatives were scored as +, 0 and -, respectively. Justifications for scoring qualitative criteria are based on the advantages and disadvantages summarized in Table 11-7. The total score in Table 11-8 is provided only as a quick overview of the scoring. The scoring includes some weighting of criteria such that each one does not contribute equally to the overall score. For example, the score for capital cost is multiplied by a factor of 3, whereas the score for footprint is multiplied by a factor of only 1.

Table 11-7 Disinfection Alternatives Advantages and Disadvantages

Parameter	Delivered Hypochlorite	Onsite Generation	UV Disinfection	Delivered PAA
Footprint	• Medium footprint for new building accommodating 1,500-gallon storage tank.	• Larger footprint for new building accommodating 6,500-gallon storage tank.	• Medium footprint for new channel located below grade. • Larger footprint for covered process area.	• Smallest building size and footprint.
Operational & Environmental Safety	• Regular handling and storage of hazardous substances. • Occasional air binding potential due to off-gassing. • Secondary containment and sprinkler required. • Hazmat training required. • Potential residual harmful to aquatic life.	• No handling or storage of toxic substances. • No hazardous materials reporting requirements. • Periodic need to clean electrodes with muriatic acid. • Potential residual harmful to aquatic life.	• No handling or storage of toxic substances. • No hazardous materials reporting requirements. • No disinfection byproducts or residual. • Periodic lamp replacement and sleeve cleaning. • Potential exposure to electrical hazards or UV irradiation.	• Regular handling and storage of hazardous substances. • Secondary containment and sprinkler required. • Hazmat training required. • No disinfection byproducts • Typical residuals not harmful to aquatic life.
Performance	• Probability exists to exceed permitted chlorine limit. • Sensitive to nitrite lock, although has not been an issue with operation of the SFF media system. • Hardness and UV-absorbing compounds not an issue. • Requires long contact time.	• Probability exists to exceed permitted chlorine limit. • Sensitive to nitrite lock, although has not been an issue with operation of the SFF media system. • Hardness and UV-absorbing compounds not an issue. • Requires long contact time.	• No issue meeting limit for residual chlorine. • No issue with nitrite lock. • Hardness and UV-absorbing compounds reduce efficacy, although not expected to be an issue. • Shortest contact time required.	• No issue with chlorine limit, but may need to occasionally quench if PAA residual high. • No issue with nitrite lock. • Hardness and UV-absorbing compounds not an issue. • Shorter contact time required.
Complexity	• Fewest components and low technical complexity. • Similar to current alkalinity dosing system. • Automated control, but some monitoring and adjustments.	• Greatest number of components and high technical complexity. • New technology to WWTP staff. • Automated control with some occasional monitoring and adjustments.	• Fewer components, but with high technical complexity. • New technology to WWTP staff. • Automated control with least monitoring and adjustments.	• Fewest components and low technical complexity. • Similar to current alkalinity dosing system. • Automated control, but some monitoring and adjustments.
Expandability	• Additional chlorine contact volume and storage tank volume required for further expansion.	• Additional chlorine contact volume, electrolytic cells, and storage tank volume required for further expansion.	• Additional UV channel and UV banks required for further expansion.	• Short contact time requirement increases capacity of existing contact tank.

Table 11-8 Disinfection Alternatives Comparison

Parameter	Delivered Hypochlorite	Onsite Generation	UV Disinfection	Delivered PAA	Weighting Factor[(a)]
Capital Cost	0	0	0	+	x3
O&M Cost	0	+	-	0	x2
Footprint	0	-	-	0	x1
Safety	-	+	+	0	x1
Performance	0	0	+	+	x1
Complexity	+	-	0	+	x1
Expandability	0	0	0	+	x1
Total Score[(a)]	**0**	**+1**	**-1**	**+6**	

Note:
(a) Total score calculated by summing the products of the weighting factor and score for each parameter.

11.3.9 Disinfection Recommendation

Based on the evaluation and scoring of alternatives, disinfection using delivered PAA solution is the apparent best alternative and is recommended for implementation given the successful completion of full-scale PAA testing. This alternative has the lowest probable construction cost and an estimated O&M cost that is only slightly higher than onsite generation. It also provides operational simplicity and receives favorable scores for performance and expandability. Because the system configuration would be similar to the existing alkalinity feed system, the WWTP staff would already be familiar with the system components.

11.4 Aeration Alternatives

As discussed in Chapter 9, the existing floating surface aerators do not provide enough air to meet the projected oxygen demand for 2036. This deficiency needs to be addressed by increasing the available oxygen supply. This will be accomplished by upgrading and replacing the existing surface aerators in the large completely mixed lagoon (Lagoon 1), which provides most of the oxygen required for treatment. The aeration alternatives discussed below were identified as reasonable solutions to upgrade and replace the existing floating surface aerators.

Applicable design and redundancy requirements from the Orange Book that were considered in this evaluation are as follows:

- There shall be a sufficient number of blowers or mechanical aerators to enable the design oxygen transfer to be maintained with the largest-capacity-unit out of service and at least two units shall be installed. (G2-8.2, Orange Book, Ecology 2008).

For consistency with this requirement, the aeration alternatives considered were sized to supply air meeting the projected oxygen demand with one aerator or blower out of service. Other assumptions applied to the aeration alternatives include:

- Lagoon Temperature of 5 -28 degrees Celsius.
- Alpha (α) factor of 0.85.
- Assume Beta (β) factor of 0.95.

- Provide an actual oxygen transfer rate of 305 pounds per hour (lbs/hr) meeting the estimated oxygen demand at a maximum month flow 0f 3.30 MGD with a dissolved oxygen residual of 2.0 mg/L at the maximum monthly average lagoon temperature of 28 degrees Celsius, based on the above temperatures and factors. Also, meet this same oxygen transfer rate with a dissolved oxygen residual of no less than 1.7 mg/L with one aerator out of service. These calculations are based on an oxygen transfer rate of 142 lbs/hr total provided by the existing floating surface aerators and submerged fixed film media in Lagoons 2, 3 and 4.
- Provide sufficient energy to "completely" mix Lagoon 1.

11.4.1 Evaluation Criteria

The evaluation and comparison of filtration alternatives considered the following criteria:

- Capital Construction Cost – opinion of probable construction cost.
- 20-Year O&M Cost – present worth of the annual O&M cost (energy, labor, and maintenance).
- Footprint – estimate of the space required and an assessment of how easily the system may fit within the available space.
- Performance – ability to efficiently meet the estimated oxygen demands and consideration of reliability.
- Complexity – assessment of system O&M complexity in normal and failure modes. Consideration was also given to the number of physical and mechanical processes involved.
- Expandability – ease with which the process could be expanded or modified to accommodate additional flow and load or comply with more stringent effluent limitations.

11.4.2 Floating Surface Aerators

One obvious alternative is to replace the existing floating surface aerators with newer and more efficient floating surface aerators. This would involve removing and replacing the existing aerators in kind. Although the City indicated they recently replaced wiring to the existing surface aerators that could be reused and the existing mooring cables and posts could also be reused, manufacturers suggested considering reconfiguration of aerator placement to maximize performance.

Two manufacturers (Aerator Solutions and Aqua-Aerobics) were contacted to provide input on cost and sizing for new floating surface aerators. In both cases, the manufacturer recommended replacing the eighteen (18) existing 15-horsepower floating surface aerators with eighteen (18) new 15-horsepower floating surface aerators, though at least one manufacturer did recommend a slightly altered configuration. Also, both manufacturers indicated that the requirements for complete mixing (requiring from 25 to 30 horsepower per million gallons of volume) controlled the size and number or aerators, rather than the oxygen requirement.

11.4.3 Floating Aspirating Aerators

Floating aspirating aerators are similar to floating surface aerators in that they float on the surface of the lagoon and are fixed in place with mooring cables. However, they utilize a different method to introduce oxygen into the wastewater. Floating surface aerators typically use a propeller and vertical draft tube pull wastewater into the unit to circulate and agitate the

wastewater. This agitation increases the air/water interface, thereby increasing the transfer of oxygen into the wastewater, and the circulation increases mixing. Floating aspirating aerators work somewhat in reverse. Water moves at a high velocity through and near the propeller at the end of an angled draft tube. This creates a low pressure zone that draws air down the angled draft tube. The air exits into the water at the propeller, which generates turbulence that breaks up the air bubbles to enhance oxygen transfer and also generates flow that disperses the oxygen and provides mixing.

RWL Water, LLC (RWL) was contacted to provide input on cost and sizing for floating aspirating aerators. RWL recommended replacing the eighteen (18) existing 15-horsepower floating surface aerators with eighteen (18) new 20-horsepower floating aspirating aerators. The floating aspirating aerators were estimated to have a lower oxygen transfer efficiency compared to the floating surface aerators, which is why they required a higher horsepower.

11.4.4 Diffused Aeration

Two types of diffused aeration were considered for this application. The first type utilizes floating laterals, from which submerged diffusers assemblies that deliver air into the lagoon are suspended. The floating laterals are connected to an onshore air header pipeline that is supplied air from blowers. There are a number of manufacturers that offer this type of diffused aeration system. One such manufacturer (Nexom) was contacted and asked about cost and sizing of a system for this application. Nexom responded that due to the size of Lagoon 1 the mixing requirements would outweigh the aeration requirements, so mechanical aeration makes the most sense. Therefore, an alternative for this type of diffused aeration was not developed further. Additionally, this type of diffused aeration system would have required a building to house the blowers.

The second type of diffused aeration considered involves a floating unit consisting of two small blowers connected to separate coarse bubble and fine bubble diffusers, as manufactured by Reliant Water. The coarse bubble diffuser and blower provide circulation for mixing and a significant amount of the air required for treatment. The fine bubble diffuser and blower provide additional air for treatment. The positioning of the diffusers is such that the coarse bubbles are above the fine bubbles and help to increase their pathway and retention in the wastewater for greater oxygen transfer. Although these units appear to have an improved mixing efficiency, such that fewer units would be required for mixing compared with the floating surface aerators, the small size of the blowers and diffusers requires 48 units to meet the projected oxygen demand.

11.4.5 Venturi Injector

A vendor had inquired about applying a Venturi style injector to aerate and mix the lagoon. This system is somewhat similar to an aspirating aerator, except rather a pump is used to provide motive force instead of a propeller. The pumped liquid passes through a Venturi mixer which draws air into the flow stream and then this mix of air and water is dispersed through nozzles. After providing information and requirements for the application, the vendor did not bring forth a proposal, suggesting that they determined this technology was not cost-effective for this application. Therefore, an alternative was not developed further for this technology.

11.4.6 Summary of Aeration Alternatives

A summary of the development of the aeration alternatives is provided in Table 11-9 below.

Table 11-9 Aeration Alternatives Design Summary[a]

Parameter	Units	Aerator Solutions	Aqua-Aerobics	RWL	Reliant Water
Design Oxygen Transfer					
Dissolved Oxygen Residual	mg/L[b]	2.0	2.0	2.0	2.0
Design Oxygen Demand	lbs/hr[c]	447	447	447	447
Oxygen Supply Lagoons 2-4	lbs/hr	142	142	142	142
Net Oxygen Demand Lagoon 1	lbs/hr	305	305	305	305
Design Mixing Requirement					
Lagoon 1 Volume	MG[d]	10	10	10	10
Mixing Requirement	HP/MG[e]	25-30	25-30	25-30	NA[f]
Physical Information					
Aerator Model		EcoJet	AquaJet FSS	Tornado	Lagoon Master
Aerator Speed		High	High	High	NA
Number of Aerators		18	18	18	48
Number of Motors		18	18	18	96
Motor Size	HP[g]	15	15	20	2
Manufacturer SOTE	lbs/HP/hr	3.0	3.0	2.5	3.8
Design SOTE[h]	lbs/HP/hr[i]	2.0	2.0	1.7	2.8
Total Power	HP	270	270	360	192
Motor Efficiency	%	92%	92%	92%	92%
Power Required for Aeration	HP	270	270	318	192
Mixing Energy	HP/MG	27	27	36	NA
Materials		Stainless, epoxied steel, fiberglass	Stainless	Stainless	Aluminum

Notes:
(a) Summary of parameters used to develop opinions of probable cost.
(b) mg/L – milligrams per liter.
(c) lbs/hr – pounds per hour.
(d) MG – million gallons
(e) HP/MG – horsepower per million gallons
(f) NA – not applicable.
(g) HP – horsepower.
(h) Applied 25-33% reduction to value provided by manufacturer to account for degradation over planning horizon.
(i) lbs/HP/hr – pounds per horsepower per hour.

Construction cost estimates were prepared for each alternative by using a combination of commercial cost estimating software, published cost indexes, cost estimating guides, vendor quotations, and bids or estimates from similar facilities at other treatment plants. The estimates

provided in Table 11-10 include markups for site work, mechanical/yard piping, electrical, instrumentation and control, mobilization, contractor overhead and profit, contingency, and sales taxes.

11.4.7 Aeration Probable Construction and O&M Costs

An opinion of probable construction cost was prepared for each alternative by using a combination of published cost indexes, cost estimating guides, vendor quotations, and bids or estimates from similar facilities at other treatment plants. The estimates provided in Table 11-10 include markups for installation, mobilization, contractor overhead and profit, contingency, and sales tax.

An estimate of the annual O&M costs associated with each alternative was prepared using published O&M cost information, desktop estimates of energy requirements based on typical operation, vendor estimates of labor to perform routine maintenance activities, and labor estimates for operations from similar facilities at other treatment plants. O&M cost estimates apply to the evaluated alternative only. Administration, laboratory work, energy, chemical usage, labor, and maintenance associated with other processes and facilities at the WWTP are not included in these estimates.

These estimates are conceptual and should therefore be considered to have an accuracy of +50 to -30 percent, consistent with order-of-magnitude estimates as defined by the American Association of Cost Engineers (AACE).

Table 11-10 Aeration Alternatrives Cost Summary[a]

Parameter	Units	Aerator Solutions	Aqua-Aerobics	RWL	Reliant Water
Average Energy Use					
Yearly Energy Consumption	kWh/year	1,763,730	1,763,730	2,351,640	1,254,208
Energy Unit Cost	$/kWh	0.075	0.075	0.075	0.075
Yearly Energy Cost[b]	$/year	132,280	132,280	176,373	94,066
Labor					
Monthly Labor Requirements[c]	hrs/month	12	12	12	32
Labor Unit Cost	$/hour	50	50	50	50
Yearly Labor Cost	$/year	7,200	7,200	7,200	9,600
Total Yearly O&M Cost[d,e]	**$/year**	**139,500**	**139,500**	**183,600**	**113,300**
Equipment	$	188,000	172,000	239,076	854,208
Installation Allowance	$	47,000	43,000	59,769	213,552
Mobilization and OH&P	$	35,250	32,250	44,827	160,164
Contingency and Taxes	$	128,153	117,246	162,969	582,282
Probable Construction Cost[f]	**$**	**398,000**	**364,000**	**507,000**	**1,810,000**
Probable Project Cost[i]	**$**	**518,000**	**474,000**	**659,000**	**2,353,000**
20-Year O&M Cost[f,g]	**$**	**2,075,000**	**2,075,000**	**2,731,000**	**1,685,000**
20-Year NPV[h]	**$**	**2,593,000**	**2,549,000**	**3,390,000**	**4,038,000**

Table 11-10 Aeration Alternatrives Cost Summary[(a)]

Parameter	Units	Aerator Solutions	Aqua-Aerobics	RWL	Reliant Water

Notes:
(a) All costs are in 2017 dollars. See Appendix I for further breakdown of construction and O&M estimates.
(b) Yearly energy consumption estimates are based on all units running for complete mixing/aeration.
(c) Based on 8 hours per year per aerator unit.
(d) Sum of energy and labor cost.
(e) Cost rounded to nearest hundred dollars. The totals presented may not appear to add up to the components listed due to rounding.
(f) Cost rounded to nearest thousand dollars. The totals presented may not appear to add up to the components listed due to rounding.
(g) Assumes 3 percent effective interest (interest less inflation) per year for present value of future annuity calculation for the 20-year life cycle periods.
(h) Sum of project cost and O&M cost.
(i) The project cost includes a 30% markup added to the construction cost to cover planning, design and construction services during the project.

Both floating the Aerator Solutions and Aqua-Aerobics surface aerator alternatives have a similarly low probable construction cost and net present value. Although the Reliant Water diffused aeration alternative has the lowest annual O&M cost, it has a substantially higher probable construction cost and net present value due to the number and cost of the diffused air units.

11.4.8 Comparison of Aeration Alternatives

An overview of advantages and disadvantages associated with each alternative is provided in Table 11-11. Comparative scoring is provided in Table 11-12. Each alternative was scored for each criterion in comparison with the other evaluated alternatives. The most favorable, neutral, and least favorable alternatives were scored as (+), 0, and (-), respectively. Justifications for scoring qualitative criteria are based on the advantages and disadvantages summarized in Table 11-11. The total score in Table 11-12 is provided only as a quick overview of the scoring. The scoring includes some weighting of criteria such that each one does not contribute equally to the overall score. For example, the score for capital cost is multiplied by a factor of 3, whereas the score for footprint is multiplied by a factor of only 1.

Table 11-11 Aeration Alternative Advantages and Disadvantages

Parameter	Aerator Solutions (Floating Surface Aerator)	Aqua-Aerobics (Floating Surface Aerator)	RWL (Floating Aspirating Aerator)	Reliant Water (Diffused Aeration)
Footprint	• All units are contained within the footprint of Lagoon 1.	• All units are contained within the footprint of Lagoon 1.	• All units are contained within the footprint of Lagoon 1.	• All units are contained within the footprint of Lagoon 1.
Performance	• Higher oxygen transfer efficiency. • No significant increase in mixing efficiency. • Lower total horsepower. • Same number of units required as current installation.	• Higher oxygen transfer efficiency. • No significant increase in mixing efficiency. • Lower total horsepower. • Same number of units required as current installation.	• Lower oxygen transfer efficiency. • No significant increase in mixing efficiency. • Highest total horsepower. • Same number of units required as current installation.	• Highest oxygen transfer efficiency. • Increase in mixing efficiency. • Lowest total horsepower. • Almost 3 times as many units required compared to current installation.
Complexity	• Same as existing installation.	• Same as existing installation.	• Essentially same as existing installation.	• Greater complexity with many more motors, dual blowers and dual coarse and fine bubble diffusers.
Expandability	• Additional and/or larger aerators could be installed in Lagoon 1.	• Additional and/or larger aerators could be installed in Lagoon 1.	• Additional and/or larger aerators could be installed in Lagoon 1.	• Less ability to accommodate additional units due to the number of units required to be installed initially. • Larger units currently not available.

Table 11-12 Aeration Alternatives Comparison

Parameter	Aerator Solutions	Aqua-Aerobics	RWL	Reliant Water	Weighting Factor
Capital Cost	+	+	0	-	x3
O&M Cost	0	0	-	+	x2
Footprint	+	+	+	+	x1
Performance	0	0	-	+	x1
Complexity	+	+	+	0	x1
Expandability	+	+	+	-	x1
Total Score[(a)]	**+6**	**+6**	**+1**	**0**	

Note:
(a) Total score calculated by summing the products of the weighting factor and score for each parameter.

11.4.9 Aeration Recommendation

Based on the evaluation and scoring of alternatives, replacing and upgrading the existing floating surface aerators with new floating surface aerators is the apparent best alternative and is recommended for implementation. This alternative has the lowest probable construction cost, lowest net present value and second lowest estimated annual O&M cost. It also provides operational simplicity and receives favorable scores for footprint and expandability. Although it is not rated as highly for performance as diffused aeration, the new floating surface aerators are expected to have a significantly higher oxygen transfer efficiency compared to the existing aerators due to improvements in the technology and the fact that the existing aerators are worn and less efficient than they were initially. As the new aerators will be very similar to the existing aerators in their components and functionality, the WWTP staff will already be very familiar with this technology.

Chapter 12 Recommended Improvements

12.1 Introduction

This chapter provides a compilation of specific projects, improvements, and programs the City should implement, providing the tools necessary for long-range project planning and budgeting. These projects are derived from analyses of the collection system and WWTP and discussions with the City's operations and engineering staff. Each project is accompanied by a planning level opinion of probable project cost (OPPC) and is scheduled as either part of the 6-year or Long-Term Capital Improvement Plan (CIP). The City should review the CIP plan periodically to adjust for significant changes in the priority of each project, its cost, and scope.

Improvements identified in this section and Plan are organized into categories for collection (C), pumping (P) and wastewater treatment (W). The need for each improvement project is classified among one or more of the following five categories:

- **Capacity:** Improvements classified as insufficient in capacity are determined based on whether or not the infrastructure can effectively convey or treat the incoming flow or load. Gravity sewer pipes are considered to have insufficient capacity when the surcharging depth is within five feet of the rim of the manhole, or equal to the pipe diameter (d/D of 1) for manholes less than 5 feet deep. Force mains are considered to have insufficient capacity when the velocities exceed 8 feet per second. Pump stations are considered to have insufficient capacity when firm capacity is exceeded causing the backup pump to turn on. Treatment processes are considered deficient if they cannot treat the incoming load or pass the incoming flow. As described in Chapter 7, the conveyance system was evaluated using existing flows and flows projected for 2022, 2036, and buildout conditions, including changes expected to occur in the system, such as stormwater separation along Avenue F prior to 2022. The evaluations determined system deficiencies when subjected to these existing and future flow conditions. Following identification of system deficiencies, the computer model was used to evaluate and select system improvements to alleviate the system deficiencies.

- **Obsolescence:** Improvements classified as obsolete are based on the age of the infrastructure. Pump station mechanical and electrical equipment is expected to have a typical usable life of 20 to 25 years. WWTP mechanical and electrical equipment such as panels and switchgear are expected to have a typical service life of 20 to 25 years. Instrumentation and controls are expected to have a service life of 15 to 20 years. Structures are expected to have a typical usable life of 50 years. Pipes are expected to have a typical usable life of 100 years.

- **Operations & Maintenance (O&M):** O&M projects will replace facilities identified by the City staff as having unacceptably high maintenance requirements, both in terms of frequency and in magnitude, or poor performance.

- **Developer:** Projects identified as developer dependent are needed to serve new developments but are not needed to provide continuation of service to existing customers.

- **General:** General improvement projects are those identified by City staff for various reasons that do not fall within any of the previous four categories. These projects may be needed to simplify system operation, consolidate and/or eliminate redundant facilities, reduce or eliminate non-critical O&M concerns, or to meet ongoing sewer system management needs.

Projects are also identified as either "Replacement", "Upgrade", or "Expansion" or a combination of the three. This gives a quick indication of the project type when considering the appropriate funding source.

- **Replacement:** These projects are generally intended to replace like infrastructure with like, they are typically the result of obsolete equipment that has exhausted its useful life or creates excessively high maintenance.

- **Upgrade:** These projects are normally targeted at reducing maintenance or improving operations. This may include new equipment or a replacement of equipment that is still functional but operating inefficiently.

- **Expansion:** These projects can include new equipment or a replacement of equipment, but their driving force is to provide additional capacity for growth.

When possible, sewer system improvement projects should be coordinated with other utility and roadway improvements to minimize disruption and reduce associated costs such as road and surface restoration. Due to the number of projects scheduled in the 6-year CIP, the City should periodically evaluate its progress in completing those projects and determine if current staffing levels and funding are adequate to complete those projects.

12.2 Capital Improvement Program (CIP)

The CIP is presented for two timeframes:

- 6-Year CIP from 2020 to 2025
- Long-Term CIP from 2026 to 2036

Though the model used 2022, 2036 and buildout populations for the hydraulic analysis, the City requested that the 6-year CIP start in 2020. Although the 6-year CIP at the start of this planning effort was planned to be 2017 to 2022, shifting the timeline for the 6-year CIP by 3 years does not affect the proposed 6-year CIP projects since the population growth over three years does not significantly affect the model results for system deficiencies.

12.2.1 6-Year CIP (2020 – 2025)

The projects recommended for the 6-year CIP are described in Table 12-1 and illustrated on Figure 12-1. Improvements that would be completed by developers are expected to be privately funded and so are not listed in this section. Project priorities were developed with input from the City.

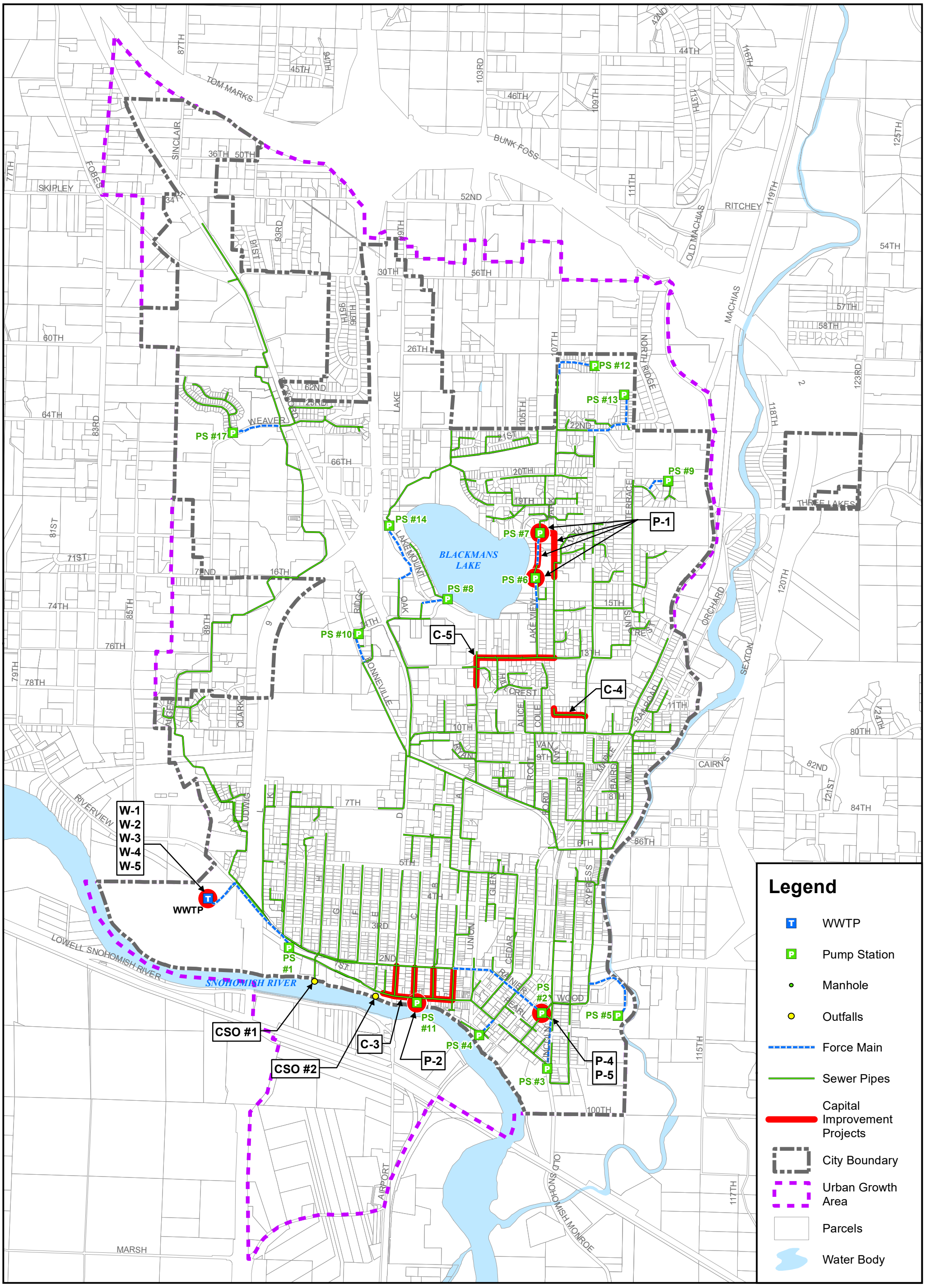
Table 12-1 6-Year Capital Improvement Plan (2020-2025)

CIP No.	Project	Type	Priority Rank	Replacement	Upgrade	Expansion	Project Description
C-1	Capital Reserve	O&M / Obsolescence	1	☑	☐	☐	General roll-over fund to handle unanticipated projects
C-2	General Conveyance System Repair and Replacements (2020-2025)	O&M / Obsolescence	1	☑	☐	☐	Annual allowance for general sewer conveyance repairs and replacements Projects typically accompany annual Road Preservation Projects
C-3	CSO 2 Separation Program	Capacity	3/4	☐	☑	☐	Annual allowance to install stormwater pipes and separate the CSS in the CSO Pump Station basin
C-4	Emerson Street Utility Improvement	Capacity	1	☑	☐	☐	Re-connect side sewers to non-failed sewer main in Emerson Street
C-5	Upsize Gravity Main at 13th Street and Avenue A	Capacity	4	☑	☐	☐	Upsize approximately 1,600 ft. of pipe from 13th St and Park Ave to south of Ave A and Smithson Pl.
C-6	Western Gravity Flow Diversion	Capacity	4	☐	☐	☑	Planning for the Western Diversion, project to be constructed in the Long-Term CIP
P-1	Champagne Pump Station (PS 7) Upgrade and New Force Main	Capacity	1	☐	☑	☑	Upgrade Champagne Pump Station (PS 7) New Force Main from Champagne Pump Station to Park Ave Decommission Hill Park Pump Station (PS 6)
P-2	Kla-Ha-Ya Pump Station (PS 11) Decommissioning	Obsolescence	1	☐	☑	☐	Decommission the Kla-Ha-Ya Pump Station
P-3	Mobile 80 kW Generator	Redundancy	1	☑	☑	☐	Replace City's trailer mounted 80 kW mobile power generator
P-4	Rainier Pump Station (PS 2) Generator	O&M	2	☐	☑	☐	Install a permanent 100 kW generator set including housing and equipment pad
P-5	Rainier Pump Station (PS 2) Upgrades	Capacity / Obsolescence	2	☑	☑	☐	Replace piping and add flow metering Replace pumps to restore capacity Replace instrumentation Rehabilitate wet well

Table 12-1 6-Year Capital Improvement Plan (2020-2025)

CIP No.	Project	Type	Priority Rank	Replacement	Upgrade	Expansion	Project Description
W-1	WWTP NPDES Re-rating	Capacity	2	☐	☐	☑	Re-rate the WWTP's max month flow and BOD5 limits on the NPDES permit
W-2	Upgrade/Expand Effluent Filtration	O&M / Capacity	2/3	☐	☑	☑	Upgrade and expand the existing effluent sand filters with the EcoWash system or convert to Fuzzy Filters
W-3	General WWTP Improvements (2020-2025)	O&M / Obsolescence / Capacity	3	☑	☐	☑	Annual allowance for general WWTP repairs, including aerator replacement
W-4	Replace the Chlorine Gas Disinfection System	O&M / Obsolescence	2	☑	☑	☐	Replace the current chlorine gas system with a peracetic acid (PAA) system
W-5	Add Non-Potable Water Air Gaps	O&M	1	☐	☑	☐	Add non-potable water air gaps for cross-connection control

Priority Ranks are as follows:
1) Out of regulation with DOE - potential regulatory issues or needed immediately
2) On the verge of being out of regulation with DOE - high probability
3) Medium risk - medium probability
4) (4) Low risk - low probability



Sewer System & Basins: City of Snohomish.
GIS Base data: Snohomish County.
Data sources supplied may not reflect current or actual conditions. This map is a geographic representation based on information available. It does not represent survey data. No warranty is made concerning the accuracy, currency, or completeness of data depicted on this map.
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6-Year Capital Improvement Projects

General Sewer Plan and
Wastewater Facilities Plan
City of Snohomish, Washington
February 2019

Figure
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12.2.2 6-year CIP Project Descriptions

CIP C-1 – Capital Reserve

This reserve of funds is meant to cover the expense of unanticipated projects (e.g., FEMA levy improvements, regulatory requirements, etc.) and not general maintenance and repairs. The funds are available in the first year of the 6-year CIP and whatever amount is not used rolls over into the next year.

CIP C-2 – General Conveyance Systems Repairs and Replacements (2020 – 2025)
This allowance is for general sewer conveyance repairs and replacements. The projects typically accompany annual Road Preservation Projects. The City often replaces old sewer pipes during road renovation projects to minimize project costs in anticipation of future pipe failures in the 70+ year old pipes.

CIP C-3 – CSO 2 Separation Program

The CSO 2 Separation Program involves installing stormwater pipes in the portion of the CSS tributary to the CSO Pump Station (PS 1). The stormwater in those areas would be diverted away from the sewer system and into the stormwater system that conveys stormwater to the stormwater utility lagoon adjacent to the WWTP. This program includes separation of storm and sanitary sewers along 1st Street and Avenues A, B, C and D between 1st Street and 2nd Street. In addition to installing new stormwater pipelines, the existing sewer pipelines will also be replaced with new sanitary sewer pipelines. Stormwater will be conveyed to an existing stormwater wet well at the CSO Pump Station. An existing force main would transfer stormwater from the wet well to the existing stormwater main that would then convey it to the stormwater utility lagoon. The actual cost of the separations will be split between the City sewer and stormwater funds. This separation program has higher priority than the CSO 1 separation program, because CSO 2 is closer to the NPDES permit limit of averaging 1 CSO event per year over a 5-year period.

CIP C-4 – Emerson Street Utility Improvement

This project will re-connect 15 residential side sewers to the non-failed sewer main in Emerson Street.

CIP C-5 – Upsize Gravity Main at 13th Street and Avenue A

This project will replace approximately 1,600 feet of 8-inch gravity pipe with 10-inch and 12-inch PVC pipe from 13th Street and Park Avenue to south of Avenue A and Smithson Place. The OPPC is estimated based on open trench construction.

CIP C-6 – Western Gravity Flow Diversion

This project is scheduled to be designed and constructed under the Long-Term CIP, but project planning and permitting are scheduled for the 6-year CIP. Section 12.2.4 provides a detailed description of the project. The goal of the project is to provide capacity in the sewer collection system to support development in the UGA north of the current City limits and in the portion of the North Planning Area south of State Route 2.

CIP P-1 – Champagne Pump Station (PS 7) Upgrade and New Force Main

This project includes abandoning the force main running south from the Champagne Pump Station (PS 7) to Hill Park Pump Station (PS 6), installing a new 4-inch force main approximately 760 feet east along 18th Street and south down Park Avenue to connect to the existing gravity system on Park Avenue, upgrading approximately 560 feet of gravity pipeline from the connection point of the force main south along Park Avenue, installing a gravity pipe to convey flow from the Hill Park Pump Station to the Champagne Pump Station, and decommissioning the Hill Park Pump Station. This project also includes upgrading the Champagne Pump Station by replacing the existing pumps with larger pumps, replacing the existing electrical equipment, adding flow metering and rehabilitating the wet well. This alternative greatly reduces the flow conveyed to the Hill Park Pump Station, which allows the remaining flow collected at the Hill Park Pump Station to be conveyed by gravity to the Champagne Pump Station and the Hill Park Pump Station decommissioned.

The need to upgrade the Champagne Pump Station is due to high flows during large storms, causing the backup pump at times to run simultaneously with the duty pump, which then provides no redundancy for these peak flow events. Since the contributing area is supposed to be all sewage with no known stormwater connections, this information suggests that there may be unknown sources of stormwater directly connected to the sewer pipes upstream of the pump station that should be removed. This project could be reduced in size or possibly eliminated if stormwater sources were located and disconnected such that peak flows to the pump station were substantially reduced. To accomplish this, a survey to determine potential sources of stormwater would need to be conducted. If sources were found and disconnected, temporary flow metering could be installed upstream of the pump station to re-calibrate the model and determine if inflow has been decreased enough to keep the backup pump from running during a 25-year storm.

CIP P-2 – Kla-Ha-Ya Pump Station (PS 11) Decommissioning

The Kla-Ha-Ya Pump Station does not meet current DOE standards for reliability and is subject to high infiltration and inflow due to its location on the riverbank. Therefore, the City plans to decommission the pump station once a gravity line has been installed to connect the side sewers of the parcels currently served by the pump station to a downstream manhole.

CIP P-3 – Mobile 80 kW Generator

This project is to replace the City's 1977 trailer-mounted 80 kW generator used to provide emergency power to the Rainier Pump Station, which is not currently equipped with an emergency generator on site. The OPPC is based on purchase of a new trailer-mounted generator. However, this project may not be necessary, since a permanent generator is to be added to Rainier Pump Station (see CIP P-4), but the City may still desire a new portable generator for continued use at other pump stations.

CIP P-4 – Rainier Pump Station (PS 2) Generator

The Rainier Pump Station currently uses a portable power generator for its backup power. This project adds a permanent 100 kW generator set including sound attenuating enclosure and equipment pad. Due to the limited area around the Rainier Pump Station, the project includes purchasing a nearby undeveloped lot to locate the generator. The OPPC assumes purchase of the entire lot. During project development, alternatives such as easements or purchasing only a portion of the lot can be explored.

CIP P-5 – Rainier Pump Station (PS 2) Upgrades

The Rainier Pump Station was re-built in 1987, which exceeds the typical 25-year life expectancy of mechanical equipment and will be at the 50-year life expectancy of structural facilities within the planning horizon. The electrical equipment at the Rainier Pump Station was upgraded in 2014/2015 so it should be fine for the planning horizon. The instrumentation and controls will have met their life expectancy by the end of the planning horizon, so new instrumentation and controls are included as part of the upgrade. It is recommended that funds be budgeted for the rehabilitation/replacement of mechanical equipment and instrumentation and an inspection of structural facilities based on its age. Firm capacity will be restored to the original design of 3,000 gpm or greater to convey projected flows. The OPPC for this project includes replacing the pumps with larger pumps, replacing the piping, replacing the instrumentation, adding flow metering, and rehabilitating the wet well.

CIP W-1 – WWTP NPDES Re-rating

This project's goal is to get the WWTP's maximum month flow and BOD_5 limits increased in the NPDES permit to cover the projected flows and loads through the planning horizon. The WWTP will exceed its current rated capacity for maximum month flow and BOD_5 loading by 2022, as discussed in Chapters 8 and 9. The analysis of the WWTP showed that only improvements to the aeration system and effluent filtration would be needed for the plant to accommodate the higher flows and BOD_5 loading, which are covered in the 6-year CIP (see CIP W-2 and W-3). This project allocates funds for a study to verify capacity, as needed for the re-rating, upon completion of the aeration and filtration improvements.

CIP W-2 – Upgrade/Expand Effluent Filtration

This project is to upgrade the existing effluent filtration process to improve performance and efficiency and increase the filtration capacity. The existing continuous backwash sand filters will either be replaced with a compressible media system (e.g., Schreiber Fuzzy Filters). This improvement is meant to increase capacity, increase capture of solids, reduce use of chemicals, and reduce reject flow from the filters back into the lagoons. The increased capacity will significantly enhance solids removal in the effluent, allowing filtration of all effluent during both dry and wet weather conditions, except under peak flow conditions during periods of high rainfall when a portion of the effluent may bypass filtration. The proposed Fuzzy Filters would have a capacity of 4.2 MGD at a hydraulic loading rate of 30 gpm/sf. Chapter 11 provides further discussion on the evaluation of filtration alternatives and selection of this improvement.

CIP W-3 – General WWTP Improvements (2020-2025)

This is an annual budgeted amount for general repairs to the WWTP. The amount budgeted each year is increased by 3.25% to keep up with inflation and rising construction costs. This annual budget includes planned replacement of at least three existing surface aerators per year at a cost of $10,000 to $12,000 each. The City will invest in replacing failed or problematic aerators, rather than fixing the existing aerators due to obsolescence and capacity issues. Over the course of the 6-year CIP all 18 aerators in Lagoon #1 will be replaced. This replacement schedule will ensure that sufficient aeration capacity is available to meet the projected demands. Chapter 11 provides further discussion on the evaluation of aeration alternatives and selection of this improvement.

CIP W-4 – Replace the Chlorine Gas Disinfection System

This project is to replace the current chlorine gas disinfection system due to safety concerns and improvements required to enhance safety and comply with current codes. The alternative analysis in Chapter 11 determined that replacement with a peracetic acid (PAA) system is the most cost-effective alternative. Successful full-scale testing demonstrated that PAA dosing is comparable to values suggested by initial bench-scale testing, such that use of PAA economically viable and competitive with the next highest ranked alternative of delivered sodium hypochlorite. Additionally, a PAA system provides simpler operation and easier chemical handling compared to delivered sodium hypochlorite. The cost for this project within the 6-year CIP excludes planning, which was completed in 2018, and design, which will be completed in 2019.

CIP W-5 – Add Non-Potable Water Air Gaps

The City needs to add two air gaps at the WWTP to provide cross connection control for protection of the public potable water system and onsite domestic potable water system from possible contamination. One will isolate the non-potable water system at the south end of the WWTP and the other will isolate the non-potable water system at the north end of the WWTP. The north end air gap systems will be housed in a small pre-engineered metal building near the headworks. The south end air gap system will be housed in a pre-engineered metal building near the new PAA system.

12.2.3 Long-Term CIP (2026-2036)

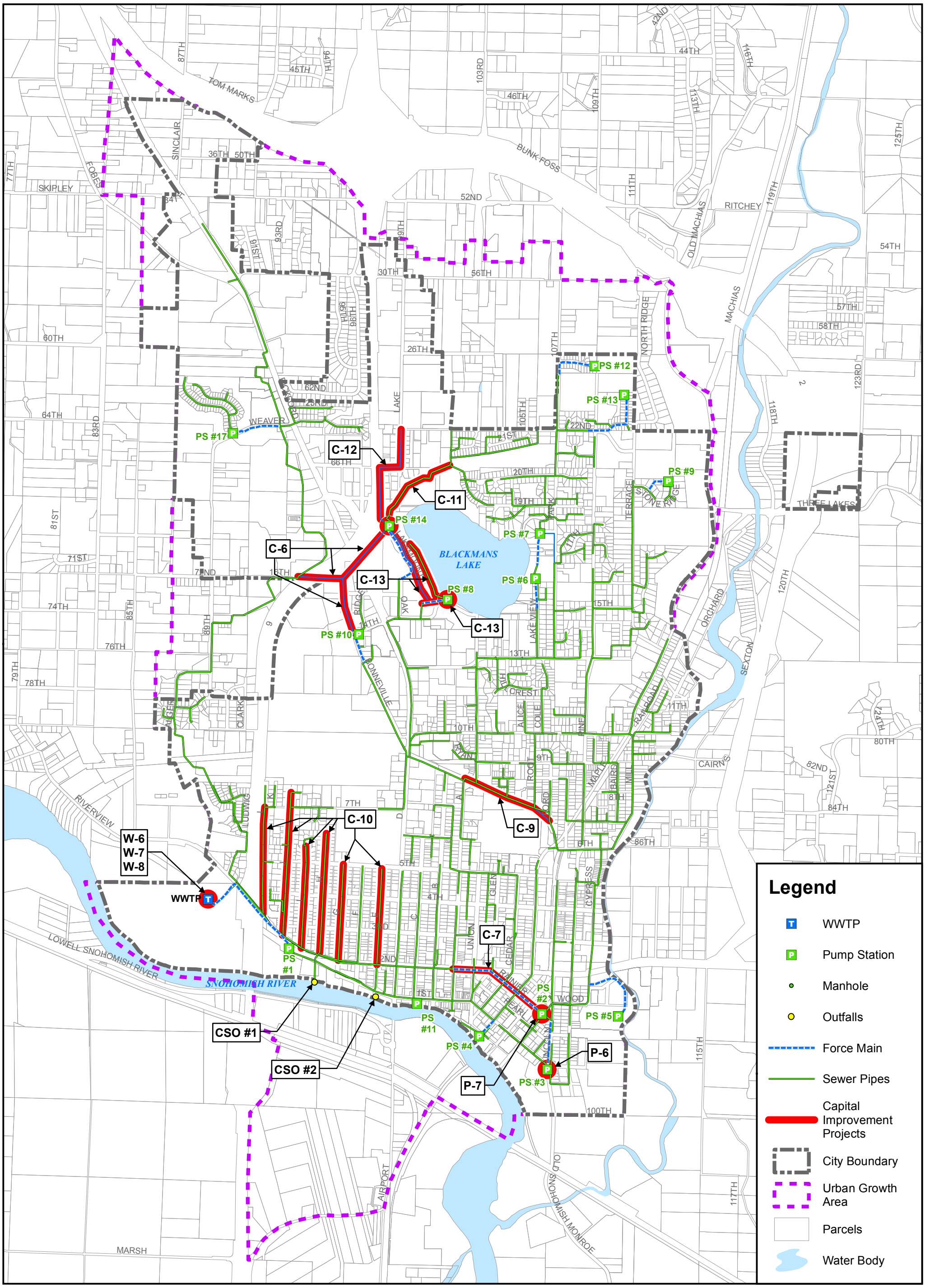
The projects recommended for the Long-Term CIP are described in Table 12-2 and illustrated on Figure 12-2. Improvements that would be completed by developers are expected to be privately funded and so are not listed in this section. Project priorities were developed with input from the City.

Table 12-2 Long-Term Capital Improvement Plan (2026-2036)

CIP No.	Project	Type	Replacement	Upgrade	Expansion	Project Description
C-6	Western Gravity Flow Diversion	Capacity	☐	☐	☑	Add approximately 4,000 ft. of pipe to divert flow to the West Trunk
						Decommission the Casino and Bonneville Pump Stations
C-7	Rainier Pump Station (PS 2) Force Main Replacement	Obsolescence	☑	☐	☐	Replace the existing 1,800 ft. AC force main with a PVC force main
C-8	General Conveyance System Repair and Replacements (2026-2036)	O&M / Obsolescence	☑	☐	☐	Annual allowance for general sewer conveyance repairs and replacements
						Projects typically accompany annual Road Preservation Projects
C-9	Interurban Trail Conveyance System Upgrade	Obsolescence	☑	☐	☑	Replace and upsize 1,600 ft. of pipe along the Interurban Trail
C-10	CSO 1 Separation Program	Capacity	☐	☑	☐	Annual allowance to install stormwater pipes and separate the CSS in the Gravity CSO basin
C-11	CIPP Pipe Restoration North of Blackmans Lake	Obsolescence	☐	☑	☐	Line approximately 1,700 ft. of pipe north of Blackmans Lake with CIPP
C-12	Pipe Extension North of the Casino Pump Station (PS 14)	Capacity	☐	☐	☑	Install a 2,100 ft. long pipe extension north of the Casino Pump Station
C-13	New Gravity Line and Decommissioning the Ferguson Pump Station (PS 8)	Obsolescence	☑	☐	☐	Install 1,450 ft. pipe from the Ferguson Pump Station to the Casino Pump Station
						Add new grinder pumps and laterals to residences contributing to the Ferguson Pump Station
						Decommission and abandon the Ferguson Pump Station, force main, and tributary gravity line
P-6	Lincoln Pump Station (PS 3) Replacement	Obsolescence	☑	☑	☐	Replace mechanical equipment and add flow metering
						Replace electrical/controls and add emergency generator
						Rehabilitate wet well
P-7	Rainier Pump Station (PS 2) Wet Well Replacement/Expansion	Capacity / Obsolescence	☑	☑	☐	Replace and expand existing wet well

Table 12-2 Long-Term Capital Improvement Plan (2026-2036)

CIP No.	Project	Type	Replacement	Upgrade	Expansion	Project Description
W-6	Replace MCCs and Retrofit the Influent Screw Pumps with VFDs	O&M	☑	☑	☐	Retrofit the existing WWTP influent screw pumps with variable frequency drives (VFDs) Replace EMCC-1, EMCC-2, and NMCC-1
W-7	Biosolids Removal	O&M	☑	☐	☐	Clean out biosolids from the WWTP Lagoons
W-8	General WWTP Improvements (2026-2036)	O&M / Obsolescence	☑	☐	☐	Annual allowance for general WWTP repairs, including aerator replacement



Sewer System & Basins: City of Snohomish.
GIS Base data: Snohomish County.
Data sources supplied may not reflect current or actual conditions. This map is a geographic representation based on information available. It does not represent survey data. No warranty is made concerning the accuracy, currency, or completeness of data depicted on this map.
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City of Snohomish Washington

Long-Term Capital Improvement Projects

General Sewer Plan and
Wastewater Facilities Plan
City of Snohomish, Washington
February 2019

Figure
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12.2.4 Long-Term CIP Project Descriptions

CIP C-6 – Western Gravity Flow Diversion

The purpose of this project is to provide capacity in the sewer collection system to support development in the UGA north of the current City limits and in the portion of the North Planning Area south of State Route 2. The project would add approximately 3,000 linear feet of gravity pipe to route flow from these future developments to the West Trunk just west of State Route 9 along 16th Street. About 1,250 feet of this new gravity pipe would be an 18-inch pipe installed using horizontal directional drilling (HDD) between the Casino Pump Station (PS 14), located near the intersection of Lakemount Drive and 99th Ave SE, and the intersection of 16th Street and Bonneville Ave just east of State Route 9. This route passes through primarily public right-of-way, but also some designated commercial and park areas at either end. Although open trench installation of this section of pipe could potentially be utilized, it is not a prudent or cost effective method for installation due to terrain and physical obstructions. However, the 800-foot long 18-inch pipe section extending west along 16th Street, which crosses under State Route 9 to connect with the west trunk, would be installed via open-trench method. Installation of these 18-inch gravity pipelines would allow the Casino Pump Station (PS 14) to be decommissioned.

This project would also include approximately 950 feet of 12-inch gravity pipe installed via open-trench method to route flow from the Bonneville Pump Station (PS 10), located at 14th Street and Bonneville Avenue, to the new 18-inch gravity pipeline at the intersection of 16th Street and Bonneville Avenue just east of State Route 9. This would allow the Bonneville Pump Station to also be decommissioned.

CIP C-7 – Rainier Pump Station (PS 2) Force Main Replacement

This project is to replace the existing approximately 1,800-foot long, 8-inch asbestos concrete (AC) force main, which runs from the Rainier Pump Station to 2nd Street and Avenue A, with a PVC force main. The existing AC main is being replaced due to its age and the unreliability of AC pipe. The OPPC is estimated based on open trench construction and the removal of the existing AC pipe.

CIP C-8 – General Conveyance System Repair and Replacements (2026-2036)

This allowance is for general sewer conveyance repairs and replacements. The projects typically accompany annual Road Preservation Projects. The City often replaces old sewer pipes during road renovation projects to minimize project costs in anticipation of future pipe failures in the 70+ year old pipes.

CIP C-9 – Interurban Trail Conveyance System Upgrade

This project replaces approximately 1,660 feet of existing gravity pipe to remove a sag currently in the pipe and increase capacity. Construction would be done via open trench installation with larger 12-inch and 15-inch HDPE pipe and installing several new manholes between Avenue A and Ford Avenue along the Interurban Trail. The new pipe would be installed parallel to the existing line and would only require bypass pumping while the new pipe is being connected to the existing line and when the laterals from Avenue A, Glen Avenue and Root Avenue are connected to the main. The OPCC assumes the existing pipe would be abandoned in place and filled with controlled density fill (CDF).

CIP C-10 – CSO 1 Separation Program

The continuation of the CSO 1 Separation Program involves installing stormwater pipes in the gravity portion of the CSS to separate storm and sanitary sewers along Avenue E through Riverview Lane between 2nd and 5th Streets. In addition to installing new stormwater pipelines, the existing sewer pipelines will also be replaced with new sanitary sewer pipelines. The stormwater in those areas would be diverted away from the sewer system and into an existing stormwater main that will convey it to a stormwater utility lagoon adjacent to the WWTP. These separation projects will be coordinated with planned roadway and utility improvements along these same streets. The actual cost of the separations will be split between the City sewer and stormwater funds.

CIP C-11 – CIPP Pipe Restoration North of Blackmans Lake

This project would use cured-in-place pipe (CIPP) to restore the integrity of 1,700 lineal feet of pipe running from the north end of Blackmans Lake to the Casino Pump Station (PS 14). CIPP coats the existing pipe in an epoxy composed of resin and fiberglass that restores the integrity of the pipe and makes it watertight, reducing or eliminating I&I. The OPPC for this project assumes that UV cured CIPP is used to shorten the curing time, thus reducing the amount of bypass pumping needed.

CIP C-12 – Pipe Extension North of the Casino Pump Station (PS 14)

This project adds approximately 2,100 lineal feet of 18-inch pipe north of the Casino Pump Station for future development. The pipe would run down Lake Avenue from 22nd Street to Lakemount Drive and connect to the Casino Pump Station.

CIP C-13 – New Gravity Line and Decommissioning the Ferguson Pump Station (PS 8)

This project eliminates the need for the Ferguson Pump Station by adding a gravity line and residential grinder pumps to route flow to the Casino Pump Station (PS 14). The new 1,450 lineal feet of 8-inch gravity pipe would run north from the end of Lakemount Drive to the Casino Pump Station. Thirteen residences along Blackmans Lake would have new grinder pumps and laterals connecting to the new 8-inch gravity pipe, while the current laterals running east towards Blackmans Lake would be filled with CDF and abandoned. Ferguson Pump Station and the manholes along the existing influent gravity line would be partially demolished and the remaining portions filled with gravel or CDF. The pump station's force main and influent gravity pipe would be abandoned in place and filled with CDF.

CIP P-6 – Lincoln Pump Station (PS 3) Replacement

The Lincoln Pump Station was built in 1958 and last upgraded in 1981. The Lincoln Pump Station exceeds the typical 25-year life expectancy of mechanical equipment and will exceed the 50-year life expectancy of structural facilities within the planning horizon. The electrical equipment at the station was upgraded in 2010 and will reach its typical 25-year life expectancy by 2035. It is recommended that funds be budgeted for the rehabilitation and/or replacement of mechanical and electrical equipment, and an inspection/rehabilitation of the wet well structure based on its age. Firm capacity should be 280 gpm. The OPPC for this project includes replacing the pumps and electrical equipment, adding flow metering, adding an emergency generator, and rehabilitating the wet well.

CIP P-7 – Rainier Pump Station (PS 2) Wet Well Replacement/Expansion

This project allows for the replacement and/or expansion of the aging Rainier Pump Station wet well to double the wet well capacity. The increased wet well capacity would reduce cycling and wear on the pumps at the station and allow the pump station to operate more effectively at its full capacity.

CIP W-6 – Replace MCCs and Retrofit the Influent Screw Pumps with VFDs

This project is to retrofit the existing WWTP influent screw pumps with variable frequency drives (VFDs) to reduce energy consumption, which would be located in EMCC-1. Also included is the replacement of EMCC-1, EMCC-2, and NMCC-1, all of which are currently over 20 years old. NMCC-2 was recently replaced in 2012.

CIP W-7 – Biosolids Removal

The biosolids were removed from the WWTP lagoons in 2015, but it is anticipated that they will need to be cleared again during the planning horizon.

CIP W-8 – General WWTP Improvements (2026-2036)

This is an annual budgeted amount for general repairs to the wastewater treatment plant (WWTP). The amount budgeted each year is increased by 3.25% to keep up with inflation and rising construction costs.

12.2.5 Sewer Extensions into Undeveloped Basins

New sewer extensions will be needed to serve new developments expected in unsewered areas of the City, UGA and the portion of the North Planning Area south of State Route 2. Specific plans for the sewer extensions have not been prepared and will be the responsibility of the developer. Some of the developments may require local pump stations. The downstream existing pump stations and conveyance have adequate capacity to handle the projected additional flows.

Major land developers will prepare site-specific plans for street layouts, residential lot distribution, commercial parcels, sensitive area delineations, required setbacks with buffers, and other land use intentions for approval by the permitting authorities. These land use decisions, and the timing of when specific parcels are developed will influence the sewer collection facilities within these basins. These sewer extensions will be funded and built by developers and are not expected to require significant financial investment by the City. The contractor would be granted a reduced connection fee or credit for adding the extensions to the sewer system.

12.3 Opinions of Probable Project Cost

Opinions of probable project costs (OPPC) for the 6-year and Long-Term CIP are listed in Table 12-3 and Table 12-4, respectively. These projects have been defined to a preliminary planning level with approximate sizing and dimensions and should be considered to have an accuracy of +50 to -30 percent, consistent with order-of-magnitude estimates used for planning purposes as defined by the American Association of Cost Engineers (AACE). All projects will require further definition and design refinement as part of the design process.

The OPPC for each improvement are provided in Appendix J. Each OPPC includes markups for 9.1% Washington State sales tax (WSST), and a 35% construction contingency is applied to the construction costs. A 30% factor is added to the construction cost to get the project cost. The 30% factor includes 15% for planning/engineering and 15% for construction management. These markups were consistently applied to each OPPC, unless stated otherwise herein. Construction costs were estimated from bid results for similar projects in the Puget Sound area, equipment vendor quotes and RS Means cost data. Costs associated with financing, easements, right-of-way, or property acquisition are not included, unless specifically noted. Actual costs can and will differ from the opinions of probable project costs. Volatility in the bidding climate, the number of contractors bidding on a project, and their approach to bidding and completing the work will all impact actual project costs.

Table 12-3 6-Year (2020-2025) CIP Opinion of Probable Project Costs[(a)]

CIP No.	Project	Opinion of Probable Project Cost						Total Project Cost
		2020	2021	2022	2023	2024	2025	
C-1	Capital Reserve	$2,000,000	$0	$0	$0	$0	$0	$2,000,000
C-2	General Conveyance System Repair and Replacements (2020-2025)	$360,000	$375,000	$385,000	$400,000	$415,000	$430,000	$2,365,000
C-3	CSO 2 Separation Program[(b)]	$0	$0	$0	$0	$265,000	$2,025,000	$2,315,000
C-4	Emerson Street Utility Improvement	$100,000	$0	$0	$0	$0	$0	$100,000
C-5	Upsize Gravity Main at 13th Street and Avenue A	$0	$0	$0	$0	$0	$1,050,000	$1,050,000
C-6	Western Gravity Flow Diversion (Planning)	$0	$0	$0	$0	$0	$119,000	$119,000
P-1	Champagne Pump Station (PS 7) Upgrade and New Force Main	$170,000	$1,290,000	$0	$0	$0	$0	$1,460,000
P-2	Kla-Ha-Ya Pump Station (PS 11) Decommissioning	$42,000	$0	$0	$0	$0	$0	$42,000
P-3	Mobile 80 kW Generator	$64,000	$0	$0	$0	$0	$0	$64,000
P-4	Rainier Pump Station (PS 2) Generator (100kW)	$0	$0	$70,000	$510,000	$0	$0	$580,000
P-5	Rainier Pump Station (PS 2) Upgrades	$0	$0	$90,000	$670,000	$0	$0	$760,000
W-1	WWTP NPDES Re-rating	$0	$75,000	$0	$0	$0	$0	$75,000
W-2	Upgrade/Expand Effluent Filtration	$0	$250,000	$2,070,000	$0	$0	$0	$2,320,000
W-3	General WWTP Improvements (2020-2025)	$360,000	$375,000	$385,000	$400,000	$415,000	$430,000	$2,365,000
W-4	Replace the Chlorine Gas Disinfection System	$1,290,000	$0	$0	$0	$0	$0	$1,290,000
W-5	Add Non-Potable Water Air Gaps	$0	$0	$0	$500,000	$0	$0	$500,000
Total Opinion of Probable Project Cost		**$4,386,000**	**$2,365,000**	**$3,000,000**	**$2,480,000**	**$1,095,000**	**$4,079,000**	**$17,405,000**

Note:
(a) All costs are in 2018 dollars including future projects.
(b) Half of each CSO Separation Program is being paid for by the sewer fund while the other half is being paid from the stormwater fund. The cost presented in the table is the portion allocated from the sewer fund.

Table 12-4 Long-Term (2026-2036) CIP Opinion Of Probable Project Costs[a]

CIP No.	Project	Opinion of Probable Project Cost
C-6	Western Gravity Flow Diversion	$2,981,000
C-7	Rainier Pump Station (PS 2) Force Main Replacement	$760,000
C-8	General Conveyance System Repair and Replacements (2026-2036)	$8,115,000
C-9	Interurban Trail Conveyance System Upgrade	$1,019,000
C-10	CSO 1 Separation Program[b]	$4,500,000
C-11	CIPP Pipe Restoration North of Blackmans Lake	$451,000
C-12	Pipe Extension North of the Casino Pump Station (PS 14)	$1,479,000
C-13	New Gravity Line and Decommissioning the Ferguson Pump Station (PS 8)	$2,088,000
P-6	Lincoln Pump Station (PS 3) Replacement	$390,000
P-7	Rainier Pump Station (PS 2) Wet Well Replacement/Expansion	$230,000
W-6	Replace MCCs and Retrofit the Influent Screw Pumps with VFDs	$3,490,000
W-7	Biosolids Removal	$800,000
W-8	General WWTP Improvements (2026-2036)	$8,115,000
Total Opinion of Probable Project Cost		**$34,418,000**

Note:
(a) All costs are in 2018 dollars including future projects.
(b) Half of each CSO Separation Program is being paid for by the sewer fund while the other half is being paid from the stormwater fund. The cost presented in the table is the portion allocated from the sewer fund.

The financial impact to the City and sewer rates to complete the 6-year CIP is discussed in Chapter 13.

Chapter 13 Financial Plan

13.1 Introduction

To be added upon completion of the financial analysis

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APPENDICES

Appendix A

SEPA Checklist

SEPA ENVIRONMENTAL CHECKLIST

Purpose of checklist:

Governmental agencies use this checklist to help determine whether the environmental impacts of your proposal are significant. This information is also helpful to determine if available avoidance, minimization or compensatory mitigation measures will address the probable significant impacts or if an environmental impact statement will be prepared to further analyze the proposal.

***Instructions for applicants*:**

This environmental checklist asks you to describe some basic information about your proposal. Please answer each question accurately and carefully, to the best of your knowledge. You may need to consult with an agency specialist or private consultant for some questions. You may use "not applicable" or "does not apply" only when you can explain why it does not apply and not when the answer is unknown. You may also attach or incorporate by reference additional studies reports. Complete and accurate answers to these questions often avoid delays with the SEPA process as well as later in the decision-making process.

The checklist questions apply to all parts of your proposal, even if you plan to do them over a period of time or on different parcels of land. Attach any additional information that will help describe your proposal or its environmental effects. The agency to which you submit this checklist may ask you to explain your answers or provide additional information reasonably related to determining if there may be significant adverse impact.

Instructions for Lead Agencies:

Please adjust the format of this template as needed. Additional information may be necessary to evaluate the existing environment, all interrelated aspects of the proposal and an analysis of adverse impacts. The checklist is considered the first but not necessarily the only source of information needed to make an adequate threshold determination. Once a threshold determination is made, the lead agency is responsible for the completeness and accuracy of the checklist and other supporting documents.

Use of checklist for nonproject proposals: [help]

For nonproject proposals (such as ordinances, regulations, plans and programs), complete the applicable parts of sections A and B plus the SUPPLEMENTAL SHEET FOR NONPROJECT ACTIONS (part D). Please completely answer all questions that apply and note that the words "project," "applicant," and "property or site" should be read as "proposal," "proponent," and "affected geographic area," respectively. The lead agency may exclude (for non-projects) questions in Part B - Environmental Elements –that do not contribute meaningfully to the analysis of the proposal.

A. Background [help]

1. Name of proposed project, if applicable: [help]

   City of Snohomish – General Sewer Plan and Wastewater Facilities Plan

2. Name of applicant: [help]

   City of Snohomish, Public Works Department

3. Address and phone number of applicant and contact person: [help]

Andrew Sics, P.E.
Senior Utilities Engineer
City of Snohomish
116 Union Avenue
Snohomish, WA 98290
360-282-3174

4. Date checklist prepared: [help]

April 2018 (revised December 2018)

5. Agency requesting checklist: [help]

City of Snohomish, Public Works Department
Washington State Department of Ecology

6. Proposed timing or schedule (including phasing, if applicable): [help]

The proposed date for adoption of the General Sewer Plan and Wastewater Facilities Plan (Plan) by the Snohomish City Council is expected to be in 2019.

7. Do you have any plans for future additions, expansion, or further activity related to or connected with this proposal? If yes, explain. [help]

This is a non-project action adopting the Plan. In addition to the capital improvement plan (CIP) outlined in the Plan, incremental sanitary sewer facilities may be constructed in conjunction with private development, as they occur.

8. List any environmental information you know about that has been prepared, or will be prepared, directly related to this proposal. [help]

Snohomish County 2035 Comprehensive Plan (Adopted in 2015)
City of Snohomish General Sewer Plan and Wastewater Facilities Plan (May 2005)
City of Snohomish Comprehensive Plan (Revised March 2016)

9. Do you know whether applications are pending for governmental approvals of other proposals directly affecting the property covered by your proposal? If yes, explain. [help]

Property owners and developers have and are expected to apply for approval of development that will require sanitary sewer service. These developments are not addressed specifically in the Plan, but the Plan accounts for necessary sewer collection and treatment improvements to support such development. All project-level improvements will be subject to environmental review at the time of their application. As such, no pending proposals affect this non-project action.

10. List any government approvals or permits that will be needed for your proposal, if known. [help]

The Plan must be reviewed and approved by the Washington State Department of Ecology. The plan must also be reviewed by Snohomish County and the Snohomish City Council.

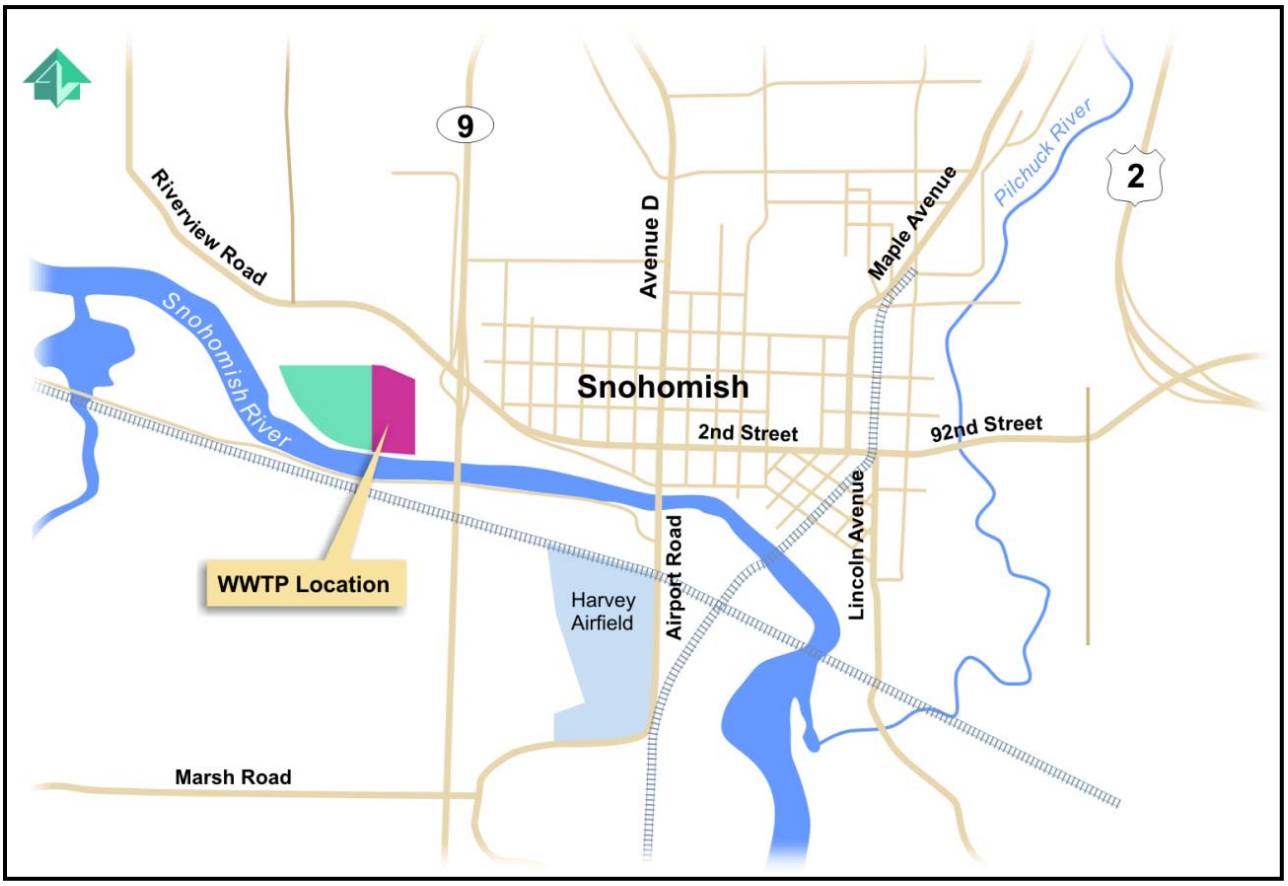
11. Give brief, complete description of your proposal, including the proposed uses and the size of the project and site. There are several questions later in this checklist that ask you to describe certain aspects of your proposal. You do not need to repeat those answers on this page. (Lead agencies may modify this form to include additional specific information on project description.) [help]

The Plan includes and/or considers the following regarding the City's sewer collection and treatment systems:

- **Current and historical information about the City's sewer collection and treatment systems**
- **Current and projected future land uses and populations**
- **Current and projected future wastewater flow rates and loads**
- **Applicable permits and regulations**
- **Current and projected future collection system hydraulics**
- **Current and projected future wastewater treatment capacity needs and performance**
- **Collection and treatment system reliability and obsolescence**
- **Evaluation of wastewater treatment alternatives**
- **Identification of recommended collection and treatment system improvements related to necessary upgrades, replacement and/or expansion**
- **Costs of recommended improvements**
- **Recommended capital improvement plan (CIP)**
- **Financial analysis of rate impacts resulting from recommended CIP**

12. Location of the proposal. Give sufficient information for a person to understand the precise location of your proposed project, including a street address, if any, and section, township, and range, if known. If a proposal would occur over a range of area, provide the range or boundaries of the site(s). Provide a legal description, site plan, vicinity map, and topographic map, if reasonably available. While you should submit any plans required by the agency, you are not required to duplicate maps or detailed plans submitted with any permit applications related to this checklist. [help]

The City of Snohomish sewer service area covered in the Plan includes the area within the City limits and portions of the designated Urban Growth Area (UGA). The City of Snohomish is located in Snohomish County with the City of Everett about 5 miles northwest. The Snohomish River borders the city to the south. The north end of the city extends towards Highway 2 and the city is bordered by the Pilchuck River on the east side. The portion of the UGA south of the Snohomish River is not part of the sewer service area considered in the Plan. The current UGA does not extend north of Highway 2.



B. ENVIRONMENTAL ELEMENTS [help]

1. Earth [help]

a. General description of the site: [help]

(circle one): Flat, rolling, hilly, steep slopes, mountainous, other _____

b. What is the steepest slope on the site (approximate percent slope)? [help]

The City's topography is generally moderately sloped up and away from the river valley. Though there are steep slopes in specific locations, this non-project action will not impact slopes, and any subsequent project work will be reviewed separately for SEPA compliance where required.

c. What general types of soils are found on the site (for example, clay, sand, gravel, peat, muck)? If you know the classification of agricultural soils, specify them and note any agricultural land of long-term commercial significance and whether the proposal results in removing any of these soils. [help]

Soils and soil types are not impacted by this non-project action. An extensive discussion of the soils and their properties can by found in the USDA soil survey of Snohomish County area.

d. Are there surface indications or history of unstable soils in the immediate vicinity? If so, describe. [help]

Unstable and steep slopes are not impacted by this non-project action. Any subsequent project work will include a separate site specific review will determine impacts to soils and slopes and SEPA compliance.

e. Describe the purpose, type, total area, and approximate quantities and total affected area of any filling, excavation, and grading proposed. Indicate source of fill. [help]

No filling or grading is proposed as part of this non-project action. Fill or grading related to subsequent project work will be reviewed separately for SEPA compliance.

f. Could erosion occur as a result of clearing, construction, or use? If so, generally describe. [help]

No clearing or construction is proposed as part of this non-project action.

g. About what percent of the site will be covered with impervious surfaces after project construction (for example, asphalt or buildings)? [help]

No construction is proposed as part of this non-project action.

h. Proposed measures to reduce or control erosion, or other impacts to the earth, if any: [help]

No specific measures are proposes as part of this non-project action. Subsequent project work will be evaluated separately as part of a site specific review for compliance with SEPA and other applicable regulations.

2. Air [help]

a. What types of emissions to the air would result from the proposal during construction, operation, and maintenance when the project is completed? If any, generally describe and give approximate quantities if known. [help]

This non-project action will not impact air quality. Air quality will be evaluated separately for subsequent project work as part of a site specific review for SEPA compliance.

b. Are there any off-site sources of emissions or odor that may affect your proposal? If so, generally describe. [help]

This non-project action will not generate odors or emissions. Potential for odors or emissions will be evaluated separately for subsequent project work as part of a site specific review for SEPA compliance.

c. Proposed measures to reduce or control emissions or other impacts to air, if any: [help]

This non-project action will not generate odors or emissions. Potential for odors or emissions will be evaluated separately for subsequent project work as part of a site specific review for SEPA compliance.

3. **Water** [help]

a. Surface Water:

1) Is there any surface water body on or in the immediate vicinity of the site (including year-round and seasonal streams, saltwater, lakes, ponds, wetlands)? If yes, describe type and provide names. If appropriate, state what stream or river it flows into. [help]

The City of Snohomish is bordered by the Snohomish River to the south and the Pilchuck River to the east. Blackmans Lake is located in the middle of the city limits. There are also numerous streams, creeks and wetlands throughout the city. Impacts on shoreline, surface water, seasonal streams and wetlands will be evaluated separately for subsequent project work as part of a site specific review for SEPA compliance.

2) Will the project require any work over, in, or adjacent to (within 200 feet) the described waters? If yes, please describe and attach available plans. [help]

This non-project action will not impact shoreline. Potential shoreline impacts will be evaluated separately for subsequent project work as part of a site specific review for SEPA compliance.

3) Estimate the amount of fill and dredge material that would be placed in or removed from surface water or wetlands and indicate the area of the site that would be affected. Indicate the source of fill material. [help]

This non-project action will not involve filling or dredging. Potential impacts from fill and dredging activities will be evaluated separately for subsequent project work, as applicable, as part of a site specific review for SEPA compliance.

4) Will the proposal require surface water withdrawals or diversions? Give general description, purpose, and approximate quantities if known. [help]

This non-project action will not involve surface water withdrawals or diversions. Potential impacts from surface water withdrawals or diversions will be evaluated separately for subsequent project work as part of a site specific review for SEPA compliance. However, none of the potential projects in the recommended CIP involve surface water withdrawals or diversions.

5) Does the proposal lie within a 100-year floodplain? If so, note location on the site plan. [help]

Portions of the city bordering the Snohomish and Pilchuck Rivers are within the 100-year floodplain. This non-project action does not impact floodplain areas specifically. Any subsequent project work within the floodplain will comply with applicable floodplain management regulations and city code and will be evaluated as part of a site specific review for SEPA compliance. It should be noted that the WWTP site is within the 100-year floodplain. Although the WWTP is protected by berms, it is required that future infrastructure at the WWTP susceptible to flood damage must be a

couple feet above grade for additional flood protection. This will need to be applied to potential projects at the WWTP in the recommended CIP.

6) Does the proposal involve any discharges of waste materials to surface waters? If so, describe the type of waste and anticipated volume of discharge. [help]

This non-project action does not directly affect the discharge of waste material to surface waters. However, the potential projects in the recommended CIP are aimed at further reducing combined sewer overflows to the Snohomish River and further improving effluent quality from the WWTP that is discharged to the Snohomish River. Any subsequent project work will need to comply with the City's NPDES permit and will be evaluated as part of a project specific review for SEPA compliance.

b. Ground Water:

1) Will groundwater be withdrawn from a well for drinking water or other purposes? If so, give a general description of the well, proposed uses and approximate quantities withdrawn from the well. Will water be discharged to groundwater? Give general description, purpose, and approximate quantities if known. [help]

This non-project action does not directly affect groundwater. Any subsequent project work will be evaluated as part of a project specific review for SEPA compliance. It is expected that potential projects in the recommended CIP may require temporary withdrawal of groundwater for the purpose of dewatering excavations during construction, but no permanent groundwater withdrawals or discharges to groundwater are envisioned.

2) Describe waste material that will be discharged into the ground from septic tanks or other sources, if any (for example: Domestic sewage; industrial, containing the following chemicals. . . ; agricultural; etc.). Describe the general size of the system, the number of such systems, the number of houses to be served (if applicable), or the number of animals or humans the system(s) are expected to serve. [help]

This non-project action will not require discharge of waste material to groundwater. Potential projects in the recommended CIP are meant to expand sewer service to include areas where septic systems are currently used, which would reduce the discharge of waste material to groundwater. Existing health regulations control the location, type and density of development where septic tanks exist or may be proposed.

c. Water runoff (including stormwater):

1) Describe the source of runoff (including storm water) and method of collection and disposal, if any (include quantities, if known). Where will this water flow? Will this water flow into other waters? If so, describe. [help]

This non-project action will not impact surface and stormwater. It is expected that potential projects in the recommended CIP will reduce the occurrence of combined sewers by separating sanitary and storm sewers in some locations. Additionally, specific projects may increase or decrease the impervious area and resulting runoff. Impacts on stormwater flow and runoff from subsequent project work will be evaluated as part of a project specific review for SEPA compliance.

2) Could waste materials enter ground or surface waters? If so, generally describe. [help]

This non-project action will not impact ground or surface waters. Potential impacts to ground and surface waters will be evaluated as part of a project specific review for SEPA compliance.

3) Does the proposal alter or otherwise affect drainage patterns in the vicinity of the site? If so, describe. [help]

This non-project action will not impact drainage patterns. However, potential projects in the recommended CIP will reduce the occurrence of combined sewers by separating sanitary and storm sewers in some locations. Possible impacts to drainage patterns will be evaluated as part of a project specific review for SEPA compliance.

d. Proposed measures to reduce or control surface, ground, and runoff water, and drainage pattern impacts, if any: [help]

This non-project action will not impact surface water, groundwater or drainage patterns. Measures to reduce or control such impacts will be evaluated as part of a project specific review for SEPA compliance.

4. **Plants** [help]

a. Check the types of vegetation found on the site: [help]

__X__deciduous tree: alder, maple, aspen, other
__X__evergreen tree: fir, cedar, pine, other
__X__shrubs
__X__ grass
_____ pasture
_____ crop or grain
__X__ Orchards, vineyards or other permanent crops.
__X__ wet soil plants: cattail, buttercup, bullrush, skunk cabbage, other
__X__water plants: water lily, eelgrass, milfoil, other
__X__other types of vegetation (landscape plantings)

b. What kind and amount of vegetation will be removed or altered? [help]

This non-project action will not directly affect vegetation. However, potential projects in the recommended CIP may require removal or alteration of existing vegetation. Impacts to vegetation will be evaluated as part of a project specific review for SEPA compliance.

c. List threatened and endangered species known to be on or near the site. [help]

This non-project action will not directly impact threatened or endangered species. The potential for such impacts will be evaluated as part of a project specific review for SEPA compliance.

d. Proposed landscaping, use of native plants, or other measures to preserve or enhance vegetation on the site, if any: [help]

Landscaping is not proposed as part of this non-project action. Applicable regulations for open space and plantings will be considered as part of a project specific review for SEPA compliance.

e. List all noxious weeds and invasive species known to be on or near the site. [help]

This non-project action has no impact on noxious weeds or invasive species. Flora will be evaluated for subsequent project work as part of a site specific review for SEPA compliance.

5. **Animals** [help]

a. List any birds and other animals which have been observed on or near the site or are known to be on or near the site. [help]

Examples include:

birds: hawk, heron, eagle, songbirds, other: **ducks, crows, gulls**
mammals: deer, bear, elk, beaver, other: **opossums, bats, rabbits, gophers, skunks, voles, mice, coyotes and domestic animals**
fish: bass, salmon, trout, herring, shellfish, other _____

b. List any threatened and endangered species known to be on or near the site. [help]

This non project action will not have a direct impact on wildlife. Impacts to wildlife will be evaluated for subsequent project work as part of a site specific review for SEPA compliance. Chinook salmon and bull trout both use the bordering waterways for incoming and outgoing migration.

c. Is the site part of a migration route? If so, explain. [help]

The Pacific Flyway is a major north-south migratory route for birds that encompasses most of western Washington, including the City of Snohomish. Impacts to wildlife will be evaluated for subsequent project work as part of a site specific review for SEPA compliance.

d. Proposed measures to preserve or enhance wildlife, if any: [help]

This non-project action will not directly impact wildlife, so there are not proposed measures to preserve or enhance wildlife. Such measures will be evaluated for subsequent project work as part of a site specific review for SEPA compliance.

e. List any invasive animal species known to be on or near the site. [help]

This non-project action will not have an effect on invasive animal species. Potential impacts will be evaluated for subsequent project work as part of a site specific review for SEPA compliance.

6. **Energy and Natural Resources** [help]

a. What kinds of energy (electric, natural gas, oil, wood stove, solar) will be used to meet the completed project's energy needs? Describe whether it will be used for heating, manufacturing, etc. [help]

No energy is required for this non-project action. Energy consumption will be evaluated as part of a project specific review for SEPA compliance. Potential projects in the recommended CIP may increase or also decrease energy consumption.

b. Would your project affect the potential use of solar energy by adjacent properties? If so, generally describe. [help]

This non-project action will not affect solar access. Solar access will be evaluated for subsequent project work as part of a site specific review for SEPA compliance.

c. What kinds of energy conservation features are included in the plans of this proposal? List other proposed measures to reduce or control energy impacts, if any: [help]

None. The city used the Washington State Energy Code to enhance electricity conservation. Energy conservation features will be considered as part of a project specific review for SEPA compliance.

7. **Environmental Health** [help]

a. Are there any environmental health hazards, including exposure to toxic chemicals, risk of fire and explosion, spill, or hazardous waste, that could occur as a result of this proposal? If so, describe. [help]

This non-project action does not involve any environmental health hazards. Potential environmental health hazards will be evaluated as part of a project specific review for SEPA compliance. Potential projects in the recommended CIP involve elimination of chlorine gas at the WWTP in exchange for a less hazardous chemical (peracetic acid) for disinfection.

1) Describe any known or possible contamination at the site from present or past uses. [help]

This non-project action does not impact know contamination or potential for contamination. These impacts will be evaluated as part of a project specific review for SEPA compliance.

2) Describe existing hazardous chemicals/conditions that might affect project development and design. This includes underground hazardous liquid and gas transmission pipelines located within the project area and in the vicinity. [help]

This non-project action does not impact existing hazardous chemical or conditions. Such impacts will be evaluated as part of a project specific review for SEPA compliance. Potential projects in the recommended CIP involve elimination of chlorine gas at the WWTP in exchange for a less hazardous chemical (peracetic acid) for disinfection.

3) Describe any toxic or hazardous chemicals that might be stored, used, or produced

during the project's development or construction, or at any time during the operating life of the project. [help]

This non-project action does not impact storage, use or production of hazardous chemicals. Such impacts will be evaluated as part of a project specific review for SEPA compliance. Potential projects in the recommended CIP involve eliminating storage and use of chlorine gas for disinfection at the WWTP in exchange for a less hazardous chemical (peracetic acid).

4) Describe special emergency services that might be required. [help]

This non-project action does not require any special emergency services. The potential need for such services will be evaluated as part of a project specific review for SEPA compliance.

5) Proposed measures to reduce or control environmental health hazards, if any: [help]

No measures to reduce or control environmental health hazards are necessary as part of this non-project action.

b. Noise [help]

1) What types of noise exist in the area which may affect your project (for example: traffic, equipment, operation, other)? [help]

This non-project action will not impact noise levels. Impact to noise levels will be considered as part of a project specific review for SEPA compliance.

2) What types and levels of noise would be created by or associated with the project on a short-term or a long-term basis (for example: traffic, construction, operation, other)? Indicate what hours noise would come from the site. [help]

This non-project action will not create or impact noise levels. Impact to noise levels will be considered as part of a project specific review for SEPA compliance.

3) Proposed measures to reduce or control noise impacts, if any: [help]

Noise levels are regulated by city code. Subsequent project work will need to comply with regulations for noise under the city code.

8. **Land and Shoreline Use** [help]

a. What is the current use of the site and adjacent properties? Will the proposal affect current land uses on nearby or adjacent properties? If so, describe. [help]

Land use in the City of Snohomish is primarily residential and commercial with some industrial, parks and open space. This non-project action will not impact current land uses. Additionally, potential projects in the recommended CIP are also not expected to impact current land uses. Impacts to current land uses will be evaluated as part of a project specific review for SEPA compliance.

b. Has the project site been used as working farmlands or working forest lands? If so, describe. How much agricultural or forest land of long-term commercial significance will be converted to

other uses as a result of the proposal, if any? If resource lands have not been designated, how many acres in farmland or forest land tax status will be converted to nonfarm or nonforest use? [help]

There are no designated forest or agricultural land use areas in the city limits or UGA. However, there are a couple areas classified as Urban Horticulture land use at the south end of the city. Impacts to current land uses will be evaluated as part of a project specific review for SEPA compliance.

1) Will the proposal affect or be affected by surrounding working farm or forest land normal business operations, such as oversize equipment access, the application of pesticides, tilling, and harvesting? If so, how: [help]

This non-project action will not affect or be affected by surrounding forest or farmland. The impact to or from surrounding forest or farmland will be considered as part of a project specific review for SEPA compliance.

c. Describe any structures on the site. [help]

This non-project action does not impact any structures. Impacts to existing structures will be evaluated as part of a project specific review for SEPA compliance. Potential projects in the recommended CIP involve work at the WWTP and pump stations where structures currently exist.

d. Will any structures be demolished? If so, what? [help]

No structures will be demolished as part of this non-project action. Potential projects in the recommended CIP (e.g., replacement of the Rainier Pump Station wet well and the Ferguson Pump Station) may require demolition of existing structures, but such demolition activities will be evaluated as part of a project specific review for SEPA compliance.

e. What is the current zoning classification of the site? [help]

Current zoning classifications within the city limits and UGA include single family, low density residential, medium density residential, high density residential, Pilchuck district, business park, commercial, historic business, mixed use, industry, airport industry, parks, open space and urban horticulture. Zoning in the City of Snohomish is according to the official zoning map, adopted March 2015.

f. What is the current comprehensive plan designation of the site? [help]

The comprehensive plan land use designations are the same as the zoning classifications identified above with the addition of designations for right-of-way and open water.

g. If applicable, what is the current shoreline master program designation of the site? [help]

Not applicable for this non-project action.

h. Has any part of the site been classified as a critical area by the city or county? If so, specify. [help]

There are environmentally sensitive areas located in the city limits and UGA. These can include wetlands, floodplain, geologically hazardous areas, creeks and rivers. These are identified in the City of Snohomish Comprehensive Plan. Environmentally sensitive areas will be evaluated for subsequent project work as part of a site specific review for SEPA compliance.

i. Approximately how many people would reside or work in the completed project? [help]

Not applicable to this non-project action.

j. Approximately how many people would the completed project displace? [help]

None for this non-project action.

k. Proposed measures to avoid or reduce displacement impacts, if any: [help]

Not applicable to this non-project action.

L. Proposed measures to ensure the proposal is compatible with existing and projected land uses and plans, if any: [help]

None applicable for this non-project action.

m. Proposed measures to reduce or control impacts to agricultural and forest lands of long-term commercial significance, if any: [help]

None applicable for this non-project action.

9. **Housing** [help]

a. Approximately how many units would be provided, if any? Indicate whether high, middle, or low-income housing. [help]

Not applicable to this non-project action.

b. Approximately how many units, if any, would be eliminated? Indicate whether high, middle, or low-income housing. [help]

Not applicable to this non-project action.

c. Proposed measures to reduce or control housing impacts, if any: [help]

None applicable for this non-project action.

10. **Aesthetics** [help]

a. What is the tallest height of any proposed structure(s), not including antennas; what is the principal exterior building material(s) proposed? [help]

This non-project action has no effect on structure height. Any addition or modification of structures for subsequent project work will need to comply with the city building code.

b. What views in the immediate vicinity would be altered or obstructed? [help]

This non-project action will not impact views. Impacts to views must comply with city code and will be evaluated as part of a project specific review for SEPA compliance.

b. Proposed measures to reduce or control aesthetic impacts, if any: [help]

None applicable for this non-project action.

11. **Light and Glare** [help]

a. What type of light or glare will the proposal produce? What time of day would it mainly occur? [help]

This non-project action will not produce any light or glare. Light and glare impacts will be considered as part of a project specific review for SEPA compliance.

b. Could light or glare from the finished project be a safety hazard or interfere with views? [help]

This non-project action will not produce any light or glare. Light and glare impacts will be considered as part of a project specific review for SEPA compliance.

c. What existing off-site sources of light or glare may affect your proposal? [help]

This non-project action will not produce any light or glare. Light and glare impacts will be considered as part of a project specific review for SEPA compliance.

d. Proposed measures to reduce or control light and glare impacts, if any: [help]

None applicable for this non-project action.

12. **Recreation** [help]

a. What designated and informal recreational opportunities are in the immediate vicinity? [help]

Not applicable to this non-project action.

b. Would the proposed project displace any existing recreational uses? If so, describe. [help]

This non-project action will not displace any recreational uses. Displacement of recreational uses will be considered as part of a project specific review for SEPA compliance/

c. Proposed measures to reduce or control impacts on recreation, including recreation opportunities to be provided by the project or applicant, if any: [help]

None applicable for this non-project action.

13. **Historic and cultural preservation** [help]

a. Are there any buildings, structures, or sites, located on or near the site that are over 45 years old listed in or eligible for listing in national, state, or local preservation registers ? If so, specifically describe. [help]

This non-project action will not impact any existing buildings, structures or sites. Such impacts will be considered for subsequent project work as part of a site specific review for SEPA compliance.

b. Are there any landmarks, features, or other evidence of Indian or historic use or occupation? This may include human burials or old cemeteries. Are there any material evidence, artifacts, or areas of cultural importance on or near the site? Please list any professional studies conducted at the site to identify such resources. [help]

This non-project action will not impact any existing landmarks or historic resources. Such impacts will be considered for subsequent project work as part of a site specific review for SEPA compliance.

c. Describe the methods used to assess the potential impacts to cultural and historic resources on or near the project site. Examples include consultation with tribes and the department of archeology and historic preservation, archaeological surveys, historic maps, GIS data, etc. [help]

Not applicable to this non-project action.

d. Proposed measures to avoid, minimize, or compensate for loss, changes to, and disturbance to resources. Please include plans for the above and any permits that may be required. [help]

None applicable for this non-project action.

14. **Transportation** [help]

a. Identify public streets and highways serving the site or affected geographic area and describe proposed access to the existing street system. Show on site plans, if any. [help]

State Route 2 and State Route 9 are the main connectors between the City and the surrounding areas. There is also an extensive system of arterials and local streets. The location of and access to streets and highways will be evaluated for subsequent project work as part of the site specific review for SEPA compliance.

b. Is the site or affected geographic area currently served by public transit? If so, generally describe. If not, what is the approximate distance to the nearest transit stop? [help]

The City is served by Community Transit, which provides fixed routes, paratransit, zone service and van pools.

c. How many additional parking spaces would the completed project or non-project proposal have? How many would the project or proposal eliminate? [help]

This non-project action will not impact parking. Parking requirements for subsequent project work will need to comply with city code.

d. Will the proposal require any new or improvements to existing roads, streets, pedestrian, bicycle or state transportation facilities, not including driveways? If so, generally describe (indicate whether public or private). [help]

This non-project action will not necessitate new or improved streets. Transportation facilities will be evaluated for subsequent project work as part of the site specific review for SEPA compliance.

e. Will the project or proposal use (or occur in the immediate vicinity of) water, rail, or air transportation? If so, generally describe. [help]

The various modes of transportation will be evaluated for subsequent project work as part of the site specific review for SEPA compliance, but are not expected to utilize transportation.

f. How many vehicular trips per day would be generated by the completed project or proposal? If known, indicate when peak volumes would occur and what percentage of the volume would be trucks (such as commercial and non-passenger vehicles). What data or transportation models were used to make these estimates? [help]

This non-project action will not impact vehicular trips. Trip generation and cumulative impact will be evaluated for subsequent project work as part of the site specific review for SEPA compliance, but are not expected to have any significant impact.

g. Will the proposal interfere with, affect or be affected by the movement of agricultural and forest products on roads or streets in the area? If so, generally describe. [help]

This non-project action will not impact movement of agricultural or forest products. Such impacts will be evaluated for subsequent project work as part of the site specific review for SEPA compliance.

h. Proposed measures to reduce or control transportation impacts, if any: [help]

None applicable for this non-project action.

15. **Public Services** [help]

a. Would the project result in an increased need for public services (for example: fire protection, police protection, public transit, health care, schools, other)? If so, generally describe. [help]

This non-project action will not impact public services. The need for public services will be evaluated for subsequent project work as part of the site specific review for SEPA compliance.

b. Proposed measures to reduce or control direct impacts on public services, if any. [help]

None applicable for this non-project action.

16. **Utilities** [help]

a. Circle utilities currently available at the site: [help]
electricity, natural gas, water, refuse service, telephone, sanitary sewer, septic system, other _____

b. Describe the utilities that are proposed for the project, the utility providing the service, and the general construction activities on the site or in the immediate vicinity which might be needed. [help]

This non-project action will not directly impact utilities. However, potential projects in the recommended CIP will directly impact sanitary sewer and may require new or modified electricity, water and/or telephone utilities to support the project. The provision of utilities will be evaluated for subsequent project work as part of the site specific review for SEPA compliance.

C. Signature [help]

The above answers are true and complete to the best of my knowledge. I understand that the lead agency is relying on them to make its decision.

Signature: _____

Name of signee _____

Position and Agency/Organization _____

Date Submitted: _____

D. supplemental sheet for nonproject actions [help]

(**IT IS NOT NECESSARY** to use this sheet for project actions)

Because these questions are very general, it may be helpful to read them in conjunction with the list of the elements of the environment.

When answering these questions, be aware of the extent the proposal, or the types of activities likely to result from the proposal, would affect the item at a greater intensity or at a faster rate than if the proposal were not implemented. Respond briefly and in general terms.

1. How would the proposal be likely to increase discharge to water; emissions to air; production, storage, or release of toxic or hazardous substances; or production of noise?

This non-project action will not effect discharges to water, emissions to air or production, storage or release of hazardous substances. However, potential projects in the recommended CIP are meant to accommodate growth within the sewer service area. This growth will result in increased discharge from the WWTP to the Snohomish River. Potential projects in the recommended CIP are also meant to further reduce the occurrence of combined sewer overflows and inflow into the sanitary sewer system, which would conversely decrease discharge to the Snohomish River in certain instances. Potential projects in the recommended CIP

are meant to eliminate the use of chlorine gas for disinfection at the WWTP and replace it with a less hazardous chemical as well.

Proposed measures to avoid or reduce such increases are:

Effects on discharges to water, emissions to air or production, storage or release of hazardous substances and recommended mitigation measures will be evaluated as part of a project specific review for SEPA compliance.

2. How would the proposal be likely to affect plants, animals, fish, or marine life?

This non-project action will not affect plants, animals, fish or marine life. However, as stated above, potential projects in the recommended CIP may impact discharge to the Snohomish River where fish and marine life are present. Such impacts will be evaluated as part of a project specific review for SEPA compliance.

Proposed measures to protect or conserve plants, animals, fish, or marine life are:

Effects on plants, animals, fish or marine life will be evaluated as part of a project specific review for SEPA compliance. This may include a new mixing zone study to evaluate the impacts on increased volume of discharge and changes in discharge characteristics from the WWTP into the Snohomish River.

3. How would the proposal be likely to deplete energy or natural resources?

This non-project action will not impact energy or natural resources. Impacts to energy and natural resources will be evaluated as part of a project specific review for SEPA compliance.

Proposed measures to protect or conserve energy and natural resources are:

Subsequent project will be reviewed under the Washington State Energy Code and city code for compliance with applicable regulations aimed at conserving energy and natural resources.

4. How would the proposal be likely to use or affect environmentally sensitive areas or areas designated (or eligible or under study) for governmental protection; such as parks, wilderness, wild and scenic rivers, threatened or endangered species habitat, historic or cultural sites, wetlands, floodplains, or prime farmlands?

This non-project action does not impact environmentally sensitive areas or areas designated for protection. Effects on such areas will be evaluated as part of a project specific review for SEPA compliance.

Proposed measures to protect such resources or to avoid or reduce impacts are:

Although potential projects in the recommended CIP are not anticipated to impact environmentally sensitive areas or areas designated for protection, the impacts to these areas will be evaluated as part of a project specific review for SEPA compliance to determine if mitigation measures are necessary and applicable.

5. How would the proposal be likely to affect land and shoreline use, including whether it would allow or encourage land or shoreline uses incompatible with existing plans?

This non-project action will not impact shoreline and land use. Impacts from subsequent project work will be evaluated as part of a site specific review for SEPA compliance.

Proposed measures to avoid or reduce shoreline and land use impacts are:

Although potential projects in the recommended CIP are not anticipated to significantly impact shoreline and land use, impacts to shoreline and land use will be evaluated as part of a project specific review for SEPA compliance to determine if any identified impacts can be reduced or avoided altogether.

6. How would the proposal be likely to increase demands on transportation or public services and utilities?

This non-project action will not impact demands on transportation, public services or utilities. Impacts from subsequent project work will be evaluated as part of a site specific review for SEPA compliance.

Proposed measures to reduce or respond to such demand(s) are:

Not applicable to this non-project action. Potential projects in the recommended CIP are not anticipated to impact demands for transportation or public services. However, potential projects in the recommended CIP are meant to increase the capacity, reliability and performance of sanitary sewer service. Additionally, new or modified electrical or water service may be needed in support of those projects. Impacts to utilities, transportation and public services from subsequent project work will be evaluated as part of a site specific review for SEPA compliance. Additionally, subsequent project work will be reviewed by the city planning department, public works department and local health district.

7. Identify, if possible, whether the proposal may conflict with local, state, or federal laws or requirements for the protection of the environment.

This non-project action is consistent with all local, state and federal requirements for protection of the environment. Subsequent project work will also be reviewed to maintain consistency with local, state and federal requirements.

Appendix B

Washington State Department of Ecology Agreed Order No. 10467 and Notice of Compliance

STATE OF WASHINGTON
DEPARTMENT OF ECOLOGY
Northwest Regional Office • 3190 160th Ave SE • Bellevue, WA 98008-5452 • 425-649-7000
711 for Washington Relay Service • Persons with a speech disability can call 877-833-6341

March 10, 2015

The Honorable Karen Guzak
Mayor, City of Snohomish
116 Union Avenue
Snohomish, WA 98290

Dear Mayor Guzak:

RE: City of Snohomish Wastewater Treatment Plant Agreed Orders

Enclosed is Notice of Compliance with Agreed Order No. 7973 (Near-Term Improvements), and Amended Agreed Order No. 10467, amending Agreed Order No. 7974 (Wastewater System Improvements). If you have any questions concerning the content of the document, please contact Amy Jankowiak at 425-649-7195 or amy.jankowiak@ecy.wa.gov.

Sincerely,

Kevin C. Fitzpatrick
Water Quality Section Manager

Enclosure

By Certified Mail No.: 7008 1140 0000 2359 9067

cc: Larry Bauman, City Manager, Snohomish
Steve Schuller, Public Works Director, Snohomish
Grant Weed, City Attorney, Snohomish
Josh Baldi, Northwest Regional Director, Ecology
Mark Henley, Municipal Unit Supervisor, Ecology
Amy Jankowiak, Compliance Specialist, Ecology
Shawn McKone, Facility Manager, Ecology
Ron Lavigne, Assistant Attorney General
Central Files: City of Snohomish WWTP; Permit No.WA0029548; WQ 6.4

STATE OF WASHINGTON
DEPARTMENT OF ECOLOGY

IN THE MATTER OF A COMPLIANCE ORDER AGAINST City of Snohomish	))))	NOTICE OF COMPLIANCE WITH AGREED ORDER NO. 7973 AND AMENDED AGREED ORDER NO. 10467 AMENDING AGREED ORDER NO. 7974

To: The Honorable Karen Guzak
Mayor, City of Snohomish
116 Union Avenue
Snohomish, WA 98290

For the site located at: City of Snohomish Wastewater Treatment Plant at
2115 Second Street, Snohomish, WA 98203

Agreed Orders numbered 7973 and 7974 were signed and effective with the City of Snohomish on September 10, 2010, and Agreed Order number 10467, which amended Agreed Order number 7974, was signed and effective on February 10, 2014; all requiring compliance with certain conditions imposed in accordance with chapter 90.48 of the Revised Code of Washington (RCW).

The Department of Ecology has determined that the City of Snohomish has satisfied the conditions in Agreed Order number 7973 and Agreed Order number 10467 amending Agreed Order number 7974.

As part of this compliance of the Agreed Orders, conditions 12-15 Options A and B of Amended Agreed Order 10467 and 7974 for the connection to the City of Everett are not required.

Dated March 10, 2015, at Bellevue, Washington

Kevin C. Fitzpatrick
Water Quality Section Manager

STATE OF WASHINGTON
DEPARTMENT OF ECOLOGY
Northwest Regional Office • 3190 160th Avenue SE • Bellevue, Washington 98008-5452 • (425) 649-7000

January 21, 2014

The Honorable Karen Guzak
Mayor, City of Snohomish
116 Union Avenue
Snohomish, WA 98290

Dear Mayor Guzak:

RE: City of Snohomish Wastewater Treatment Plant

Enclosed are two originals of Amended Agreed Order No. 10467, which amends Agreed Order No. 7974, originally effective on September 10, 2010. The enclosed Order may not be appealed by the City of Snohomish. Please review and sign the Amended Agreed Order and return it to the Department of Ecology, Northwest Regional Office at the address below by February 10, 2014. I will countersign the Amended Agreed Order, and one of the originals will be sent to you promptly. The Amended Agreed Order will be effective on the date the agreement has been signed by both parties.

All correspondence relating to this document should be directed to Enforcement Coordinator at Department of Ecology, Northwest Regional Office, 3190 – 160th Avenue SE, Bellevue, WA 98008-5452. If you have any questions concerning the content of the document, please contact Amy Jankowiak at 425-649-7195 or amy.jankowiak@ecy.wa.gov.

Sincerely,

Kevin C. Fitzpatrick
Water Quality Section Manager

Enclosures: two originals of Amended Agreed Order No. 10467
Registered Mail: RB 336 146 650 US

cc: Larry Bauman, City Manager, Snohomish
Steve Schuller, Public Works Director, Snohomish
Grant Weed, City Attorney, Snohomish
Daniel Weinberg, Support Services Director, Snohomish
Lori Terry Gregory, Counsel, Foster Pepper LLC
Larry Altose, Ecology PIO
Josh Baldi, Northwest Regional Director, Ecology
Mark Henley, Municipal Unit Supervisor, Ecology
Amy Jankowiak, Compliance Specialist, Ecology
Shawn McKone, Facility Manager, Ecology
Donald A. Seeberger, Interim Water Quality Program Manager, Ecology
Ron Lavigne, Assistant Attorney General
Central Files: City of Snohomish WWTP; Permit No.WA0029548; WQ 6.4

STATE OF WASHINGTON
DEPARTMENT OF ECOLOGY

IN THE MATTER OF AN ADMINISTRATIVE ORDER AGAINST: City of Snohomish	))))	AMENDED AGREED ORDER No. 10467 AMENDMENT NO. 1 OF AGREED ORDER No. 7974

To: The Honorable Karen Guzak
Mayor, City of Snohomish
116 Union Avenue
Snohomish, WA 98290

For the site located at: City of Snohomish Wastewater Treatment Plant at 2115 Second Street, Snohomish, WA 98203

This is Amendment No. 1 to Agreed Order No. 7974 between the Department of Ecology (Ecology) and the City of Snohomish (City) to achieve and maintain long-term compliance with the City's NPDES Permit No. WA0029548 (Permit), Chapter 90.48 Revised Code of Washington (RCW) and Chapters 173-221 and 173-220 Washington Administrative Code (WAC) by taking certain actions which are described below.

Agreed Order No. 7974, effective September 10, 2010 (attached), is amended in part as follows:

IV. COMPLIANCE SCHEDULE AND ACTIONS REQUIRED

In accordance with RCW 90.48.120(2) IT IS AGREED, that the City must take the following actions by the dates (milestones) set forth below. These actions are necessary to satisfy the requirements of Chapters 173-220 WAC, 173-221 WAC, and 173-240 WAC. The City has participated in defining these actions and the milestones by which they shall be completed. The City must also submit permit applications and documents needing approvals from the various government agencies in a timely fashion in order to meet the milestone dates for the various tasks.

Wastewater System Improvements and Compliance Schedule:

This Agreed Order contains a Compliance Schedule for completing wastewater system improvements in order to provide for ongoing compliance with the City's current and future Permit requirements.

Compliance Schedule:

1. **Submit a CSO Separation Plan by February 28, 2014.**

   The CSO Separation Plan must include a prioritized list of all projects in areas identified for separation and timelines for project sequencing, planning, design, and construction completion milestones. The City may include portions of the City's Draft Stormwater Comprehensive Plan that includes a section on the City's proposed Phase 2 CSO Separation Project as well as portions of previous plans to provide detail on all separation projects. The report shall include project area descriptions and drawings, estimates of what percentage of stormwater flows will be removed from the combined sanitary sewer system, updated construction cost estimates, and a preliminary analysis of the modifications required to the existing 30-acre lagoon to treat stormwater in accordance with the City's adopted Stormwater Management Manual.

2. **Submit a CSO Flow Management Plan by February 28, 2014.**

The CSO Flow Management Plan must include strategies for meeting the State regulation (WAC 173-245-020(22) of no more than one untreated CSO discharge per outfall per year on average from the City's two CSO outfalls while conveying wet weather flows to the wastewater treatment plant (WWTP) without exceeding the plant's permitted design capacity for influent flow, BOD and TSS loading and the approved peak flow design criteria for the plant as listed in approved engineering reports. The plan will describe when the City will be prepared to turn-down or shut off the CSO pump station and conduct an overflow to the Snohomish River through CSO overflow No. 2, while remaining in compliance with WAC 173-245-020(22).

3. **Submit a Draft Biosolids Sampling Plan by February 28, 2014.**

The Draft Biosolids Sampling Plan must, at a minimum, include the following details: a diagram showing proposed sample locations in each lagoon for sludge depth and sludge quality; a description of sampling protocols and parameters to be tested by an accredited lab using standard methods in accordance with WAC 173-308-140 "Biosolids Sampling and Analysis." Ecology will provide written comments within 30 days of receipt of the Draft Biosolids Sampling Plan.

4. **Submit a Final Biosolids Sampling Plan by April 30, 2014.**

The Final Biosolids Sampling Plan must incorporate and respond to comments by Ecology.

5. **Submit a Preliminary SFF Media Performance Assessment Report (Preliminary Media Report) by February 28, 2014.**

The Preliminary Media Report must provide an initial assessment of the SFF Media System's compliance with the "Performance and Design Requirements" in the Ecology-approved Project Manual for the Wastewater Treatment Plant Near-Term Improvements (Approved December 8, 2011). Part 2.02B of Section 13010 of the approved specifications state "The SFF Media System shall be required to treat the design flows and loadings listed in Table 1 while meeting the performance requirements specified in Table 2." Both tables are attached to this amendment. Table 1 provides anticipated influent quality for flow, $CBOD_5$, Ammonia-N, TKN, TSS, pH, and Temperature. Table 2 provides required effluent quality for filtered and unfiltered $CBOD_5$ and Ammonia-N. The report must summarize all influent and effluent sampling data collected since the beginning of the performance demonstration period, which started on July 1, 2013. The report must evaluate compliance with the performance standards, verify that influent quality was consistent with the design flows and loadings, and identify any trends that may point to positive or negative impacts on effluent limit parameters listed in the NPDES permit. Sampling for this report must conform to the *Snohomish WWTP Near-Term Improvements Sampling and Testing Plan* submitted to Ecology on May 21, 2013. The period must include July-December 2013 and should include any January 2014 data if possible. If the Preliminary Media Report determines that performance does not comply with the required performance criteria in Table 2 at a 95% confidence level, the City must submit a **diagnostic report**, as set forth in task 6 below.

6. **Submit a Diagnostic Report on SFF Media Filter by April 30, 2014, if necessary.**

If a diagnostic report is required under preceding task, the City must submit a Diagnostic Report by February 28, 2014, that identifies measures the City will take to correct deficiencies in order to achieve compliance with the required performance in the approved design documents and with the City's NPDES Permit effluent limitations.

7. **Submit a Disinfection System Upgrade Plan by April 30, 2014.**

The Disinfection System Upgrade Plan must include an evaluation of the following disinfection-related items for potential upgrades: alarm annunciation, sensors and PLCs, adequate electrical supply and backup power, reliability and redundancy of disinfection, chlorine feed system for adequate disinfection including alternatives to the existing practice of using chlorine gas, strategies to prevent river flooding into the contact chamber including a description of the current protocol, plans to prevent flooding and measures to ensure effluent is conveyed through the outfall when the river level is high, and an assessment of the capacity of the facility's generator and switch gears and whether the systems are adequately sized to operate all vital components during a power outage to meet reliability requirements.

8. **Submit a Draft Biosolids Management Plan by August 29, 2014.**

The Draft Biosolids Management Plan must include the following details: all results for the sampling completed in accordance with the sampling plan; an assessment of biosolids quality; an evaluation of the depth of the sludge layer in each lagoon; a calculation of the dry weight of solids in each lagoon; a description of the sludge removal and disposal options with at least three options identified and preferred alternative selected from the options and evaluated to confirm viability; estimated schedule for solids removal and disposal; proposed method of removal; and proposals for treatment and/or beneficial use of biosolids removed from the lagoons including location of disposal. The plan must also include an application for coverage under the General Biosolids Permit.

9. **Complete a Lagoon Lining Evaluation Scope of Work by August 31, 2014.**

The Scope of Work must include the following minimum content: name of testing company, qualifications of the testing company, detailed description of testing strategies, identification of conditions that may limit the test results, a schedule for on-site work, and a schedule for report development. The scope must also require the testing company to present recommendations for methods of correcting leaks detected during testing.

10. **Complete a Lagoon Lining Evaluation by October 31, 2014.**

The lagoon lining evaluation must evaluate the integrity of all the plant's lagoon liners and be conducted by a reputable company with at least 10 years experience in lagoon liner integrity assessments and leak detection evaluations. The testing must use an electrical-based leak location assessment method specified in ASTM Standard D7007, or other equivalent method. Testing must be capable of detecting a hole 3 millimeters (approximately $1/8^{th}$ inch) in its longest dimension, unless the testing contractor determines that site conditions make this level of accuracy unobtainable. The contractor must provide equipment calibration records that validate the accuracy achievable based on site conditions. The lagoon lining evaluation must also assess the general condition of the liners, including exposed side slopes and tops of the liners.

11. **Submit a Lagoon Lining Evaluation Report by December 31, 2014.**

The Lagoon Lining Evaluation Report must include a plan and schedule for addressing any leaks or other deficiencies in the lagoon liner(s) as a result of the evaluation. Potential options for addressing liner leaks or deficiencies include, but are not limited, to repair, patching, and replacement.

Note: All submittals (i.e. Documents #1-11 listed above) must be stand-alone documents. To implement improvements identified in the above submittal reports as necessary and that are mutually determined by Ecology and the City, both parties agree to either amend this Agreed Order or put such new milestones into another Agreed Order. Should Ecology require improvements that the City does not mutually agree to, Ecology will affect these improvements through modifications of the facility's NPDES Permit, or in unilateral Administrative Orders. Unilateral Administrative Orders, modifications to NPDES Permits or re-issued NPDES Permits may be appealed before the Pollution Control Hearings Board.

Preliminary Media Report Results and Compliance Schedule:

If the Preliminary Media Report determines that a Diagnostic Report is triggered pursuant to paragraphs 5-6 above, but the SFF Media System may meet the performance criteria by implementing changes prescribed in the Diagnostic Report, the compliance schedule will follow the "Option B" timeline that results in an 18-month delay to the original Agreed Order No. 7974 schedule.

If the Preliminary Media Report determines that the SFF Media System is preliminarily meeting the SFF Media System performance criteria identified in No. 5 above and NPDES permit compliance was achieved for the same period, then the compliance schedule will follow the "Option B" timeline that results in an 18-month delay to the original Agreed Order No. 7974 schedule. Additionally, the City will not be required to work on "Option B" milestones until after July 31, 2014.

If the Preliminary Media Report determines that the SFF Media System failed to meet the SFF Media System performance criteria identified in No. 5 above and the media vendor is unable to present a remedy by March 31, 2014, then the compliance schedule will follow the "Option A" timeline that results in a nine month delay to the original Agreed Order No. 7974 schedule and the City will resume taking measures to connect to the City of Everett plant on that schedule.

Subject to the previous three paragraphs above, Ecology has sole discretion in determining whether a nine month extension, eighteen month extension or if another amendment will be granted or necessary and will notify the City by letter via certified mail.

Option A (9-month delay)

12. **Submit "bid ready" Plans and Specifications and draft Construction Quality Assurance Plan by September 23, 2014.**
13. **Bid construction by September 30, 2014.**
14. **Award construction by April 30, 2015.**
15. **Substantial Completion of Construction by August 13, 2017.**

Option B (18-month delay)

12. **Submit "bid ready" Plans and Specifications and draft Construction Quality Assurance Plan by June 23, 2015.**
13. **Bid construction by June 30, 2015.**
14. **Award construction by January 31, 2016.**
15. **Substantial Completion of Construction by May 13, 2018.**

Conditions to rescind Compliance Schedule items 12-15 (Everett connection):

If the results of the full year of sampling conform with standards and the City is in compliance with all of the compliance schedule items 1-11, and other NPDES permit requirements, as of January 31, 2015, items No. 12-15 for Option A and Option B are not required. A letter verifying the same will be issued by Ecology.

If the results of the full year of sampling do not conform with the standards, but may meet requirements with changes prescribed in the Diagnostic Report and the City is in compliance with its NPDES permit and compliance schedule items 1-11, Ecology will consider whether or not Compliance Schedule items No. 12-15 for Option A or Option B will be required. The decision will be at Ecology's discretion and will consider the long-term effectiveness of the SFF Media System as well as overall permit and Agreed Order compliance and will notify the City of its decision by letter via certified mail.

VI. STIPULATED PENALTIES

If the City of Snohomish fails to meet milestone #2, 7, 8, 10 or 15 (if required), as set forth in the Compliance Schedule in Section IV of this Agreed Order, the City agrees to pay Ecology a stipulated penalty in the amount of $10,000 for each week that the date is missed. Ecology may waive this penalty if the City has demonstrated a good faith effort to meet the deadline AND Ecology concurs that the delay is attributable to delays in acquiring permits/approvals from government agencies, or other unforeseeable causes that are outside of the City's control. If the City believes delays during construction may impact the ability to achieve milestone #14 (if required) and such delay is necessary and customary based on standard construction practices, such delay must be documented through a Change Order and submitted to Ecology for approval in order for Ecology to waive stipulated penalties for that milestone.

If the City fails to submit monthly progress reports by the 15th of each month, the City agrees to pay Ecology a stipulated penalty in the amount of $500 for each late or missing monthly report. Ecology may waive this penalty if the City has demonstrated a good faith effort to meet the deadline AND Ecology concurs that the delay was outside of the City's control.

If stipulated penalties are imposed, Ecology shall notify the City in writing through a notice of Stipulated Penalty. Stipulated Penalties are due within 30 days of receipt of Ecology's notification or, in the event of a dispute, within 30 days of the completion of dispute resolution process under Section X of Agreed Order 7974 as amended, for such amount as may be determined to be due, and are not otherwise appealable.

XII. ORDER AMENDMENTS

This Order may be Amended only by mutual written agreement of the parties.

XIII. THIRD PARTY RIGHT TO APPEAL

By signing this Agreed Order, the City may not appeal this Agreed Order, however, a third party may.

A party other than the City has a right to appeal this Order to the Pollution Control Hearing Board (PCHB) within 30 days of the date of receipt of this Order. The appeal process is governed by Chapter 43.21B RCW and Chapter 371-08 WAC. "Date of receipt" is defined in RCW 43.21B.001(2).

To appeal you must do the following within 30 days of the date of receipt of this Order:

- File your appeal and a copy of this Order with the PCHB (see addresses below). Filing means actual receipt by the PCHB during regular business hours.
- Serve a copy of your appeal and this Order on Ecology in paper form - by mail or in person (see addresses below). E-mail is not accepted.

You must also comply with other applicable requirements in Chapter 43.21B RCW and Chapter 371-08 WAC.

Your appeal alone will not stay the effectiveness of this Order. Stay requests must be submitted in accordance with RCW 43.21B.320.

ADDRESS AND LOCATION INFORMATION

Street Addresses:	Mailing Addresses:
Department of Ecology Attn: Appeals Processing Desk 300 Desmond Drive SE Lacey WA 98503 **Pollution Control Hearings Board** 4224 – 6th Avenue SE Rowe Six, Building 2 Lacey WA 98503	**Department of Ecology** Attn: Appeals Processing Desk PO Box 47608 Olympia WA 98504-7608 **Pollution Control Hearings Board** PO Box 40903 Olympia WA 98504-0903

No other provision of Agreed Order No. 7974 is affected by this amendment.

SIGNATURES

_____ _____

Karen Guzak, Mayor
City of Snohomish

Date

_____ _____

Kevin C. Fitzpatrick
Water Quality Section Manager
Northwest Regional Office
Department of Ecology

Date

Appendix C

Wastewater Financial Gap Analysis

Appendix D

Population Analysis Methodology Memo

MEMORANDUM

Date: April 18, 2016

To: Brooke Eidem; Max Selin; Tim Jackson, City of Snohomish

CC: Tom Giese, BHC Consultants

From: Abby Weber, BHC Consultants

Subject: Population Analysis; City of Snohomish General Sewer Plan Update

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The data and methodology used to establish current and projected population estimates for two contributing populations – residents and employees – for the City of Snohomish General Sewer Plan Update are presented in this memo.

Year 2016 is the "current year". The baseline year is the most recent year from which data is available, however, the baseline year differs for each contributing population. In instances where 2016 data was not available, a population was calculated by interpolation between the baseline and future projections. The target years for population projections are 2022 (used for 6-year CIP), 2036 (used for 20 year CIP), and build-out.

The projection methodology uses Snohomish County's adopted 2035 Population and Employment Growth Targets for the City of Snohomish UGA for target year 2035, and uses the adopted targets as an interpolation point for 2022, 2036 and build-out.

Conversations with the City of Snohomish regarding demographic data and local planning activities have informed this methodology and ensured consistency with comprehensive planning efforts.

DATA

The following resources and data were utilized in establishing baseline population and projections:

- 2010 Census
    - Population data by Census Block
    - Average household size, 3.41
- City of Snohomish
    - TAZ population and employment data, 2015-2035
- Snohomish County
    - 2012 Snohomish County Buildable Lands Report (BLR)
        - Parcel-level housing unit and employment capacity estimates
    - Countywide Planning Policies for Snohomish County
        - Adopted 2035 Population and Employment Growth Targets

- Parcel-level land use data
- Office of Financial Management (OFM)
  - 2015 City population estimate
  - 2015 Small Area Estimate Program (SAEP) Census Block-Group population estimates
- Puget Sound Regional Council (PSRC)
  - 2014 basin-level Covered Employment Estimates

RESIDENTIAL POPULATION

Baseline

Year 2015 served as the residential population baseline. Population estimates were calculated using County land use data, 2010 Census population by block, and OFM 2015 SAEP population growth estimates by block-group, and adjusted based on 2015 TAZ housing unit data.

Census block-groups consist of multiple Census blocks. Census block and block-group data were joined spatially using GIS, and a 2015 population estimate was established for each block. The residential density was calculated for each census block (the ratio of population to total residential acreage). Residential parcels were identified using Snohomish County Property Use Codes and GIS parcel data. The residential density was then applied to the acreage of each individual residential parcel to produce a 2015 population estimate per parcel. The parcel population values were re-aggregated by basin.

Census-based basin population estimates were evaluated against to 2015 TAZ housing unit counts and aerial imagery, and adjusted accordingly.

Projected

Population figures were interpolated for 2016 and 2022 between the 2015 baseline estimates and 2035 projections for each basin. Growth targets were distributed to buildable residential lands, and added to the baseline for the total 2035 population estimate.

Snohomish County adopted a 2035 Population Growth Target of 10,657 for the City of Snohomish UGA. The target was distributed throughout the service area based on development capacity to establish 20-year projections for each basin. The population analysis utilized the Snohomish County 2012 Buildable Lands Report (BLR) data to establish the development capacity per parcel. BLR data was obtained from the County for the City of Snohomish UGA. The BLR identifies parcels as vacant, partially used, or re-developable. The BLR provides the additional housing unit capacity per parcel.

The development capacity is calculated for each parcel as its additional capacity divided by the total Snohomish UGA capacity, resulting in the percentage of total growth captured per parcel, or development capacity. The 2035 population growth target was distributed to each parcel based on development capacity, and re-aggregated by basin. Target-based population estimates were evaluated against to 2035 TAZ housing unit counts, and adjusted accordingly.

Population figures were then interpolated for 2016 and 2022, and extrapolated for 2036, between 2015 baseline data and 2035 projections.

Build-Out

Build-out calculations also utilized the BLR. Prior to market reductions, the BLR housing unit capacity equates to the build-out capacity. The parcel-level housing unit capacity was aggregated by basin, and increased by a factor of 15% to account for previous market-factor reductions. The build-out population was calculated for each basin as a function of adjusted housing unit capacity and average household size.

EMPLOYMENT POPULATION

Baseline

Year 2014 served as the baseline employment population year. Covered employment estimates for 2014 were obtained from Puget Sound Regional Council (PSRC) staff by custom data request. Covered employment refers to positions covered by the WA Unemployment Insurance Act, and accounts for approximately 85-90% of all employment. Covered Employment basin estimates were adjusted based on 2015 TAZ employment data.

Projected

Employment population figures were interpolated for 2016 and 2022 between the 2014 baseline estimates and 2035 projections for each basin. Growth targets were distributed to buildable commercial and industrial lands, and added to the baseline for the total 2035 population estimate.

Snohomish County adopted a 2035 Employment Growth Target of 6,941 for the City of Snohomish UGA. This adopted target was distributed throughout the service area based on development capacity to establish 20-year projections for each basin. The employment analysis utilizes Snohomish County BLR data to establish the commercial and industrial development capacity per parcel. BLR GIS data was obtained for the City of Snohomish UGA, which provides the additional employment capacity per parcel.

The development capacity is calculated for each parcel as its additional employment capacity divided the total UGA employment capacity, resulting in the percentage of total employment growth captured per parcel. The 2035 employment growth target was distributed to each parcel based on development capacity, and re-aggregated by basin. Target-based employment estimates were evaluated against to 2035 TAZ housing unit counts, and adjusted accordingly. Employment figures were then interpolated for 2016 and 2022, and extrapolated for 2036, between baseline data and 2035 employment growth targets.

Build-Out

Similar to residential growth, employment build-out calculations also utilized the BLR. The BLR GIS data provides the additional employment capacity for each parcel. The BLR parcel-level employment capacity was aggregated by basin, and increased by a factor of 15-percent to

account for prior market-reductions. The revised employment capacity is added to the baseline to establish the build-out employment population.

Appendix E

WWTP DMR Flow Data Summary

City of Snohomish
WWTP DMR Flow Data Summary

Month	Flow Rate (mgd)									
	2006	**2007**	**2008**	**2009**	**2010**	**2011**	**2012**	**2013**	**2014**	**2015**
January	2.51	1.66	1.65	1.90	1.72	2.18	1.81	2.24	1.46	1.50
February	1.57	1.19	1.23	0.93	1.30	1.48	2.04	1.32	1.63	1.71
March	1.19	1.74	1.42	1.38	1.16	2.27	2.29	1.55	2.56	1.21
April	1.06	1.07	1.47	1.39	1.38	1.77	1.48	1.51	1.27	0.98
May	0.96	0.84	1.04	1.10	1.29	1.52	1.46	0.85	1.05	0.70
June	0.87	0.83	1.13	0.73	1.49	1.15	1.47	0.83	0.65	0.65
July	0.66	0.74	0.75	0.67	0.71	0.86	0.88	0.62	0.65	0.57
August	0.66	0.76	0.79	0.70	0.72	0.76	0.81	0.63	0.67	0.67
September	0.78	0.79	0.76	0.78	1.05	0.77	0.72	0.85	0.76	0.65
October	0.81	0.98	0.84	1.24	0.95	0.87	1.09	0.74	1.40	0.79
November	2.32	1.15	1.75	2.16	1.42	1.39	1.87	1.13	1.77	1.84
December	2.11	2.35	1.70	1.31	1.99	1.03	2.73	1.08	2.10	2.67
Average Annual	**1.29**	**1.17**	**1.21**	**1.19**	**1.26**	**1.34**	**1.55**	**1.11**	**1.33**	**1.16**
Maximum Month	**2.51**	**2.35**	**1.75**	**2.16**	**1.99**	**2.27**	**2.73**	**2.24**	**2.56**	**2.67**

Appendix F

WWTP Hydraulic Analysis

HYDRAULIC PROFILE - 2036 PEAK HOUR/DAY FLOW

PROJECT: Snohomish WWTP
PIPING SYSTEM: From River to Mechanical Screen
Project Number: 16-10487.00
Date: 10/18/2016 (Updated 3/28/2018)
Engineer: Greg Mockos

BASIC DATA

Total Headloss = Friction Loss + Minor Losses

Friction Loss = $10.44 \times L \times Q^{1.85} / C_{HW}^{1.85} \times D^{4.8655}$ (Hazen-Williams Equation)

Minor Losses = $K \times V^2/2g$

Friction Factor, C_w: 120

Downstream WS Elevation 22.75

OUTFALL PIPE WITH 4 DIFFUSERS (ORIFICES) TO OUTFALL MH

Flow	9.42	mgd
Flow	6542	gpm
Length of Pipe	132.5	ft
Diameter	30	in
Velocity	2.97	fps

Minor Losses	No.	K	K_{total}
Entrance Loss	1	0.5	0.5
Exit Loss	0	1.0	0.0
Total K			0.5

Outfall Orifice Port Loss

$Q = CA*(2gH)^{1/2}$ $H = (Q/CA)^2/2g$

Q = flow, cfs
C = discharge coefficient
A = area of port, ft^2
H = head, ft

Number of Ports: 4
Width: 12 inches

Flow/Port 3.64 cfs
C = 0.75
A = 0.79 ft^2
Total Port Area: 3.14 ft^2

Port Loss, ft: **0.59 ft**
Port Exit Loss, ft: **0.33 ft**
Total Headloss in Piping Section, ft **0.22 ft**

WS Elevation in Manhole at Start of Outfall: **23.89 ft**

FROM OUTFALL MANHOLE TO CCB

Flow:	6542	gpm		14.57493186
Length of Pipe	**7.5**	ft		
Diameter	**30**	in		
Velocity	2.97	fps		

Minor Losses	No.	K	K_{total}
Entrance Loss	**1**	0.5	0.5
Exit Loss	**1**	1.0	1.0
Total K			1.5

Total Headloss in Piping Section, ft **0.21 ft**

WS Elevation Downstream of CCB Flap Gate **24.11 ft**

CCB FLAP GATE HEADLOSS

Flow:	6542	gpm
Length of Pipe	**1**	ft
Diameter	**30**	in
Velocity	2.97	fps

Fittings	No.	K	K_{total}
Swing Check Valve	1	2	2
Total K			2.0

Total Flap Gate Headloss, ft **0.27 ft**

WS Elevation Downstream of CCB Weir, ft **24.38 ft**

Sharp Crested Weir - Submerged

$Q/Q_1 = [1-(H_2/H_1)^{1.5}]^{0.385}$

$Q_1 = 3.33\ (L-0.2H_1)H_1^{3/2}$

Location/Description:

Weir Length:		**7.25**	ft
Weir Crest Elevation:		**17.79**	
Downstream WS El.:		**24.38**	
	H_2 =	6.5921	ft
	H_1 =	6.5934	ft
	Q_1 =	334.39	cfs
	Q =	14.57	cfs
	Q =	9.42	mgd

Total Weir Headloss, ft **0.00 ft**

WS Elevation Upstream of CCB Weir, ft **24.38 ft**

FROM WEIR TO CCB INLET

CCB Sluice Gate (square gate)

$Q = CA*(2gH)^{1/2}$ $H = (Q/CA)^2/2g$

Q = flow, cfs
C = discharge coefficient
A = area of port, ft^2
H = head, ft

Number of Sluice Gates:	**2**	
Width:	**24**	inches
Flow/Port	14.57	cfs
C =	**0.60**	
A =	4.00	ft^2
Total Area:	8.00	ft^2

Sluice Gate Loss, ft: **0.14 ft**

WS Elevation Upstream of V-Notch Weir, ft **24.53 ft**

CCB Channel #1

Input	n =	**0.015**	
	Channel Width =	**3.50**	ft
	Channel Depth =	17.58	ft
	Channel Length =	**25.6**	ft

Results

p =	38.67	ft
a =	61.54	ft

Flow =	3271	gpm
Flow =	7.29	cfs
Velocity =	0.12	fps
Slope, s =	0.0000008	ft/ft

Headloss, ft **0.0000 ft**
WS Elevation @ Upstream End **24.53 ft**

CCB Channel Bend #1

Depth @ Downstream End	17.73	ft
Width of Channel	**3.5**	ft
Velocity	0.1175	fps
Velocity Head	0.0002	ft

Bend Loss = K x $V^2/2g$ K = 0.5

Bend Loss **0.000 ft**
Total # of Bends **2**
Total CCB Bend Loss **0.000**
WS Elevation Channel #1 **24.53 ft**

CCB Channel #2

Input	n =	**0.015**		
	Channel Width =	**3.50**	ft	
	Channel Depth =	17.73	ft	
	Channel Length =	**51**	ft	
Results				
	p =	38.95	ft	
	a =	62.04	ft	
Flow =	3271	gpm		
Flow =	7.29	cfs		
Velocity =	0.12	fps		
Slope, s =	0.0000008	ft/ft		
Headloss, ft				**0.0000 ft**
WS Elevation @ Upstream End				**24.53 ft**

CCB Channel Bend #2

Depth @ Downstream End			17.73 ft	
Width of Channel			**3.5** ft	
Velocity			0.1175 fps	
Velocity Head			0.0002 ft	
Bend Loss = K x $V^2/2g$		K =	0.5	
Bend Loss				**0.000 ft**
Total # of Bends				**2**
Total CCB Bend Loss				**0.000**
WS Elevation Channel #2				**24.53 ft**

CCB Channel #3

Input	n =	**0.015**		
	Channel Width =	**3.50**	ft	
	Channel Depth =	17.73	ft	
	Channel Length =	**51**	ft	
Results				
	p =	38.95	ft	
	a =	62.04	ft	
Flow =	3271	gpm		
Flow =	7.29	cfs		
Velocity =	0.12	fps		
Slope, s =	0.0000008	ft/ft		
Headloss, ft				**0.0000 ft**
WS Elevation @ Upstream End				**24.53 ft**

CCB Channel Bend #3

Depth @ Downstream End		17.73	ft
Width of Channel		3.5	ft
Velocity		0.1175	fps
Velocity Head		0.0002	ft
Bend Loss = K x $V^2/2g$	K =	0.5	
Bend Loss			**0.000 ft**
Total # of Bends			**2**
Total CCB Bend Loss			**0.000**
WS Elevation Channel #3			**24.53 ft**

CCB Channel #4

Input	n =	**0.015**	
	Channel Width =	**3.50**	ft
	Channel Depth =	17.73	ft
	Channel Length =	**54.5**	ft
Results	p =	38.95	ft
	a =	62.05	ft
Flow =	3271	gpm	
Flow =	7.29	cfs	
Velocity =	0.12	fps	
Slope, s =	0.0000008	ft/ft	
Headloss, ft			**0.0000 ft**
WS Elevation @ Upstream End			**24.53 ft**

CCB Channel Bend #4

Depth @ Downstream End		17.73	ft
Width of Channel		3.5	ft
Velocity		0.1175	fps
Velocity Head		0.0002	ft
Bend Loss = K x $V^2/2g$	K =	0.5	
Bend Loss			**0.000 ft**
Total # of Bends			**2**
Total CCB Bend Loss			**0.000**
WS Elevation Channel #4			**24.53 ft**

FROM CCB TO CHLORINE MIXING MH TEE

Flow:	3271	gpm
Length of Pipe	**27.5**	ft
Diameter	**24**	in
Velocity	2.32	fps

Minor Losses	No.	K	K_{total}	
90^{o} Bend	1	0.3	0.3	
Run of Standard Tee	1	0.6	0.6	
Entrance Loss	1	0.5	0.5	
Exit Loss	1	1.0	1.0	
Total K			2.4	

Total Headloss in Piping Section, ft **0.23 ft**

WS Elevation in CHLORING MIXING MH: **24.75 ft**

FROM CHLORINE MIXING MH TEE TO CHLORINE MIXING MH

Flow:	6542	gpm
Length of Pipe	7.25	ft
Diameter	24	in
Velocity	4.64	fps

Minor Losses	No.	K	K_{total}
Entrance Loss	1	0.5	0.5
Total K			0.5

Total Headloss in Piping Section, ft **0.19 ft**

Chloring Mixing Baffle Elevation, ft **15.59 ft**
Head on Mixing Baffle, ft **9.35 ft**

WS Elevation in CHLORING MIXING MH: **24.94 ft**

FROM EFFLUENT CONTROL STRUCTURE (ECS) TO CHLORINE MIXING MH

Flow:	6542	gpm	14.6
Length of Pipe	20	ft	
Diameter	30	in	
Velocity	2.97	fps	

Minor Losses	No.	K	K_{total}
Entrance Loss	1	0.5	0.5
45 Bend	2	0.2	0.4
Exit Loss	1	1.0	1.0
Total K			1.9

Total Headloss in Piping Section, ft **0.28 ft**

WS Elevation in CHLORINE MIXING MH: **25.23 ft**

EFFLUENT PARSHALL FLUME

Flow:	6542	gpm
Flow:	14.57	cfs
Throat Width:	3	ft
Upstream Head:	1.13	ft

Invert of Flume	**20.47 ft**
WS Upstream of Flume	**25.23 ft**

ECS CHANNEL

Input

n =	**0.015**	
Channel Width =	**7.00**	ft
Channel Depth =	5.90	ft
Channel Length =	**19**	ft

Results

p =	18.79	ft
a =	41.27	ft

Flow =	6542	gpm
Flow =	14.57	cfs
Velocity =	0.35	fps
Slope, s =	0.0000045	ft/ft

Headloss, ft	**0.0001 ft**
WS Elevation Upstream End of ECS	**25.23 ft**

FROM LAGOON #4 OUTLET STRUCTURE TO ECS

Flow:	6542	gpm
Length of Pipe	**58.57**	ft
Diameter	**42**	in
Velocity	1.51	fps

Minor Losses	No.	K	K_{total}
60^{o} Bend	**1**	0.3	0.3
Entrance Loss	**1**	0.5	0.5
Exit Loss	**1**	1.0	1.0
Total K			1.8

Total Headloss in Piping Section, ft	**0.08 ft**
WS Elevation in Outlet Structure Effluent Chamber	**25.30 ft**

LAGOON #4 OUTLET STRUCTURE LOSSES

Submerged Lagoon Outlet pipe with slide gate (24")

$Q = CA*(2gH)^{1/2}$ $H = (Q/CA)^{2}/2g$

Q = flow, cfs
C = discharge coefficient
A = area of port, ft^2
H = head, ft

Number of Ports:	**1**	
Width:	**24**	inches

Flow/Port		14.57	cfs
C =		**0.60**	
A =		3.14	ft^2
	Total Port Area:	3.14	ft^2

Port Loss, ft: **0.93 ft**

WS Elevation in Outlet Structure Influent Chamber **26.23 ft**

Submerged Weir Loss

$Q/Q_1 = [1-(H_2/H_1)^{1.5}]^{0.385}$

$Q_1 = 3.33\ (L-0.2H_1)H_1^{3/2}$

Weir Length:		**6**	ft
Weir Crest Elevation:		**21.00**	ft
Downstream WS El.:		26.23	ft
	H_2 =	5.2314533	ft
	H_1 =	5.2354543	ft
	Q_1 =	197.58	cfs
	Q =	14.57	cfs
	Q =	9.42	mgd

Weir Headloss, ft **0.00 ft**

WS Upstream of Outlet Structure Weir (Lagoon #4 WS) **26.24 ft**

LAGOON #4 HEADLOSS

Flow:	6542	gpm		
Length of Pipe	**307.7**	ft		
Diameter	**1320**	in		(110 ft)
Velocity	0.00	fps		

Minor Losses	No.	K	K_{total}
90^o Bend	**1**	0.3	0.3
Total K			0.3

Total Lagoon Headloss, ft **0.00 ft**

WS Elevation in Lagoon #4 **26.24 ft**

Lagoon #3 Spillway Headloss

$Q/Q_1 = [1-(H_2/H_1)^{1.5}]^{0.385}$

$Q_1 = 3.33\ (L-0.2H_1)H_1^{3/2}$

Spillway Length:		**21**	ft		
Spillway Crest Elevation:		**25.50**	ft		
Downstream WS El.:		26.24	ft		
	H_2 =	0.735	ft		
	H_1 =	0.750	ft	Qp =	1395 gpm
	Q_1 =	45.08	cfs	Friction Loss =	0.002 ft
	Q =	11.47	cfs	Minor Loss =	0.011 ft
	Q =	7.41	mgd	Hp =	0.014 ft
	$H_1 - H_2$ =	0.014	ft		

Lagoon #3 Swpillway Headloss, ft **0.01 ft**

WS Elevation in Lagoon #3 **26.25**

Lagoon #2 Spillway Headloss

$Q/Q_1 = [1-(H_2/H_1)^{1.5}]^{0.385}$

$Q_1 = 3.33\ (L-0.2H_1)H_1^{3/2}$

Spillway Length:		21 ft		
Spillway Crest Elevation:		25.50 ft		
Downstream WS El.:		26.25 ft		
	H_2 =	0.749 ft		
	H_1 =	0.763 ft	Qp =	1382 gpm
	Q_1 =	46.27 cfs	Friction Loss =	0.002 ft
	Q =	11.50 cfs	Minor Loss =	0.011 ft
	Q =	7.43 mgd	Hp =	0.014 ft
	$H_1 - H_2$ =	0.014 ft		

Lagoon #2 Swpillway Headloss, ft **0.01 ft**

WS Elevation in Lagoon #2 **26.26**

Lagoon #1 Spillway Headloss

$Q/Q_1 = [1-(H_2/H_1)^{1.5}]^{0.385}$

$Q_1 = 3.33\ (L-0.2H_1)H_1^{3/2}$

Spillway Length:		21 ft		
Spillway Crest Elevation:		25.50 ft		
Downstream WS El.:		26.26 ft		
	H_2 =	0.763 ft		
	H_1 =	0.776 ft	Qp =	1370 gpm
	Q_1 =	47.45 cfs	Friction Loss =	0.002 ft
	Q =	11.52 cfs	Minor Loss =	0.011 ft
	Q =	7.45 mgd	Hp =	0.013 ft
	$H_1 - H_2$ =	0.013 ft		

Lagoon #1 Swpillway Headloss, ft **0.01 ft**

WS Elevation in Lagoon #1 **26.28**

LAGOON #1 INLET TO MH 3

Flow:	5118 gpm		
Length of Pipe	52 ft		
Diameter	24 in		
Velocity	3.63 fps		

Minor Losses	No.	K	K_{total}
11.25° Bend	1	0.1	0.1
Entrance Loss	1	0.5	0.5
Exit Loss	1	1.0	1.0
Total K			1.6

Total Headloss in Piping Section, ft **0.44 ft**

WS Elevation in Outlet Structure Effluent Chamber **26.71 ft**

MH 3 TO HEADWORKS CHANNEL

Flow:	**5118**	gpm
Length of Pipe	**107.38**	ft
Diameter	**24**	in
Velocity	3.63	fps

Minor Losses	No.	K	K_{total}
90° Bend	**2**	0.3	0.6
45° Bend	**1**	0.2	0.2
Exit Loss	**1**	1.0	1.0
Total K			1.8

Total Headloss in Piping Section, ft **0.59 ft**

Input

n =	**0.015**	
Pipe Diameter=	**2.00**	ft
Channel Length =	**5.7**	ft

Results

p =	6.28	ft
a =	3.14	ft

Flow =	5118	gpm
Flow =	11.40	cfs
Velocity =	3.63	fps
Slope, s =	0.5	ft/ft

Headloss, ft **2.87 ft**

WS Elevation in Headworks Channel Downstream of Weir Crest **30.17 ft**

HEADWORKS LOSSES

Channel Discharge

Input

n =	**0.015**	
Channel Width =	**5.00**	ft
Channel Depth =	2.17	ft
Channel Length =	**10**	ft

Results

p =	9.34	ft
a =	10.85	ft

Flow =	10236	gpm
Flow =	22.81	cfs
Velocity =	2.10	fps
Slope, s =	0.0003683	ft/ft

Headloss, ft **0.00 ft**

WS Elevation @ Upstream End **30.17 ft**

Channel Bend

Depth @ Downstream End	2.17	ft
Width of Channel	5	ft
Velocity	0.1933	fps
Velocity Head	0.0006	ft

Bend Loss = K x $V^2/2g$ K = 0.5

Bend Loss **0.000 ft**
WS Elevation Channel #2 **30.17 ft**

Screen Channel

Input

n =	0.015	
Channel Width =	5.50	ft
Channel Depth =	2.17	ft
Channel Length =	15	ft

Results

p =	9.84	ft
a =	11.94	ft

Flow =	10236	gpm
Flow =	22.81	cfs
Velocity =	1.91	fps
Slope, s =	0.0002873	ft/ft

Headloss, ft **0.0043 ft**

Screen Headloss, ft **1.07 ft**

WS Elevation @ Upstream End of Multi-Rake Screen **31.25 ft**

Screen Sluice Gate (6ft x 3 ft gate)

$Q = CA*(2gH)^{1/2}$ $H = (Q/CA)^2/2g$

Q = flow, cfs
C = discharge coefficient
A = area of port, ft^2
H = head, ft

Number of Sluice Gates:	1	
Width:	66	inches
Height	36	inches
Flow/Port	22.81	cfs
C =	0.60	
A =	16.50	ft^2
Total Area:	16.50	ft^2

Sluice Gate Loss, ft: **0.082 ft**
31.25 ft
WS Elevation @ Upstream End Sluice Gate **31.25 ft**

Screen Channel (upstream)

Input

n =	0.015	
Channel Width =	5.50	ft
Channel Depth =	2.11	ft
Channel Length =	29	ft

Results

p =	9.71	ft
a =	11.58	ft

Flow =	10236	gpm
Flow =	22.81	cfs
Velocity =	1.97	fps
Slope, s =	0.0003123	ft/ft

Headloss, ft **0.009 ft**

WS Elevation @ Upstream End of Multi-Rake Screen **31.26 ft**

Screw Pump Crest Elevation, ft **32.71 ft**

Screw Pump Bottom Elevation, ft **12 ft**

SG 2003 and 2001 submerged rectangular port (4ft x 6 ft gate)

$Q = CA*(2gH)^{1/2}$ $H = (Q/CA)^2/2g$

Q = flow, cfs
C = discharge coefficient
A = area of port, ft^2
H = head, ft

Number of Sluice Gates:	2	
Width:	72	inches
Height	48	inches
Flow/Port	22.81	cfs
C =	0.60	
A =	24.00	ft^2
Total Area:	48.00	ft^2

Sluice Gate Loss, ft: **0.010 ft**

Common Channel

Input

n =	0.015	
Channel Width =	21.50	ft
Channel Depth =	3.00	ft
Channel Length =	6.69	ft

Results

p =	27.50	ft
a =	64.50	ft

Flow =	10236	gpm
Flow =	22.81	cfs
Velocity =	0.35	fps
Slope, s =	0.0000041	ft/ft

Headloss, ft				**0.00 ft**
Parshall Flume Channel				
Input	n =	**0.015**		
	Channel Width =	**5.50**	ft	
	Channel Depth =	3.00	ft	
	Channel Length =	**10**	ft	
Results				
	p =	11.50	ft	
	a =	16.50	ft	
Flow =	10236	gpm		
Flow =	22.81	cfs		
Velocity =	1.38	fps		
Slope, s =	0.0001203	ft/ft		
Headloss, ft				**0.001 ft**
WS Elevation Downstream of Parhsall Flume				**12.01 ft**

INFLUENT PARSHALL FLUME

Flow:	10236	gpm	
Flow:	22.81	cfs	
Throat Width:	**1.5**	ft	
Upstream Head:	2.38	ft	
Invert of Flume			**12.79 ft**
WS Upstream of Flume			**15.17 ft**

Appendix G

WWTP Process Capacity Analysis

City of Snohomish WWTP Process Capacity Overview							
Component	**Design**	**Projected**			**Metcalf & Eddy**		**Orange Book**
		Existing	**2022**	**2036**	**Typical**	**Range**	
Population	NA	9,569	10,348	14,505			
Flow, MGD							
Average Annual	2.04	1.30	1.40	1.95			
Maximum Month	2.80	2.73	2.80	3.22			
Peak Day	9.90	8.30	8.10	9.42			
Peak Hour	22.80	14.38	13.67	14.74			
BOD_5, lbs/day							
Average Annual	2,892	1,996	2,160	3,030			
Maximum Month	3,960	2,724	2,950	4,130			
TSS, lbs/day							
Average Annual	3,233	1,772	1,920	2,690			
Maximum Month	4,400	2,333	2,520	3,540			
Total Kjeldahl Nitrogen (TKN), lbs/day (Actual Design includes 30% Safety Factor)							
Average annual (452 lb/day)	348	276	300	420			
Maximum Month (607 lb/day)	467	362	390	550			
Ammonia-N, lbs/day (Actual Design includes 30% Safety Factor)							
Average annual (294 lb/day)	226	161	170	240			
Maximum Month (377 lb/day)	290	194	210	290			
Influent Parshall Flume							
Throat Width, inches	18	18	18	18			
Flume Capacity, MGD	25.0	25.0	25.0	25.0			
Flume Invert Elev., feet	12.79	12.79	12.79	12.79			
Downstream WS Elev., feet at peak hour flow	12.03	12.01	12.01	12.01			
Flume Submergence Ratio at Flume Capacity at PHF	-0.24	-0.33	-0.34	-0.33			

City of Snohomish WWTP Process Capacity Overview							
Component	Design	Projected			Metcalf & Eddy		Orange Book
		Existing	2022	2036	Typical	Range	
Influent Pumps							
Type	Large Screw	Large Screw	Large Screw	Large Screw			
Number, each	2	2	2	2			
Capacity, each, MGD	15.50	15.50	15.50	15.50			
Motor size, each, HP	100	100	100	100			
Type	Small Screw	Small Screw	Small Screw	Small Screw			
Number, each	1	1	1	1			
Capacity, each, MGD	3.50	3.50	3.50	3.50			
Motor size, each, HP	25	25	25	25			
Total Capacity, MGD	34.5	34.5	34.5	34.5			
Total Firm Capacity, MGD	19.0	19.0	19.0	19.0			
Screening							
Mechanical screens							
Type	Multiple-Rake	Multiple-Rake	Multiple-Rake	Multiple-Rake			
Number, each	1	1	1	1			
Opening size, in	0.25	0.25	0.25	0.25			
Capacity, each, MGD	23.5	23.5	23.5	23.5			
Screen channel width, each, feet	5.5	5.5	5.5	5.5			
Total Capacity, MGD	23.5	23.5	23.5	23.5			
Estimated Screen Headloss at PHF, feet	1.65	1.04	0.99	1.07			
Manual Bar Screen							
Number, each	1	1	1	1			
Opening size, in	0.75	0.75	0.75	0.75			
DUAL-POWER MULTI-CELLULAR AERATED LAGOON #1 - Complete-Mixed Cell Sizing							
Type	Complete Mix	Complete Mix	Complete Mix	Complete Mix			
Number, each	1	1	1	1			
Total Volume, MG	10	10	10	10	Pg 854 M&E (4th Ed.)		
HRT Max Month Flow, days	3.57	3.66	3.57	3.11	1.5-3		
Number of Surface Aerators	18	18	18	18			
Aerator HP, ea	15	15	15	15			
Total Aerator HP	270	270	270	270			
Minimum HRT @ Max Month Flow	2.50	2.50	2.50	2.50	> 2.5		
Complete-Mix Cell Volume Required	7.00	6.83	7.00	8.05			
Mixing Sizing					Pg 854 M&E (4th Ed.)		
Total Mixing Energy Input, HP/Mgal	27	27	27	27	25 - 30		
Total Required Mixing Energy, HP	250 - 300	250 - 300	250 - 300	250 - 300			
Aeration Sizing for Winter (without nitrification)							
r (diurnal peak ratio to average oxygen demand)	1.5	1.5	1.5	1.5			
S_0 (Influent max mo BOD at peak day flow), mg/L	170	120	126	154			
S_1 (effluent BOD), mg/L	0	0	0	0			

City of Snohomish WWTP Process Capacity Overview							
Component	Design	Projected			Metcalf & Eddy		Orange Book
		Existing	2022	2036	Typical	Range	
F (solids decay factor)	0.67	0.67	0.67	0.67			
Y (BOD growth yield)	0.5	0.5	0.5	0.5			
Ro_2 (max month oxygen consumption rate), kg/hr	112	77	83	116			
Ro_2 (max month oxygen consumption rate), lb/hr	246	169	183	256			
Aeration Sizing for Summer (with nitrification)							
Influent TKN load, kg/h (based on max month load)	8.82	6.84	7.37	10.39			
Oxygen demand for nitrification, kg O2/kg N	4.6	4.6	4.6	4.6			
Achievable % nitrification in complete-mix cell	23.0%	100.0%	100.0%	10.0%			
Ro_2 (max month oxygen consumption rate), kg/hr	121	108	117	121			
Ro_2 (max month oxygen consumption rate), lb/hr	266	239	258	267	Table	10-9 M&E 3rd Ed.	
Standard oxygen transfer rate, lbs/bhp/hr	1.75	1.75	1.75	1.75		1.8-2.0	
Salinity-surface tension correction factor (Beta factor)	0.95	0.95	0.95	0.95			
Oxygen saturation, mg/L	7.81	7.81	7.81	7.81			
Operating oxygen concentration, mg/L	2.00	2.00	2.00	2.00			
Oxygen saturation at standard conditions, mg/L	9.04	9.04	9.04	9.04			
Temperature, degrees C	27.9	27.9	27.9	27.9			
Oxygen transfer correction factor (Alpha factor)	0.85	0.85	0.85	0.85			
Actual oxygen transfer rate, lbs/hp/hr	1.08	1.08	1.08	1.08			
Motor Efficiency	92%	92%	92%	92%			
Total Required Aeration Energy, HP	270	242	261	270			

City of Snohomish WWTP Process Capacity Overview							
Component	Design	Projected			Metcalf & Eddy		Orange Book
		Existing	2022	2036	Typical	Range	
DUAL-POWER MULTI-CELLULAR AERATED LAGOONS #2/#3/#4 - Partial-Mixed Cell Sizing							
Type	Partial Mix	Partial Mix	Partial Mix	Partial Mix			
Number, each	3	3	3	3			
Volume Each, MG	3.5	3.5	3.5	3.5			
Total Volume, MG	10.5	10.5	10.5	10.5	Pg 854 M&E (4th Ed.)		
HRT @ Max Month Flow, days	3.75	3.85	3.75	3.26	3		
Operating Depth, ft	10.0	10.0	10.0	10.0			
Minimum Surface Area Required for Overflow, sf	132,000	110,667	108,000	125,600			
Available Surface Area for Overflow, sf							
Mid-Depth L, ft	140	140	140	140			
Mid-Depth W, ft	335	335	335	335			
Total Surface Area for Overfow	140,700	140,700	140,700	140,700			
Available Bottom Surface Area, sf							
Side Slope	3	3	3	3			
Bottom L, ft	110	110	110	110			
Bottom W, ft	305	305	305	305			
Bottom Area, each, sf	33,550	33,550	33,550	33,550			
Bottom Area, each, m2	3,117	3,117	3,117	3,117			
Required Bottom Area, each, sf	21,150	14,597	15,797	22,159			
Mixing Sizing					Pg 854 M&E (4th Ed.)		
Min Mixing Energy Intensity, HP/Mgal	5.0	5.0	5.0	5.0	5		
Max Mixing Energy Intensity, HP/Mgal	10.0	10.0	10.0	10.0	10		
Minimum Energy, HP	52.5	52.5	52.5	52.5			
Maximum Energy, HP	105	105	105	105			
Aeration Sizing							
Benthal oxygen demand at 20 deg. C, g O2/m2/d	60	60	60	60			
Temperature sensitivity coefficient	1.05	1.05	1.05	1.05			
Max Monthly Temperature, degrees C	27.9	27.9	27.9	27.9			
Ro2, Oxygen consumption rate, kg/hr	11.44	11.44	11.44	11.44			
Ro2, Oxygen consumption rate, lbs /hr	25.22	25.22	25.22	25.22			
Required Aeration Energy per each, HP	25.5	25.5	25.5	25.5			
Total Benthal Oxygen Demand, lbs/hr	76	76	76	76			
Total Required Aeration Energy, HP	76.5	76.5	76.5	76.5			
Number of Surface Aerators Each	3	3	3	3			
Aerator HP, ea	7.5	7.5	7.5	7.5			
Total Aerator HP	68	68	68	68			
Available Actual oxygen transfer rate from Aerators, lbs/hr	67	67	67	67			

City of Snohomish WWTP Process Capacity Overview							
Component	Design	Projected			Metcalf & Eddy		Orange Book
		Existing	2022	2036	Typical	Range	
SFF Blowers							
Type	Positive Disp.	Positive Disp.	Positive Disp.	Positive Disp.			
Number, each	3	3	3	3			
Capacity, each, scfm	1,000	1,000	1,000	1,000			
Total Firm Capacity, scfm	2,000	2,000	2,000	2,000			
Normal Continuous Aeration Rate to Each SFF Module, scfm	30	30	30	30			
Max Continuous Aeration Rate to each SFF Module, scfm	37	37	37	37			
Air Scour Rate to Each SFF Module, scfm	60	60	60	60			
Total Normal SFF Aeration Rate, scfm	2,000	2,000	2,000	2,000			
Coarse bubble oxygen transfer efficiency, %	7.5%	7.5%	7.5%	7.5%			
Standard oxygen transfer rate, lbs/hr	156	156	156	156			
Alpha factor for coarse bubble aeration	0.75	0.75	0.75	0.75			
Beta factor	0.95	0.95	0.95	0.95			
Assumed DO residual, mg/L	2.0	2.0	2.0	2.0			
Available Actual oxygen transfer rate from blowers, lbs/hr	84	84	84	84			
Remaining Nitrification Oxygen Demand, lbs/hr	69	0	0	95			
Benthal Oxygen Demand, lbs/hr	76	76	76	76			
Total Oxygen Demand for Partial-Mixed Cells, lbs/hr	145	76	76	171			
Total Available Oygen Transfer for Partial-Mixed Cells, lbs/hr	151	151	151	151			
Oxygen Supply Deficiency for Partial-Mixed Cells, lbs/hr	0	0	0	19			

City of Snohomish WWTP Process Capacity Overview							
Component	Design	Projected			Metcalf & Eddy		Orange Book
		Existing	2022	2036	Typical	Range	
SFF Media							
Modules per lagoon	18	18	18	18			
Total Modules	54	54	54	54			
Media Surface Area per Module, sf	3,549	3,549	3,549	3,549			
Total Media Surface Area	191,646	191,646	191,646	191,646			
Ammonia- N Removal Rate, lbs/d/1,000 sf	3.5	3.5	3.5	3.5			
Ammonia-N Removal Capacity, lbs/d	671	671	671	671			
Unused Surface Area Capacity @ Max Month Load, sf	58,217	88,217	80,217	34,503			
Minimum BOD Removal Rate, lbs/d/1,000 sf	25.0	25.0	25.0	25.0			
Available Add'l BOD Removal Capacity, lbs/d	1,455	2,205	2,005	863			
Effluent Parshall Flume							
Throat Width, inches	36	36	36	36			
Flume Capacity, MGD	50.0	50.0	50.0	50.0			
Flume Invert Elev., feet	20.47	20.47	20.47	20.47			
Downstream WS Elev., feet at peak hour flow	25.48	24.68	24.58	25.23			
Flume Submergence Ratio at Flume Capacity at PHF	4.28	4.05	3.99	4.21			
Effluent Sand Filters							
Filter Feed Pump							
Type	Submersible	Submersible	Submersible	Submersible			
Motor Size, HP	10	10	10	10			
Capacity, Each, MGD	0.8	0.8	0.8	0.8			
Total Capacity, MGD	1.4	1.4	1.4	1.4			
Total Firm Capacity, MGD	0.8	0.8	0.8	0.8			
Type	Backwash	Backwash	Backwash	Backwash			
Number of Filter Cells	2	2	2	2			
Number Filter Modules per Cell	2	2	2	2			
Filter Area Per Module, sf	50	50	50	50			
Total Filter Area, sf	200	200	200	200			
Filter Loading, gpm/SF	4.86	4.86	4.86	4.86			
Total Filtration Capacity, MGD	1.4	1.4	1.4	1.4			

City of Snohomish WWTP Process Capacity Overview							
Component	Design	Projected			Metcalf & Eddy		Orange Book
		Existing	2022	2036	Typical	Range	
Filter Reject Pumps							
Type	Submersible	Submersible	Submersible	Submersible			
Number	2	2	2	2			
Motor Size, HP	3.4	3.4	3.4	3.4			
Capacity, Each, gpm	100	100	100	100			
Total Dynamic Head, ft	51	51	51	51			
Total Firm Capacity, gpm	100	100	100	100			
Chlorine Contact Basin							
Number of Chambers	2	2	2	2			
Number of Chambers in Use	2	2	2	2			
Depth based on Peak Day Flow	12.95	12.83	12.82	12.91			
Total Contact Volume in Use, gallons	126,400	125,300	125,200	126,100			
HRT (contact time) , hr							
at annual avg flow, min	90	139	129	94			60
at design max month flow, min	66	67	65	57			60
at peak day flow, min	19	22	23	20			20-30
Disinfection							
Chlorination							
Type	Chlorine (Cl_2) Gas	Chlorine (Cl_2) Gas	Chlorine (Cl_2) Gas	Chlorine (Cl_2) Gas			
Chlorine Cylinder Capacity, lbs Cl2/day	160	160	160	160			
Total Chlorinator Capacity, lbs Cl2/day	200	200	200	200			
Chlorine Mixer Capacity, lbs Cl2/day	500	500	500	500			
Peak Chlorine Dosage with Nitrification (Jul - Oct), mg Cl_2/L	3.0	3.0	3.0	3.0			
Peak Chlorine Dosage w/o Nitrification (Nov - Jun), mg/Cl2/L	1.5	1.5	1.5	1.5			
Avg Annual Chlorine Demand with Nitrification, lbs/day	51	33	35	49			
Max Month Demand with Nitrification, lbs/day	70	68	70	81			
Peak Day Demand w/o Nitrification, lbs/day	124	104	101	118			
Dechlorination							
Type	Sulfur Dioxide Gas	Sulfur Dioxide Gas	Sulfur Dioxide Gas	Sulfur Dioxide Gas			
Sulfur Dioxide (SO2) Cylinder Capacity, lbs SO2/day	160	160	160	160			
Total Sulfonator Capacity, lbs SO2/day	200	200	200	200			
Typical SO2 Dosage, mg SO_2/L	1.0	1.0	1.0	1.0			
Avg Annual SO2 Demand, lbs/day	17	11	12	16			
Max Month SO2 Demand, lbs/day	23	23	23	27			
Peak Day SO2 Demand, lbs/day	83	69	68	79			

City of Snohomish WWTP Re-Rating Comparison		
Component	**Process Capacity**	
	2036	**Re-Rating**
Population	14,505	
Flow, MGD		
Average Annual	1.95	2.00
Maximum Month	3.22	3.30
Peak Day	9.42	9.65
Peak Hour	14.74	
BOD_5, lbs/day		
Average Annual	3,030	3,105
Maximum Month	4,130	4,233
TSS, lbs/day		
Average Annual	2,690	
Maximum Month	3,540	
Total Kjeldahl Nitrogen (TKN), lbs/day (Actual Design is 30% more)		
Average annual (452 lb/day)	420	
Maximum Month (607 lb/day)	550	564
Ammonia-N, lbs/day		
Average annual (294 lb/day)	240	
Maximum Month (377 lb/day)	290	
DUAL-POWER MULTI-CELLULAR AERATED LAGOON #1 - Complete-Mixed Cell Sizing		
Type	Complete Mix	Complete Mix
Number, each	1	1
Total Volume, MG	10	10
HRT Max Month Flow, days	3.11	3.03
Number of Surface Aerators	18	18
Aerator HP, ea	15	15
Total Aerator HP	270	270
Minimum HRT @ Max Month Flow	2.50	2.50
Complete-Mix Cell Volume Required	8.05	8.25
Mixing Sizing		
Total Mixing Energy Input, HP/Mgal	27	27
Total Required Mixing Energy, HP	250 - 300	250 - 300
Aeration Sizing for Winter (without nitrification)		
r (diurnal peak ratio to average oxygen demand)	1.5	1.5
S_0 (Influent max mo BOD at peak day flow), mg/L	154	154
S_1 (effluent BOD), mg/L	0	0
F (solids decay factor)	0.67	0.67
Y (BOD growth yield)	0.5	0.5
Ro_2 (max month oxygen consumption rate), kg/hr	116	119
Ro_2 (max month oxygen consumption rate), lb/hr	256	263
Aeration Sizing for Summer (with nitrification)		
Influent TKN load, kg/h (based on max month load)	10.39	10.65
Oxygen demand for nitrification, kg O2/kg N	4.6	4.6
Achievable % nitrification in complete-mix cell	10.0%	39.0%
Ro_2 (max month oxygen consumption rate), kg/hr	121	138

City of Snohomish WWTP Re-Rating Comparison		
Component	**Process Capacity**	
	2036	**Re-Rating**
Ro_2 (max month oxygen consumption rate), lb/hr	267	305
Standard oxygen transfer rate, lbs/bhp/hr	1.75	2.0
Salinity-surface tension correction factor (Beta factor)	0.95	0.95
Oxygen saturation, mg/L	7.81	7.81
Operating oxygen concentration, mg/L	2.00	2.00
Oxygen saturation at standard conditions, mg/L	9.04	9.04
Temperature, degrees C	27.9	27.9
Oxygen transfer correction factor (Alpha factor)	0.85	0.85
Actual oxygen transfer rate, lbs/hp/hr	1.08	1.23
Motor Efficiency	92%	92%
Total Required Aeration Energy, HP	270	270
DUAL-POWER MULTI-CELLULAR AERATED LAGOONS #2/#3/#4 - Partial-Mixed Cell Sizing		
Type	Partial Mix	Partial Mix
Number, each	3	3
Volume Each, MG	3.5	3.5
Total Volume, MG	10.5	10.5
HRT @ Max Month Flow, days	3.26	3.18
Operating Depth, ft	10.0	10.0
Minimum Surface Area Required for Overflow, sf	125,600	128,720
Available Surface Area for Overflow, sf		
Mid-Depth L, ft	140	140
Mid-Depth W, ft	335	335
Total Surface Area for Overfow	140,700	140,700
Available Bottom Surface Area, sf		
Side Slope	3	3
Bottom L, ft	110	110
Bottom W, ft	305	305
Bottom Area, each, sf	33,550	33,550
Bottom Area, each, m2	3,117	3,117
Required Bottom Area, each, sf	21,794	22,708
Mixing Sizing		
Min Mixing Energy Intensity, HP/Mgal	5.0	5.0
Max Mixing Energy Intensity, HP/Mgal	10.0	10.0
Minimum Energy, HP	52.5	52.5
Maximum Energy, HP	105	105
Aeration Sizing		
Benthal oxygen demand at 20 deg. C, g O2/m2/d	60	60
Temperature sensitivity coefficient	1.05	1.05
Max Monthly Temperature, degrees C	27.9	27.9
Ro2, Oxygen consumption rate, kg/hr	11.44	11.44
Ro2, Oxygen consumption rate, lbs /hr	25.22	25.22
Required Aeration Energy per each, HP	25.5	29.7
Total Benthal Oxygen Demand, lbs/hr	76	76

City of Snohomish WWTP Re-Rating Comparison		
Component	**Process Capacity**	
	2036	**Re-Rating**
Total Required Aeration Energy, HP	76.5	89.2
Number of Surface Aerators Each	3	3
Aerator HP, ea	7.5	7.5
Total Aerator HP	68	68
Available Actual oxygen transfer rate from Aerators, lbs/hr	67	57
SFF Blowers		
Type	Positive Disp.	Positive Disp.
Number, each	3	3
Capacity, each, scfm	1,000	1,000
Total Firm Capacity, scfm	2,000	2,000
Normal Continuous Aeration Rate to Each SFF Module, scfm	30	30
Max Continuous Aeration Rate to each SFF Module, scfm	37	37
Air Scour Rate to Each SFF Module, scfm	60	60
Total Normal SFF Aeration Rate, scfm	2,000	2,000
Coarse bubble oxygen transfer efficiencty, %	7.5%	7.5%
Standard oxygen transfer rate, lbs/hr	156	156
Alpha factor for coarse bubble aeration	0.75	0.75
Beta factor	0.95	0.95
Assumed DO residual, mg/L	2.0	2.0
Available Actual oxygen transfer rate from blowers, lbs/hr	84	84
Remaining Nitrification Oxygen Demand, lbs/hr	95	66
Benthal Oxygen Demand, lbs/hr	76	76
Total Oxygen Demand for Partial-Mixed Cells, lbs/hr	171	142
Total Available Oygen Transfer for Partial-Mixed Cells, lbs/hr	151	142
Oxygen Supply Deficiency for Partial-Mixed Cells, lbs/hr	19	0
SFF Media		
Modules per lagoon	18	18
Total Modules	54	54
Media Surface Area per Module, sf	3,549	3,549
Total Media Surface Area	191,646	191,646
Ammonia- N Removal Rate, lbs/d/1,000 sf	3.5	3.5
Ammonia-N Removal Capacity, lbs/d	671	671
Unused Surface Area Capacity @ Max Month Load, sf	34,503	30,599
Minimum BOD Removal Rate, lbs/d/1,000 sf	25.0	25.0
Available Add'l BOD Removal Capacity, lbs/d	863	765

Appendix H

WWTP Lagoon Capacity Calculations

DESIGN OF DUAL-POWER MULTI-CELLULAR AERATED LAGOONS - Complete-Mixed Cell Sizing (2016)

RED INDICATED VALUES REQUIRING INPUT

This design method is based on the following articles:

Rich, L. (1982). "Design Approach to Dual-Power Aerated Lagoons." *J. Environ. Eng.* , 108(3), 532–548.

Rich, L. (1996). "Modification of Design Approach to Aerated Lagoons." *J. Environ. Eng.* , 122(2), 149–153.

Sizing for Complete-Mix Cell based on COD Removal

COD removal is based on the premise that hydrolysis of biodegradable particulate matter is the rate limiting step (i.e., slower rate than the rate at which hydrolized or soluble waste is consumed), as long as oxygen supply is not limiting.

The steady-state mass balance for the complete-mix cell is:

$$X_S/X_{B,H} = K_X(1 + b_h(V/Q))/[(V/Q)(k_H - b_h) - 1]$$ (Eqn. 3 from 1996 article)

and

$$X_{B,H} = Y_H(X_{S0} + S_{S0})/[1 + (1 - 0.92Y_H)b_h(V/Q)]$$ (Eqn. 4 from 1996 article)

(Eqn. 4 is simplified version ignoring residual effluent concentration to solve for $X_{B,H}$, then plug that in to Eqn. 3 to solve for X_s)

Where:

$X_{B,H}$ =	?	= active heterotrophic biomass COD, mg/L
X_S =	?	= effluent biodegradable particulate COD, mg/L
K_x =	0.006	= 0.03 x (1.116)$^{T-20}$ = half-saturation constant, (mg biodegradable particulate COD)/(mg biomass COD)
b_h =	0.113	= 0.62 x (1.120)$^{T-20}$ = decay coefficient, 1/day
V =	?	= volume, m^3
Q =	**10,334**	= flow rate, m^3/d (based on 2016 max month flow)
k_H =	0.69	= 3 x (1.103)$^{T-20}$ = maximum specific hydrolysis rate, 1/day
Y_H =	0.67	= growth yield, (mg biomass COD)/(mg COD oxidized)
X_{SO} =	161	= influent biodegradable particulate COD, mg/L (typcially 64% of total COD [from BioWin influent specifier], rest is soluble or unbiodegradable)
S_{SO} =	40	= influent biodegradable soluble COD, mg/L (typically 16% of total COD [from BioWin influent specifier], rest is particulate or unbiodegradable)
T =	**5**	= temperature, degrees C (assumed minimum monthly average)
COD:BOD =	**2.1**	= ratio of influent COD to influent BOD (typically ranges from 1.9 to 2.2 for domestic wastewater [from BioWin influent specifier])
BOD_{Li} =	**2724**	= influent maximum month BOD_5 load, lbs/d
BOD_{Ci} =	120	= influent maximum month BOD_5 concentration, mg/L (max month load at max month flow)

Values for K_x, b_H, k_H and Y_H are taken from the *International Associaton on Water Pollution Research and Control* "Activated Sludge Model No. 1", 1986 (as summarized in Table 1 of 1996 article)

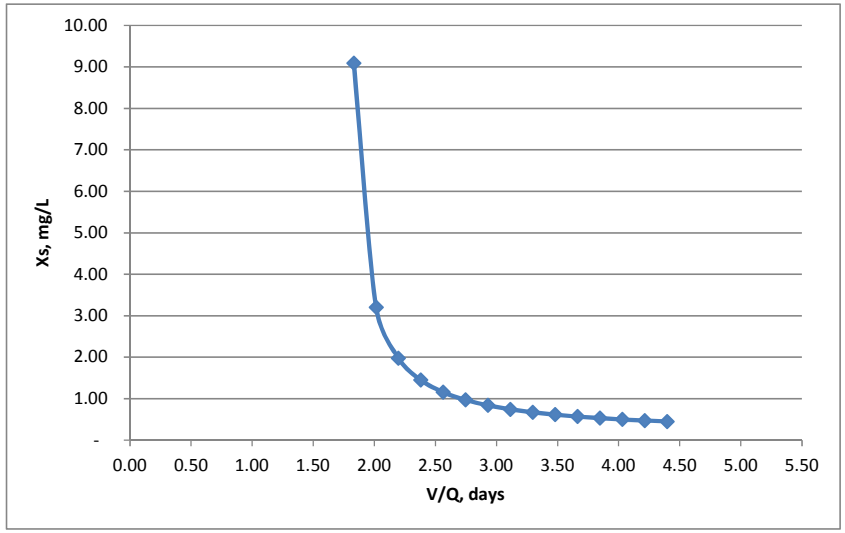
First, solve for $X_{B,H}$ based on multiple volumes using Eqn. 4:

V	V	$X_{B,H}$
(MG)	(m^3)	(mg/L)
5.00	18,927	71.93
5.50	20,820	72.43
6.00	22,712	72.93
6.50	24,605	73.44
7.00	26,498	73.95
7.50	28,391	74.48
8.00	30,283	75.01
8.50	32,176	75.55
9.00	34,069	76.09
9.50	35,961	76.65
10.00	37,854	77.21
10.50	39,747	77.78
11.00	41,640	78.36
11.50	43,532	78.94
12.00	45,425	79.54

Second, solve for X_S using the same volumes and values determined for $X_{B,H}$ using Eqn. 3:

NOTE: These values are based on max month flows and loads and minimum monthly average temperature

V	V	$X_S/X_{B,H}$	$X_{B,H}$	V/Q	X_S
(MG)	(m^3)		(mg/L)	(days)	(mg/L)
5.00	18,927	0.126391	71.93	1.83	9.09
5.50	20,820	0.044179	72.43	2.01	3.20
6.00	22,712	0.027123	72.93	2.20	1.98
6.50	24,605	0.019748	73.44	2.38	1.45
7.00	26,498	0.015634	73.95	2.56	1.16
7.50	28,391	0.013009	74.48	2.75	0.97
8.00	30,283	0.011189	75.01	2.93	0.84
8.50	32,176	0.009853	75.55	3.11	0.74
9.00	34,069	0.008831	76.09	3.30	0.67
9.50	35,961	0.008023	76.65	3.48	0.61
10.00	37,854	0.007368	77.21	3.66	0.57
10.50	39,747	0.006828	77.78	3.85	0.53
11.00	41,640	0.006373	78.36	4.03	0.50
11.50	43,532	0.005986	78.94	4.21	0.47
12.00	45,425	0.005652	79.54	4.40	0.45



Since this cell is complete-mix, there is minimal sludge accumulation, so there does not need to be signficant volume reserved for sludge accumulation, as is needed in the partial-mix cells.

Complete-Mix Cell Retention Time = **2.50** days

Complete-Mix Cell Volume = 6.83 MG

Critical Sizing for Complete-Mix Cell to Avoid Washout

The 1982 article provided the equation below to estimate the critical retention time to prevent washout of solids, since there is no solids recycle.

$$\left(\frac{V}{Q}\right)_{1C} = \frac{K_S + S_0}{\hat{\mu} S_0}$$

(Eqn. 3 from 1982 article)

Where:

K_S =	120	= saturation constant, mg/L (from Table 1, 1982 article)
S_0 =	39.4	= total influent BOD_5, mg/L (max month load at peak day flow)
μ =	3.1	= μ_{20} x $(1.10)^{T-20}$ = maximum specific growth rate at temp T, 1/d (from Eqn. 4, 1982 article)
$(V/Q)_{1C}$ =	?	= critical washout retention time, days
μ_{20} =	13.0	= maximum specific growth rate at 20 degrees C, 1/d (from Table 1, 1982 article)
T =	**5**	= temperature, degrees C (assumed minimum monthly average)
BOD_{Li} =	**2724**	= influent maximum month BOD_5 load, lbs/d (based on projected 2016 load)
PDF =	**8.30**	= peak day flow (peak day flow), MGD

$(V/Q)_{1C}$ = 1.30 days

Safety Factor = 1.92 we have a sufficiently sized lagoon at max month flow and BOD loading.

Minimum Safety 1.5

Verify OK!

DESIGN OF DUAL-POWER MULTI-CELLULAR AERATED LAGOONS - Partial-Mixed Cell Sizing (2016)

This design method is based on the following articles:
Rich, L. (1982). "Design Approach to Dual-Power Aerated Lagoons." *J. Environ. Eng.* , 108(3), 532–548.
Rich, L. (1996). "Modification of Design Approach to Aerated Lagoons." *J. Environ. Eng.* , 122(2), 149–153.

Sizing for Partial-Mix Cells based on Overflow Rate for Settling

1982 article indicates an overflow rate not exceeding 75 gpd/sf to remove most, if not all, settleable solids.
Base overflow on peak day flow to ensure adequate removal of solids under all conditions.

PDF =	**8.30**	= peak day flow, MGD
PDF =	8,300,000	= peak day flow, gpd
Max Overflow =	75	gpd/sf
Min Surface Area =	110,667	sf

Therefore, a surface area of about 132,000 sf should be provided to ensure sufficient settling at peak flows.
This area is based on the area at mid-depth. Therefore, the equivalent bottom area with 3:1 side slopes and total depth of 10 feet would be:

Mid-Depth L =	140	ft
Mid-Depth W =	335	ft
Mid-Depth Area =	140,700	sf (total mid-depth area for all 3 cells)
Total Depth =	**10**	ft
Side Slope =	**3**	:1
Bottom L =	110	ft
Bottom W =	305	ft
Bottom Area =	33,550	sf

The equivalent volume of this mid-depth area is:

Volume =	8,277,867	gal

This is approximately equivalent to the volume of the 3 lagoons (3.5 x 3= 10.5 MG).

Sizing for Partial-Mix Cells based on Sludge Loading

1982 article indicates that even with loading of biodegradable solids as high as 80 g/m^2/d, complete destruction of the solids can be achieved within an annual cycle.
The loading is estimated using Eqn. 6 from the 1986 article as follows:

$$L = \frac{YS_0Q}{nA_j\left[1 + k_d\left(\frac{V}{Q}\right)_1\right]}$$

(Eqn. 6 from 1982 article)

Where:

L =	80	=	biodegradable solids loading on partial-mix cells, g/m^2/d
Y =	0.5	=	growth yield (for BOD) (from Table 1, 1982 article)
S_0 =	184	=	total influent BOD_5, mg/L (average annual load at average annual flow)
Q =	**4920**	=	average annual flow rate, m^3/d (average annual flow)
n =	**3**	=	number of partial-mix cells
A_j =	?	=	bottom area of each partial-mix cell, m^2
k_d =	0.157	=	$k_{d20} \times (1.05)^{T-20}$ = decay rate, 1/d (Eqn. 5 from 1982 article)
k_{d20} =	0.2	=	decay rate at 20 degrees C, 1/d (from Table 1, 1982 article)
$(V/Q)_1$ =	2.5	=	hydraulic retention time in complete-mix cell, days
BOD_{Li} =	**1996**	=	average annual BOD_5 load, lbs/d
T =	**15**	=	average annual temperature, degrees C

A_j = 14,597 sf

Bottom area for sludge loading is less than is required for settling, so settling controls.

Sizing for Partial-Mix Cells based on Sludge Accumulation

1996 article indicates that measured sludge accumulation in operating facilities has been significantly less (up to 3 times) than estimated by information in the 1982 article.
The 1996 article cites sludge accumulation rates of 44 m^3/yr per L/s and 38 m^3/yr per L/s as measured over a 7-year period for two dual-power multi-cellular aerated lagoon processes in South Carolina.
Therefore, an accumulation rate of 40 m^3/yr per L/s will be used to estimate sludge accumulation. Given the high I/I and low influent BOD concentrations, this accumulation rate is likely still somewhat conservative. Over time, VSS will be reduced through digestion. It is conservatively assumed that 75% of the sludge solids is VSS and approx. 40% of VSS is destroyed, which is the minimum for Class B biosolids.

Average Flow =	**1.30**	= Average Annual Flow, MGD
Average Flow =	56.95	= Average Annual Flow, L/s
Avg. Accumulation =	**40**	= m^3/yr per L/s
Accum. Rate =	2278	= m^3/yr
Assumed % VSS =	75%	
Accum. VSS =	1709	= m^3/yr
Min. VSS Destroyed =	683	= m^3/yr
Net Accum. Rate =	1595	= m^3/yr
5-yr Accum. =	7,973	= m^3
5-yr Accum. =	2.1	= MG

So, size of the partial-mix cells should have about 3 MG of volume dedicated to sludge storage.

If approximately 3 feet of the 10 feet of total depth is allocated for sludge storage in the three partial-mix cells, the sludge storage is then as follows:

Total Depth =	10	ft
Sludge Depth =	3	ft
Partial-Mix Vol =	3.50	MG, each
Number Cells =	3	# partial-mix cells
Total Vol =	10.50	MG
Sludge Vol =	3.2	MG
Accum Duration =	7.5	yrs

3-foot sludge depth allows sufficient storage volume for approximately 7+ years conservatively and also maintains sufficient separation between the water surface and sludge layer to avoid effluent quality impacts from resuspension. Actual storage duration could be much longer if VSS destruction is greater that assumed minimum of 40% and/or % VSS is higher.

Sizing for Partial-Mix Cells based on Control of Algal Growth

The 1996 article uses the following equation to estimate algal growth rate in the partial-mix cells:

(Eqn. 5 from 1996 article)

$$(V/Q)_j = [1 - (X_{An}/X_{A0})^{-1/n}]/\mu_A$$

Where: hydraulic retention time in each partial-mix cell, days

$(V/Q)_j$ = algae in effluent of last cell, mg/L

X_{An} = algae in influent of first cell, mg/L

X_{A0} = number of cells in series

n = first-order growth rate for algae, 1/d

μ_A =

The graph below is a plot of total retention time in all partial-mix cells versus the ratio X_{An}/X_{A0}.

As stated in the 1996 article, a previous study indicated μ_A always less than 0.48 1/d.

Therefore, that value was used for μ_A.

The 1996 article states that for XAn/XA0 less than 25, significant algal growth should not result.

For 3 partial-mix cells in series, total retention time should not exceed about 4.4 days

At the current minimum monthly flow of about 600,000 gpd, that is equal to a total volume of only about 2.6 MG. Considering the wide range of flows from PDF to the minimum monthly flow to I/I and increases in flow due to growth, there is no way to both reasonably minimize retention time for algal growth, and have sufficient volume to accommodate overflow rates at peak flow and provide sufficient volume for sludge accumulation and sufficient area for sludge loading. As a result, the City has taken other measures to control algae including use of ultrasonic transducers.

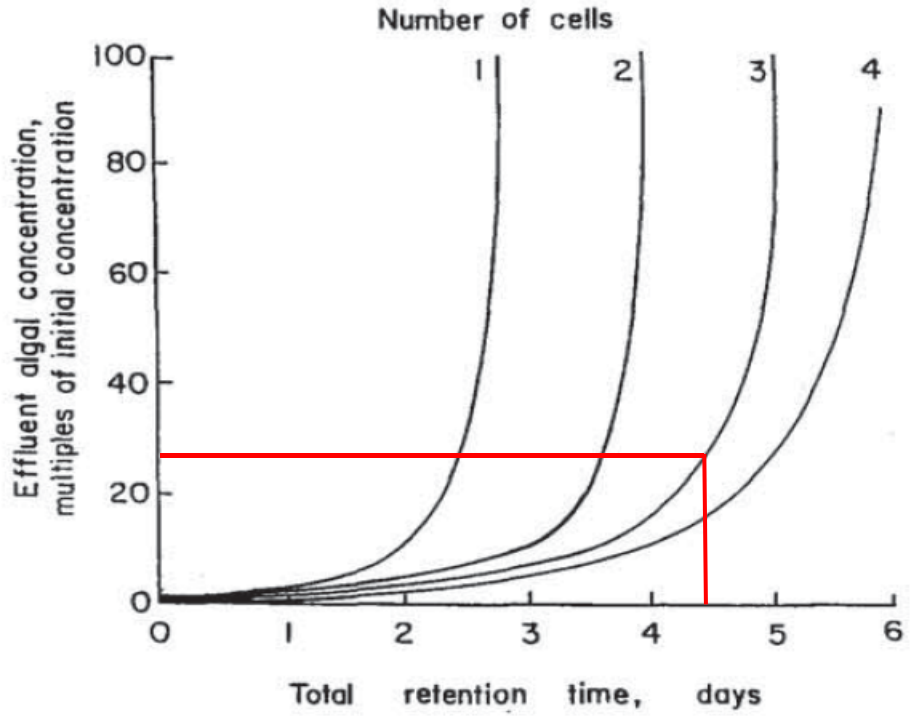


FIG. 4. Influence of Retention Time and Multicellular Configuration on Algal Growth [from Eq. (5)]

DESIGN OF DUAL-POWER MULTI-CELLULAR AERATED LAGOONS - Aeration and Mixing Sizing (2016)

This design method is based on the following articles:

Rich, L. (1982). "Design Approach to Dual-Power Aerated Lagoons." *J. Environ. Eng.* , 108(3), 532–548.
Rich, L. (1996). "Modification of Design Approach to Aerated Lagoons." *J. Environ. Eng.* , 122(2), 149–153.

For both the complete-mix and partial mix cells, the sizing of the aerators is based on the larger of the two power requirements for aeration and mixing.

Mixing Sizing for Complete-Mix Cells

For complete-mix cells, it has been found that a power level of 6 W/m^3 (30 hp/MG) is necessary to maintain solids in suspension and also maintain sufficientmixing intensity and turbidity to suppress algae growth (from 1982 article)
Therefore, the mixing energy required for the complete-mix cell ideally is:

Volume =	**10.0**	MG
Intensity =	30	hp/MG
Energy =	300	hp

Aeration Sizing for Complete-Mix Cells

Assuming substantial nitrification does not occur in the complete-mix cell (which would be the case during the winter months with higher flows and lower temperatures), the maximum oxygen consumption rate can be estimated by:

$$R_{O_2} = 4.16 \times 10^{-5} rQ(S_0 - S_1)(1.47 - 1.42\,FY)$$

(Eqn. 10 from 1982 article)

Where:

Ro_2 =	?	= oxygen consumption rate, kg/hr
r =	1.5	= diurnal peak ratio to average oxygen demand (assume 1.5 based on typical diurnal pattern)
Q =	**10,334**	= flow rate, m^3/d (max month flow)
S_0 =	120	= total influent BOD_5, mg/L (based on max month load at max month flow for 2016)
S_1 =	0	= complete-mix cell effluent soluble BOD_5, mg/L (conservatively assumed to be zero)

F =	0.67	= solids decay factor (typical value of 0.67 assumed per 1982 article)
Y =	0.5	= growth yield (for BOD) (from Table 1, 1982 article)
BOD_{Li} =	**2724**	= influent maximum month BOD_5 load, lbs/d
Ro_2 =	76.7	kg/hr
Ro_2 =	169.1	lbs/hr

The above oxygen demand represents the demand for the winter months. Compare with the demand for summer months with lower flow, same BOD load and nitrification.
Nitrification demand will be based on assuming complete nitrification and all influent organic-N hydrolyzing to ammonia:

Q =	**4,920**	= flow rate, m^3/d (based on 2016 average annual flow)
S_0 =	251	= total influent BOD_5, mg/L (max month load at peak day flow)
TKN_{Li} =	**362**	= total influent TKN load, lbs/d (max month)
TKN_{Li} =	6.84	= total influent TKN load, kg/h (max month)
Nit-O_2 =	4.6	= oxygen demand for nitrification, kg O_2/kg N
Nitrification =	100%	= % of ammonia nitrified in complete-mix cell

Ro_2 =	76.7	kg/hr	(without nitrification)
Ro_2 =	169.1	lbs/hr	Summer oxygen demand with no nitrification in complete-mix cell
Ro_2 =	108.18	kg/hr	(with nitrification)
Ro_2 =	238.5	lbs/hr	Summer oxygen demand with partial nitrification in complete-mix cell

Estimate the equivalent horsepower requirement to meet this oxygen demand using the following equation:

$$N = N_o\left(\frac{\beta C_{W_{alt}} - C_L}{C_{s_{20}}}\right) 1.024^{T-20}\alpha$$

(Eqn. 10-19 from M&E 3rd Ed.)

Where:

N =	?		= oxygen transfer rate under field conditions, lbs/hp/hr
N_0 =		**1.75**	= oxygen transfer rate under standard conditions, lbs/hp/hr (2.0 is typical design number from Table 10-9 M&E 3rd Ed., but experience has shown can be lower, use 1.5 for existing aerators)
β =		0.95	= salinity-surface tension correction factor (assume 0.9)
C_{Walt} =		**7.81**	= oxygen saturation at field temperature and elevation, mg/L (from M&E 3rd Ed. Appendix E , assuming max 25 degrees C temp)
C_L =		**2.00**	= minimum operating oxygen concentration under worst case conditions, mg/L
Cs_{20} =		**9.04**	= oxygen saturation at standard conditions, mg/L (from M&E 3rd Ed. Appendix E)
T =		**27.9**	= temperature, degrees C
α =		0.85	= oxygen transfer correction factor (assume 0.85)

N =	1.08	lbs/hp/hr	
Ro_2 with Nit =	238.5	lbs/hr	
Motor Eff. =	92%		
Req'd Energy =	242	hp	So, sufficient aeration energy to support full BOD removal and partial nitrification

Mixing Sizing for Partial-Mix Cells

For partial-mix cells, it has been found that a power level of 1 W/m^3 (5 hp/MG) is necessary to disperse dissolved oxygen throughout the lagoon and that a power level of up to 2 W/m^3 can still allow for sufficient settling of solids (from 1982 article)
Therefore, the mixing energy for the partial-mix cell should be:

Volume =	**10.5**	MG
Min Intensity =	5	hp/MG
Max Intensity =	10	hp/MG
Min Energy =	52.5	hp
Max Energy =	105.0	hp

Aeration Sizing for Partial-Mix Cells

As stated in the 1982 article, aeration demand in the partial-mix cells is dominated by residual BOD in the winter (when temperatures are low and benthal demand [i.e., oxygen consumption due to biological activity in the sludge layer] is low)
and by the benthal demand in the summer. In all but unusual conditions, the summer benthal demand is significantly larger. Therefore, the summer benthal demand will be used to determin aeration requirements for the partial-mix cells.
The benthal demand can be estimated using the following equation:

$$R_{O_2} = 4.16 \times 10^{-5} A_j B \theta^{T-20}$$

(Eqn. 12 from 1982 article)

Where:

Ro_2 =	?		= oxygen consumption rate, kg/hr
A_j =		3118	= bottom area of each partial-mix cell, m^2
B =		**60**	= rate of benthal oxygen demand at 20 deg. C, g $O_2/m^2/d$ (from 1982 article)
θ =		**1.05**	= temperature sensitivity coefficient (from 1982 article)
T =		**27.9**	= temperature, degrees C (assumed max month temperature)
Ro_2 =		11.44	kg/hr
Ro_2 =		25.23	lbs/hr
Req'd Energy =		25.5	hp

Additional air is supplied by the SFF media blowers to assist with benthal demand and nitrification.

DESIGN OF DUAL-POWER MULTI-CELLULAR AERATED LAGOONS - Complete-Mixed Cell Sizing (2022)

RED INDICATED VALUES REQUIRING INPUT

This design method is based on the following articles:

Rich, L. (1982). "Design Approach to Dual-Power Aerated Lagoons." *J. Environ. Eng.* , 108(3), 532–548.

Rich, L. (1996). "Modification of Design Approach to Aerated Lagoons." *J. Environ. Eng.* , 122(2), 149–153.

Sizing for Complete-Mix Cell based on COD Removal

COD removal is based on the premise that hydrolysis of biodegradable particulate matter is the rate limiting step (i.e., slower rate than the rate at which hydrolized or soluble waste is consumed), as long as oxygen supply is not limiting.

The steady-state mass balance for the complete-mix cell is:

$$X_S/X_{B,H} = K_X(1 + b_h(V/Q))/[(V/Q)(k_H - b_h) - 1]$$ (Eqn. 3 from 1996 article)

and

$$X_{B,H} = Y_H(X_{S0} + S_{S0})/[1 + (1 - 0.92Y_H)b_h(V/Q)]$$ (Eqn. 4 from 1996 article)

(Eqn. 4 is simplified version ignoring residual effluent concentration to solve for $X_{B,H}$, then plug that in to Eqn. 3 to solve for X_s)

Where:

$X_{B,H}$ =	?	= active heterotrophic biomass COD, mg/L
X_S =	?	= effluent biodegradable particulate COD, mg/L
K_x =	0.006	= 0.03 x (1.116)$^{T-20}$ = half-saturation constant, (mg biodegradable particulate COD)/(mg biomass COD)
b_h =	0.113	= 0.62 x (1.120)$^{T-20}$ = decay coefficient, 1/day
V =	?	= volume, m^3
Q =	**10,598**	= flow rate, m^3/d (based on 2022 max month flow)
k_H =	0.69	= 3 x (1.103)$^{T-20}$ = maximum specific hydrolysis rate, 1/day
Y_H =	0.67	= growth yield, (mg biomass COD)/(mg COD oxidized)
X_{SO} =	170	= influent biodegradable particulate COD, mg/L (typcially 64% of total COD [from BioWin influent specifier], rest is soluble or unbiodegradable)
S_{SO} =	42	= influent biodegradable soluble COD, mg/L (typically 16% of total COD [from BioWin influent specifier], rest is particulate or unbiodegradable)
T =	**5**	= temperature, degrees C (assumed minimum monthly average)
COD:BOD =	**2.1**	= ratio of influent COD to influent BOD (typically ranges from 1.9 to 2.2 for domestic wastewater [from BioWin influent specifier])
BOD_{Li} =	**2950**	= influent maximum month BOD_5 load, lbs/d
BOD_{Ci} =	126	= influent maximum month BOD_5 concentration, mg/L (max month load at max month flow)

Values for K_x, b_H, k_H and Y_H are taken from the *International Associaton on Water Pollution Research and Control* "Activated Sludge Model No. 1", 1986 (as summarized in Table 1 of 1996 article)

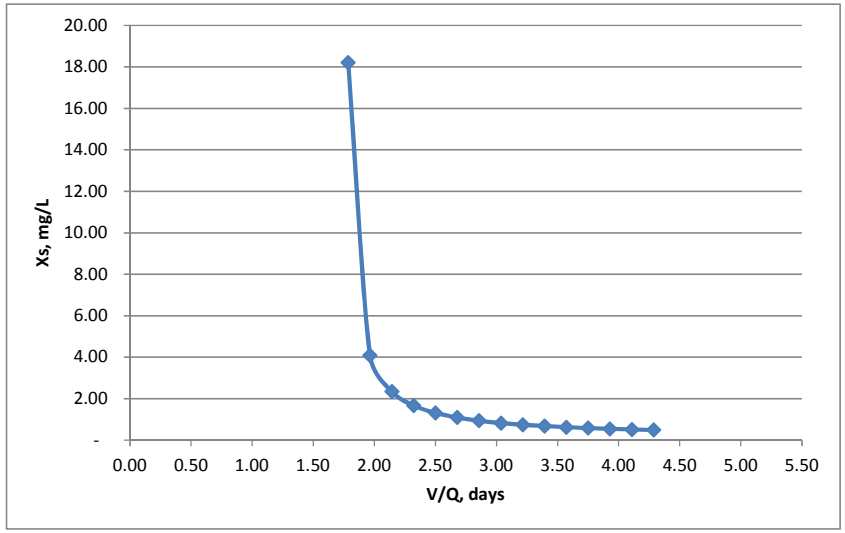
First, solve for $X_{B,H}$ based on multiple volumes using Eqn. 4:

V	V	$X_{B,H}$
(MG)	(m^3)	(mg/L)
5.00	18,927	75.83
5.50	20,820	76.34
6.00	22,712	76.85
6.50	24,605	77.38
7.00	26,498	77.90
7.50	28,391	78.44
8.00	30,283	78.98
8.50	32,176	79.53
9.00	34,069	80.09
9.50	35,961	80.66
10.00	37,854	81.23
10.50	39,747	81.82
11.00	41,640	82.41
11.50	43,532	83.01
12.00	45,425	83.62

Second, solve for X_S using the same volumes and values determined for $X_{B,H}$ using Eqn. 3:

NOTE: These values are based on max month flows and loads and minimum monthly average temperature

V	V	$X_S/X_{B,H}$	$X_{B,H}$	V/Q	X_S
(MG)	(m^3)		(mg/L)	(days)	(mg/L)
5.00	18,927	0.240079	75.83	1.79	18.21
5.50	20,820	0.053618	76.34	1.96	4.09
6.00	22,712	0.030615	76.85	2.14	2.35
6.50	24,605	0.021632	77.38	2.32	1.67
7.00	26,498	0.016845	77.90	2.50	1.31
7.50	28,391	0.013871	78.44	2.68	1.09
8.00	30,283	0.011843	78.98	2.86	0.94
8.50	32,176	0.010373	79.53	3.04	0.83
9.00	34,069	0.009258	80.09	3.21	0.74
9.50	35,961	0.008383	80.66	3.39	0.68
10.00	37,854	0.007678	81.23	3.57	0.62
10.50	39,747	0.007098	81.82	3.75	0.58
11.00	41,640	0.006613	82.41	3.93	0.54
11.50	43,532	0.006201	83.01	4.11	0.51
12.00	45,425	0.005846	83.62	4.29	0.49



Since this cell is complete-mix, there is minimal sludge accumulation, so there does not need to be signficant volume reserved for sludge accumulation, as is needed in the partial-mix cells.

Complete-Mix Cell Retention Time = **2.5** days

Complete-Mix Cell Volume = 7.00 MG

Critical Sizing for Complete-Mix Cell to Avoid Washout

The 1982 article provided the equation below to estimate the critical retention time to prevent washout of solids, since there is no solids recycle.

$$\left(\frac{V}{Q}\right)_{1C} = \frac{K_S + S_0}{\hat{\mu} S_0}$$

(Eqn. 3 from 1982 article)

Where:

K_S =	120	= saturation constant, mg/L (from Table 1, 1982 article)
S_0 =	43.7	= total influent BOD_5, mg/L (max month load at peak day flow)
μ =	3.1	= μ_{20} x $(1.10)^{T-20}$ = maximum specific growth rate at temp T, 1/d (from Eqn. 4, 1982 article)
$(V/Q)_{1C}$ =	?	= critical washout retention time, days
μ_{20} =	13.0	= maximum specific growth rate at 20 degrees C, 1/d (from Table 1, 1982 article)
T =	**5**	= temperature, degrees C (assumed minimum monthly average)
BOD_{Li} =	**2950**	= influent maximum month BOD_5 load, lbs/d (based on projected 2022 load)
PDF =	**8.10**	= peak day flow (peak day flow), MGD

$(V/Q)_{1C}$ = 1.20 days

Safety Factor = 2.08 we have a sufficiently sized lagoon at max month flow and BOD loading.

Minimum S.F. 1.5

Verify OK!

DESIGN OF DUAL-POWER MULTI-CELLULAR AERATED LAGOONS - Partial-Mixed Cell Sizing (2022)

This design method is based on the following articles:
Rich, L. (1982). "Design Approach to Dual-Power Aerated Lagoons." *J. Environ. Eng.* , 108(3), 532–548.
Rich, L. (1996). "Modification of Design Approach to Aerated Lagoons." *J. Environ. Eng.* , 122(2), 149–153.

Sizing for Partial-Mix Cells based on Overflow Rate for Settling

1982 article indicates an overflow rate not exceeding 75 gpd/sf to remove most, if not all, settleable solids.
Base overflow on peak day flow to ensure adequate removal of solids under all conditions.

PDF =	**8.10**	= peak day flow, MGD
PDF =	8,100,000	= peak day flow, gpd
Max Overflow =	75	gpd/sf
Min Surface Area =	108,000	sf

Therefore, a surface area of about 132,000 sf should be provided to ensure sufficient settling at peak flows.
This area is based on the area at mid-depth. Therefore, the equivalent bottom area with 3:1 side slopes and total depth of 10 feet would be:

Mid-Depth L =	140	ft
Mid-Depth W =	335	ft
Mid-Depth Area =	140,700	sf (total mid-depth area for all 3 cells)
Total Depth =	**10**	ft
Side Slope =	**3**	:1
Bottom L =	110	ft
Bottom W =	305	ft
Bottom Area =	33,550	sf

The equivalent volume of this mid-depth area is:

Volume =	8,078,400	gal

This is approximately equivalent to the volume of the 3 lagoons (3.5 x 3= 10.5 MG).

Sizing for Partial-Mix Cells based on Sludge Loading

1982 article indicates that even with loading of biodegradable solids as high as 80 g/m^2/d, complete destruction of the solids can be achieved within an annual cycle.
The loading is estimated using Eqn. 6 from the 1986 article as follows:

$$L = \frac{YS_0Q}{nA_j\left[1 + k_d\left(\frac{V}{Q}\right)_1\right]}$$

(Eqn. 6 from 1982 article)

Where:

L =	80	=	biodegradable solids loading on partial-mix cells, $g/m^2/d$
Y =	0.5	=	growth yield (for BOD) (from Table 1, 1982 article)
S_0 =	185	=	total influent BOD_5, mg/L (average annual load at average annual flow)
Q =	**5299**	=	average annual flow rate, m^3/d (average annual flow)
n =	**3**	=	number of partial-mix cells
A_j =	?	=	bottom area of each partial-mix cell, m^2
k_d =	0.157	=	$k_{d20} \times (1.05)^{T-20}$ = decay rate, 1/d (Eqn. 5 from 1982 article)
k_{d20} =	0.2	=	decay rate at 20 degrees C, 1/d (from Table 1, 1982 article)
$(V/Q)_1$ =	2.5	=	hydraulic retention time in complete-mix cell, days
BOD_{Li} =	**2160**	=	average annual BOD_5 load, lbs/d
T =	**15**	=	average annual temperature, degrees C

A_j = 15,797 sf

Bottom area for sludge loading is less than is required for settling, so settling controls.

Sizing for Partial-Mix Cells based on Sludge Accumulation

1996 article indicates that measured sludge accumulation in operating facilities has been significantly less (up to 3 times) than estimated by information in the 1982 article.
The 1996 article cites sludge accumulation rates of 44 m^3/yr per L/s and 38 m^3/yr per L/s as measured over a 7-year period for two dual-power multi-cellular aerated lagoon processes in South Carolina.
Therefore, an accumulation rate of 40 m^3/yr per L/s will be used to estimate sludge accumulation. Given the high I/I and low influent BOD concentrations, this accumulation rate is likely still somewhat conservative. Over time, VSS will be reduced through digestion. It is conservatively assumed that 75% of the sludge solids is VSS and approx. 40% of VSS is destroyed, which is the minimum for Class B biosolids.

Average Flow =	**1.40**	= Average Annual Flow, MGD
Average Flow =	61.33	= Average Annual Flow, L/s
Avg. Accumulation =	**40**	= m^3/yr per L/s
Accum. Rate =	2453	= m^3/yr
Assumed % VSS =	75%	
Accum. VSS =	1840	= m^3/yr
Min. VSS Destroyed =	736	= m^3/yr
Net Accum. Rate =	1717	= m^3/yr
5-yr Accum. =	8,587	= m^3
5-yr Accum. =	2.3	= MG

So, size of the partial-mix cells should have about 3 MG of volume dedicated to sludge storage.

If approximately 3 feet of the 10 feet of total depth is allocated for sludge storage in the three partial-mix cells, the sludge storage is then as follows:

Total Depth =	10	ft
Sludge Depth =	3	ft
Partial-Mix Vol =	3.50	MG, each
Number Cells =	3	# partial-mix cells
Total Vol =	10.50	MG
Sludge Vol =	3.2	MG
Accum Duration =	6.9	yrs

3-foot sludge depth allows sufficient storage volume for approximately 7+ years conservatively and also maintains sufficient separation between the water surface and sludge layer to avoid effluent quality impacts from resuspension. Actual storage duration could be much longer if VSS destruction is greater that assumed minimum of 40% and/or % VSS is higher.

Sizing for Partial-Mix Cells based on Control of Algal Growth

The 1996 article uses the following equation to estimate algal growth rate in the partial-mix cells:

(Eqn. 5 from 1996 article)

$$(V/Q)_j = [1 - (X_{An}/X_{A0})^{-1/n}]/\mu_A$$

Where: hydraulic retention time in each partial-mix cell, days

$(V/Q)_j$ = algae in effluent of last cell, mg/L

X_{An} = algae in influent of first cell, mg/L

X_{A0} = number of cells in series

n = first-order growth rate for algae, 1/d

μ_A =

The graph below is a plot of total retention time in all partial-mix cells versus the ratio X_{An}/X_{A0}.

As stated in the 1996 article, a previous study indicated μ_A always less than 0.48 1/d.

Therefore, that value was used for μ_A.

The 1996 article states that for XAn/XA0 less than 25, significant algal growth should not result.

For 3 partial-mix cells in series, total retention time should not exceed about 4.4 days

At the current minimum monthly flow of about 600,000 gpd, that is equal to a total volume of only about 2.6 MG. Considering the wide range of flows from PDF to the minimum monthly flow to I/I and increases in flow due to growth, there is no way to both reasonably minimize retention time for algal growth, and have sufficient volume to accommodate overflow rates at peak flow and provide sufficient volume for sludge accumulation and sufficient area for sludge loading. As a result, the City has taken other measures to control algae including use of ultrasonic transducers.

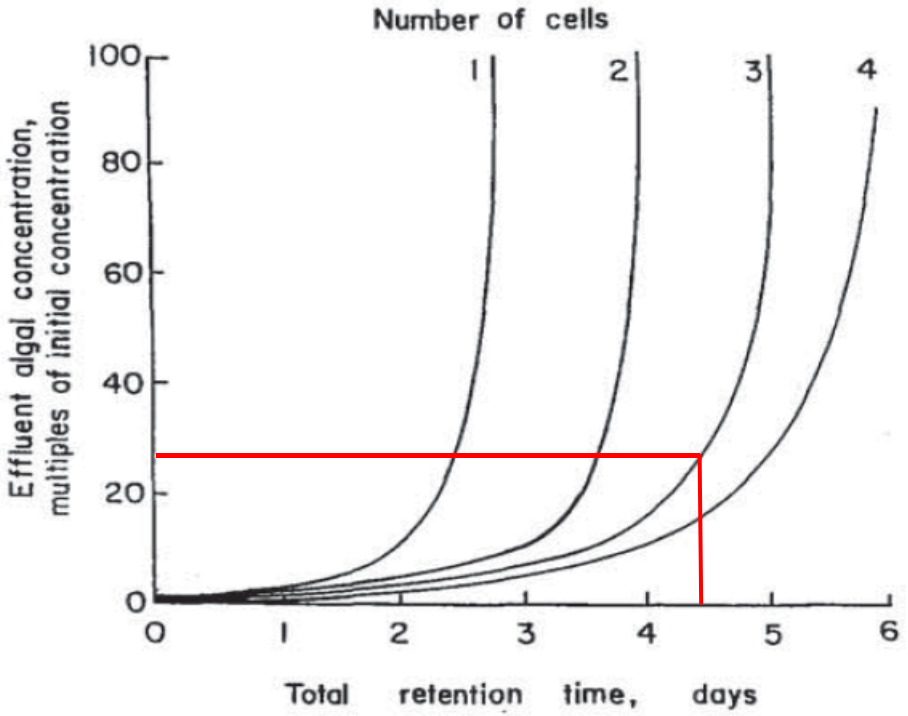


FIG. 4. Influence of Retention Time and Multicellular Configuration on Algal Growth [from Eq. (5)]

DESIGN OF DUAL-POWER MULTI-CELLULAR AERATED LAGOONS - Aeration and Mixing Sizing (2022)

This design method is based on the following articles:

Rich, L. (1982). "Design Approach to Dual-Power Aerated Lagoons." *J. Environ. Eng.* , 108(3), 532–548.
Rich, L. (1996). "Modification of Design Approach to Aerated Lagoons." *J. Environ. Eng.* , 122(2), 149–153.

For both the complete-mix and partial mix cells, the sizing of the aerators is based on the larger of the two power requirements for aeration and mixing.

Mixing Sizing for Complete-Mix Cells

For complete-mix cells, it has been found that a power level of 6 W/m^3 (30 hp/MG) is necessary to maintain solids in suspension and also maintain sufficientmixing intensity and turbidity to suppress algae growth (from 1982 article)
Therefore, the mixing energy required for the complete-mix cell ideally is:

Volume =	**10.0**	MG
Intensity =	30	hp/MG
Energy =	300	hp

Aeration Sizing for Complete-Mix Cells

Assuming substantial nitrification does not occur in the complete-mix cell (which would be the case during the winter months with higher flows and lower temperatures), the maximum oxygen consumption rate can be estimated by:

$$R_{O_2} = 4.16 \times 10^{-5} rQ(S_0 - S_1)(1.47 - 1.42\,FY)$$

(Eqn. 10 from 1982 article)

Where:

Ro_2 =	?	= oxygen consumption rate, kg/hr
r =	1.5	= diurnal peak ratio to average oxygen demand (assume 1.5 based on typical diurnal pattern)
Q =	**10,598**	= flow rate, m^3/d (max month flow)
S_0 =	126	= total influent BOD_5, mg/L (based on max month load at max month flow for 2022)
S_1 =	0	= complete-mix cell effluent soluble BOD_5, mg/L (conservatively assumed to be zero)

F =	0.67	= solids decay factor (typical value of 0.67 assumed per 1982 article)
Y =	0.5	= growth yield (for BOD) (from Table 1, 1982 article)
BOD_{Li} =	**2950**	= influent maximum month BOD_5 load, lbs/d
Ro_2 =	83.1	kg/hr
Ro_2 =	183.2	lbs/hr

The above oxygen demand represents the demand for the winter months. Compare with the demand for summer months with lower flow, same BOD load and nitrification.
Nitrification demand will be based on assuming complete nitrification and all influent organic-N hydrolyzing to ammonia:

Q =	**5,299**	= flow rate, m^3/d (based on 2022 average annual flow)
S_0 =	253	= total influent BOD_5, mg/L (max month load at peak day flow)
TKN_{Li} =	**390**	= total influent TKN load, lbs/d (max month)
TKN_{Li} =	7.37	= total influent TKN load, kg/h (max month)
Nit-O_2 =	4.6	= oxygen demand for nitrification, kg O_2/kg N
Nitrification =	100%	= % of ammonia nitrified in complete-mix cell

Ro_2 =	83.1	kg/hr	(without nitrification)
Ro_2 =	183.2	lbs/hr	Summer oxygen demand with no nitrification in complete-mix cell
Ro_2 =	116.98	kg/hr	(with nitrification)
Ro_2 =	257.9	lbs/hr	Summer oxygen demand with partial nitrification in complete-mix cell

Estimate the equivalent horsepower requirement to meet this oxygen demand using the following equation:

$$N = N_o\left(\frac{\beta C_{W_{alt}} - C_L}{C_{s_{20}}}\right) 1.024^{T-20}\alpha$$

(Eqn. 10-19 from M&E 3rd Ed.)

Where:

N =	?		= oxygen transfer rate under field conditions, lbs/hp/hr
N_0 =		**1.75**	= oxygen transfer rate under standard conditions, lbs/hp/hr (2.0 is typical design number from Table 10-9 M&E 3rd Ed., but experience has shown can be lower, use 1.5 for existing aerators)
β =		0.95	= salinity-surface tension correction factor (assume 0.9)
C_{Walt} =		**7.81**	= oxygen saturation at field temperature and elevation, mg/L (from M&E 3rd Ed. Appendix E , assuming max 25 degrees C temp)
C_L =		**2.00**	= minimum operating oxygen concentration under worst case conditions, mg/L
Cs_{20} =		**9.04**	= oxygen saturation at standard conditions, mg/L (from M&E 3rd Ed. Appendix E)
T =		**27.9**	= temperature, degrees C
α =		0.85	= oxygen transfer correction factor (assume 0.85)

N =	1.08	lbs/hp/hr	
Ro_2 with Nit =	257.9	lbs/hr	
Motor Eff. =	92%		
Req'd Energy =	261	hp	So, sufficient aeration energy to support full BOD removal and partial nitrification

Mixing Sizing for Partial-Mix Cells

For partial-mix cells, it has been found that a power level of 1 W/m^3 (5 hp/MG) is necessary to disperse dissolved oxygen throughout the lagoon and that a power level of up to 2 W/m^3 can still allow for sufficient settling of solids (from 1982 article)
Therefore, the mixing energy for the partial-mix cell should be:

Volume =	**10.5**	MG
Min Intensity =	5	hp/MG
Max Intensity =	10	hp/MG
Min Energy =	52.5	hp
Max Energy =	105.0	hp

Aeration Sizing for Partial-Mix Cells

As stated in the 1982 article, aeration demand in the partial-mix cells is dominated by residual BOD in the winter (when temperatures are low and benthal demand [i.e., oxygen consumption due to biological activity in the sludge layer] is low)
and by the benthal demand in the summer. In all but unusual conditions, the summer benthal demand is significantly larger. Therefore, the summer benthal demand will be used to determin aeration requirements for the partial-mix cells.
The benthal demand can be estimated using the following equation:

$$R_{O_2} = 4.16 \times 10^{-5} A_j B \theta^{T-20}$$

(Eqn. 12 from 1982 article)

Where:

Ro_2 =	?		= oxygen consumption rate, kg/hr
A_j =		3118	= bottom area of each partial-mix cell, m^2
B =		**60**	= rate of benthal oxygen demand at 20 deg. C, g $O_2/m^2/d$ (from 1982 article)
θ =		**1.05**	= temperature sensitivity coefficient (from 1982 article)
T =		**27.9**	= temperature, degrees C (assumed max month temperature)
Ro_2 =		11.44	kg/hr
Ro_2 =		25.23	lbs/hr
Req'd Energy =		25.5	hp

Additional air is supplied by the SFF media blowers to assist with benthal demand and nitrification.

DESIGN OF DUAL-POWER MULTI-CELLULAR AERATED LAGOONS - Complete-Mixed Cell Sizing (2036)

RED INDICATED VALUES REQUIRING INPUT

This design method is based on the following articles:

Rich, L. (1982). "Design Approach to Dual-Power Aerated Lagoons." *J. Environ. Eng.* , 108(3), 532–548.

Rich, L. (1996). "Modification of Design Approach to Aerated Lagoons." *J. Environ. Eng.* , 122(2), 149–153.

Sizing for Complete-Mix Cell based on COD Removal

COD removal is based on the premise that hydrolysis of biodegradable particulate matter is the rate limiting step (i.e., slower rate than the rate at which hydrolized or soluble waste is consumed), as long as oxygen supply is not limiting.

The steady-state mass balance for the complete-mix cell is:

$$X_S/X_{B,H} = K_X(1 + b_h(V/Q))/[(V/Q)(k_H - b_h) - 1]$$

(Eqn. 3 from 1996 article)

and

$$X_{B,H} = Y_H(X_{S0} + S_{S0})/[1 + (1 - 0.92Y_H)b_h(V/Q)]$$

(Eqn. 4 from 1996 article)

(Eqn. 4 is simplified version ignoring residual effluent concentration to solve for $X_{B,H}$, then plug that in to Eqn. 3 to solve for X_s)

Where:

$X_{B,H}$ =	?	= active heterotrophic biomass COD, mg/L
X_S =	?	= effluent biodegradable particulate COD, mg/L
K_x =	0.006	= 0.03 x (1.116)$^{T-20}$ = half-saturation constant, (mg biodegradable particulate COD)/(mg biomass COD)
b_h =	0.113	= 0.62 x (1.120)$^{T-20}$ = decay coefficient, 1/day
V =	?	= volume, m^3
Q =	**12,188**	= flow rate, m^3/d (based on 2036 max month flow)
k_H =	0.69	= 3 x (1.103)$^{T-20}$ = maximum specific hydrolysis rate, 1/day
Y_H =	0.67	= growth yield, (mg biomass COD)/(mg COD oxidized)
X_{SO} =	207	= influent biodegradable particulate COD, mg/L (typcially 64% of total COD [from BioWin influent specifier], rest is soluble or unbiodegradable)
S_{SO} =	52	= influent biodegradable soluble COD, mg/L (typically 16% of total COD [from BioWin influent specifier], rest is particulate or unbiodegradable)
T =	**5**	= temperature, degrees C (assumed minimum monthly average)
COD:BOD =	**2.1**	= ratio of influent COD to influent BOD (typically ranges from 1.9 to 2.2 for domestic wastewater [from BioWin influent specifier])
BOD_{Li} =	**4130**	= influent maximum month BOD_5 load, lbs/d
BOD_{Ci} =	154	= influent maximum month BOD_5 concentration, mg/L (max month load at max month flow)

Values for K_x, b_H, k_H and Y_H are taken from the *International Associaton on Water Pollution Research and Control* "Activated Sludge Model No. 1", 1986 (as summarized in Table 1 of 1996 article)

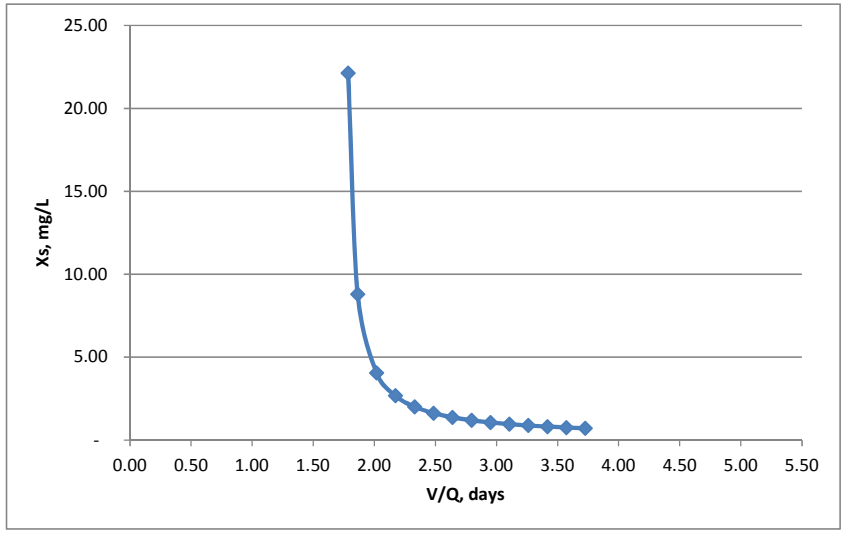
First, solve for $X_{B,H}$ based on multiple volumes using Eqn. 4:

V	V	$X_{B,H}$
(MG)	(m^3)	(mg/L)
5.00	18,927	91.52
5.50	20,820	92.05
6.00	22,712	92.58
6.50	24,605	93.12
7.00	26,498	93.67
7.50	28,391	94.22
8.00	30,283	94.78
8.50	32,176	95.35
9.00	34,069	95.92
9.50	35,961	96.50
10.00	37,854	97.09
10.50	39,747	97.68
11.00	41,640	98.28
11.50	43,532	98.89
12.00	45,425	99.51

Second, solve for X_S using the same volumes and values determined for $X_{B,H}$ using Eqn. 3:

NOTE: These values are based on max month flows and loads and minimum monthly average temperature

V	V	$X_S/X_{B,H}$	$X_{B,H}$	V/Q	X_S
(MG)	(m^3)		(mg/L)	(days)	(mg/L)
5.75	21,766	0.240288	92.05	1.79	22.12
6.00	22,712	0.095067	92.58	1.86	8.80
6.50	24,605	0.043553	93.12	2.02	4.06
7.00	26,498	0.02853	93.67	2.17	2.67
7.50	28,391	0.021366	94.22	2.33	2.01
8.00	30,283	0.017172	94.78	2.48	1.63
8.50	32,176	0.014418	95.35	2.64	1.37
9.00	34,069	0.012472	95.92	2.80	1.20
9.50	35,961	0.011023	96.50	2.95	1.06
10.00	37,854	0.009903	97.09	3.11	0.96
10.50	39,747	0.00901	97.68	3.26	0.88
11.00	41,640	0.008283	98.28	3.42	0.81
11.50	43,532	0.007678	98.89	3.57	0.76
12.00	45,425	0.007168	99.51	3.73	0.71



Since this cell is complete-mix, there is minimal sludge accumulation, so there does not need to be signficant volume reserved for sludge accumulation, as is needed in the partial-mix cells.

Complete-Mix Cell Retention Time = **2.5** days

Complete-Mix Cell Volume = 8.05 MG

Critical Sizing for Complete-Mix Cell to Avoid Washout

The 1982 article provided the equation below to estimate the critical retention time to prevent washout of solids, since there is no solids recycle.

$$\left(\frac{V}{Q}\right)_{1C} = \frac{K_S + S_0}{\hat{\mu} S_0}$$

(Eqn. 3 from 1982 article)

Where:

K_S =	120	= saturation constant, mg/L (from Table 1, 1982 article)
S_0 =	52.6	= total influent BOD_5, mg/L (max month load at peak day flow)
μ =	3.1	= μ_{20} x $(1.10)^{T-20}$ = maximum specific growth rate at temp T, 1/d (from Eqn. 4, 1982 article)
$(V/Q)_{1C}$ =	?	= critical washout retention time, days
μ_{20} =	13.0	= maximum specific growth rate at 20 degrees C, 1/d (from Table 1, 1982 article)
T =	5	= temperature, degrees C (assumed minimum monthly average)
BOD_{Li} =	4130	= influent maximum month BOD_5 load, lbs/d (based on projected 2036 load)
PDF =	9.42	= peak day flow (peak day flow), MGD

$(V/Q)_{1C}$ = 1.05 days

Safety Factor = 2.37 we have a sufficiently sized lagoon at max month flow and BOD loading.

Minimum Safety 1.5

Verify OK!

DESIGN OF DUAL-POWER MULTI-CELLULAR AERATED LAGOONS - Partial-Mixed Cell Sizing (2036)

This design method is based on the following articles:
Rich, L. (1982). "Design Approach to Dual-Power Aerated Lagoons." *J. Environ. Eng.* , 108(3), 532–548.
Rich, L. (1996). "Modification of Design Approach to Aerated Lagoons." *J. Environ. Eng.* , 122(2), 149–153.

Sizing for Partial-Mix Cells based on Overflow Rate for Settling

1982 article indicates an overflow rate not exceeding 75 gpd/sf to remove most, if not all, settleable solids.
Base overflow on peak day flow to ensure adequate removal of solids under all conditions.

PDF =	**9.42**	= peak day flow, MGD
PDF =	9,420,000	= peak day flow, gpd
Max Overflow =	75	gpd/sf
Min Surface Area =	125,600	sf

Therefore, a surface area of about 132,000 sf should be provided to ensure sufficient settling at peak flows.
This area is based on the area at mid-depth. Therefore, the equivalent bottom area with 3:1 side slopes and total depth of 10 feet would be:

Mid-Depth L =	140	ft
Mid-Depth W =	335	ft
Mid-Depth Area =	140,700	sf (total mid-depth area for all 3 cells)
Total Depth =	**10**	ft
Side Slope =	**3**	:1
Bottom L =	110	ft
Bottom W =	305	ft
Bottom Area, each =	33,550	sf

The equivalent volume of this mid-depth area is:

Volume =	9,394,880	gal

This is approximately equivalent to the volume of the 3 lagoons (3.5 x 3= 10.5 MG).

Sizing for Partial-Mix Cells based on Sludge Loading

1982 article indicates that even with loading of biodegradable solids as high as 80 g/m^2/d, complete destruction of the solids can be achieved within an annual cycle.
The loading is estimated using Eqn. 6 from the 1986 article as follows:

$$L = \frac{YS_0Q}{nA_j\left[1 + k_d\left(\frac{V}{Q}\right)_1\right]}$$

(Eqn. 6 from 1982 article)

Where:

L =	80	= biodegradable solids loading on partial-mix cells, $g/m^2/d$
Y =	0.5	= growth yield (for BOD) (from Table 1, 1982 article)
S_0 =	186	= total influent BOD_5, mg/L (average annual load at average annual flow)
Q =	**7380**	= average annual flow rate, m^3/d (average annual flow)
n =	**3**	= number of partial-mix cells
A_j =	?	= bottom area of each partial-mix cell, m^2
k_d =	0.157	= $k_{d20} \times (1.05)^{T-20}$ = decay rate, 1/d (Eqn. 5 from 1982 article)
k_{d20} =	0.2	= decay rate at 20 degrees C, 1/d (from Table 1, 1982 article)
$(V/Q)_1$ =	2.5	= hydraulic retention time in complete-mix cell, days
BOD_{Li} =	**3030**	= average annual BOD_5 load, lbs/d
T =	**15**	= average annual temperature, degrees C

A_j = 22,159 sf

Bottom area for sludge loading is less than is required for settling, so settling controls.

Sizing for Partial-Mix Cells based on Sludge Accumulation

1996 article indicates that measured sludge accumulation in operating facilities has been significantly less (up to 3 times) than estimated by information in the 1982 article.
The 1996 article cites sludge accumulation rates of 44 m^3/yr per L/s and 38 m^3/yr per L/s as measured over a 7-year period for two dual-power multi-cellular aerated lagoon processes in South Carolina.
Therefore, an accumulation rate of 40 m^3/yr per L/s will be used to estimate sludge accumulation. Given the high I/I and low influent BOD concentrations, this accumulation rate is likely still somewhat conservative. Over time, VSS will be reduced through digestion. It is conservatively assumed that 75% of the sludge solids is VSS and approx. 40% of VSS is destroyed, which is the minimum for Class B biosolids.

Average Flow =	**1.95**	= Average Annual Flow, MGD
Average Flow =	85.43	= Average Annual Flow, L/s
Avg. Accumulation =	**40**	= m^3/yr per L/s
Accum. Rate =	3417	= m^3/yr
Assumed % VSS =	75%	
Accum. VSS =	2563	= m^3/yr
Min. VSS Destroyed =	1025	= m^3/yr
Net Accum. Rate =	2392	= m^3/yr
5-yr Accum. =	11,960	= m^3
5-yr Accum. =	3.2	= MG

So, size of the partial-mix cells should have about 3.3 MG of volume dedicated to sludge storage.

If approximately 3 feet of the 10 feet of total depth is allocated for sludge storage in the three partial-mix cells, the sludge storage is then as follows:

Total Depth =	10	ft
Sludge Depth =	3	ft
Partial-Mix Vol =	3.50	MG, each
Number Cells =	3	# partial-mix cells
Total Vol =	10.50	MG
Sludge Vol =	3.2	MG
Accum Duration =	5.0	yrs

3-foot sludge depth allows sufficient storage volume for approximately 5+ years conservatively and also maintains sufficient separation between the water surface and sludge layer to avoid effluent quality impacts from resuspension. Actual storage duration could be much longer if VSS destruction is greater that assumed minimum of 40% and/or % VSS is higher.

Sizing for Partial-Mix Cells based on Control of Algal Growth

The 1996 article uses the following equation to estimate algal growth rate in the partial-mix cells:

(Eqn. 5 from 1996 article)

$$(V/Q)_j = [1 - (X_{An}/X_{A0})^{-1/n}]/\mu_A$$

Where:

hydraulic retention time in each partial-mix cell, days

$(V/Q)_j$ = algae in effluent of last cell, mg/L

X_{An} = algae in influent of first cell, mg/L

X_{A0} = number of cells in series

n = first-order growth rate for algae, 1/d

μ_A =

The graph below is a plot of total retention time in all partial-mix cells versus the ratio X_{An}/X_{A0}.

As stated in the 1996 article, a previous study indicated μ_A always less than 0.48 1/d.

Therefore, that value was used for μ_A.

The 1996 article states that for XAn/XA0 less than 25, significant algal growth should not result.
For 3 partial-mix cells in series, total retention time should not exceed about 4.4 days

At the current minimum monthly flow of about 600,000 gpd, that is equal to a total volume of only about 2.6 MG.
Considering the wide range of flows from PDF to the minimum monthly flow to I/I and increases in flow due to growth, there is no way to both reasonably minimize retention time for algal growth, and have sufficient volume to accommodate overflow rates at peak flow and provide sufficient volume for sludge accumulation and sufficient area for sludge loading. As a result, the City has taken other measures to control algae including use of ultrasonic transducers.

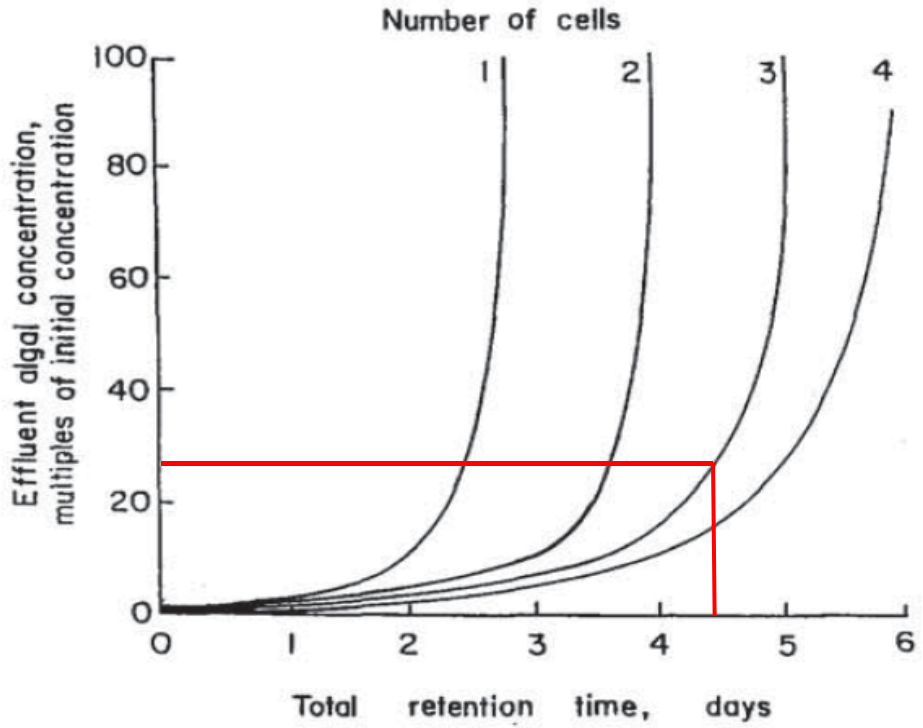


FIG. 4. Influence of Retention Time and Multicellular Configuration on Algal Growth [from Eq. (5)]

DESIGN OF DUAL-POWER MULTI-CELLULAR AERATED LAGOONS - Aeration and Mixing Sizing (2036)

This design method is based on the following articles:

Rich, L. (1982). "Design Approach to Dual-Power Aerated Lagoons." *J. Environ. Eng.* , 108(3), 532–548.
Rich, L. (1996). "Modification of Design Approach to Aerated Lagoons." *J. Environ. Eng.* , 122(2), 149–153.

For both the complete-mix and partial mix cells, the sizing of the aerators is based on the larger of the two power requirements for aeration and mixing.

Mixing Sizing for Complete-Mix Cells

For complete-mix cells, it has been found that a power level of 6 W/m^3 (30 hp/MG) is necessary to maintain solids in suspension and also maintain sufficientmixing intensity and turbidity to suppress algae growth (from 1982 article)
Therefore, the mixing energy required for the complete-mix cell ideally is:

Volume =	**10.0**	MG
Intensity =	30	hp/MG
Energy =	300	hp

Aeration Sizing for Complete-Mix Cells

Assuming substantial nitrification does not occur in the complete-mix cell (which would be the case during the winter months with higher flows and lower temperatures), the maximum oxygen consumption rate can be estimated by:

$$R_{O_2} = 4.16 \times 10^{-5} rQ(S_0 - S_1)(1.47 - 1.42\,FY)$$

(Eqn. 10 from 1982 article)

Where:

Ro_2 =	?	= oxygen consumption rate, kg/hr
r =	1.5	= diurnal peak ratio to average oxygen demand (assume 1.5 based on typical diurnal pattern)
Q =	**12,188**	= flow rate, m^3/d (max month flow)
S_0 =	154	= total influent BOD_5, mg/L (based on max month load at max month flow for 2036)
S_1 =	0	= complete-mix cell effluent soluble BOD_5, mg/L (conservatively assumed to be zero)

F =	0.67	= solids decay factor (typical value of 0.67 assumed per 1982 article)
Y =	0.5	= growth yield (for BOD) (from Table 1, 1982 article)
BOD_{Li} =	**4130**	= influent maximum month BOD_5 load, lbs/d
Ro_2 =	116.3	kg/hr
Ro_2 =	256.5	lbs/hr

The above oxygen demand represents the demand for the winter months. Compare with the demand for summer months with lower flow, same BOD load and nitrification.
Nitrification demand will be based on assuming complete nitrification and all influent organic-N hydrolyzing to ammonia:

Q =	**7,380**	= flow rate, m^3/d (based on 2036 average annual flow)
S_0 =	254	= total influent BOD_5, mg/L (max month load at peak day flow)
TKN_{Li} =	**550**	= total influent TKN load, lbs/d (max month)
TKN_{Li} =	10.39	= total influent TKN load, kg/h (max month)
Nit-O_2 =	4.6	= oxygen demand for nitrification, kg O_2/kg N
Nitrification =	10%	= % of ammonia nitrified in complete-mix cell

Ro_2 =	116.3	kg/hr	(without nitrification)
Ro_2 =	256.5	lbs/hr	Summer oxygen demand with no nitrification in complete-mix cell
Ro_2 =	121.09	kg/hr	(with nitrification)
Ro_2 =	267.0	lbs/hr	Summer oxygen demand with partial nitrification in complete-mix cell

Estimate the equivalent horsepower requirement to meet this oxygen demand using the following equation:

$$N = N_o\left(\frac{\beta C_{W_{alt}} - C_L}{C_{s_{20}}}\right) 1.024^{T-20}\alpha$$

(Eqn. 10-19 from M&E 3rd Ed.)

Where:

N =	?		= oxygen transfer rate under field conditions, lbs/hp/hr
N_0 =		**1.75**	= oxygen transfer rate under standard conditions, lbs/hp/hr (2.0 is typical design number from Table 10-9 M&E 3rd Ed., but experience has shown can be lower, use 1.5 for existing aerators)
β =		0.95	= salinity-surface tension correction factor (assume 0.9)
C_{Walt} =		**7.81**	= oxygen saturation at field temperature and elevation, mg/L (from M&E 3rd Ed. Appendix E , assuming max 25 degrees C temp)
C_L =		**2.00**	= minimum operating oxygen concentration under worst case conditions, mg/L
Cs_{20} =		**9.04**	= oxygen saturation at standard conditions, mg/L (from M&E 3rd Ed. Appendix E)
T =		**27.9**	= temperature, degrees C
α =		0.85	= oxygen transfer correction factor (assume 0.85)

N =	1.08	lbs/hp/hr
Ro_2 with Nit =	267.0	lbs/hr
Motor Eff. =	92%	
Req'd Energy =	270	hp

Mixing Sizing for Partial-Mix Cells

For partial-mix cells, it has been found that a power level of 1 W/m^3 (5 hp/MG) is necessary to disperse dissolved oxygen throughout the lagoon and that a power level of up to 2 W/m^3 can still allow for sufficient settling of solids (from 1982 article)
Therefore, the mixing energy for the partial-mix cell should be:

Volume =	**10.5**	MG
Min Intensity =	5	hp/MG
Max Intensity =	10	hp/MG
Min Energy =	52.5	hp
Max Energy =	105.0	hp

Aeration Sizing for Partial-Mix Cells

As stated in the 1982 article, aeration demand in the partial-mix cells is dominated by residual BOD in the winter (when temperatures are low and benthal demand [i.e., oxygen consumption due to biological activity in the sludge layer] is low)
and by the benthal demand in the summer. In all but unusual conditions, the summer benthal demand is significantly larger. Therefore, the summer benthal demand will be used to determin aeration requirements for the partial-mix cells.
The benthal demand can be estimated using the following equation:

$$R_{O_2} = 4.16 \times 10^{-5} A_j B \theta^{T-20}$$

(Eqn. 12 from 1982 article)

Where:

Ro_2 =	?		= oxygen consumption rate, kg/hr
A_j =		3118	= bottom area of each partial-mix cell, m^2
B =		**60**	= rate of benthal oxygen demand at 20 deg. C, g $O_2/m^2/d$ (from 1982 article)
θ =		**1.05**	= temperature sensitivity coefficient (from 1982 article)
T =		**27.9**	= temperature, degrees C (assumed max month temperature)
Ro_2 =		11.44	kg/hr
Ro_2 =		25.23	lbs/hr
Req'd Energy =		25.5	hp

Additional air is supplied by the SFF media blowers to assist with benthal demand and nitrification.

DESIGN OF DUAL-POWER MULTI-CELLULAR AERATED LAGOONS - Complete-Mixed Cell Sizing (Re-Rated at 3.30 MGD)

RED INDICATED VALUES REQUIRING INPUT

This design method is based on the following articles:

Rich, L. (1982). "Design Approach to Dual-Power Aerated Lagoons." *J. Environ. Eng.* , 108(3), 532–548.

Rich, L. (1996). "Modification of Design Approach to Aerated Lagoons." *J. Environ. Eng.* , 122(2), 149–153.

Sizing for Complete-Mix Cell based on COD Removal

COD removal is based on the premise that hydrolysis of biodegradable particulate matter is the rate limiting step (i.e., slower rate than the rate at which hydrolized or soluble waste is consumed), as long as oxygen supply is not limiting.

The steady-state mass balance for the complete-mix cell is:

$$X_S/X_{B,H} = K_X(1 + b_h(V/Q))/[(V/Q)(k_H - b_h) - 1]$$

(Eqn. 3 from 1996 article)

and

$$X_{B,H} = Y_H(X_{S0} + S_{S0})/[1 + (1 - 0.92Y_H)b_h(V/Q)]$$

(Eqn. 4 from 1996 article)

(Eqn. 4 is simplified version ignoring residual effluent concentration to solve for $X_{B,H}$, then plug that in to Eqn. 3 to solve for X_s)

Where:

$X_{B,H}$ =	?	= active heterotrophic biomass COD, mg/L
X_S =	?	= effluent biodegradable particulate COD, mg/L
K_x =	0.006	= 0.03 x (1.116)$^{T-20}$ = half-saturation constant, (mg biodegradable particulate COD)/(mg biomass COD)
b_h =	0.113	= 0.62 x (1.120)$^{T-20}$ = decay coefficient, 1/day
V =	?	= volume, m^3
Q =	**12,491**	= flow rate, m^3/d (based on 3.3 MGD max month flow)
k_H =	0.69	= 3 x (1.103)$^{T-20}$ = maximum specific hydrolysis rate, 1/day
Y_H =	0.67	= growth yield, (mg biomass COD)/(mg COD oxidized)
X_{SO} =	207	= influent biodegradable particulate COD, mg/L (typcially 64% of total COD [from BioWin influent specifier], rest is soluble or unbiodegradable)
S_{SO} =	52	= influent biodegradable soluble COD, mg/L (typically 16% of total COD [from BioWin influent specifier], rest is particulate or unbiodegradable)
T =	**5**	= temperature, degrees C (assumed minimum monthly average)
COD:BOD =	**2.1**	= ratio of influent COD to influent BOD (typically ranges from 1.9 to 2.2 for domestic wastewater [from BioWin influent specifier])
BOD_{Li} =	**4233**	= influent maximum month BOD_5 load, lbs/d
BOD_{Ci} =	154	= influent maximum month BOD_5 concentration, mg/L (max month load at max month flow)

Values for K_x, b_H, k_H and Y_H are taken from the *International Associaton on Water Pollution Research and Control* "Activated Sludge Model No. 1", 1986 (as summarized in Table 1 of 1996 article)

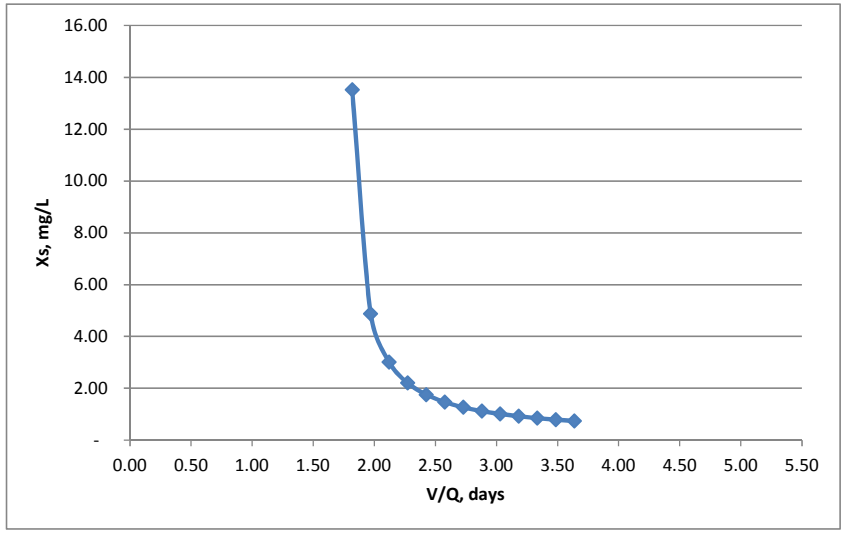
First, solve for $X_{B,H}$ based on multiple volumes using Eqn. 4:

V	V	$X_{B,H}$
(MG)	(m^3)	(mg/L)
5.00	18,927	91.41
5.50	20,820	91.92
6.00	22,712	92.44
6.50	24,605	92.96
7.00	26,498	93.49
7.50	28,391	94.03
8.00	30,283	94.58
8.50	32,176	95.13
9.00	34,069	95.68
9.50	35,961	96.24
10.00	37,854	96.81
10.50	39,747	97.39
11.00	41,640	97.97
11.50	43,532	98.56
12.00	45,425	99.16

Second, solve for X_S using the same volumes and values determined for $X_{B,H}$ using Eqn. 3:

NOTE: These values are based on max month flows and loads and minimum monthly average temperature

V	V	$X_S/X_{B,H}$	$X_{B,H}$	V/Q	X_S
(MG)	(m^3)		(mg/L)	(days)	(mg/L)
6.00	22,712	0.146304	92.44	1.82	13.52
6.50	24,605	0.052406	92.96	1.97	4.87
7.00	26,498	0.032269	93.49	2.12	3.02
7.50	28,391	0.023489	94.03	2.27	2.21
8.00	30,283	0.018572	94.58	2.42	1.76
8.50	32,176	0.015429	95.13	2.58	1.47
9.00	34,069	0.013245	95.68	2.73	1.27
9.50	35,961	0.011641	96.24	2.88	1.12
10.00	37,854	0.010412	96.81	3.03	1.01
10.50	39,747	0.00944	97.39	3.18	0.92
11.00	41,640	0.008653	97.97	3.33	0.85
11.50	43,532	0.008002	98.56	3.49	0.79
12.00	45,425	0.007455	99.16	3.64	0.74



Since this cell is complete-mix, there is minimal sludge accumulation, so there does not need to be signficant volume reserved for sludge accumulation, as is needed in the partial-mix cells.

Complete-Mix Cell Retention Time = **2.5** days

Complete-Mix Cell Volume = 8.25 MG

Critical Sizing for Complete-Mix Cell to Avoid Washout

The 1982 article provided the equation below to estimate the critical retention time to prevent washout of solids, since there is no solids recycle.

$$\left(\frac{V}{Q}\right)_{1C} = \frac{K_S + S_0}{\hat{\mu} S_0}$$

(Eqn. 3 from 1982 article)

Where:

K_S =	120	= saturation constant, mg/L (from Table 1, 1982 article)
S_0 =	52.6	= total influent BOD_5, mg/L (max month load at peak day flow)
μ =	3.1	= μ_{20} x $(1.10)^{T-20}$ = maximum specific growth rate at temp T, 1/d (from Eqn. 4, 1982 article)
$(V/Q)_{1C}$ =	?	= critical washout retention time, days
μ_{20} =	13.0	= maximum specific growth rate at 20 degrees C, 1/d (from Table 1, 1982 article)
T =	5	= temperature, degrees C (assumed minimum monthly average)
BOD_{Li} =	4233	= influent maximum month BOD_5 load, lbs/d (based on projected 2036 load)
PDF =	9.65	= peak day flow (peak day flow), MGD

$(V/Q)_{1C}$ = 1.05 days

Safety Factor = 2.37 we have a sufficiently sized lagoon at max month flow and BOD loading.

Min. S.F. = 1.5

Verify OK!

DESIGN OF DUAL-POWER MULTI-CELLULAR AERATED LAGOONS - Partial-Mixed Cell Sizing (Re-Rated at 3.30 MGD)

This design method is based on the following articles:
Rich, L. (1982). "Design Approach to Dual-Power Aerated Lagoons." *J. Environ. Eng.* , 108(3), 532–548.
Rich, L. (1996). "Modification of Design Approach to Aerated Lagoons." *J. Environ. Eng.* , 122(2), 149–153.

Sizing for Partial-Mix Cells based on Overflow Rate for Settling

1982 article indicates an overflow rate not exceeding 75 gpd/sf to remove most, if not all, settleable solids.
Base overflow on peak day flow to ensure adequate removal of solids under all conditions.

PDF =	9.65	= peak day flow, MGD
PDF =	9,654,037	= peak day flow, gpd
Max Overflow =	75	gpd/sf
Min Surface Area =	128,720	sf

Therefore, a surface area of about 125,000 sf should be provided to ensure sufficient settling at peak flows, which is less than the available surface area.
This area is based on the area at mid-depth. Therefore, the equivalent bottom area with 3:1 side slopes and total depth of 10 feet would be:

Mid-Depth L =	140	ft
Mid-Depth W =	335	ft
Mid-Depth Area =	140,700	sf (total mid-depth area for all 3 cells)
Total Depth =	10	ft
Side Slope =	3	:1
Bottom L =	110	ft
Bottom W =	305	ft
Bottom Area, each =	33,550	sf

The equivalent volume of the required mid-depth area is:

Volume =	9,628,293	gal

This is less than the available volume of the 3 lagoons (3.5 x 3= 10.5 MG).

Sizing for Partial-Mix Cells based on Sludge Loading

1982 article indicates that even with loading of biodegradable solids as high as 80 g/m^2/d, complete destruction of the solids can be achieved within an annual cycle.
The loading is estimated using Eqn. 6 from the 1986 article as follows:

$$L = \frac{YS_0Q}{nA_j\left[1 + k_d\left(\frac{V}{Q}\right)_1\right]}$$

(Eqn. 6 from 1982 article)

Where:

L =	80	= biodegradable solids loading on partial-mix cells, $g/m^2/d$
Y =	0.5	= growth yield (for BOD) (from Table 1, 1982 article)
S_0 =	186	= total influent BOD_5, mg/L (average annual load at average annual flow)
Q =	**7571**	= average annual flow rate, m^3/d (average annual flow)
n =	**3**	= number of partial-mix cells
A_j =	?	= bottom area of each partial-mix cell, m^2
k_d =	0.157	= $k_{d20} \times (1.05)^{T-20}$ = decay rate, 1/d (Eqn. 5 from 1982 article)
k_{d20} =	0.2	= decay rate at 20 degrees C, 1/d (from Table 1, 1982 article)
$(V/Q)_1$ =	2.5	= hydraulic retention time in complete-mix cell, days
BOD_{Li} =	**3105**	= average annual BOD_5 load, lbs/d
T =	**15**	= average annual temperature, degrees C

A_j = 22,708 sf

Bottom area for sludge loading is less than is required for settling, so settling controls.

Sizing for Partial-Mix Cells based on Sludge Accumulation

1996 article indicates that measured sludge accumulation in operating facilities has been significantly less (up to 3 times) than estimated by information in the 1982 article.
The 1996 article cites sludge accumulation rates of 44 m^3/yr per L/s and 38 m^3/yr per L/s as measured over a 7-year period for two dual-power multi-cellular aerated lagoon processes in South Carolina.
Therefore, an accumulation rate of 40 m^3/yr per L/s will be used to estimate sludge accumulation. Given the high I/I and low influent BOD concentrations, this accumulation rate is likely still somewhat conservative. Over time, VSS will be reduced through digestion. It is conservatively assumed that 75% of the sludge solids is VSS and approx. 40% of VSS is destroyed, which is the minimum for Class B biosolids.

Average Flow =	**2.00**	= Average Annual Flow, MGD
Average Flow =	87.62	= Average Annual Flow, L/s
Avg. Accumulation =	**40**	= m^3/yr per L/s
Accum. Rate =	3505	= m^3/yr
Assumed % VSS =	75%	
Accum. VSS =	2629	= m^3/yr
Min. VSS Destroyed =	1051	= m^3/yr
Net Accum. Rate =	2453	= m^3/yr
5-yr Accum. =	12,267	= m^3
5-yr Accum. =	3.2	= MG

So, size of the partial-mix cells should have about 3.2 MG of volume dedicated to sludge storage.

If approximately 3 feet of the 10 feet of total depth is allocated for sludge storage in the three partial-mix cells, the sludge storage is then as follows:

Total Depth =	10	ft
Sludge Depth =	3	ft
Partial-Mix Vol =	3.50	MG, each
Number Cells =	3	# partial-mix cells
Total Vol =	10.50	MG
Sludge Vol =	3.2	MG
Accum Duration =	4.9	yrs

3-foot sludge depth allows sufficient storage volume for approximately 5 years conservatively and also maintains sufficient separation between the water surface and sludge layer to avoid effluent quality impacts from resuspension. Actual storage duration could be much longer if VSS destruction is greater that assumed minimum of 40% and/or % VSS is higher.

Sizing for Partial-Mix Cells based on Control of Algal Growth

The 1996 article uses the following equation to estimate algal growth rate in the partial-mix cells:

(Eqn. 5 from 1996 article)

$$(V/Q)_j = [1 - (X_{An}/X_{A0})^{-1/n}]/\mu_A$$

Where:

hydraulic retention time in each partial-mix cell, days

$(V/Q)_j$ = algae in effluent of last cell, mg/L

X_{An} = algae in influent of first cell, mg/L

X_{A0} = number of cells in series

n = first-order growth rate for algae, 1/d

μ_A =

The graph below is a plot of total retention time in all partial-mix cells versus the ratio X_{An}/X_{A0}.

As stated in the 1996 article, a previous study indicated μ_A always less than 0.48 1/d.

Therefore, that value was used for μ_A.

The 1996 article states that for XAn/XA0 less than 25, significant algal growth should not result.
For 3 partial-mix cells in series, total retention time should not exceed about 4.4 days

At the current minimum monthly flow of about 600,000 gpd, that is equal to a total volume of only about 2.6 MG.
Considering the wide range of flows from PDF to the minimum monthly flow to I/I and increases in flow due to growth, there is no way to both reasonably minimize retention time for algal growth, and have sufficient volume to accommodate overflow rates at peak flow and provide sufficient volume for sludge accumulation and sufficient area for sludge loading. As a result, the City has taken other measures to control algae including use of ultrasonic transducers.

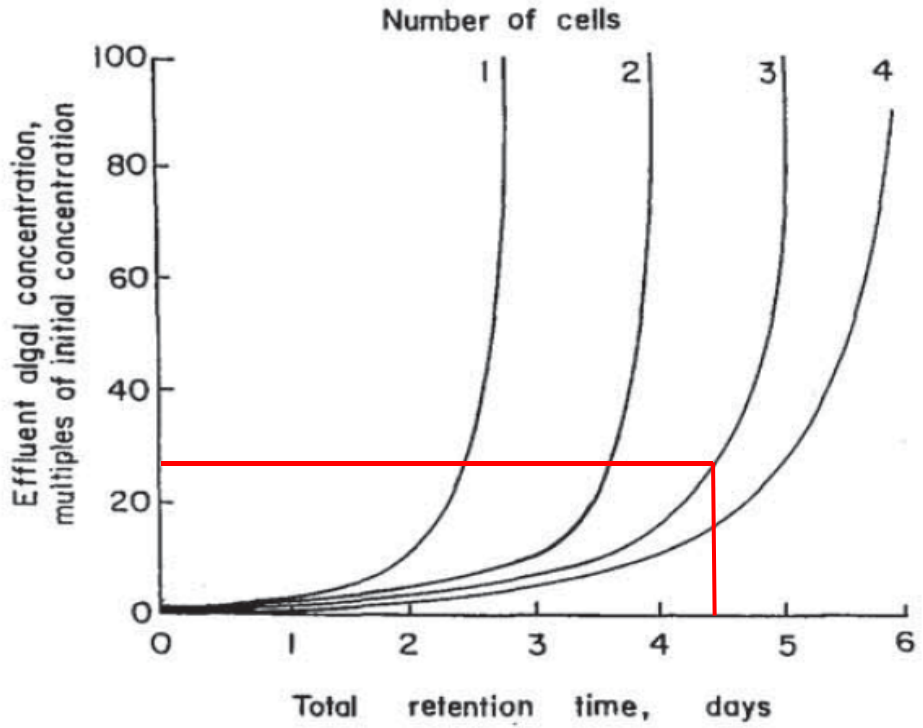


FIG. 4. Influence of Retention Time and Multicellular Configuration on Algal Growth [from Eq. (5)]

DESIGN OF DUAL-POWER MULTI-CELLULAR AERATED LAGOONS - Aeration and Mixing Sizing (Re-Rated at 3.30 MGD

This design method is based on the following articles:
Rich, L. (1982). "Design Approach to Dual-Power Aerated Lagoons." *J. Environ. Eng.* , 108(3), 532–548.
Rich, L. (1996). "Modification of Design Approach to Aerated Lagoons." *J. Environ. Eng.* , 122(2), 149–153.

For both the complete-mix and partial mix cells, the sizing of the aerators is based on the larger of the two power requirements for aeration and mixing.

Mixing Sizing for Complete-Mix Cells

For complete-mix cells, it has been found that a power level of 6 W/m^3 (30 hp/MG) is necessary to maintain solids in suspension and also maintain sufficientmixing intensity and turbidity to suppress algae growth (from 1982 article)
Therefore, the mixing energy required for the complete-mix cell ideally is:

Volume =	**10.0**	MG
Intensity =	30	hp/MG
Energy =	300	hp

Aeration Sizing for Complete-Mix Cells

Assuming substantial nitrification does not occur in the complete-mix cell (which would be the case during the winter months with higher flows and lower temperatures), the maximum oxygen consumption rate can be estimated by:

$$R_{O_2} = 4.16 \times 10^{-5} rQ(S_0 - S_1)(1.47 - 1.42\,FY)$$

(Eqn. 10 from 1982 article)

Where:

Ro_2 =	?	= oxygen consumption rate, kg/hr
r =	1.5	= diurnal peak ratio to average oxygen demand (assume 1.5 based on typical diurnal pattern)
Q =	**12,491**	= flow rate, m^3/d (max month flow)
S_0 =	154	= total influent BOD_5, mg/L (based on max month load at max month flow)
S_1 =	0	= complete-mix cell effluent soluble BOD_5, mg/L (conservatively assumed to be zero)

F =	0.67	= solids decay factor (typical value of 0.67 assumed per 1982 article)
Y =	0.5	= growth yield (for BOD) (from Table 1, 1982 article)
BOD_{Li} =	**4233**	= influent maximum month BOD_5 load, lbs/d (modified for 3.25)

Ro_2 =	119.2	kg/hr
Ro_2 =	262.8	lbs/hr

The above oxygen demand represents the demand for the winter months. Compare with the demand for summer months with lower flow, same BOD load and nitrification.
Nitrification demand will be based on assuming complete nitrification and all influent organic-N hydrolyzing to ammonia:

Q =	**7,571**	= flow rate, m^3/d (based on average annual flow associated with 3.25 MGD max mo. flow)
S_0 =	254	= total influent BOD_5, mg/L (max month load at peak day flow)
TKN_{Li} =	**564**	= total influent TKN load, lbs/d (max month, modified for 3.25 MGD)
TKN_{Li} =	10.66	= total influent TKN load, kg/h (max month)
Nit-O_2 =	4.6	= oxygen demand for nitrification, kg O_2/kg N
Nitrification =	39.0%	= % of ammonia nitrified in complete-mix cell

Ro_2 =	119.2	kg/hr	(without nitrification)
Ro_2 =	262.8	lbs/hr	Summer oxygen demand with no nitrification in complete-mix cell

Ro_2 =	138.33	kg/hr	(with nitrification)
Ro_2 =	305	lbs/hr	Summer oxygen demand with partial nitrification in complete-mix cell

Estimate the equivalent horsepower requirement to meet this oxygen demand using the following equation:

$$N = N_o\left(\frac{\beta C_{W_{alt}} - C_L}{C_{s_{20}}}\right)1.024^{T-20}\alpha$$

(Eqn. 10-19 from M&E 3rd Ed.)

Where:

N =	?		= oxygen transfer rate under field conditions, lbs/hp/hr
N_0 =	**2.0**		= oxygen transfer rate under standard conditions, lbs/hp/hr (3.0 per manufacturers for new and higher efficiency aerators)
β =	0.95		= salinity-surface tension correction factor (assume 0.9)
C_{Walt} =	**7.81**		= oxygen saturation at field temperature and elevation, mg/L (from M&E 3rd Ed. Appendix E , assuming max 25 degrees C temp)
C_L =	**2.00**		= operating oxygen concentration, mg/L
Cs_{20} =	**9.04**		= oxygen saturation at standard conditions, mg/L (from M&E 3rd Ed. Appendix E)
T =	**27.9**		= temperature, degrees C
α =	0.85		= oxygen transfer correction factor (assume 0.85)
N =	1.23	lbs/hp/hr	
Ro_2 with Nit =	305.0	lbs/hr	
Motor Eff. =	92%		
Req'd Energy =	270	hp	So, sufficient aeration energy to support full BOD removal and partial nitrification

Mixing Sizing for Partial-Mix Cells

For partial-mix cells, it has been found that a power level of 1 W/m^3 (5 hp/MG) is necessary to disperse dissolved oxygen throughout the lagoon and that a power level of up to 2 W/m^3 can still allow for sufficient settling of solids (from 1982 article)
Therefore, the mixing energy for the partial-mix cell should be:

Volume =	**10.5**	MG
Min Intensity =	5	hp/MG
Max Intensity =	10	hp/MG
Min Energy =	52.5	hp
Max Energy =	105.0	hp

Aeration Sizing for Partial-Mix Cells

As stated in the 1982 article, aeration demand in the partial-mix cells is dominated by residual BOD in the winter (when temperatures are low and benthal demand [i.e., oxygen consumption due to biological activity in the sludge layer] is low)
and by the benthal demand in the summer. In all but unusual conditions, the summer benthal demand is significantly larger. Therefore, the summer benthal demand will be used to determin aeration requirements for the partial-mix cells.
The benthal demand can be estimated using the following equation:

$$R_{O_2} = 4.16 \times 10^{-5} A_j B \theta^{T-20}$$

(Eqn. 12 from 1982 article)

Where:

Ro_2 =	?		= oxygen consumption rate, kg/hr
A_j =		3118	= bottom area of each partial-mix cell, m^2
B =		**60**	= rate of benthal oxygen demand at 20 deg. C, g $O_2/m^2/d$ (from 1982 article)
θ =		**1.05**	= temperature sensitivity coefficient (from 1982 article)
T =		**27.9**	= temperature, degrees C (assumed max month temperature)
Ro_2 =		11.44	kg/hr
Ro_2 =		25.23	lbs/hr
N =		0.92	lbs/hp/hr (for existing aerators)
Req'd Energy =		29.7	hp

Additional air is supplied by the SFF media blowers to assist with benthal demand and nitrification.

Appendix I

WWTP Alternative Life Cycle Costs

Costs for Filtration Improvements

	Units	DynaSand	ECOwash	(2) 7x7 Fuzzy Filter	FlexFilter	Notes
Equipment Cost						
On-Site Package	$	259,500	338,500	512,500	494,000	ECO includes 55,000 to retrofit existing
Media		84,000	84,000			compressible repl. Every 10 years.
Air Compressors	$	16,500	26,500		86,000	Use spare SFF media blower for Fuzzy Filter
Turbidity Monitoring				15,000	15,000	
Installation Labor	$	3,360	3,360	incl.	incl.	Labor on Dynasand quote was light. Used 4 workers, 80 hours, $50/hr
Filtration Package	***$***	***363,360***	***452,360***	***527,500***	***595,000***	
Filter Feed Pumps	$	56,619	56,619	56,619	56,619	Upsize FF pumps above 1.4 MGD
Reject Well Pumps	$	19,376	19,376	19,376		
FF Pipe	$	20,000	20,000	20,000	20,000	Upsize FF pipe above 1.4 MGD
Reject Pipe	$	47,502				LF * $7/LF * dia.
Alum Pump	$	1,800	1,800	1,800	1,800	
2500 gal. Alum Tank	$	35,200	35,200	35,200	35,200	Tank only needed for continuous backwash, other methods use less pre-treatment, double wall, insulated
Total Equipment Cost	**$**	**543,900**	**585,400**	**660,500**	**708,600**	
Structure						
Filter Cell Dimension	ft x ft	14x7	14x7	7x7		
Filter Cell Surface Area	sq. ft	100	100	49	90	
# of New Cells	#	2	2	2	3	
Total Filter Cell Surface Area	sq. ft	200	200	98	270	
Floc tank Length, width	ft	14	14	14	14	
Floc tank depth	ft	13	13	13	13	
Floc Tank Surface Area	sq. ft	196	196	196	196	
Floc tank Volume	gal	19,059	19,059	19,059	19,059	
Retention Time at 2.8 MGD	min	10	10	10	10	
Floc Tank Wall Volume	cu.ft	1,092	1,092	1,092	1,092	
Floc Tank Foundation Volume	cu.ft	490	490	490	490	
Floc Tank Concrete Volume	CY	40	40	40	40	
Added width around for slab	ft			2		
Added footprint	sq.ft			112		
Added foundation	cu.ft			280		
Total new surface area for grating	sq. ft	396	396	196	466	
Grating unit price	$/sq.ft	58	58	58	58	

Void depth below new cell					36	2 ft deep for Fuzzy, no dimension inform for Flex
Void fill unit and Foundation unit	$/CY	445	445	445	445	
Wall unit	$/CY	765	765	765	765	
Modify Outdoor Canopy Space for larger alum tank	$	10,000	10,000	10,000	10,000	
Concrete (w/ JH pricing)	$	104,791	104,791	0	52,395	Flex: Void fill plus other chamber modifications, use 1/2 of Dynasand new basin. For 3rd cell, use same concrete amt. as for new sand filter cell
Grating	$	22,964	22,964	11,366	27,023	Based on filter (except Fuzzy) and floc tank surface area
Void fill below filter	$	0	0	0	16,137	
New slab foundation	$	0	0	8,645	0	
Floc Tank	$	38,997	38,997	38,997	38,997	Floc: assume 10 min. HRT, 14x14x13 tank
Structures Total	***$***	***$176,800***	***$176,800***	***$69,100***	***$144,600***	
Subtotal Construction Cost	**$**	**720,700**	**762,200**	**729,600**	**853,200**	
Special Foundations	$	43,492	43,492	0	43,492	piling for new tank (filter, floc tank)
Site Works (5% of Subtotal, 10% for Flex)	$	36,035	38,110	36,480	42,660	
Electrical (15% of Subtotal)	$	108,105	114,330	109,440	127,980	
Mechanical (10% of Subtotal)	$	72,070	76,220	72,960	85,320	
Control (5% of Subtotal, instruments included in Quote)	$	36,035	38,110	36,480	42,660	
Subtotal Including Site Work, Electrical, Mechanical and Control	**$**	**1,016,500**	**1,072,500**	**985,000**	**1,195,400**	
Incidentals - OH&P, Mobilization, etc (15% of Structures and Equipment)	$	152,475	160,875	147,750	179,310	
Equipment, Structures, Incidentals Total	**$**	**1,169,000**	**1,234,000**	**1,133,000**	**1,375,000**	
Sales Tax (9.2%)	$	107,548	113,528	104,236	126,500	
OPCC Subtotal	**$**	**1,277,000**	**1,348,000**	**1,238,000**	**1,502,000**	
Contingency (35% of OPCC Subtotal)	$	409,150	431,900	396,550	481,250	
Total OPCC (2013)	**$**	**1,687,000**	**1,780,000**	**1,635,000**	**1,984,000**	
Plannning (5% of Total OPCC)	$	84,350	89,000	81,750	99,200	
Design and Permitting (10% of Total OPCC)	$	168,700	178,000	163,500	198,400	
Construction Mangmt. (15% of Total OPCC)	$	253,050	267,000	245,250	297,600	
Total Project Cost (2013)	**$**	**2,194,000**	**2,314,000**	**2,126,000**	**2,580,000**	
RSMeans Cost Index (2013)		203.4	203.4	203.4	203.4	
RSMeans Cost Index (2018)		221.5	221.5	221.5	221.5	
Adjusted Total Project Cost (2018)	**$**	**2,390,000**	**2,520,000**	**2,316,000**	**2,810,000**	

O&M Cost						
Air compressor/Blower	HP	5	5	40	40	
Backwash duration	min	60	22	15	75	
Backwash #		continuous	once per hour	3 to 5 per day	2 to 5 per day	
Backwash #	ea	24	24	4	4	
Backwash flow	gpm	292	292	980	75	
Backwash cycle on time	hr/day	24	9	1	5	
Daily blower power	kWh/day	90	33	30	149	
Annual blower power consumption	kWh/year	32675	11981	10892	54458	
Backwash Volume	MG/year	153.48	56.27	21.46	8.21	
HP consumption at	gpm	300	300	300	100	
Reject HP	HP	7.60	7.60	7.60	3.40	
Annual RJ flow power consumption	kWh/year	48341	17725	6760	3472	
Annual Power Consumption	kWh/year	81,016	29,706	17,652	57,930	Parkson based on backwash minutes and HP of compressor
Energy Cost	$/kWh	0.075	0.075	0.075	0.075	
Total Annual Energy Cost	$/Year	6,100	2,200	1,300	4,300	
						Alternatives other than continuous
Alum Consumption	lbs/day	226.0	180.8	180.8	180.8	backwash use less pretreatment
Alum Consumption at 1.9 MGD	lbs/year	82,490	65,992	65,992	65,992	
Price per Pound	cents/lb	0.24	0.24	0.24	0.24	24 cents/lb. today's price
# of lbs per delivery	Ea	9,000	9,000	9,000	9,000	
# of Deliveries/year	#	9	7	7	7	
Cost of Alum	$/year	$19,800	$15,838	$15,838	$15,838	
Fuel Surcharge per delivery	$/delivery	0	0	0	0	
Annual Fuel Surcharge	$/year	0	0	0	0	
Annual Alum Cost	$	$19,800	$15,838	$15,838	$15,838	
						Alternatives other than continuous
Polymer Consumption	gals/day	5.3	4.2	4.2	4.2	backwash use less pretreatment
Polymer Consumption at 1.9 MGD	gals/year	1,935	1,548	1,548	1,548	
Polymer Contained in barrel	gals	55	55	55	55	
# of barrels	Ea	36	29	29	29	
# of barrels Ordered per Delivery	Ea	4	4	4	4	
# of Deliveries	#	9	7	7	7	
Price for Barrels	$/barrel	$1,221	$1,221	$1,221	$1,221	$111/5 gal jug today's price
Cost of Polymer	$/year	$43,956	$35,409	$35,409	$35,409	
Fuel Surcharge per delivery	$/delivery	0	0	0	0	
Annual Fuel Surcharge	$/year	0	0	0	0	
Annual Polymer Cost	$	$43,956	$35,409	$35,409	$35,409	
Sand loss	%/year	1	1			Assumed
Additional Sand	$	840	840			
Annual Chemical Cost	$	64,600	52,100	51,200	51,200	

Monthly Labor Hours Requested	hrs/months	24	22	24	24	Duane said 8 hrs/mo., but says more now. Added 16 hrs/mo. for continuous and 14 hrs/mo. for ECO, since ECO replaces airlift less often. 16 hrs for comp. media plus 8 hrs for blower
Labor Cost	$/hr	50	50	50	50	
Total Annual Labor Cost	$/Year	14,400	13,200	14,400	14,400	
Media	**$**			**52,800**	**12,150**	Repl. Every 10 years.
Motor and actuator	$			24,000	24,000	Repl. Every 10 years.
Sum	$			76,800	36,150	
Life	years			10	10	
Replacement NPV	$			57,200	26,900	
Yearly Replacement Conversion				***6,706***	***3,154***	
Equal Annuity Factor of Present Value for the Period				0.12	0.12	
Internal Moveable Parts	**$**	**12,000**	**12,000**			
Life	years	5	7			Airlift $1500 ea
Replacement NPV	$	10,400	9,800			Continuous, replace 3-5 yrs
Yearly Replacement Conversion	***$***	***2,271***	***1,573***			ECO 7-10 yrs
Equal Annuity Factor of Present Value for the Period		0.22	0.16			
Internal Moveable Parts	**$**	**12,000**	**12,000**			
Life	years	10	14			
Replacement NPV	$	9,000	8,000			
Yearly Replacement Conversion	***$***	***1,055***	***708***			
Equal Annuity Factor of Present Value for the Period		0.12	0.09			
Internal Moveable Parts	**$**	**12,000**				
Life	years	15				
Replacement NPV	$	7,800				
Yearly Replacement Conversion	***$***	***653***				
Equal Annuity Factor of Present Value for the Period		0.08				
Yearly Replacement Conversion for 20 years	$	3,980	2,280	6,710	3,150	
Total Yearly O&M Cost (20 years)	**$/Year**	**89,100**	**69,800**	**73,600**	**73,100**	
Discount Rate (2010 rate for federal water projects)	%	3.000%	3.000%	3.000%	3.000%	
Period	years	20	20	20	20	
Present value factor of an equal annuity for the period		14.88	14.88	14.88	14.88	
20-Year O&M Cost	**$**	**1,326,000**	**1,038,000**	**1,095,000**	**1,088,000**	
20-Year NPV	**$**	**3,716,000**	**3,558,000**	**3,411,000**	**3,898,000**	

Delivered Sodium Hypochlorite Solution Cost Estimate

	Units	Cost	Notes
Equipment Cost			
1500-gallon Double-Contained Hypo Tank	$	12,000	PolyProcessing
Metering Pumps	EA	2	2 Duty, 1 Standby
Price of Metering Pump Skid	$	100,000	Based on TMG
Sodium Hypochlorite Feed System	**LS**	**112,000**	
Price of Water Champs	$	50,000	TMG
Instrumentation	$	0	Already installed
Air scrubber rehabilitation	$	40,000	Powell
Total Equipment Cost	**$**	**202,000**	
Structure Cost			
Construction			
Building Length	ft	20	
Building Width	ft	30	
Building Footprint	sq.ft	600	
Unit Cost for Building	$/sq.ft	468	
Cost for Building	*$*	*280688*	
Sump Volume Required	cu.ft	201	
Sump Depth	ft	1	
Wall Enclosed Sump	cu.ft	341	
Unit Cost for Sump Wall	$/CY	556	
Cost for sump	*$*	*7022*	
Total Building Cost	$	287,710	
CCT repair	LS	225,000	
Structures Total	***$***	***512700***	
Subtotal Construction Cost	**$**	**714,700**	
Special Foundations (0% of Subtotal)	$	0	
Site Works (5% of Subtotal)	$	35,735	
Electrical (15% of Subtotal)	$	107,205	
Mechanical (10% of Subtotal)	$	71,470	
Control (5% of Subtotal, instruments included in Quote)	$	35,735	
Subtotal Including Site Work, Electrical, Mechanical and Control	**$**	**964,900**	
Incidentals - OH&P, Mobilization, etc (15% Struct & Eqpt)	$	144,700	
Equipment, Structures, Incidentals Total	**$**	**1,110,000**	
Sales Tax (9.2%)	$	102,120	
OPCC Subtotal	**$**	**1,213,000**	
Contingency (35% of OPCC Subtotal)	$	424,550	
Total OPCC (2013)	**$**	**1,638,000**	
Plannning (5% of Total OPCC)	$	81,900	
Design and Permitting (10% of Total OPCC)	$	163,800	
Construction Mangmt. (15% of Total OPCC)	$	245,700	
Total Project Cost (2013)	**$**	**2,130,000**	
RSMeans Cost Index (2013)	$	203.4	
RSMeans Cost Index (2018)	$	221.5	
Adjusted Total Project Cost (2018)	**$**	**2,320,000**	
O&M Cost			
Hypo Dose at 2036 AAF	mg/L	2.00	
2036 AAF	MGD	1.9	
Hypo Dose at 2036 AAF	lbs/day	31.7	
Active Hypo Content	lb (NaOCl)/(lb sol)	12.5%	
Specific Gravity	-	1.20	
Hypo Weight per Gal	lb NaOCl/gal solution	10.0	
Cl2 Equivalent per Gal	lb (Cl2)/gal sol	1.05	
Hypo Doserate at 2036 AAF	gal Sol/day	30.2	

Estimated Power Consumption at AAF	kW	7.46	2 Water Champs, 5 HP each
Daily Power Consumption at AAF	kWh/day	179	
Annual Power Consumption at AAF	kWh	65350	
Engery Cost	$/kWh	0.075	
Total Annual Energy Cost	$/Year	4,900	
Hypo Consumption at 2036 AAF	gal/day	30.2	
Annual Hypo Consumption at 2036 AAF	gal/year	11035	
Price for Hypo	$/gal	1.87	
Volume per Delivery	gallon	900	
Delivery Times per Year	Ea	13	
Cost of Hypo	$/year	20635	
Fuel Surcharge per delivery	$/delivery	40	
Annual Fuel Surcharge	$/year	520	
Annual Hypo Cost	$	21,200	
Sulfur Dioxide Consumption at 2036 AAF	lbs/day	15.9	
Annual Sulfur Dioxide Consumption at 2036 AAF	lbs/year	5793	
Sulfur Dioxide Contained in Cylinder	lbs	150	
# of 150 lbs Cylinders	Ea	39	
# of Cylinders Ordered per Delivery	Ea	8	
# of Deliveries	#	5	
Price for 150 lbs Cylinder	$/cylinder	320.00	
Delivery Times per Year	Ea	5	
Cost of SO2	$/year	12480	
Fuel Surcharge per delivery	$/delivery	52	
Annual Fuel Surcharge	$/year	254	
Annual SO2 Cost	$	12,734	
Total Annual Chemical Cost	$	33,900	
Monthly Labor Hours Requested	hrs/months	20	12 (Hypo) 8 (SO2)
Labor Cost	$/hr	50	
Total Annual Labor Cost	$/Year	12,000	
Tank replacement	$	12,000	
Life or tank	years	10	10 years life
Present worth	$	12,000	
Equal Annuity Factor of Present Value for the Period		0.12	
Yearly replacement cost	$/year	1,400	
Total Yearly O&M Cost for 20 & 10 years	$/Year	52,200	
Discount Rate (2010 rate for federal water projects)	%	3.000%	
Period	years	20	
Present value factor of an equal annuity for the period		14.88	
20-Year O&M Cost	**$**	**777,000**	
20-Year Present Worth	***$***	***3,097,000***	

Onsite Sodium Hypochlorite Generation Cost Estimate

	Units	Cost	Notes
Equipment Cost			
On-Site Package		138,500	MicrOclor
Hypo tank (6550 gallon)	$	30,800	PolyProcessing, single wall
Price of Metering Pump Skid	$	100,000	Based on TMG
On-Site Generation System Package	**$**	**269,300**	
Price of Water Champ	$	50,000	TMG
Instrumentation	$	0	Already installed
Air scrubber rehabilitation	$	40,000	Powell
Total Equipment Cost	**$**	**359,300**	
Structure Cost			
Building Length	ft	22	
Building Width	ft	17	
Building Footprint	sq.ft	374	
Building Cost	$/sq.ft	453	
Total Building Cost	$	169,278	
CCT repair	LS	225,000	
Structures Total	***$***	***394300***	
Subtotal Construction Cost	**$**	**753,600**	
Special Foundations (0% of Subtotal)	$	0	
Site Works (5% of Subtotal)	$	37,680	
Electrical (15% of Subtotal)	$	113,040	
Mechanical (10% of Subtotal)	$	75,360	
Control (5% of Subtotal, instruments included in Quote)	$	37,680	
Subtotal Including Site Work, Electrical, Mechanical and Control	**$**	**1,017,400**	
Incidentals - OH&P, Mobilization, etc (15% Struct & Eqpt)	$	152,600	
Equipment, Structures, Incidentals Total	**$**	**1,170,000**	
Sales Tax (9.2%)	$	107,640	
OPCC Subtotal	**$**	**1,278,000**	
Contingency (35% of OPCC Subtotal)	$	447,300	
Total OPCC (2013)	**$**	**1,726,000**	
Plannning (5% of Total OPCC)	$	86,300	
Design and Permitting (10% of Total OPCC)	$	172,600	
Construction Mangmt. (15% of Total OPCC)	$	258,900	
Total Project Cost (2013)	**$**	**2,244,000**	
RSMeans Cost Index (2013)	$	203.4	
RSMeans Cost Index (2018)	$	221.5	
Adjusted Total Project Cost (2018)	**$**	**2,444,000**	
O&M Cost			
2036 PD DWF Use as Design Production	ppd	200	
2036 AA Use	ppd	31.7	
Estimated Power Consumption at AAF (with 2 Water Champs, 5 HP each)	kW	7.46	
Daily Power Consumption at AAF	kWh/day	179	
Annual Power Consumption at AAF	kWh	65350	
Power Consumption per One Pounds Cl2 Production	kWh/lbs Cl2	2.0	
Estimated Power Consumption at AA Use	kWh/day	63.49	
Annual Power Consumption	kWh/year	88523	
Engery Cost	$/kWh	0.075	
Total Annual Energy Cost	$/Year	11,500	

Salt Consumption per One Pounds Cl2 Production	lbs	3.0	
Salt Use at AA Use	lbs	95	
Annual Salt Use	lbs	34760	
# of 50 lbs bags per year	Ea	695	
Delivery Times per Year	Ea	24	
Bi-Weekly Delivery	Ea	29	
Cost of Bag	$/bag	5	
	$/year	3,476	
Fuel Surcharge per delivery	$/delivery	40	
	$/year	960	
Annual Salt Cost	*$*	*4,400*	
Sulfur Dioxide Consumption at 2036 AAF	lbs/day	15.9	
Annual Sulfur Dioxide Consumption at 2036 AAF	lbs/year	5793	
Sulfur Dioxide Contained in Cylinder	lbs	150	
# of 150 lbs Cylinders	Ea	39	
# of Cylinders Ordered per Delivery	Ea	8	
# of Deliveries	#	5	
Price for 150 lbs Cylinder	$/cylinder	320.00	
Delivery Times per Year	Ea	5	
Cost of SO2	$/year	12480	
Fuel Surcharge per delivery	$/delivery	52	
Annual Fuel Surcharge	$/year	254	
Annual SO2 Cost	*$*	*12,734*	
Total Annual Chemical Cost	$	17,100	
Monthly Labor Hours Requested	hrs/months	20	120 to 150 hours per year for Hypo
Labor Cost	$/hr	50	
Total Annual Labor Cost	$/Year	12,000	
One electrolytic cell replacement	$	25,000	$5000 per cell
Life or electrolytic cell	years	10	7 -10 years in full service
Present worth(P)	$	18,700	
Equal Annuity Factor of Present Value for the Period		0.12	
Yearly replacement cost (A)	$/year	2,200	
Tank replacement	$	30,800	
Life or tank	years	10	10 years life
Present worth	$	30,800	
Equal Annuity Factor of Present Value for the Period		0.12	
Yearly replacement cost	$/year	3,600	
Total Yearly O&M Cost for 20 & 10 years	$/Year	46,400	
Discount Rate (2010 rate for federal water projects)	%	3.000%	
Period	years	20	
Present value factor of an equal annuity for the period		14.88	
20-Year O&M Cost	**$**	**690,000**	
20-Year Present Worth	***$***	***3,134,000***	

UV Cost Estimate

	Units	Cost	Notes
Equipment Cost			
UV Equipment Package Cost	**$**	**595,400**	Calgon Carbon
Cost of Hoist	$	10,000	
Total Equipment Cost	**$**	**605,400**	
Structure Cost			
Construction			
Wall Thickness	ft	1	
Slab Thickness	ft	1	
Inlet and Outlet Box Length	ft	10	
Lengthwise Wall Length	ft	60.4	
Lengthwise Wall Depth	ft	6	
Lengthwise Wall Volume	cu.ft	725	
Widthwise Wall Length	ft	4.3	
Widthwise Wall Depth	ft	6	
Widthwise Wall Volume	cu.ft	51	
Wall Unit Cost	$/CY	1300	
Wall Cost	$	37363	
Bottom Slab Length	ft	62.4	
Bottom Slab Width	ft	8.3	
Bottom Slab Volume	cu.ft	515	
Walkway Width	ft	5	
Top Slab Length	ft	72	
Top Slab Width	ft	18	
Tank Length	ft	60	
Tank Width	ft	6	
Top Slab Volume	cu.ft	944	
Slab Unit Cost	$/CY	556	
Slab Cost	$	30043	
Grating Length	ft	58	
Grating Width	ft	4	
Grating Area	sq.ft	248	
Grating Unit Cost	$/SF	58	
Grating Cost	$	533	
UV Canopy Dimentsions	LxW	20x30	
UV Canopy Footprint	sq.ft	600	
UV Canopy Unit Cost	$/SF	75	
Cost of UV Canopy	$	45000	
Structures Total	***$***	***112900***	
Subtotal Construction Cost	**$**	**718,300**	
Special Foundations (0% of Subtotal)	$	0	
Site Works (5% of Subtotal)	$	35,915	
Electrical (15% of Subtotal)	$	107,745	
Mechanical (10% of Subtotal)	$	71,830	
Control (5% of Subtotal, instruments included in Quote)	$	35,915	
Subtotal Including Site Work, Electrical, Mechanical and Control	**$**	**969,800**	
Incidentals - OH&P, Mobilization, etc (15% Struct & Eqpt)	$	145,500	
Equipment, Structures, Incidentals Total	**$**	**1,116,000**	
Sales Tax (9.2%)	$	102,672	
OPCC Subtotal	**$**	**1,219,000**	
Contingency (35% of OPCC Subtotal)	$	426,650	
Total OPCC (2013)	**$**	**1,646,000**	
Plannning (5% of Total OPCC)	$	82,300	
Design and Permitting (10% of Total OPCC)	$	164,600	
Construction Mangmt. (15% of Total OPCC)	$	246,900	
Total Project Cost (2013)	**$**	**2,140,000**	
RSMeans Cost Index (2013)	$	203.4	
RSMeans Cost Index (2018)	$	221.5	
Adjusted Total Project Cost (2018)	**$**	**2,331,000**	

O&M Cost			
2036 Peak Day Flow (Attenuated Peak Hour Flow)	MGD	9.42	
2036 Max Month Flow	MGD	3.22	
Total SS	mg/L	30	
Design UV Dose at Peak Day Flow at End of Lamp Life	mJ/cm2	40	
UVT	%	50%	
Quartz Fouling Factor		0.9	from Calgon Carbon
End of Lamp Life Factor		0.9	
Number of Channels		1	
Channel Length	inches	581	
Channel Width	inches	42	
Channel Width2	inches	84	
Equivalent Width	inches	51	
Channel Depth	inches	72	
Number of Banks in Duty per Channel	EA	2	
Number of Banks on Stand-by per Channel	EA	1	
Modules per Bank	EA	7	
Number of Lamps per Module	EA	8	
Total Number of Duty Lamps	EA	112	
Total Number of Lamps	EA	168	
Power Consumption	kW	98.16	
Power Consumption per PDC	kW	32.72	
Estimated Headloss through Channel at Peak Flow	inches	7.32	
Annual Replacement Parts			from Calgon Carbon
Lamp	EA	41	
	$/Ea	250	
	$	10,225	
Ballast	EA	17	
	$/Ea	400	
	$	6,720	
Quartz	EA	3	
	$/Ea	90	
	$	306	
Scraper	EA	34	
	$/Ea	85	
	$	2,856	
Annual Replacement Cost		20,200	
Estimated Power Consumption at AAF	kW	31.10	from Calgon Carbon
Daily Power Consumption at AAF	kWh/day	746	
Annual Power Consumption at AAF	kWh	272436	
Engery Cost	$/kWh	0.075	
Total Annual Energy Cost	$/Year	20,400	
Monthly Labor Hours Requested	hrs/months	30	356.4 man hours per yr
Labor Cost	$/hr	$50	
Total Annual Labor Cost	$/Year	18,000	
Total Yearly O&M Cost	$/Year	58,600	
Discount Rate (2010 rate for federal water projects)	%	3.000%	
Period	years	20	
Present value factor of an equal annuity for the period		14.88	
20-Year O&M Cost	**$**	**872,000**	
20-Year Present Worth	***$***	***3,203,000***	

Delivered Peracetic Acid Solution Cost Estimate

	Units	Cost	Notes
Equipment Cost			
300-gallon Totes (Provided by PAA Supplier)	$	0	
Metering Pumps	EA	2	2 Duty, 1 Standby
Price of pre-packaged PAA system building	$	125,000	Based on TMG
PAA Feed System	**LS**	**125,000**	
Diffusers and Control Valves	$	12,600	
Piping	$	12,800	
Carrier Water Pump	$	5,000	
Emergency Shower	$	4,000	
Instrumentation and Programming	$	65,000	
Total Equipment Cost	**$**	**224,400**	
Structure Cost			
Construction			
Containment Area Canopy, Length	ft	16	
Containment Area Canopy, Width	ft	10	
Canopy Footprint	sq.ft	160	
Unit Cost for Canopy	$/sq.ft	200	
Cost for Canopy	*$*	*32000*	From City
Foundation/Containment Concrete	CY	27	
Unit Cost for Concrete Foundation/Containment	$/CY	600	
Grating Area	sq.ft	60	
Unit cost of Grating	$/sq.ft	50	
Cost for Foundation/Containment	*$*	*19,200*	
Total Building Cost	$	51,200	
Demoltion	LS	10,000	
CCT repair	LS	225,000	
Structures Total	***$***	***286200***	
Subtotal Construction Cost	**$**	**510,600**	
Special Foundations (0% of Subtotal)	$	0	
Site Works (5% of Subtotal)	$	25,530	
Electrical (15% of Subtotal, self-performed by City)	$	76,590	
Mechanical (10% of Subtotal)	$	51,060	
Control (5% of Subtotal)	$	25,530	
Subtotal Including Site Work, Electrical, Mechanical and Control	**$**	**689,400**	
Incidentals - OH&P, Mobilization, etc (15% Struct & Eqpt)	$	103,400	
Equipment, Structures, Incidentals Total	**$**	**793,000**	
Sales Tax (9.2%)	$	72,956	
OPCC Subtotal	**$**	**866,000**	
Contingency (35% of OPCC Subtotal)	$	303,100	
Total OPCC (2018)	**$**	**1,170,000**	
Plannning (0% of Total OPCC, completed w/ PAA testing)	$	0	
Design and Permitting (15% of Total OPCC)	$	175,500	
Construction Mangmt. (10% of Total OPCC)	$	117,000	
Total Project Cost (2018)	**$**	**1,463,000**	
O&M Cost			
PAA Dose at 2036 AAF	mg/L	0.9	
2036 AAF	MGD	1.9	
PAA Dose at 2036 AAF	lbs/day	14.3	
Active Hypo Content	lb (NaOCl)/(lb sol)	15.0%	
Specific Gravity	-	1.16	
PAA Weight per Gal	lbs/gal solution	9.7	
PAA Equivalent per Gal	lbs PAA/gal sol	1.45	
PAA Dose Rate at 2036 AAF	gal Sol/day	9.8	

Estimated Power Consumption at AAF	kW	7.46	One 10 HP water booster pump
Daily Power Consumption at AAF	kWh/day	179	
Annual Power Consumption at AAF	kWh	65350	
Engery Cost	$/kWh	0.075	
Total Annual Energy Cost	$/Year	4,900	
PAA Consumption at 2036 AAF	gal/day	9.8	
Annual PAA Consumption at 2036 AAF	gal/year	3593	
Price for PAA	$/lb	1.06	
Price for PAA	$/gal	10.26	
Volume per Delivery	gallon	330	
Delivery Times per Year	Ea	11	
Cost of PAA	$/year	36880	
Fuel Surcharge per delivery	$/delivery	0	
Annual Fuel Surcharge	$/year	0	
Annual PAA Cost	$	36,900	
Sulfur Dioxide Consumption at 2036 AAF	lbs/day	0.0	
Annual Sulfur Dioxide Consumption at 2036 AAF	lbs/year	0	
Sulfur Dioxide Contained in Cylinder	lbs	150	
# of 150 lbs Cylinders	Ea	0	
# of Cylinders Ordered per Delivery	Ea	8	
# of Deliveries	#	0	
Price for 150 lbs Cylinder	$/cylinder	320.00	
Delivery Times per Year	Ea	0	
Cost of SO2	$/year	0	
Fuel Surcharge per delivery	$/delivery	52	
Annual Fuel Surcharge	$/year	0	
Annual SO2 Cost	$	0	
Total Annual Chemical Cost	$	36,900	
Monthly Labor Hours Requested	hrs/months	12	12 (PAA)
Labor Cost	$/hr	50	
Total Annual Labor Cost	$/Year	7,200	
Tank replacement	$	0	
Life or tank	years	10	10 years life
Present worth	$	0	
Equal Annuity Factor of Present Value for the Period		0.12	
Yearly replacement cost	$/year	0	
Total Yearly O&M Cost for 20 & 10 years	$/Year	49,000	
Discount Rate (2010 rate for federal water projects)	%	3.000%	
Period	years	20	
Present value factor of an equal annuity for the period		14.88	
20-Year O&M Cost	**$**	**729,000**	
20-Year Present Worth	***$***	***2,192,000***	

Costs for Aeration Improvements

Design

Manufacturer	Aerator Solutions	Aqua-Aerobics	RWL	Reliant Water
Type	Floating Surface Aerator	Floating Surface Aerator	Floating Aspirating Aerator	Diffused Aeration
Aerator Model	EcoJet	AquaJet FSS Endura Series	Tornado	Lagoon Master
Aerator Speed	High	High	High	N/A
Number of Aerators	18	18	18	48
Number of Motors	18	18	18	96
Motor Size, HP	15	15	20	2
Mfr. Standard Oxygen Transfer Rate, lbs/HP/hr	3.0	3.0	2.5	3.8
Design Standard Oxygen Transfer Rate, lbs/HP/hr	2.0	2.0	1.7	2.8
Total Power, HP	270	270	360	192
Power Required to Meet Design AOTR, HP	270	270	318	192
Mixing Energy, HP/MG	27	27	36	N/A
Aerator Materials	Stainless steel w/ epoxy-coated steel cone and fiberglass float (stainless steel option for both)	Stainless steel	Stainless steel	Aluminum frame and diffusers
Mooring Cables Included?	Yes	Yes	No	No
Electrical Cables Included?	Yes	Yes	No	No
Electrical Disconnects Included?	Yes	No	No	No

Capital Cost

Equipment Price	$ 188,000	$ 172,000	$ 239,076	$ 854,208
Installation Allowance (25%)	$ 47,000	$ 43,000	$ 59,769	$ 213,552
OH&P, Mobilization, etc. (15%)	$ 35,250	$ 32,250	$ 44,827	$ 160,164
Subtotal	$ 270,250	$ 247,250	$ 343,672	$ 1,227,924
Sales Tax (9.2%)	$ 24,863	$ 22,747	$ 31,618	$ 112,969
OPCC Subtotal	**$ 295,113**	**$ 269,997**	**$ 375,290**	**$ 1,340,893**
Contingency (35% of OPCC Subtotal)	$ 103,290	$ 94,499	$ 131,351	$ 469,313
Total OPCC	**$ 398,403**	**$ 364,496**	**$ 506,641**	**$ 1,810,206**
Plannning (5% of Total OPCC)	$ 19,920	$ 18,225	$ 25,332	$ 90,510
Design and Permitting (10% of Total OPCC)	$ 39,840	$ 36,450	$ 50,664	$ 181,021
Construction Mangmt. (15% of Total OPCC)	$ 59,760	$ 54,674	$ 75,996	$ 271,531
Total Project Cost	**$ 518,000**	**$ 474,000**	**$ 659,000**	**$ 2,353,000**

O&M Cost

Annual Power Consumption, kWh/yr	1,763,730	1,763,730	2,351,640	1,254,208
Energy Cost, $/kWh	$ 0.075	$ 0.075	$ 0.075	$ 0.075
Total Annual Energy Cost	**$ 132,280**	**$ 132,280**	**$ 176,373**	**$ 94,066**
Typical Annual Labor per Aerator, hrs	8	8	8	8
Typical Total Annual Labor, hrs	144	144	144	384
Labor Rate, $/hr	$ 50	$ 50	$ 50	$ 50
Total Annual Labor Cost	**$ 7,200**	**$ 7,200**	**$ 7,200**	**$ 19,200**
Total Annual O&M Cost	**$ 139,480**	**$ 139,480**	**$ 183,573**	**$ 113,266**
Discount Rate, %	3.00%	3.00%	3.00%	3.00%
Period, years	20	20	20	20
Present Value Factor	14.88	14.88	14.88	14.88
20-Year O&M Cost	**$ 2,075,000**	**$ 2,075,000**	**$ 2,731,000**	**$ 1,685,000**
20-Year Present Worth	**$ 2,593,000**	**$ 2,549,000**	**$ 3,390,000**	**$ 4,038,000**

Appendix J

CIP Detailed Cost Estimates

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP C-3: CSO 2 Separation Program
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 2/18/19

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Mobilization	$204,900	1	LS	$204,900
2	Temporary Erosion & Sediment Control	$41,000	1	LS	$41,000
3	Dewatering	$41,000	1	LS	$41,000
4	30-inch PVC Stormwater Pipe, SDR 35 (Open Cut)	$748	1,200	LF	$897,600
5	12-inch PVC Stormwater Pipe, SDR 35 (Open Cut)	$182	1,800	LF	$327,600
6	15-inch PVC Sanitary Sewer Pipe, SDR 35 (Open Cut)	$217	1,200	LF	$260,400
7	8-inch PVC Sanitary Sewer Pipe, SDR 35 (Open Cut)	$170	1,800	LF	$306,000
8	48-inch Manhole	$8,807	10	EA	$88,070
9	CSBC	$32	1,233	TN	$39,260
10	HMA Overlay	$190	683	TN	$129,833
11	Traffic Control	$41,000	1	LS	$41,000
12	General Restoration	$41,000	1	LS	$41,000
	Subtotal				$2,417,664
	Sales Tax	9.1%			$220,007
	ESTIMATED CONSTRUCTION COST				$2,637,671
	Construction Contingency	35%			$923,185
	TOTAL ESTIMATED CONSTRUCTION COST				$3,561,000

Planning	5%	$178,000
Design and Permitting	10%	$356,000
Services During Construction	15%	$534,000
TOTAL ESTIMATED PROJECT COST		$4,629,000
Notes		$2,315,000

1. Gen. Rest., Dewatering, Traffic Control, Erosion Control at 2% Construction Costs
2. Mobilization is assumed to be 10% of Construction
3. Pipe cost includes all fittings, pipe, bedding, excavation, haul, and pavement restoration

The opinion of probable cost herein is based on our perception of current conditions at the project location. This opinion reflects our professional opinion of costs at this time and is subject to change as the project design progresses. BHC Consultants has no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. BHC Consultants cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the costs presented as shown.

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP C-5: Upsize Gravity Main at 13th Street and Avenue A
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 2/18/19

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Mobilization	$46,800	1	LS	$46,800
2	Temporary Erosion & Sediment Control	$9,400	1	LS	$9,400
3	Dewatering	$9,400	1	LS	$9,400
3	10-inch PVC Sewer Pipe, SDR 35	$180	1,008	LF	$181,440
4	12-inch PVC Sewer Pipe, SDR 35	$182	624	LF	$113,568
5	Rehabilitate 48-inch Manhole (Average 6.5 ft Deep)	$6,685	10	EA	$66,849
6	Side Sewer Connections	$2,971	3	EA	$8,913
7	Connection to Existing Manhole	$1,592	3	EA	$4,775
8	CSBC	$32	671	TN	$21,358
9	HMA Trench Patch	$190	372	TN	$70,629
10	Traffic Control	$5,700	1	LS	$5,700
11	General Restoration	$5,700	1	LS	$5,700
	Subtotal				$544,532
	Sales Tax	9.1%			$49,552
	ESTIMATED CONSTRUCTION COST				$594,084
	Construction Contingency	35%			$207,929
	TOTAL ESTIMATED CONSTRUCTION COST				$802,000

Planning	5%	$40,000
Design and Permitting	10%	$80,000
Services During Construction	15%	$120,000
TOTAL ESTIMATED PROJECT COST		$1,042,000

Notes
1. Gen. Rest., Dewatering, Traffic Control, Erosion Control at 2% Construction Costs
2. Mobilization is assumed to be 10% of Construction
3. Pipe cost includes all fittings, pipe, bedding, excavation, haul, and pavement restoration

The opinion of probable cost herein is based on our perception of current conditions at the project location. This opinion reflects our professional opinion of costs at this time and is subject to change as the project design progresses. BHC Consultants has no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. BHC Consultants cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the costs presented as shown.

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP C-6: Western Gravity Flow Diversion
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 2/18/19

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Mobilization	$137,100	1	LS	$137,100
2	Temporary Erosion & Sediment Control	$27,400	1	LS	$27,400
3	Dewatering	$27,400	1	LS	$27,400
4	18-inch Horizontal Directional Drilling (including pipe and pit)	$500	1,250	LF	$625,000
5	Unforseen Obstructions or Soil Conditions	$200,000	1	LS	$200,000
6	12-inch PVC Sewer Pipe, SDR 35 (Open Cut)	$182	950	LF	$172,900
7	18-inch PVC Sewer Pipe, SDR 35 (Open Cut)	$233	800	LF	$186,400
8	48-inch Manhole	$8,807	5	EA	$44,035
9	Pump Station Decommissioning	$42,000	2	EA	$84,000
10	Rehabilitate 48-inch Manhole (Average 6.5 ft Deep)	$6,685	2	EA	$13,370
11	CSBC	$32	329	TN	$10,469
12	HMA Trench Patch	$190	182	TN	$34,622
13	Traffic Control	$27,400	1	LS	$27,400
14	General Restoration	$27,400	1	LS	$27,400
	Subtotal				$1,617,497
	Sales Tax	9.1%			$147,192
	ESTIMATED CONSTRUCTION COST				$1,764,689
	Construction Contingency	35%			$617,641
	TOTAL ESTIMATED CONSTRUCTION COST				$2,382,000

Planning	5%	$119,000
Design and Permitting	10%	$238,000
Services During Construction	15%	$357,000
TOTAL ESTIMATED PROJECT COST		$3,096,000

Notes
1. Gen. Rest., Dewatering, Traffic Control, Erosion Control at 2% Construction Costs
2. Mobilization is assumed to be 10% of Construction
3. Pipe cost includes all fittings, pipe, bedding, excavation, haul, and pavement restoration

The opinion of probable cost herein is based on our perception of current conditions at the project location. This opinion reflects our professional opinion of costs at this time and is subject to change as the project design progresses. BHC Consultants has no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. BHC Consultants cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the costs presented as shown.

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP C-7: Rainier Pump Station (PS 2) Force Main Replacement
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 2/18/19

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Mobilization	$33,500	1	LS	$33,500
2	Temporary Erosion & Sediment Control	$6,700	1	LS	$6,700
3	Dewatering	$6,700	1	LS	$6,700
4	8-inch PVC C900 Force Main	$132	1,813	LF	$239,259
5	Air Release/Vacuum Valve Vault	$33,530	0	EA	$0
6	8-inch Dia. Force Main Blow-off Assembly	$10,611	0	EA	$0
7	48-inch Manhole	$8,807	2	EA	$17,614
8	HMA Trench Patch	$190	413	TN	$78,444
9	Traffic Control	$6,700	1	LS	$6,700
10	General Restoration	$6,700	1	LS	$6,700
	Subtotal				$395,617
	Sales Tax	9.1%			$36,001
	ESTIMATED CONSTRUCTION COST				$431,618
	Construction Contingency	35%			$151,066
	TOTAL ESTIMATED CONSTRUCTION COST				$583,000

Planning	5%	$29,000
Design and Permitting	10%	$58,000
Services During Construction	15%	$87,000
TOTAL ESTIMATED PROJECT COST		$757,000

Notes
1. Gen. Rest., Dewatering, Traffic Control, Erosion Control at 2% Construction Costs
2. Mobilization is assumed to be 10% of Construction
3. Pipe cost includes all fittings, pipe, bedding, excavation, haul, and pavement restoration

The estimate of probable cost herein is based on our perception of current conditions at the project location. This estimate reflects our professional opinion of accurate costs at this time and is subject to change as the project design matures. BHC Consultants has no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. BHC Consultants cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the costs presented as shown.

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP C-9: Interurban Trail Conveyance System Upgrade
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 2/18/19

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Mobilization	$45,100	1	LS	$45,100
2	Temporary Erosion & Sediment Control	$9,000	1	LS	$9,000
3	Dewatering	$9,000	1	LS	$9,000
4	12-inch PVC Sewer Pipe, SDR 35	$182	210	LF	$38,220
5	15-inch PVC Sewer Pipe, SDR 35	$217	1,450	LF	$314,650
6	48-inch Manhole (Average Depth 11ft)	$8,807	6	EA	$52,842
7	Connection to Existing Manhole	$1,592	2	EA	$3,183
8	Pipe Connections	$4,244	3	EA	$12,733
9	AbandonPipe	$9,900	1	LS	$9,900
10	Top Soil and Hydroseed	$11	1,844	SY	$19,571
11	Traffic Control	$9,000	1	LS	$9,000
12	General Restoration	$9,000	1	LS	$9,000
	Subtotal				$532,200
	Sales Tax	9.1%			$48,430
	ESTIMATED CONSTRUCTION COST				$580,630
	Construction Contingency	35%			$203,220
	TOTAL ESTIMATED CONSTRUCTION COST				$784,000

Planning	5%	$39,000
Design and Permitting	10%	$78,000
Services During Construction	15%	$118,000
TOTAL ESTIMATED PROJECT COST		$1,019,000

Notes
1. Gen. Rest., Dewatering, Traffic Control, Erosion Control at 2% Construction Costs
2. Mobilization is assumed to be 10% of Construction
3. Pipe cost includes all fittings, pipe, bedding, excavation, haul, and pavement restoration

The opinion of probable cost herein is based on our perception of current conditions at the project location. This opinion reflects our professional opinion of costs at this time and is subject to change as the project design progresses. BHC Consultants has no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. BHC Consultants cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the costs presented as shown.

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP C-11: CIPP Pipe Restoration North of Blackmans Lake
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 2/18/19

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Mobilization	$19,900	1	LS	$19,900
2	Temporary Erosion & Sediment Control	$4,000	1	LS	$4,000
3	Dewatering	$4,000	1	LS	$4,000
4	Temporary Sewer Bypass	$50,000	1	LS	$50,000
5	15-inch CIPP	$80	1,700	LF	$136,000
6	Manhole Rehabilitation	$6,685	2	EA	$13,370
7	Traffic Control	$4,000	1	LS	$4,000
8	General Restoration	$4,000	1	LS	$4,000
	Subtotal				$235,270
	Sales Tax	9.1%			$21,410
	ESTIMATED CONSTRUCTION COST				$256,679
	Construction Contingency	35%			$89,838
	TOTAL ESTIMATED CONSTRUCTION COST				$347,000

Planning	5%	$17,000
Design and Permitting	10%	$35,000
Services During Construction	15%	$52,000
TOTAL ESTIMATED PROJECT COST		$451,000

Notes
1. Gen. Rest., Dewatering, Traffic Control, Erosion Control at 2% Construction Costs
2. Mobilization is assumed to be 10% of Construction
3. Pipe cost includes all fittings, pipe, bedding, excavation, haul, and pavement restoration

The opinion of probable cost herein is based on our perception of current conditions at the project location. This opinion reflects our professional opinion of costs at this time and is subject to change as the project design progresses. BHC Consultants has no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. BHC Consultants cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the costs presented as shown.

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP C-12: Pipe Extension North of the Casino Pump Station (PS 14)
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 2/18/19

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Mobilization	$67,100	1	LS	$67,100
2	Temporary Erosion & Sediment Control	$13,400	1	LS	$13,400
3	Dewatering	$13,400	1	LS	$13,400
4	18-inch PVC Sewer Pipe, SDR 35	$233	2,100	LF	$489,300
5	48-inch Manhole (Average 6.5 ft Deep)	$8,807	7	EA	$61,649
6	Connection to Pump Station	$1,592	1	EA	$1,592
7	CSBC	$32	863	TN	$27,482
8	HMA Trench Patch	$190	478	TN	$90,883
9	Traffic Control	$3,600	1	LS	$3,600
10	General Restoration	$3,600	1	LS	$3,600
	Subtotal				$772,006
	Sales Tax	9.1%			$70,253
	ESTIMATED CONSTRUCTION COST				$842,259
	Construction Contingency	35%			$294,791
	TOTAL ESTIMATED CONSTRUCTION COST				$1,137,000

Planning	5%	$57,000
Design and Permitting	10%	$114,000
Services During Construction	15%	$171,000
TOTAL ESTIMATED PROJECT COST		$1,479,000

Notes
1. Gen. Rest., Dewatering, Traffic Control, Erosion Control at 2% Construction Costs
2. Mobilization is assumed to be 10% of Construction
3. Pipe cost includes all fittings, pipe, bedding, excavation, haul, and pavement restoration

The opinion of probable cost herein is based on our perception of current conditions at the project location. This opinion reflects our professional opinion of costs at this time and is subject to change as the project design progresses. BHC Consultants has no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. BHC Consultants cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the costs presented as shown.

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP C-13: New Gravity Line and Decommissioning the Ferguson Pump Station (PS 8)
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 2/18/19

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Mobilization	$93,200	1	LS	$93,200
2	Temporary Erosion & Sediment Control	$18,600	1	LS	$18,600
3	Dewatering	$18,600	1	LS	$18,600
4	8-inch PVC Sewer Pipe, SDR 35	$170	1,450	LF	$246,500
5	48-inch Manhole (Average 6.5 ft Deep)	$8,807	5	EA	$44,035
6	Connection to Pump Station	$1,592	1	EA	$1,592
7	Side Sewer and Grinder Pumps	$42,444	12	EA	$509,323
8	Pump Station Decommissioning	$42,000	1	LS	$42,000
9	Abandon Existing Force Main	$700	1	LS	$700
10	Abandon Existing Gravity Pipe	$6,600	1	LS	$6,600
11	CSBC	$32	596	TN	$18,976
12	HMA Trench Patch	$190	330	TN	$62,753
13	Traffic Control	$13,700	1	LS	$13,700
14	General Restoration	$13,700	1	LS	$13,700
	Subtotal				$1,090,278
	Sales Tax	9.1%			$99,215
	ESTIMATED CONSTRUCTION COST				$1,189,493
	Construction Contingency	35%			$416,323
	TOTAL ESTIMATED CONSTRUCTION COST				$1,606,000

Planning	5%	$80,000
Design and Permitting	10%	$161,000
Services During Construction	15%	$241,000
TOTAL ESTIMATED PROJECT COST		$2,088,000

Notes
1. Gen. Rest., Dewatering, Traffic Control, Erosion Control at 2% Construction Costs
2. Mobilization is assumed to be 10% of Construction
3. Pipe cost includes all fittings, pipe, bedding, excavation, haul, and pavement restoration

The opinion of probable cost herein is based on our perception of current conditions at the project location. This opinion reflects our professional opinion of costs at this time and is subject to change as the project design progresses. BHC Consultants has no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. BHC Consultants cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the costs presented as shown.

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP P-1: Champagne Pump Station (PS 7) Upgrade and New Force Main
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 2/18/19

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Mobilization	$64,100	1	LS	$64,100
2	Temporary Erosion & Sediment Control	$12,800	1	LS	$12,800
3	Dewatering	$12,800	1	LS	$12,800
4	4-inch PVC C900 Force Main	$115	757	LF	$87,055
5	Air Release/Vacuum Valve Vault	$33,530	1	EA	$33,530
6	4-inch Dia. Force Main Blow-off Assembly	$10,717	2	EA	$21,434
7	10-inch PVC Sewer Pipe, SDR 35	$180	559	LF	$100,620
8	8-inch PVC Sewer Pipe, SDR 35	$170	550	LF	$93,500
9	48-inch Manhole	$8,807	2	EA	$17,614
10	Pump Station Decommissioning	$42,000	1	LS	$42,000
11	HMA Trench Patch	$190	172	TN	$32,761
12	Sandblast, Caulk Jts and Coat Int Wet Well, Pipe & Equip.	$12,733	1	LS	$12,733
13	SST Pump Rails	$4,244	1	LS	$4,244
14	Disch Piping in Wet Well, incl. support/thrust restraint	$7,428	1	LS	$7,428
15	Gorman Rupp Pumps for PS 7 390 gpm	$23,250	2	EA	$46,500
16	Primary Power Supply	$12,733	1	LS	$12,733
17	UG Power & Controls to Wet Well & Vaults	$9,550	1	LS	$9,550
18	Level Controls in Wet Well	$2,122	1	LS	$2,122
19	Pump Inst. & Controls in NEMA 3R Enclosures	$47,749	1	LS	$47,749
20	MCC in NEMA 3R Enclosures	$26,527	1	LS	$26,527
21	Telemetry	$12,733	1	LS	$12,733
22	Flow Meter Vault	$22,283	1	LS	$22,283
23	Misc. Yard Piping (water, vault drains, site SD)	$7,428	1	LS	$7,428
24	Traffic Control	$12,800	1	LS	$12,800
25	General Restoration	$12,800	1	LS	$12,800
26	Minor Landscaping	$6,000	1	LS	$6,000
	Subtotal				$762,000
	Sales Tax	9.1%			$69,342
	OPINION OF PROBABLE CONSTRUCTION COST				$831,342
	Construction Contingency	35%			$290,970
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST				$1,122,000

Planning	5%	$56,000
Design and Permitting	10%	$112,000
Services During Construction	15%	$168,000
TOTAL OPINION OF PROBABLE ALLIED COST		$336,000
TOTAL OPINION OF PROJECT COST		$1,458,000

Notes
1. Gen. Rest., Dewatering, Traffic Control, Erosion Control at 2% Construction Costs
2. Mobilization is assumed to be 10% of Construction
3. Pipe cost includes all fittings, pipe, bedding, excavation, haul, and pavement restoration

The opinion of probable cost herein is based on our perception of current conditions at the project location. This opinion reflects our professional opinion of costs at this time and is subject to change as the project design progresses. BHC Consultants has no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. BHC Consultants cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the costs presented as shown.

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP No. P-3: Mobile 80 kW Generator
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 2/18/19

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Generator Set 80 kW	$38,239	1	EA	$38,239
2	Trailer for Portable Generator	$10,611	1	LS	$10,611
	Subtotal				$49,000
	Sales Tax	9.1%			$4,000
	ESTIMATED CONSTRUCTION COST				$53,000
	Construction Contingency	20%			$11,000
	TOTAL ESTIMATED CONSTRUCTION COST				$64,000
	Planning	0%			$0
	Design and Permitting	0%			$0
	Services During Construction	0%			$0
	TOTAL ESTIMATED PROJECT COST				$64,000

Notes
1. Gen. Rest., Traffic Control, Erosion Control at 2% Construction Costs

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City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP No. P-4: Rainier Pump Station (PS 2) Generator (100kW)
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 2/18/19

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Mobilization	$25,800	1	LS	$25,800
2	Temporary Erosion & Sediment Control	$5,200	1	LS	$5,200
3	Generator Set 100 kW	$42,458	1	EA	$42,458
4	Generator Housing	$10,611	1	EA	$10,611
5	Equipment Pad	$10,611	1	LS	$10,611
6	Property for generator	$193,887	1	LS	$193,887
7	Traffic Control	$5,200	1	LS	$5,200
8	General Restoration	$5,200	1	LS	$5,200
	Subtotal				$299,000
	Sales Tax	9.1%			$27,000
	ESTIMATED CONSTRUCTION COST				$326,000
	Construction Contingency	35%			$114,000
	TOTAL ESTIMATED CONSTRUCTION COST				$440,000

Planning	5%	$22,000
Design and Permitting	10%	$44,000
Services During Construction	15%	$66,000
TOTAL ESTIMATED PROJECT COST		$572,000

Notes
1. Gen. Rest., Traffic Control, Erosion Control at 2% Construction Costs
2. Mobilization is assumed to be 10% of Construction

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City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP No. P-5: Rainier Pump Station (PS 2) Upgrades
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 2/18/19

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Mobilization	$34,100	1	LS	$34,100
2	Temporary Erosion & Sediment Control	$6,800	1	LS	$6,800
3	Electrical Equipment Foundation (6-in gravel & 6-in reinf. conc. pad)	$1,592	1	LS	$1,592
4	Flow Meter Vault	$22,283	1	LS	$22,283
5	Level Controls in Wet Well	$1,592	1	LS	$1,592
6	Sandblast, Caulk Jts and Coat Int Wet Well, Pipe & Equip.	$12,733	2	EA	$25,466
7	SST Pump Rails	$4,244	2	EA	$8,489
8	Disch Piping in Wet Well, incl. support/thrust restraint	$7,428	2	EA	$14,855
9	Pump - Cornell 8NHTR	$87,540	3	EA	$262,620
10	Level Controls in Wet Well	$2,122	2	EA	$4,244
11	Traffic Control	$6,800	1	LS	$6,800
12	General Restoration	$6,800	1	LS	$6,800
	Subtotal				$395,640
	Sales Tax	9.1%			$36,003
	ESTIMATED CONSTRUCTION COST				$431,643
	Construction Contingency	35%			$151,075
	TOTAL ESTIMATED CONSTRUCTION COST				$583,000

Planning	5%	$29,000
Design and Permitting	10%	$58,000
Services During Construction	15%	$87,000
TOTAL ESTIMATED PROJECT COST		$757,000

Notes
1. Gen. Rest., Dewatering, Traffic Control, Erosion Control at 2% Construction Costs
2. Mobilization is assumed to be 10% of Construction

The opinion of probable cost herein is based on our perception of current conditions at the project location. This opinion reflects our professional opinion of costs at this time and is subject to change as the project design progresses. BHC Consultants has no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. BHC Consultants cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the costs presented as shown.

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP No P-6: Lincoln Pump Station (PS 3) Replacement
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 2/18/19

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Mobilization	$16,800	1	LS	$16,800
2	Temporary Erosion & Sediment Control	$3,400	1	LS	$3,400
3	Sandblast, Caulk Jts and Coat Int Wet Well, Pipe & Equip.	$12,733	1	LS	$12,733
4	UG Power & Controls to Wet Well & Vaults	$9,550	1	LS	$9,550
5	Eng-Generator Foundation	$3,183	1	LS	$3,183
6	Weather/Acoustical Enclosure w/Eng-Gen, fuel tank, critical silencer, ATS)	$79,582	1	LS	$79,582
7	Pump - Flygt NP 3102 SH 3 ~ Adaptive 256 280gpm/37ft	$14,280	2	EA	$28,560
8	Pump Standard Accessories	$3,735	2	EA	$7,470
9	Flow Meter Vault	$26,527	1	LS	$26,527
10	Drain for Valve Box	$212	1	LS	$212
11	Traffic Control	$3,400	1	LS	$3,400
12	General Restoration	$3,400	1	LS	$3,400
13	Minor Landscaping	$6,000	1	LS	$6,000
	Subtotal				$201,000
	Sales Tax	9.1%			$18,291
	OPINION OF PROBABLE CONSTRUCTION COST				$219,291
	Construction Contingency	35%			$76,752
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST				$296,000

Planning	5%	$15,000
Design and Permitting	10%	$30,000
Services During Construction	15%	$44,000
TOTAL OPINION OF PROBABLE ALLIED COST		$89,000
TOTAL OPINION OF PROJECT COST		$385,000

Notes
Notes
1. Gen. Rest., Dewatering, Traffic Control, Erosion Control at 2% Construction Costs
2. Mobilization is assumed to be 10% of Construction

The opinion of probable cost herein is based on our perception of current conditions at the project location. This opinion reflects our professional opinion of costs at this time and is subject to change as the project design progresses. BHC Consultants has no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. BHC Consultants cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the costs presented as shown.

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP No. P-7: Rainier Pump Station (PS 2) Wet Well Replacement/Expansion
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 2/18/19

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Mobilization	$10,200	1	LS	$10,200
2	Temporary Erosion & Sediment Control	$2,000	1	LS	$2,000
3	Wet Well	$25,563	2	EA	$51,125
4	Construction sequencing, bypass pumping, excavation, reconnections, etc.	$51,125	1	LS	$51,125
5	Traffic Control	$2,000	1	LS	$2,000
6	General Restoration	$2,000	1	LS	$2,000
	Subtotal				$118,451
	Sales Tax	9.1%			$10,779
	ESTIMATED CONSTRUCTION COST				$129,230
	Construction Contingency	35%			$45,230
	TOTAL ESTIMATED CONSTRUCTION COST				$174,000

Planning	5%	$9,000
Design and Permitting	10%	$17,000
Services During Construction	15%	$26,000
TOTAL ESTIMATED PROJECT COST		$226,000

Notes
1. Gen. Rest., Dewatering, Traffic Control, Erosion Control at 2% Construction Costs
2. Mobilization is assumed to be 10% of Construction

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP No. W-5: Add Non-Potable Water Air Gaps
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 2/18/19

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Mobilization	$23,000	1	LS	$23,000
2	Pumps and Controls	$80,000	1	LS	$80,000
3	Piping and Valves	$70,000	1	LS	$70,000
4	Tank	$5,000	2	EA	$10,000
5	Building Foundation	$1,000	8	CY	$8,000
6	Pre-Engineered Metal Building	$350	100	SF	$35,000
7	HVAC/Mechanical	$10,000	1	LS	$10,000
8	Electrical and Lighting	$21,000	1	LS	$21,000
9	General Restoration	$5,000	1	LS	$5,000
	Subtotal				$262,000
	Sales Tax	9.1%			$24,000
	ESTIMATED CONSTRUCTION COST				$286,000
	Construction Contingency	35%			$100,000
	TOTAL ESTIMATED CONSTRUCTION COST				$386,000
	Planning	5%			$19,000
	Design and Permitting	10%			$39,000
	Services During Construction	15%			$58,000
	TOTAL ESTIMATED PROJECT COST				$502,000

Notes
1. Gen. Rest., Traffic Control, Erosion Control at 2% Construction Costs
2. Mobilization is assumed to be 10% of Construction

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City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP No. W-6: Replace MCCs and Retrofit the Influent Screw Pumps with VFDs
Prepared by: T. Neier
Reviewed by: T. Giese
Date: 4/20/2018

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Mobilization	$162,300	1	LS	$162,300
2	EMCC-1	$636,653	1	LS	$636,653
3	EMCC-2	$381,992	1	LS	$381,992
4	NMCC	$297,105	1	LS	$297,105
5	VFDs for 3 Influent Screw Pumps	$307,716	1	LS	$307,716
6	General Restoration	$32,500	1	LS	$32,500
	Subtotal				$1,818,000
	Sales Tax	9.1%			$165,000
	ESTIMATED CONSTRUCTION COST				$1,983,000
	Construction Contingency	35%			$694,000
	TOTAL ESTIMATED CONSTRUCTION COST				$2,677,000
	Planning	5%			$134,000
	Design and Permitting	10%			$268,000
	Services During Construction	15%			$402,000
	TOTAL ESTIMATED PROJECT COST				$3,481,000

Notes
1. Gen. Rest., Traffic Control, Erosion Control at 2% Construction Costs
2. Mobilization is assumed to be 10% of Construction

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Appendix K

Snohomish WWTP NPDES Permit

Issuance Date: May 21, 2018
Effective Date: July 1, 2018
Expiration Date: June 30, 2023

National Pollutant Discharge Elimination System
Waste Discharge Permit No. WA0029548

State of Washington
DEPARTMENT OF ECOLOGY
Northwest Regional Office
3190 160th Avenue SE
Bellevue, WA 98008-5452

In compliance with the provisions of
The State of Washington Water Pollution Control Law
Chapter 90.48 Revised Code of Washington
and
The Federal Water Pollution Control Act
(The Clean Water Act)
Title 33 United States Code, Section 1342 et seq.

City of Snohomish Wastewater Treatment Plant

116 Union Avenue
Snohomish, WA 98290

is authorized to discharge in accordance with the Special and General Conditions that follow.

Plant Location:
2115 2nd Street
Snohomish, WA 98290

Receiving Water:
Snohomish River at River Mile 12.1

Treatment Type:
Aerated Lagoon with Submerged Fixed-Film Media System

Rachel McCrea

Rachel McCrea
Water Quality Section Manager
Northwest Regional Office
Washington State Department of Ecology

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Summary of Permit Report Submittals

Refer to the Special and General Conditions of this permit for additional submittal requirements.

Permit Section	Submittal	Frequency	First Submittal Date
S3.A	Discharge Monitoring Report (DMR)	Monthly	August 15, 2018
S3.A	Discharge Monitoring Report (DMR)	Annual	January 15, 2020
S3.F	Reporting Permit Violations	As necessary	
S4.B	Plans for Maintaining Adequate Capacity	As necessary	
S4.D	Notification of New or Altered Sources	As necessary	
S4.E	Infiltration and Inflow Evaluation	1/permit cycle	June 30, 2019
S5.F	Bypass Notification	As necessary	
S5.G	Operations and Maintenance Manual Update	1/permit cycle	May 1, 2022
S6.E	Industrial User Survey Submittal	1/permit cycle	May 1, 2022
S8.1	Compliance Schedule: Engineering Report	1/permit cycle	July 31, 2018
S8.2	Compliance Schedule: Plans and Specifications	1/permit cycle	May 31, 2019
S8.2	Compliance Schedule: Construction completion	1/permit cycle	August 1, 2020
S9.A	Effluent Mixing Plan of Study	1/permit cycle	August 1, 2020
S9.A	Effluent Mixing Report	1/permit cycle	August 1, 2021
S10.C	Combined Sewer Overflow Report	Annually	September 15, 2018
S11.A	Acute Toxicity Effluent Test Results - Submit with Permit Renewal Application	2/permit cycle	April 30, 2021 and October 31, 2021 submittals for testing conducted in February and August 2021.
S12.A	Chronic Toxicity Effluent Test Results with Permit Renewal Application	2/permit cycle	April 30, 2021 and October 31, 2021 submittals for testing conducted in February and August 2021.
S13	Application for Permit Renewal	1/permit cycle	January 1, 2023
G1	Notice of Change in Authorization	As necessary	
G4	Reporting Planned Changes	As necessary	
G5	Engineering Report for Construction or Modification Activities	As necessary	
G7	Notice of Permit Transfer	As necessary	
G10	Duty to Provide Information	As necessary	
G20	Compliance Schedules	As necessary	
G21	Contract Submittal	As necessary	

Special Conditions

S1. Discharge limits

S1.A. Effluent limits

All discharges and activities authorized by this permit must comply with the terms and conditions of this permit. The discharge of any of the following pollutants more frequently than, or at a level in excess of, that identified and authorized by this permit violates the terms and conditions of this permit.

Beginning on the effective date of this permit, the Permittee may discharge treated domestic wastewater to the Snohomish River at the permitted location subject to compliance with the following limits:

Effluent Limits: Outfall 001 Latitude: 47.912552 Longitude: -122.110974		
Parameter	**Average Monthly [a]**	**Average Weekly [b]**
Carbanaceous Biochemical Oxygen Demand (5-day) ($CBOD_5$)	25 milligrams/liter (mg/L) 85% removal of influent $CBOD_5$	40 mg/L
$CBOD_5$ *Effective November – June Only*	584 pounds/day (lbs/day)	934 lbs/day
Total Suspended Solids (TSS) *April through October*	30 mg/L 701 lbs/day 85% removal of influent TSS	45 mg/L 1,051 lbs/day
Total Suspended Solids (TSS) *November through March*	30 mg/L 701 lbs/day 72.2% removal of influent TSS	45 mg/L 1,051 lbs/day
Parameter	**Daily Minimum**	**Daily Maximum**
pH [c]	6.4 standard units	9.0 standard units
Parameter	**Monthly Geometric Mean**	**Weekly Geometric Mean**
Fecal Coliform Bacteria [d]	200/100 milliliter (mL)	400/100 mL
Parameter	**Average Monthly**	**Maximum Daily [e]**
NBOD+CBOD [f] *Effective July – October Only*	134 lbs/day	301 lbs/day
Total Residual Chlorine [g]	83 micrograms/liter (µg/L)	209 µg/L
Total Residual Peracetic Acid (PAA) Interim limit [h]		1.0 mg/L

a	Average monthly effluent limit means the highest allowable average of daily discharges over a calendar month. Calculate the average value by adding the value of each daily discharge measured during a calendar month and dividing by the total number of daily discharges. See footnote d for fecal coliform calculations.
b	Average weekly discharge limitation means the highest allowable average of "daily discharges" over a calendar week, calculated as the sum of all "daily discharges" measured during a calendar week divided by the number of "daily discharges" measured during that week. See footnote d for fecal coliform calculations.
c	Indicates the range of permitted values. Do not average pH values.
d	Ecology provides directions to calculate the monthly and the 7-day geometric mean in publication No. 04-10-020, Information Manual for Treatment Plant Operators available at: https://fortress.wa.gov/ecy/publications/documents/0410020.pdf

Effluent Limits: Outfall 001 Latitude: 47.912552 Longitude: -122.110974	
e	Maximum daily effluent limit is the highest allowable daily discharge. The daily discharge is the average discharge of a pollutant measured during a calendar day. For pollutants with limits expressed in units of mass, calculate the daily discharge as the total mass of the pollutant discharged over the day. This does not apply to pH.
f	NBOD+CBOD is calculated using the following equation: **NBOD+CBOD (lbs/day) = (2.1 * total ammonia-N (lbs/day)) + $CBOD_5$(lbs/day)** Calculate total ammonia-N and $CBOD_5$ using measurements from the same composite sample.
g	Once PAA is establish as the primary disinfection method, chlorine limits apply only during periods when PAA is not available and the permittee uses chlorine for disinfection.
h	Cease using peracetic acid and revert to chlorine disinfection immediately upon completion of the pilot testing. The limit for residual Peracetic Acid is effective beginning on the first calendar day following submission of written notice of the PAA system construction completion and lasting through the remaining term of this permit).

S1.B. Mixing zone authorization

Mixing zone for Outfall 001

Ecology classifies the Snohomish River in the vicinity of Outfall 001 as "Estuarine". The following paragraphs define the maximum boundaries of the authorized mixing zones:

Chronic mixing zone

The chronic mixing zone extends 214 feet downstream and upstream from the outfall, and extends 90 feet across the river channel as measured from the north bank. The mixing zone extends from the discharge ports to the top of the water surface. The concentration of pollutants at the edge of the chronic zone must meet chronic aquatic life criteria and human health criteria.

Acute mixing zone

The acute mixing zone extends 21.4 feet downstream and upstream from the outfall as measured from each port and extends 61.4 feet across the river channel as measured from the north bank. The mixing zone extends from the discharge ports to the top of the water surface. The concentration of pollutants at the edge of the acute zone must meet acute aquatic life criteria.

Mixing Zone Size and Location	
	
Available Dilution (dilution factor)	
Acute Aquatic Life Criteria	11
Chronic Aquatic Life Criteria	28
Human Health Criteria - Carcinogen	28
Human Health Criteria - Non-carcinogen	28

S2. Monitoring requirements

S2.A. Monitoring schedule

The Permittee must monitor in accordance with the following schedule and the requirements specified in Appendix A.

Parameter	Units & Speciation	Minimum Sampling Frequency	Sample Type
(1) Wastewater influent			
Wastewater Influent means the raw sewage flow from the collection system into the treatment facility. Sample the wastewater entering the headworks of the treatment plant excluding any side-stream returns from inside the plant.			
Flow	MGD	Continuous [1]	Metered
Biochemical Oxygen Demand (BOD_5)	mg/L	1/week	24-hr composite [2]
Biochemical Oxygen Demand (BOD_5	lbs/day	1/week	Calculated [3]
$CBOD_5$	mg/L	2/week	24-hr composite
Total Suspended Solids (TSS)	mg/L	2/week (April – October)	24-hr composite
Total Suspended Solids (TSS)	lbs/day	2/week (April – October)	Calculated [3]
Total Suspended Solids (TSS)	mg/L	4/week (***November – March***)	24-hr composite
Total Suspended Solids (TSS)	lbs/day	4/week (***November – March***)	Calculated [3]

Parameter	Units & Speciation	Minimum Sampling Frequency	Sample Type
(2) Final wastewater effluent			
Final Wastewater Effluent means wastewater exiting the last treatment process or operation. Typically, this is after or at the exit from the chlorine contact chamber or other disinfection process. The Permittee may take effluent samples for the $CBOD_5$ analysis before or after the disinfection process. If taken after, the Permittee must dechlorinate and reseed the sample.			
Flow	MGD	Continuous	Metered
$CBOD_5$	mg/L	2/week	24-hr composite
$CBOD_5$	lbs/day	2/week	Calculated [3]
$CBOD_5$	% removal	1/month	Calculated [4]
TSS	mg/L	2/week	24-hr composite
TSS	lbs/day	2/week	Calculated [3]
TSS	% removal	1/month	Calculated [4]
Total Ammonia	mg/L as N	2/week	24-hr composite
Total Ammonia	lbs/day	2/week	Calculated [3]
NBOD+CBOD (July-Oct. only)	lbs/day	2/week	Calculated [5]
Chlorine (Total Residual)	µg/L	Daily	Grab [6]
Fecal Coliform [7]	#Organisms /100 ml	2/week	Grab
pH [8]	Standard Units	Daily	Grab
Daily Maximum Temperature	Degrees centigrade (°C)	Continuous	Measurement [9]
7-DADMax Temperature	°C	Daily	Calculated [10]
Final PAA Residual[11]	mg/L	Daily	Grab
(3) Whole effluent toxicity testing – final wastewater effluent			
Acute Toxicity Testing	See Condition S11 for testing requirements.	2/permit cycle, dates specified in S11	24-hr composite
Chronic Toxicity Testing	See Condition S12 for testing requirements.	2/permit cycle, dates specified in S12	24-hr composite
(4) Effluent characterization – final wastewater effluent			
Total Phosphorus	mg/L as P	1/month	24-hr composite
Soluble Reactive Phosphorus	mg/L as P	1/month	24-hr composite
Nitrate plus Nitrite Nitrogen	mg/L as N	1/month	24-hr composite
Total Kjeldahl Nitrogen (TKN)	mg/L as N	1/month	24-hr composite
(5) Permit renewal application requirements – final wastewater effluent			
Dissolved Oxygen	mg/L	Once per year	Grab
Oil and Grease	mg/L	Once per year	Grab
Total Dissolved Solids	mg/L	Once per year	Grab
Total Hardness	mg/L	Once per year	Grab
Cyanide	micrograms/liter (µg/L)	Once per year	Grab
Total Phenolic Compounds	µg/L	Once per year	Grab
Priority Pollutants (PP) – Total Metals	µg/L; nanograms(ng/L) for mercury	Once per year	24-hr composite Grab for mercury
PP – Volatile Organic Compounds	µg/L	Once per year	Grab
PP – Acid-extractable Compounds	µg/L	Once per year	24-hr composite
PP – Base-neutral Compounds	µg/L	Once per year	24-hr composite

[1]	Continuous means uninterrupted except for brief lengths of time for calibration, for power failure, or for unanticipated equipment repair or maintenance.

Parameter	Units & Speciation	Minimum Sampling Frequency	Sample Type

2	24-hour composite means a series of individual samples collected over a 24-hour period into a single container, and analyzed as one sample.
3	Calculate mass discharge concurrently with the respective concentration measurement using the following formula: Concentration (mg/L) X Flow (MGD) X Conversion Factor (8.34) = lbs/day
4	% removal = $\frac{\text{(avg. influent concentration (mg/L) – avg. effluent concentration (mg/L))}}{\text{Influent concentration (mg/L)}} \times 100$ Calculate the percent (%) removal of $CBOD_5$ and TSS using the above equation.
5	Calculate NBOD+CBOD using the following equation: NBOD+CBOD (lbs/day) = (2.1 * total ammonia-N (lbs/day)) + $CBOD_5$(lbs/day) Calculate total ammonia-N and $CBOD_5$ using measurements from the same composite sample.
6	Grab means an individual sample collected over a fifteen (15) minute, or less, period.
7	Do not report fecal coliform as too numerous to count (TNTC). Report a numeric value consistent with Ecology's *Information Manual for Wastewater Treatment Plant Operators*, Publication Number 04-10-020 available at: https://fortress.wa.gov/ecy/publications/documents/0410020.pdf
8	Report the daily pH and the minimum and maximum for the monitoring period.
9	Determine and report a daily maximum from continuous measurements integrated over a maximum half-hour interval. Continuous monitoring instruments must achieve an accuracy of 0.2 degrees C and the Permittee must verify accuracy annually.
10	Calculate a 7-DAD Max for each day by averaging each day's maximum temperature value with the values from the six (6) preceding days.
11	The final PAA residual is the concentration of PAA in the effluent prior to discharge.

S2.B. Combined sewer overflow (CSO) monitoring schedule

The Permittee must monitor all discharges from CSO outfalls listed in Special Condition S10 using the following monitoring schedule. Permittees must use automatic flow monitoring equipment to collect the information required below. Permittee must calibrate flow monitoring equipment according to requirements in Condition S2.D.

Parameter	Units	Minimum Sampling Frequency	Sample Type
CSO discharge is defined as any untreated CSO which will exit or has exited the CSO outfall.			
Volume Discharged	Gallons	Per Event [c]	Measurement/Calculation [a,b]
Discharge Duration	Hours	Per Event [c]	Measurement
Storm Duration	Hours	Per Event [d]	Measurement
Precipitation	Inches	Per Event [c]	Measurement/Calculation [b]

Footnotes for CSO Monitoring:

a	Flow measurement must be continuous, except for brief lengths of time for calibration, for power failure, or for unanticipated equipment repair or maintenance. During periods of interrupted service, a calculation may be used to estimate the discharge volume. An explanation must be provided in the monthly DMR for all disruptions in flow measurement.
b	"Measurement/Calculation" means the total volume of the discharge or amount of precipitation event as estimated by direct measurement or indirectly by calculation (i.e. flow weirs, pressure transducers, tipping bucket). Precipitation must be measured by the nearest possible precipitation-measuring device and actively monitored during the period of interest.
c	"Per Event" means a unique flow event as defined in the *Permit Writer's Manual* https://fortress.wa.gov/ecy/publications/summarypages/92109.html. Ecology defines the minimum inter-event period (MIET) as 24 hours. A CSO event is considered to have ended only after at least 24 hours has elapsed since the last measured occurrence of an overflow.
d	Storm duration is the amount of total time when precipitation occurred that contributed to a discharge event. It is determined on a case-by-case basis.

Ecology recommends that the City continue tracking any public complaints or comments about observed water quality impacts from CSO events that may be received and making observations to confirm there are no noticeable toxic effects or aesthetic impacts following a CSO event.

S2.C. Sampling and analytical procedures

Samples and measurements taken to meet the requirements of this permit must represent the volume and nature of the monitored parameters. The Permittee must conduct representative sampling of any unusual discharge or discharge condition, including bypasses, upsets, and maintenance-related conditions that may affect effluent quality.

Sampling and analytical methods used to meet the monitoring requirements specified in this permit must conform to the latest revision of the *Guidelines Establishing Test Procedures for the Analysis of Pollutants* contained in 40 CFR Part 136 (or as applicable in 40 CFR subchapters N [Parts 400–471] or O [Parts 501-503]) unless otherwise specified in this permit . Ecology may only specify alternative methods for parameters without permit limits and for those parameters without an EPA approved test method in 40 CFR Part 136.

S2.D. Flow measurement, field measurement, and continuous monitoring devices

The Permittee must:

1. Select and use appropriate flow measurement, and continuous monitoring devices and methods consistent with accepted scientific practices.
2. Install, calibrate, and maintain these devices to ensure the accuracy of the measurements is consistent with the accepted industry standard and the manufacturer's recommendation for that type of device.
3. Calibrate continuous monitoring instruments weekly unless it can demonstrate a longer period is sufficient based on monitoring records. The Permittee:
    a. May calibrate apparatus for continuous monitoring of dissolved oxygen by air calibration.
    b. Must calibrate continuous chlorine measurement instruments using a grab sample analyzed in the laboratory within 15 minutes of sampling.
4. Calibrate flow monitoring devices at a minimum frequency of at least one calibration per year.
5. Maintain calibration records for at least three years.

S2.E. Laboratory accreditation

The Permittee must ensure that all monitoring data required by Ecology for permit specified parameters is prepared by a laboratory registered or accredited under the provisions of chapter 173-50 WAC, *Accreditation of Environmental Laboratories*. Flow, temperature, settleable solids, conductivity, pH, and internal process control parameters are exempt from this requirement. The Permittee must obtain accreditation for conductivity and pH if it must receive accreditation or registration for other parameters.

S2.F. Request for reduction in monitoring

The Permittee may request a reduction of the sampling frequency after twelve (12) months of monitoring. Ecology will review each request and at its discretion grant the request when it reissues the permit or by a permit modification.

The Permittee must:

1. Provide a written request.
2. Clearly state the parameters for which it is requesting reduced monitoring.
3. Clearly state the justification for the reduction.

S3. Reporting and recording requirements

The Permittee must monitor and report in accordance with the following conditions. Falsification of information submitted to Ecology is a violation of the terms and conditions of this permit.

S3.A. Discharge monitoring reports

The first monitoring period begins on the effective date of the permit (unless otherwise specified). The Permittee must:

1. Summarize, report, and submit monitoring data obtained during each monitoring period on the electronic discharge monitoring report (DMR) form provided by Ecology within the Water Quality Permitting Portal. Include data for each of the parameters tabulated in Special Conditions S2.A (treatment plant monitoring), S2.B (CSO monitoring), and as required by the form. Report a value for each day sampling occurred (unless specifically exempted in the permit) and for the summary values (when applicable) included on the electronic form.
2. Ensure that DMRs are electronically submitted no later than the dates specified below, unless otherwise specified in this permit.
3. The Permittee must also submit an electronic copy of the laboratory report as an attachment using WQWebDMR. The contract laboratory reports must also include information on the chain of custody, QA/QC results, and documentation of accreditation for the parameter.
4. Submit DMRs for parameters with the monitoring frequencies specified in S2 (monthly, quarterly, annual, etc.) at the reporting schedule identified below. The Permittee must:
    a. Submit **monthly** DMRs by the 15th day of the following month.
    b. Submit **annual DMRs**, unless otherwise specified in the permit, by January 15 for the previous calendar year. The annual sampling period is the calendar year.
5. Enter the "No Discharge" reporting code for an entire DMR, for a specific monitoring point, or for a specific parameter as appropriate, if the Permittee did not discharge wastewater or a specific pollutant during a given monitoring period.

6. Report single analytical values below detection as "less than the detection level (DL)" by entering < followed by the numeric value of the detection level (e.g. < 2.0) on the DMR. If the method used did not meet the minimum DL and quantitation level (QL) identified in the permit, report the actual QL and DL in the comments or in the location provided.
7. Report single analytical values between the detection level (DL) and the quantitation level (QL) by entering the estimated value, the code for estimated value/below quantitation limit (j) and any additional information in the comments. Submit a copy of the laboratory report as an attachment using WQWebDMR.
8. Not report zero for bacteria monitoring. Report as required by the laboratory method.
9. Calculate and report an arithmetic average value for each day for bacteria if multiple samples were taken in one day.
10. Calculate the geometric mean values for bacteria (unless otherwise specified in the permit) using:
    a. The reported numeric value for all bacteria samples measured above the detection value except when it took multiple samples in one day. If the Permittee takes multiple samples in one day it must use the arithmetic average for the day in the geometric mean calculation.
    b. The detection value for those samples measured below detection.
11. Report the test method used for analysis in the comments if the laboratory used an alternative method not specified in the permit and as allowed in Appendix A.
12. Calculate average values and calculated total values (unless otherwise specified in the permit) using:
    a. The reported numeric value for all parameters measured between the detection value and the quantitation value for the sample analysis.
    b. One-half the detection value (for values reported below detection) if the lab detected the parameter in another sample from the same monitoring point for the reporting period.
    c. Zero (for values reported below detection) if the lab did not detect the parameter in another sample for the reporting period.
13. Report single-sample grouped parameters (for example: priority pollutants, PAHs, pulp and paper chlorophenolics, TTOs) on the WQWebDMR form and include: sample date, concentration detected, detection limit (DL) (as necessary), and laboratory quantitation level (QL) (as necessary).

S3.B. Permit submittals and schedules

The Permittee must use the Water Quality Permitting Portal – Permit Submittals application (unless otherwise specified in the permit) to submit all other written permit-required reports by the date specified in the permit.

When another permit condition requires submittal of a paper (hard-copy) report, the Permittee must ensure that it is postmarked or received by Ecology no later than the dates specified by this permit. Send these paper reports to Ecology at:

Water Quality Permit Coordinator
Department of Ecology
Northwest Regional Office
3190 160th Avenue SE
Bellevue, WA 98008-5452

S3.C. Records retention

The Permittee must retain records of all monitoring information for a minimum of three (3) years. Such information must include all calibration and maintenance records and all original recordings for continuous monitoring instrumentation, copies of all reports required by this permit, and records of all data used to complete the application for this permit. The Permittee must extend this period of retention during the course of any unresolved litigation regarding the discharge of pollutants by the Permittee or when requested by Ecology.

S3.D. Recording of results

For each measurement or sample taken, the Permittee must record the following information:

1. The date, exact place, method, and time of sampling or measurement.
2. The individual who performed the sampling or measurement.
3. The dates the analyses were performed.
4. The individual who performed the analyses.
5. The analytical techniques or methods used.
6. The results of all analyses.

S3.E. Additional monitoring by the Permittee

If the Permittee monitors any pollutant more frequently than required by Special Condition S2 of this permit, then the Permittee must include the results of such monitoring in the calculation and reporting of the data submitted in the Permittee's DMR unless otherwise specified by Special Condition S2.

S3.F. Reporting permit violations

The Permittee must take the following actions when it violates or is unable to comply with any permit condition:

1. Immediately take action to stop, contain, and cleanup unauthorized discharges or otherwise stop the noncompliance and correct the problem.
2. If applicable, immediately repeat sampling and analysis. Submit the results of any repeat sampling to Ecology within thirty (30) days of sampling.

a. Immediate reporting

The Permittee must immediately report to Ecology and the Local Health Jurisdiction at the numbers listed below, all:

- Failures of the disinfection system.
- Collection system overflows.
- Plant bypasses discharging to marine surface waters.
- Any other failures of the sewage system (pipe breaks, etc.).
- This reporting requirement does not apply to Combined Sewer Overflows (CSO), discharging from outfalls, as a result of precipitation, and as authorized in condition S10.A of this permit. These reporting requirements do apply to Dry Weather Overflows from CSO outfalls that are prohibited by the Nine Minimum Controls listed in S10.B.

Northwest Regional Office	425-649-7000
Snohomish Health District	425-339-5250 (business hours) 425-339-5295 (after business hours)

Additionally, for any sanitary sewer overflow (SSO) that discharges to a municipal separate storm sewer system (MS4), the Permittee must notify the appropriate MS4 owner or operator.

b. Twenty-four-hour reporting

The Permittee must report the following occurrences of noncompliance by telephone, to Ecology at the telephone numbers listed above, within 24 hours from the time the Permittee becomes aware of any of the following circumstances:

1. Any noncompliance that may endanger health or the environment, unless previously reported under immediate reporting requirements.
2. Any unanticipated bypass that causes an exceedance of an effluent limit in the permit (See Part S5.F, “Bypass Procedures”).
3. Any upset that causes an exceedance of an effluent limit in the permit (See G.15, “Upset”).
4. Any violation of a maximum daily or instantaneous maximum discharge limit for any of the pollutants in Section S1.A of this permit.
5. Any overflow prior to the treatment works, whether or not such overflow endangers health or the environment or exceeds any effluent limit in the permit.

c. Report within five days

The Permittee must also submit a written report within five days of the time that the Permittee becomes aware of any reportable event under subparts a or b, above. The report must contain:

1. A description of the noncompliance and its cause.

2. The period of noncompliance, including exact dates and times.
3. The estimated time the Permittee expects the noncompliance to continue if not yet corrected.
4. Steps taken or planned to reduce, eliminate, and prevent recurrence of the noncompliance.
5. If the noncompliance involves an overflow prior to the treatment works, an estimate of the quantity (in gallons) of untreated overflow.

d. Waiver of written reports

Ecology may waive the written report required in subpart c, above, on a case-by-case basis upon request if the Permittee has submitted a timely oral report.

e. All other permit violation reporting

The Permittee must report all permit violations, which do not require immediate or within 24 hours reporting, when it submits monitoring reports for S3.A ("Reporting"). The reports must contain the information listed in subpart c, above. Compliance with these requirements does not relieve the Permittee from responsibility to maintain continuous compliance with the terms and conditions of this permit or the resulting liability for failure to comply.

S3.G. Other reporting

a. Spills of oil or hazardous materials

The Permittee must report a spill of oil or hazardous materials in accordance with the requirements of RCW 90.56.280 and chapter 173-303-145. You can obtain further instructions at the following website: https://ecology.wa.gov/About-us/Get-involved/Report-an-environmental-issue/Report-a-spill.

b. Failure to submit relevant or correct facts

Where the Permittee becomes aware that it failed to submit any relevant facts in a permit application, or submitted incorrect information in a permit application, or in any report to Ecology, it must submit such facts or information promptly.

S3.H. Maintaining a copy of this permit

The Permittee must keep a copy of this permit at the facility and make it available upon request to Ecology inspectors.

S4. Facility loading

S4.A. Design criteria

The flows or waste loads for the permitted facility must not exceed the following design criteria:

Maximum Month Design Flow (MMDF)	2.8 MGD
BOD_5 Influent Loading for Maximum Month	3,960 lb/day
TSS Influent Loading for Maximum Month	4,400 lb/day

S4.B. Plans for maintaining adequate capacity

a. Conditions triggering plan submittal

The Permittee must submit a plan and a schedule for continuing to maintain capacity to Ecology when:

1. The actual flow or waste load reaches 85 percent of any one of the design criteria in S4.A for three consecutive months.
2. The projected plant flow or loading would reach design capacity within five years.

b. Plan and schedule content

The plan and schedule must identify the actions necessary to maintain adequate capacity for the expected population growth and to meet the limits and requirements of the permit. The Permittee must consider the following topics and actions in its plan.

1. Analysis of the present design and proposed process modifications
2. Reduction or elimination of excessive infiltration and inflow of uncontaminated ground and surface water into the sewer system
3. Limits on future sewer extensions or connections or additional waste loads
4. Modification or expansion of facilities
5. Reduction of industrial or commercial flows or waste loads

Engineering documents associated with the plan must meet the requirements of WAC 173-240-060, "Engineering Report," and be approved by Ecology prior to any construction.

If the Permittee intends to apply for state or federal funding for the design or construction of a facility project, the plan may also need to meet the environmental review requirements as described in 40 CFR 35.3040 and 40 CFR 35.3045, and it may also need to demonstrate cost effectiveness as required by WAC 173-95-730. The plan must specify any contracts, ordinances, methods for financing, or other arrangements necessary to achieve this objective.

S4.C. Duty to mitigate

The Permittee must take all reasonable steps to minimize or prevent any discharge or sludge use or disposal in violation of this permit that has a reasonable likelihood of adversely affecting human health or the environment.

S4.D. Notification of new or altered sources

1. The Permittee must submit written notice to Ecology whenever any new discharge or a substantial change in volume or character of an existing discharge into the wastewater treatment plant is proposed which:

a. Would interfere with the operation of, or exceed the design capacity of, any portion of the wastewater treatment plant.

b. Is not part of an approved general sewer plan or approved plans and specifications.

c. Is subject to pretreatment standards under 40 CFR Part 403 and Section 307(b) of the Clean Water Act.

2. This notice must include an evaluation of the wastewater treatment plant's ability to adequately transport and treat the added flow and/or waste load, the quality and volume of effluent to be discharged to the treatment plant, and the anticipated impact on the Permittee's effluent [40 CFR 122.42(b)].

S4.E. Infiltration and inflow evaluation

1. The Permittee must conduct an infiltration and inflow evaluation. Refer to the U.S. EPA publication, I/I Analysis and Project Certification, available as Publication No. 97-03 at: https://fortress.wa.gov/ecy/publications/SummaryPages/9703.html

2. The Permittee may use monitoring records to assess measurable infiltration and inflow.

3. The Permittee must prepare a report summarizing any measurable infiltration and inflow. If infiltration and inflow have increased by more than 15 percent from that found in the previous report based on equivalent rainfall, the report must contain a plan and a schedule to locate the sources of infiltration and inflow and to correct the problem.

4. The Permittee must submit a report summarizing the results of the evaluation and any recommendations for corrective actions by June 30, 2019.

S5. Operation and maintenance

The Permittee must at all times properly operate and maintain all facilities and systems of treatment and control (and related appurtenances), which are installed to achieve compliance with the terms and conditions of this permit. Proper operation and maintenance also includes keeping a daily operation logbook (paper or electronic), adequate laboratory controls, and appropriate quality assurance procedures. This provision of the permit requires the Permittee to operate backup or auxiliary facilities or similar systems only when the operation is necessary to achieve compliance with the conditions of this permit.

S5.A. Certified operator

This permitted facility must be operated by an operator certified by the state of Washington for at least a Class II plant. This operator must be in responsible charge of the day-to-day operation of the wastewater treatment plant. An operator certified for at least a Class I plant must be in charge during all regularly scheduled shifts. The Permittee must notify Ecology when the operator in charge at the facility changes. It must provide the new operator's name and certification level and provide the name of the operator leaving the facility.

S5.B. Operation and maintenance program

The Permittee must:

1. Institute an adequate operation and maintenance program for the entire sewage system.
2. Keep maintenance records on all major electrical and mechanical components of the treatment plant, as well as the sewage system and pumping stations. Such records must clearly specify the frequency and type of maintenance recommended by the manufacturer and must show the frequency and type of maintenance performed.
3. Make maintenance records available for inspection at all times.

S5.C. Short-term reduction

The Permittee must schedule any facility maintenance, which might require interruption of wastewater treatment and degrade effluent quality, during non-critical water quality periods and carry this maintenance out according to the approved O&M manual or as otherwise approved by Ecology.

If a Permittee contemplates a reduction in the level of treatment that would cause a violation of permit discharge limits on a short-term basis for any reason, and such reduction cannot be avoided, the Permittee must:

1. Give written notification to Ecology, if possible, thirty (30) days prior to such activities.
2. Detail the reasons for, length of time of, and the potential effects of the reduced level of treatment.

This notification does not relieve the Permittee of its obligations under this permit.

S5.D. Electrical power failure

The Permittee must ensure that adequate safeguards prevent the discharge of untreated wastes or wastes not treated in accordance with the requirements of this permit during electrical power failure at the treatment plant and/or sewage lift stations. Adequate safeguards include, but are not limited to, alternate power sources, standby generator(s), or retention of inadequately treated wastes.

The Permittee must maintain Reliability Class II (EPA 430-99-74-001) at the wastewater treatment plant. Reliability Class II requires a backup power source sufficient to operate all vital components and critical lighting and ventilation during peak wastewater flow conditions. Vital components used to support the secondary processes (i.e., mechanical aerators or aeration basin air compressors) need not be operable to full levels of treatment, but must be sufficient to maintain the biota.

S5.E. Prevent connection of inflow

The Permittee must strictly enforce its sewer ordinances and not allow the connection of inflow (roof drains, foundation drains, etc.) to the sanitary sewer system.

S5.F. Bypass procedures

A bypass is the intentional diversion of waste streams from any portion of a treatment facility. This permit prohibits all bypasses except when the bypass is for essential maintenance, as authorized in special condition S5.F.1, or is approved by Ecology as an anticipated bypass following the procedures in S5.F.2.

1. Bypass for essential maintenance without the potential to cause violation of permit limits or conditions.

   This permit allows bypasses for essential maintenance of the treatment system when necessary to ensure efficient operation of the system. The Permittee may bypass the treatment system for essential maintenance only if doing so does not cause violations of effluent limits. The Permittee is not required to notify Ecology when bypassing for essential maintenance. However the Permittee must comply with the monitoring requirements specified in special condition S2.B.

2. Anticipated bypasses for non-essential maintenance

   Ecology may approve an anticipated bypass under the conditions listed below. This permit prohibits any anticipated bypass that is not approved through the following process.

   a. If a bypass is for non-essential maintenance, the Permittee must notify Ecology, if possible, at least ten (10) days before the planned date of bypass. The notice must contain:

      - A description of the bypass and the reason the bypass is necessary.
      - An analysis of all known alternatives which would eliminate, reduce, or mitigate the potential impacts from the proposed bypass.
      - A cost-effectiveness analysis of alternatives.
      - The minimum and maximum duration of bypass under each alternative.
      - A recommendation as to the preferred alternative for conducting the bypass.
      - The projected date of bypass initiation.
      - A statement of compliance with SEPA.
      - A request for modification of water quality standards as provided for in WAC 173-201A-410, if an exceedance of any water quality standard is anticipated.
      - Details of the steps taken or planned to reduce, eliminate, and prevent recurrence of the bypass.

b. For probable construction bypasses, the Permittee must notify Ecology of the need to bypass as early in the planning process as possible. The Permittee must consider the analysis required above during the project planning and design process. The project-specific engineering report as well as the plans and specifications must include details of probable construction bypasses to the extent practical. In cases where the Permittee determines the probable need to bypass early, the Permittee must continue to analyze conditions up to and including the construction period in an effort to minimize or eliminate the bypass.

c. Ecology will determine if the Permittee has met the conditions of special condition S5.F.2 a and b and consider the following prior to issuing a determination letter, an administrative order, or a permit modification as appropriate for an anticipated bypass:

- If the Permittee planned and scheduled the bypass to minimize adverse effects on the public and the environment.
- If the bypass is unavoidable to prevent loss of life, personal injury, or severe property damage. "Severe property damage" means substantial physical damage to property, damage to the treatment facilities which would cause them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of a bypass.
- If feasible alternatives to the bypass exist, such as:
  - The use of auxiliary treatment facilities.
  - Retention of untreated wastes.
  - Stopping production.
  - Maintenance during normal periods of equipment downtime, but not if the Permittee should have installed adequate backup equipment in the exercise of reasonable engineering judgment to prevent a bypass which occurred during normal periods of equipment downtime or preventative maintenance.
  - Transport of untreated wastes to another treatment facility.

S5.G. Operations and maintenance (O&M) manual

a. O&M manual submittal and requirements

The Permittee must:

1. Update the operations and maintenance (O&M) manual to incorporate the PAA disinfection system, if the Permittee decides to fully implement PAA as its primary method of disinfection. Submit the updated O&M manual to Ecology electronically and no later than May 1, 2022.

2. Ensure that the Operations and Maintenance (O&M) manual meets the requirements of 173-240-080 WAC.
3. Review the O&M manual at least annually.
4. Submit to Ecology for review and approval substantial changes or updates to the O&M manual whenever it incorporates them into the manual.
5. Keep the approved O&M manual at the permitted facility.
6. Follow the instructions and procedures of this manual.

b. O&M manual components

In addition to the requirements of WAC 173-240-080(1) through (5), the O&M manual must be consistent with the guidance in Table G1-3 in the *Criteria for Sewage Works Design* (Orange Book), 2008. The O&M manual must include:

1. Emergency procedures for cleanup in the event of wastewater system upset or failure.
2. A review of system components which if failed could pollute surface water or could impact human health. Provide a procedure for a routine schedule of checking the function of these components.
3. Wastewater system maintenance procedures that contribute to the generation of process wastewater.
4. Reporting protocols for submitting reports to Ecology to comply with the reporting requirements in the discharge permit.
5. Any directions to maintenance staff when cleaning or maintaining other equipment or performing other tasks which are necessary to protect the operation of the wastewater system (for example, defining maximum allowable discharge rate for draining a tank, blocking all floor drains before beginning the overhaul of a stationary engine).
6. The treatment plant process control monitoring schedule.
7. Minimum staffing adequate to operate and maintain the treatment processes and carry out compliance monitoring required by the permit.

S6. Pretreatment

S6.A. General requirements

The Permittee must work with Ecology to ensure that all commercial and industrial users of the publicly owned treatment works (POTW) comply with the pretreatment regulations in 40 CFR Part 403 and any additional regulations that the Environmental Protection Agency (U.S. EPA) may promulgate under Section 307(b) (pretreatment) and 308 (reporting) of the Federal Clean Water Act.

S6.B. Duty to enforce discharge prohibitions

1. Under federal regulations (40 CFR 403.5(a) and (b)), the Permittee must not authorize or knowingly allow the discharge of any pollutants into its POTW which may be reasonably expected to cause pass through or interference, or which otherwise violate general or specific discharge prohibitions contained in 40 CFR Part 403.5 or WAC 173-216-060.
2. The Permittee must not authorize or knowingly allow the introduction of any of the following into their treatment works:
    a. Pollutants which create a fire or explosion hazard in the POTW (including, but not limited to waste streams with a closed cup flashpoint of less than 140 degrees Fahrenheit or 60 degrees Centigrade using the test methods specified in 40 CFR 261.21).
    b. Pollutants which will cause corrosive structural damage to the POTW, but in no case discharges with pH lower than 5.0, or greater than 11.0 standard units, unless the works are specifically designed to accommodate such discharges.
    c. Solid or viscous pollutants in amounts that could cause obstruction to the flow in sewers or otherwise interfere with the operation of the POTW.
    d. Any pollutant, including oxygen-demanding pollutants, (BOD_5, etc.) released in a discharge at a flow rate and/or pollutant concentration which will cause interference with the POTW.
    e. Petroleum oil, non-biodegradable cutting oil, or products of mineral origin in amounts that will cause interference or pass through.
    f. Pollutants which result in the presence of toxic gases, vapors, or fumes within the POTW in a quantity which may cause acute worker health and safety problems.
    g. Heat in amounts that will inhibit biological activity in the POTW resulting in interference but in no case heat in such quantities such that the temperature at the POTW headworks exceeds 40 degrees Centigrade (104 degrees Fahrenheit) unless Ecology, upon request of the Permittee, approves, in writing, alternate temperature limits.
    h. Any trucked or hauled pollutants, except at discharge points designated by the Permittee.
    i. Wastewaters prohibited to be discharged to the POTW by the Dangerous Waste Regulations (chapter 173-303 WAC), unless authorized under the Domestic Sewage Exclusion (WAC 173-303-071).
3. The Permittee must also not allow the following discharges to the POTW unless approved in writing by Ecology:
    a. Noncontact cooling water in significant volumes.
    b. Stormwater and other direct inflow sources.

c. Wastewaters significantly affecting system hydraulic loading, which do not require treatment, or would not be afforded a significant degree of treatment by the system.

4. The Permittee must notify Ecology if any industrial user violates the prohibitions listed in this section (S6.B), and initiate enforcement action to promptly curtail any such discharge.

S6.C. Wastewater discharge permit required

The Permittee must:

1. Establish a process for authorizing non-domestic wastewater discharges that ensures all SIUs in all tributary areas meet the applicable state waste discharge permit (SWDP) requirements in accordance with chapter 90.48 RCW and chapter 173-216 WAC.
2. Immediately notify Ecology of any proposed discharge of wastewater from a source, which may be a significant industrial user (SIU) [see fact sheet definitions or refer to 40 CFR 403.3(v)(i)(ii)].
3. Require all SIUs to obtain a SWDP from Ecology prior to accepting their non-domestic wastewater, or require proof that Ecology has determined they do not require a permit.
4. Require the documentation as described in S6.C.3 at the earliest practicable date as a condition of continuing to accept non-domestic wastewater discharges from a previously undiscovered, currently discharging and unpermitted SIU.
5. Require sources of non-domestic wastewater, which do not qualify as SIUs but merit a degree of oversight, to apply for a SWDP and provide it a copy of the application and any Ecology responses.
6. Keep all records documenting that its users have met the requirements of S6.C.

S6.D. Identification and reporting of existing, new, and proposed industrial users

1. The Permittee must take continuous, routine measures to identify all existing, new, and proposed SIUs and potential significant industrial users (PSIUs) discharging or proposing to discharge to the Permittee's sewer system (see *Appendix C* of the fact sheet for definitions).
2. Within 30 days of becoming aware of an unpermitted existing, new, or proposed industrial user who may be a significant industrial user (SIU), the Permittee must notify such user by registered mail that, if classified as an SIU, they must apply to Ecology and obtain a State Waste Discharge Permit. The Permittee must send a copy of this notification letter to Ecology within this same 30-day period.
3. The Permittee must also notify all Potential SIUs (PSIUs), as they are identified, that if their classification should change to an SIU, they must apply to Ecology for a State Waste Discharge Permit within 30 days of such change.

S6.E. Industrial user survey

The Permittee must complete an industrial user survey listing all SIUs and potential significant industrial users (PSIUs) discharging to the POTW. The Permittee must submit the survey to Ecology by May 1, 2022. At a minimum, the Permittee must develop the list of SIUs and PSIUs by means of a telephone book search, a water utility billing records search, and a physical reconnaissance of the service area. Information on PSIUs must include, at a minimum, the business name, telephone number, address, description of the industrial process(s), and the known wastewater volumes and characteristics.

S7. Solid wastes

S7.A. Solid waste handling

The Permittee must handle and dispose of all solid waste material in such a manner as to prevent its entry into state ground or surface water.

S7.B. Leachate

The Permittee must not allow leachate from its solid waste material to enter state waters without providing all known, available, and reasonable methods of treatment, nor allow such leachate to cause violations of the State Surface Water Quality Standards, Chapter 173-201A WAC, or the State Ground Water Quality Standards, Chapter 173-200 WAC. The Permittee must apply for a permit or permit modification as may be required for such discharges to state ground or surface waters.

S8. Compliance schedule

Upon completion of the peracetic acid (PAA) full-scale pilot test on April 10, 2018, if the Permittee decides to fully implement PAA as its primary method of disinfection, the Permittee must:

1. Submit an Engineering Report, including the results of the pilot test, the long-term feasibility assessment of using PAA as disinfection process, and the basis for the design of the PAA disinfection system by July 31, 2018.
2. Submit detailed Plans and Specifications of the PAA disinfection system by May 31, 2019. Complete construction of PAA disinfection system by August 1, 2020.
3. The Engineering Report and the Plans and Specifications must comply with the requirements of the WAC 173-240-060 and WAC 173-240-070, respectively.

S9. Mixing study

S9.A. General requirements

The Permittee must

1. Update the Effluent Mixing Zone Study (Cosmopolitan, 2000). Submit a Plan of Study to Ecology for review by August 1, 2020, prior to initiation of the effluent mixing study.

2. Use the Guidance for Conducting Mixing Zone Analyses (Appendix C of Ecology's *Permit Writer's Manual*, 2015) and the protocols identified in S8.C.
3. Include the results of the effluent mixing study in the Effluent Mixing Report and submit it to Ecology for approval by August 1, 2021.
4. If the results of the mixing study, toxicity tests, and chemical analysis indicate that the concentration of any pollutant(s) exceeds or has a reasonable potential to exceed the state water quality standards, chapter 173-201A WAC, Ecology may modify this permit to impose effluent limits to meet the water quality standards.

S9.B. Reporting requirements

The mixing zone study must include:

1. A statement confirming that AKART has been applied to the discharge.
2. A description of the size of the mixing zone allowed under WAC 173-201A.
3. An analysis showing how mixing zones have been minimized based on using the lowest dilution from hydraulic limitation, width limitations, distance limitation and that predicted by the model.
4. A clear description of the critical conditions used for dilution factors:
    a. For ambient freshwater (unidirectional flow) use 7Q10 flows for acute, chronic and non-carcinogen pollutants, and harmonic flow for carcinogens.
    b. For ambient marine waters (and reversing flows e.g., tidally-influenced rivers) use 10th or 90th percentile current velocity for acute and 50th percentile tidal current velocity for chronic, carcinogens and non-carcinogens.
    c. Generally, use depth of outfall at 7Q10 flows (rivers) or at MLLW (marine environment). For assessing human health in freshwater, depths of outfall should be established at the applicable flow (e.g. harmonic mean flow or 30Q5 flows). For tidally influenced rivers a combination of MLLW and critical river flows should be used to establish depth of outfall.
    d. Use density profile that gives the lowest dilution. Valuate both maximum and minimum stratification. For human health, use average density profiles to estimate dilution.
    e. For unidirectional flow use centerline dilution factor for acute and chronic conditions, while flux average for human health dilution factors. For marine environment or rivers with reversing flows, use flux-average dilution factors for all conditions.
5. Diffuser information:
    a. Location, orientation, description and dimension of diffusers and ports.

b. Port elevation above bottom and the depth of the diffuser/port below water surface based on either 7Q10 flow (for rivers) or MLLW (for marine or tidally-influenced river reaches).

c. Plan view maps showing the mixing zone size and dimensions in relation to the diffuser.

d. Schematic of waterbody cross-section, showing channel width, depth, and diffuser location in relation to shoreline and bottom.

e. Report on the integrity of the diffuser and the ports being modeled.

6. Discharge characteristics:

   a. Existing and projected maximum daily, maximum monthly average, and annual average flows.

   b. Discharge density (temperature and salinity).

7. Ambient water characteristics:

   a. Critical stream flow statistics (7Q10, 30Q5, harmonic flow) or marine current velocities (10^{th}, 90^{th}, and 50^{th} percentiles over a neap and spring tide and directions).

   b. Velocity profile in the vicinity of the diffuser.

   c. Temporal density (temperature and salinity) profiles near the diffuser. May need to consider both seasonal and tidal variability.

   d. Manning's roughness coefficient, if used.

   e. Available information regarding background concentrations of chemical substances in the receiving water (for which there are criteria in chapter 173-201A WAC.

8. Model selection and results:

   a. Model selection and application discussion. Consider model applicability to single or multiport diffuser, opposing port configuration, submerged, surface or above-surface discharge, buoyant or non-buoyant discharge, and potential plume attachment to boundaries.

   b. Description of mixing and plume dynamics (nearfield, farfield, tidal buildup/reflux).

   c. Sensitivity analysis.

   d. Calibration to empirical data (tracer studies), if applicable.

   e. Provide model output and summary table of results.

S9.C. Protocols

The Permittee must determine the dilution ratio using protocols outlined in the following references, approved modifications thereof, or by another method approved by Ecology:

1. Doneker, R.L. and G.H. Jirka, *CORMIX User Manual: A Hydrodynamic Mixing Zone Model and Decision Support System for Pollutant Discharges into Surface Waters,* EPA-823-K-07-001, Dec. 2007. http://www.mixzon.com/downloads/.

   A complete list of general reference for CORMIX is at: http://www.cormix.info/references.php

2. Frick, W.E., Roberts, P.J.W., Davis, L.R., Keyes, D.J., Baumgartner, George, K.P. 2003. *Dilution Models for Effluent Discharges, 4th Edition (Visual Plumes).* Ecosystems Research Div., USEPA, Athens, GA, USA.

3. Ecology, Water Quality Program, *Permit Writer's Manual*. 2015. Washington State Department of Ecology. Publication No. 92-109, Revised January 2015. https://fortress.wa.gov/ecy/publications/summarypages/92109.html.

4. Ecology, Guidance for conducting mixing zone analysis (Appendix C, Water Quality Program *Permit Writer's Manual*. 2015). https://fortress.wa.gov/ecy/publications/summarypages/92109.html.

5. Kilpatrick, F.A., and E.D. Cobb, *Measurement of Discharge Using Tracers, Chapter A16, Techniques of Water-Resources Investigations of the USGS*, Book 3, Application of Hydraulics, USGS, U.S. Department of the Interior, Reston, VA, 1985.

6. Wilson, J.F., E.D. Cobb, and F.A. Kilpatrick, *Fluorometric Procedures for Dye Tracing, Chapter A12. Techniques of Water-Resources Investigations of the USGS*, Book 3, Application of Hydraulics, USGS, U.S. Department of the Interior, Reston, VA, 1986.

S10. Combined sewer overflows

S10.A. Authorized combined sewer overflow (CSO) discharge locations

Beginning on the effective date of this permit, the Permittee may discharge domestic wastewater from the following list of combined sewer overflow (CSOs) outfalls which, represent occasional point sources of pollutants as a result of overloading of the combined sewer system during precipitation events. The permit prohibits discharges not caused by precipitation. This permit does not authorize a discharge from a CSO that causes adverse impacts that threaten characteristic uses of the receiving water as identified in the water quality standards, chapter 173-201A WAC.

Outfall ID	Location	Receiving Water Body
Outfall #002 (CSO #1)	Latitude: 47.912119 Longitude: -122.104180	Snohomish River
Outfall #003 (CSO #2)	Latitude: 47.911389 Longitude: -122.100000	Snohomish River

S10.B. Nine minimum controls

In accordance with chapter 173-245 WAC and US EPA CSO control policy (59 FR 18688), the Permittee must implement and document the following nine minimum controls (NMC) for CSOs. The Permittee must document compliance with the NMC in the annual CSO report as required in Special Condition S10.C.

The Permittee must comply with the following technology-based requirements; the Permittee must:

1. Implement proper operation and maintenance programs for the sewer system and all CSO outfalls to reduce the magnitude, frequency, and duration of CSOs. The program must consider regular sewer inspections; sewer, catch basin, and regulator cleaning; equipment and sewer collection system repair or replacement, where necessary; and disconnection of illegal connections.
2. Implement procedures that will maximize use of the collection system for wastewater storage that can be accommodated by the storage capacity of the collection system in order to reduce the magnitude, frequency, and duration of CSOs.
3. Review and modify, as appropriate, its existing pretreatment program to minimize CSO impacts from the discharges from non-domestic users.
4. Operate the Permittee's wastewater treatment plant at maximum treatable flow during all wet weather flow conditions to reduce the magnitude, frequency, and duration of CSOs. The Permittee must deliver all flows to the treatment plant within the constraints of the treatment capacity of the POTW.
5. Not discharge (prohibited) overflows from CSO outfalls except as a result of precipitation events. The Permittee must report each dry weather overflow to the permitting authority immediately per Special Condition S3.E. When it detects a dry weather overflow, the Permittee must begin corrective action immediately and inspect the dry weather overflow each subsequent day until it has eliminated the overflow.
6. Implement measures to control solid and floatable materials in CSOs.
7. Implement a pollution prevention program focused on reducing the impact of CSOs on receiving waters.
8. Implement a public notification process to inform the citizens of when and where CSOs occur. The process must include (a) mechanism to alert persons of the occurrence of CSOs and (b) a system to determine the nature and duration of conditions that are potentially harmful for users of receiving waters due to CSOs.
9. Monitor CSO outfalls to characterize CSO impacts and the efficacy of CSO controls. This must include collection of data that it will use to document the existing baseline conditions, evaluate the efficacy of the technology-based controls, and determine the baseline conditions upon which it will base the long-term control plan. This data must include:
    a. Characteristics of the combined sewer system, including the population served by the combined portion of the system and locations of all CSO outfalls in the CSS.
    b. Total number of CSO events, and the frequency and duration of CSOs for a representative number of events.
    c. Locations and designated uses of receiving water bodies.

d. Water quality data for receiving water bodies.

e. Water quality impacts directly related to CSO (e.g., beach closing, floatables, wash-up episodes, fish kills).

S10.C. Combined sewer overflow annual report

The Permittee must submit a CSO Annual Report to Ecology for review and approval by April 15th of each year. The CSO Annual Report must cover the previous calendar year (January – December). The report must comply with the requirements of WAC 173-245-090(1). It must include documentation of compliance with the Nine Minimum Controls for CSOs described in Special Condition S10.B and summarize the results of post construction monitoring required in Special Condition S10.E. The CSO Annual report must include the following information:

1. A summary of the number and volume of untreated discharge events per outfall for that year.
2. A summary of the 5-year moving average number of untreated discharge events per outfall, calculated once annually.
3. A summary of each event-based reporting form (provided by Ecology through WQWebDMR) for all CSO discharges for the reporting period, summarizing all data collected according to the monitoring schedule in Special Condition S2.B.
4. An explanation of the previous year's CSO reduction accomplishments, if any.
5. A list of CSO reduction projects planned for the next year, if any.

S10.D. Requirements for controlled combined sewer overflows

a. CSOs identified as controlled

Based on monitoring data, the CSO outfalls listed in S10.A meet the requirement of "greatest reasonable reduction" as defined in chapter WAC 173-245-020(22). Frequency of overflow events at these CSO outfalls, as a result of precipitation events, must continue to meet the performance standard.

b. Performance standards for controlled CSO outfalls

The performance standard for each controlled CSO outfall is not more than one discharge event per outfall per year on average, due to precipitation. Ecology evaluates compliance with the performance standard annually based on a 5-year moving average. The Permittee must report the running 5-year average number of overflow events per year during this permit term from these CSO outfalls in the CSO Annual Report required in Section S10.C.

c. CSO post construction monitoring

The Permittee must implement its post construction compliance monitoring program, as approved in 2015, to verify the effectiveness of CSO controls and to demonstrate compliance with water quality standards and protection of designated uses. The Permittee must include data collected under this program in its annual report submitted under special condition S10.C.

S11. Acute toxicity

S11.A. Testing when there is no permit limit for acute toxicity

The Permittee must:

1. Conduct acute toxicity testing on final effluent once in the last winter (February 2021) and once in the last summer (August 2021) prior to submission of the application for permit renewal.
2. Conduct acute toxicity testing on a series of at least five concentrations of effluent, including 100% effluent and a control.
3. Use each of the following species and protocols for each acute toxicity test:

Acute Toxicity Tests	Species	Method
Fathead minnow 96-hour static-renewal test	*Pimephales promelas*	EPA-821-R-02-012
Daphnid 48-hour static test	*Ceriodaphnia dubia, Daphnia pulex,* or *Daphnia magna*	EPA-821-R-02-012

4. Submit the results to Ecology electronically by April 30, 2021 and October 31, 2021, respectively.

S11.B. Sampling and reporting requirements

1. The Permittee must submit all reports for toxicity testing in accordance with the most recent version of Ecology Publication No. WQ-R-95-80, *Laboratory Guidance and Whole Effluent Toxicity Test Review Criteria*. Reports must contain toxicity data, bench sheets, and reference toxicant results for test methods. In addition, the Permittee must submit toxicity test data in electronic format (CETIS export file preferred) for entry into Ecology's database.
2. The Permittee must collect 24-hour composite effluent samples for toxicity testing. The Permittee must cool the samples to 0 - 6 degrees Celsius during collection and send them to the lab immediately upon completion. The lab must begin the toxicity testing as soon as possible but no later than 36 hours after sampling was completed.
3. The laboratory must conduct water quality measurements on all samples and test solutions for toxicity testing, as specified in the most recent version of Ecology Publication No. WQ-R-95-80, *Laboratory Guidance and Whole Effluent Toxicity Test Review Criteria*.
4. All toxicity tests must meet quality assurance criteria and test conditions specified in the most recent versions of the EPA methods listed in Subsection C and the Ecology Publication No. WQ-R-95-80, *Laboratory Guidance and Whole Effluent Toxicity Test Review Criteria*. If Ecology determines any test results to be invalid or anomalous, the Permittee must repeat the testing with freshly collected effluent.
5. The laboratory must use control water and dilution water meeting the requirements of the EPA methods listed in Section A or pristine natural water of sufficient quality for good control performance.

6. The Permittee must conduct whole effluent toxicity tests on an unmodified sample of final effluent.
7. The Permittee must chemically dechlorinate final effluent samples for whole effluent toxicity testing with sodium thiosulfate just prior to test initiation. Do not add more sodium thiosulfate than is necessary to neutralize the chlorine. Provide in the test report the calculations to determine the amount of sodium thiosulfate necessary to just neutralize the chlorine in the sample.
8. The Permittee may choose to conduct a full dilution series test during compliance testing in order to determine dose response. In this case, the series must have a minimum of five effluent concentrations and a control. The series of concentrations must include the acute critical effluent concentration (ACEC). The ACEC equals 9.1% effluent.
9. All whole effluent toxicity tests, effluent screening tests, and rapid screening tests that involve hypothesis testing must comply with the acute statistical power standard of 29% as defined in WAC 173-205-020. If the test does not meet the power standard, the Permittee must repeat the test on a fresh sample with an increased number of replicates to increase the power.

S12. Chronic toxicity

S12.A. Testing when there is no permit limit for chronic toxicity

The Permittee must:

1. Conduct chronic toxicity testing on final effluent once in the last winter (February 2021) and once in the last summer (August 2021) prior to submission of the application for permit renewal.
2. Conduct chronic toxicity testing on a series of at least five concentrations of effluent and a control. This series of dilutions must include the acute critical effluent concentration (ACEC). The ACEC equals 9.1% effluent. The series of dilutions should also contain the CCEC of 3.6% effluent.
3. Compare the ACEC to the control using hypothesis testing at the 0.05 level of significance as described in Appendix H, EPA/600/4-89/001.
4. Submit the results to Ecology by April 30, 2021 and October 31, 2021, respectively.
5. Perform chronic toxicity tests with all of the following species and the most recent version of the following protocols:

Freshwater Chronic Test	Species	Method
Fathead minnow survival and growth	*Pimephales promelas*	EPA-821-R-02-013
Water flea survival and reproduction	*Ceriodaphnia dubia*	EPA-821-R-02-013

S12.B. Sampling and reporting requirements

1. The Permittee must submit all reports for toxicity testing in accordance with the most recent version of Ecology Publication No. WQ-R-95-80, *Laboratory Guidance and Whole Effluent Toxicity Test Review Criteria*. Reports must contain toxicity data, bench sheets, and reference toxicant results for test methods. In addition, the Permittee must submit toxicity test data in electronic format (CETIS export file preferred) for entry into Ecology's database.
2. The Permittee must collect 24-hour composite effluent samples for toxicity testing. The Permittee must cool the samples to 0 - 6 degrees Celsius during collection and send them to the lab immediately upon completion. The lab must begin the toxicity testing as soon as possible but no later than 36 hours after sampling was completed.
3. The laboratory must conduct water quality measurements on all samples and test solutions for toxicity testing, as specified in the most recent version of Ecology Publication No. WQ-R-95-80, *Laboratory Guidance and Whole Effluent Toxicity Test Review Criteria*.
4. All toxicity tests must meet quality assurance criteria and test conditions specified in the most recent versions of the EPA methods listed in Section C and the Ecology Publication no. WQ-R-95-80, *Laboratory Guidance and Whole Effluent Toxicity Test Review Criteria*. If Ecology determines any test results to be invalid or anomalous, the Permittee must repeat the testing with freshly collected effluent.
5. The laboratory must use control water and dilution water meeting the requirements of the EPA methods listed in Subsection C or pristine natural water of sufficient quality for good control performance.
6. The Permittee must conduct whole effluent toxicity tests on an unmodified sample of final effluent.
7. The Permittee must chemically dechlorinate final effluent samples for whole effluent toxicity testing with sodium thiosulfate just prior to test initiation. Do not add more sodium thiosulfate than is necessary to neutralize the chlorine. Provide in the test report the calculations to determine the amount of sodium thiosulfate necessary to just neutralize the chlorine in the sample.
8. The Permittee may choose to conduct a full dilution series test during compliance testing in order to determine dose response. In this case, the series must have a minimum of five effluent concentrations and a control. The series of concentrations must include the CCEC and the ACEC. The CCEC and the ACEC may either substitute for the effluent concentrations that are closest to them in the dilution series or be extra effluent concentrations. The CCEC equals 3.6% effluent. The ACEC equals 9.1% effluent.
9. All whole effluent toxicity tests that involve hypothesis testing must comply with the chronic statistical power standard of 39% as defined in WAC 173-205-020. If the test does not meet the power standard, the Permittee must repeat the test on a fresh sample with an increased number of replicates to increase the power.

S13. Application for permit renewal or modification for facility changes

The Permittee must submit an application for renewal of this permit by January 1, 2023. The Permittee must submit a paper copy and an electronic copy (preferably as a PDF).

The Permittee must also submit a new application or addendum at least one hundred eighty (180) days prior to commencement of discharges, resulting from the activities listed below, which may result in permit violations. These activities include any facility expansions, production increases, or other planned changes, such as process modifications, in the permitted facility.

General Conditions

G1. Signatory requirements

1. All applications submitted to Ecology must be signed and certified.

   a. In the case of corporations, by a responsible corporate officer. For the purpose of this section, a responsible corporate officer means:

      - A president, secretary, treasurer, or vice-president of the corporation in charge of a principal business function, or any other person who performs similar policy or decision making functions for the corporation, or

      - The manager of one or more manufacturing, production, or operating facilities, provided, the manager is authorized to make management decisions which govern the operation of the regulated facility including having the explicit or implicit duty of making major capital investment recommendations, and initiating and directing other comprehensive measures to assure long-term environmental compliance with environmental laws and regulations; the manager can ensure that the necessary systems are established or actions taken to gather complete and accurate information for permit application requirements; and where authority to sign documents has been assigned or delegated to the manager in accordance with corporate procedures.

   b. In the case of a partnership, by a general partner.

   c. In the case of sole proprietorship, by the proprietor.

   d. In the case of a municipal, state, or other public facility, by either a principal executive officer or ranking elected official.

   Applications for permits for domestic wastewater facilities that are either owned or operated by, or under contract to, a public entity shall be submitted by the public entity.

2. All reports required by this permit and other information requested by Ecology must be signed by a person described above or by a duly authorized representative of that person. A person is a duly authorized representative only if:

   a. The authorization is made in writing by a person described above and submitted to Ecology.

   b. The authorization specifies either an individual or a position having responsibility for the overall operation of the regulated facility, such as the position of plant manager, superintendent, position of equivalent responsibility, or an individual or position having overall responsibility for environmental matters. (A duly authorized representative may thus be either a named individual or any individual occupying a named position.)

3. Changes to authorization. If an authorization under paragraph G1.2, above, is no longer accurate because a different individual or position has responsibility for the overall operation of the facility, a new authorization satisfying the requirements of paragraph G1.2, above, must be submitted to Ecology prior to or together with any reports, information, or applications to be signed by an authorized representative.

4. Certification. Any person signing a document under this section must make the following certification:

   "I certify under penalty of law, that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gathered and evaluated the information submitted. Based on my inquiry of the person or persons who manage the system or those persons directly responsible for gathering information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations."

G2. Right of inspection and entry

The Permittee must allow an authorized representative of Ecology, upon the presentation of credentials and such other documents as may be required by law:

1. To enter upon the premises where a discharge is located or where any records must be kept under the terms and conditions of this permit.
2. To have access to and copy, at reasonable times and at reasonable cost, any records required to be kept under the terms and conditions of this permit.
3. To inspect, at reasonable times, any facilities, equipment (including monitoring and control equipment), practices, methods, or operations regulated or required under this permit.
4. To sample or monitor, at reasonable times, any substances or parameters at any location for purposes of assuring permit compliance or as otherwise authorized by the Clean Water Act.

G3. Permit actions

This permit may be modified, revoked and reissued, or terminated either at the request of any interested person (including the Permittee) or upon Ecology's initiative. However, the permit may only be modified, revoked and reissued, or terminated for the reasons specified in 40 CFR 122.62, 40 CFR 122.64 or WAC 173-220-150 according to the procedures of 40 CFR 124.5.

1. The following are causes for terminating this permit during its term, or for denying a permit renewal application:
   a. Violation of any permit term or condition.
   b. Obtaining a permit by misrepresentation or failure to disclose all relevant facts.
   c. A material change in quantity or type of waste disposal.
   d. A determination that the permitted activity endangers human health or the environment, or contributes to water quality standards violations and can only be regulated to acceptable levels by permit modification or termination.

e. A change in any condition that requires either a temporary or permanent reduction, or elimination of any discharge or sludge use or disposal practice controlled by the permit.

f. Nonpayment of fees assessed pursuant to RCW 90.48.465.

g. Failure or refusal of the Permittee to allow entry as required in RCW 90.48.090.

2. The following are causes for modification but not revocation and reissuance except when the Permittee requests or agrees:

a. A material change in the condition of the waters of the state.

b. New information not available at the time of permit issuance that would have justified the application of different permit conditions.

c. Material and substantial alterations or additions to the permitted facility or activities which occurred after this permit issuance.

d. Promulgation of new or amended standards or regulations having a direct bearing upon permit conditions, or requiring permit revision.

e. The Permittee has requested a modification based on other rationale meeting the criteria of 40 CFR Part 122.62.

f. Ecology has determined that good cause exists for modification of a compliance schedule, and the modification will not violate statutory deadlines.

g. Incorporation of an approved local pretreatment program into a municipality's permit.

3. The following are causes for modification or alternatively revocation and reissuance:

a. When cause exists for termination for reasons listed in 1.a through 1.g of this section, and Ecology determines that modification or revocation and reissuance is appropriate.

b. When Ecology has received notification of a proposed transfer of the permit. A permit may also be modified to reflect a transfer after the effective date of an automatic transfer (General Condition G7) but will not be revoked and reissued after the effective date of the transfer except upon the request of the new Permittee.

G4. Reporting planned changes

The Permittee must, as soon as possible, but no later than one hundred eighty (180) days prior to the proposed changes, give notice to Ecology of planned physical alterations or additions to the permitted facility, production increases, or process modification which will result in:

1. The permitted facility being determined to be a new source pursuant to 40 CFR 122.29(b).

2. A significant change in the nature or an increase in quantity of pollutants discharged.

3. A significant change in the Permittee's sludge use or disposal practices. Following such notice, and the submittal of a new application or supplement to the existing application, along with required engineering plans and reports, this permit may be

modified, or revoked and reissued pursuant to 40 CFR 122.62(a) to specify and limit any pollutants not previously limited. Until such modification is effective, any new or increased discharge in excess of permit limits or not specifically authorized by this permit constitutes a violation.

G5. Plan review required

Prior to constructing or modifying any wastewater control facilities, an engineering report and detailed plans and specifications must be submitted to Ecology for approval in accordance with chapter 173-240 WAC. Engineering reports, plans, and specifications must be submitted at least one hundred eighty (180) days prior to the planned start of construction unless a shorter time is approved by Ecology. Facilities must be constructed and operated in accordance with the approved plans.

G6. Compliance with other laws and statutes

Nothing in this permit excuses the Permittee from compliance with any applicable federal, state, or local statutes, ordinances, or regulations.

G7. Transfer of this permit

In the event of any change in control or ownership of facilities from which the authorized discharge emanate, the Permittee must notify the succeeding owner or controller of the existence of this permit by letter, a copy of which must be forwarded to Ecology.

1. Transfers by Modification

   Except as provided in paragraph (2) below, this permit may be transferred by the Permittee to a new owner or operator only if this permit has been modified or revoked and reissued under 40 CFR 122.62(b)(2), or a minor modification made under 40 CFR 122.63(d), to identify the new Permittee and incorporate such other requirements as may be necessary under the Clean Water Act.

2. Automatic Transfers

   This permit may be automatically transferred to a new Permittee if:

   a. The Permittee notifies Ecology at least thirty (30) days in advance of the proposed transfer date.

   b. The notice includes a written agreement between the existing and new Permittees containing a specific date transfer of permit responsibility, coverage, and liability between them.

   c. Ecology does not notify the existing Permittee and the proposed new Permittee of its intent to modify or revoke and reissue this permit. A modification under this subparagraph may also be minor modification under 40 CFR 122.63. If this notice is not received, the transfer is effective on the date specified in the written agreement.

G8. Reduced production for compliance

The Permittee, in order to maintain compliance with its permit, must control production and/or all discharges upon reduction, loss, failure, or bypass of the treatment facility until the facility is restored or an alternative method of treatment is provided. This requirement applies in the situation where, among other things, the primary source of power of the treatment facility is reduced, lost, or fails.

G9. Removed substances

Collected screenings, grit, solids, sludges, filter backwash, or other pollutants removed in the course of treatment or control of wastewaters must not be resuspended or reintroduced to the final effluent stream for discharge to state waters.

G10. Duty to provide information

The Permittee must submit to Ecology, within a reasonable time, all information which Ecology may request to determine whether cause exists for modifying, revoking and reissuing, or terminating this permit or to determine compliance with this permit. The Permittee must also submit to Ecology upon request, copies of records required to be kept by this permit.

G11. Other requirements of 40 CFR

All other requirements of 40 CFR 122.41 and 122.42 are incorporated in this permit by reference.

G12. Additional monitoring

Ecology may establish specific monitoring requirements in addition to those contained in this permit by administrative order or permit modification.

G13. Payment of fees

The Permittee must submit payment of fees associated with this permit as assessed by Ecology.

G14. Penalties for violating permit conditions

Any person who is found guilty of willfully violating the terms and conditions of this permit is deemed guilty of a crime, and upon conviction thereof shall be punished by a fine of up to ten thousand dollars ($10,000) and costs of prosecution, or by imprisonment in the discretion of the court. Each day upon which a willful violation occurs may be deemed a separate and additional violation.

Any person who violates the terms and conditions of a waste discharge permit may incur, in addition to any other penalty as provided by law, a civil penalty in the amount of up to ten thousand dollars ($10,000) for every such violation. Each and every such violation is a separate and distinct offense, and in case of a continuing violation, every day's continuance is deemed to be a separate and distinct violation.

G15. Upset

Definition – "Upset" means an exceptional incident in which there is unintentional and temporary noncompliance with technology-based permit effluent limits because of factors beyond the reasonable control of the Permittee. An upset does not include noncompliance to the extent caused by operational error, improperly designed treatment facilities, inadequate treatment facilities, lack of preventive maintenance, or careless or improper operation.

An upset constitutes an affirmative defense to an action brought for noncompliance with such technology-based permit effluent limits if the requirements of the following paragraph are met.

A Permittee who wishes to establish the affirmative defense of upset must demonstrate, through properly signed, contemporaneous operating logs, or other relevant evidence that:

1. An upset occurred and that the Permittee can identify the cause(s) of the upset.
2. The permitted facility was being properly operated at the time of the upset.
3. The Permittee submitted notice of the upset as required in Special Condition S3.F.
4. The Permittee complied with any remedial measures required under S3.F of this permit.

In any enforcement action the Permittee seeking to establish the occurrence of an upset has the burden of proof.

G16. Property rights

This permit does not convey any property rights of any sort, or any exclusive privilege.

G17. Duty to comply

The Permittee must comply with all conditions of this permit. Any permit noncompliance constitutes a violation of the Clean Water Act and is grounds for enforcement action; for permit termination, revocation and reissuance, or modification; or denial of a permit renewal application.

G18. Toxic pollutants

The Permittee must comply with effluent standards or prohibitions established under Section 307(a) of the Clean Water Act for toxic pollutants within the time provided in the regulations that establish those standards or prohibitions, even if this permit has not yet been modified to incorporate the requirement.

G19. Penalties for tampering

The Clean Water Act provides that any person who falsifies, tampers with, or knowingly renders inaccurate any monitoring device or method required to be maintained under this permit shall, upon conviction, be punished by a fine of not more than $10,000 per violation, or by imprisonment for not more than two (2) years per violation, or by both.

If a conviction of a person is for a violation committed after a first conviction of such person under this condition, punishment shall be a fine of not more than $20,000 per day of violation, or by imprisonment of not more than four (4) years, or by both.

G20. Compliance schedules

Reports of compliance or noncompliance with, or any progress reports on, interim and final requirements contained in any compliance schedule of this permit must be submitted no later than fourteen (14) days following each schedule date.

G21. Service agreement review

The Permittee must submit to Ecology any proposed service agreements and proposed revisions or updates to existing agreements for the operation of any wastewater treatment facility covered by this permit. The review is to ensure consistency with chapters 90.46 and 90.48 RCW as required by RCW 70.150.040(9). In the event that Ecology does not comment within a thirty-day (30) period, the Permittee may assume consistency and proceed with the service agreement or the revised/updated service agreement.

Appendix A

LIST OF POLLUTANTS WITH ANALYTICAL METHODS, DETECTION LIMITS AND QUANTITATION LEVELS

The Permittee must use the specified analytical methods, detection limits (DLs) and quantitation levels (QLs) in the following table for permit and application required monitoring unless:

- Another permit condition specifies other methods, detection levels, or quantitation levels.
- The method used produces measurable results in the sample and EPA has listed it as an EPA-approved method in 40 CFR Part 136.

If the Permittee uses an alternative method, not specified in the permit and as allowed above, it must report the test method, DL, and QL on the discharge monitoring report or in the required report.

If the Permittee is unable to obtain the required DL and QL in its effluent due to matrix effects, the Permittee must submit a matrix-specific detection limit (MDL) and a quantitation limit (QL) to Ecology with appropriate laboratory documentation.

When the permit requires the Permittee to measure the base neutral compounds in the list of priority pollutants, it must measure all of the base neutral pollutants listed in the table below. The list includes EPA required base neutral priority pollutants and several additional polynuclear aromatic hydrocarbons (PAHs). The Water Quality Program added several PAHs to the list of base neutrals below from Ecology's Persistent Bioaccumulative Toxics (PBT) List. It only added those PBT parameters of interest to Appendix A that did not increase the overall cost of analysis unreasonably.

Ecology added this appendix to the permit in order to reduce the number of analytical "non-detects" in permit-required monitoring and to measure effluent concentrations near or below criteria values where possible at a reasonable cost.

The lists below include conventional pollutants (as defined in CWA section 502(6) and 40 CFR Part 122.), toxic or priority pollutants as defined in CWA section 307(a)(1) and listed in 40 CFR Part 122 Appendix D, 40 CFR Part 401.15 and 40 CFR Part 423 Appendix A), and nonconventionals. 40 CFR Part 122 Appendix D (Table V) also identifies toxic pollutants and hazardous substances which are required to be reported by dischargers if expected to be present. This permit Appendix A list does not include those parameters. The list also includes pulp and paper pollutants identified in 40 CFR Part 430 and the dioxin and furan congeners identified using EPA Method 1613.

CONVENTIONAL POLLUTANTS

Pollutant	CAS Number (if available)	Recommended Analytical Protocol	Detection (DL)[1] *µg/L unless specified*	Quantitation Level (QL)[2] *µg/L unless specified*
Biochemical Oxygen Demand		SM5210-B		2 mg/L
Biochemical Oxygen Demand, Soluble		SM5210-B [3]		2 mg/L
Fecal Coliform		SM 9221E,9222	N/A	Specified in method - sample aliquot dependent
Oil and Grease (HEM) (Hexane Extractable Material)		1664 A or B	1,400	5,000
pH		SM4500-H^+ B	N/A	N/A
Total Suspended Solids		SM2540-D		5 mg/L

NONCONVENTIONAL POLLUTANTS

Pollutant & CAS No. (if available)	CAS Number (if available)	Recommended Analytical Protocol	Detection (DL)[1] µg/L unless specified	Quantitation Level (QL)[2] µg/L unless specified
Alkalinity, Total		SM2320-B		5 mg/L as CaCO3
Aluminum, Total	7429-90-5	200.8	2.0	10
Ammonia, Total (as N)		SM4500-NH3-B and C/D/E/G/H		20
Barium Total	7440-39-3	200.8	0.5	2.0
BTEX (benzene +toluene + ethylbenzene + m,o,p xylenes)		EPA SW 846 8021/8260	1	2
Boron, Total	7440-42-8	200.8	2.0	10.0
Chemical Oxygen Demand		SM5220-D		10 mg/L
Chloride		SM4500-Cl B/C/D/E and SM4110 B		Sample and limit dependent
Chlorine, Total Residual		SM4500 Cl G		50.0
Cobalt, Total	7440-48-4	200.8	0.05	0.25
Color		SM2120 B/C/E		10 color units
Dissolved oxygen		SM4500-OC/OG		0.2 mg/L
Flow		Calibrated device		
Fluoride	16984-48-8	SM4500-F E	25	100
Hardness, Total		SM2340B		200 as CaCO3
Iron, Total	7439-89-6	200.7	12.5	50
Magnesium, Total	7439-95-4	200.7	10	50
Manganese, Total	7439-96-5	200.8	0.1	0.5
Molybdenum, Total	7439-98-7	200.8	0.1	0.5
Nitrate + Nitrite Nitrogen (as N)		SM4500-NO3- E/F/H		100
Nitrogen, Total Kjeldahl (as N)		SM4500-N_{org}B/C and SM4500NH_3-B/C/D/EF/G/H		300
NWTPH Dx [4]		Ecology NWTPH Dx	250	250
NWTPH Gx [5]		Ecology NWTPH Gx	250	250

NONCONVENTIONAL POLLUTANTS

Pollutant & CAS No. (if available)	CAS Number (if available)	Recommended Analytical Protocol	Detection (DL)[1] µg/L unless specified	Quantitation Level (QL)[2] µg/L unless specified
Phosphorus, Total (as P)		SM 4500 PB followed by SM4500-PE/PF	3	10
Salinity		SM2520-B		3 practical salinity units or scale (PSU or PSS)
Settleable Solids		SM2540-F		Sample and limit dependent
Soluble Reactive Phosphorus (as P)		SM4500-P E/F/G	3	10
Sulfate (as mg/L SO_4)		SM4110-B		0.2 mg/L
Sulfide (as mg/L S)		SM4500-S^2F/D/E/G		0.2 mg/L
Sulfite (as mg/L SO_3)		SM4500-SO3B		2 mg/L
Temperature (max. 7-day avg.)		Analog recorder or use micro-recording devices known as thermistors		0.2º C
Tin, Total	7440-31-5	200.8	0.3	1.5
Titanium, Total	7440-32-6	200.8	0.5	2.5
Total Coliform		SM 9221B, 9222B, 9223B	N/A	Specified in method - sample aliquot dependent
Total Organic Carbon		SM5310-B/C/D		1 mg/L
Total dissolved solids		SM2540 C		20 mg/L

PRIORITY POLLUTANTS	PP #	CAS Number (if available)	Recommended Analytical Protocol	Detection (DL)[1] *µg/L unless specified*	Quantitation Level (QL)[2] *µg/L unless specified*
METALS, CYANIDE & TOTAL PHENOLS					
Antimony, Total	114	7440-36-0	200.8	0.3	1.0
Arsenic, Total	115	7440-38-2	200.8	0.1	0.5
Beryllium, Total	117	7440-41-7	200.8	0.1	0.5
Cadmium, Total	118	7440-43-9	200.8	0.05	0.25
Chromium (hex) dissolved	119	18540-29-9	SM3500-Cr C	0.3	1.2
Chromium, Total	119	7440-47-3	200.8	0.2	1.0
Copper, Total	120	7440-50-8	200.8	0.4	2.0
Lead, Total	122	7439-92-1	200.8	0.1	0.5
Mercury, Total	123	7439-97-6	1631E	0.0002	0.0005
Nickel, Total	124	7440-02-0	200.8	0.1	0.5
Selenium, Total	125	7782-49-2	200.8	1.0	1.0
Silver, Total	126	7440-22-4	200.8	0.04	0.2
Thallium, Total	127	7440-28-0	200.8	0.09	0.36
Zinc, Total	128	7440-66-6	200.8	0.5	2.5
Cyanide, Total	121	57-12-5	335.4	5	10
Cyanide, Weak Acid Dissociable	121		SM4500-CN I	5	10
Cyanide, Free Amenable to Chlorination (Available Cyanide)	121		SM4500-CN G	5	10
Phenols, Total	65		EPA 420.1		50

PRIORITY POLLUTANTS	PP #	CAS Number (if available)	Recommended Analytical Protocol	Detection (DL)[1] *µg/L unless specified*	Quantitation Level (QL)[2] *µg/L unless specified*
ACID COMPOUNDS					
2-Chlorophenol	24	95-57-8	625.1	3.3	9.9
2,4-Dichlorophenol	31	120-83-2	625.1	2.7	8.1
2,4-Dimethylphenol	34	105-67-9	625.1	2.7	8.1
4,6-dinitro-o-cresol (2-methyl-4,6,-dinitrophenol)	60	534-52-1	625.1/1625B	24	72
2,4 dinitrophenol	59	51-28-5	625.1	42	126
2-Nitrophenol	57	88-75-5	625.1	3.6	10.8
4-Nitrophenol	58	100-02-7	625.1	2.4	7.2
Parachlorometa cresol (4-chloro-3-methylphenol)	22	59-50-7	625.1	3.0	9.0
Pentachlorophenol	64	87-86-5	625.1	3.6	10.8
Phenol	65	108-95-2	625.1	1.5	4.5
2,4,6-Trichlorophenol	21	88-06-2	625.1	2.7	8.1

PRIORITY POLLUTANTS	PP #	CAS Number (if available)	Recommended Analytical Protocol	Detection (DL)[1] *µg/L unless specified*	Quantitation Level (QL)[2] *µg/L unless specified*
VOLATILE COMPOUNDS					
Acrolein	2	107-02-8	624	5	10
Acrylonitrile	3	107-13-1	624	1.0	2.0

PRIORITY POLLUTANTS	PP #	CAS Number (if available)	Recommended Analytical Protocol	Detection (DL)[1] *µg/L unless specified*	Quantitation Level (QL)[2] *µg/L unless specified*
VOLATILE COMPOUNDS					
Benzene	4	71-43-2	624.1	4.4	13.2
Bromoform	47	75-25-2	624.1	4.7	14.1
Carbon tetrachloride	6	56-23-5	624.1/601 or SM6230B	2.8	8.4
Chlorobenzene	7	108-90-7	624.1	6.0	18.0
Chloroethane	16	75-00-3	624/601	1.0	2.0
2-Chloroethylvinyl Ether	19	110-75-8	624	1.0	2.0
Chloroform	23	67-66-3	624.1 or SM6210B	1.6	4.8
Dibromochloromethane (chlordibromomethane)	51	124-48-1	624.1	3.1	9.3
1,2-Dichlorobenzene	25	95-50-1	624	1.9	7.6
1,3-Dichlorobenzene	26	541-73-1	624	1.9	7.6
1,4-Dichlorobenzene	27	106-46-7	624	4.4	17.6
Dichlorobromomethane	48	75-27-4	624.1	2.2	6.6
1,1-Dichloroethane	13	75-34-3	624.1	4.7	14.1
1,2-Dichloroethane	10	107-06-2	624.1	2.8	8.4
1,1-Dichloroethylene	29	75-35-4	624.1	2.8	8.4
1,2-Dichloropropane	32	78-87-5	624.1	6.0	18.0
1,3-dichloropropene (mixed isomers) (1,2-dichloropropylene) [6]	33	542-75-6	624.1	5.0	15.0
Ethylbenzene	38	100-41-4	624.1	7.2	21.6
Methyl bromide (Bromomethane)	46	74-83-9	624/601	5.0	10.0
Methyl chloride (Chloromethane)	45	74-87-3	624	1.0	2.0
Methylene chloride	44	75-09-2	624.1	2.8	8.4
1,1,2,2-Tetrachloroethane	15	79-34-5	624.1	6.9	20.7
Tetrachloroethylene	85	127-18-4	624.1	4.1	12.3
Toluene	86	108-88-3	624.1	6.0	18.0
1,2-Trans-Dichloroethylene (Ethylene dichloride)	30	156-60-5	624.1	1.6	4.8
1,1,1-Trichloroethane	11	71-55-6	624.1	3.8	11.4
1,1,2-Trichloroethane	14	79-00-5	624.1	5.0	15.0
Trichloroethylene	87	79-01-6	624.1	1.9	5.7
Vinyl chloride	88	75-01-4	624/SM6200B	1.0	2.0

PRIORITY POLLUTANTS	PP #	CAS Number (if available)	Recommended Analytical Protocol	Detection (DL)[1] *µg/L unless specified*	Quantitation Level (QL)[2] *µg/L unless specified*
BASE/NEUTRAL COMPOUNDS (compounds in bold are Ecology PBTs)					
Acenaphthene	1	83-32-9	625.1	1.9	5.7
Acenaphthylene	77	208-96-8	625.1	3.5	10.5
Anthracene	78	120-12-7	625.1	1.9	5.7
Benzidine	5	92-87-5	625.1	44	132
Benzyl butyl phthalate	67	85-68-7	625.1	2.5	7.5
Benzo(*a*)anthracene	72	56-55-3	625.1	7.8	23.4
Benzo(b)fluoranthene (3,4-benzofluoranthene) [7]	74	205-99-2	610/625.1	4.8	14.4

PRIORITY POLLUTANTS	**PP #**	**CAS Number (if available)**	**Recommended Analytical Protocol**	**Detection (DL)[1] *µg/L unless specified***	**Quantitation Level (QL) [2] *µg/L unless specified***
BASE/NEUTRAL COMPOUNDS (compounds in bold are Ecology PBTs)					
Benzo(j)fluoranthene [7]		**205-82-3**	625	0.5	1.0
Benzo(k)fluoranthene (11,12-benzofluoranthene) [7]	75	207-08-9	610/625.1	2.5	7.5
Benzo(r,s,t)pentaphene		**189-55-9**	625	1.3	5.0
Benzo(*a*)pyrene	73	50-32-8	610/625.1	2.5	7.5
Benzo(*ghi*)Perylene	79	191-24-2	610/625.1	4.1	12.3
Bis(*2-chloroethoxy)*methane	43	111-91-1	625.1	5.3	15.9
Bis(*2-chloroethyl*)ether	18	111-44-4	611/625.1	5.7	17.1
Bis(*2-chloroisopropyl*)ether	42	39638-32-9	625	0.5	1.0
Bis(*2-ethylhexyl*)phthalate	66	117-81-7	625.1	2.5	7.5
4-Bromophenyl phenyl ether	41	101-55-3	625.1	1.9	5.7
2-Chloronaphthalene	20	91-58-7	625.1	1.9	5.7
4-Chlorophenyl phenyl ether	40	7005-72-3	625.1	4.2	12.6
Chrysene	76	218-01-9	610/625.1	2.5	7.5
Dibenzo (a,h)acridine		**226-36-8**	610M/625M	2.5	10.0
Dibenzo (a,j)acridine		**224-42-0**	610M/625M	2.5	10.0
Dibenzo(a-*h*)anthracene (1,2,5,6-dibenzanthracene)	82	53-70-3	625.1	2.5	7.5
Dibenzo(a,e)pyrene		192-65-4	610M/625M	2.5	10.0
Dibenzo(a,h)pyrene		189-64-0	625M	2.5	10.0
3,3-Dichlorobenzidine	28	91-94-1	605/625.1	16.5	49.5
Diethyl phthalate	70	84-66-2	625.1	1.9	5.7
Dimethyl phthalate	71	131-11-3	625.1	1.6	4.8
Di-n-butyl phthalate	68	84-74-2	625.1	2.5	7.5
2,4-dinitrotoluene	35	121-14-2	609/625.1	5.7	17.1
2,6-dinitrotoluene	36	606-20-2	609/625.1	1.9	5.7
Di-n-octyl phthalate	69	117-84-0	625.1	2.5	7.5
1,2-Diphenylhydrazine (*as Azobenzene*)	37	122-66-7	1625B	5.0	20
Fluoranthene	39	206-44-0	625.1	2.2	6.6
Fluorene	80	86-73-7	625.1	1.9	5.7
Hexachlorobenzene	9	118-74-1	612/625.1	1.9	5.7
Hexachlorobutadiene	52	87-68-3	625.1	0.9	2.7
Hexachlorocyclopentadiene	53	77-47-4	1625B/625	2.0	4.0
Hexachloroethane	12	67-72-1	625.1	1.6	4.8
Indeno(*1,2,3-cd*)Pyrene	83	193-39-5	610/625.1	3.7	11.1
Isophorone	54	78-59-1	625.1	2.2	6.6
3-Methyl cholanthrene		**56-49-5**	625	2.0	8.0
Naphthalene	55	91-20-3	625.1	1.6	4.8
Nitrobenzene	56	98-95-3	625.1	1.9	5.7
N-Nitrosodimethylamine	61	62-75-9	607/625	2.0	4.0
N-Nitrosodi-n-propylamine	63	621-64-7	607/625	0.5	1.0
N-Nitrosodiphenylamine	62	86-30-6	625	1.0	2.0
Perylene		**198-55-0**	625	1.9	7.6
Phenanthrene	81	85-01-8	625.1	5.4	16.2
Pyrene	84	129-00-0	625.1	1.9	5.7
1,2,4-Trichlorobenzene	8	120-82-1	625.1	1.9	5.7

PRIORITY POLLUTANT	PP #	CAS Number (if available)	Recommended Analytical Protocol	Detection (DL)[1] *µg/L unless specified*	Quantitation Level (QL)[2] *µg/L unless specified*
DIOXIN					
2,3,7,8-Tetra-Chlorodibenzo-P-Dioxin (2,3,7,8 TCDD)	129	1746-01-6	1613B	1.3 pg/L	5 pg/L

PRIORITY POLLUTANTS	PP #	CAS Number (if available)	Recommended Analytical Protocol	Detection (DL)[1] *µg/L unless specified*	Quantitation Level (QL)[2] *µg/L unless specified*
PESTICIDES/PCBs					
Aldrin	89	309-00-2	608.3	4.0 ng/L	12 ng/L
alpha-BHC	102	319-84-6	608.3	3.0 ng/L	9.0 ng/L
beta-BHC	103	319-85-7	608.3	6.0 ng/L	18 ng/L
gamma-BHC (Lindane)	104	58-89-9	608.3	4.0 ng/L	12 ng/L
delta-BHC	105	319-86-8	608.3	9.0 ng/L	27 ng/L
Chlordane [8]	91	57-74-9	608.3	14 ng/L	42 ng/L
4,4'-DDT	92	50-29-3	608.3	12 ng/L	36 ng/L
4,4'-DDE	93	72-55-9	608.3	4.0 ng/L	12 ng/L
4,4' DDD	94	72-54-8	608.3	11ng/L	33 ng/L
Dieldrin	90	60-57-1	608.3	2.0 ng/L	6.0 ng/L
alpha-Endosulfan	95	959-98-8	608.3	14 ng/L	42 ng/L
beta-Endosulfan	96	33213-65-9	608.3	4.0 ng/L	12 ng/L
Endosulfan Sulfate	97	1031-07-8	608.3	66 ng/L	198 ng/L
Endrin	98	72-20-8	608.3	6.0 ng/L	18 ng/L
Endrin Aldehyde	99	7421-93-4	608.3	23 ng/L	70 ng/L
Heptachlor	100	76-44-8	608.3	3.0 ng/L	9.0 ng/L
Heptachlor Epoxide	101	1024-57-3	608.3	83 ng/L	249 ng/L
PCB-1242 [9]	106	53469-21-9	608.3	0.065	0.095
PCB-1254	107	11097-69-1	608.3	0.065	0.095
PCB-1221	108	11104-28-2	608.3	0.065	0.095
PCB-1232	109	11141-16-5	608.3	0.065	0.095
PCB-1248	110	12672-29-6	608.3	0.065	0.095
PCB-1260	111	11096-82-5	608.3	0.065	0.095
PCB-1016 [9]	112	12674-11-2	608.3	0.065	0.095
Toxaphene	113	8001-35-2	608.3	240 ng/L	720 ng/L

1. Detection level (DL) or detection limit means the minimum concentration of an analyte (substance) that can be measured and reported with a 99% confidence that the analyte concentration is greater than zero as determined by the procedure given in 40 CFR part 136, Appendix B.

2. Quantitation Level (QL) also known as Minimum Level of Quantitation (ML) – The lowest level at which the entire analytical system must give a recognizable signal and acceptable calibration point for the analyte. It is equivalent to the concentration of the lowest calibration standard, assuming that the lab has used all method-specified sample weights, volumes, and cleanup procedures. The QL is calculated by multiplying the MDL by 3.18 and rounding the result to the number nearest to (1, 2, or 5) x 10^n, where n is an integer. (64 FR 30417).
   ALSO GIVEN AS:
   The smallest detectable concentration of analyte greater than the Detection Limit (DL) where the accuracy (precision & bias) achieves the objectives of the intended purpose. (Report of the Federal Advisory Committee on Detection and Quantitation Approaches and Uses in Clean Water Act Programs Submitted to the US Environmental Protection Agency, December 2007).

3. Soluble Biochemical Oxygen Demand method note: First, filter the sample through a Millipore Nylon filter (or equivalent) - pore size of 0.45-0.50 um (prep all filters by filtering 250 ml of laboratory grade deionized water through the filter and discard). Then, analyze sample as per method 5210-B.

4. NWTPH Dx – Northwest Total Petroleum Hydrocarbons Diesel Extended Range – see http://www.ecy.wa.gov/biblio/97602.html

5. NWTPH Gx – Northwest Total Petroleum Hydrocarbons Gasoline Extended Range – see http://www.ecy.wa.gov/biblio/97602.html

6. 1, 3-dichloroproylene (mixed isomers) – You may report this parameter as two separate parameters: cis-1, 3-dichlorpropropene (10061-01-5) and trans-1, 3-dichloropropene (10061-02-6).

7. Total Benzofluoranthenes – Because Benzo(b)fluoranthene, Benzo(j)fluoranthene and Benzo(k)fluoranthene co-elute you may report these three isomers as total benzofluoranthenes.

8. Chlordane – You may report alpha-chlordane (5103-71-9) and gamma-chlordane (5103-74-2) in place of chlordane (57-74-9). If you report alpha and gamma-chlordane, the DL/PQLs that apply are 14/42 ng/L.

9. PCB 1016 & PCB 1242 – You may report these two PCB compounds as one parameter called PCB 1016/1242.

Appendix L

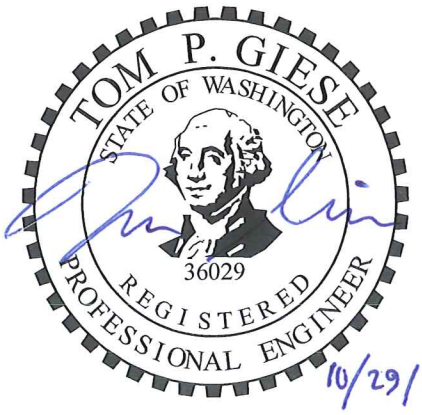
Snohomish CSO Management Plan

SNOHOMISH
WASHINGTON

City of Snohomish

Combined Sewer Overflow Management Plan

October 2014



10/29/14

Prepared by:

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CITY OF SNOHOMISH
DRAFT COMBINED SEWER OVERFLOW MANAGEMENT PLAN
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This report has been prepared for the use of the client for the specific purposes identified in the report. The conclusions, observations, and recommendations contained herein attributed to BHC Consultants constitute the opinions of BHC Consultants. To the extent that statements, information, and opinions provided by the client or others have been used in the preparation of this report, BHC Consultants has relied upon the same to be accurate, and for which no assurances are intended and no representations or warranties are made. BHC Consultants makes no certification and gives no assurances except as explicitly set forth in this report

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CHAPTER 1
INTRODUCTION

PURPOSE

This Combined Sewer Overflow (CSO) Management Plan (Plan) identifies flow management strategies that are intended to limit the occurrence of CSO events to no more than one per outfall per year on average, while also conveying wet weather flows to the wastewater treatment plant (WWTP) without exceeding its capacity or permitted limits. This Plan is prepared as an updated and expanded version of the initial Plan submitted in February 2014. This plan also responds to comments by the Washington State Department of Ecology (Ecology) on the initial Plan. That initial Plan was submitted to Ecology in compliance with Amended Agreed Order No. 10467 (Amendment No. 1 to Agreed Order No. 7974).

BACKGROUND

Combined sewage is a mixture of sanitary sewage from residential and commercial customers and storm water from rainfall. It had been historically common to combine these two water streams into a single conveyance system, therefore the term, Combined Sewage. Before the construction of the CSO Pump Station, millions of gallons of combined sewage, including untreated sanitary sewage, overflowed into the Snohomish River every year. Since the completion of the CSO Pump Station in April 2011, the City of Snohomish (City) has had zero CSO overflows over three winters, in which time all combined sewage has been conveyed through the CSO Pump Station and gravity trunkline to the WWTP. However, there has been an occurrence of a short duration overflow during late summer/early fall each of the past three years due to brief periods (less than one hour) of very high intensity rainfall, which is discussed in detail in Chapter 4. The WWTP staff routinely keeps the water level in the treatment lagoons as low as practical to ensure sufficient volume is available to attenuate peak flows from storm events to avoid overloading or stressing the treatment processes and to allow more efficient operation.

The initial Plan described how the City intended to operate the CSO pump station to control the occurrence of CSO events, so as not to overload the WWTP. The City submitted the initial Plan on February 18, 2014, prior to the milestone deadline of February 28, 2014, as required by Amended Agreed Order No. 10467. In response to the initial Plan, Ecology requested additional items be added to the Plan (primarily hydraulic flow modeling and a WWTP hydraulic capacity evaluation). These additional items were not delineated in the Amended Agreed Order. Feedback was provided both verbally, during an April 29, 2014 meeting with Ecology, and in written format through a briefing provided by Ecology at that same meeting, also dated April 29, 2014. The verbal feedback reinforced the written feedback contained in the briefing. Following the meeting with Ecology, an outline for an expanded and updated version of the Plan was prepared and submitted to Ecology for review on May 27, 2014. Ecology responded in an e-mail dated June 17, 2014, that the submitted outline was acceptable as the basis for the Plan.

ORGANIZATION

As indicated in the outline submitted to Ecology, this Plan consists of the following:

- **Chapter 1 – Introduction.**
- **Chapter 2 – WWTP Hydraulic Evaluation.** Describes the configuration and operation of the existing WWTP, analyzes storage volumes available for flow attenuation, analyzes historical data and performance at peak flows, estimates hydraulic capacities by unit

process, determines overall hydraulic constraints, and identifies current sampling procedures.

- **Chapter 3 – CSO Pump Station Evaluation.** Describes current configuration and operation of the CSO Pump Station, identifies relevant control elements, identifies the associated storage volume and normal operating conditions and discusses how the storage volume and operating conditions change during storm events.
- **Chapter 4 – Hydraulic Modeling of the Combined Sewer System.** Provides an overview of model development, discusses model calibration, summarizes the modeling results based on simulating 27 years of historical rainfall and compares rainfall data used for model calibration with the original design storm.
- **Chapter 5 – Recommendations.** Recommends operational adjustments and system improvements, as necessary, to reduce the occurrence of CSO events without exceeding the hydraulic capacity of the WWTP and discusses a general operational strategy for controlling flow through the WWTP.

CHAPTER 2
WWTP HYDRAULIC EVALUATION

This Chapter describes the configuration and operation of the existing WWTP, analyzes storage volumes available for flow attenuation, analyzes historical data and performance at peak flows, estimates hydraulic capacities by unit process, determines overall hydraulic constraints, and identifies current sampling procedures.

EXISTING WWTP

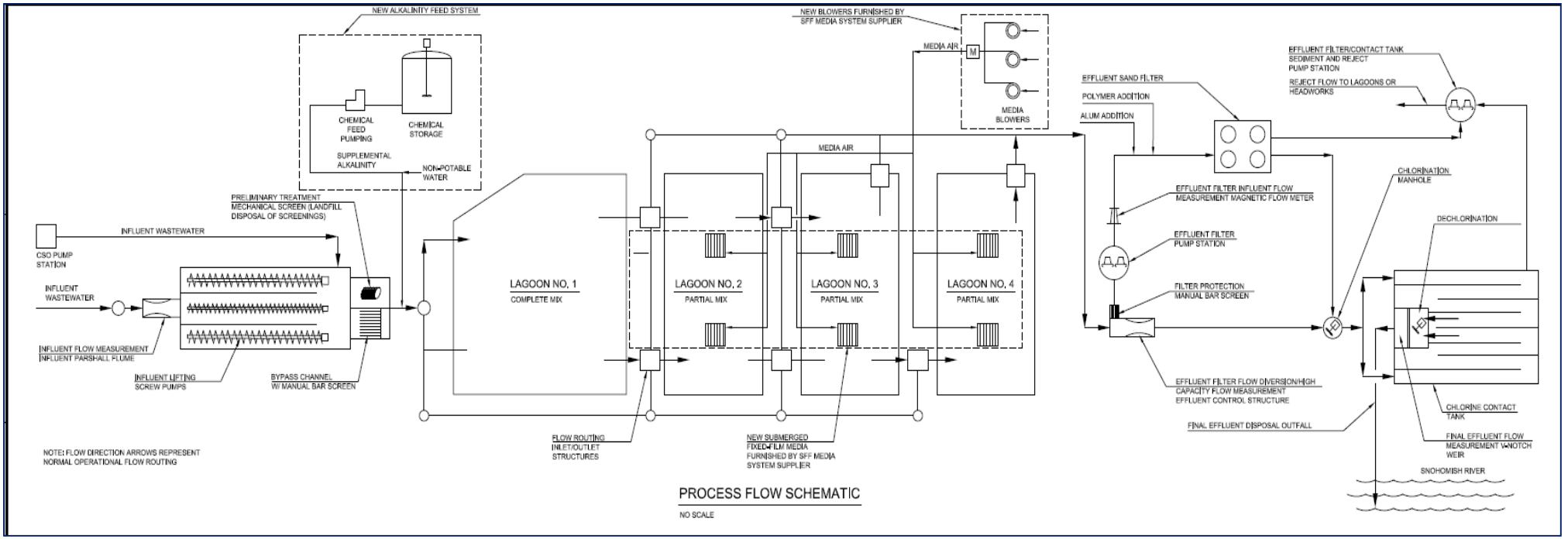
The City currently operates a four-stage lagoon WWTP, which was retrofitted with a submerged fixed-film (SFF) media system in 2012. The WWTP also includes influent pumping and flow measurement, screening, addition of supplemental alkalinity, effluent filtration, and chlorine disinfection followed by dechlorination.

Prior to the lagoons, influent flow is received at the headworks through both a gravity sewer pipeline and a force main from the CSO Pump Station. The influent gravity sewer flow passes through a Parshall flume for flow measurement and then is pumped to a mechanical screen using influent screw pumps. Flow from the CSO Pump Station is received directly upstream of the mechanical screen. A magnetic flow meter at the CSO Pump Station measures the pumped flow. The mechanical screen is a multi-rake bar screen with ¼-inch openings. There is a manual bar screen with ¾-inch openings in an adjacent channel that can be utilized when the mechanical screen requires maintenance. Following screening, supplemental alkalinity is dosed into the wastewater prior to entering the lagoons. Supplemental alkalinity supports nitrification in the lagoons by preventing a drop in pH, which could lead to accumulation of nitrite and occurrence of nitrite-lock in the disinfection process, particularly during high influent flows when influent alkalinity can be weak due to dilution from storm water.

The four aerated lagoons at the WWTP comprise a hybrid lagoon process called a dual-powered multi-cellular (DPMC) aerated lagoon system, which combines aspects of suspended growth and facultative treatment. The first lagoon is a low-rate, suspended-growth treatment lagoon, with a normal volume of approximately 10 million gallons. Three partially mixed facultative lagoons in series follow the first lagoon, providing a normal volume of approximately 3.5 million gallons each. These three facultative lagoons provide additional biological treatment, as well as settling, storage, and digestion of solids. Additionally, each of these three partially mixed lagoons has been retrofitted with 18 Submerged Fixed Film (SFF) media modules for enhanced treatment. The SFF media modules provide a surface for growth of biomass to increase the population of microorganisms in the lagoons, thereby increasing the effectiveness of treatment and reducing washout of biomass at high flows.

Following treatment in the lagoons, effluent is normally pumped to upflow, deep bed sand filters and then flows by gravity to the disinfection system. Effluent pumped to the filters is dosed with aluminum sulfate and polymer to increase solids capture, thereby reducing associated carbonaceous biochemical oxygen demand (CBOD) and total suspended solids (TSS) in the effluent. Currently, capacity of the filters is limited to about 0.8 million gallons per day (MGD), though minor modifications are planned that could increase filter capacity to 1.4 MGD. Effluent flow in excess of the filter capacity flows by gravity directly to the disinfection system. The disinfection system uses chlorine gas that is mixed with the effluent in a chlorination manhole upstream of the chlorine contact tank. Sulfur dioxide gas is mixed with the effluent near the end of the chlorine contact tank to eliminate the chlorine residual prior to discharge through a 30-inch outfall into the Snohomish River. A schematic of the WWTP is shown in Figure 2-1 below.

FIGURE 2-1: WWTP SCHEMATIC



Source: *City of Snohomish Submerged Fixed-Film Media Performance Assessment Report* (Kennedy/Jenks, August 2014)

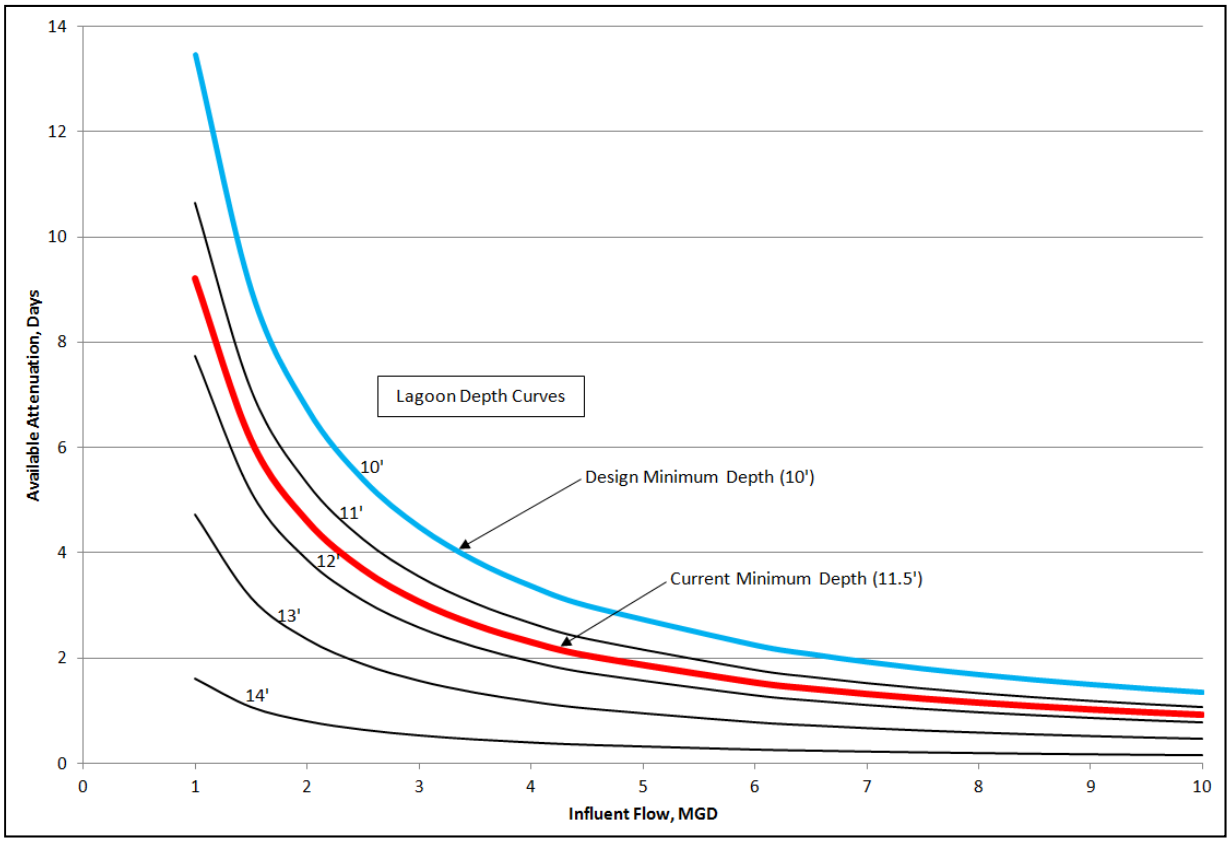
FLOW ATTENUATION

Due to the size of the lagoons and the fact that the normal operating water level is up to 6 feet below the top of the surrounding berms, there is considerable storage available in the lagoons. The City takes advantage of this storage by normally operating at water depths of 10.0 to 11.5 feet and allowing the depth to increase up to about 14.5 feet under high flow conditions by restricting effluent discharge. Limiting effluent discharge allows more time for treatment in the lagoons, allows a higher percentage of the effluent flow to be filtered and provides greater contact time for disinfection, all of which contribute to improved effluent quality. A water depth of 14.5 feet is right at the invert of the internal spillways between lagoons, which is a practical limit as it maintains a reasonable 1.5 feet of freeboard from the top of the berms and prevents short circuiting across the surface of the lagoons.

Although the normal design operating depth is 10.0 feet, the City currently operates at a normal depth of 11.5 feet. Following installation and startup of the SFF media system, sediment in the lagoons has been redistributed such that much of the sediment is now piled higher along the sides of the lagoons, but less is in the center of the lagoons around the SFF media modules. As a result, the City has been maintaining higher lagoon levels to provide a greater buffer between the sediment layer and the water surface in an effort to minimize the impact of periodic lagoon turnover events. The City is currently planning to remove sediment from the lagoons in 2015, after which the City can consider returning to the normal design operating depth of 10.0 feet.

Available attenuation in days at various operating depths and influent flows is illustrated in Figure 2-2 below assuming a maximum water depth of 14.5 feet.

FIGURE 2-2: AVAILABLE ATTENUATION VERSUS INFLUENT FLOW



A summary of the storage volume available for attenuation at various operating depths is shown in Table 2-1 below.

TABLE 2-1
STORAGE VOLUME AVAILABLE FOR ATTENUATION VS. OPERATING DEPTH

Operating Depth	Storage Volume Available for Attenuation
10.0 Feet	13,454,000 Gallons
11.0 Feet	10,650,000 Gallons
11.5 Feet	9,209,000 Gallons
12.0 Feet	7,742,000 Gallons
13.0 Feet	4,727,000 Gallons
14.0 Feet	1,603,000 Gallons
14.5 Feet	0 Gallons

As mentioned previously, the WWTP staff can restrict effluent discharge and use the storage volume available for attenuation to maximize effluent quality. Discharge from Lagoon 4 is controlled by manually adjusting a canal gate at the Lagoon 4 outlet structure. When the WWTP staff partially closes this canal gate, effluent flow backs up into the lagoons and causes the water level in the lagoons to rise. A manually adjustable weir is located upstream of this canal gate. The normal position of this weir is at its full height, which maintains a minimum operating depth of 10.0 feet. At its lowest position, the operating depth in the lagoons can be as low as 7.75 feet, although the water level in the lagoons cannot be much below 10 feet and maintain proper submergence of the SFF media modules. There is also a canal gate on the inlet to each of the two chambers of the chlorine contact tank. These canal gates are normally fully open, but are used at times to isolate the chlorine contact tank. When level in the lagoons exceeds a depth of 10 feet and all canal gates are fully open, lagoon level is controlled by the v-notch weir at the end of the chlorine contact tank.

The WWTP staff normally maintains the operating depth in the lagoons between 10.0 and 11.5 feet so that there is a large storage volume available for attenuation to accommodate high flows during storm events. As already discussed, effluent discharge is restricted during high influent flows, rather than letting effluent flow match influent flow. This procedure maximizes effluent quality by gaining a greater level of treatment through longer hydraulic retention times in the lagoons and chlorine contact tank and having a greater fraction of the flow filtered. After the high flows subside, the WWTP staff allows the rate of effluent discharge to exceed the influent flow rate so that the operating depths in the lagoons are brought back down to their normal levels to provide a storage volume available for attenuation for the next storm event.

The WWTP staff considers current lagoon levels, current influent flow rates and rainfall, and forecasted rainfall when selecting the level of effluent discharge restriction. For instance, effluent discharge would be less restricted if a prolonged storm is forecast so that use of storage volume for attenuation could be spread out over the entire storm. Conversely, effluent discharge may be substantially restricted if current flows are very high and no significant storms are forecast in the immediate future.

HISTORICAL PERFORMANCE

Historically, the WWTP has produced effluent of acceptable quality during high flows. Figures 2-3 and 2-4 below depict effluent quality between January 2006 and June 2014 pertaining to CBOD, TSS and fecal coliform during high flows. For the purpose of this evaluation, high flows are defined as those over 4.0 MGD during a 24-hour period. Data points are limited to those high flow periods where effluent sampling occurred that day or the immediate day following.

FIGURE 2-3: EFFLUENT CBOD AND TSS AT HIGH FLOWS

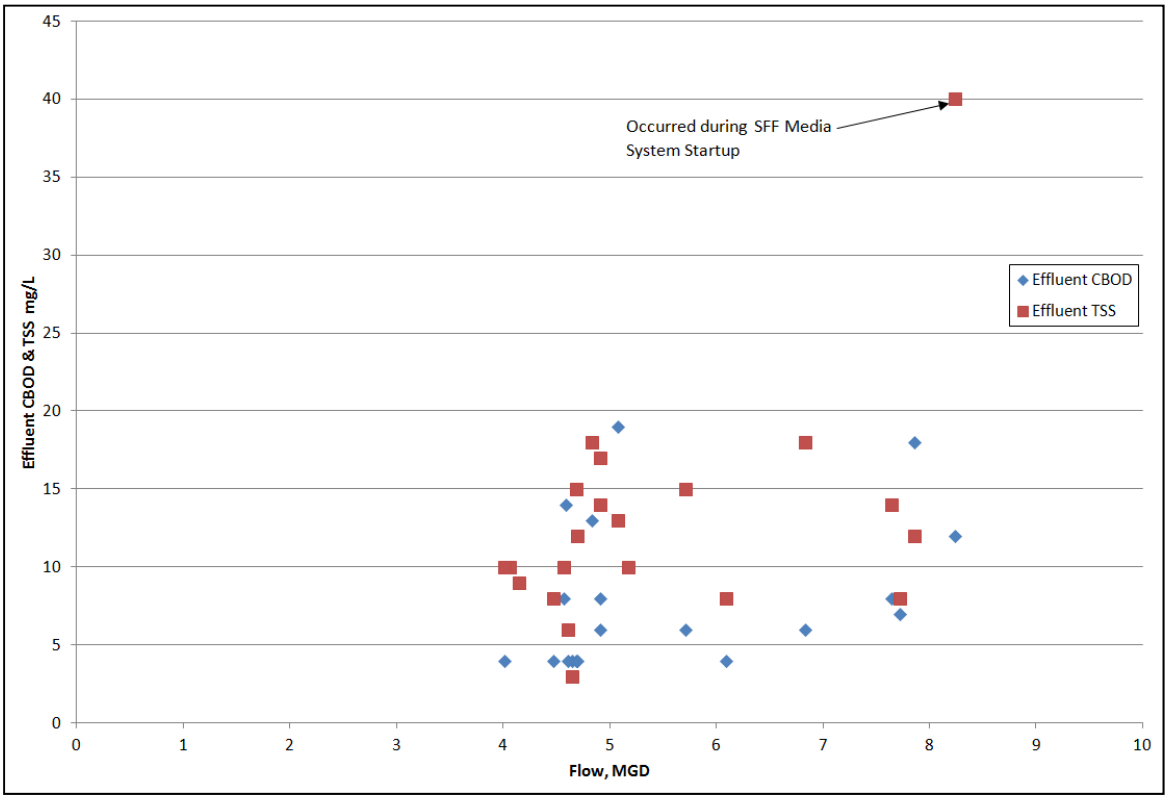
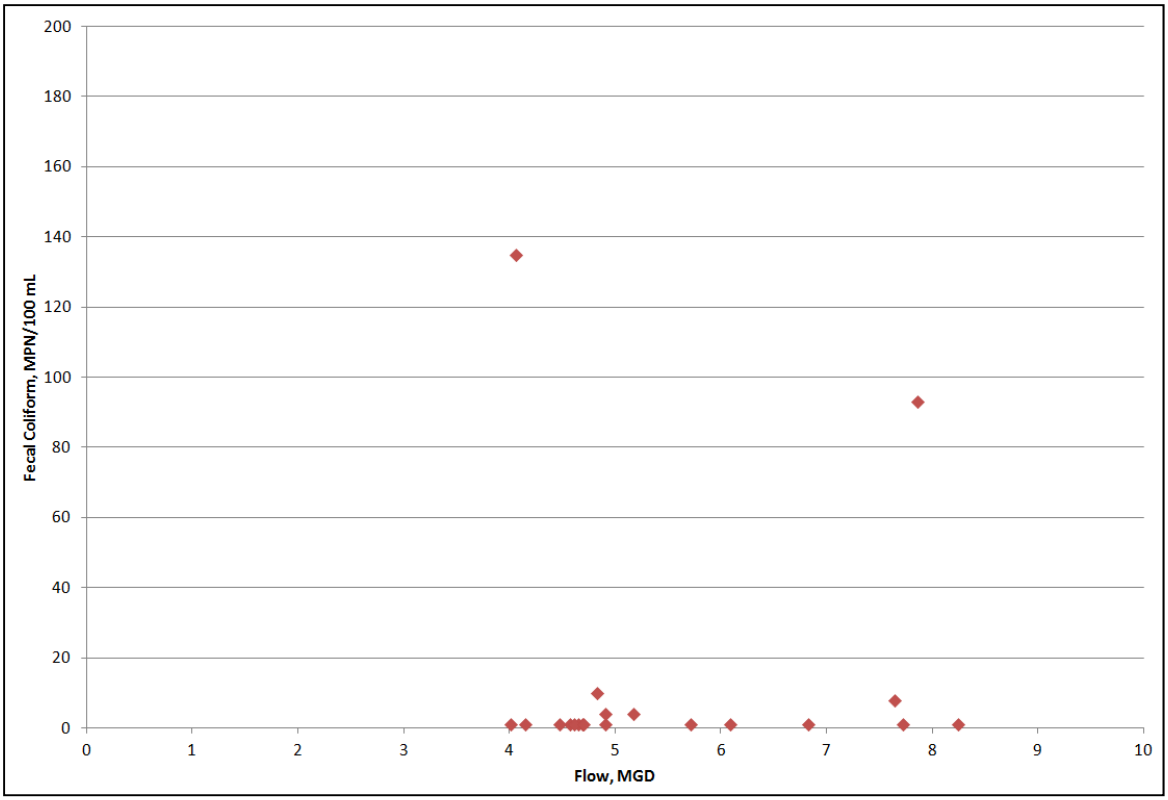


FIGURE 2-4: EFFLUENT FECAL COLIFORM AT HIGH FLOWS



As shown in Figure 2-3, the effluent CBOD and TSS concentrations during high flow periods have been consistently below the monthly average National Pollutant Discharge Elimination System (NPDES) permit limit of 30 milligrams per liter (mg/L) for TSS and 25 mg/L for CBOD and the weekly average limits of 40 mg/L CBOD and 45 mg/L TSS. Similarly, data in Figure 2-4 demonstrates that effluent fecal coliform concentrations during high flow periods have been consistently below the monthly geometric mean of 200 per 100 milliliters and 7-day geometric mean of 400 per 1000 milliliters required by the NPDES permit. There are no daily concentration limits applicable to CBOD, TSS or fecal coliform. In fact, all of the concentrations during high flow periods have been below the monthly average limits, except for an occurrence of effluent TSS exceeding 30 mg/L that was concurrent with startup of the SFF media system. This indicates that high flows do not contribute significantly to effluent concentrations exceeding monthly average or weekly average permit limits. The occurrence of TSS concentration during high flow exceeding 30 mg/L was attributed to operation of the SFF media modules, which stirred settled sediment in the lagoons. Effluent TSS was higher than normal during startup, even at lower flows, until redistribution of the sediment was complete. Therefore, it is believed that under normal circumstances the high flow would not have resulted in such a spike in effluent TSS.

In addition to maintaining effluent CBOD and TSS concentrations below the permitted limit, the WWTP must also achieve a minimum 85 percent removal of CBOD and TSS on a monthly average. The monthly average percent removal versus the monthly average flow is shown for CBOD and TSS in Figures 2-5 and 2-6 below, respectively. The data is segmented into two parts: before completion of installation and startup of the SFF media system (January 2006 through December 2012) and after completion of installation and startup of the SFF media

system (January 2013 through June 2014). Installation and startup of the SFF media system was completed at the end of December 2012.

FIGURE 2-5: MONTHLY AVERAGE CBOD PERCENT REMOVAL VERSUS MONTHLY AVERAGE FLOW

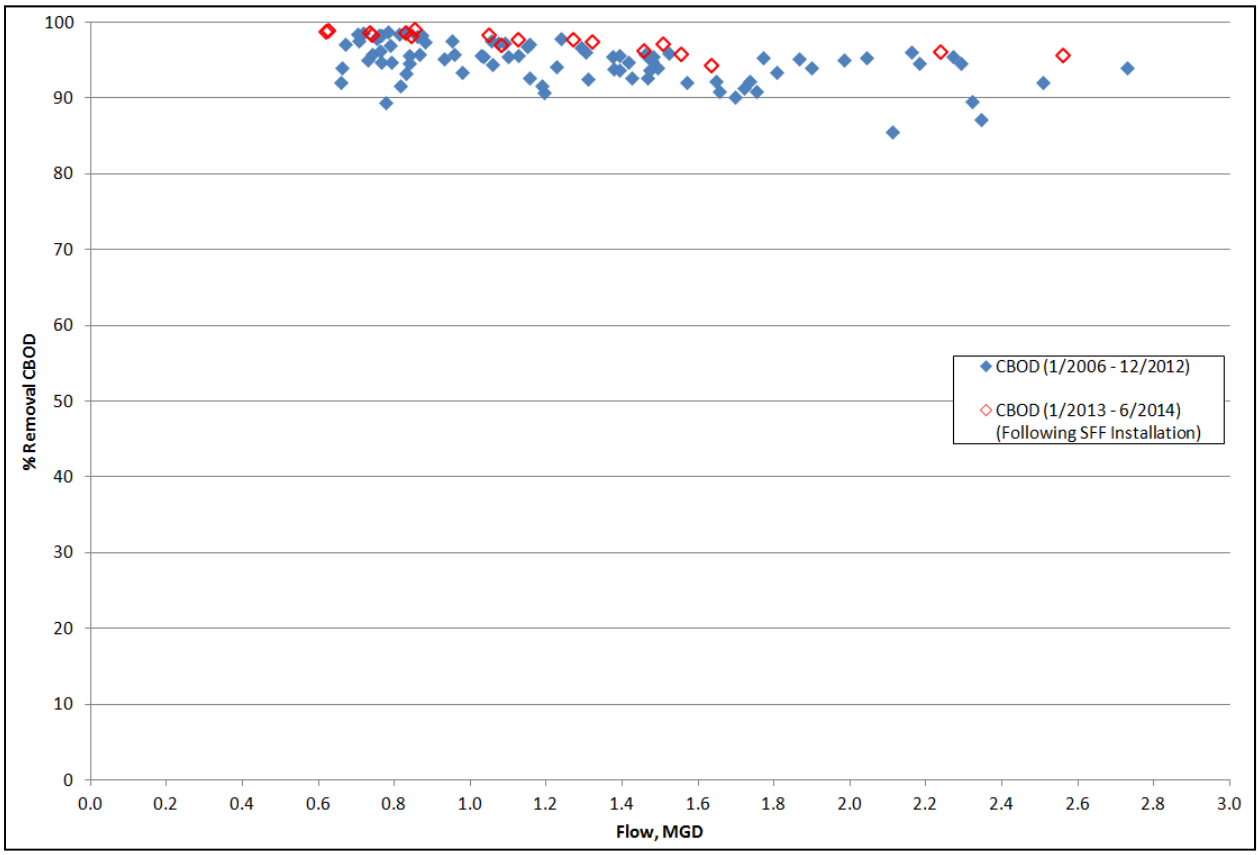
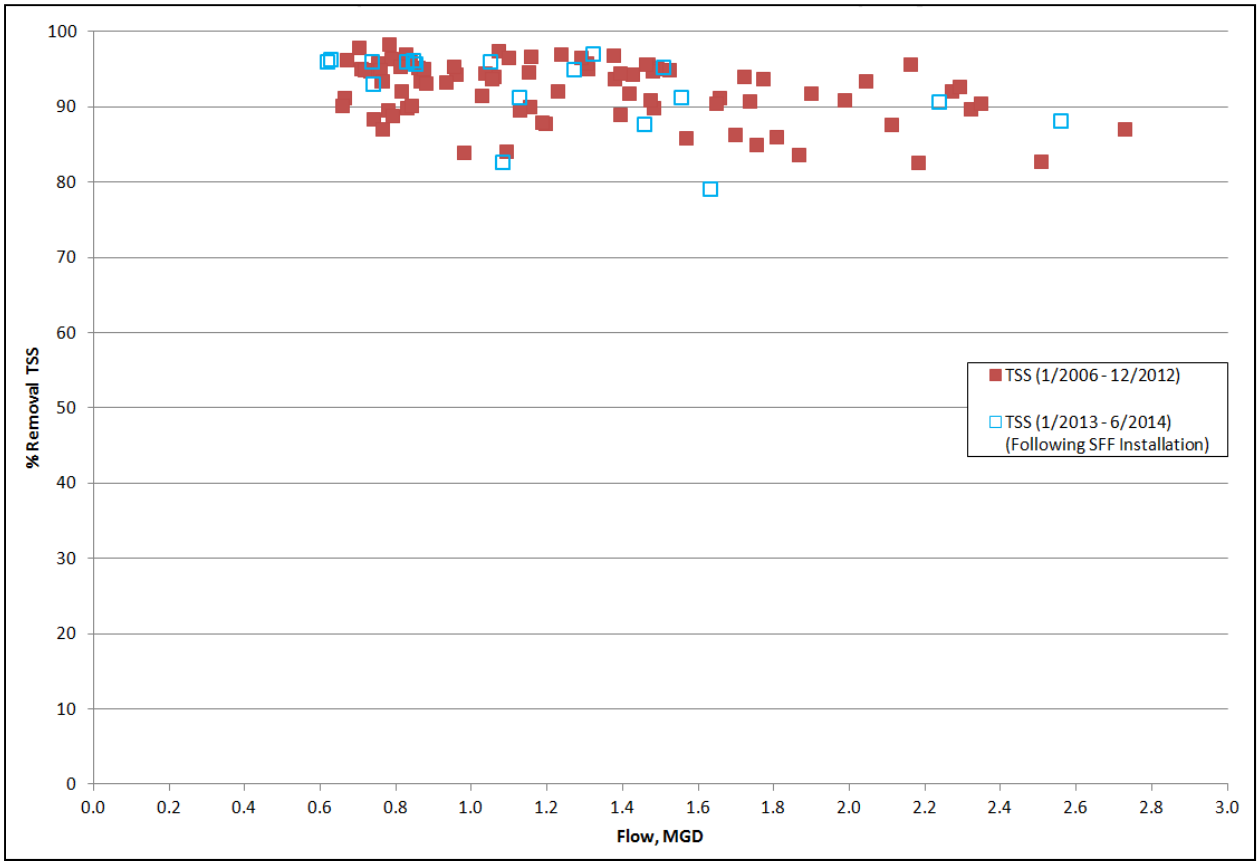


FIGURE 2-6: MONTHLY AVERAGE TSS PERCENT REMOVAL VERSUS MONTHLY AVERAGE FLOW



As shown in Figure 2-5, the removal of CBOD has routinely exceeded the 85 percent minimum required by the NPDES permit, even as flows approached the permitted influent flow limit of 2.8 MGD. In fact, the removal of CBOD has shown a noticeable improvement since installation and startup of the SFF media system was completed. As for TSS, the percent removal before and after installation and startup of the SFF media system has been about the same, with months in both cases where the minimum 85 percent removal was not achieved. Although more of the excursions are at higher monthly average flows, there are also excursions at lower monthly average flows. So, the ability to achieve a minimum 85 percent removal does not appear to be linked solely to flow and hydraulic capacity and periods of high flow. Figure 2-6 does however, indicate that percent removal is somewhat influenced by flow, that is, higher percent removal at lower flows.

A comparison of the percent removal of CBOD and TSS for a given day at high flows (greater than 4 MGD) is shown in Figure 2-7 below. Figure 2-7 also indicates that percent removal is influenced by flow, that is, higher percent removal at lower flows. In contrast, as shown in Figure 2-8, effluent TSS concentration does not appear to have any significant correlation with flow (r-squared value of 0.0061). There is, however, a noticeable correlation between influent flow and influent TSS concentration (r-squared value of 0.4738), as evident in Figure 2-9. Higher influent flow tends to result in a lower influent concentrations, because the increased flow is typically storm water from the combined system with little waste load. The conclusion, therefore, is that lower influent concentrations mean that a lower than normal effluent concentration must be achieved to yield the same percent removal. Since effluent TSS

concentrations generally remain about the same regardless of flow and cannot be significantly improved, the typical result is a lower percent removal with the lower influent concentrations.

FIGURE 2-7: PERCENT REMOVAL OF CBOD AND TSS AT HIGH FLOWS

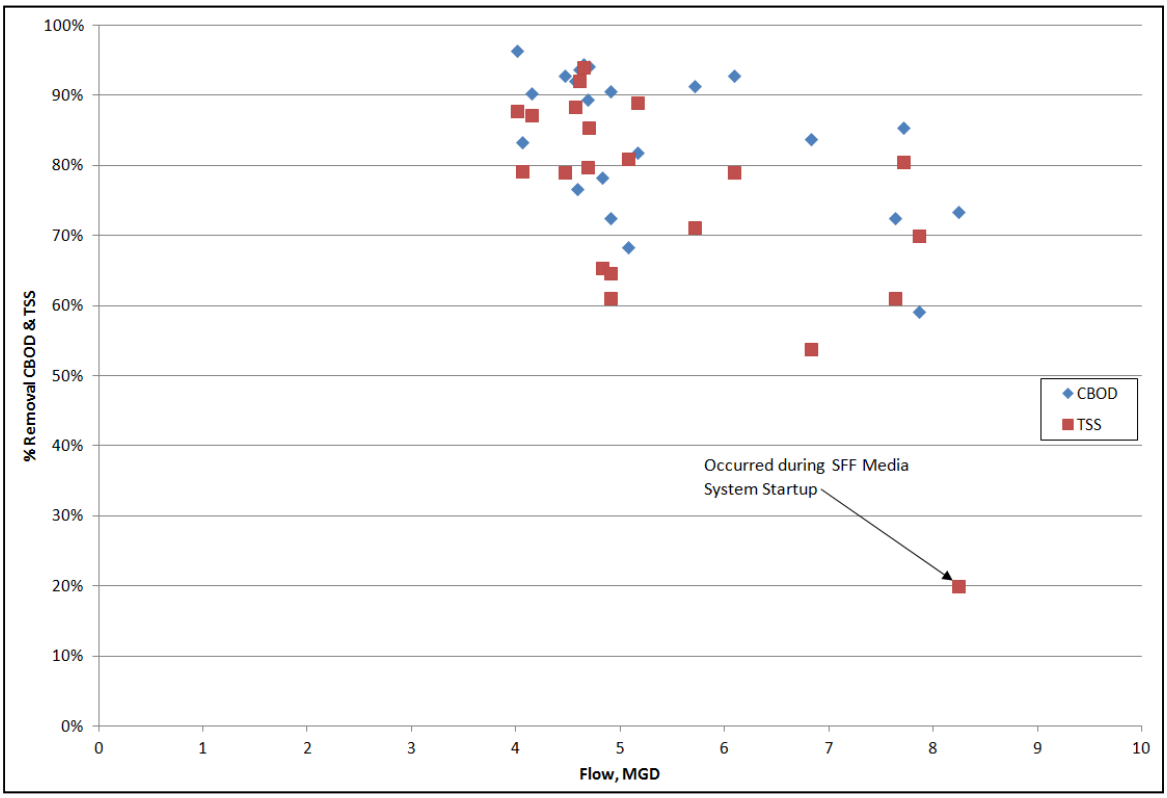


FIGURE 2-8: EFFLUENT TSS CONCENTRATION VERSUS FLOW

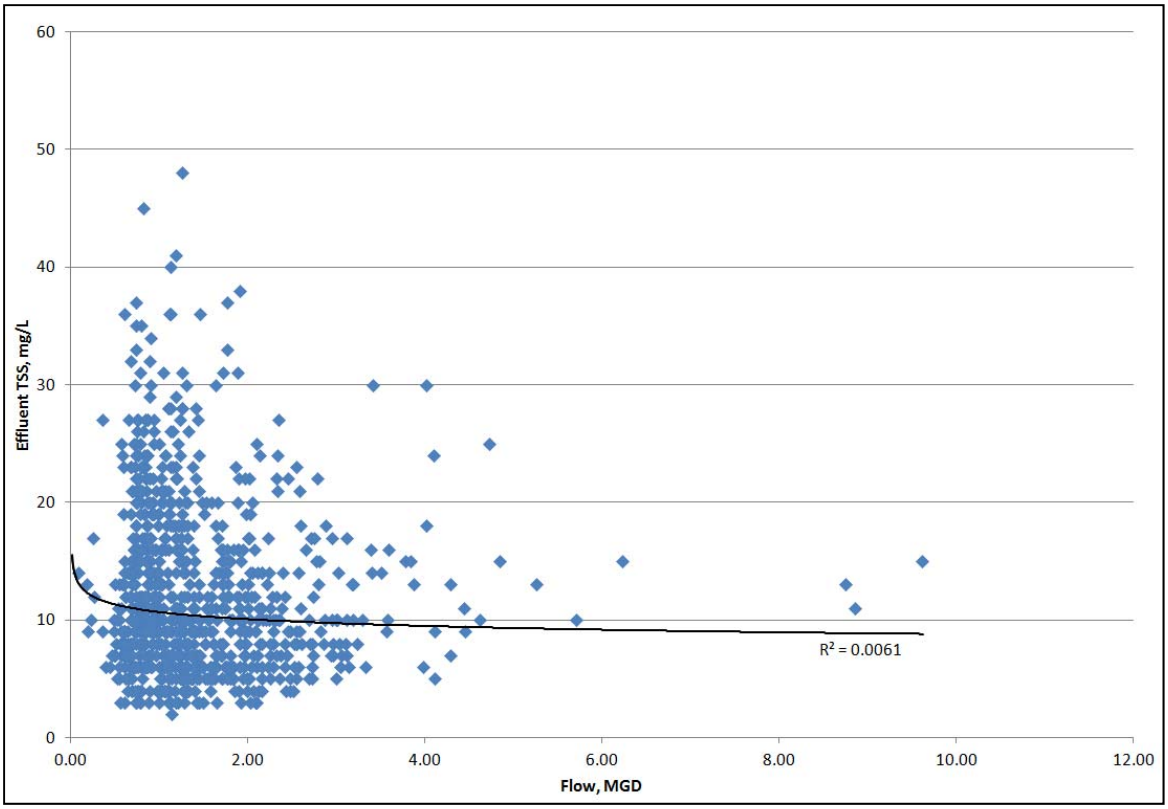
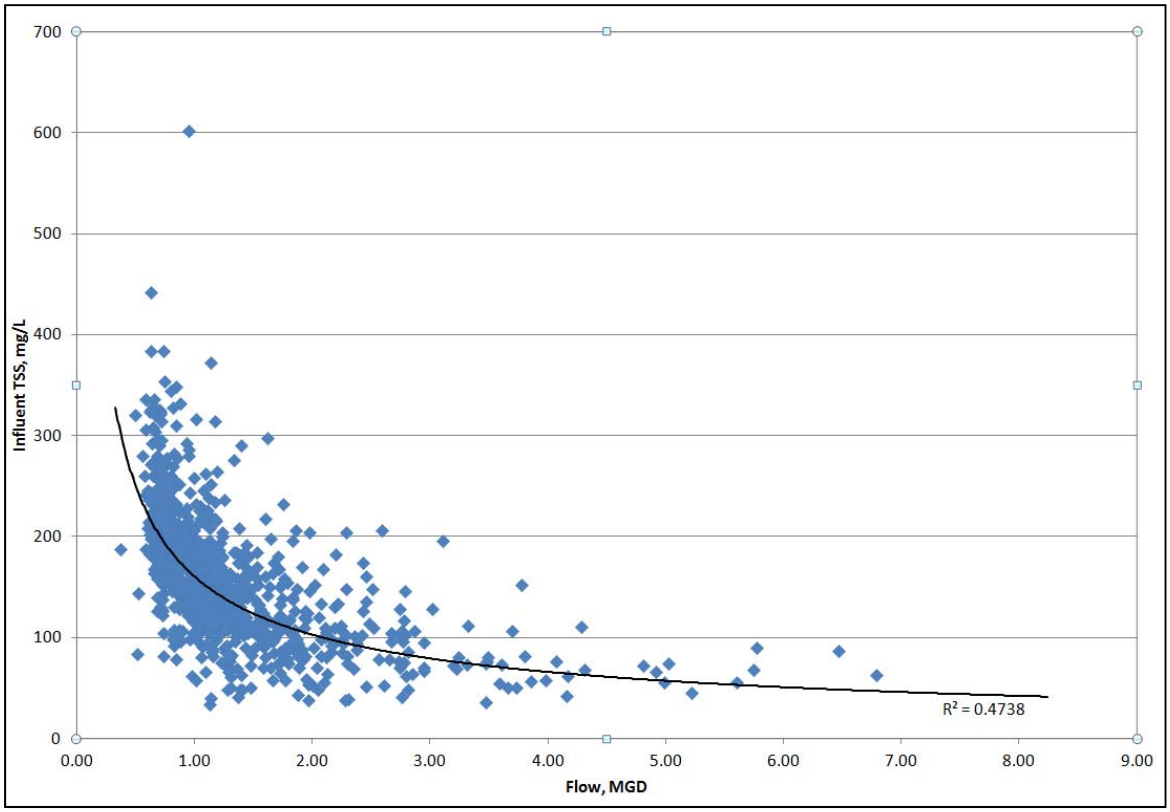


FIGURE 2-9: INFLUENT TSS CONCENTRATION VERSUS FLOW



So, knowing that influent flow impacts the percent removal that can be achieved, Figure 2-10 below compares the impact of the severity of a single high flow event on the monthly average percent removal. Figure 2-10 displays the highest daily flow for a given month versus the percent removal of TSS for that month. Although there is a slight correlation (r-squared value of 0.1792), the trend is more pronounced at the lower flow values, where highest daily flow for that month is closest to the average flow for the month. For months with the highest daily flow value above about 3 MGD, there appears to be no noticeable trend. This indicates that the occurrence of a single high flow event generally has little impact on the percent removal that is achieved. Even comparing monthly average flow with percent removal, there was not a strong correlation, as evident from Figures 2-5 and 2-6 above. From this discussion, it is possible to conclude that there are other factors besides flow that influence percent removal of TSS.

Percent removal of CBOD has consistently exceeded 94% since startup of the SFF media system was completed. Therefore, further analysis of CBOD percent removal is not warranted.

Since installation and startup of the SFF media system, there have been two occurrences of the monthly average TSS removal not meeting the 85 percent minimum. In both of these cases, there was a significant drop in the effluent temperature immediately preceding a spike in effluent TSS (see Figure 2-11 below). This sudden drop in temperature created density currents that led to turnover in the lagoon and re-suspension of settled sediments. Re-suspension of sediment from lagoon turnover has been more pronounced since installation and startup of the SFF media system, because the subsequent redistribution of the sediment from startup of this system has resulted in sediment that is less compacted and more easily re-suspended. The City is in the process of planning for removal of the sediment, which should alleviate this issue.

FIGURE 2-10: MONTHLY AVERAGE TSS PERCENT REMOVAL VERSUS MONTHLY PEAK DAY FLOW

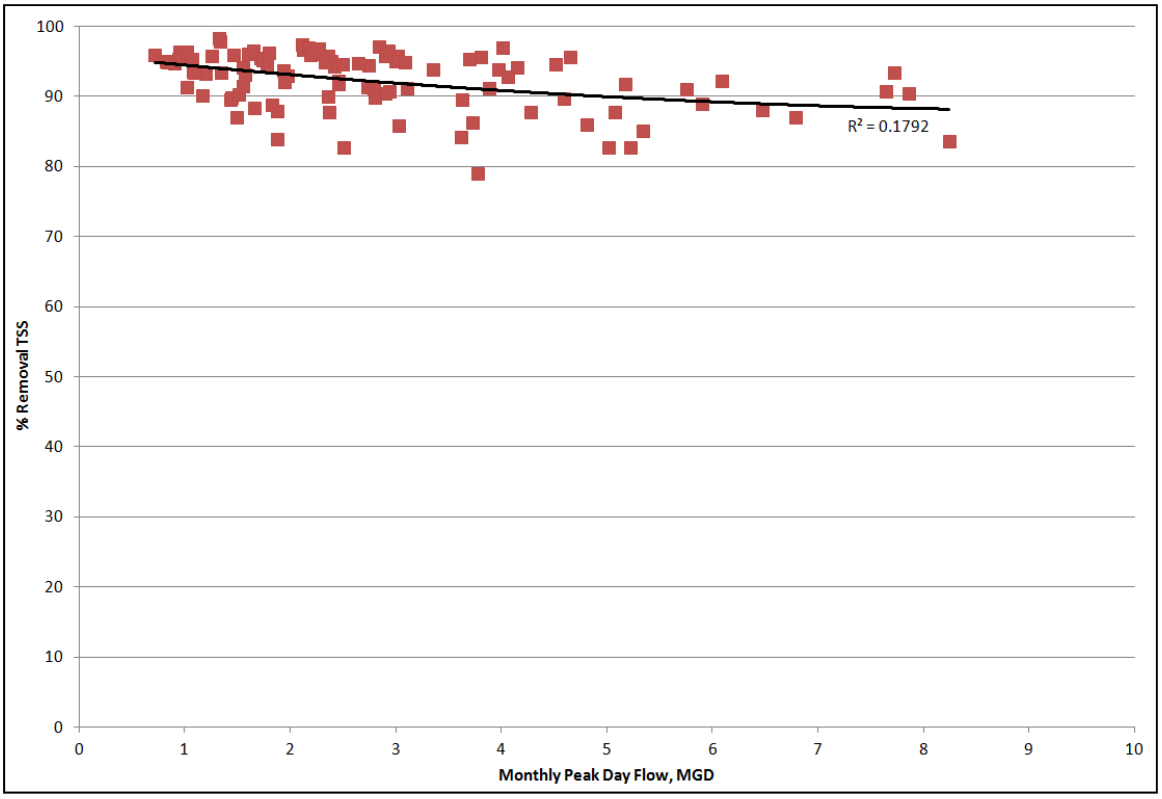
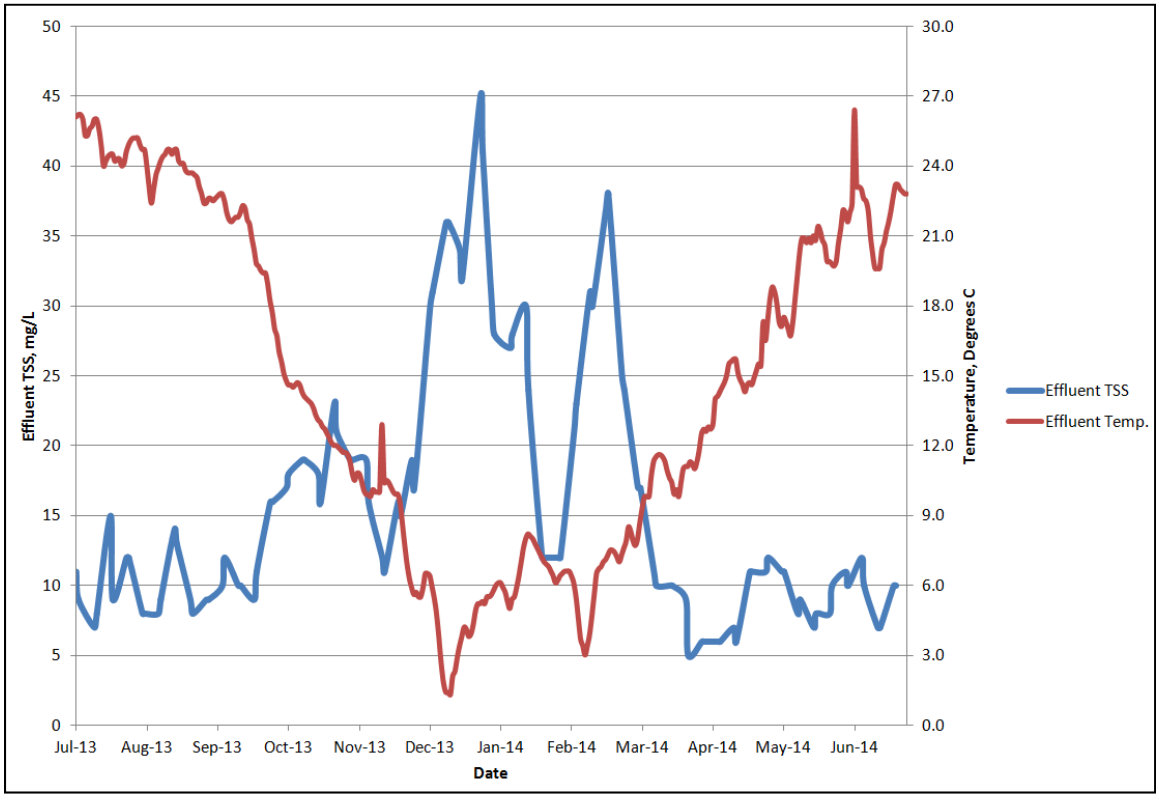
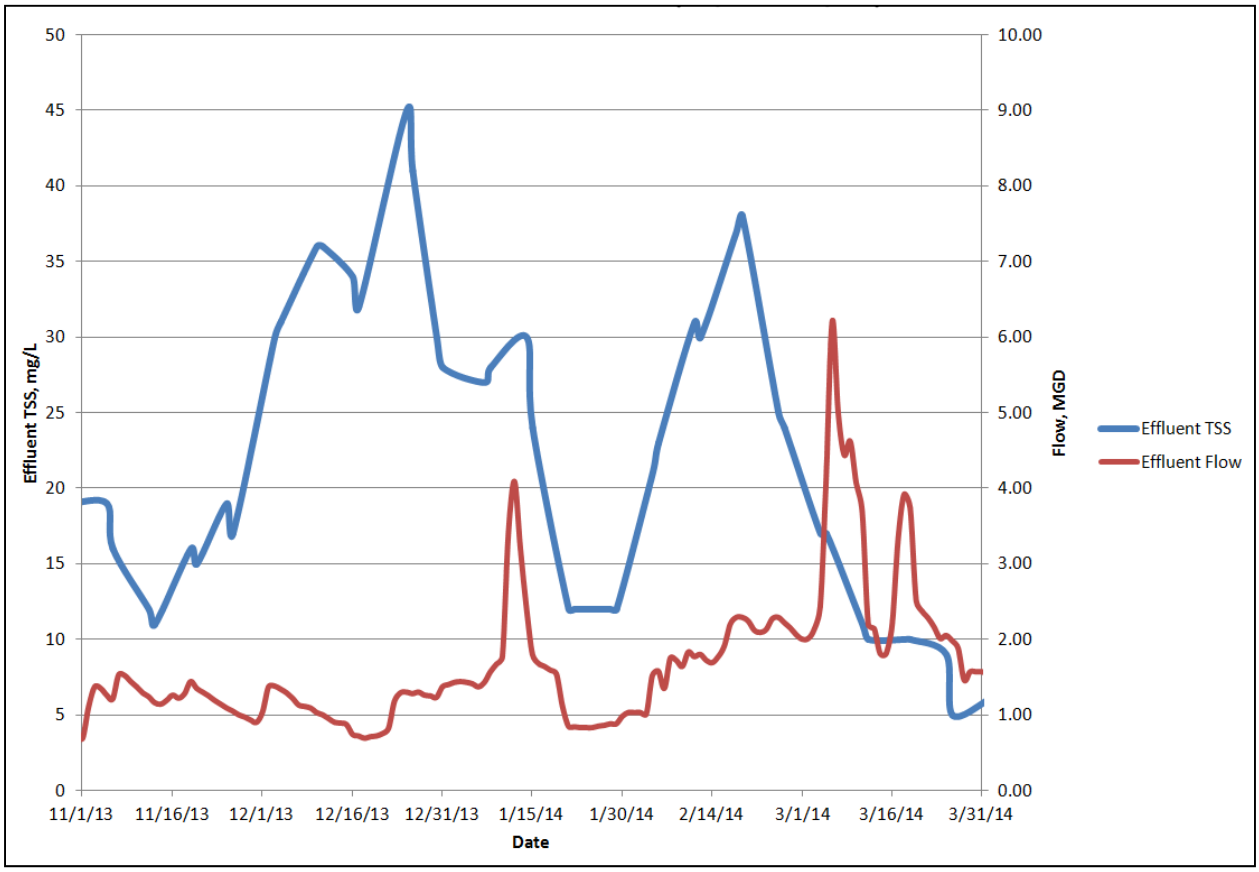


FIGURE 2-11: EFFLUENT TSS CONCENTRATION AND EFFLUENT TEMPERATURE



For comparison, a plot of effluent flow versus effluent TSS is provided in Figure 2-12 covering the same critical period and shows no correlation between flow (and contributing rainfall and infiltration/inflow) and effluent TSS. Thus, the conclusion is that temperature and subsequent density currents are the primary contributing factor to elevated TSS concentrations in the effluent.

FIGURE 2-12: EFFLUENT TSS CONCENTRATION AND EFFLUENT FLOW



In addition to impacts from lagoon turnover, it has been documented that, unlike domestic wastewater, storm water contains a wider range of particle sizes and densities (Lenhart, 2007). So, this means that storm water can contain more particles that do not readily settle compared to domestic wastewater. If that is the case, a higher component of storm water (such as is the case for high flows) means a lesser percent removal of TSS and possibly CBOD (if particles are organic in nature) can be achieved. So, not only does the more dilute concentration at high flows make achieving a certain percent removal more difficult, but the influent particles can also be more difficult to settle and remove.

WWTP HYDRAULIC CAPACITY

A summary of the hydraulic capacity of each unit process is provided in the following paragraphs. One basis of comparison is projected future flows and loads, which are summarized in Table 2-2 below. These projected future flows are from revised flow projections included in the Draft City of Snohomish General Sewer Plan and Wastewater Facilities Plan Amendment (Kennedy/Jenks, 2013). A second basis of comparison is the permitted influent flows and loads, from Section S4.A of the current NPDES permit, which are as follows:

- Influent Maximum Month Flow 2.8 MGD
- Influent Maximum Month CBOD Loading 3,960 lbs/day
- Influent Maximum Month TSS Loading 4,400 lbs/day

TABLE 2-2
PROJECTED 2033 WASTEWATER FLOWS AND LOADS

2033 Influent	Flow (MGD)	CBOD (lbs/d)	TSS (lbs/d)	TKN (lbs/d)
Average Annual Flow (AAF)	1.90	3,042	2,358	350
Maximum Month Flow (MMF)	2.79	3,832	3,278	456
Peak Day Flow (PDF)	8.13	6,609	6,906	866
Peak Hour Flow (PHF) (Total)	22.10			
Peak Hour Flow (PHF) (Gravity Only)	14.70			
Average Dry Weather Flow (ADWF)	1.23			
Maximum Month Dry Weather Flow (MMDWF)	1.89			
Peak Day Dry Weather Flow (PDDWF)	5.44			

Influent Flow Measurement

Flow is received at the WWTP through a 30-inch gravity sewer and an 18-inch force main conveying flow from the CSO Pump Station. Flow conveyed through the force main is received just upstream of influent screening, whereas gravity flow is received at a lower elevation and must be pumped up to the influent screening. The gravity flow is measured using an 18-inch Parshall flume, which has a capacity of 25 MGD. Of the projected 2033 PHF of 22.10 MGD, approximately one-third (7.4 MGD) is expected to be conveyed through the 18-inch force main and the remaining two-thirds (14.7 MGD) is expected to be received through the 30-inch gravity sewer. The 14.7 MGD expected to be received through the gravity sewer is well within the capacity of the 18-inch Parshall flume. Flow through the force main is measured by a 16-inch magnetic flow meter located at the CSO Pump Station.

Influent Pumping

Three influent screw pumps are used to convey gravity sewer flow up to the influent screening. Two of the screw pumps are 100 horsepower pumps, each with a capacity of 15.5 MGD. The third is a smaller 25 horsepower pump with a capacity of 3.5 MGD. The firm capacity of the influent pumps with one large pump out of service is 19.0 MGD, which provides sufficient capacity to convey the expected gravity sewer portion of the projected 2033 PHF.

Influent Screening

The influent screen is a mechanical multi-rake bar screen with ¼-inch clear spacing between the bars. The screen has a capacity of 23.5 MGD, which is sufficient to handle the projected 2033 PHF. Additionally, there is a manual bar screen with ¾-inch clear spacing between the bars. The manual bar screen can be utilized when the mechanical screen is out of service for maintenance.

Supplemental Alkalinity Dosing

Following screening, influent wastewater is dosed with supplemental alkalinity in the form of magnesium hydroxide. The addition of supplemental alkalinity ensures sufficient alkalinity exists to buffer pH as alkalinity is consumed during nitrification. Without supplemental alkalinity, there can be too little alkalinity remaining and pH will drop and inhibit further nitrification. Inhibition of nitrification can also lead to accumulation of nitrite and the persistence of nitrite-lock in the disinfection process. Nitrite-lock is a condition where ammonia is converted to nitrite, but full nitrification is not completed to convert the nitrite to nitrate. As a result, the chlorine used for disinfection oxidizes the nitrite to nitrate. This can substantially increase the required chlorine dose and destabilize the disinfection process.

Historically, the WWTP has dosed approximately 75 mg/L of magnesium hydroxide to maintain the effluent pH near 7.0. The alkalinity pumping rate is flow-paced to maintain a consistent dose of 75 mg/L. There are two alkalinity pumps, one duty and one standby. Each pump has a capacity to pump 35 gallons per hour (840 gallons per day each, or 1,680 gallons per day total). The pumps were not designed to have capacity to dose 75 mg/L at the projected 2033 PHF. Instead, they were sized to maintain the dosing set point over a 24-hour period, since short fluctuations in the dosing would not yield a significant change in effluent pH. Additionally, pumps large enough to meet the dosing set point for PHF would not have sufficient turndown to operate effectively at the much lower flows that are typical. At the projected 2033 PDF of 8.13 MGD, 662 gallons per day of magnesium hydroxide would need to be pumped into the influent, which is within the capacity of an individual alkalinity pump. A single pump could accommodate a PDF of up to 10.3 MGD at a dose of 75 mg/L.

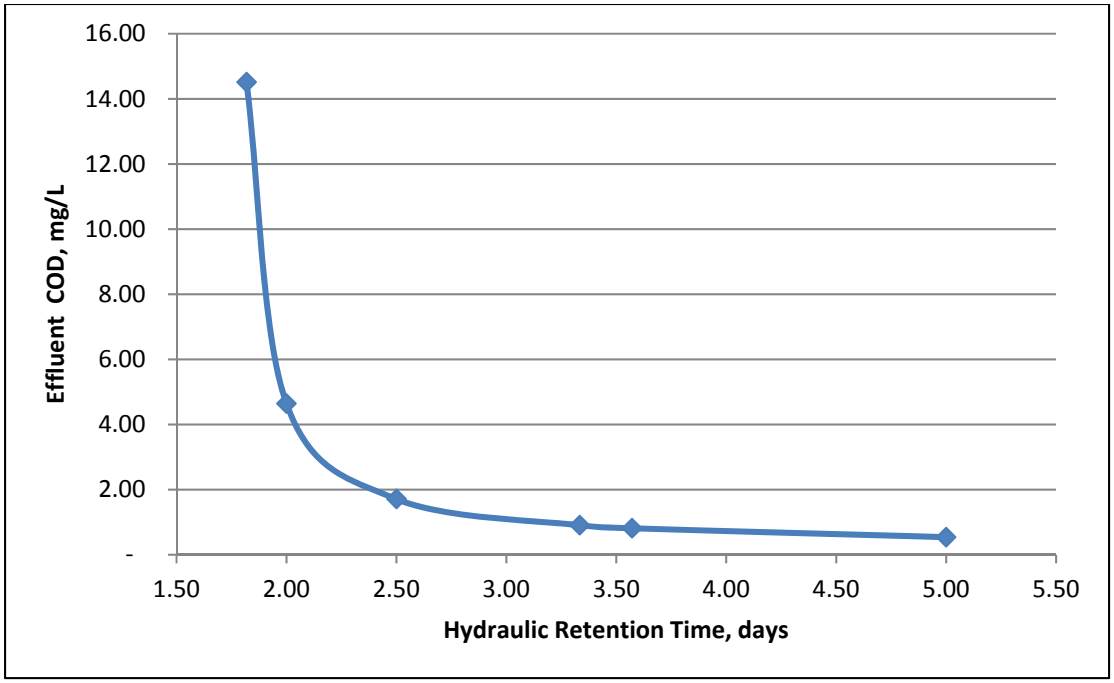
DPMC Aerated Lagoons

After screening and dosing supplemental alkalinity, the influent wastewater is conveyed to the treatment lagoons. As mentioned previously, the DPMC aerated lagoons consist of a complete mixed, suspended growth lagoon with a normal volume of approximately 10.0 million gallons followed by three partially mixed, facultative lagoons in series with a normal volume of approximately 3.5 million gallons each. These volumes are provided at an operating depth of 10 feet.

Complete Mixed Lagoon

There are two primary hydraulic considerations regarding sizing and capacity of the complete mixed lagoon. The first is to ensure that sufficient volume is available at MMF to treat the maximum month loads to meet the permit requirements for CBOD concentrations in the effluent and CBOD percent removal. The second is that there is sufficient volume to prevent washout of the suspended growth biomass at peak flows. The design methodology for DPMC aerated lagoons put forth by Linvil Rich (Rich, 1982 and 1996) are used to evaluate these two conditions. An evaluation of the theoretical sizing based on treatment of CBOD using Rich's methodology is summarized in Figure 2-13 below. Detailed calculations are presented in Appendix A.

FIGURE 2-13: EFFLUENT COD CONCENTRATION VERSUS HYDRAULIC RETENTION TIME



Rich's methodology uses chemical oxygen demand (COD) instead of CBOD for sizing of the complete mixed lagoon (Rich, 1996). For domestic wastewater, the COD:CBOD ratio is typically 1.9 to 2.2. For this evaluation a ratio of 2.1 was used. As demonstrated in Figure 2-13, the effluent COD concentration rises sharply with a hydraulic retention time less than 2.5 days, suggesting this is the practical limit for hydraulic retention time. With a volume of 10 million gallons, a minimum hydraulic retention time of 2.5 days translates to a maximum monthly flow of 4.0 MGD, well in excess of the permitted capacity of 2.8 MGD. The fact that CBOD removal has historically averaged over 95 percent suggests that this capacity assessment is reasonable. Variations in the historical performance are likely due primarily to the presence of algae or re-suspension of sediment that are at times generated within the process and contribute to effluent CBOD, but that were not part of the influent waste load. It should be noted that this assessment of capacity does not account for contributions of the SFF media system to CBOD removal.

Regarding acceptable peak flow to avoid washout of the suspended growth biomass, Rich's sizing methodology (Rich, 1982) calculates a critical hydraulic retention time of 1.0 days. This translates to an allowable PDF of 10.0 MGD, which provides sufficient capacity for the projected 2033 PDF. PHF is not considered, because the volume of the lagoon is such that hourly peaks are readily attenuated.

Partially Mixed Lagoons

The primary hydraulic consideration for the partially mixed lagoons is providing sufficient volume and surface area for solids settling. The partially mixed lagoons must have sufficient surface area so that the typical overflow rate is low enough to allow for removal of settleable solids. Also, the depth must be sufficient to allow for sufficient separation between the sediment and water surface, plus account for storage of sediment. The normal 10-foot (3-meter) operating depth factors in 3 feet for storage of sediment and 7 feet of water column and is consistent with depths recommended by Rich (Rich, 1996). Regarding surface area, Rich indicates that a

surface overflow rate of 75 gallons per day per square foot (gpd/sf) is sufficient to remove most, if not all, settleable solids (Rich, 1982). A single partially mixed lagoon can maintain a surface overflow rate of 75 gpd/sf or less (based on area at mid-depth) at flows up to 3.6 MGD, which is in excess of the projected 2033 MMF and permitted MMF. Considering all three partially mixed lagoons in series, assuming settleable solids do not get moved further up into the water column as flow passes between lagoons, an overflow rate of 75 gpd/sf could be maintained at flows up to 10.7 MGD, which is in excess of the projected 2033 PDF. In actuality, the pipeline velocities at the PDF are such that there would likely be some currents generated out the outlet to the downstream lagoon, such that particles may move back up into the water column upon entry into the next lagoon. That affect could be mitigated by using both outlets between Lagoons 2 and 3 and making use of the overflow pipes at each outlet as well, allowing pipeline velocities to remain around 1 foot per second. It could also be mitigated by operating the partially mixed lagoons in parallel, but this configuration is not readily supported by the existing layout, nor is it the most efficient mode of operation under most other circumstances. However, as discussed above, the impact of a single occurrence of high flow is generally not significant in achieving permit compliance.

SFF Media Modules

The SFF media modules located in the partially mixed lagoons are sized based on a peak ammonia load 620 pounds per day during the critical dry weather months (July through October). This loading is based on a peak day dry weather flow (PDDWF) of 2.75 MGD and a maximum influent ammonia concentration of 27 mg/L. The number of SFF media modules installed is sufficient to accommodate this load, plus a 30 percent safety factor. So, the total theoretical load is 806 pounds per day.

Although more recent flow measurements since design of the SFF media system has revised the PDDWF upward to 5.44 MGD, the total ammonia load capacity of the SFF media modules is still more than sufficient to treat current and projected loads. This is because as flows increase, the concentration of ammonia drops, since the majority of the high flows is storm water. For flows in excess of 1.0 MGD, the influent ammonia concentrations are consistently below 20 mg/L and are below 10 mg/L at flows above 2.0 MGD. For example, the influent ammonia concentration at a total daily flow of 4.91 MGD was 4 mg/L. Since the addition of the SFF media modules, the effluent ammonia has been consistently well below 1 mg/L during the critical dry weather months.

The average ratio of ammonia to total Kjeldahl nitrogen (TKN) is 0.6. Given the projected 2033 peak day TKN load is 866 pounds per day, the associated ammonia load would be 520 pounds per day, which is well within the capacity of the SFF media modules. Although some of the TKN will hydrolyze and form ammonia, there is sufficient excess ammonia removal capacity to accommodate this. The highest ammonia load measured to date has been 281 pounds per day. As a result, excess capacity is being used to enhance CBOD removal. The Draft City of Snohomish General Sewer Plan and Wastewater Facilities Plan Amendment (Kennedy/Jenks, 2013) suggested that excess capacity of the SFF media modules, when considering the peak day ammonia load, equates to 800 to 1,600 pounds per day of CBOD removal, or approximately 0.6 to 1.1 MGD of additional CBOD loading capacity under the projected maximum month CBOD concentration of approximately 170 mg/L.

Sand Filters

The sand filters are designed to treat only a portion of the lagoon effluent. They currently have capacity to treat approximately 0.8 MGD, though with some relatively minor modifications (i.e., replacement of the existing static mixer and upgrade of the reject pumps) they are

expected to be able to treat up to 1.4 MGD. The City is targeting these modifications to be made in 2015. Any flow that is not treated through the sand filters is conveyed directly to the disinfection process.

Disinfection

Lagoon effluent and filtered effluent enter a chlorine mixing manhole where chlorine gas is mixed with the effluent using an induction mixer. The rate of chlorine dose is currently not limited by the size of the induction mixer or the size of the chlorinators controlling the chlorine dose, but rather by the number of 150-pound chlorine cylinders that are connected to the chlorinator. The induction mixer has a capacity of 500 pounds per day and the two chlorinators have a total capacity of 200 pounds per day. There are four cylinders currently connected and up to 40 pound per day can be withdrawn from each, for a total chlorine dosing capacity of 160 pounds per day.

The historical chlorine dosing rate has been 1.5 mg/L during wet weather and 3.0 mg/L during dry weather. The chlorine dose is lower during the wet weather, because effluent ammonia concentrations are higher due to lower nitrification rates and cycled aeration of the SFF media modules and so chlorine can combine with the ammonia to form the disinfectant monochloramine. However, when effluent ammonia is low during the dry weather months due to near complete nitrification in the lagoons and little monochloramine is formed, more chlorine must be added. The additional chlorine first reacts with organic-nitrogen that is present to produce organo-chloramines, which essentially have no disinfection power. Once all of the organics are oxidized with chlorine (i.e., the breakpoint has been reached), further addition of chlorine will result in a free chlorine residual for disinfection. Based on these dosing rates, the required chlorine dose is 102 pounds per day at the projected 2033 PDF and 136 pounds per day at the projected 2033 PDDWF, which is within the capacity of the current system. Based on these dosing rates, a peak wet weather flow of approximately 12.8 MGD could be accommodated.

After dosing of chlorine, the chlorinated effluent is conveyed into a chlorine contact tank to allow time for the effluent to be disinfected prior to discharge. The chlorine contact tank normally has an active volume of approximately 123,000 gallons. During flood conditions in the Snohomish River, the level in the contact tank can rise such that the volume is increased to approximately 166,200 gallons. Ecology's design guideline for chlorine contact tanks is to provide a minimum 20 minutes of contact time at peak flows. At the normal operating volume, this translates to a peak flow not exceeding 8.9 MGD, which is greater than the projected 2033 PDF.

At the end of the chlorine contact tank is a small chamber where sulfur dioxide gas is mixed with the effluent for dechlorination prior to discharge. The sulfur dioxide dosing system is configured much the same as the chlorine dosing system. A total of four 150-pound cylinders are connected to the sulfonator, which has a capacity of 90 pounds per day. The City is in the process of adding a second sulfonator of equal capacity. The sulfur dioxide gas is mixed with the effluent using an induction mixer having a capacity of 1,500 pounds per day. So, with the installation of the second sulfonator, the limiting capacity will be 160 pounds per day based on the four cylinders, same as the chlorine system.

Currently, the City manually sets the dosing of sulfur dioxide at the same rate as chlorine to ensure sufficient dechlorination. Based on the current practice of manual dosing, the sulfur dioxide system would see the same demands as the chlorine system. However, the City is in the process of automating dosing of chlorine and sulfur dioxide, such that the sulfur dioxide dose will be able to track with the actual measured residual. This will result in a lower demand

and even greater capacity for the sulfur dioxide system. It is estimated that with automated control the peak demand for sulfur dioxide will be approximately 70 pounds per day.

Effluent Flow Measurement and Discharge

A V-notch weir at the end of the chlorine contact tank is used to measure effluent flow. The 120 degree and 30-inch deep V-notch has capacity to measure flows up to about 27.5 MGD, which is well in excess of the projected 2033 PDF.

Effluent is discharged to the Snohomish River through a 30-inch outfall pipeline. Hydraulic analyses performed during design of the SFF media system concluded that the outfall could accommodate a peak day flow of approximately 8.8 MGD at the 25-year flood elevation, which was reported in the 1995 WWTP design documents (NAVD 88 elevation of about 25.75 feet). A flow rate of 8.8 MGD was approximately the maximum flow that could be achieved at that river elevation without flooding back into the headworks through the precast overflow trenches, though the internal lagoon spillways and V-notch weir at the end of the chlorine contact tank would be submerged. At the normal mean high-high water level in the river, there is no submergence of the spillways or V-notch weir. In either case, the outfall capacity is greater than the projected 2033 PDF. Historically, there have been two occasions of extreme flooding when the river level has prevented discharge by gravity. One occurred in November 2006 (peak day during that month was 4.6 MGD) and the other in January 2009 (peak day of 5.2 MGD for the month). There have been many days with higher peak flows, so river elevation has a significant impact on capacity of the outfall. The City overcomes the limitation of reduced outfall capacity during high river elevation by implementing portable pumping when needed to increase effluent flow.

WWTP Hydraulic Capacity Summary

A summary of the unit process capacities as discussed above is provided in Table 2-3 below.

TABLE 2-3
WWTP HYDRAULIC CAPACITY SUMMARY

Projected 2033 Influent Flows		
PHF (Total)	22.10	MGD
PHF (Gravity Only)	14.70	MGD
PDF	8.13	MGD
MMF	2.79	MGD
Influent Flow Measurement		
Type	Parshall Flume	
Number	1	
Size	18	inches
Capacity	25.0	MGD
Projected 2033 PHF	14.7	MGD
Sufficient Capacity?	Yes	
Influent Pumping		
Type	Screw	
Number Large Pumps	2	
Number Small Pumps	1	
Size Large Pumps	100	HP

TABLE 2-3
WWTP HYDRAULIC CAPACITY SUMMARY

Size Small Pump	25	HP
Large Pump Capacity, Each	15.5	MGD
Small Pump Capacity, Each	3.5	MGD
Total Firm Capacity	19.0	MGD
Projected 2033 PHF	14.7	MGD
Sufficient Capacity?	Yes	
Influent Screening		
Type	Multi-Rake Bar	
Number	1	
Opening Size	1/4	inches
Capacity	23.50	MGD
Projected 2033 PHF	22.10	MGD
Sufficient Capacity?	Yes	
Supplemental Alkalinity		
Slurry Unit Weight	12.8	lbs/gal
% Magnesium Hydroxide in Slurry	60%	
Historical Dosing Rate	75	mg/L
Dose at Projected 2033 PDF	5,085	lbs/day
Pumping Rate at Projected 2033 MMF	662	gpd
Number of Alkalinity Pumps	2	
Type of Alkalinity Pumps	Hose	
Pump Capacity, Each	840	gpd
PDF at Pump Capacity	10.3	MGD
Sufficient Capacity?	Yes	
Complete-Mix Lagoon		
Number	1	
Volume at Normal Level	10	MG
HRT at 2033 MMF	3.6	days
Min. HRT Required at MM CBOD	2.5	days
MMF at Min. HRT and MM CBOD	4.0	MGD
HRT at 2033 PDF	1.23	days
Min. HRT to Avoid Washout	1.0	days
Sufficient Capacity?	Yes	
Partial-Mix Lagoons		
Number	3	
Volume at Normal Level, EA	3.5	MG
Area at Mid-Depth per Lagoon	47,488	sf
Total Area at Mid-Depth	142,464	sf
Recommended Max Overflow	75	gpd/sf
Overflow per Lagoon at 2033 MMF	59	gpd/sf

TABLE 2-3
WWTP HYDRAULIC CAPACITY SUMMARY

MMF at 75 gpd/sf per Lagoon	3.6	gpd/sf
Overflow all Lagoons at 2033 PDF	57	gpd/sf
PDF at 75 gpd/sf all Lagoons	10.7	MGD
Sufficient Capacity?	Yes	
SFF Media Modules		
Number	54	
Design Ammonia Removal Capacity	620	lbs/day
Ammonia Capacity w/ Safety Factor	806	lbs/day
Projected 2033 Peak Day Ammonia Load	520	lbs/day
Historical Peak Ammonia Load	281	lbs/day
Sufficient Capacity?	Yes	
Sand Filters		
Number	2	
Capacity, EA	0.4	MGD
Total Capacity	0.8	MGD
Sufficient Capacity?	N/A	
Chlorine Contact Tank		
Number	1	
Volume at Max Level	166,200	gallons
Volume at Normal PDF Level	123,000	gallons
Minimum HRT	20	minutes
Flow at Min HRT & Normal PDF Level	8.9	MGD
Projected 2033 PDF	8.13	MGD
Sufficient Capacity?	Yes	
Chlorine Dosing		
Historical Wet Weather Dosing Rate	1.5	mg/L
Historical Dry Weather Dosing Rate	3.0	mg/L
Dose at Projected 2033 PDF	102	lbs/day
Dose at Projected 2033 PDDWF	136	lbs/day
Maximum Available Dose	160	lbs/day
PDF at Wet Weather Dosing Rate	12.8	MGD
Sufficient Capacity?	Yes	
Sulfur Dioxide Dosing		
Historical Wet Weather Dosing Rate	1.5	mg/L
Historical Dry Weather Dosing Rate	3.0	mg/L
Dose at Projected 2033 PDF	102	lbs/day
Dose at Projected 2033 PDDWF	136	lbs/day
Maximum Available Dose	160	lbs/day
PDF at Wet Weather Dosing Rate	12.8	MGD
Sufficient Capacity?	Yes	

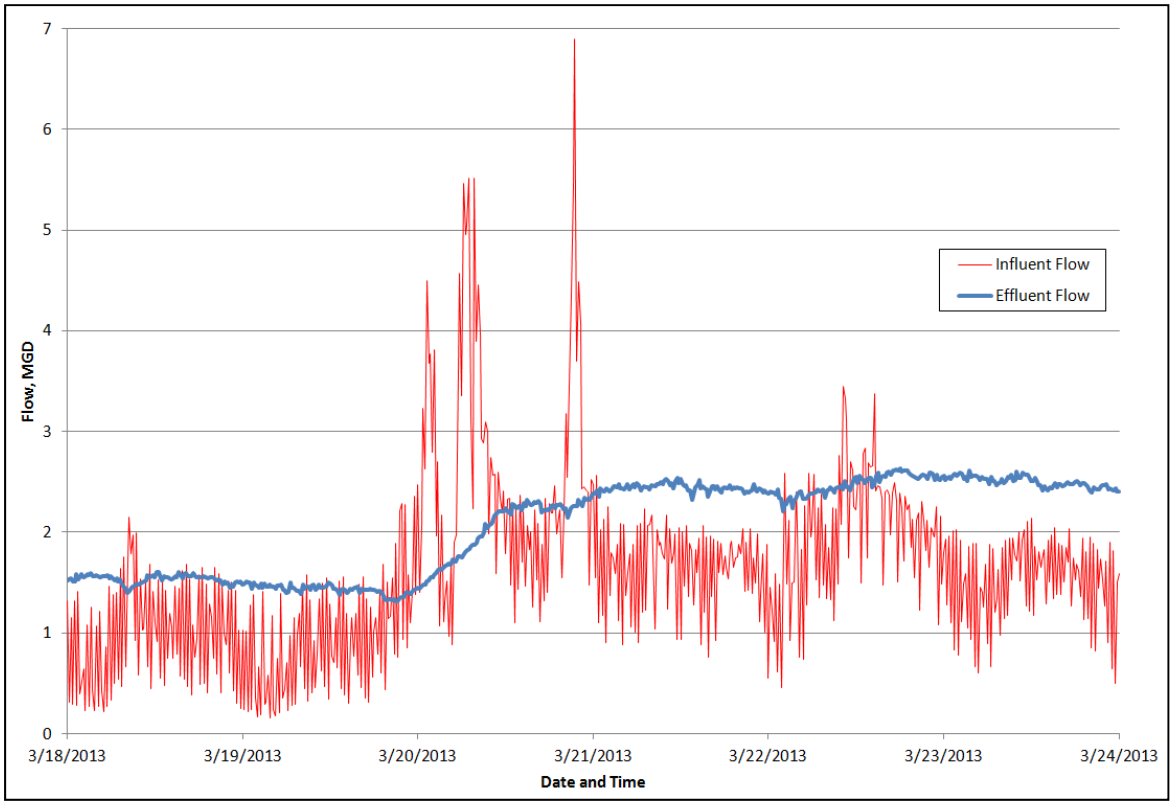
TABLE 2-3
WWTP HYDRAULIC CAPACITY SUMMARY

Effluent Flow Measurement		
Type	V-Notch	
Number	1	
Height	2.5	feet
Angle	120	degrees
Capacity	27.50	MGD
Projected 2033 PDF	8.13	MGD
Sufficient Capacity?	Yes	
Outfall		
Number	1	
Diameter	30	inches
Number of Ports	4	
Capacity	8.8	MGD
Sufficient Capacity?	Yes	

Estimate of Hydraulic Capacity

As mentioned previously, the lagoons are operated below their maximum water levels, providing excess storage capacity to attenuate hourly peak flows. The attenuation is sufficient that downstream processes need only consider PDF, and not PHF, in terms of hydraulic capacity. Unit processes upstream of the lagoons must still consider PHF when evaluating hydraulic capacity since the attenuation occurs in the lagoons. A comparison of influent and effluent flow measurements at 15-minute intervals for a one week time period in March 2013 is shown in Figure 2-14 to demonstrate the flow attenuation that occurs between the influent and effluent flow. As effluent flow is shown to be generally higher on average than the influent flow, this represents a period where the WWTP staff was actively lowering the levels in the lagoons to provide more attenuation volume.

FIGURE 2-14: ATTENUATION BETWEEN INFLUENT AND EFFLUENT FLOW



The PHF is limited to the capacity of the mechanical screen, which is 23.5 MGD. This available capacity is greater than the projected 2033 PHF of 22.10 MGD. However, if the hydraulic capacity of the screen were to be exceeded, flow can also be routed through the manual bar screen. If stop logs isolating the manual bar screen are in place, excess flow can go over the stop logs and through the manual bar screen without overflowing the screening channels. There is also a third channel that will route flow past the screens if flows exceed the capacity of both the mechanical and manual bar screens. The influent sewage can flow through this third channel without overflowing the screening channels.

The PDF capacity of the WWTP is limited by the size of the chlorine contact tank, which has a capacity 8.9 MGD. The WWTP could sustain sufficient treatment at this flow rate even if there were limited excess volume in the lagoons to attenuate only hourly and diurnal peaks and this flow rate were to persist beyond a single day, in which case the daily influent flow and daily effluent flow would be equal (i.e., no net storage of flow over a 24-hour period). This available capacity is greater than the projected 2033 PHF of 8.13 MGD. Under flood conditions discharge through the outfall may be more limiting. When this occurs, the City uses portable pumps to discharge effluent to the river. When instantaneous flows exceed 8.9 MGD, attenuation volume must be used to control effluent flow. The storage volume available for attenuation in the lagoons may also be used at flows less than 8.9 MGD to optimize performance of the downstream disinfection and filtration processes.

The MMF capacity of the WWTP is limited by the size of the partial mixed lagoons, which have a capacity of 3.6 MGD. This is based on not exceeding a surface overflow rate of 75 gpd/sf in any one partially mixed lagoon, which will provide sufficient retention time to remove most, if not

all, settleable solids within each individual lagoon. This capacity exceeds the projected 2033 MMF of 2.79 MGD and the permitted MMF of 2.80 MGD. The estimate of the overall WWTP hydraulic capacity is summarized in Table 2-4 below.

TABLE 2-4
ESTIMATE OF WWTP HYDRAULIC CAPACITY

Flow Condition	Flow (MGD)	Limitation
Maximum Month Flow (MMF)	3.6	Partially Mixed Lagoons (Surface Overflow Rate)
Peak Day Flow (PDF)	8.9	Chlorine Contact Tank (Hydraulic Retention Time)
Peak Hour Flow (PHF)	23.5	Mechanical Screening Capacity

WWTP SAMPLING PROCEDURES

The WWTP staff currently collects 24-hour composite samples of the influent and effluent twice each week, in accordance with the NPDES permit requirements. The samples are routinely collected on Tuesday and Wednesday of each week, though this schedule may be modified due to holidays or maintenance activities at the WWTP. The predictability of this schedule allows for more efficient testing and processing of the samples. However, this schedule can lead to skewed results if high flow events happen to occur during one or more of the sampling periods. A high flow event that occurred over a 2 day period is representative of about 7 percent of a month, but could represent 25 percent of the sampling data set for that month if it coincided with the two sampling days. This could significantly skew the percent removal calculations so that they are not near what would actually be representative for the month. Therefore, the WWTP staff is considering testing TSS in particular on a more frequent basis, since it is felt that percent removal of TSS has been inaccurately skewed by the limited data set during certain months in the past. It may also be worth considering separating the two sampling events by a couple days so they are not back-to-back, perhaps then making them more representative of the average.

In addition to composite sampling, the WWTP staff also collects daily grab samples of effluent to analyze for total chlorine residual and pH and twice weekly (Monday and Wednesday) effluent grab samples to analyze for fecal coliform, in accordance with the NPDES permit. Additionally, online instrumentation continuously measures influent flow, effluent flow and effluent temperature, which are also required by the NPDES permit.

CHAPTER 3
CSO PUMP STATION EVALUATION

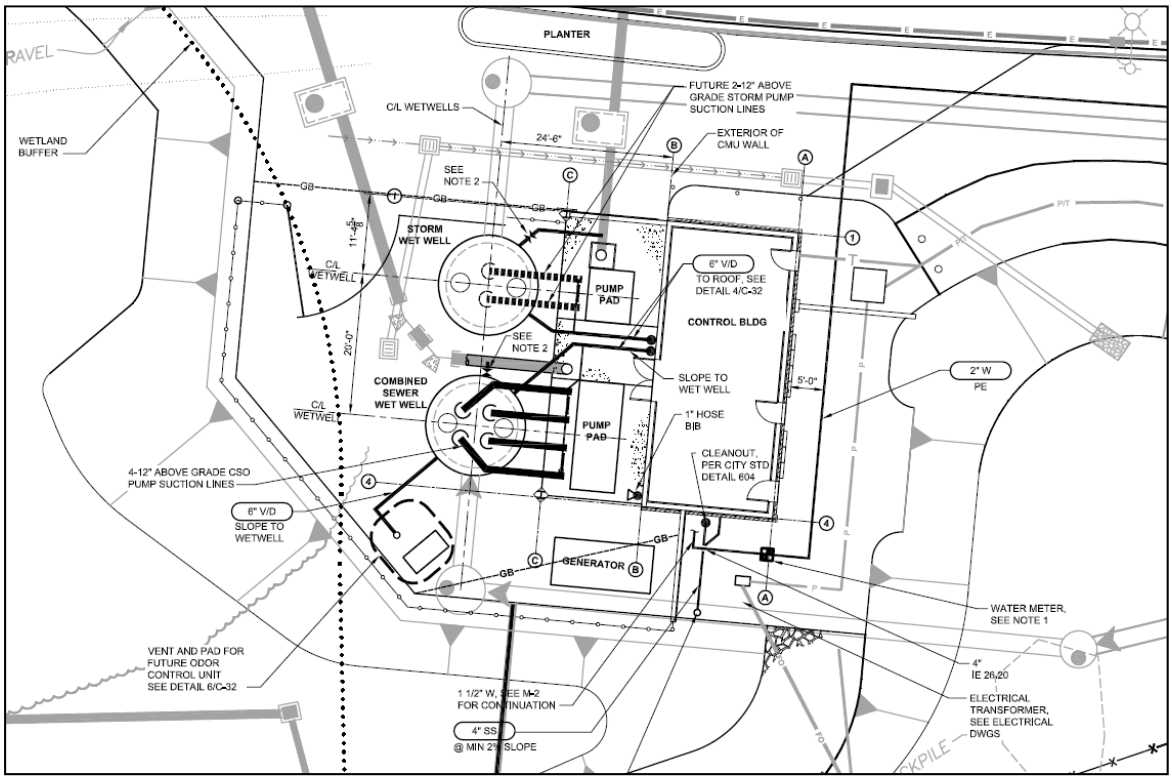
This Chapter describes the current configuration and operation of the CSO Pump Station, identifies relevant control elements of the pump station, identifies the associated storage volume and normal operating conditions of the pump station, and discusses how the storage volume and operating conditions change during storm events.

EXISTING CSO PUMP STATION

The CSO Pump Station began operation in September 2011. Prior to startup of the station, millions of gallons of combined sewage overflowed into the Snohomish River every year through two CSO outfalls (i.e., CSO #1 and CSO #2). Both CSO #1 and CSO #2 outfalls still exist. Control structures are in place to divert combined sewage to the CSO Pump Station. Only when flow of combined sewage reaches extreme levels does a portion of the flow get diverted to one or both of the CSO outfalls. Since start-up of CSO Pump Station, combined sewage has on average been diverted to the CSO outfalls less than once per year.

A layout of the CSO Pump Station is shown in Figure 3-1. The CSO Pump Station includes a 12-foot diameter and approximately 18.5-foot deep wet well; four above grade, self-priming, centrifugal pumps; interconnecting piping between the pumps and wet well; an 18-inch discharge force main; a tributary 30-inch gravity pipeline approximately 1,750 feet long; a control building and a standby generator. Each pump is connected to a variable frequency drive (VFD). The VFDs can be used to vary pump output with wet well level, but are normally used to allow soft start of the pumps, after which the pumps run at a set speed. There is a second wet well of equal size with space for two above grade, self-priming, centrifugal pumps that will be used in the future to pump storm water, once the tributary combined sewers are separated.

FIGURE 3-1: CSO PUMP STATION LAYOUT



CSO CONTROL ELEMENTS

Each of the two CSO outfalls is preceded by a hydraulic control structure to limit discharge of combined sewage. The control structure for CSO #1 is located south of the Avenue H and Second Street intersection. All flow passing through this control structure is conveyed by gravity to the WWTP and, therefore, does not contribute to CSO Pump Station flows. The majority of a storm water trunkline is already in place to convey storm water to a future wetland treatment system. The control structure will be modified in the future once the tributary storm water and sanitary sewer basins are separated and all storm water is conveyed to the treatment wetland. It should be noted that both the CSO #1 and CSO #2 outfalls already discharge some small amount of separated storm water collected from a few catch basins and drainage ditches.

A vault receives combined sewage immediately upstream of the CSO #1 outfall. A weir within the vault controls discharge of combined sewage through a 24-inch overflow pipeline to CSO #1. The top of the weir is set at elevation 40.15 feet. Two 18-inch sanitary sewer pipes convey combined sewage from the vault approximately 1,900 feet to a 30-inch pipeline that extends approximately 570 feet to the WWTP headworks. For the combined sewage to overflow the weir and discharge to the CSO #1 outfall, the 18-inch and 30-inch gravity pipelines downstream of the vault must be completely surcharged, which provides a total volume of approximately 72,000 gallons for flow attenuation. Because the City is not confident in allowing further surcharging, this is considered the maximum attenuation volume available. Because the elevation of this control structure is well above the Snohomish River flood elevation, there is no risk of backflow from the river into the combined sewer system through the CSO #1 outfall.

The 30-inch gravity pipeline conveying combined sewage to the CSO Pump Station is connected to the CSO #2 outfall by means of a 24-inch overflow pipeline located south of the Avenue E and First Street intersection. The invert of the overflow pipeline is at elevation 21.00 feet, which is above the crown of the 30-inch gravity pipeline tributary to the CSO Pump Station. Therefore, the tributary 30-inch pipeline must be completely surcharged prior to an overflow at CSO #2. This storage volume provides approximately 65,000 gallons for flow attenuation at the CSO Pump Station, including the wet well volume. The overflow pipeline includes a flap gate to prevent backflow from the Snohomish River into the 30-inch gravity pipeline and combined sewer system.

Level instruments in the CSO Pump Station wet well are used to control operation of the pumps. A summary of the control set points for the wet well are summarized in Table 3-1 below.

TABLE 3-1
CSO PUMP STATION WET WELL SET POINTS

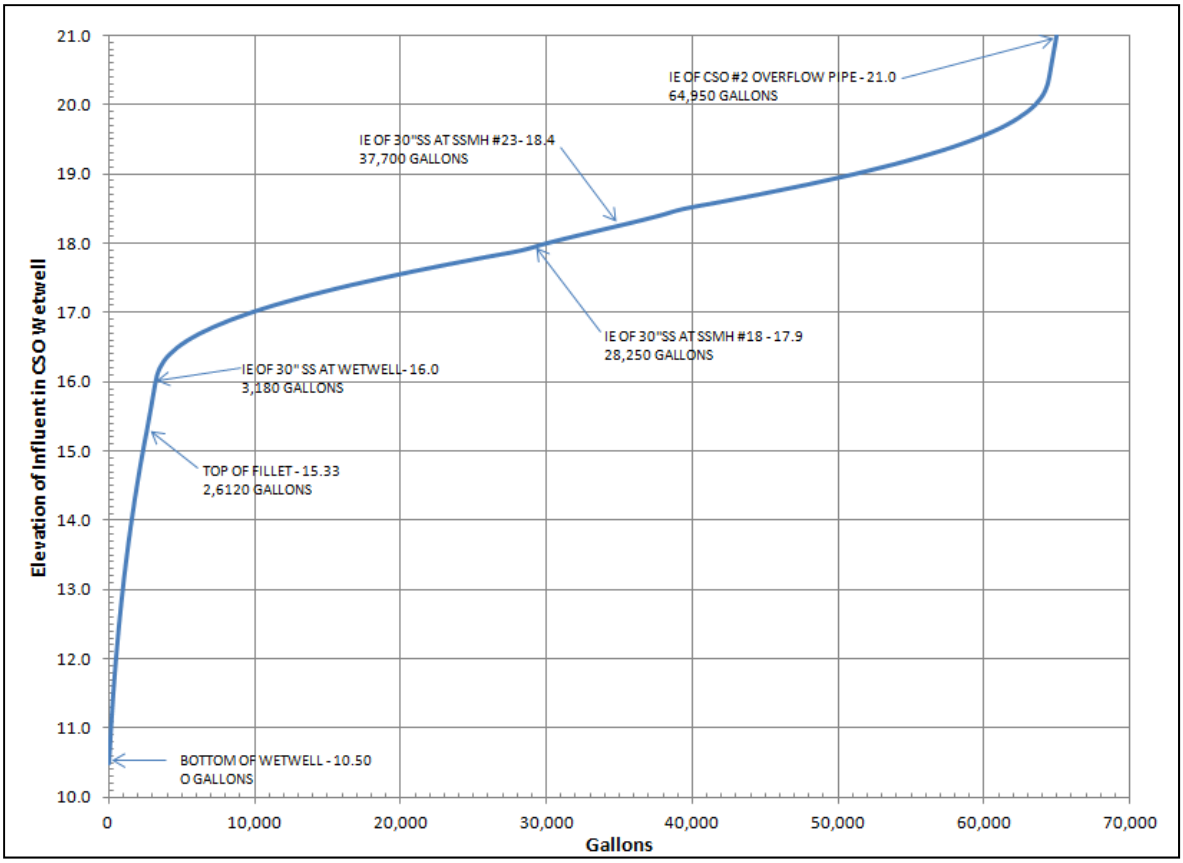
Control Set Point	Wet Well Depth	Elevation
CSO #2 Overflow	10.5 feet	21.0 feet
High-High Level Alarm and Backup Pump Start	8.0 feet	18.5 feet
High Level Alarm	7.6 feet	18.1 feet
Lag-Lag Pump Start	7.0 feet	17.5 feet
Lag Pump Start	6.5 feet	17.0 feet
Lag-Lag Pump Stop	6.0 feet	16.5 feet
Lead Pump Start	5.5 feet	16.0 feet
Lag Pump Stop	5.0 feet	15.5 feet
Lead Pump Stop	3.5 feet	14.0 feet
Low Level Alarm	3.3 feet	13.8 feet
Low-Low Level Alarm	3.2 feet	13.7 feet

Because much of the 30-inch pipeline tributary to the CSO Pump Station is at an elevation that overlaps with the active volume of the wet well, the pipeline provides additional storage volumes to attenuate flow conveyed to the pump station. As with piping tributary to CSO #1, it is assumed that no further surcharging of the system is allowed. A summary of the volume of both the wet well and pipeline between the primary control set points is shown in Table 3-2 below. Additionally, the relationship between stored volume and depth in the wet well is shown in Figure 3-2 below.

TABLE 3-2
CSO PUMP STATION STORAGE VOLUMES

Volume Description	Wet Well	30-inch Pipe	Total
Between Lead Pump Stop and Lead Pump Start	1,640 gallons	0 gallons	1,640 gallons
Between Lead Pump Start and Lag Pump Start	850 gallons	5,650 gallons	6,500 gallons
Between Lag Pump Start and Lag-Lag Pump Start	425 gallons	8,565 gallons	8,990 gallons
Between Lag-Lag Pump Start and Wet Well High-High Level	850 gallons	19,850 gallons	20,700 gallons
Total Between Wet Well High-High Level and CSO #2 Overflow	2,115 gallons	23,460 gallons	25,575 gallons
Total Between Lead Pump Stop and CSO #2 Overflow	5,880 gallons	57,525 gallons	63,405 gallons

FIGURE 3-2: STORAGE VOLUME VS. WET WELL DEPTH



CSO PUMP STATION OPERATION

The CSO Pump Station is operated based on level in the wet well. The four centrifugal pumps are designated as lead, lag, lag-lag and standby. These designations are automatically rotated to distribute runtime of the pumps. When the level in the wet well reaches the Lead Pump Start elevation, the lead pump is started initially at full speed to prime the pump, but then ramps down to a set operating speed (currently 72 percent of full speed or approximately 850 rpm). When the lead pump is running at or above the Lag Pump Start elevation for more than 30 seconds, the lag pump will be started initially at full speed, and then slowed to a set operating speed (currently 72 percent of full speed). If the lead and lag pumps are running at or above the Lag-Lag Pump Start elevation for more than 30 seconds, the lag-lag pump will be started initially at full speed, and then slowed to a set operating speed (currently 72 percent of full speed). If the level in the wet well reaches the High Level, then all four pumps are called to run via a hardwired interlock with a high level float switch and will remain running until the level drops to the Low Level and all pumps are called to stop via a hardwired interlock with a low level float switch.

Under normal operation, when the wet well level drops to the Lag-Lag Pump Stop elevation, the lag-lag pump is stopped and the lag and lead pump continue running at their set operating speed. When the wet well level drops to the Lag Pump Stop elevation, the lag pump is stopped and the lead pump continues to operate at its set operating speed. When the wet well level drops to the Lead Pump Stop elevation, the lead pump is stopped and the pump designations rotate. A summary of the range of pump speeds and associated discharge rates for the various level set points is provided in Table 3-3 below.

TABLE 3-3
CSO PUMP SPEEDS AND DISCHARGE RATES

Control Set Point	Set Operating Speed	No. Pumps Running	Total Discharge Rate
Lead Pump Start to Lag Pump Start	850 rpm	1	~2,350 gpm
Lag Pump Start to Lag-Lag Pump Start	850 rpm	2	~3,650 gpm
Lag-Lag Pump Start to High Level	850 rpm	3	~4,500 gpm
Backup Pump Start at High Level	850 rpm	4	~5,100 gpm
High Level to Lag-Lag Pump Stop	850 rpm	3	~4,500 gpm
Lag-Lag Pump Stop to Lag Pump Stop	850 rpm	2	~3,650 gpm
Lag Pump Stop to Lead Pump Stop	850 rpm	1	~2,350 gpm

With just sanitary sewage entering the CSO Pump Station, typically only the lead pump is required to operate on a periodic basis. When storm water is combined with the sanitary sewage, the lag and lag-lag pumps can be required to operate to convey the increased flow. Other than the number of pumps that are required to run and the frequency and duration with which they run, there is no difference in how the CSO Pump Station operates normally and during storm events that produce high volumes of combined sewage.

CHAPTER 4
HYDRAULIC MODELING OF THE COMBINED SEWER SYSTEM

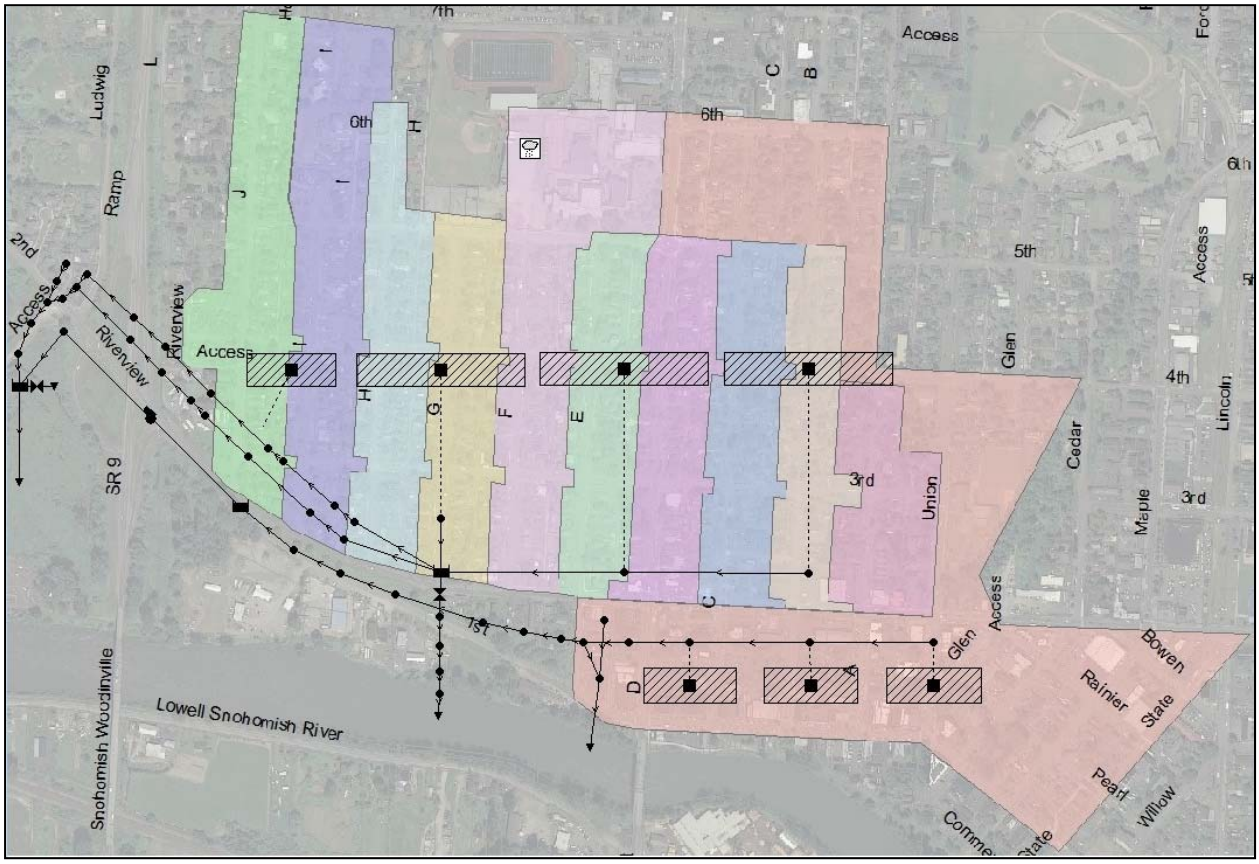
This Chapter provides an overview of model development, discusses model calibration, summarizes the modeling results based on simulating 27 years of historical rainfall and compares rainfall data used for model calibration with the original design storm.

MODEL DEVELOPMENT AND CALIBRATION

The Storm Water Management Model (SWMM), developed by the U.S. Environmental Protection Agency (EPA), has been selected for dynamic modeling of the combined sewer collection system. The latest version of EPA's SWMM (version 5.1) was chosen because of its relative ease of use given the amount and quality of data available. EPA's SWMM is a dynamic hydraulic model capable of simulating unsteady state (time-varying) flow conditions under both open channel (partially full pipe) and surcharged conditions. The dynamic model represents pipe systems mathematically as a series of nodes and links representing structures and piping, respectively. It combines two-dimensional runoff calculations from sub-catchment areas that simulate the natural rainfall-runoff processes with one-dimensional piping calculations simulating hydraulic performance of the combined sewer system.

The City provided drawings and maps that were used as the basis for constructing the dynamic model. Because sufficient information was not available on the complete network of pipes within the combined sewer system and to simplify the modeling effort, only the main combined sewer trunklines connecting the WWTP, CSO Pump Station and CSO outfalls were represented in the model (see Figure 4-1). The rest of the collection system is represented as a series of sub-catchment areas that feed into the trunklines. The characteristics of the sub-catchment areas entered into the model are reflective of the areas being represented. The storage volume for attenuation in the WWTP lagoons is also represented in the model with a maximum effluent flow of 8.9 MGD, representing the hydraulic limit identified in the WWTP hydraulic evaluation. As a result, any influent flow in excess of 8.9 MGD will utilize some of the attenuation volume in the WWTP lagoons. There are no limits set on the influent flow. As discussed previously, influent flow in excess of the mechanical screen capacity is temporarily routed through the manual bar screen.

FIGURE 4-1: CITY OF SNOHOMISH COMBINED SEWER SYSTEM HYDRAULIC MODEL



The three CSO events that have occurred since construction of the combined sewer system improvements and CSO Pump Station were completed (10/16/2012, 09/05/2013 and 09/03/2014) and associated rainfall and system flow data were used to calibrate the model. The first two CSO events had overflows that occurred at both outfalls. The most recent CSO event had an overflow only at the CSO #2 outfall. CSO flow data and rainfall data were provided by ADS Environmental Services (ADS), which monitors this information for the City. ADS monitors an electronic rain gauge near the CSO Pump Station and has installed flow monitoring equipment at both outfalls. Each outfall has a level transducer to measure water level in the combined sewer system and an in-pipe flow monitor that measures depth and velocity in the overflow pipe connected to the CSO outfall. Because there is a weir upstream of the CSO #1 outfall, the level transducer can also be used to calculate flow over the weir.

ADS provided rainfall data for months during which CSO events occurred, as well as a few additional months when CSO events did not occur. The City provided available flow data for the CSO Pump Station measuring the pumped flow and Parshall flume at the WWTP measuring the gravity flow. Unfortunately, the CSO Pump Station flow data is only available for two of the three CSO events and Parshall flume flow data is only available for one of the three CSO events, which made model calibration somewhat subjective. The model calibration correctly predicts CSO events when they historically occurred and does not predict CSO events when they had not occurred, but the volume of the overflows does not match in all instances. Further adjustment of model parameters does not improve the calibration, as any improvement in one calibration point causes a greater deviation in another. Additional information would be needed

to significantly improve model calibration, which would require significantly more time and expense to obtain. Additional information could include field investigations of the sub-catchment areas to improve their characterization in the model, surveying existing trunklines to verify physical characteristics used in the model, verifying accuracy of the monitoring equipment, gathering additional flow data for calibration, etc.

MODEL RESULTS

The calibrated model was used to predict whether or not the CSO outfalls are controlled with the combined sewer system as it currently exists and whether or not the flow of effluent from the WWTP would exceed the hydraulic capacity of 8.9 MGD. As defined by Washington Administrative Code Title 173, Chapter 245, Section 020, Paragraph (22) (WAC 173-245-020(22)), each CSO outfall must not have more than one untreated discharge per year on average to achieve "the greatest reasonable reduction" and be considered controlled. Based on the WWTP hydraulic evaluation in Chapter 2, it is expected that as long as the effluent flow rate does not exceed 8.9 MGD, the WWTP discharge will meet the current NPDES permit limits.

Historical rainfall data from a period of 26.7 years was collected and modeled to develop a long-term simulation that would be representative of actual conditions. Rainfall data for short periods of time (i.e., a few years) can be uncharacteristic and over- or under-predict the occurrence of CSO events. Rainfall data for the period between November 21, 1987 and August 15, 2014 (all available data) was obtained from a rain gauge located at the Silver Lake Water District Office at 2210 132nd Street SE in Everett, which is monitored by Snohomish County.

To assist with calibration, help calculate CSO volumes and conservatively catch any possible CSO event, the model was run at 1 minute time steps. Given that the model calibration may be conservative (i.e., may be predicting a CSO when one might not actually occur) and considering the accuracy of the model given the limited data available for calibration and the typical level of precision for such an analysis, the predictions are considered less accurate for the lower duration CSO events. Generally, the accuracy of the models is best represented by 5 to 15 minute time steps, in which case overflows with a duration below that time step interval would be excluded. It is expected that model calibration could be improved as more data becomes available in the coming years.

CSO events during a 26.7-year simulation (using all available rainfall data) for each outfall are summarized in Tables 4-1 and 4-2 below. Based on these results, it appears that the CSO #1 outfall is currently controlled, but the CSO #2 outfall is not controlled as currently operated or configured per the requirement of WAC 173-245-020(22). However, modeling predicts that the CSO #2 outfall can be controlled with improvements already planned for implementation by the City.

TABLE 4-1
CSO OUTFALL #1 OVERFLOW PREDICTIONS
(26.7 YEARS OF RAINFALL)

Overflow Characteristics	No Separation		Avenues F, G, H Separated	
Volume (gal)	**Total**	**# Per Yr**	**Total**	**Per Yr**
> 0	34	1.27	18	0.67
> 5,000	29	1.08	15	0.56
> 10,000	27	1.01	13	0.49
> 15,000	26	0.97	12	0.45
Duration (min)	**Total**	**# Per Yr**	**Total**	**Per Yr**
> 1	34	1.27	18	0.67
> 2	33	1.23	17	0.64
> 4	30	1.12	15	0.56
> 5	29	1.08	15	0.56
> 10	24	0.90	9	0.34
> 15	15	0.56	3	0.11

TABLE 4-2
CSO OUTFALL #2 OVERFLOW PREDICTIONS
(26.7 YEARS OF RAINFALL)

Overflow Characteristics	3 Pumps (72% Speed)		4 Pumps (72% Speed)		3 Pumps (100% Speed)		4 Pumps (100% Speed)	
Volume (gal)	**Total**	**Avg./Yr**	**Total**	**Avg./Yr**	**Total**	**Avg./Yr**	**Total**	**Avg./Yr**
> 0	55	2.06	48	1.80	37	1.38	35	1.31
> 5,000	47	1.76	44	1.65	31	1.16	29	1.08
> 10,000	44	1.65	40	1.50	30	1.12	28	1.05
> 15,000	42	1.57	39	1.46	28	1.05	27	1.01
Duration (min)	**Total**	**Avg./Yr**	**Total**	**Avg./Yr**	**Total**	**Avg./Yr**	**Total**	**Avg./Yr**
> 1	55	2.06	48	1.80	37	1.38	34	1.27
> 2	55	2.06	47	1.76	35	1.31	33	1.23
> 4	54	2.02	44	1.65	32	1.20	30	1.12
> 5	51	1.91	44	1.65	31	1.16	29	1.08
> 10	42	1.57	39	1.46	27	1.01	25	0.94
> 15	30	1.12	26	0.97	15	0.56	13	0.49

The 26.7-year simulations include a total of 9 predicted overflows for the CSO #2 outfall during 1992 alone. Of these 9 predicted overflows, a total of 8 are predicted to occur within just a 2-month period, meaning that there was essentially one CSO event per week. This extremely tight collection of CSO events appears to be due to a stream of high intensity storms that happened to be spaced just far enough apart to generate isolated CSO events. Due to the anomalous nature of this 2-month period from 1992, the model results were also analyzed considering just the last 20-years of rainfall, which is a typical approach for this sort of evaluation.

CSO events during a 20-year simulation (using rainfall data from only the last 20 years) are summarized in Tables 4-3 and 4-4 below. These results similarly, but even more convincingly, indicate that the CSO #1 outfall is currently controlled, but the CSO #2 outfall is still not controlled as currently operated or configured per the requirement of WAC 173-245-020(22).

TABLE 4-3
CSO OUTFALL #1 OVERFLOW PREDICTIONS
(20 YEARS OF RAINFALL)

Overflow Characteristics	No Separation		Avenues F, G, H Separated	
Volume (gal)	**Total**	**# Per Yr**	**Total**	**Per Yr**
> 0	23	1.15	8	0.40
> 5,000	19	0.95	6	0.30
> 10,000	17	0.85	4	0.20
> 15,000	16	0.80	4	0.20
Duration (min)	**Total**	**# Per Yr**	**Total**	**Per Yr**
> 1	23	1.15	8	0.40
> 2	22	1.10	7	0.35
> 4	20	1.00	6	0.30
> 5	19	0.95	6	0.30
> 10	14	0.70	3	0.15
> 15	8	0.40	2	0.10

TABLE 4-4
CSO OUTFALL #2 OVERFLOW PREDICTIONS
(20 YEARS OF RAINFALL)

Overflow Characteristics	3 Pumps (72% Speed)		4 Pumps (72% Speed)		3 Pumps (100% Speed)		4 Pumps (100% Speed)	
Volume (gal)	**Total**	**Avg./Yr**	**Total**	**Avg./Yr**	**Total**	**Avg./Yr**	**Total**	**Avg./Yr**
> 0	41	2.05	35	1.75	24	1.20	24	1.20
> 5,000	33	1.65	31	1.55	19	0.95	19	0.95
> 10,000	32	1.60	27	1.35	18	0.90	18	0.90
> 15,000	30	1.50	26	1.30	16	0.80	17	0.85
Duration (min)	**Total**	**Avg./Yr**	**Total**	**Avg./Yr**	**Total**	**Avg./Yr**	**Total**	**Avg./Yr**
> 1	41	2.05	35	1.75	24	1.20	24	1.20
> 2	41	2.05	34	1.70	22	1.10	22	1.10
> 4	40	2.00	31	1.55	20	1.00	20	1.00
> 5	38	1.90	31	1.55	19	0.95	19	0.95
> 10	29	1.45	26	1.30	16	0.80	15	0.75
> 15	20	1.00	17	0.85	9	0.45	8	0.40

As indicated by the modeling results presented in Table 4-1, the average number of overflows to the CSO #1 outfall is predicted to be approximately one per year under current conditions, when considering results based on 5 to 15 minute time steps. Therefore, it appears that the CSO #1 outfall is currently controlled. Any separation of the system will further reduce the occurrence of overflows. The City already has separated the storm and sanitary sewers along Avenue F. Currently the storm sewer along Avenue F is connected to the combined sewer system, but will be connected to an existing storm sewer trunkline once the planned storm water wetlands that have already been designed are constructed, which is expected to occur in 2016. Additionally, the City has plans to separate all storm and sanitary sewers between Avenues E and L, which will provide even greater reduction in overflows to the CSO #1 outfall. For example, the model predicts about one overflow only every other year on average if the storm and sanitary sewers along just Avenues F, G and H are separated. To facilitate these ongoing separation projects, the City recently completed an extension of the 18-inch storm water trunkline from Avenue H to Avenue E (approximately 1,000 lineal feet) in summer 2014

As indicated in Table 4-2, the average number of overflows to the CSO #2 outfall is predicted to exceed one per year under current operation of the CSO Pump Station, whether up to 3 or 4 pumps are running at 72 percent of full speed. Although all four pumps are called to operate when the water level in the wet well triggers the high level float switch, the firm capacity of the pump station is based on 3 pumps. During the September 3, 2014 overflow to the CSO #2 outfall the backup pump was out of service, and so only 3 pumps were operating. Given that the pumps only operate at 72 percent of full speed, the system was also modeled with up to 3 and 4 pumps running at full speed. Allowing the pumps to run at higher speeds reduces surcharging and increases conveyance capacity. With this modification, the model predicts that even with just 3 pumps operating at full speed overflows to the CSO #2 outfall would likely average about one per year, when evaluating model results based on a 5 to 15 minute time step.

The model flow predictions throughout the 26.7 year-simulation were used to evaluate flow attenuation capacity in the WWTP. The evaluation assumed that effluent flow would increase with influent flow up to a limit of 8.9 MGD, after which the storage volume in the lagoons available for attenuation would be utilized to avoid exceeding the effluent flow capacity. Also, to be conservative, it was assumed that all four pumps at the CSO Pump Station would be operating at up to 100% capacity and that no sewer separation had yet occurred. Based on these parameters, the maximum storage volume required during some of the highest periods of influent flow throughout the 26.7-year simulation period is shown in Table 4-5 below. The highest instantaneous flow and highest hourly average flow during these periods of high influent flow are also shown in Table 4-5.

TABLE 4-5
PEAK WWTP FLOW CAPACITY AND STORAGE REQUIREMENTS

Time Period	Storage Requirements (Million Gallons)	Highest Instantaneous Flow (MGD)	Highest Hourly Average Flow (MGD)
1992	0.35	30.85	16.09
1996	0.53	30.88	18.35
2002	0.27	22.01	12.64
2004	0.64	27.98	23.36

As is evident from this evaluation, the storage volume currently available is more than enough to attenuate flow so as not to exceed an effluent flow rate of 8.9 MGD. Additionally, the highest hourly average flow rate into the WWTP during the 26.7-year simulation is predicted to be less than the 23.5 MGD capacity of the mechanical screen. Although the model predicts that instantaneous flow can exceed that capacity for a short duration (less than 20 minutes), excess flow can pass through the manual bar screen. Similarly, the model predicts that the portion of instantaneous flow from the influent gravity pipeline can reach approximately 20 MGD for short durations, which is just above the firm capacity of the influent pumps. Were one of the large pumps to be out of service at this time, the wet well would provide some flow attenuation and at worst flow could pass through the precast overflow trenches directly to Lagoon 1 if necessary.

COMPARISON TO DESIGN STORM

As mentioned previously, there have been no overflows reported during the wet weather months since completion of the CSO Pump Station and associated collection system improvements. However, three overflow events at the CSO #2 outfall and two at the CSO #1 outfall have occurred during the past three years in September and October. The 1.01-year storm interval used for design of the improvements was estimated to have a rainfall intensity of 0.41 inches per hour with a 1-hour duration. This rainfall intensity was determined by referencing rainfall intensity maps for 1-hour duration storms available from the National Oceanic and Atmospheric Administration (NOAA). In comparison, precipitation data from NOAA for the City of Snohomish indicates intensities of only 0.07 and 0.16 inches per hour for the 2-year storm interval with 24-hour and 6-hour durations, respectively, and 0.15 and 0.32 inches per hour for the 100-year storm interval with 24-hour and 6-hour durations, respectively.

The characteristics of the storms that generated the three overflows appear to be different compared to the design storm. Whereas the design storm assumes a constant rate of rainfall for the 1-hour duration, the storm that occurred on September 5, 2013 had a peak rainfall intensity equivalent to 1.20 inches per hour and an actual maximum rainfall of 0.77 inches within a 1-hour period. Similarly, for the storm that occurred on October 16, 2012 also had a peak rainfall intensity of 1.20 inches per hour, but the actual maximum rainfall within a 1-hour period was only 0.42 inches. The storm that occurred on September 3, 2014 had a peak rainfall intensity equivalent to 0.72 inches per hour, but the actual maximum rainfall within a 1-hour period was only 0.36 inches. During the first two CSO events, the total rainfall within a 1-hour period exceeded that of the design storm, meaning the storm interval was greater than 1.01 years. During the most recent CSO event, the total rainfall within a 1-hour period was slightly less than that of the design storm, but it appears from flow records that the output of the CSO Pump Station was lower than normal. The City confirmed that the backup pump was out of service and so only 3 pumps were running even after the high level float switch in the wet well was triggered. If the backup pump had been in service, a CSO event might not have occurred.

The relatively short duration storms with high peak intensities that generated the three CSO events are characteristic of “summer” showers occurring during normally dry weather months, which are typically not the focus of collection system sizing. Based on the occurrence of the three CSO events, it appears that peak intensities of these showers can exceed the hydraulic conveyance capacity, quickly filling storage volumes available for attenuation and surcharge the system causing an overflow.

CHAPTER 5
RECOMMENDATIONS

This Chapter recommends operational adjustments and system improvements, as necessary, to reduce the occurrence of CSO events without exceeding the hydraulic capacity of the WWTP and discusses a general operational strategy for controlling flow through the WWTP.

OPERATIONAL ADJUSTMENTS

Based on the modeling results presented in Chapter 4, the current configuration and operation of the combined sewer system requires some modification for the CSO outfalls to be considered controlled. Although not all of the adjustments discussed herein are necessary for the outfalls to be controlled, the City is interested in making additional improvements to the system to further reduce the occurrence of CSO events and improve reliability and efficiency of the system. To that end, the City is considering specific operational adjustments as discussed below, as well as improvements to the system that are discussed in the following section.

Increase WWTP Lagoon Attenuation Volume

As mentioned previously, the City has been maintaining higher lagoon levels to provide a greater buffer between the sediment layer and the water surface in an effort to minimize the impact of periodic lagoon turnover events. The City plans to remove sediment from the lagoons in 2015, after which the normal operating level in the lagoons can be reduced from the current depth of 11.5 feet to the original design depth of 10.0 feet. This will increase the storage volume available for attenuation from 9.2 MG to 13.5 MG, an almost 50 percent increase. A greater storage volume will allow the City to maintain a lower effluent flow during storm events, which will increase the efficiency and performance of the lagoons and filtration and disinfection processes.

Online Monitoring of Lagoon Level

Currently, the City has staff gauges installed to allow visual tracking of lagoon level. The City has indicated they are planning to install a level transducer for real-time monitoring of level in the lagoons through the supervisory control and data acquisition (SCADA) system. It is also recommended that the programmable logic controller be programmed to calculate storage volume available for attenuation in real-time and generate a curve (revised hourly) to be displayed in the SCADA system that plots the projected attenuation duration versus effluent flow given the current storage volume available for attenuation and the rolling average of influent flow over the previous 24-hours. This will allow the WWTP staff to determine the appropriate effluent flow rate from the lagoons to maintain the desired storage when considering current flows and forecasted rainfall. If more storage is deemed necessary, the effluent flow rate can be increased to reduce the lagoon level. If there is more storage available than needed, the effluent flow rate can be reduced to maximize performance and efficiency of the lagoons and filtration and disinfection processes.

Modify CSO Pump Station Control Strategy

Pump Speed Control

The current control strategy for the CSO Pump Station does not maximize use of the pumps to minimize the potential for an overflow at the CSO #2 outfall. The controls are currently set to limit pump operation to a constant 72 percent of full speed. While this certainly helps to reduce wear and stress on the pumps and energy consumption and improves efficiency at the WWTP, it also increases the potential for an overflow at the CSO #2 outfall by limiting throughput and

causing surcharging upstream. Based on available information, it is estimated that the carrying capacity of the 30-inch pipeline between the overflow pipe and the CSO Pump Station wet well is 11.1 to 17.4 MGD over the expected range of 0.007 to 0.011 for the Manning's roughness factor associated with aged PVC pipe (13.6 MGD based on a typical value of 0.009). This capacity is in excess of the current capacity of the CSO Pump Station, which is approximately 7.3 MGD with the pumps operating at 72 percent speed. Modifying the control strategy to allow the pumps to increase speed to 100 percent would likely increase the throughput of the CSO Pump Station to over 10 MGD based upon field test data for the pumps. This additional throughput would reduce surcharging and potential for overflow at the CSO #2 outfall. As discussed in Chapter 4, the model predicts that this modification should result in the CSO #2 outfall being controlled per the requirements of WAC 173-245-020(22). Facilitating this modification would likely involve adjusting the control strategy so that pump control (i.e., number of pumps in operation and pump speed) was based on maintaining one or more target levels in the wet well.

Rain Gauge Feedback Control

Another potential modification to the CSO Pump Station control strategy is to communicate information on rainfall from the electronic rain gauge that will be installed at the WWTP to the CSO Pump Station. Because there is some delay between when high intensity rainfall is measured and when it is received at the CSO Pump Station, there is an opportunity to optimize conditions for maximum throughput and flow attenuation. This could involve pumping down the wet well to near the minimum operating level to maximize available attenuation volume. This might also involve starting 2 or all 3 pumps in advance and running them at reduced speed so they are ready to quickly ramp up as level in the wet well starts to increase. Although this modification would not be expected to improve control of longer duration or larger volume CSO events, it could reduce the occurrence of short duration and low volume CSO events, as identified previously in Table 4-2.

SYSTEM IMPROVEMENTS

Storm Sewer Separation Projects

The two parallel 18-inch diameter sanitary sewer pipes that convey combined sewer flow along Second Street between the overflow control vault for the CSO #1 outfall and the 30-inch sanitary sewer trunkline leading to the WWTP are estimated to have a total open channel flow capacity of about 5.3 MGD. During even moderate storms the pipes must surcharge to generate enough head to force the flow through the pipes. This is a limitation of the system as it must maintain a relatively shallow slope to pass underneath Highway 9. As a result the parallel 18-inch sanitary sewer pipes were designed recognizing that surcharge conditions would occur with some regularity. It is estimated that under surcharge conditions the parallel 18-inch diameter pipes could convey a total of about 13 MGD without creating an overflow to the CSO #1 outfall. It appears that this capacity has been exceeded on two occasions leading to an overflow to the CSO #1 outfall. It is likely that initial rainfall leads to the pipes operating under surcharged conditions, such that when the peak rainfall intensity occurs there is little storage volume available for attenuation in the pipes and a short peak of very high intensity can lead to an overflow.

The City has already separated the storm and sanitary sewers along Avenue F, L, and K and a portion of Avenue J, but the new storm sewer pipes are still connected to the sanitary sewer pipes until a new storm water treatment wetland is constructed to receive the storm water. The storm water treatment wetland and extension of the existing storm sewer trunkline from the storm water structure at the entrance of the WWTP to the proposed storm water treatment

wetland have already been designed and the City is working to secure funding for construction. Once these improvements are constructed, the City will connect the separated storm sewers to the existing storm sewer trunkline. This will decrease flow through the parallel 18-inch sanitary sewer pipes. As indicated by the model predictions in Chapter 4, some separation is necessary for control of the CSO #1 outfall per the requirements of WAC 173-245-020(22). Additionally, the City is looking at securing funding to design separation of the rest of the storm and sanitary sewers from Avenues E to L that are tributary to the parallel 18-inch pipes. Such a project would further reduce flow through those pipes and the potential for overflow to the CSO #1 outfall. As mentioned previously, the City has just recently extended the 18-inch storm sewer trunkline between Avenues E and H to facilitate these separations.

Automated Lagoon Effluent Flow Control/Level Adjustment

As discussed in Chapter 2, the flow of effluent from the lagoons and the level in the lagoons can be controlled by manually adjusting the canal gate at the Lagoon 4 outlet structure. The Draft City of Snohomish General Sewer Plan and Wastewater Facilities Plan Amendment (Kennedy/Jenks, 2013) considered a future improvement to install a modulating weir in the Lagoon 4 outlet structure. The new weir would have a larger operating range than the existing weir, which would allow active control of the lagoon level and effluent flow. The weir could then be remotely or automatically adjusted via the SCADA system to control level and flow. Alternatively, the existing canal gate at the Lagoon 4 outlet structure could be retrofitted with an actuator to allow remote or automated control of lagoon level and effluent flow, similar to the existing manual control. Either would allow the WWTP staff to easily adjust level and effluent flow for improved performance and efficiency by maximizing retention time in the lagoons, minimizing peak flows through the filtration and disinfection processes, and maintaining sufficient storage volume for attenuation in the lagoons. Additionally, a control strategy could be programmed that would automate adjustment of the weir or canal gate to control to a targeted lagoon level and/or vary lagoon level to maintain a storage volume that would provide a specified duration of attenuation at the rolling average of influent flow over the previous 24 hours.

GENERAL OPERATIONAL STRATEGY FOR WWTP FLOW CONTROL

The following is an outline of the general operational strategy to help optimize performance and efficiency of the WWTP without exceeding hydraulic capacity:

- Maintain lagoon level at the normal operating level (currently a depth of 11.5 feet, reduce to 10.0 feet after removal of lagoon sediment) to increase storage volume available for attenuation.
- Monitor lagoon level during storm events. Once the lagoon level transducer is installed refer to real-time calculations of storage volume available for attenuation and the curve displayed in SCADA showing projected attenuation duration versus effluent flow (based on the storage volume available for attenuation and the rolling 24-hour average for influent flow). This information and the near-term weather forecast will be used to judge whether the effluent flow should be increased and lagoon level dropped (if not already at the normal operating level) to increase the storage volume available for attenuation or if lagoon level can be increased and effluent flow decreased to maximize treatment efficiency and performance.
- Manually adjust the existing canal gate at the Lagoon 4 outlet structure to achieve the effluent flow that will result in the desired lagoon level and storage volume available for attenuation. If the canal gate is retrofitted with an actuator and programming is enabled for automated control, adjust control parameters as needed to provide the desired result

(target level in the lagoons to be maintained or, projected attenuation duration if variable level control is enabled).

- If lagoon level is high (little storage volume available for attenuation), the effluent flow is already at the allowed maximum (8.9 MGD) and the near-term forecast is for continuation of high rainfall, consideration will be given to reducing output from the CSO pump station to avoid exceeding the hydraulic capacity of the WWTP. It is expected that this would be an extremely rare circumstance. The procedure to reduce flow from the CSO pump station is provided below. Although this procedure could be automated, it is recommended it remain manually initiated because it could result in an overflow to the CSO #2 outfall.

  1. The WWTP staff will contact the Public Works Manager to inform them of the need to change operation of the CSO Pump Station.
  2. The WWTP staff will contact the Lead of the Collection Department who will then manually change operation of the pumps using the VFD controls and/or hand-off-auto switch for each pump.
  3. The WWTP staff will contact the Lead of the Collection Department once it has been determined that the CSO Pump Station can resume normal operations.
  4. The Lead of the Collection Department will return the VFD controls and hand-off-auto switches to their normal settings and positions.

CHAPTER 6
REFERENCES

Kennedy/Jenks Consultants (July 2013), “Draft City of Snohomish General Sewer Plan and Wastewater Facilities Plan Amendment”.

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