City of Snohomish

September 2020

General Sewer Plan
and
Wastewater Engineering Report

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ACKNOWLEDGEMENTS

The General Sewer Plan and Wastewater Engineering Report was developed under the direct supervision of Tom Giese, P.E. of BHC Consultants, LLC.

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LIST OF ABBREVIATIONS

2005 Plan	2005 General Sewer Plan and Wastewater Facilities Plan
2010 Plan	2010 City of Snohomish General Sewer Plan and Wastewater Facilities Plan Update
AACE	American Association of Cost Engineers
AC	Asbestos Concrete
ammonia-N	Ammonia-Nitrogen
ATS	Automatic Transfer Switch
BLR	Buildable Lands Report
BOD	Biochemical Oxygen Demand
BOD_5	5-day Biochemical Oxygen Demand
BSF	Base Sanitary Flow
BUF	Beneficial Use Facility
C	Collection
$CBOD_5$	5-day Carbonaceous Biochemical Oxygen Demand
ccf	Charge per Hundred Cubic Feet
CDF	Controlled Density Fill
CERB	Community Economic Revitalization Board
CFR	Code of Federal Regulations
cfs	Cubic Feet Per Second
cfu	Colony Forming Units
CIP	Capital Improvement Plan
CIPP	Cured-In-Place Pipe
City	City of Snohomish
CO_2	Carbon Dioxide
CPPs	Countywide Planning Policies
CSO	Combined Sewer Overflow
CSS	Combined Sewer System
CT	Concentration-Time
DAHP	Department of Archaeology and Historic Preservation
DMPC	Dual-Powered, Multi-Cellular
DMR	Discharge Monitoring Report
DNR	Department of Natural Resources
DO	Dissolved Oxygen
DWF	Dry Weather Flow
Ecology	Washington State Department of Ecology
EFH	Essential Fish Habitat
EMCC	Emergency Motor Control Center
EPA	Environmental Protection Agency
Everett	City of Everett

FEMA	Federal Emergency Management Agency
FIRMs	Flood Insurance Rate Maps
fps	Feet per Second
FTE	Full-Time Employee
GMA	Growth Management Act
GOIA	Governor's Office of Indian Affairs
gpcd	Gallons per Capita per Day
gped	Gallons per Employee per Day
gpm/sf	Gallons per Minute per Square Foot
GWI	Groundwater Infiltration
H_2O	Water
HDD	Horizontal Directional Drilling
HMI	Human-Machine Interface
hp	Horsepower
HP/MG	Horsepower per Million Gallons
HPA	Hydraulic Project Approval
HRT	Hydraulic Retention Time
HUD	Housing and Urban Development
HVAC	Heating, Ventilation, and Air Conditioning
I/I	Infiltration and Inflow
kW	Kilowatts
JARPA	Joint Aquatic Resource Permit Application
lbs/hr	Pounds per Hour
MBBR	Moving Bed Bioreactor
MCC	Motor Control Center
MCE	Meter Capacity Equivalent
MG	Million Gallons
mg/L	Milligrams per Liter
MGD	Million Gallons per Day
mL	Milliliter
MMF	Maximum Month Flow
MNE	Microvi MicroNiche Engineering
NAVD	North American Vertical Datum
NBOD	Nitrogenous BOD
NEPA	National Environmental Policy Act
NH_3	Ammonia-N
NHPA	National Historic Preservation Act
NMCC	Normal Motor Control Center
NOAA	National Oceanic and Atmospheric Administration
NOC	Notice of Construction
NPDES	National Pollutant Discharge Elimination System

NPV	Net Present Value
NTU	Nephelometric Turbidity Units
O&M	Operations and Maintenance
OFM	Office of Financial Management
OPPC	Opinion of Probable Project Costs
Orange Book	Criteria for Sewage Works Design (Ecology, 2008)
P	Pumping
PAA	Peracetic acid
PHF	Peak Hour Flow
Plan	General Sewer Plan and Wastewater Engineering Report
PLC	Programmable Logic Controller
ppcd	Pounds per Capita per Day
ppd	Pounds per Day
PS	Pump Station
PSCAA	Puget Sound Clean Air Agency
PSRC	Puget Sound Regional Council
PSRP	Process to Significantly Reduce Pathogens
PVC	Polyvinyl Chloride
PWTF	Public Works Trust Fund
RCW	Revised Code of Washington
RDII	Rainfall Dependent Inflow and Infiltration
RWL	RWL Water, LLC
SAEP	Small Area Estimate Program
SCADA	Supervisory Control and Data Acquisition
SEPA	State Environmental Policy Act
SERP	State Environmental Review Process
SFF	Submerged Fixed-Film
SMC	City of Snohomish Municipal Code
SMPs	Shoreline Master Programs
SO_2	Sulfur Dioxide
SOTE	Standard Oxygen Transfer Efficiency
SR	State Route
SRT	Solids Retention Time
TAZ	Traffic Analysis Zone
TDH	Total Dynamic Head
TF/SC	Trickling Filter/Solids Contact
TIN	Total Inorganic Nitrogen
TKN	Total Kjeldahl Nitrogen
TMDL	Total Maximum Daily Load
TSS	Total Suspended Solids
UFC	Uniform Fire Code

UGA	Urban Growth Area
USGS	United States Geological Survey
UV	Ultraviolet
V	Volt
VFD	Variable Frequency Drive
W	Wastewater Treatment
WAC	Washington Administrative Code
WDFW	Washington State Department of Fish and Wildlife
WPCF	Water Pollution Control Facility
WSST	Washington State sales tax
WWF	Wet Weather Flow
WWTP	Wastewater Treatment Plant

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Executive Summary

This *General Sewer Plan and Wastewater Engineering Report* (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. Based on population projections, influent flow and load projections, and consideration of the recent influent flow trend, the currently permitted capacity of the wastewater treatment plant (WWTP) is estimated to be sufficient to 2028. 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, though additional improvements are required to meet pending nitrogen limits associated with the Puget Sound Nutrient Reduction Project. The permitted capacity will need to be increased during the planning horizon to allow treatment of projected flows and loads. It is recommended that a re-rating of the permitted capacity be undertaken with the planning and design of improvements for nitrogen removal, as these two elements are interrelated. 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 Plan (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 disinfection with peracetic acid. 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).

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

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. 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 Engineering Reports (WAC 173-240-060) as shown in Table ES-1 and Table 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 Engineering Report 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 9.4.8
3m	Discussion of sludge disposal	Section 9.7
3n	Provisions for future needs	Chapters 9 and 12
3o	Staffing and testing requirements	Section 9.8
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 its residents. The water system consists of a storage and distribution system that is separated into six pressure zones. 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, the City purchases all of its water wholesale from the City of Everett via six interties to Everett's Transmission Line No. 5. No additional treatment is required for Everett's water supply and the City does not own or operate any water supply wells.

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
  - CSO Control Policy
- 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)
  - Department of Archaeology and Historic Preservation Approval
- 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
  - Shoreline Management Program
- City Design and Construction Standards

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-5). 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, current, and projected flows and loadings for the WWTP are 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.

As shown in Table ES-5, the projected maximum month flow for 2022 is equal to the permitted capacity of 2.80 MGD. It should be noted that the projections for maximum month flow highly influenced by infiltration and inflow and are based on a peaking factor that represents the historical worst case condition. Although this is considered appropriate for modeling the collection system and examining long-term capacity needs, as a similar peaking factor is expected to occur again at some point in the future, such a conservative peaking factor is less appropriate for considering capacity needs in the short-term. Therefore, recent trends for maximum month flow were examined to project a more appropriate timeline in the short-term for when the maximum month flow would likely reach the permit limit of 2.80 MGD. Based on recent trends, it is estimated the permit limit would be reached in 2028.

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, addition of a denitrification process for future nitrogen removal requirements, 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 $56 million, including an estimated $26 million to purchase capacity at the Everett WPCF (costs adjusted to 2018 dollars). 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. Additionally, it is not guaranteed that the City of Everett still has adequate capacity at their WWTP to accept the City's wastewater and the associated connection charge could increase substantially with the pending nitrogen removal requirements, for which the City of Everett will need to make improvements. 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, adding denitrification, 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 achieving denitrification for nitrogen removal include:

- Denitrifying Sand Filters
- Microvi MNE Biocatalyst
- Moving Bed Bioreactor

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.
- Adding a moving bed bioreactor to achieve denitrification following nitrification in the existing lagoon process
- 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 (2019 thru 2024) and Long-Term (2025 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. Project W-4 to replace the existing chlorine gas disinfection system was just recently completed with the construction of a new peracetic acid disinfection system. However, this project was left in the 6-year CIP as it occurred during the 6-year CIP period.

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

CIP No.	Project	Opinion of Probable Project Cost
C-1	General Conveyance System Repair and Replacements (2019-2024)	$2,278,000
C-2	CSO 2 Separation Program	$2,315,000
C-3	Emerson Street Utility Improvement	$100,000
C-4	Upsize Gravity Main at 13th Street and Avenue A	$1,050,000
C-5	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	Nitrogen Removal Planning/Optimization	$100,000
W-2	Upgrade/Expand Effluent Filtration	$2,320,000
W-3	General WWTP Improvements (2019-2024)	$2,278,000

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

CIP No.	Project	Opinion of Probable Project Cost
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	**$15,256,000**

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

CIP No.	Project	Opinion of Probable Project Cost
C-5	Western Gravity Flow Diversion	$2,981,000
C-6	Rainier Pump Station (PS 2) Force Main Replacement	$760,000
C-7	General Conveyance System Repair and Replacements (2025-2036)	$6,109,000
C-8	Interurban Trail Conveyance System Upgrade	$1,019,000
C-9	CSO 1 Separation Program	$4,500,000
C-10	CIPP Pipe Restoration North of Blackmans Lake	$451,000
C-11	Pipe Extension North of the Casino Pump Station (PS 14)	$1,479,000
C-12	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	Nitrogen Removal Improvements	$4,840,000
W-9	General WWTP Improvements (2025-2036)	$6,109,000
	Total Opinion of Probable Project Cost	**$35,246,000**

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 finances of the sewer utility, including six-year past financial performance, current financial structure, financing resources and funding assistance, financial forecast, and analysis of current and projected rates to finance the capital improvements identified in Chapter 12. The results of this analysis indicate that rates can be held flat from 2020-2022 and provide revenue sufficient to cover all utility financial obligations before 2.00 percent annual increases are implemented from 2023 forward. It is important to remember this analysis is based on projected growth rates. If the future growth rates change, the existing rate strategy may need to be updated and revised. It is recommended that the City regularly review and update the key assumptions of the financial plan to ensure adequate revenues are collected to meet all financial obligations.

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 Engineering Report* (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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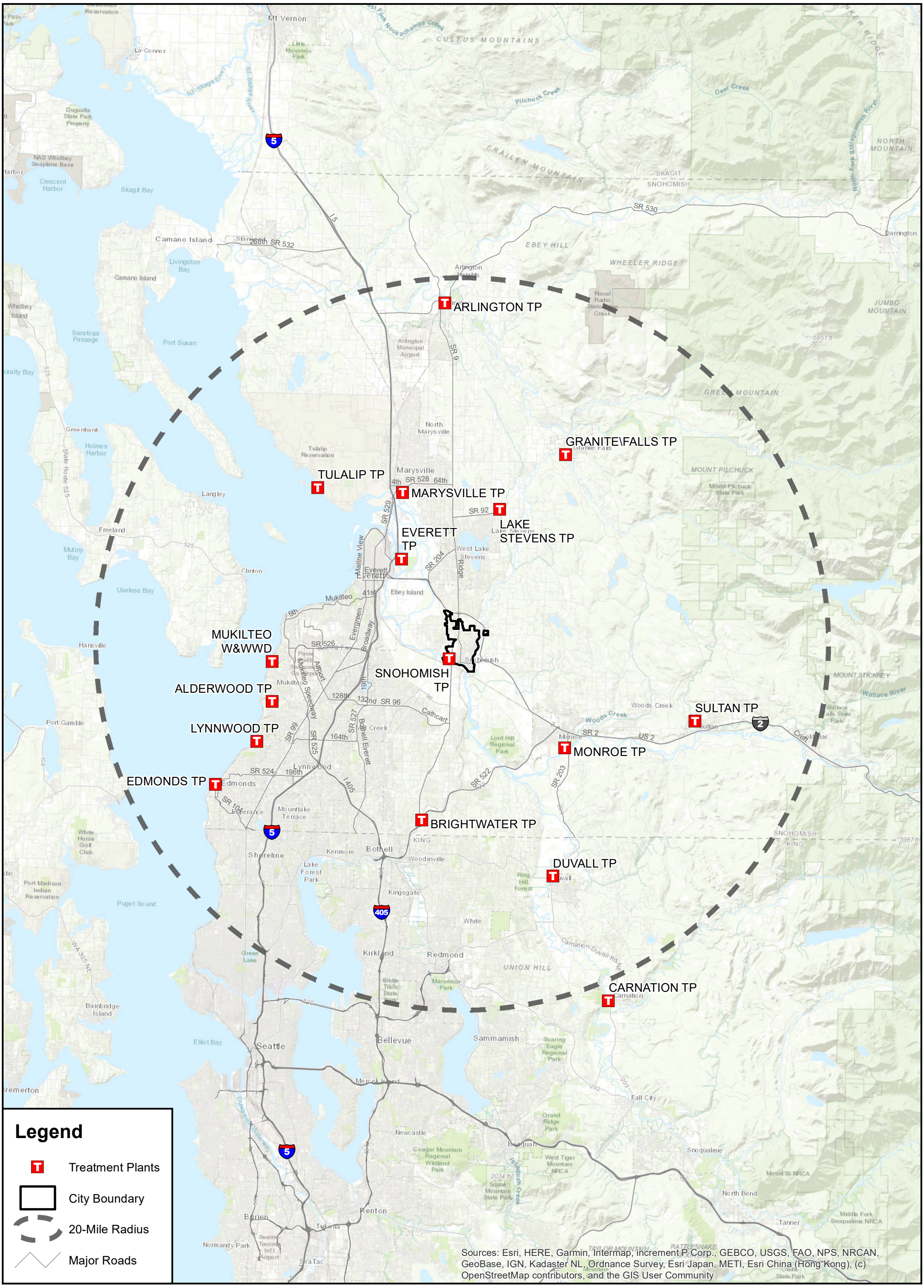

Sewer System & Basins: City of Snohomish.
GIS Base data: Snohomish County.
Data sources supplied may not reflect current or actual conditions. This map is a geographic representation based on information available. It does not represent survey data. No warranty is made concerning the accuracy, currency,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

General Sewer Plan and
Wastewater Engineering Report
City of Snohomish, Washington
September 2020

Figure
1-1

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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 engineering report 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 Engineering Report 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 9.4.8
3m	Discussion of sludge disposal	Section 9.7
3n	Provisions for future needs	Chapters 9 and 12
3o	Staffing and testing requirements	Section 9.8
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.

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 Combined Sewer Overflow (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 Plan. 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.

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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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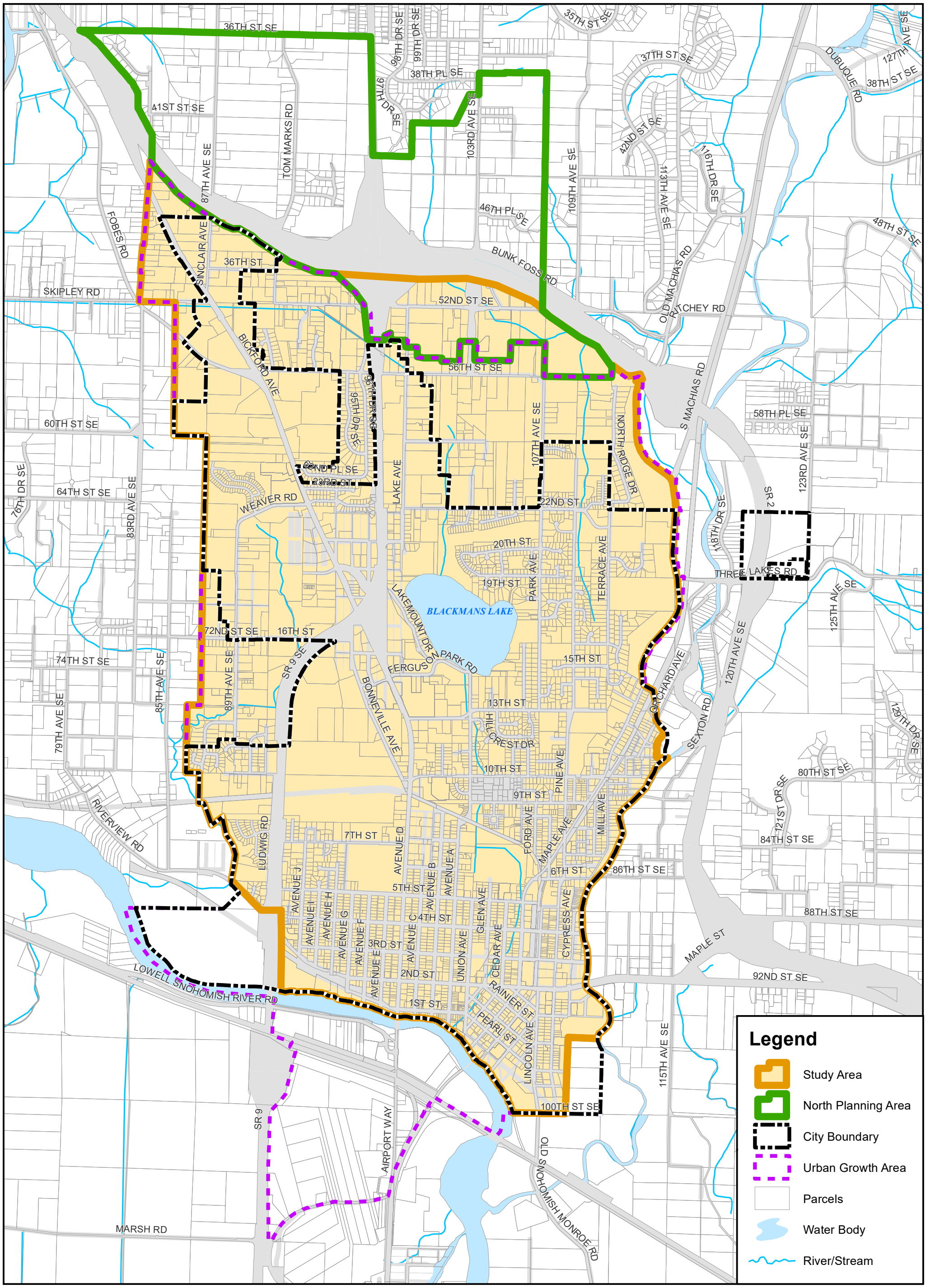

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Study Area

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Figure
2-1

P:\Mapping\Maps_Generated\Snohomish_City\19-10615.00\003\maps\Fig 2-1 Study Area - 11x17.mxd 8/27/2020 ctolentino

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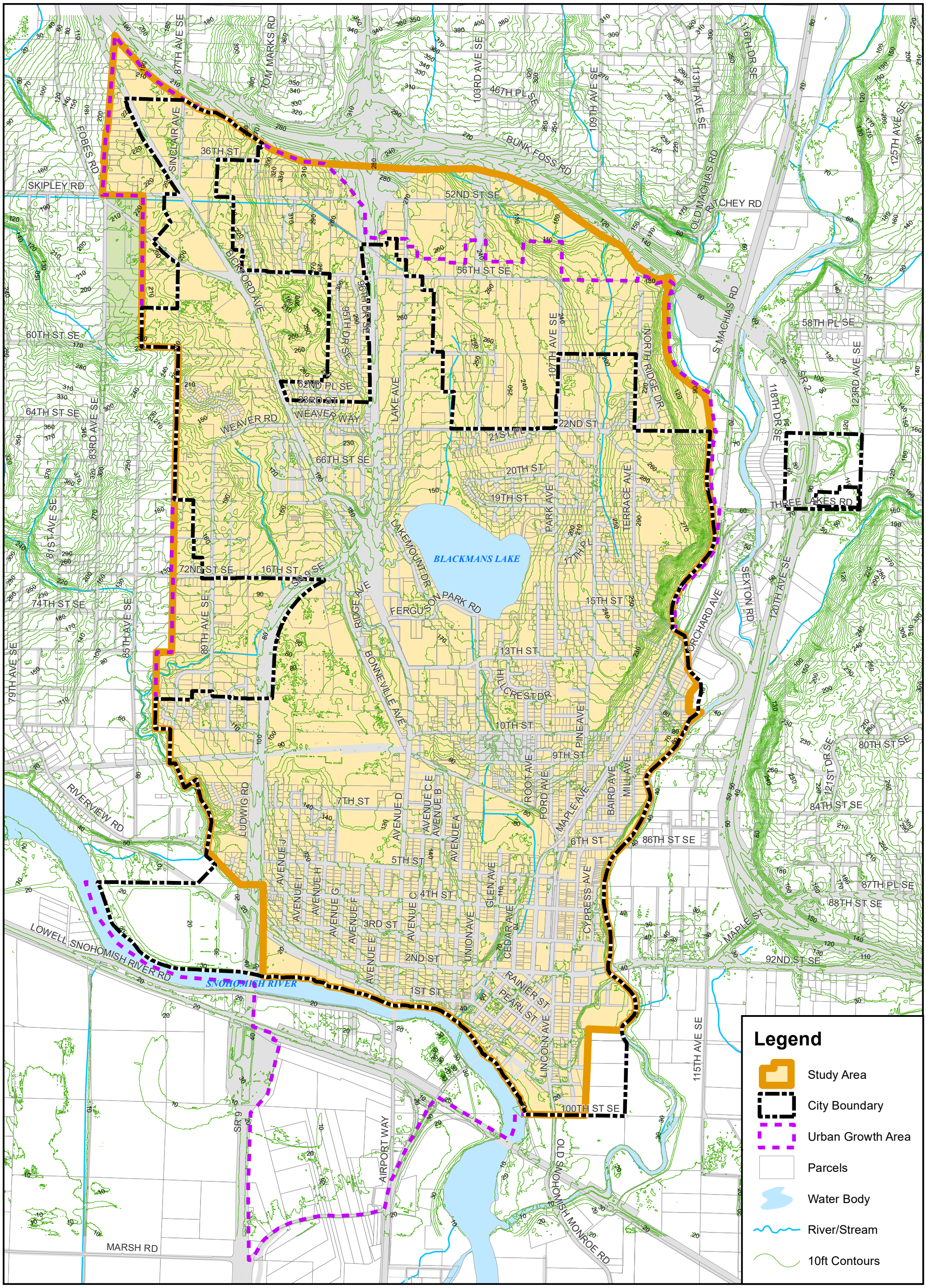

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Study Area Topography

General Sewer Plan and
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Figure
2-2

P:\Mapping\Maps_Generated\Snohomish_City\19-10615.00\003\maps\Fig 2-2 Study Area Topography - 11x17.mxd 8/27/2020 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.

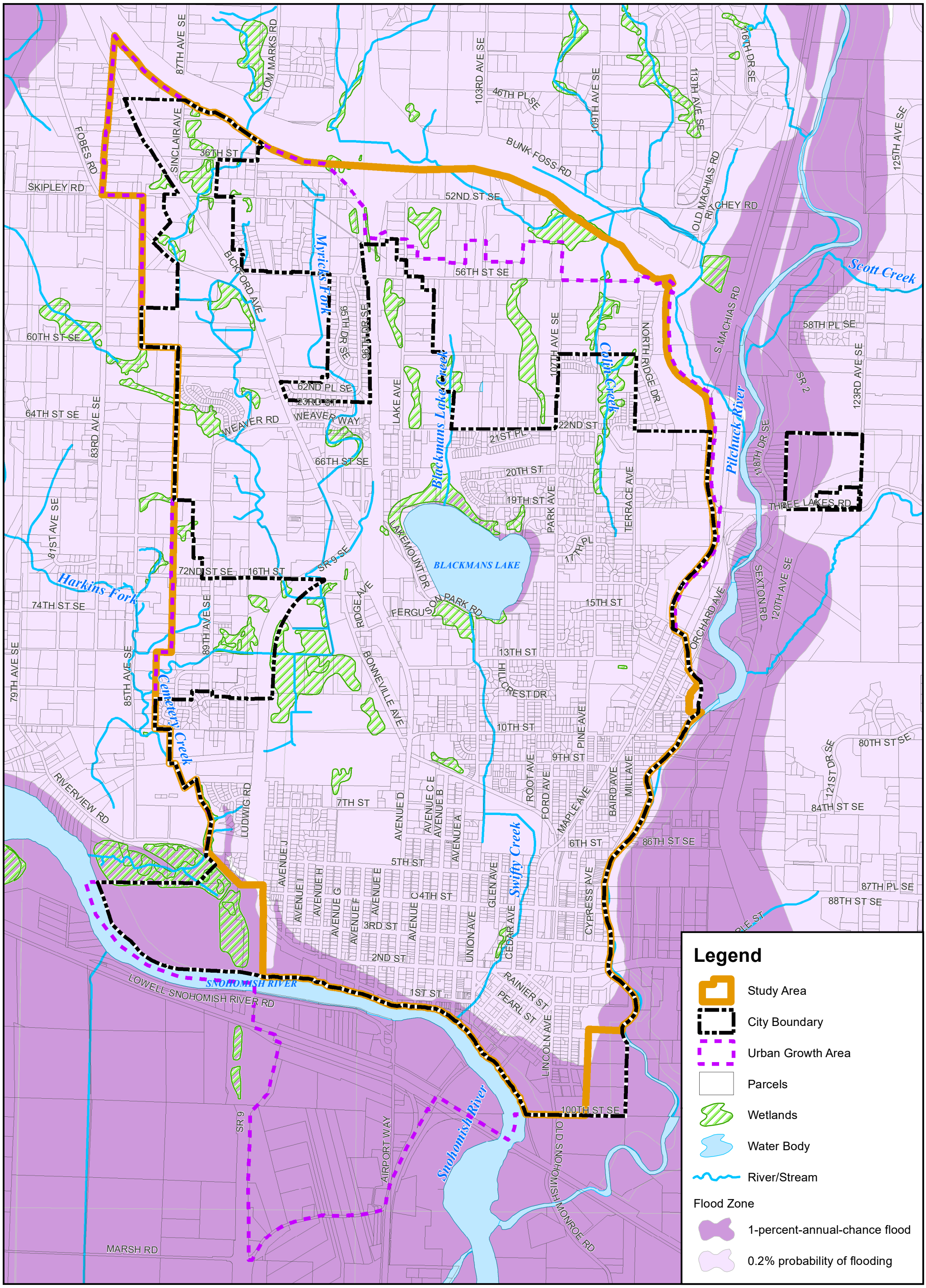
The average annual discharge of the river at Snohomish (River Mile 12.7) is estimated to be 11,000 cubic feet per second (cfs); the median discharge at this site is roughly 8,800 cfs (USGS, 1985). These discharges rank the Snohomish as the second largest river (behind the Skagit) in the Puget Sound/Hood 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.

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.

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.

Although the WWTP does not directly discharge to Puget Sound, Ecology's Salish Sea modeling includes the WWTP as a point source input. This model has been used to predict dissolved oxygen impairments in Puget Sound and inform the ongoing Puget Sound Nutrient Reduction Project. The outcome of this project will have a direct impact on future treatment requirements for the WWTP. Ecology has indicated that the WWTP will require coverage under the Puget Sound Nutrient General Permit, currently under development and anticipated to be issued as early as mid-2021. This nutrient general permit will initially cap the discharge of total inorganic nitrogen (TIN) at existing levels and require the City to take steps to optimize nitrogen removal and initiate planning of improvements to meet future TIN effluent limits. These future limits are expected to be included in the second 5-year cycle of the nutrient general permit.



Wetlands: Snohomish County. Flood Hazard Area: FEMA/WA Dept. of Ecology 2010
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Study Area Surface Waters and Critical Areas

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Figure **2-3**

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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 and ten zone valves that divide the water system into six pressure zones. The City purchases water wholesale from the City of Everett via six interties to Everett's Transmission Line No. 5. Refer to Figure 2-4 for the location of these facilities. No additional treatment is required for Everett water supply and the City does not own or operate any water supply wells.

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Everett Transmission Line

Bickford Ave. Intertie

King Charley's Intertie

Everett Transmission Line

Park Ave. Intertie

Terrace Ave. Intertie

Everett Transmission Line

NEPA Pallet Intertie

Bickford Ave. PRV (Inactive)

10th St. & Ave. D PRV

9th St. & Ave. A PRV

Reservoir No. 3

Reservoir No. 2 PRV

Reservoir No. 2

Reservoir No. 3
2.7 MG
Base Elevation: 296.5'
Overflow Elevation: 362.0'

Reservoir No. 2
5.0 MG
Base Elevation: 197.4'
Overflow Elevation: 218.2'

BLACKMANS LAKE

SNOHOMISH RIVER

Legend

Water System

- Reservoir
- PRV
- Intertie
- Zone Valve
- Water Pipes
- Everett Transmission Line

Base Layers

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

Water System: City of Snohomish.
GIS Base data: Snohomish County.
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City of Snohomish Water System
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Figure **2-4**

P:\Mapping\Maps_Generated\Snohomish_City\19-10615.00\003\maps\Fig 2-4 City of Snohomish Water System - 11x17.mxd 8/29/2020 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.

The City currently runs a water use efficiency program, included as part of the 2020 *City of Snohomish Water System Plan* (see Appendix F of that plan). The water use efficiency program contains mandatory measures including source meters, service meters, meter calibration, water loss control action plan, and customer education with annual consumer confidence reports. Other measures selected by the City for implementation include high meter reading notification, rebate program, indoor and outdoor conservation kits, lawn watering calendars, and additional customer education in the form of events, workshops, and advertising.

2.8 Other Wastewater Facilities

Under the requirements of WAC 173-240-050 for General Sewer Plans, Snohomish must identify existing 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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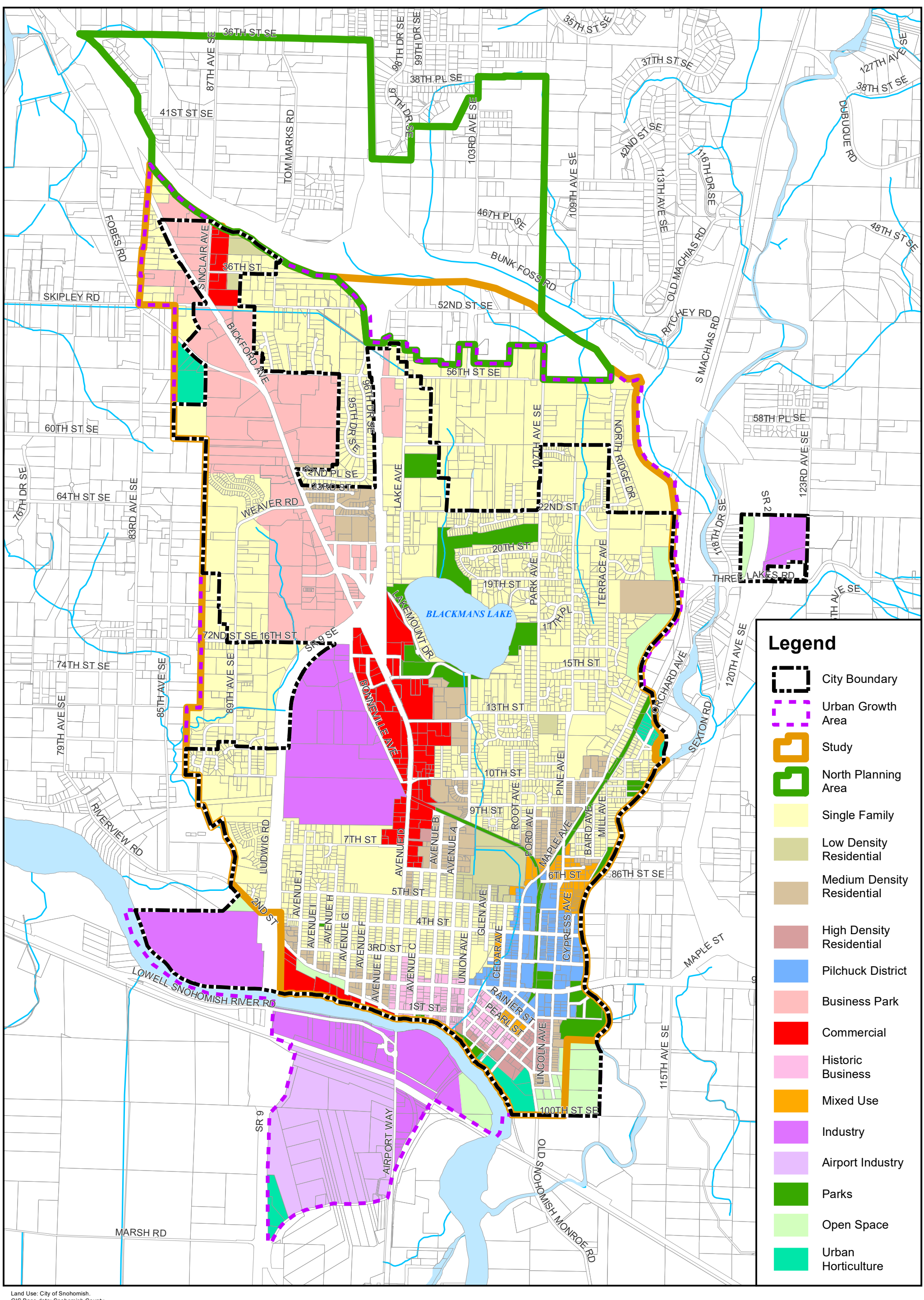

Land Use: City of Snohomish.
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Land Use Map

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Figure
3-1

P:\Mapping\Maps_Generated\Snohomish_City\19-10615.00\003\maps\Fig 3-1 Land Use - 11x17.mxd 8/27/2020 ctolentino

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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 identification. A detailed methodology and list of resources can be found in Appendix C.

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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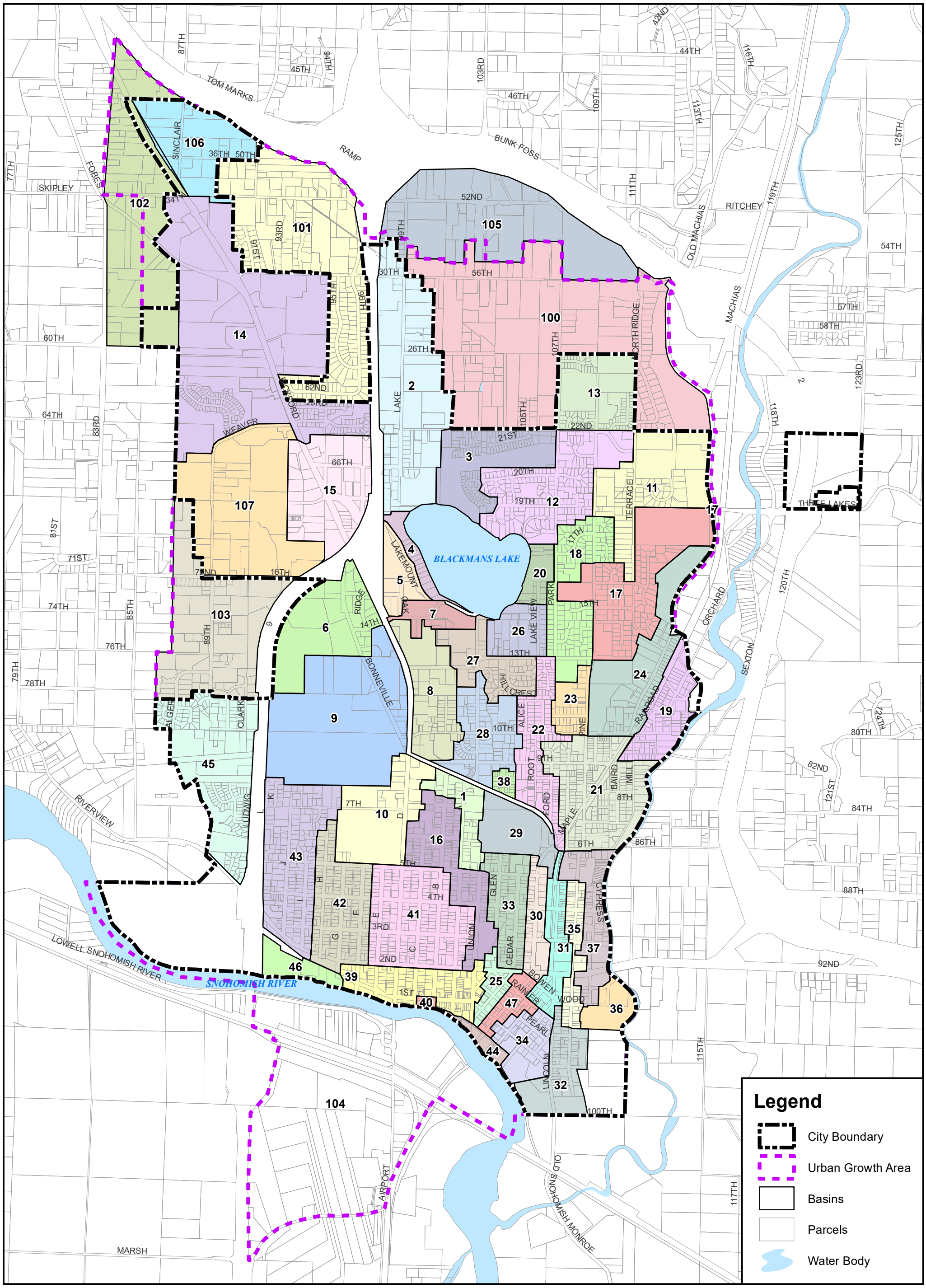

Sewer System & Basins: City of Snohomish.
GIS Base data: Snohomish County.
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Sub-Basins Map

General Sewer Plan and
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September 2020

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
- The pH of the effluent shall be between 6.0 and 9.0.

There can be exceptions to these regulations when treatment works receive combined sewer flows or certain industrial wastes and more stringent limits can be imposed to comply with water quality standards. However, in general, these are the minimum federal requirements for effluent quality. As discussed in Section 4.2, Ecology administers these regulations under the NPDES and has authority to establish alternative limits when applicable.

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 public outreach and participation program as part of the development and approval of this General Sewer Plan and Wastewater Engineering Report. Following a SEPA review for this Plan conducted by the City, public notice of the determination of non-significance was published on March 13, 2020. The public written comment period for this notice extended until March 26, 2020. Following review by Ecology and additional opportunities to comment before the City Council, the City Council is to make its decision on adoption of the Plan in mid-2020.

4.1.15 CSO Control Policy

EPA oversees a national framework for controlling CSOs through the NPDES permitting program. The CSO control policy was established to eliminate dry weather CSOs, bring CSOs in compliance with CWA requirements, and minimize impacts of CSOs. Key components of the policy include the Nine Minimum Control Measures and the implementation of a post-construction monitoring plan. The Nine Minimum Control Measures provides information on minimum technology-based controls for use in addressing CSO problems. The City has successfully maintained an average of less than 1 CSO event per outfall per year following construction of the CSO Pump Station (Pump Station No. 1) in 2011. The City maintains a post-construction monitoring plan (completed in 2015) currently in place as required by EPA CSO control policy.

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. As the NPDES authority, Ecology can establish alternative limits on a case-by-case basis when applicable. Washington's chief water pollution control law, RCW 90.48, is administered by Ecology, and contains relevance on planning, design, and operation of WWTPs. Current and potential future water quality regulations, such as nutrient discharge limits, are considered in development of this Plan, consistent with this law.

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

WAC 173-201A establishes water quality standards for surface waters of the State of Washington consistent with public health, recreation, and maintaining a healthy population of fish, shellfish, and wildlife, pursuant to the provisions of RCW 90.48. All surface waters are protected by numeric and narrative criteria, designated uses, and an antidegradation policy. Numeric and narrative criteria are assigned to protect each water body based on the designated uses. Designated uses for fresh waters, such as the Snohomish River, can include aquatic life,

recreation, water supply, and miscellaneous uses. Criteria for protection of fresh water uses are defined in WAC 173-201A-200. Designated uses for marine waters, such as Puget Sound, can include aquatic life, shellfish harvesting, recreation, and miscellaneous uses. Criteria for protection of marine water uses are defined in WAC 173-201A-210. The antidegradation policy has a three tiered approach that ensures existing and designated uses are maintained and protected, waters of a higher quality are not degraded unless in the overriding public interest, and degradation of outstanding resource waters is prevented.

The Snohomish River, into which the WWTP discharges, is designated core summer habitat for aquatic life use, primary contact for recreation use, for all recreation uses, and for all miscellaneous uses. The 1999 Snohomish River Estuary Total Maximum Daily Load Study performed by Ecology implemented mass load limits 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. Increases in the effluent discharge beyond the current permitted capacity, whether in flow or concentrations of constituents, associated with meeting future flow and load projections will require the City to conduct a new mixing zone study and reasonable potential analysis to confirm that these increases will still comply with applicable water quality standards.

In 2016, Ecology revised water quality standards under WAC 173-201A to protect humans from exposure to specific chemicals as a result of either consuming fish or drinking water that contain the pollutants (from a water supply or ingestion during recreational activities) including metals (e.g., copper, mercury and zinc) and organic compounds (e.g., Bis [2-ethylhexyl] phthalate and chloroform).

For many municipalities, metals such as copper, mercury, and zinc may be present in the wastewater in significant quantities depending on local industrial discharges, introduction of stormwater runoff, characteristics of the potable water system. 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. 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. With regard to organic compounds, processes with a longer solids and/or hydraulic retention time have been shown to increase removal of most types of organic pollutants. However, as with metals, these compounds are typically best dealt with at the source if possible.

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. With the recent installation of a new peracetic acid disinfection system, the discharge no longer contains residual chlorine.

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
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

Table 4-4
Current NPDES Permit Effluent Limits

Parameter	Effluent Limits
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

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 based on historical data as part of the next mixing zone study to better represent actual conditions and more accurately predict subsequent impacts from the WWTP.

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.

As discussed in Chapter 2, Ecology's ongoing Puget Sound Nutrient Reduction Project will include issuing a nutrient general permit (as early as mid-2021) to regulate discharge of TIN from point sources into Puget Sound. The City's WWTP is on the preliminary list of point sources identified by Ecology. The nutrient general permit will initially cap the discharge of TIN at existing levels and require the City to take steps to optimize nitrogen removal and initiate planning of improvements to meet future TIN effluent limits. These future limits are expected to be included in the second 5-year cycle of the nutrient general permit. Initial evaluations by Ecology have considered TIN limits of 8 milligrams per liter as nitrogen (mg-N/L) and 3 mg-N/L, suggesting that concentration limits of around 8 mg-N/L might be applied to WWTPs that are smaller and/or in less critical areas and that limits of around 3 mg-N/L might be applied to WWTPs that are larger and/or in more critical areas. However, Ecology has stated that all facilities should give consideration to how a lower limit of 3 mg-N/L could be achieved.

The City's WWTP is currently capable of nitrification using the existing submerged fixed-film media system. This process has successfully nitrified effluent from later spring through early fall. From late fall to early spring the City has historically changed to intermittent aeration to save energy, and so nitrification has dropped off during this period. The process should be

capable of continuing full nitrification between late fall and early spring, but this will need to be confirmed and demonstrated. To achieve expected limits for TIN, a denitrification process must be added to the WWTP, as discussed in Chapter 9. Chapter 11 evaluates alternatives for denitrification and the recommended alternative is included in the capital improvement plan in Chapter 12.

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

The Reclaimed Water Rule (WAC 173-219) is an implementation of Chapter 90.46 of the RCW. It was created to help meet the growing demand for clean water throughout the state by providing a regulatory framework for the generation, storage, distribution, and use of reclaimed water, as authorized by Ecology and the Department of Health. Under this rule, there are two primary classifications of reclaimed water, Class A and B. Class A reclaimed water must be oxidized, coagulated, filtered, and disinfected. Class B reclaimed water must be oxidized and disinfected. The degree to which these treatment methods must be applied to meet particular water quality limits depends on the specific beneficial use of the reclaimed water, as defined by the rule.

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. Additionally, Executive Order 05-05 requires that capital projects using funds appropriated by the State's consider how proposed projects may impact significant cultural and historical places, similar to a federal Section 106 review. This would include projects funded through Ecology's grant and loan programs or through the Public Works Trust Fund, along with other state funding sources. Agencies are required to notify the Department of Archaeology and Historic Preservation (DAHP), the Governor's Office of Indian Affairs (GOIA), and any concerned tribes of proposed plans. After an initial review, DAHP will determine if further action is required.

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 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. A 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 a 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 March 11, 2020 for the SEPA review associated with this Plan, which was publicly noticed as described in Section 4.1.14 above. As this SEPA review is for non-project action, additional SEPA review will be required during design of specific project actions identified in this Plan.

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.

4.4 Design and Construction Standards

All sewer system improvements are designed and constructed in accordance with Department of Ecology *Criteria for Sewage Works Design*, the City's *Engineering Design and Construction Standards* (see Appendix L) and meet or exceed the design standards referenced in WAC 173-240.

4.5 Pretreatment

Pretreatment systems may be required to reduce, eliminate, or alter the nature of a pollutant's properties prior to discharging to the City's sewer collection system. Pretreatment systems include grease traps, interceptors, oil/water separators and other units to treat metals, solvents, fats, oils, grease, excessive BOD or TSS, and other constituents. Pretreatment systems are to be installed in compliance with Chapter 15.08 of the City's Municipal Code.

Pretreatment is regulated by EPA under Part 403 and by Ecology in the State of Washington. Industrial dischargers are required to obtain a pretreatment permit to be administered by Ecology. Under WAC 173-240-050 (3)(i), general sewer plans are required to discuss current and future industrial dischargers that may affect the collection or treatment system. At this time, there are no significant industrial users in the City and no known plans for a significant industrial user to connect to the City's sewer system.

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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 is of particular interest because the NPDES permit limits the influent maximum month flow so that flows do not exceed the approved design capacity.
- **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.

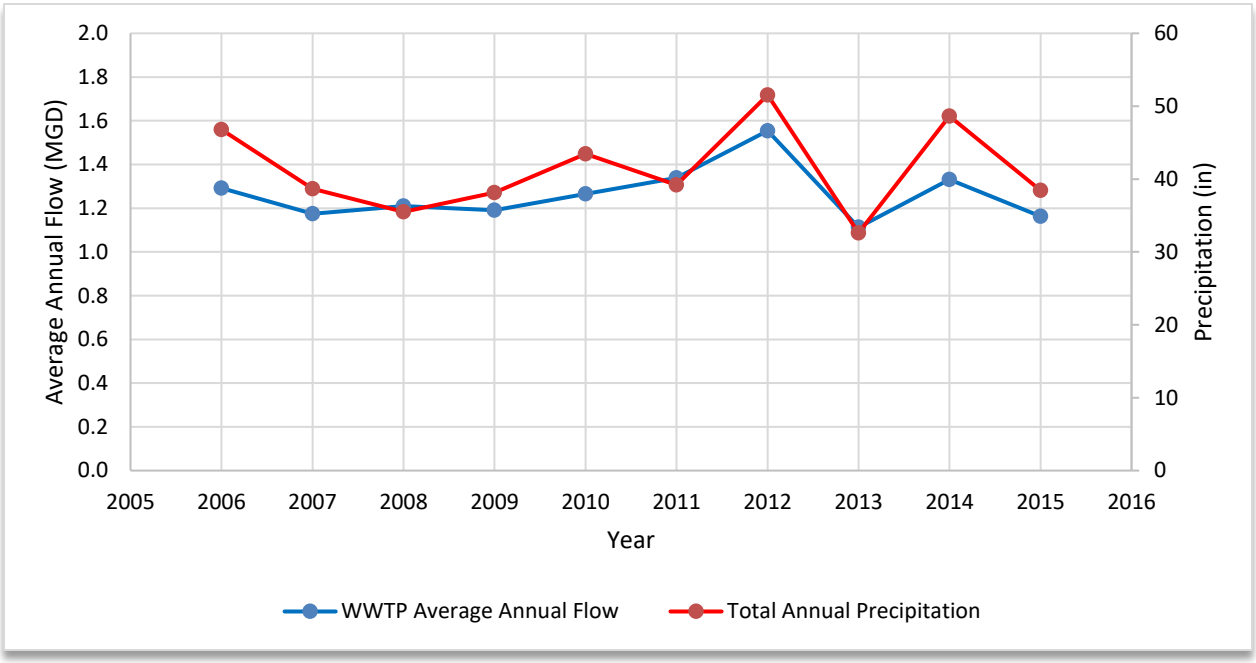

Figure 5-1 Average Annual Flow and Total Annual Rainfall

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.

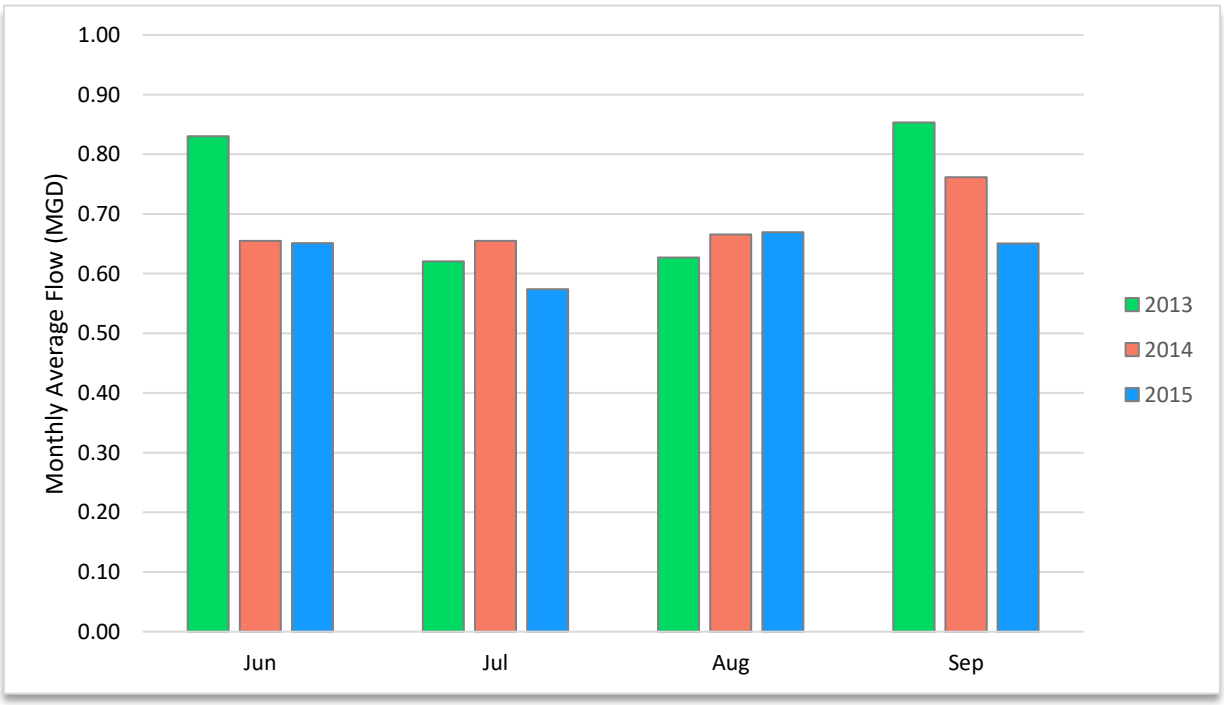

Figure 5-2 Average Flow During Dry Weather Months

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 Avenue 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 D. 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.

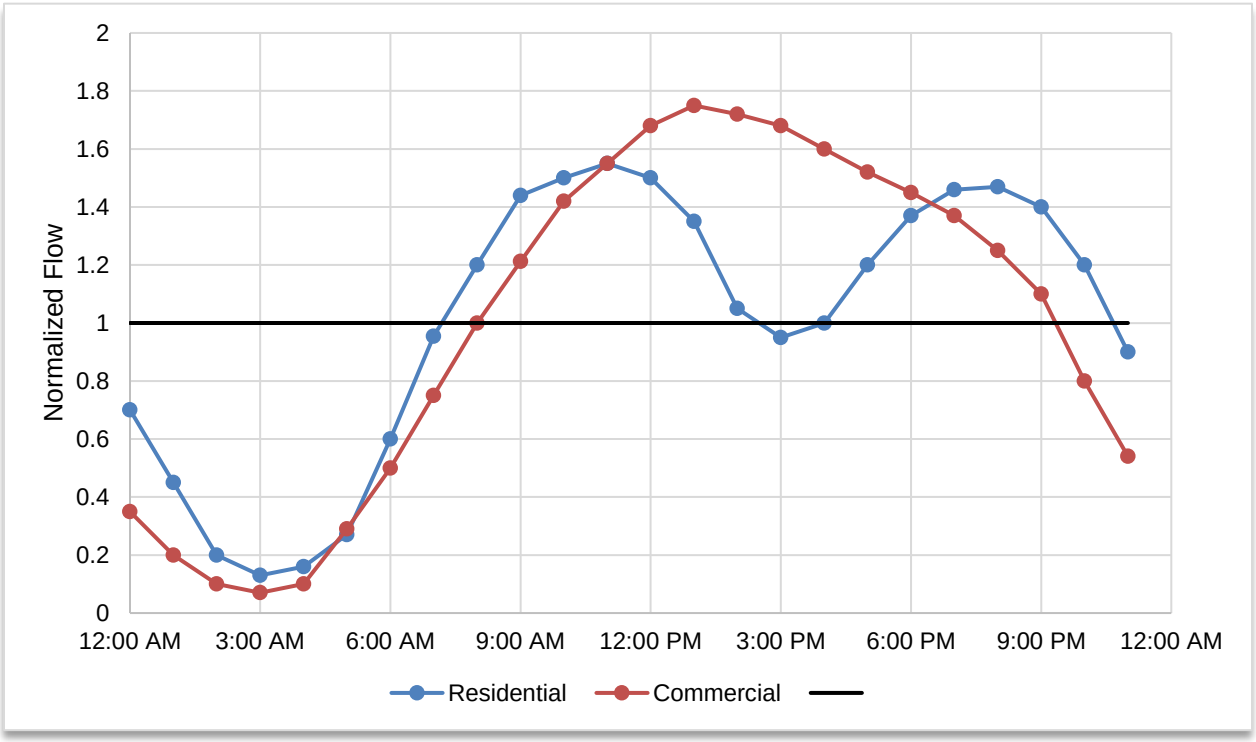

Figure 5-3 Residential and Commercial Sewer Diurnal Curves

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 Avenue 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 within the separated sewer service area suggests that there are a significant number of sources that have stormwater directly connected to the sewer system (i.e., roof drains), particularly within residential areas north of Blackmans Lake, that result in collection system deficiencies along Avenue A and at the Champagne Pump Station. Stormwater connections will be removed as they are found during sewer system improvement projects if there is an existing storm system in the area. Any remodeling or reconstruction project will be required to remove any stormwater connections or install a stormwater system to mitigate the impact of stormwater on the City sanitary sewer system. Additionally, capital improvement projects are proposed to address these capacity deficiencies.

As identified in Chapter 12, the City is in the process of separating the storm and sanitary sewers within the combined sewer service area 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, with the primary focus being CSO #2 as it has a greater potential for overflow events but is still considered controlled. 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 only about half the estimated 306 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

As discussed above, the model calibration (see Chapter 7) accurately predicted the historical average annual and maximum month flows. The historical average annual flow of 1.30 MGD is based on the average flow between 2006 and 2015, as shown in Figure 5-1 above. The historical maximum month flow is based on the highest maximum month of 2.73 MGD that occurred in December 2012, as noted in Table 5-4. The historical peak day flow is based on the highest daily flow of 8.30 MGD on December 8, 2015, also noted in Table 5-4. Because extensive historical hourly flow data was not available, the model predicted peak hour flow was adjusted by the same proportion as the model predicted peak day flow was to match the historical peak day flow. These flows are considered appropriate for modeling the collection system and examining long-term capacity needs, because they represent peaking factors for the historical worst case conditions. With those peaking factors having occurred in the past, they are expected to occur again at some point in the future. However, such conservative peaking factors are less appropriate for considering capacity needs in the short-term. As discussed in Chapter 8, the model predicts the influent maximum month flow would reach the NPDES permit limit of 2.80 MGD in just a couple year by 2022. Therefore, Chapter 8 also examines recent trends in the maximum month flow that need to be taken into consideration in the short-term and projects a more appropriate timeline for how soon influent flows would likely reach the permit limit of 2.80 MGD.

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, which is expected to reduce high I/I. Other actions the City may take to reduce high I/I, which would reduce peak flows presented herein, include removing direct connections of storm water if found in areas identified as high I/I. As discussed in Chapter 7, sub-basins adjacent to the east side of Blackmans Lake are one such area where the sewer systems are separated, but there appears to be significantly higher I/I than other separated areas.

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
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

Table 5-9 Total Sub-Basin Population Served					
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

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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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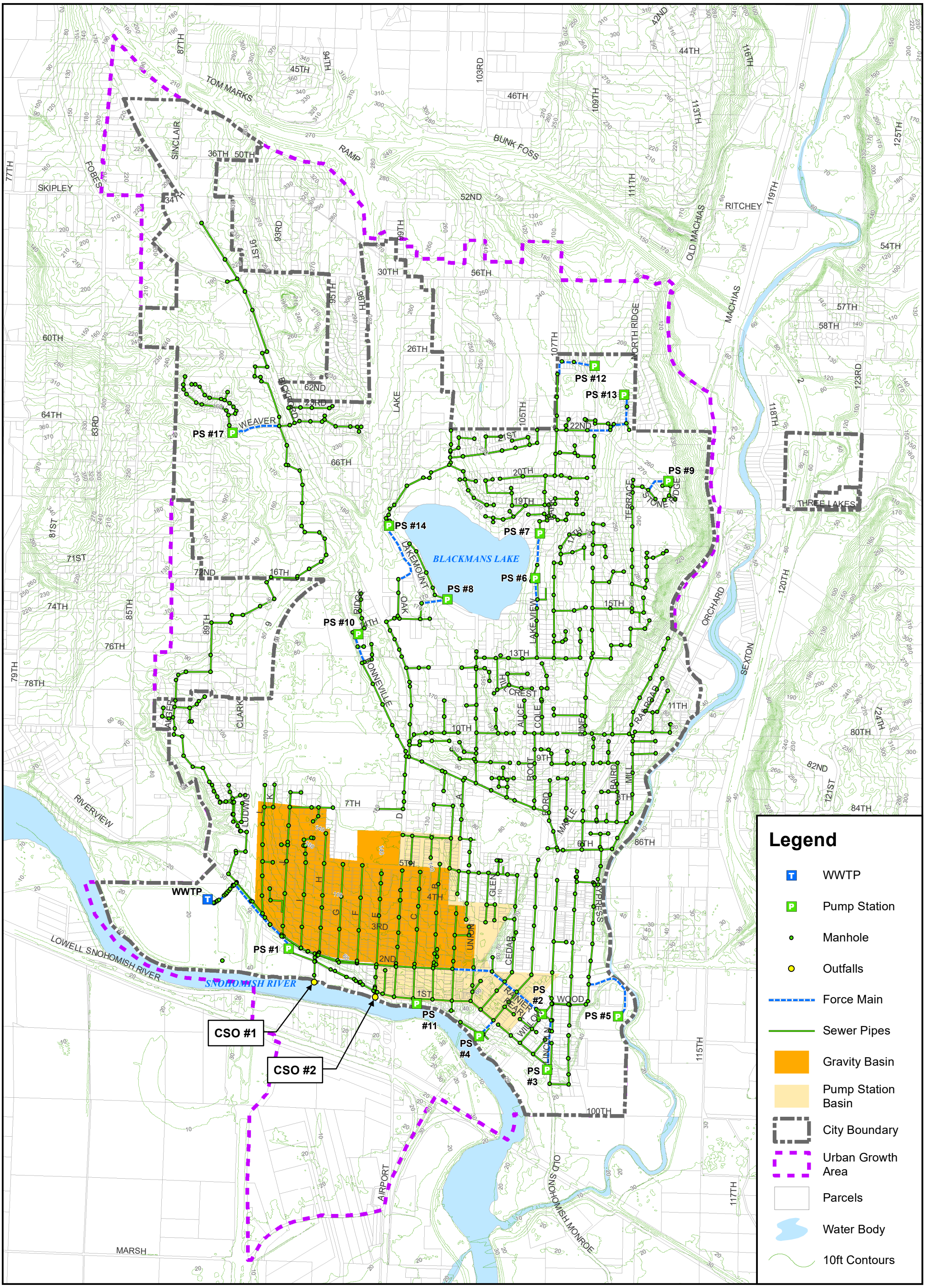

Sewer System & Basins: City of Snohomish.
GIS Base data: Snohomish County.
Data sources supplied may not reflect current or actual conditions. This map is a geographic representation based on information available. It does not represent survey data. No warranty is made concerning the accuracy, currency,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

General Sewer Plan and
Wastewater Engineering Report
City of Snohomish, Washington
September 2020

Figure
6-1

P:\Mapping\Maps_Generated\Snohomish_City\19-10615.00\003\maps\Fig 6-1 Existing Wastewater Collection System - 11x17.mxd 8/27/2020 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 (in)	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 (in)	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.

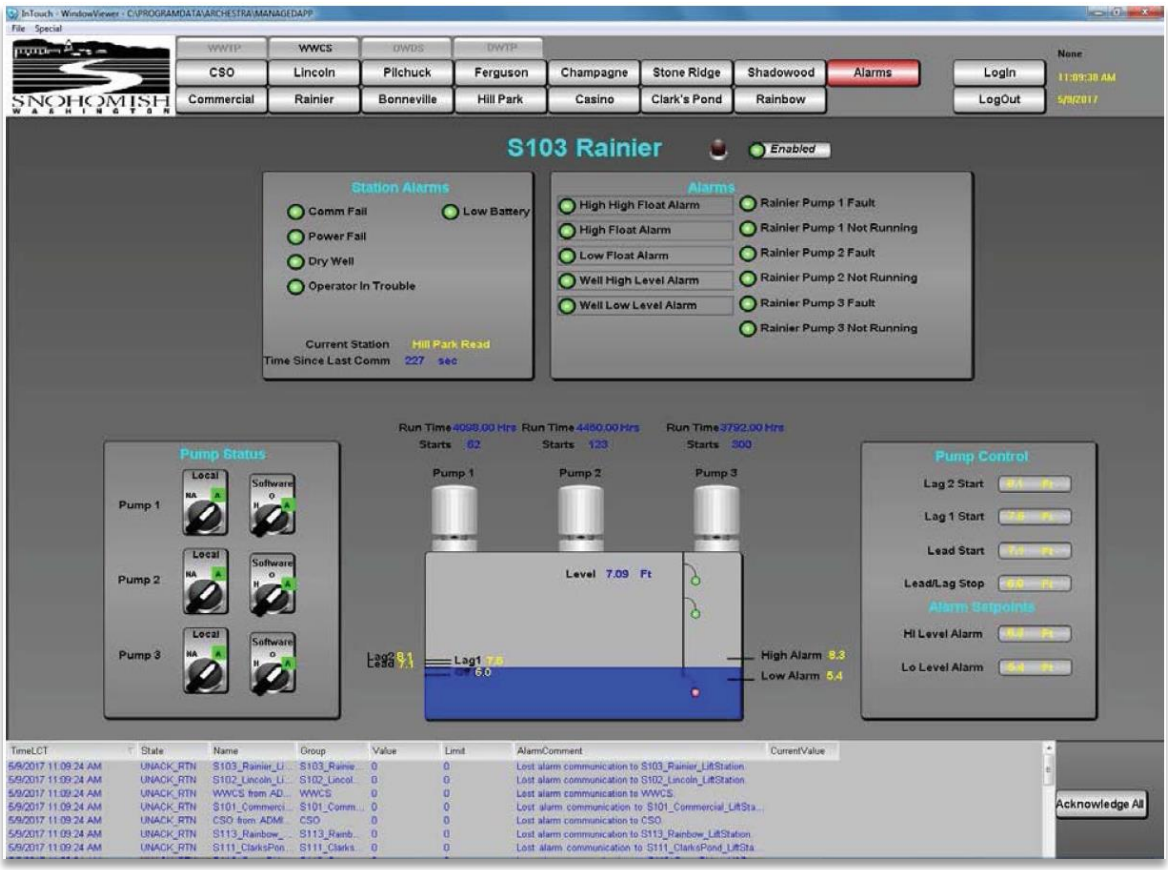

Figure 6-2 APPSVR02 Sample Screen Shot – Rainier Pump Station

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.

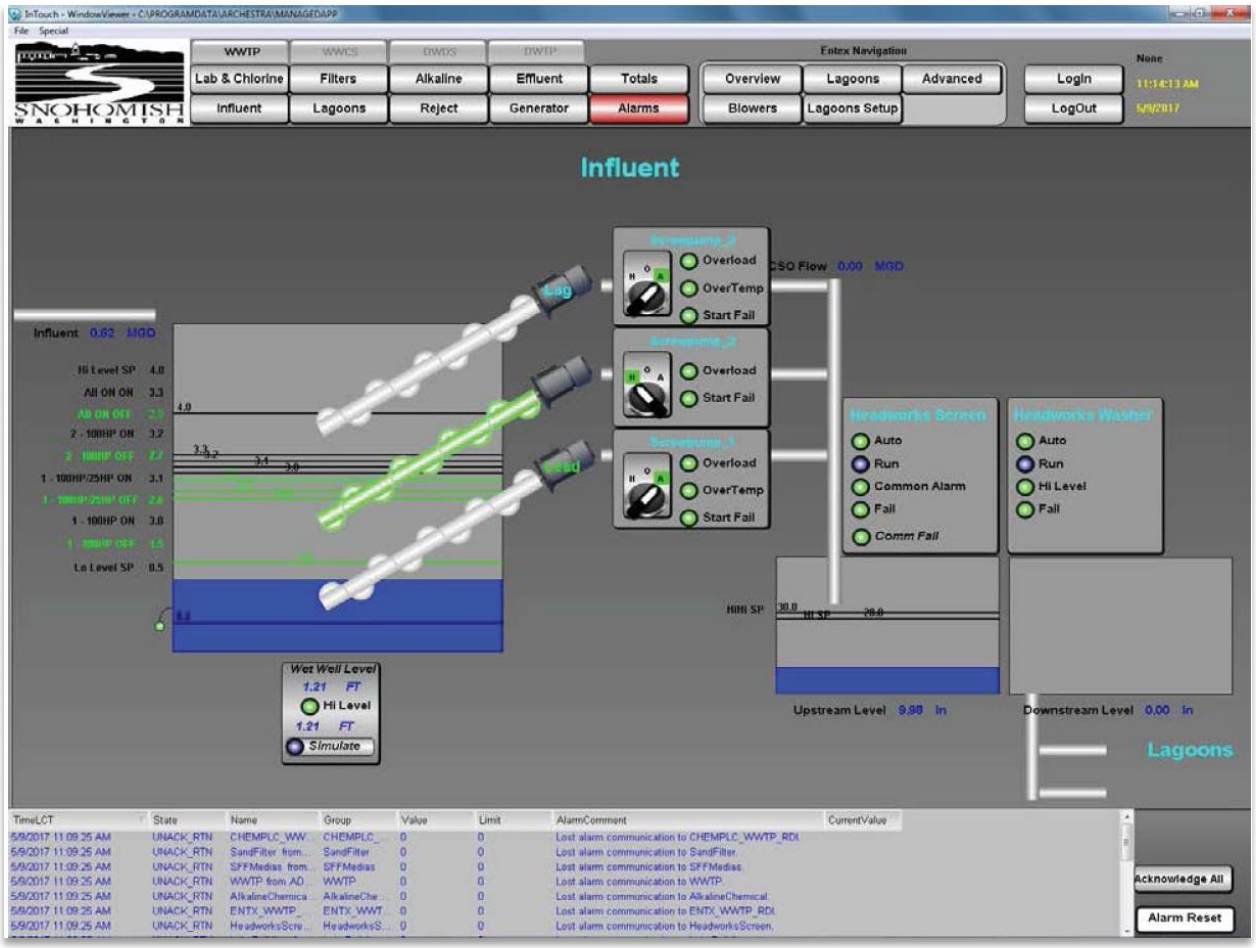

Figure 6-3 APPSVR01 Sample Screen Shot – WWTP Influent

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.

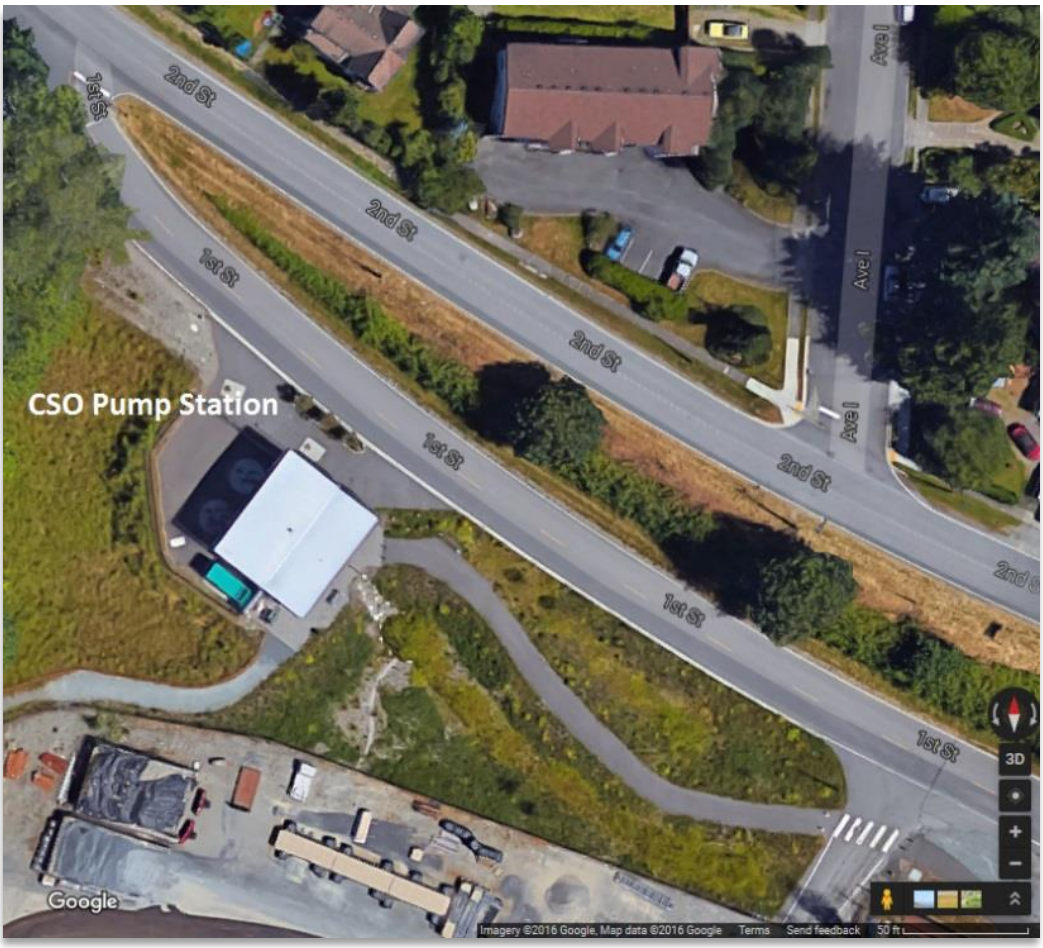

Figure 6-4 CSO Pump Station Location

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.

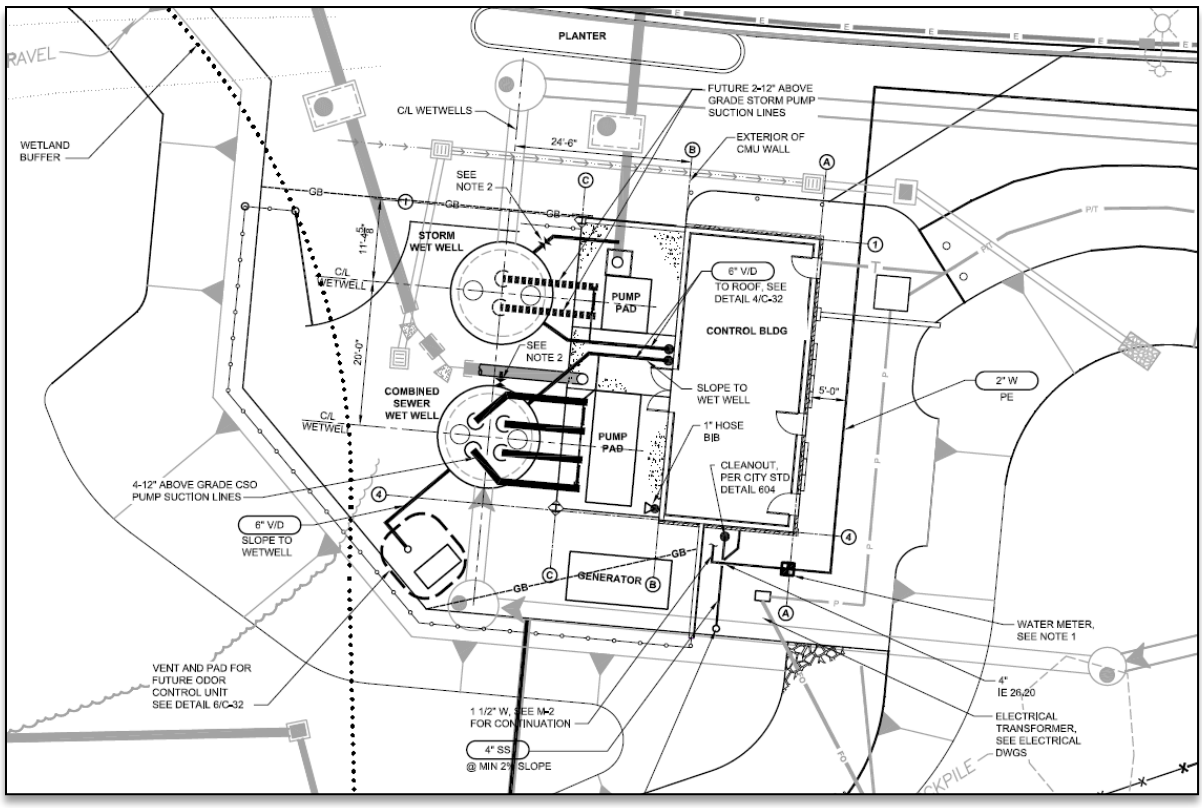

Figure 6-5 CSO Pump Station Layout

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.

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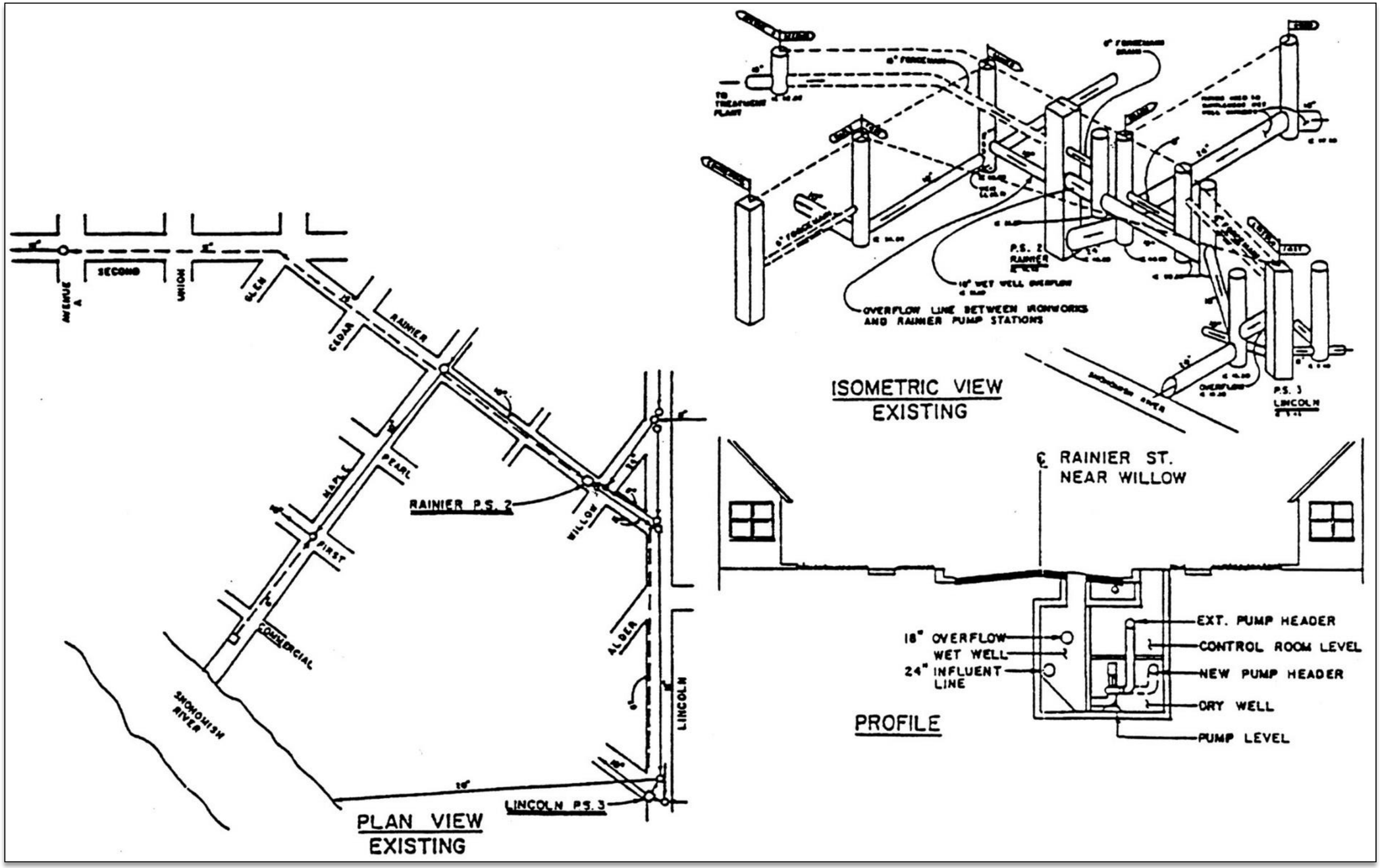

Figure 6-6 Rainier Pump Station Diagram (2005 Plan)

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6.4.1 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.

6.4.2 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.

6.4.3 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.

6.4.4 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.

6.4.5 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.

6.4.6 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.

6.4.7 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.

6.4.8 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.

6.4.9 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.

6.4.10 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.

6.4.11 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.

6.4.12 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.

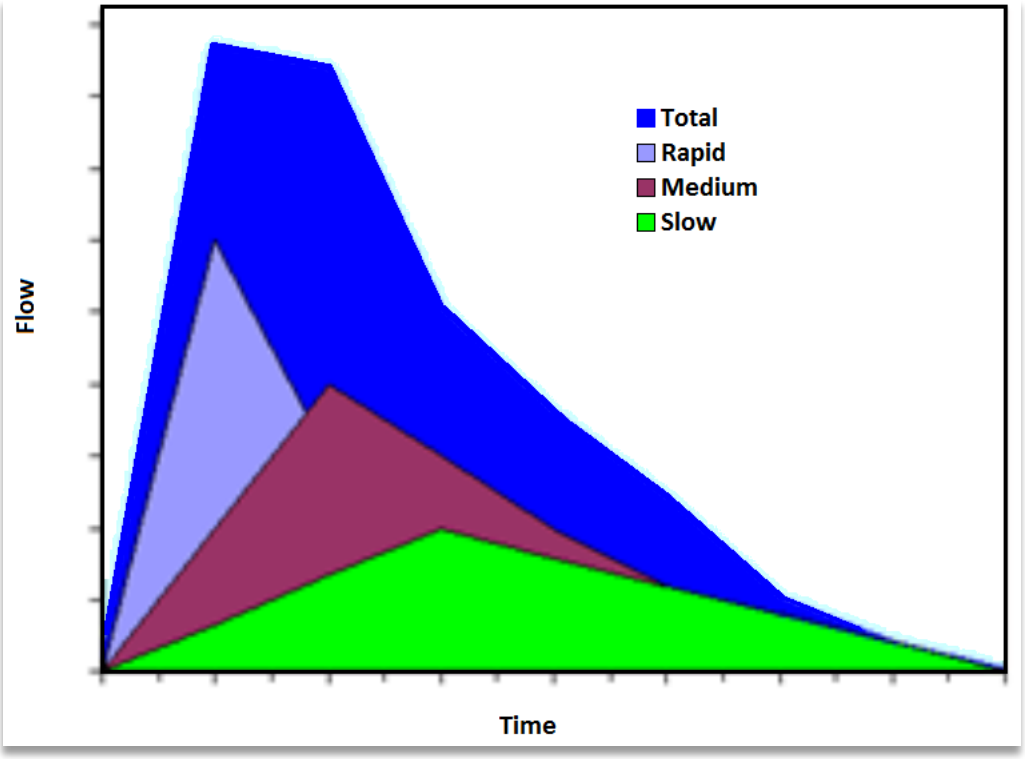


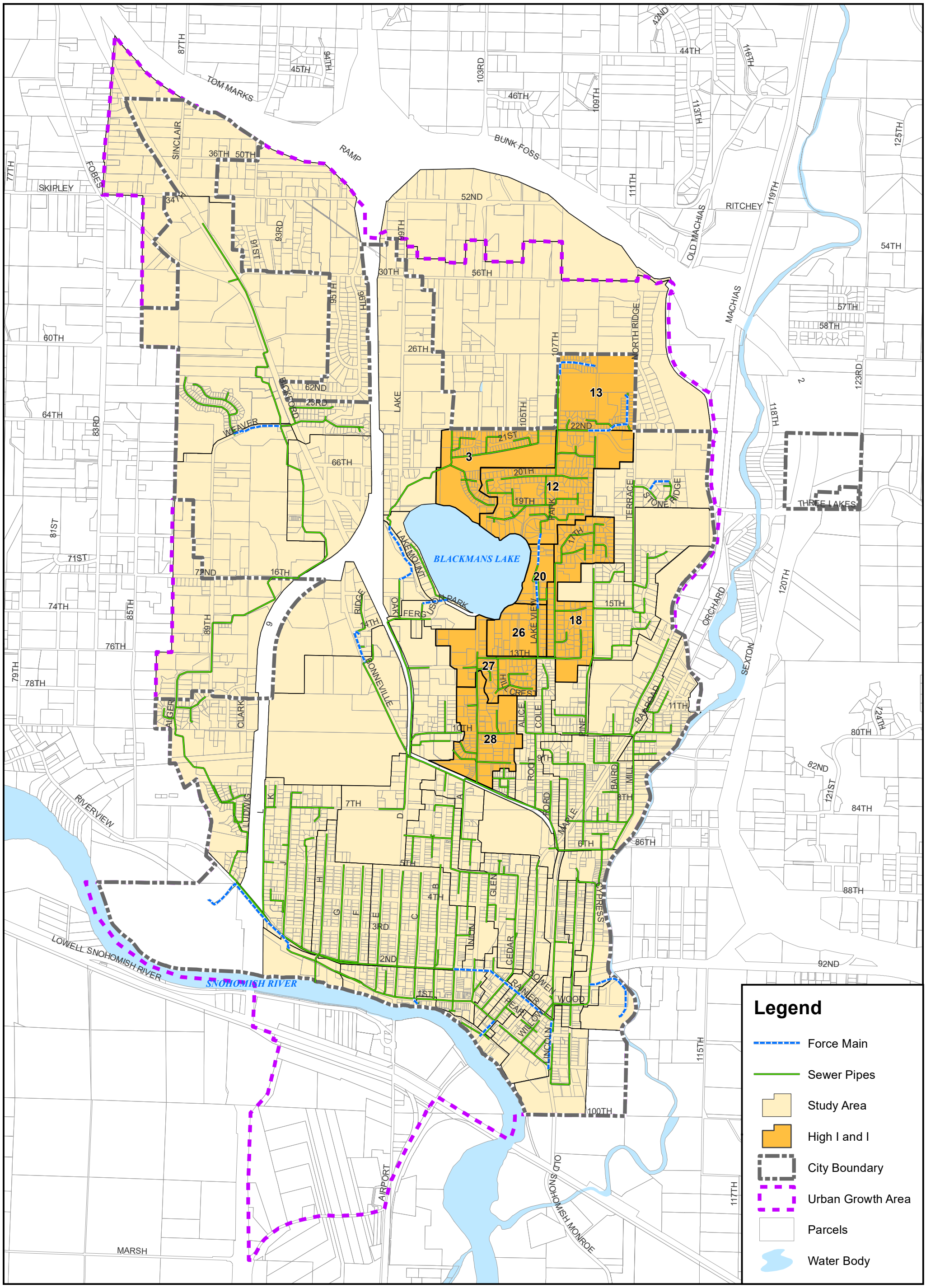
Figure 7-1 Example RTK Method Hydrographs for Modeling RDII

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.



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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0 750 1,500 Feet

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City of Snohomish Washington

Sub-Basins with High I/I

General Sewer Plan and
Wastewater Engineering Report
City of Snohomish, Washington
September 2020

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 8/27/2020 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.

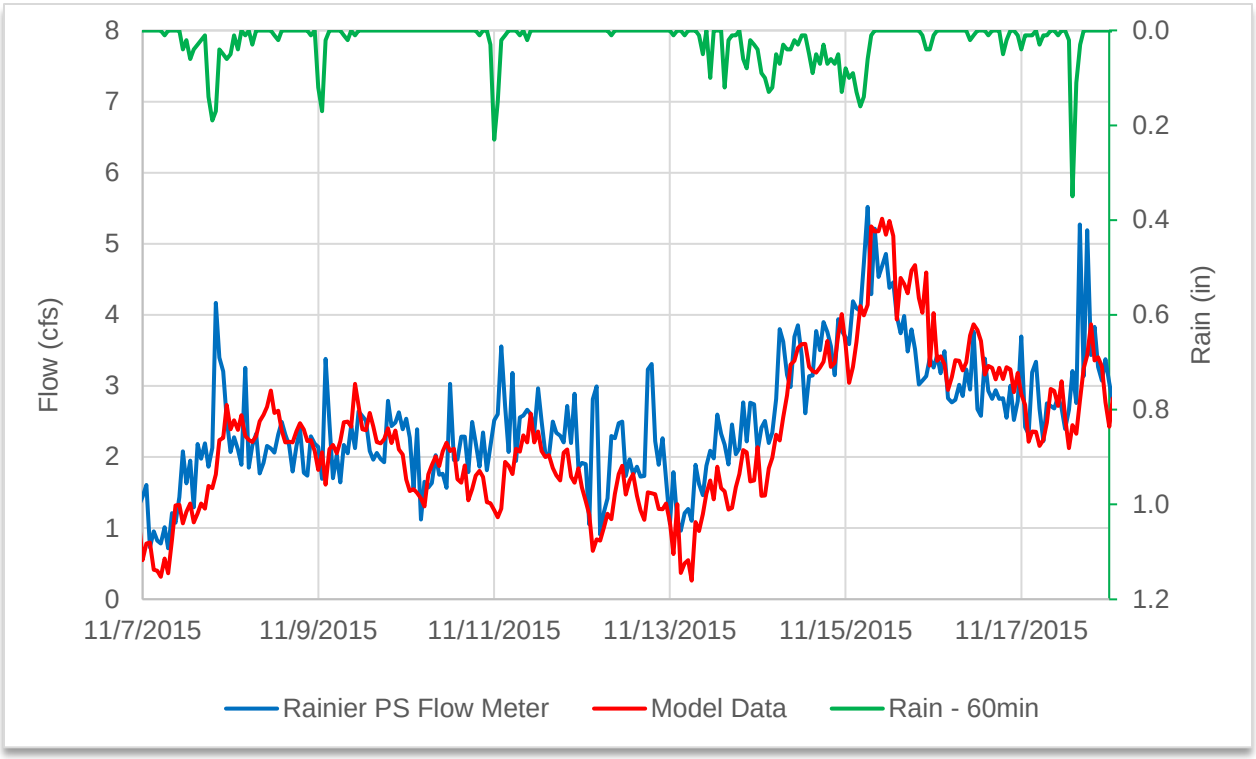


Figure 7-3 Flow At Rainier Pump Station

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.

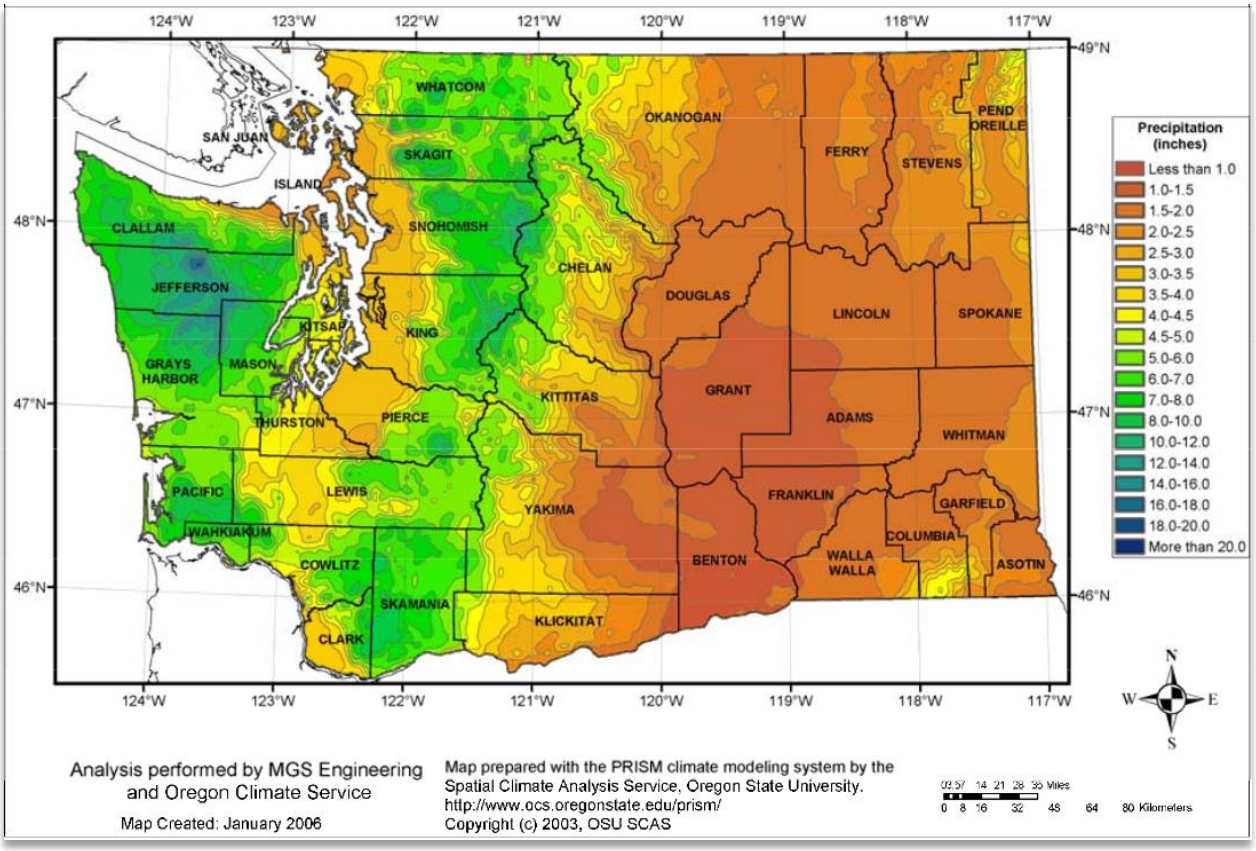


Figure 7-4 Washington State 25-Year, 24-Hour Isopluvial Map

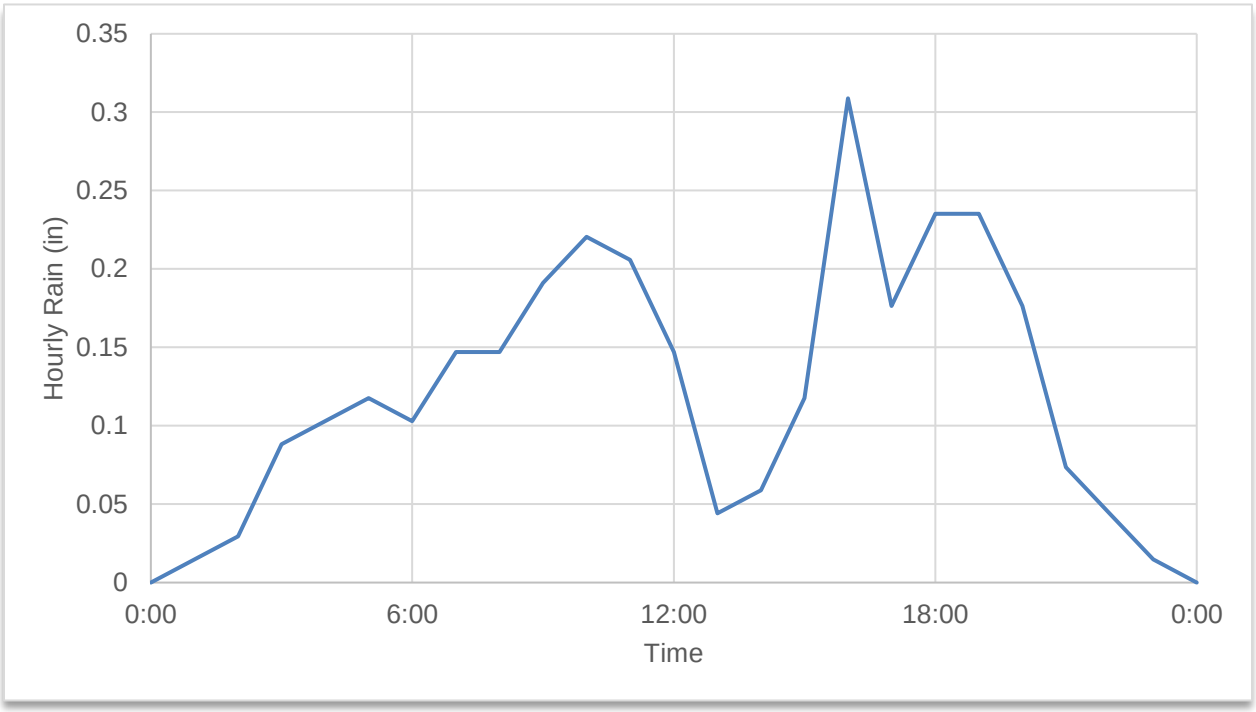


Figure 7-5 25-Year Design Storm

7.8.3 Existing (2016) Collection System Performance

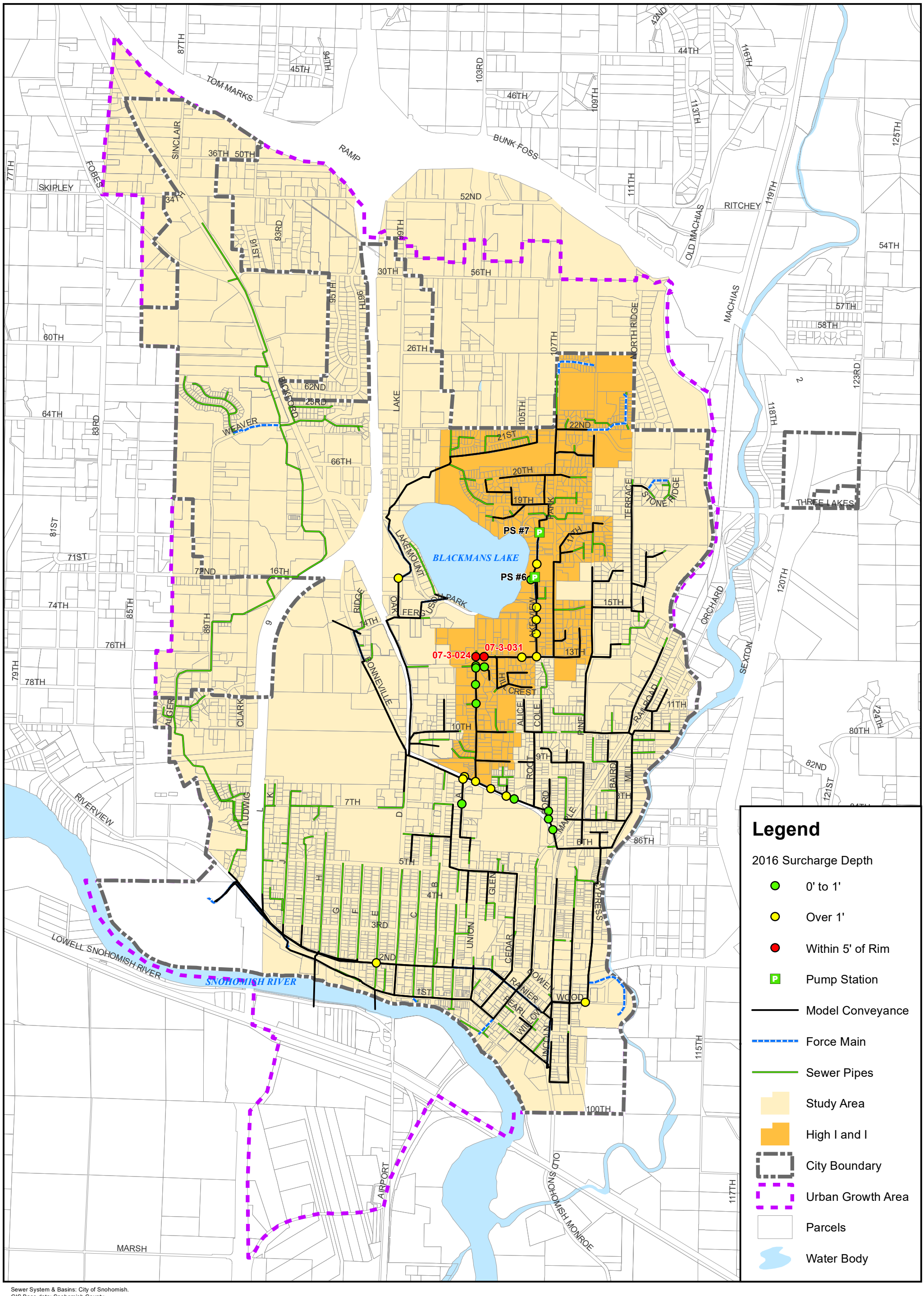
The existing collection system was evaluated to calibrate the model to 2016 flows and populations and determine if there are current deficiencies. Table 7-3 provides a summary of the existing (2016) gravity piping deficiencies. The current pump station deficiencies are listed in Table 7-4 showing the 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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0 750 1,500 Feet

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Existing (2016) Conveyance System Surcharging & Deficiencies

General Sewer Plan and
Wastewater Engineering Report
City of Snohomish, Washington
September 2020

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 has recently completed 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. The stormwater pipe installed along Avenue F and was recently extended to the stormwater utility lagoon for final separation 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 plan 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 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
Min. Monthly Average	0.66	86	1,235	85	1,262	75	1,081
Max. 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
Max. 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
Max. 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
Min. Monthly Average	0.57	103	1,448	94	448	90	818
Max. 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
Max. 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
Max. 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.

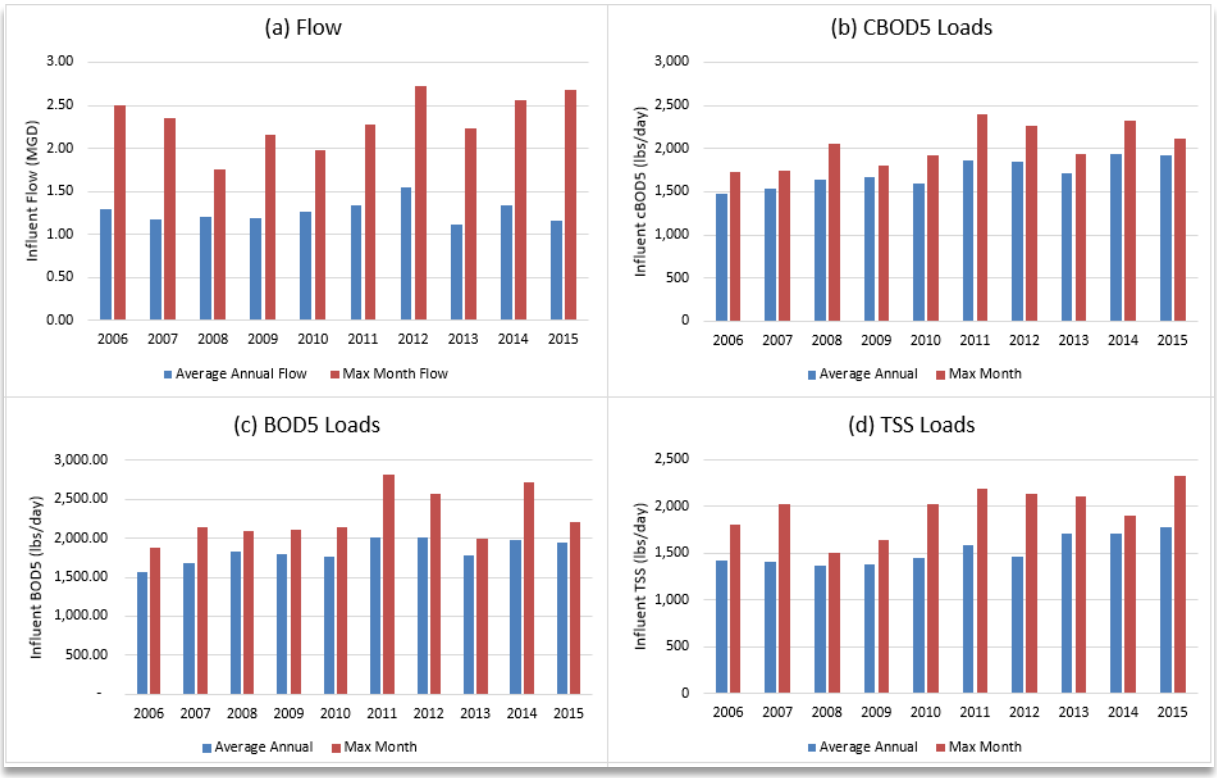

Figure 8-1 WWTP Annual Flow And Loading Trending from 2006 Through 2015

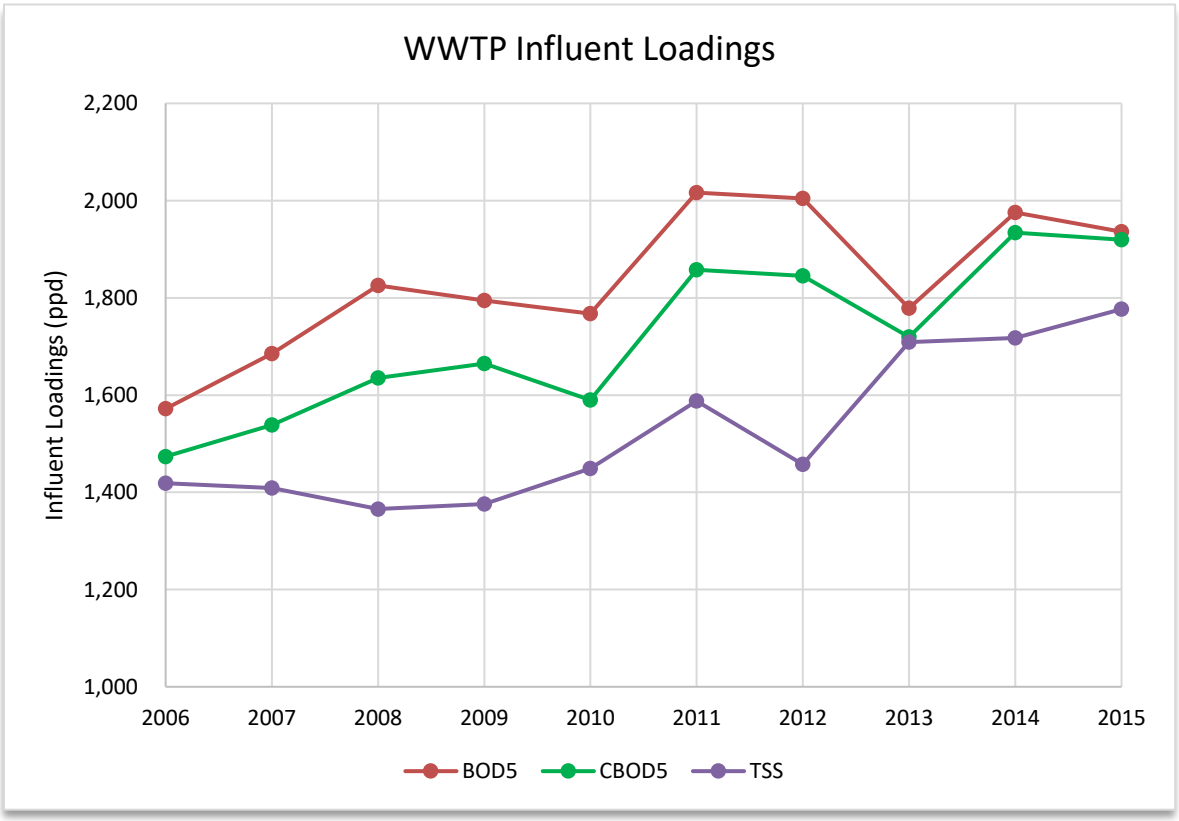

Figure 8-2 WWTP Annual Influent Loadings from 2006 Through 2015

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

Flow and load projections were compared with historical data from the WWTP discharge monitoring reports, as presented on Figure 8-3, Figure 8-5, Figure 8-6, and Figure 8-6. 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.

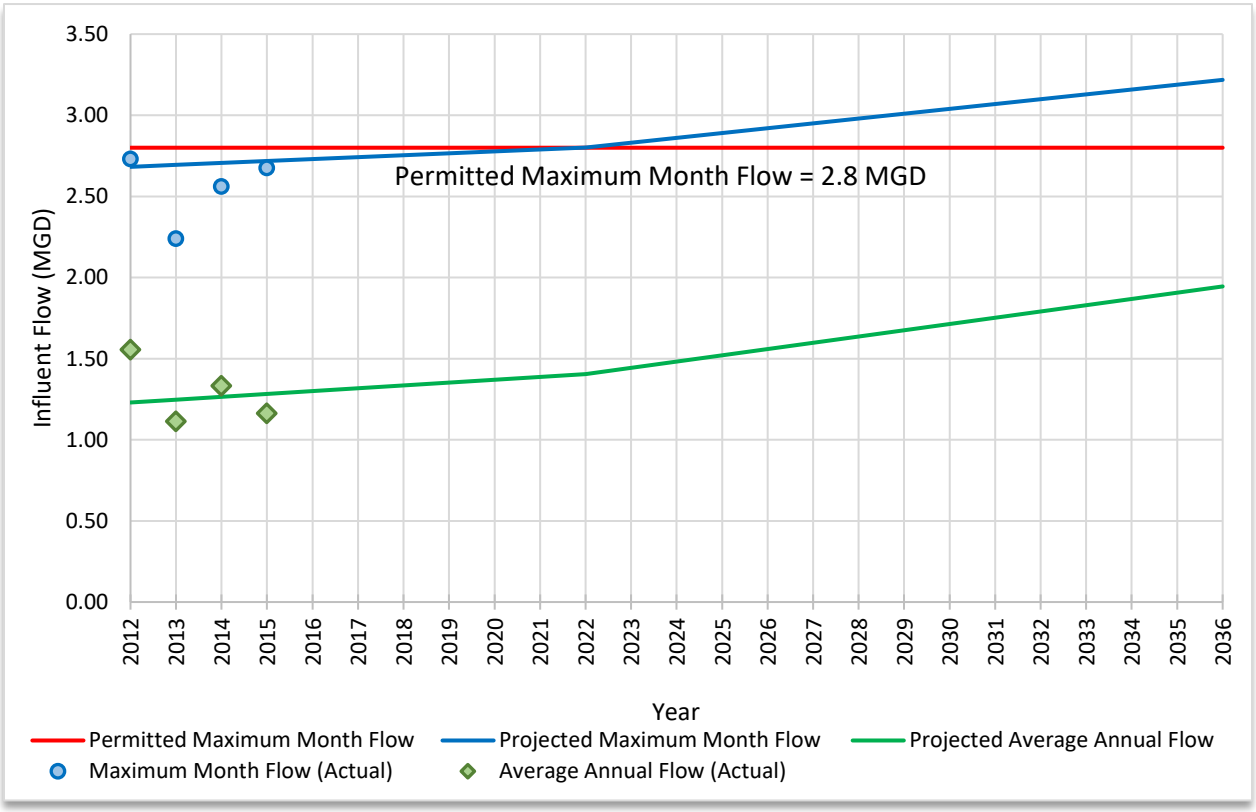

Figure 8-3 Maximum Month Influent Flow Projection with Historical Data

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 a series of extreme storm events 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. Based on this data from 2012 – 2015, the WWTP is projected to surpass its permitted flow of 2.8 MGD in 2023, which falls within the planning horizon. As additional influent flow data has become available, data from 2016 to mid-2020 was evaluated to compare with the maximum month flow projections presented in Figure 8-3.

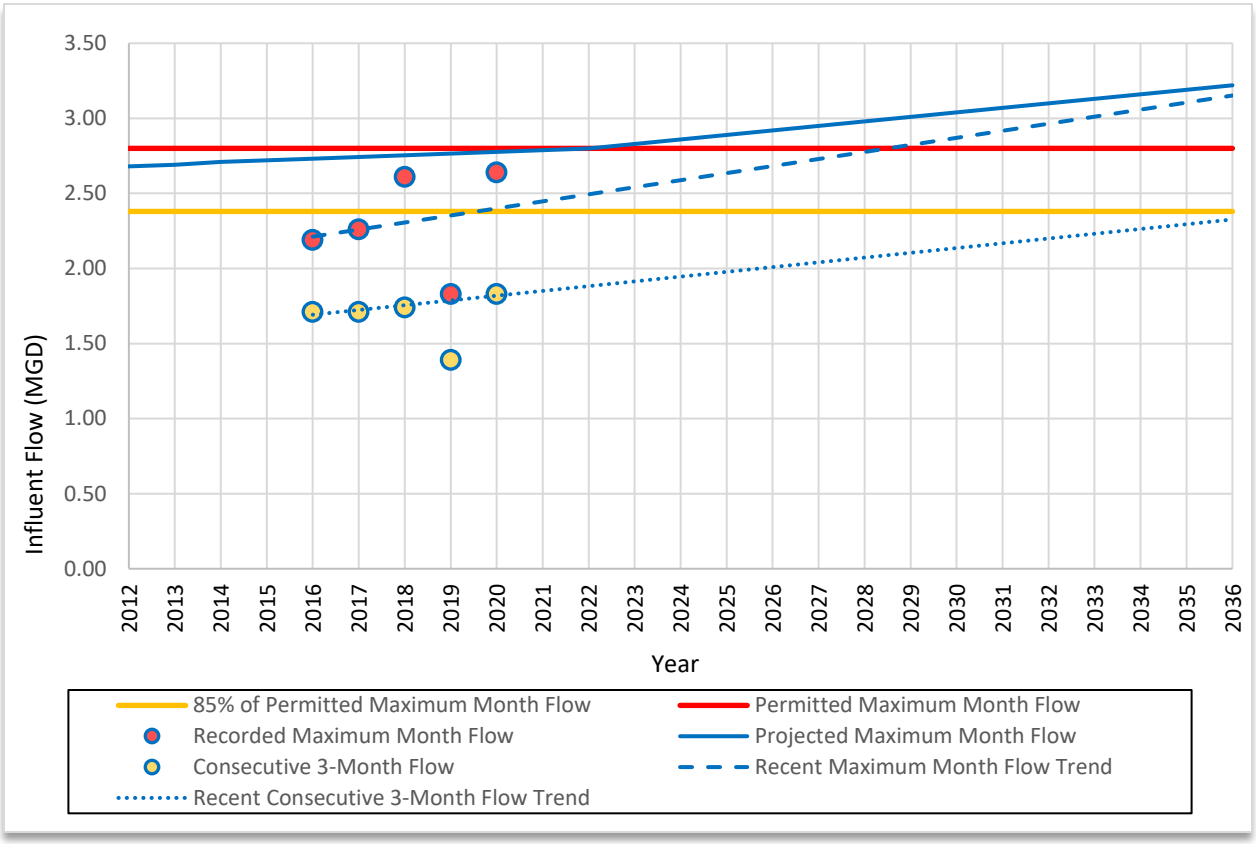

Figure 8-4 Maximum Month Influent Flow Projection with Recent Data

In Figure 8-4, the projections from Figure 8-3 are illustrated as the blue "Projected Maximum Month Flow" line. Figure 8-4 also includes the recorded maximum flow and the flow for the highest three consecutive month period (lowest flow of the three months) for each year between 2016 and 2020 (through July 2020). Trendlines extrapolate these recent flow trends compared to the permitted maximum month flow and 85% of the permitted maximum month flow. The recent trend for maximum month flow suggests that the WWTP will probably not exceed permitted capacity during any month until about 2028. The recent trend for three consecutive months suggest that the WWTP will probably not reach 85% of the permitted maximum month flow for three consecutive months, which requires the City to begin implementing capacity improvements regardless of other factors, until the end of the planning horizon. It is more likely that a single month will exceed the permitted maximum month flow before 85% of the permitted maximum month flow is exceeded for three consecutive months.

Whether or not the influent flow exceeds the permitted maximum month flow by 2023, as has been projected, or not until 2028 as recent trends would suggest, depends almost entirely on the weather and rainfall within each monthly period. Because the projection for maximum month flow is based on the highest peaking factor (in December 2012) for at least the last 15 years, it is likely that such a high peaking factor will not be experienced again before 2028. Additionally, the storm sewers along Avenue F have since been separated from the sanitary sewers and the City is planning to undertake additional combined sewer separation projects in the future, as outlined in Chapter 12 of this Plan, which will reduce the peaking factors. Therefore, even though projections indicate the permitted maximum month flow could theoretically be exceeded by 2023, it is acknowledged that a more realistic timeline for reaching the permitted flow capacity is around the year 2028. This later timeline will also allow the City to combine and coordinate increasing capacity to treat future flows with re-rating the existing

lagoon process (as discussed in Chapter 9) to account for the full capacity of the submerged fixed-film media and upgrading treatment to remove nitrogen (as discussed in Chapter 11) so that all three of these important elements are integrated into cohesive and cost-effective improvements. It is expected that the next step in developing these improvements will be conducting planning and optimization for nitrogen removal (as discussed in Chapter 12), which is anticipated to be a requirement of the forthcoming nutrient general permit to be issued by Ecology as early as June 2021.

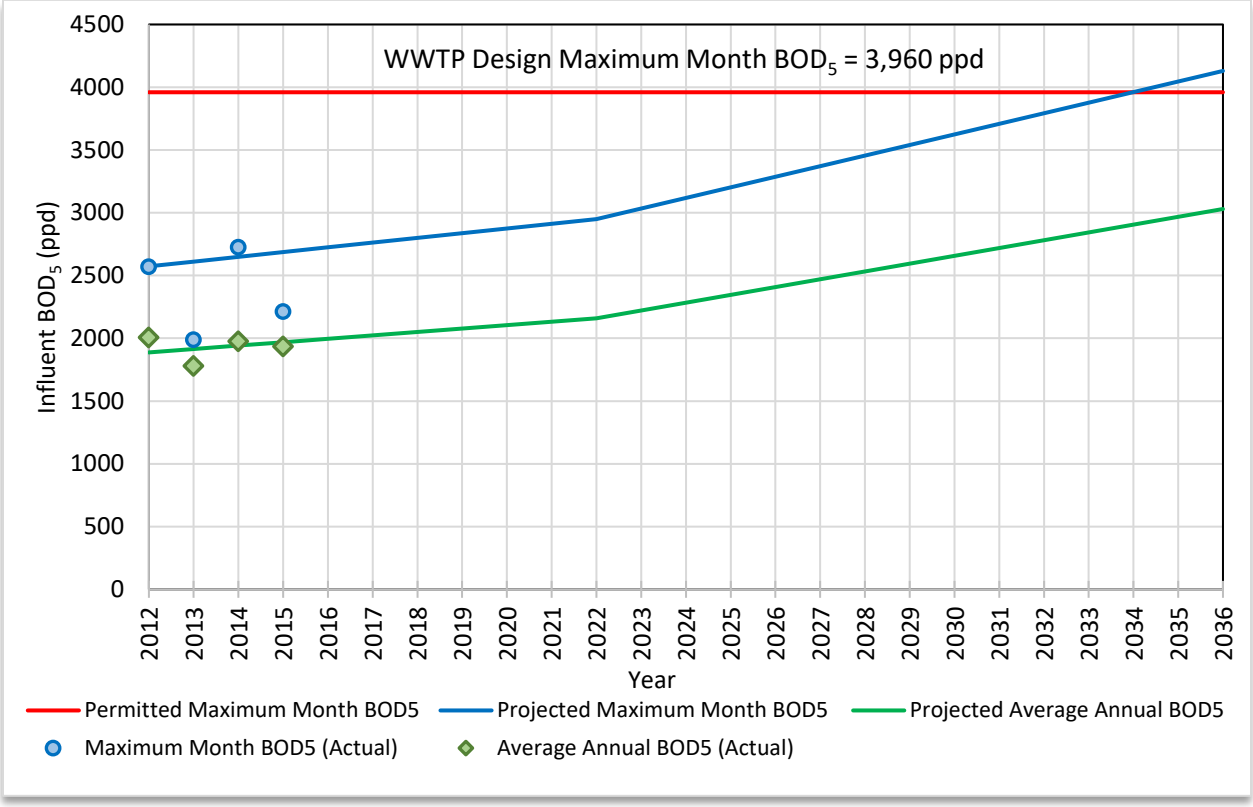

Figure 8-5 Maximum Month Influent BOD_5 Load Projection with Historical Data

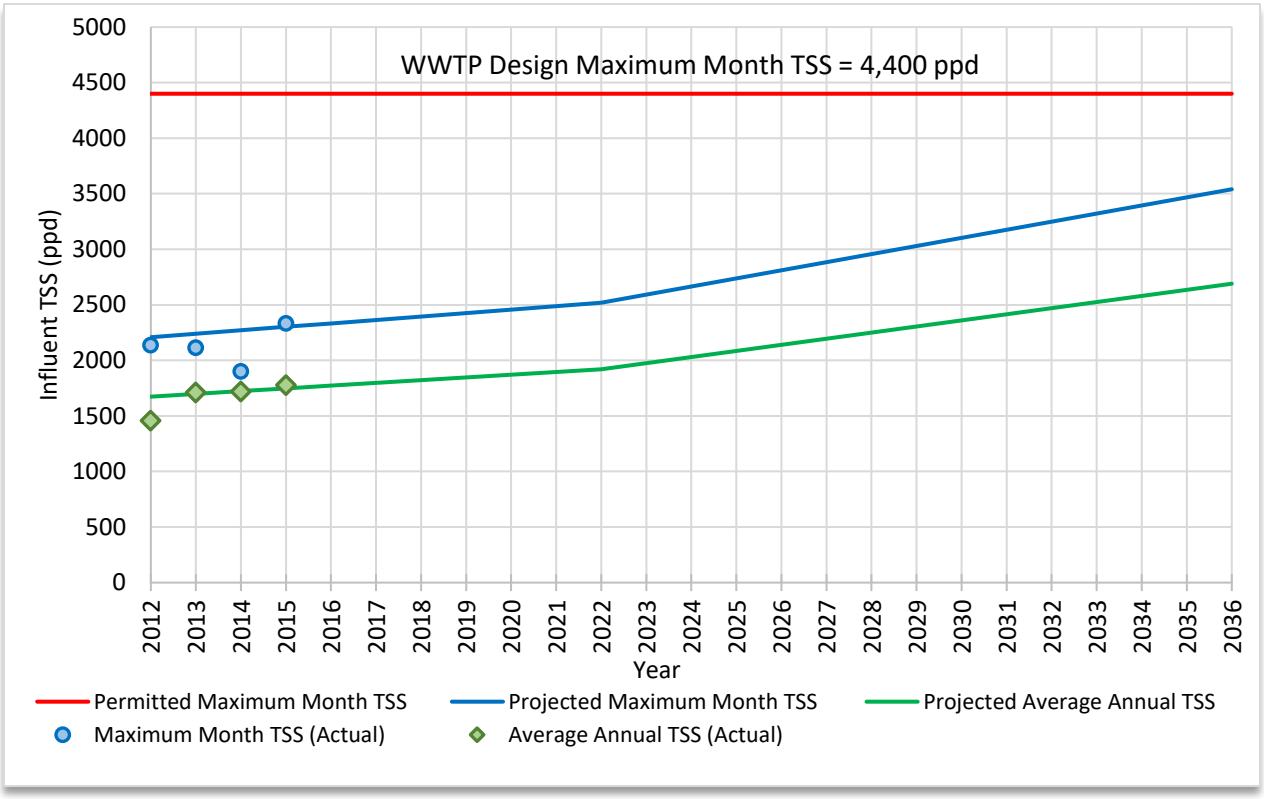

Figure 8-6 Maximum Month Influent TSS Load Projection with Historical Data

The planning horizon for this Plan extends through 2036. As indicated by the projections, the WWTP appears to have sufficient flow capacity through 2028, based on the current permitted capacity. After 2028 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.

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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.

The City recently installed a new generator to provide additional backup power at the south end of the WWTP in 2018 and just completed replacement of the chlorination and dechlorination systems for disinfection with a peracetic acid system in 2020.

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

BLOWER BLDG

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

CONTACT TANK

PAA STORAGE CANOPY

VAULT

FILTER PUMP STATION

PAA BUILDING

EFFLUENT CONTROL STRUCTURE

OUTLET STRUCTURE, TYP

DECANT STRUCTURE

GRAVEL PATH

INFLUENT LIFT STATION

CHAINLINK FENCE

CHEMICAL BLDG

GENERATOR BUILDING

WATER ACCESS BOX

LUMINARE ON CONC PAD

SURFACE AERATOR, TYP

LAGOON 1

LAGOON NO. 2

LAGOON NO. 3

LAGOON NO. 4

LAGOON DRAIN STRUCTURE

LAGOON DRAIN STRUC

SNOHOMISH RIVER

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 Engineering Report

September 2020

Figure

9-1

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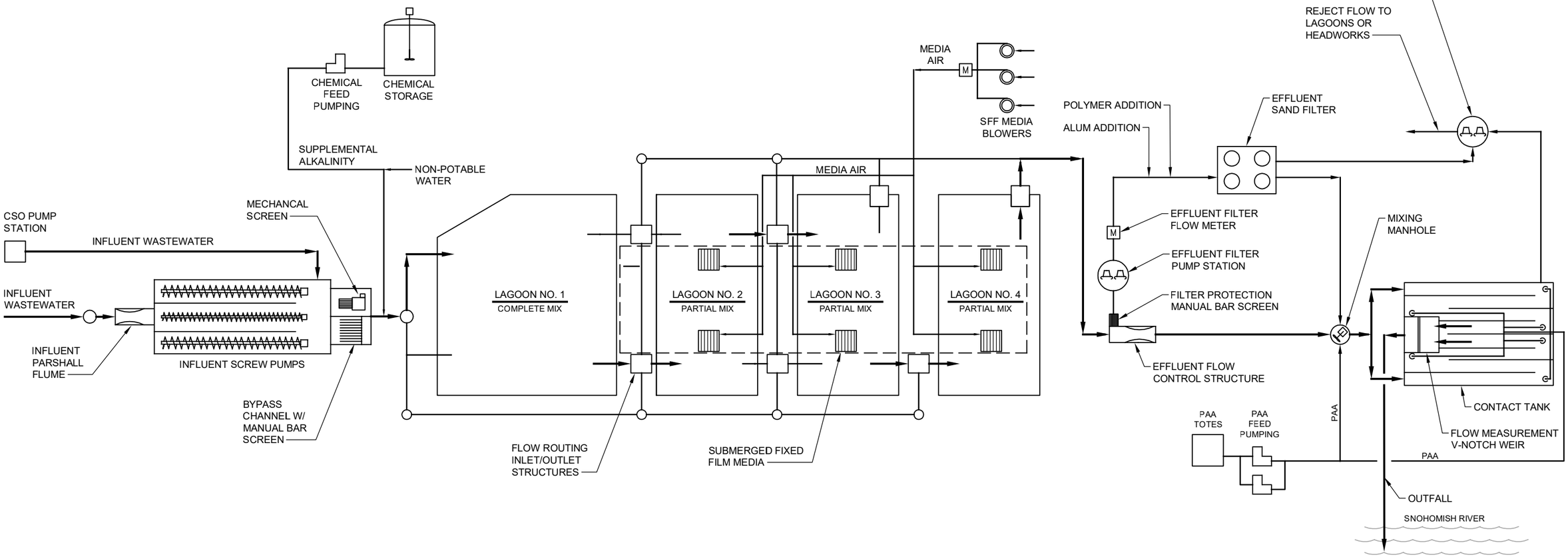

NOTE:

FLOW DIRECTION ARROWS REPRESENT NORMAL OPERATIONAL FLOW ROUTING.

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WWTP
Process Flow Diagram
General Sewer Plan and Wastewater Engineering Report
September 2020

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 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 E) 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 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 would need to be performed for higher flows before the WWTP could be re-rated for a higher flow. Further detail regarding a potential re-rating is provided in Sections 9.4.1 and 9.10 below.

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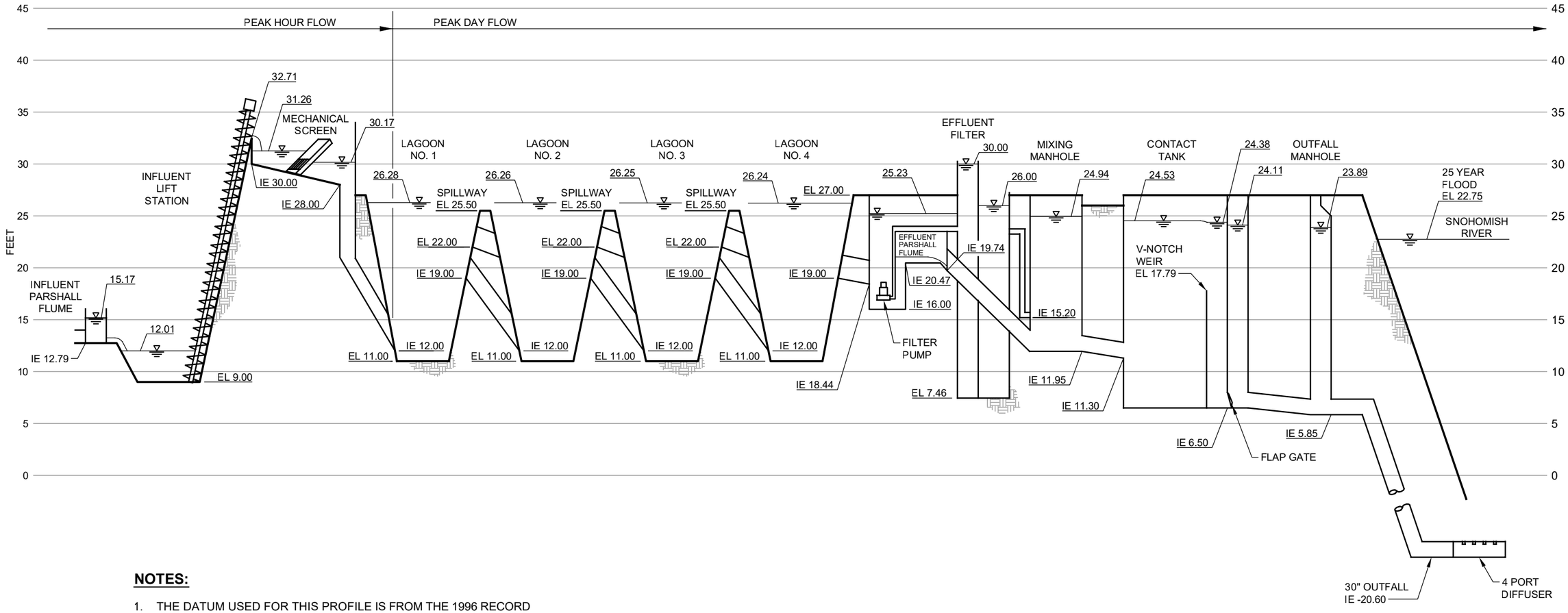

NOTES:

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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City of Snohomish Washington

WWTP
Hydraulic Profile
General Sewer Plan and
Wastewater Engineering Report
September 2020

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 at the end of the planning horizon in 2036 is higher than the WWTP's permitted capacity of 2.8 MGD. As discussed in Chapter 8, recent trends suggest the permitted capacity will be reached in 2028. As mentioned 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 data 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 are higher than projected for 2022 due to subsequent 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 F. 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 F.

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 F.

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. Specifically, the slopes of the lagoons are formed from top to bottom of quarry spalls, geotextile, top sand bedding layer, PVC liner, and bottom sand bedding layer. The bottom of the lagoons consist of the same elements, except there is a 4-inch layer of controlled density fill in place of the quarry spalls. 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 F. Additionally, detailed calculations of lagoon capacities are provided in Appendix G. 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**			
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 G.

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

Component	Design	Projected		
		2016	2022	2036
Type	**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. It is expected the facility will be able to achieve nitrification in the lagoons long-term, with a separate process added for denitrification of lagoon effluent to remove nitrogen to lower levels. See Chapter 11 for discussion and evaluation of alternatives to achieve denitrification.

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 F 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 Disinfection

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. Dosing of peracetic acid (PAA) is the existing disinfection process. PAA is a strong oxidant that is even more potent than chlorine. Because of this, it requires a significantly lower dose and typically less contact time to achieve the same level of disinfection. In general, residual PAA does not affect effluent toxicity as it diminishes quickly and does not form disinfection byproducts. Therefore, it does not need to be removed as with chlorine (i.e., dechlorination) unless there is an unusually high residual (typically greater than 1 mg/L). Because the average PAA dose at the WWTP is less than 1 mg/L, the residual is always well below 1 mg/L such that chemicals for quenching, such as sodium bisulfite or sulfur dioxide, are not required. Additionally, PAA does not react with forms of nitrogen, so there is not potential for nitrite lock or formation of weaker chloramine compounds.

A 15% PAA solution is stored in 300-gallon totes under a three-sided canopy to shade the totes from direct sunlight to avoid premature degradation and provide an area for the operators that is protected from wind and rain when switching between totes. Two totes are stored on a containment vault, though only one tote is in use at a time. The second tote is a backup. Based on the average annual flow and typical PAA dose required, less than 10 gallons per day of 15% PAA solution is required, such that each tote generally lasts a month or more.

There are two PAA metering pumps located in the PAA Building, one duty and one backup. Each pump is sized to deliver up to 3.2 gallons per hour, or about 77 gallons per day. At the projected 2036 peak day flow of 9.42 MGD and a dose of 1 mg/L, the PAA dose would be 56 gallons per day of 15% PAA solution. Although there may be times when the PAA dose exceeds 1 mg/l (perhaps up to 2 mg/L), a higher PAA dose is not required during peak flow events which yield diluted effluent and low fecal coliform concentrations prior to disinfection.

PAA can be dosed into one of four different locations. The most commonly used location is the mixing manhole just upstream of the contact tank. Neat PAA solution is pumped into the mixing manhole on the upstream side of a baffle that induces mixing of the chemical. The effluent then is discharged through two pipes, each conveying half the flow to each of the two chambers in the contact tank. Each chamber consists of a serpentine plug-flow channel with four passes providing sufficient contact time to reduce the fecal coliform count. Within each chamber there are three diffusers that are arranged identically. These diffusers provide alternate dosing locations for the PAA during low flow months. Introducing PAA further downstream reduces the contact time, which can be beneficial at low flows to maintain a measurable residual and prevent re-growth. PAA delivered through the diffusers is diluted with carrier water to improve diffusion of the chemical and add energy to the discharge from the diffusers for improved distribution. The diffusers are arranged vertically and consist of a number of small diameter holes to distribute the PAA throughout the cross section of the channel.

The contact tank has 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 of the two chambers is equipped with an inlet isolation valves, electrically actuated outlet gate, and drain valve. 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. A summary of the disinfection capacity evaluation is provided in Table 9-5 and 9-6 with details provided in Appendix F.

Table 9-5
WWTP Contact Tank Capacity Overview

Component	Design	Projected		
		Existing (2016)	2022	2036
Contact Tank				
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
Typical PAA Dose, mg/L	1	1	1	1
CT Value, mg-min/L				
at annual avg. flow	40	139	129	94
at max. month flow	40	67	65	57
at peak day flow	20	22	23	20

Previous testing of the PAA system established that a concentration-time (CT) value of 40 mg-min/L appears capable of maintaining fecal coliform concentrations at or below a conservative target of 50 cfu/100 mL and a CT value of 20 mg-min/L appears capable of maintaining fecal coliform concentrations at or below the permit limit of 200 cfu/100 mL. Although the WWTP normally uses both of the chlorine tank chambers, the WWTP could often utilize a single chamber to attain an adequate CT value if a chamber needs to be taken out of service for maintenance. However, the City will typically perform scheduled maintenance on both chambers at the same time during the dry weather months and hold wastewater in the upstream lagoons. Both chambers should be utilized to achieve sufficient disinfection during peak day flows, but if only one chamber were available a higher PAA dose could be used, which could potentially necessitate using both PAA metering pumps. Also, consideration could be given to raising the effluent V-notch weir to increase the depth in the contact tank and the resulting contact time. There are no identified capacity deficiencies with the existing contact tank throughout the planning horizon.

Similarly, there are no identified capacity deficiencies with the existing PAA dosing system. As shown in Table 9-6, the available PAA storage capacity is sufficient to provide PAA for nearly 2 months of use at a typical dose of about 1 mg/L and the City has space to store a third tote under the existing canopy too if desired. Additionally, a single PAA metering pump has sufficient capacity to deliver the typical dose of about 1 mg/L at peak flows. Additionally, the backup pump could be utilized if for some reason a significantly higher dose was required during peak flows.

Table 9-6
WWTP PAA Dosing Capacity Overview

Component	Design	Projected		
		Existing	2022	2036
Type	15% PAA Solution			
PAA Storage				
PAA Tote Volume, gallons	300	300	300	300
Number of Totes	2	2	2	2
Total PAA Volume, gallons	600	600	600	600
Average PAA Use, gpd	12	8	9	12
PAA Storage at Average Use, days	50	75	67	50
Peak PAA Use, gpd	59	49	48	56
PAA Storage at Peak Use, days	10	12	13	11
PAA Metering				
Number of Metering Pumps	2	2	2	2
Metering Pump Capacity Each, gpd	77	77	77	77
Metering Pump Total Capacity	154	154	154	154
Metering Pump Firm Capacity, gpd	77	77	77	77

Deficiencies

The existing contact tank can provide a CT value of at least 20 mg-min-L, as recommended from prior testing, to meet the permit limit for fecal coliform at peak day flows up to 9.5 MGD, which is slightly greater than 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. There are no deficiencies with the existing disinfection system. As mentioned previously, actuated gates have been installed at the outlets from the contact tank chambers, which can help to maximize flow attenuation in the lagoons and minimize flow to the contact tank to improve overall disinfection efficiency. Additionally, the contact tank recently underwent minor structural repairs to seal some cracks in the divider wall between the two chambers that made it difficult to isolate one chamber while keeping the other in service.

9.4.8 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 require 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). Previously, EMCC2 received emergency power from the emergency generator located near the Laboratory/Office Building through EMCC1. However, the City has since installed a second emergency generator at the south end of the plant that is capable of powering both EMCC2 and NMCC2. This second generator is 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 remains 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 1
- All three 7.5-horsepower surface aerators in Lagoon 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, the PAA disinfection system replaced both the previous chlorination and dechlorination systems with a simpler process. Additionally, before the addition of the SFF media, the City routinely seeded the lagoons with nitrifying bacteria to try and maintain a higher

level of nitrification. Because such recent improvements have simplified operations 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. 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.

The use of Class A reclaimed water would likely require removal of total nitrogen to less than 10 mg/L on a monthly average, as well as 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 currently have processes in place to remove nitrogen, but as discussed previously and in Chapter 11, this will be a requirement within the planning horizon due to the pending nutrient general permit. Additionally, Class A reclaimed water only requires nitrogen removal for certain uses, such as groundwater recharge, wetland enhancement, and surface water augmentation. 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. Additionally, the recently installed PAA disinfection process is not approved by Ecology of DOH as an appropriate, reliable disinfection technology for water reclamation. .Therefore, 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.

As discussed in Chapter 8, it is expected that the permitted maximum month flow will be reached in 2028. The City plans to address this capacity need and the pending requirements for nitrogen removal together, as these items are interrelated and improvements to address one will likely affect the other. Part of addressing the capacity need will be utilizing the existing processes to their full potential. At the time the SFF media system was installed in the lagoons, Ecology did not want to consider its impact on WWTP capacity, as it was still considered a new and developmental technology that needed to prove its effectiveness. Having proven its effectiveness over the last 8 years, it is appropriate now to determine how addition of the SFF media system has affected capacity of the WWTP to achieve its current permit limits. This re-rating effort would be undertaken simultaneously with planning and design of improvements to meet pending nitrogen removal requirements. As discussed below, initial analyses suggest that improvements identified herein would allow the WWTP to meet current permit limits at a maximum month flow of up to 3.30 MGD.

9.10.2 Analysis 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 F. 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 G. 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 BOD_5
- 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 and likely more. 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 has 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 at least 2.8 MGD, the effluent filters would filter a minimum of 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.

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 would 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 F and G. 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 to include the analysis in this Plan in a future engineering report that will address both this proposed re-rating effort and improvements for nitrogen removal. As discussed previously, this Plan has established the technical basis and proposed re-rating for the WWTP. Regarding evaluation and monitoring to demonstrate performance, the City could 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. The City could 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 $CBOD_5$, 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 would 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 with the filters operating at full capacity the resulting effluent TSS at the re-rated maximum month flow is projected to be 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: If some or all of the improvements identified herein as being necessary to support the re-rating are made before the actual re-rating is required, the City may be able to demonstrate performance of these improvements without requiring provisional approval. Otherwise, the City will conduct the necessary evaluations during a provisional approval period.

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 in 2009 dollars. Adjusting for the ENR Construction Cost Index, this cost becomes $50 million in 2018 dollars. 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. 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 was estimated to amount to an additional cost of about $64 million in 2009 dollars, which is equivalent to $82 million in 2018 dollars. 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 (approximately 10 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 $130 million, including approximately $50 million in capital costs to construct the conveyance system and an assumed $82 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 into 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.

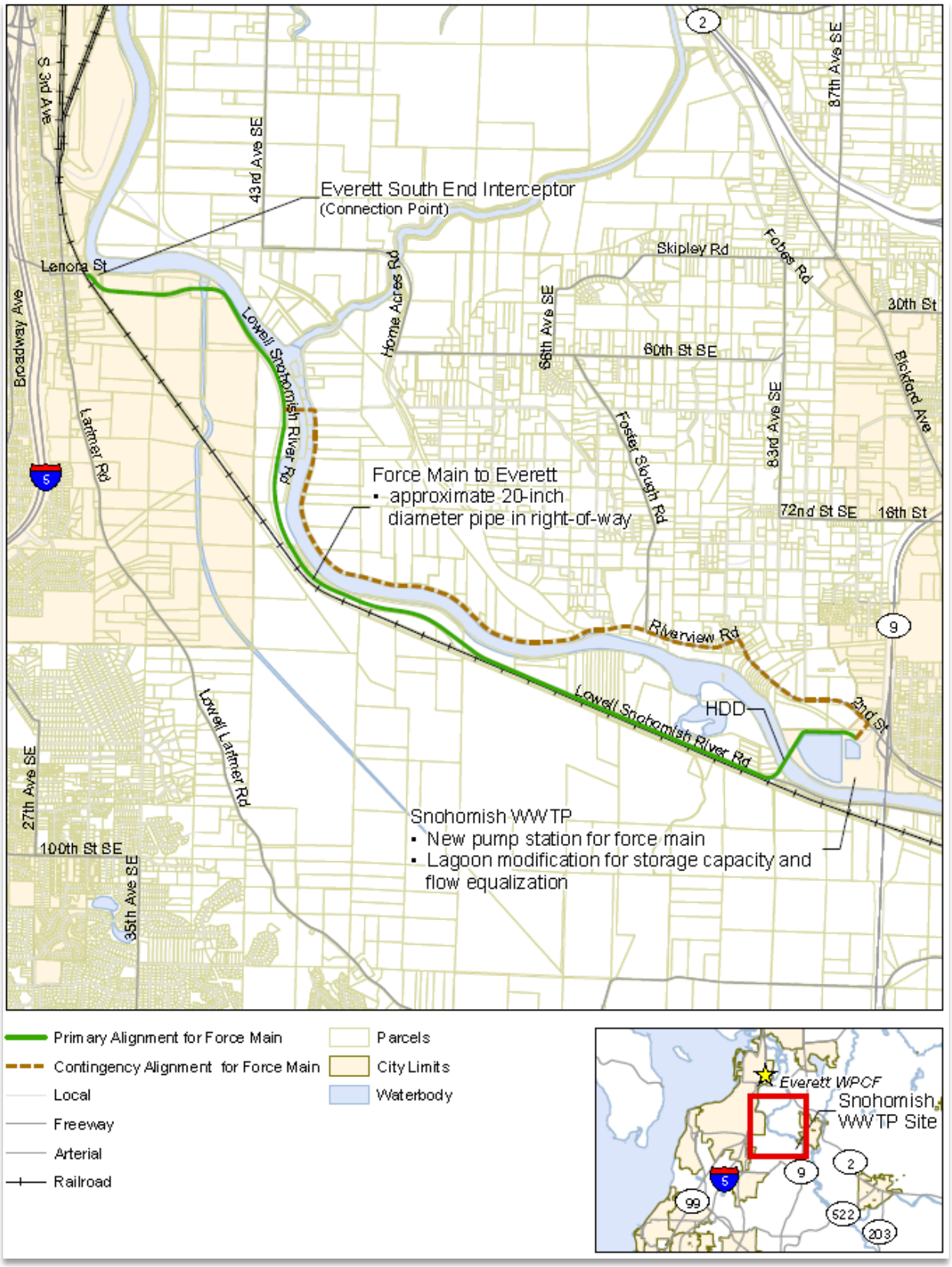


Figure 10-2 Snohomish to Everett Conveyance Route (2010 Plan)

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 in 2009 dollars. Adjusting for the ENR Construction Cost Index, this cost becomes $30 million in 2018 dollars. 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. It is assumed the initial Everett connection charge (capitalized cost) will be approximately $20 million in 2009 dollars, which is equivalent to $26 million in 2018 dollars, for a total capitalized cost of about $56 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 and the City had already prepared some initial design documents for this alternative, it was chosen as the favored alternative for regional wastewater treatment. However, now being 10-years removed from the original evaluation of regionalization, the City would need to confirm with the City of Everett that adequate capacity remains at their WWTP to accept all of the flow from the Snohomish WWTP and determine how much the connection charge has increased with the pending nitrogen removal requirements that the City of Everett also faces before proceeding further with any consideration of regionalization.

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 $56 million in 2018 dollars, including an estimated $26 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 decided to proceed 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 SO_2 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.
- Replaced old automatic transfer switches for the generator at the north end of the WWTP.
- Added a second emergency generator at the south end of the WWTP.
- Recently replaced the old chlorine disinfection system and dechlorination system with a peracetic acid (PAA) disinfection to improve safety, simplify the disinfection process and comply with current codes.

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 be 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.

Additionally, 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.

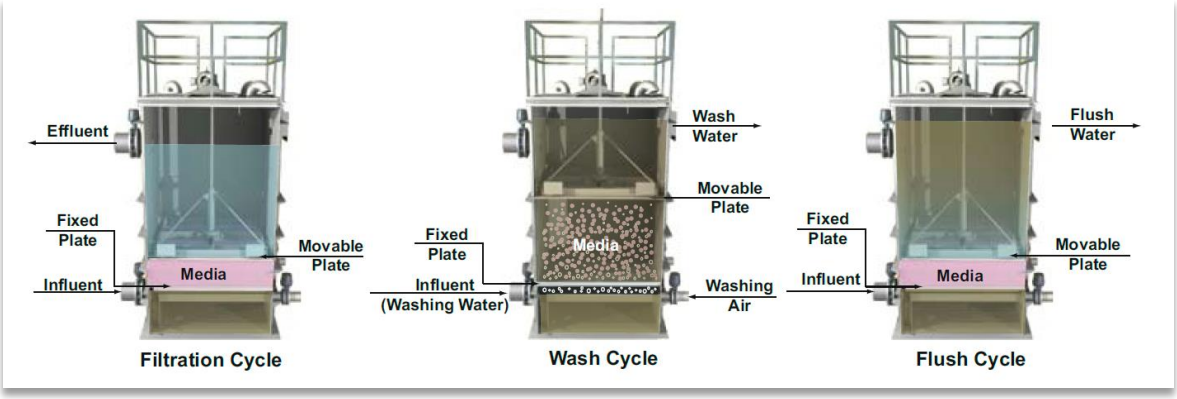


Figure 11-1 Schreiber Fuzzy Filter

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. A copy of the pilot test report is included in Appendix M. 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 wit-h 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

Table 11-1
Filtration Alternatives Design Summary[a]

Parameter	Units	DynaSand	EcoWash	Fuzzy Filter	FlexFilter
Reject Flow Duration	minutes/cycle	continuous	8 to 22	15	75
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

Opinions of probable construction cost and project cost (see Appendix H) 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 11-2 include markups for site work, mechanical/yard piping, electrical, instrumentation and control, mobilization, contractor overhead and profit, contingency, sales tax, design, and construction management.

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
Yearly Chemical Cost[b]	$/year	64,600	52,100	51,200	51,200

Table 11-2
Filtration Alternatives Cost Summary[a]

Parameter	Units	Parkson DynaSand® (Sand)	Parkson EcoWash™ (Sand)	Schreiber Fuzzy Filter (Compressible)	WWetco Flex Filter™ (Compressible)
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 H 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 H 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. As discussed in Chapter 4, the City is expected to face future nitrogen limits. The sand filters could potentially be utilized as denitrification filters to achieve some level of nitrogen removal prior to discharge. This could be used as part of a nitrogen removal strategy to meet future limits resulting from the Puget Sound Nutrient Source Reduction Project.

11.3 Nitrogen Removal Alternatives

Due to Ecology's current work on the Puget Sound Nutrient Source Reduction Project, it is expected that a nutrient general permit is to be issued by Ecology as early as mid-2021. The initial general permit, in which Ecology has indicated the City of Snohomish will be included, is expected to include a cap on the discharge of TIN based on current WWTP performance and design capabilities. To that end, this section investigates the feasibility of different technologies to achieve nitrogen removal to a level of 8 mg/L of TIN, but which is also capable of being upgraded/expanded to achieve TIN of 3 mg/L, consistent with the range of potential limits discussed in Chapter 4.

The WWTP currently nitrifies in compliance with its NPDES permit, from July through October. Nitrification is achieved using an aerated submerged fixed-film process. Aeration is continuous during the summer months when ammonia limits are in place and is reduced to intermittent aeration in the winter to reduce energy use. Complete nitrification is expected to be possible in the lagoon throughout the winter months using continuous aeration, though this still must be confirmed. It appears that some amount of denitrification could be occurring in the lagoons, but it has not yet been determined if this can be increased and optimized by adjusting the aeration rates/patterns to the submerged fixed-film media while still achieving complete nitrification. Effluent data indicates a typical effluent nitrate concentration of about 25 mg/L. As discussed above, optimization of aeration could yield more denitrification within the existing process, but it is conservatively assumed the effluent nitrate concentration would remain around 25 mg/L.

Three denitrification processes were identified for evaluation that could be added to supplement the existing secondary treatment process within available space and achieve the anticipated effluent TIN limit of 8 mg/L: 1) denitrifying sand filters, 2) Microvi MicroNiche Engineering™ (MNE) biocatalyst, and 3) a denitrifying moving bed bioreactor (MBBR). All three alternatives require the addition of supplemental carbon to reduce nitrate to nitrogen gas, since insufficient carbon remains in the lagoon effluent to support sufficient denitrification. Given that all three processes would see the same effluent nitrate concentrations at the same flows and operate under similar kinetics, each process is estimated to use the same amount of supplemental carbon, which is based on a carbon to nitrogen ratio of 4. It is assumed that supplemental carbon would be added as a non-hazardous proprietary carbohydrate-based product (i.e., MicroC) for ease of handling and storage. Because Ecology has indicated that the TIN limit will be based on an average annual mass limit, the denitrification process would be sized for at least 2.0 MGD to accommodate the projected average annual flow for the planning horizon of 1.95 MGD. This will allow cost-effective sizing and more significantly limit use of supplemental carbon to what is necessary to achieve the average annual mass load limit.

11.3.1 Evaluation Criteria

The evaluation and comparison of denitrification 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.
- Process Integration – consideration of how the denitrification process integrates with existing facility operations and whether adjustments to upstream or downstream processes are required to ensure permit compliance and successful operation.
- Performance – ability to comply with anticipated effluent TIN limit; assessment of effluent quality under different flow and temperature conditions; assessment of reliability and longevity.
- Complexity – assessment of system O&M complexity 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 (e.g., effluent TIN limit of 3 mg/L).

11.3.2 Denitrifying Sand Filters

As the existing sand filters at the facility are to be abandoned in favor of compressible media filters, they will be available for other uses. The four existing filter modules can be used for denitrification, however, to achieve treatment of a flow of 2.0 MGD, twelve additional filter modules are needed. A total of 16 filter modules provides four redundant filter modules and reduces the loading rate to 2.4 gpm/sf (with 12 filter modules in operation) as recommended by the manufacturer when operating the filters for denitrification. Supplemental carbon would be dosed into the filter influent from an outdoor storage tank located near the existing Chlorination Building. It is assumed that supplemental carbon dosing equipment, excluding the storage tank, would be housed inside the existing Chlorination Building in the rooms that previously housed the chlorination and dechlorination equipment. The existing reject pump station would be upgraded to accommodate the additional reject water from the additional filters.

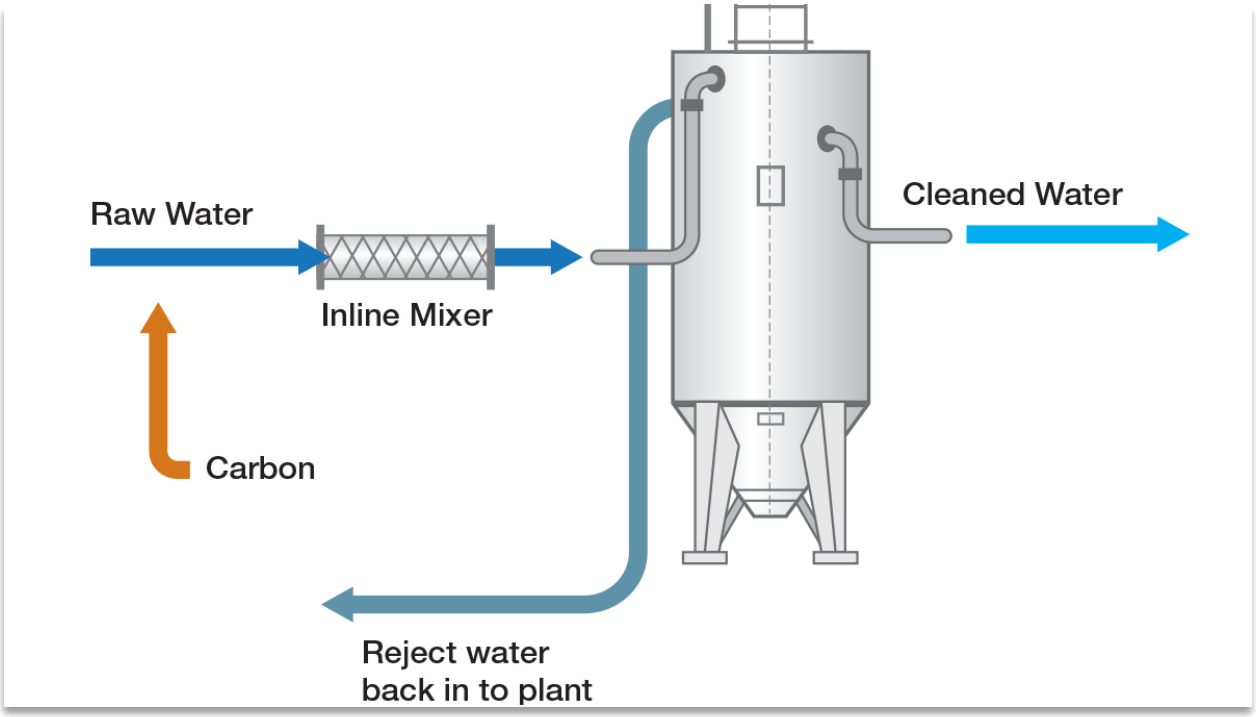


Figure 11-2 Denitrifying Sand Filter Schematic

The denitrifying filters were preliminarily sized to meet a target of less than 8 mg/L effluent TIN based on an average annual temperature of 15 °C. The lagoon effluent temperature can vary seasonally from 5 to 24 °C. Therefore, it is expected that denitrification will improve at the higher temperatures and degrade at the lower temperatures due to the impact of temperature on biological kinetics. However, it is unclear if the improvement in performance at warmer temperatures will adequately balance the degraded performance at lower temperatures. As a point of reference, to achieve denitrification to 8 mg/L TIN at 5 °C, a total of thirty-six additional filter modules would be needed instead of twelve additional modules.

Operationally, the denitrifying sand filters function the same as the existing sand filters. Compressed air is provided to airlift pumps for backwashing media. For this application, backwashing is used to control biomass wasting. Too frequent of backwashing can prevent the denitrifying organisms from growing, whereas too infrequent of backwashing can clog the filters with biomass growth. Lagoon effluent into the filters needs to be low in TSS, as the solids can interfere with the fixed film growing on the media. Therefore, this process would be utilized

downstream of the proposed compressed media filtration process and head produced by the filter feed pumps associated with the compressed media filtration process would be used to feed the denitrifying filters.

11.3.3 Microvi MNE Biocatalyst

Microvi utilizes a new technology consisting of stainless-steel reactors filled with an engineered biocatalyst. The biocatalyst uses a high density of selected, highly efficient microorganisms to achieve denitrification with virtually no sludge production. Each reactor uses a motorized mixer to mix nitrate rich influent, supplemental carbon, and the biocatalyst. The biocatalyst has a 5-year warranty; however, it is expected to require replacement every 8-10 years. The replacement cost represents a significant maintenance expense that must be factored into the O&M costs. Figure 11-3 shows a picture of one of Microvi's reactors filled with biocatalyst.

Figure 11-3 Microvi Biocatalyst

For this application, the Microvi process would utilize three trains of four reactors each for a total of 12 reactors. Each reactor would be 9.5 feet in diameter, 15 feet tall, and have a 3-HP mixer. The footprint of this arrangement is approximately 50-feet by 60-feet. All of the tanks would be installed on a common concrete slab foundation. As with the denitrifying filters alternative, supplemental carbon would be dosed from an outdoor storage tank located near the existing Chlorination Building that would house the supplemental carbon dosing equipment. Although discussions with the manufacturer indicated that it may be possible to repurpose the existing sand filters tanks for use as Microvi reactors, it is unclear if this arrangement would yield cost savings. Therefore, it was assumed all new reactors would be provided.

The Microvi process is tightly controlled via instrumentation that controls external carbon dose based on nitrate concentration and flow. Any TSS in the Microvi influent is expected to pass through the reactors, and the reactors themselves are not expected to produce any additional TSS. Therefore, the Microvi process would be utilized downstream of the proposed compressed media filtration process and head produced by the filter feed pumps associated with the compressed media filtration process would be used to feed the Microvi reactors.

11.3.4 Moving Bed Bioreactor

A MBBR application would consist of two common-wall trains, each with one anoxic zone and one aerobic zone. The overall footprint of the concrete MBBR basins would approximately 58 feet by 40 feet. Figure 11-4 illustrates a schematic of the MBBR process. Within the MBBR, the denitrifying biomass attach to the HDPE carrier media forming a biofilm. The carrier media is kept in the zones using retention sieves that allow treated wastewater and suspended solids to pass, but blocks passage of the media. This eliminates the need for recycle pumping to return biomass to the MBBR tanks.

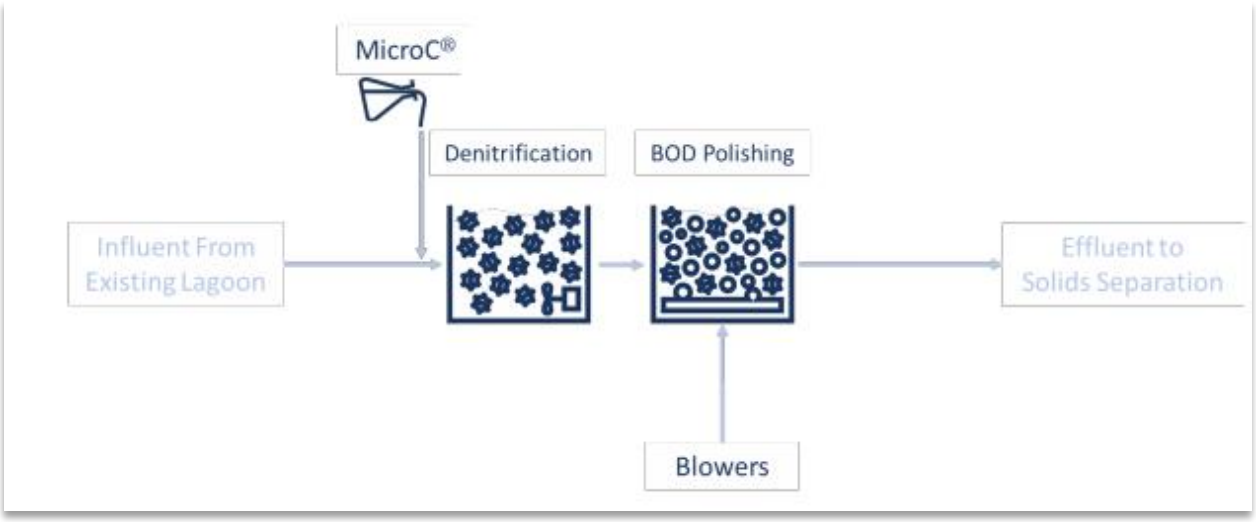


Figure 11-4 MBBR Schematic

Supplemental carbon (e.g., MicroC) is added to drive denitrification in the anoxic zones, which each would be mixed using a 7.5-HP mixer to keep the carrier media in suspension. The aerobic zone removes residual $CBOD_5$ that was not used in the denitrification reaction. These zones would be aerated using blowers and coarse bubble diffusers. A new building would be constructed to house the blowers, supplemental carbon dosing equipment, and controls.

Some TSS is expected to slough off the media resulting in an increase of about 30 mg/L compared to influent TSS. For this reason, it is recommended that the MBBR process be fed from the bypass outlet structure of Lagoon 3, treated in the MBBR, and then conveyed to Lagoon 4 for settling of solids before being filtered and disinfected. Based on preliminary sizing, the MBBR media fill is approximately 48%, close to the manufacturer's recommended maximum of 50%, indicating that additional treatment capacity more media could not be added to increase capacity. However, because effluent from the MBBR would be treated in Lagoon 4, which includes some aeration, it is probable that the aerobic zones of the MBBR could be eliminated or converted to additional anoxic volume. Eliminating the aerobic zones would reduce cost. However, converting them to additional anoxic volume would still reduce cost some by

eliminating the blowers and diffusers and eliminating or downsizing the new building, and would also allow for increased capacity for denitrification up to 2.8 MGD or increased level of treatment to produce nitrate concentrations less than 2 mg/L at 2.0 MGD, which is expected to allow the WWTP to meet effluent TIN of 3 mg/L.

11.3.5 Denitrification Probable Construction and O&M Costs

Opinions of probable construction cost and project cost (see Appendix H) were 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 11-5 include markups for equipment installation, electrical, instrumentation and control, mobilization, contingency, sales tax, design, and construction management. Contractor overhead and profit is included in the unit prices of the breakdown provided in Appendix H.

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-supplied information on parts cost and replacement interval, and labor estimates for operations from similar facilities at other treatment plants. Maintenance costs were assumed to be equal to 2% of the purchase price of each vendor equipment package. 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 11-5
Denitrification Alternatives Cost Summary[a]

Parameter	Units	WesTech SuperSand (Sand)	Microvi MNE (Biocatalyst)	Nexom BioPortz (MBBR)
Average Energy Use				
Yearly Energy Consumption	kWh/year	182,494	235,325	139,128
Energy Unit Cost	$/kWh	0.075	0.075	0.075
Yearly Energy Cost[b]	$/year	13,687	17,649	10,435
Labor				
Monthly Labor Requirements	hrs/month	60	36	34
Labor Unit Cost	$/hour	50	50	50
Yearly Labor Cost	$/year	36,000	21,600	20,400
Chemical				
Micro C Consumption	gals/year	66,409	66,409	66,409
Yearly Micro C Cost	$/year	232,432	232,432	232,432
Annual Media Lost	%/year	1	NA	NA
Yearly Media Replacement Cost	$/year	3,360	50,000	NA
Yearly Chemical Cost[b]	$/year	235,792	282,432	232,432

Table 11-5
Denitrification Alternatives Cost Summary[a]

Parameter	Units	WesTech SuperSand (Sand)	Microvi MNE (Biocatalyst)	Nexom BioPortz (MBBR)
Maintenance				
Yearly Maintenance Cost[c]	$/year	15,379	60,000	13,200
Total Yearly O&M Cost[d,e]	**$/year**	**301,000**	**382,000**	**276,000**
Construction Cost[f]				
Probable Construction Cost (2020)[f]	**$**	**2,812,000**	**7,763,000**	**3,873,000**
Probable Project Cost (2020)[f,i]	**$**	**3,520,000**	**9,700,000**	**4,840,000**
20-Year O&M Cost (2020)[f,g]	**$**	**4,478,000**	**5,683,000**	**4,106,000**
20-Year NPV[f,h]	**$**	**7,998,000**	**15,383,000**	**8,946,000**

Notes:
(a) All costs are in 2020 dollars unless specified.
(b) Yearly energy and chemical use based on operation at rated capacity of 2.0 MGD.
(c) 2% of quoted equipment price used for annual maintenance cost.
(d) Sum of energy, labor, chemical and maintenance 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.
(h) NPV – net present value. Sum of 2020 project cost and O&M costs.
(i) The project cost includes a 25% markup added to the construction cost to cover planning, design, and construction services during the project.

Based on preliminary sizing MBBR has the lowest O&M cost, but denitrifying sand filters have a lower probable construction cost and its net present value is about 10% lower than MBBR. Microvi has both the highest probable construction cost and O&M cost.

11.3.6 Comparison of Denitrification Alternatives

The advantages and disadvantages of each evaluated alternative are listed in Table 11-8. Comparative scoring is provided in Table 11-7. 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 11-6. The total score in Table 11-7 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 11-6
Filtration Alternative Advantages and Disadvantages

Parameter	WesTech SuperSand® (Sand)	Microvi MNE ™ (Biocatalyst)	Nexom BioPortz™ (MBBR)
Footprint	▪ Approximately 1,400 square feet. ▪ Fits into existing footprint the best out of all alternatives, but could require significantly more filters to maintain sufficiently low TIN during winter. ▪ No new building required.	▪ Approximately 3,150 square feet. ▪ Large reactor footprint. ▪ No new building required.	▪ Approximately 2,770 square feet. ▪ Large tank footprint. ▪ Requires new building to house blowers.
Process Integration	▪ Requires filtration upstream to ensure low influent TSS to process. ▪ Does not increase effluent TSS.	▪ No filtration of process effluent required. ▪ No solids produced, so does not increase effluent TSS.	▪ Some aeration needed for $CBOD_5$ polishing. ▪ Solids slough off media and increase effluent TSS requiring settling in lagoon and prior to filtration.
Performance	▪ Capable of meeting effluent nitrate of 5 mg/L at 15 °C. ▪ May require significantly more filters to maintain sufficiently low TIN during winter.	▪ Capable of consistently maintaining effluent nitrate of 5 mg/L or less.	▪ Capable of meeting effluent nitrate of 5 mg/L at 5 °C.
Complexity	▪ Backwash wasting required. ▪ City familiarity with sand filter operation.	▪ Mixers in each Microvi reactor. ▪ No wasting required.	▪ Mixers in each anoxic zone of MBBR and blowers for aerobic zones. ▪ No wasting required.
Expandability	▪ Additional sand filters must be added to increase denitrification capacity, which would require significant reconfiguration due to space constraints for more filters.	▪ Additional bioreactors can be added to increase denitrification capacity, though available space is limited for substantially more reactors.	▪ Converting aerobic zone to anoxic zone increases capacity from 2.0 to 2.8 MGD and allows for some additional media fill to increase level of treatment. ▪ Additional capacity beyond 2.8 MGD requires more concrete basins. Some space available.

Table 11-7
Denitrification Alternatives Comparison

Parameter	WesTech SuperSand (Sand)	Microvi MNE (Biocatalyst)	Nexom BioPortz (MBBR)	Weighting Factor
Capital Cost	+	-	0	x3
O&M Cost	0	-	+	x2
Footprint	0	-	0	x1
Process Integration	0	+	-	x1
Performance	-	+	+	x2
Complexity	0	+	0	x1
Expandability	-	0	+	x1
Total Score[(a)]	**0**	**-2**	**4**	

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

11.3.7 Denitrification Recommendation

Based on the evaluation and scoring of alternatives, denitrification using a MBBR is the apparent best alternative and is recommended for implementation. While this alternative does not have the lowest probable construction cost, it has the lowest estimated O&M cost and the life cycle cost is only about 10% higher than the lowest life cycle cost. It also provides good performance and offers significant advantages for expandability.

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 11-8 below.

Table 11-8
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 11-9 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

Opinions of probable construction cost and project cost (see Appendix H) were 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 11-9 include markups for equipment installation, mobilization, contractor overhead and profit, contingency, sales tax, design, and construction management.

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 11-9
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**

Table 11-9
Aeration Alternatrives Cost Summary[a]

Parameter	Units	Aerator Solutions	Aqua-Aerobics	RWL	Reliant Water
20-Year NPV[h]	**$**	**2,593,000**	**2,549,000**	**3,390,000**	**4,038,000**

Notes:
(a) All costs are in 2017 dollars. See Appendix H 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 11-10. Comparative scoring is provided in Table 11-11. 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 11-10. The total score in Table 11-11 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 11-10
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 11-11
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 the recent stormwater separation along Avenue F. 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 enhance facilities to improve 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 Plan (CIP)

The CIP is presented for two timeframes:

- 6-Year CIP from 2019 to 2024
- Long-Term CIP from 2025 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 2019. 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 2 years does not affect the proposed 6-year CIP projects since the population growth over two years does not significantly affect the model results for system deficiencies.

12.2.1 6-Year CIP (2019 – 2024)

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.

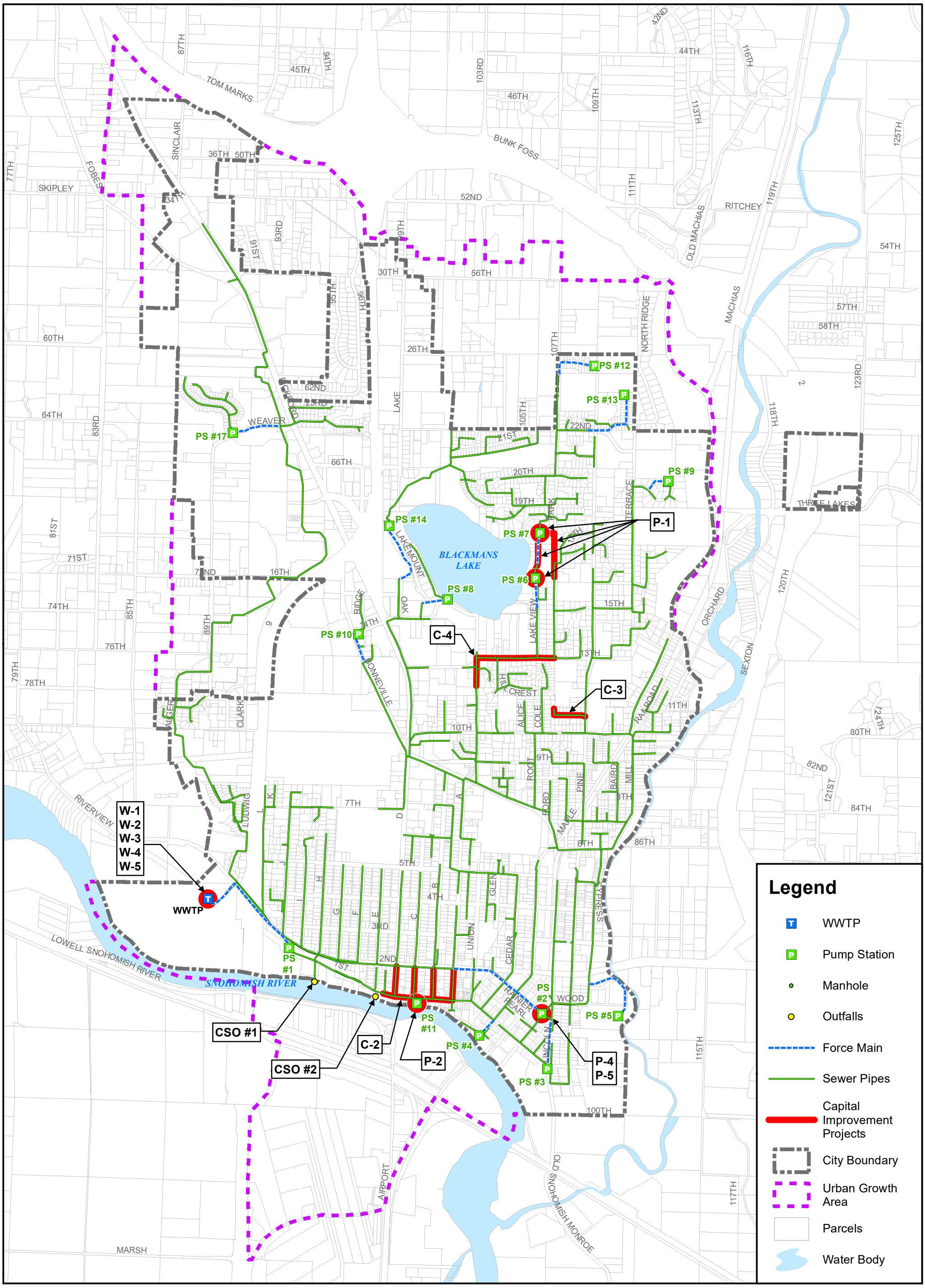
Table 12-1
6-Year Capital Improvement Plan (2019-2024)

CIP No.	Project	Type	Priority Rank	Replacement	Upgrade	Expansion	Project Description
C-1	General Conveyance System Repair and Replacements (2019-2024)	O&M / Obsolescence	1	☑	☐	☐	Annual allowance for general sewer conveyance repairs and replacements Projects typically accompany annual Road Preservation Projects
C-2	CSO 2 Separation Program	Capacity	3/4	☐	☑	☐	Annual allowance to install stormwater pipes and separate the CSS in the CSO Pump Station basin
C-3	Emerson Street Utility Improvement	Capacity	1	☑	☐	☐	Re-connect side sewers to non-failed sewer main in Emerson Street
C-4	Upsize Gravity Main at 13th Street and Avenue A	Capacity	4	☑	☐	☐	Upsize approximately 1,600 ft. of pipe from 13th Street and Park Ave to south of Ave A and Smithson Pl.
C-5	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	O&M / 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 (2019-2024)

CIP No.	Project	Type	Priority Rank	Replacement	Upgrade	Expansion	Project Description
W-1	Nitrogen Removal Planning/ Optimization	Capacity	2	☐	☐	☑	Conduct initial planning for implementation of nitrogen removal and examine optimization of the existing processes for nitrogen removal
W-2	Upgrade/Expand Effluent Filtration	O&M / Capacity	2/3	☐	☑	☑	Replace the existing effluent sand filters with compressible media filters to upgrade performance and increase capacity
W-3	General WWTP Improvements (2019-2024)	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) disinfection 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 Ecology - potential regulatory issues or needed immediately
2) On the verge of being out of regulation with Ecology - high probability
3) Medium risk - medium probability
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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City of Snohomish Washington

6-Year Capital Improvement Projects

General Sewer Plan and Wastewater Engineering Report
City of Snohomish, Washington
September 2020

Figure
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12.2.2 6-year CIP Project Descriptions

CIP C-1 – General Conveyance Systems Repairs and Replacements (2019 – 2024)

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. The amount budgeted each year is increased by 3.25% to keep up with inflation and rising construction costs.

CIP C-2 – 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-3 – Emerson Street Utility Improvement

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

CIP C-4 – 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-5 – 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 Ecology 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 – Nitrogen Removal Planning/Optimization

This project allocates funds to conduct initial planning for implementation of nitrogen removal, which will include examining optimization of the existing processes to improve existing nitrogen removal and refining the preliminary design criteria utilized in this Plan as it pertains to sizing, layout and integration of a new nitrogen removal process. It is expected such a planning effort will be required by Ecology within the first cycle of the pending nutrient general permit, as discussed in Chapter 4. This effort, or a subsequent engineering report, would also build upon the proposed re-rating discussed in this plan, as at that time it would be possible to consider holistically the capacity needs to meet current limits and pending nitrogen limits, which should be known at the time of this evaluation.

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 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. A pilot study of a Fuzzy Filter system was completed in 2018, the results of which are provided in Appendix M.

CIP W-3 – General WWTP Improvements (2019-2024)

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

As stated in Chapter 9, this project was recently completed in 2020. The project replaced the current chlorine gas disinfection system due to safety concerns and improvements required to enhance safety and comply with current codes with a peracetic acid disinfection system. This new system provides simpler operation and easier/safer chemical handling compared to the previous chlorine system and eliminates the need for dechlorination. The design and construction costs for this project are retained in the 6-year CIP as they occurred within the 6-year CIP period. Cost for planning is excluded as that work was completed in 2018.

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 (2025-2036)

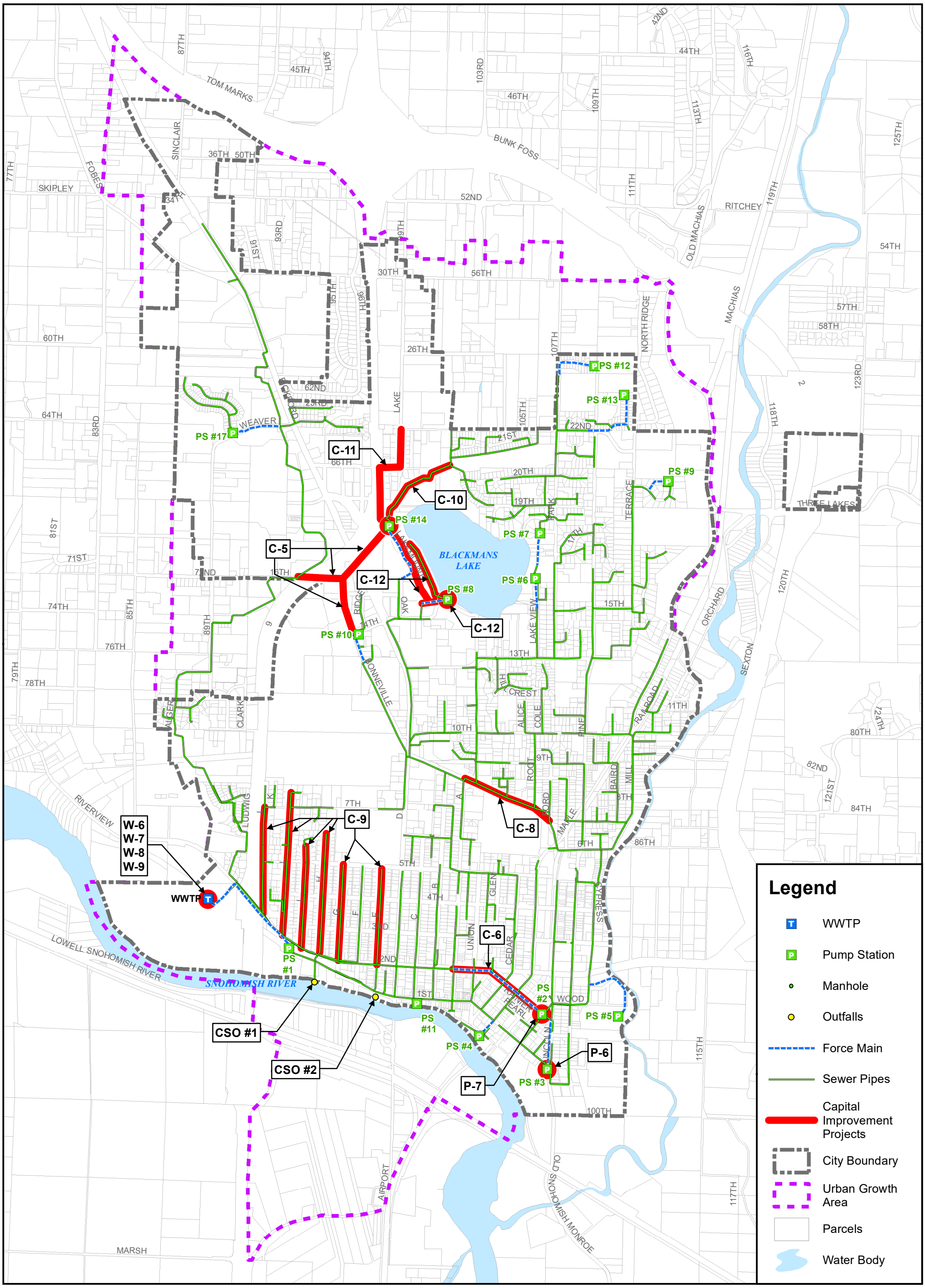
The projects recommended for the Long-Term CIP are described in Table 12-2 and illustrated on Figure 12-2. Improvements that would be completed by developers are expected to be privately funded and so are not listed in this section. Project priorities were developed with input from the City.

Table 12-2
Long-Term Capital Improvement Plan (2025-2036)

CIP No.	Project	Type	Replacement	Upgrade	Expansion	Project Description
C-5	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-6	Rainier Pump Station (PS 2) Force Main Replacement	Obsolescence	☑	☐	☐	Replace the existing 1,800 ft. AC force main with a PVC force main
C-7	General Conveyance System Repair and Replacements (2025-2036)	O&M / Obsolescence	☑	☐	☐	Annual allowance for general sewer conveyance repairs and replacements
						Projects typically accompany annual Road Preservation Projects
C-8	Interurban Trail Conveyance System Upgrade	Obsolescence	☑	☐	☑	Replace and upsize 1,600 ft. of pipe along the Interurban Trail
C-9	CSO 1 Separation Program	Capacity	☐	☑	☐	Annual allowance to install stormwater pipes and separate the CSS in the Gravity CSO basin
C-10	CIPP Pipe Restoration North of Blackmans Lake	Obsolescence	☐	☑	☐	Line approximately 1,700 ft. of pipe north of Blackmans Lake with CIPP
C-11	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-12	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	O&M / Obsolescence	☑	☑	☐	Replace and expand existing wet well

Table 12-2
Long-Term Capital Improvement Plan (2025-2036)

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	Nitrogen Removal Improvements	O&M / Capacity	☐	☑	☑	Optimize and add process for denitrification
W-9	General WWTP Improvements (2025-2036)	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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Long-Term Capital Improvement Projects

General Sewer Plan and
Wastewater Engineering Report
City of Snohomish, Washington
September 2020

Figure

12-2

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12.2.4 Long-Term CIP Project Descriptions

CIP C-5 – 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-6 – 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-7 – General Conveyance System Repair and Replacements (2025-2036)

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. The amount budgeted each year is increased by 3.25% to keep up with inflation and rising construction costs.

CIP C-8 – 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-9 – 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-10 – 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-11 – 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-12 – 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 – Nitrogen Removal Improvements

An amount of $4.84 million is budgeted to implement improvements at the WWTP for nitrogen removal based on the recommendation in Chapter 11 of adding a MBBR process for denitrification. The improvements may also involve some modifications to optimize existing processes with respect to nitrogen removal.

CIP W-9 – General WWTP Improvements (2025-2036)

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 I. 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 (2019-2024) CIP Opinion of Probable Project Costs[a]

CIP No.	Project	Opinion of Probable Project Cost						Total Project Cost
		2019	2020	2021	2022	2023	2024	
C-1	General Conveyance System Repair and Replacements (2019-2024)	$350,000	$361,000	$373,000	$385,000	$398,000	$411,000	$2,278,000
C-2	CSO 2 Separation Program[b]	$0	$0	$0	$0	$265,000	$2,050,000	$2,315,000
C-3	Emerson Street Utility Improvement	$100,000	$0	$0	$0	$0	$0	$100,000
C-4	Upsize Gravity Main at 13th Street and Avenue A	$0	$0	$0	$0	$0	$1,050,000	$1,050,000
C-5	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	Nitrogen Removal Planning/ Optimization	$0	$100,000	$0	$0	$0	$0	$100,000
W-2	Upgrade/Expand Effluent Filtration	$0	$250,000	$2,070,000	$0	$0	$0	$2,320,000
W-3	General WWTP Improvements (2019-2024)	$350,000	$361,000	$373,000	$385,000	$398,000	$411,000	$2,278,000
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		**$2,366,000**	**$2,362,000**	**$2,976,000**	**$2,450,000**	**$1,061,000**	**$4,041,000**	**$15,256,000**

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 (2025-2036) CIP Opinion Of Probable Project Costs[a]

CIP No.	Project	Opinion of Probable Project Cost
C-5	Western Gravity Flow Diversion	$2,981,000
C-6	Rainier Pump Station (PS 2) Force Main Replacement	$760,000
C-7	General Conveyance System Repair and Replacements (2025-2036)	$6,109,000
C-8	Interurban Trail Conveyance System Upgrade	$1,019,000
C-9	CSO 1 Separation Program[b]	$4,500,000
C-10	CIPP Pipe Restoration North of Blackmans Lake	$451,000
C-11	Pipe Extension North of the Casino Pump Station (PS 14)	$1,479,000
C-12	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	Nitrogen Removal Improvements	$4,840,000
W-9	General WWTP Improvements (2025-2036)	$6,109,000
Total Opinion of Probable Project Cost		**$35,246,000**

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

This chapter was prepared by FCS GROUP to determine the total cost of providing sewer service to the City of Snohomish customers. The purpose of the financial plan is to demonstrate the financial viability of the sewer utility to meet the system needs outlined in this Plan. This analysis considers historical performance, the sufficiency of utility revenues to meet current and future O&M needs, policy obligations, and the impact of executing the CIP. Furthermore, this chapter provides a review of the utility's current rate structure with respect to rate adequacy and customer affordability. The following financial plan demonstrates the ability of the sewer utility to remain financially secure through the end of the planning period with sufficient funds to construct, operate, maintain and manage the system on a continuing basis, in full compliance with federal, state and local requirements.

13.2 Past Financial Performance

This section includes a historical summary of financial performance as reported by the City on the fund resources and uses arising from cash transactions specific to the sewer utility.

13.2.1 Comparative Financial Statements

The City's enterprise funds operate on a cash basis; therefore, annual balance sheets and income statements are not prepared. Table 13-1 summarizes the available level of financial detail for the previous 6 years (2012 through 2017).

13.2.2 Findings and Trends

The following noteworthy findings and trends are identified to demonstrate the historical performance and condition of the utility:

- Despite several years of rate decreases, Charges for Goods and Services have increased annually from 2012-2016, before dropping fairly significantly in 2017. Since the reported revenue also includes connection charge revenue, the decrease in revenues is likely indicative of stagnation in growth and a reduction in new construction in 2017.
- 2012 saw a large inflow of intergovernmental revenues and non-revenues. The increase in intergovernmental revenues was related to Department of Ecology and FEMA grants to fund the capital program while the increase in non-revenues represents an interfund loan and Public Works Trust Fund proceeds, also to fund the capital program.
- With the exception of 2012, operating expenditures have been fairly stable, fluctuating between $1.7 and $1.8 million.
- The operations and maintenance (O&M) coverage ratio (service revenues divided by operating expenses) ranged from a low of 1.22 in 2012 to a high of 1.87 in 2016 before ending 2017 at 1.66. A ratio above 100 provides capacity for capital improvements without drawing down cash reserves.
- In each year of the historical review, cash and investments have increased; starting at a low of $5.0 million in 2012 and increasing to $8.8 million by 2017.

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Table 13-1
Summary of Historical Fund Resources and Uses Arising from Cash Transactions

Sewer Fund 402	2012	2013	2014	2015	2016	2017
BEGINNING CASH AND CASH INVESTMENTS	**$ 4,008,109**	**$ 5,387,821**	**$ 5,900,832**	**$ 6,147,650**	**$ 6,739,557**	**$ 7,639,092**
OPERATING REVENUES						
Taxes	$ -	$ -	$ -	$ -	$ -	$ -
Licenses & Permits	1,350	1,100	2,400	4,350	3,450	1,700
Intergovernmental Revenues	5,180,176	169,569	-	-	-	-
Charges for Goods and Services	4,244,309	4,556,094	4,607,443	4,852,595	5,397,361	4,145,519
Fines & Penalties	-	-	-	-	-	-
Miscellaneous Revenues	2,327	11,679	8,003	16,739	21,428	292,887
Total Operating Revenues	**$ 9,428,162**	**$ 4,738,442**	**$ 4,617,846**	**$ 4,873,684**	**$ 5,422,240**	**$ 4,440,106**
OPERATING EXPENDITURES						
General Government	$ -	$ -	$ -	$ -	$ -	$ -
Public Safety	-	-	-	-	-	-
Utilities	2,661,588	1,738,720	1,818,842	1,841,800	1,850,685	1,750,412
Transportation	-	-	-	-	-	-
Natural and Economic Environment	-	-	-	-	-	-
Social Services	-	-	-	-	-	-
Culture and Recreation	-	-	-	-	-	-
Intergovernmental Payments	-	-	-	-	-	-
Total Operating Expenditures	**$ 2,661,588**	**$ 1,738,720**	**$ 1,818,842**	**$ 1,841,800**	**$ 1,850,685**	**$ 1,750,412**
Net Operating Increase (Decrease):	**$ 6,766,574**	**$ 2,999,722**	**$ 2,799,004**	**$ 3,031,884**	**$ 3,571,554**	**$ 2,689,694**
NONOPERATING REVENUES						
Other Financing Sources	$ 300,000	$ 1,651,694	$ 339,073	$ 633,420	$ -	$ -
Debt Proceeds	-	-	-	-	-	-
Transfers-In	-	518,791	1,500,000	20,000	-	-
Total Nonoperating Revenues		**$ 2,170,485**	**$ 1,839,073**	**$ 653,420**	**$ -**	**$ -**
NONOPERATING EXPENDITURES						
Other Financing Uses	$ 823,900	$ 1,288,500	$ 1,574	$ -	$ -	$ -
Debt Service	830,143	1,379,905	1,370,146	2,109,768	1,028,618	749,068
Capital Expenditures	5,699,934	1,483,790	1,285,427	983,628	970,490	732,776
Transfers - Out	-	505,000	1,735,500	-	650,000	36,000
Total Nonoperating Expenses	**$ 7,353,977**	**$ 4,657,195**	**$ 4,392,647**	**$ 3,093,396**	**$ 2,649,108**	**$ 1,517,844**
Increase (Decrease) in Cash and Investments	**$ (587,403)**	**$ 513,012**	**$ 245,431**	**$ 591,908**	**$ 922,446**	**$ 1,171,850**
Nonrevenues	$ 1,614,015	$ -	$ -	$ -	$ -	$ -
Nonexpenditures	16,000	-	-	-	-	-
ENDING CASH AND INVESTMENTS	**$ 5,018,721**	**$ 5,900,832**	**$ 6,147,650**	**$ 6,739,557**	**$ 7,662,004**	**$ 8,810,942**

Notes:
1) Beginning cash balances include prior period adjustments

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13.3 Current Financial Structure

This section summarizes the current financial structure used as the baseline for the capital financing strategy and financial forecast developed for this Plan.

13.3.1 Financial Plan

The City's sewer utility is responsible for funding all of its related costs through user fees. It does not depend on general tax revenues or general fund resources. The primary source of funding for the sewer utility is derived from ongoing charges for service, with additional revenue coming from sewer permit and inspection fees and interest income. The City controls the level of user charges by ordinance, and subject to statutory authority, can adjust user charges as needed to meet financial objectives.

The financial plan can only provide a qualified assurance of financial feasibility if it considers the total system costs of providing sewer service, both operating and capital. To meet these objectives, the following elements have been completed.

1. **Capital Funding Plan**: Identifies the total CIP obligations of the planning period. The plan defines a strategy for funding the CIP, including an analysis of available resources from rate revenues, existing reserves, connection charges, debt financing and any special resources that may be readily available (e.g., grants, developer contribution, etc.). The capital funding plan impacts the financial plan through the use of debt financing (resulting in annual debt service) and the assumed rate revenue available for capital funding.
2. **Financial Forecast**: The financial plan ultimately evaluates the sufficiency of utility revenues in meeting all obligations, including cash uses such as operating expenses, debt service, capital outlays and reserve conditions, as well as any coverage requirements associated with long-term debt. Included in the financial plan is a reserve analysis that forecasts cash flow and fund balance activity along with testing for satisfaction of actual or recommended minimum fund balance policies. The plan identifies the future adjustments required to fully fund all utility obligations in the projection period.

13.3.2 Capital Funding Plan

The capital improvement plan developed for this GSP identifies $23.0 million in project costs over the 10-year period and $49.7 million for the remainder of the planning period, based on costs presented in Chapter 12 that have been escalated from 2018 dollars to future dollars.

This capital improvement plan consists of the CSO 2 separation program, Champagne pump station upgrade and new force main, Rainier pump station generator and upgrades, upgrades to expand effluent filtration, gravity flow diversion to the west trunk and various renewal and replacement projects.

A summary of the initial 10-year and total CIP is shown in Table 13-2. As shown, each year has varied capital cost obligations depending on construction schedules and infrastructure planning needs. Approximately 31.6 percent of the capital costs are included in the 10-year planning period. Table 13-3 provides the detail for the 10-year CIP. Costs in both Table 13-2 and Table 13-3 are escalated from 2018 dollars to the future dollar amount assuming an effective escalation rate of 3.25 percent.

Table 13-2
10-Year and Total CIP

Year	Escalated $
2019	$ 2,437,665
2020	2,512,634
2021	3,268,679
2022	2,778,405
2023	1,242,324
2024	4,885,381
2025	1,270,921
2026	1,312,226
2027	1,354,873
2028	1,934,746
10-Year Total	**$ 22,997,853**
2029-2036	49,712,127
Total CIP	**$ 72,709,980**

Table 13-3
10 Year CIP (Escalated $)

Project	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	Total
CSO 2 Separation Program					$ 310,288	$ 2,478,355					**$ 2,788,643**
General Conveyance System Repair and Replacements	360,601	384,022	409,683	436,606	466,018	496,880	635,460	656,113	677,437	699,453	**5,222,274**
Upsize Gravity Main at 13th Street and Avenue A						1,269,401					**1,269,401**
Gravity Flow Diversion to the West Trunk						143,865					**143,865**
Champagne Pump Station (PS 7) Upgrade and New Force Main	175,149	1,372,268									**1,547,418**
Rainier Pump Station (PS 2) Generator (100kW)			76,884	578,362							**655,246**
Rainier Pump Station (PS 2) Upgrades			98,851	759,809							**858,660**
Nitrogen Removal Planning/Optimization		106,377									**106,377**
Upgrade/Expand Effluent Filtration		265,943	2,273,577								**2,539,521**
General WWTP Improvements	360,601	384,022	409,683	436,606	466,018	496,880	635,460	656,113	677,437	699,453	**5,222,274**
Replace the Chlorine Gas Disinfection System	1,329,073										**1,329,073**
Lincoln Pump Station (PS 3) Replacement										535,839	**535,839**
Emerson Street Utility Improvement	103,029										**103,029**
Kla-Ha-Ya Pump Station Decommissioning	43,272										**43,272**
Mobile 80 kW Generator	65,939										**65,939**
Add Non-Potable Water Air Gaps				567,021							**567,021**
Total	**$ 2,437,665**	**$ 2,512,634**	**$ 3,268,679**	**$ 2,778,405**	**$ 1,242,324**	**$ 4,885,381**	**$ 1,270,921**	**$ 1,312,226**	**$ 1,354,873**	**$ 1,934,746**	**$ 22,997,853**

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13.3.3 Capital Financing Strategy

An ideal capital funding strategy would include the use of grants and low-cost loans when debt issuance is required. However, these resources are very limited and competitive in nature and do not provide a reliable source of funding for planning purposes. It is recommended that the City pursue these funding avenues but assume bond financing to meet the needs of the City when available cash resources are insufficient. The capital financing strategy developed to fund the capital improvement plan identified in this Plan assumes the following funding resources:

- Accumulated capital cash reserves;
- Annual revenue collections from connection charges;
- Annual transfers of excess cash (over minimum balance targets) from the Operating Fund, if any;
- Interest earnings on Capital Fund balances; and
- New revenue bond proceeds.

Based on the information provided by the City, the sewer utility began 2019 with $9.2 million in total funds. The cash resources described above are anticipated to fund 100 percent of the 10-year CIP and 68.0 percent of the total CIP. Three new debt issuances (2031, 2033 and 2035), totaling $23.25 million in additional proceeds are anticipated in the planning period. Table 13-4 presents the corresponding capital financing strategy.

Table 13-4
Capital Funding Strategy

Year	Capital Expenditures Escalated	Revenue Bond Financing	Cash Funding	Total Financial Resources
2019	**$ 2,437,665**	$ -	$ 2,437,665	**$ 2,437,665**
2020	**2,512,634**	-	2,512,634	**2,512,634**
2021	**3,268,679**	-	3,268,679	**3,268,679**
2022	**2,778,405**	-	2,778,405	**2,778,405**
2023	**1,242,324**	-	1,242,324	**1,242,324**
2024	**4,885,381**	-	4,885,381	**4,885,381**
2025	**1,270,921**	-	1,270,921	**1,270,921**
2026	**1,312,226**	-	1,312,226	**1,312,226**
2027	**1,354,873**	-	1,354,873	**1,354,873**
2028	**1,934,746**	-	1,934,746	**1,934,746**
Subtotal	**$ 22,997,853**	**$ -**	**$ 22,997,853**	**$ 22,997,853**
2029-2036	**49,712,127**	23,250,000	26,462,127	**49,712,127**
Total	**$ 72,709,980**	**$ 23,250,000**	**$ 49,459,980**	**$ 72,709,980**

13.4 Available Funding Assistance and Financing Resources

Feasible long-term capital funding strategies must be defined to ensure that adequate resources are available to fund the CIP identified in this Plan. In addition to the utility's resources, such as accumulated cash reserves, capital revenues, and rate revenues designated for capital purposes, capital needs can be met from outside sources such as grants, low-interest loans and bond financing. The following is a summary of the City's sewer utility resources and outside resources.

13.4.1 Utility Resources

Sewer utility resources appropriate for funding capital needs include accumulated cash in the capital reserve, rate revenue designated for capital spending purposes, and capital-related charges such as connection charge revenue. The first two resources will be discussed in the **Financial Policies** section. Capital-related charges are discussed below.

Connection Charges

A connection charge, such as the utility connection fee and capital facility charge, refer to a one-time charge imposed on new customers as a condition of connecting to the sewer system. The purpose of the connection charge is two-fold: 1) to promote equity between new and existing customers; and 2) to provide a source of revenue to fund capital projects. Revenue can only be used to fund utility capital projects or to pay debt service incurred to finance those projects. In 2019, the City charged all new customers a utility connection fee of $6,340 per meter capacity equivalent (MCE) and a capital facility charge of $2,975 per MCE for a total connection charge of $9,315 per MCE. In addition to the utility connection fee and capital facility charge, customers connecting to an area of the City served by Cemetery Creek Trunk Line will have to pay an additional charge of $8,288 per MCE.

Local Facilities Funds

While a connection charge is the manner in which new customers pay their share of general facilities costs, local facilities funds are used to pay the costs of local facilities that connect each property to the system's infrastructure. Local facilities funding is often overlooked in a rate forecast because it is funded upfront by either connecting customers, developers, or through an assessment to properties, but never from rates. Although these funding mechanisms do not provide a capital revenue source toward funding CIP costs in this analysis, the discussion of these charges is included in this chapter, as they represent a potential funding source for future projects. None of the mechanisms described below are anticipated as part of the current financial plan, but may be considered in the future.

A number of mechanisms can be considered for funding local facilities. One of the following scenarios typically occurs: (a) the utility charges a connection fee based on the cost of the local facilities under the same authority as the connection charge; (b) a developer funds extension of the system to its development and turns those facilities over to the utility (contributed capital); or (c) a local assessment is set up called a Utility Local Improvement District, which collects tax revenue from benefited properties.

A local facilities charge is a variation of the connection charge authorized through RCW 35.92.025. It is a City-imposed charge to recover the cost related to service extension to local properties. Often called a front-footage charge and imposed on the basis of footage of main "fronting" a particular property, it is usually implemented as a reimbursement mechanism to a city for the cost of a local facility that directly serves a property. It is a form of connection charge and thus can accumulate up to 10 years of interest. It typically applies to instances when no developer-installed facilities are needed through developer extension due to the prior existence of available mains already serving the developing property.

The developer extension is a requirement that a developer install on-site and sometimes off-site improvements as a condition of extending service. These are in addition to the connection charge required and must be built to City standards, similar to local facility connections. Part of the agreements between the City and the developer for the developer to extend service might include a latecomer agreement, resulting in a latecomer charge to new connections to the developer extension.

Latecomer charges are a variation of developer extensions whereby new customers connecting to a developer-installed improvement make a payment to the City based on their share of the developer's cost. The City passes this charge on to the developer who installed the facilities. This is part of the developer extension process, and defines the allocation of costs and records latecomer obligations on the title of affected properties. No interest is allowed, and the reimbursement agreement cannot exceed 20 years in duration.

Utility Local Improvement District formation is another mechanism for funding infrastructure that assesses benefited properties based on the special benefit received by the construction of specific facilities. Most often used for local facilities, some Utility Local Improvement Districts also recover related general facilities costs. Substantial legal and procedural requirements can make this a relatively expensive process and there are mechanisms by which a Utility Local Improvement District can be rejected.

13.4.2 Outside Resources

This section outlines various grant and loan opportunities available to the City through federal and state agencies and bond financing options to fund the CIP identified in this Plan. The short-term capital program identified in this plan does not depend on any of these funding sources, but they may be pursued in the future to help fund the long-term capital program.

Grants and Low Cost Loans

Historically, federal and state grant programs were available to local utilities for capital funding assistance. However, these assistance programs have been mostly eliminated, substantially reduced in scope and amount or replaced by loan programs. Remaining miscellaneous grant programs are generally lightly funded and heavily subscribed. Nonetheless, even the benefit of low-interest loans makes the effort of applying worthwhile. Grants and low-cost loans for Washington State utilities are available from the Washington State Department of Ecology (Ecology) and the Department of Commerce. Each department offers programs for which the City might be eligible. They are primarily targeted as sewer programs or low-income and/or rural communities.

Department of Ecology

Ecology's Water Quality Program administers funding programs that provide low-interest loans, grants or loan and grant combinations for projects that protect, preserve and enhance water quality in Washington State. Low-interest loans and forgivable principal loan funding are available through the Clean Water State Revolving Fund. Grants are available for financially distressed communities through the Centennial Clean Water Program.

Department of Commerce

The Department of Commerce has three grant and loan programs for which the City might eligible. These programs are:

- The Community Development Block Grants General Purpose Grant;
- The Community Economic Revitalization Board (CERB) Grant and Loan Program; and
- The Public Works Trust Fund (PWTF) Loan Program.

Community Development Block Grants General Purpose Grants

These grants are made available to small cities, towns and counties in Washington State carrying out significant community and economic development projects that principally benefit low and moderate income persons.

- Eligible applicants are Washington State cities and towns with population less than 50,000 and counties with populations less than 200,000 that are non-entitlement jurisdictions or are not participants in Housing and Urban Development (HUD) Urban County Entitlement Consortium.
- Eligible projects include public facilities of water, wastewater, and storm sewer and streets.

Community Economic Revitalization Board

CERB primarily offers low-cost loans. Grants are made available only to the extent that a loan is not reasonably possible. The CERB targets public facilities funding for economically disadvantaged communities, specifically targeting job creation and retention. Priority criteria include the unemployment rates, number of jobs created and/or retained, wage rates, projected private investment and estimated state and local revenues generated by the project. Traditional construction projects are offered at a maximum dollar limit per project of $2.0 million. A local match of 50 percent is targeted.

- Eligible applicants include cities, towns, port districts, special purpose districts, federally recognized Indian tribes, and municipal corporations.
- Public facilities, bridges, roads, domestic and industrial water, earth stabilization, sanitary sewer, storm sewer, railroad, telecommunications, electricity, transportation, natural gas, buildings or structures and port facilities are all eligible.

Terms do not exceed 20 years, including available payment deferral of interest and principal for up to 5 years. Interest rates vary between 1.00-3.00 percent based on debt service coverage ratio, Distressed County and length of loan term.

Public Works Trust Fund

Cities, counties, special purpose districts, public utility districts, and quasi-municipal governments are eligible to receive loans from the PWTF. Eligible projects include repair, replacement, and construction of infrastructure for domestic water, wastewater, stormwater, solid waste, road, and bridge projects that improve public health and safety, respond to environmental issues, promote economic development, or upgrade system performance.

PWTF loans are available at interest rates ranging from 0.40 percent to 1.58 percent depending on the repayment term, with reduced interest rates available for all projects located in communities that have been declared a natural disaster. The standard loan offer is 1.58 percent interest repaid over a 20-year term. All loan terms are subject to negotiation and Board approval. Currently, no local match is required, and the maximum loan amount is $10M per jurisdiction per biennium.

The next funding cycle for the PWTF loan process begins in the summer of 2020.

Information regarding the application process, as well as rates and terms, are posted on the PWTF website in early spring. Further detail is available at http://www.pwb.wa.gov.

Bond Financing

General Obligation Bonds

General obligation bonds are bonds secured by the full faith and credit of the issuing agency, committing all available tax and revenue resources to debt repayments. With this high level of commitment, general obligation bonds have relatively low interest rates and few financial restrictions. However, the authority to issue general obligation bonds is restricted in terms of the amount and use of the funds, as prescribed in RCW 39.36.020.

While bonding capacity can limit availability of general obligation bonds for utility purposes, these can sometimes play a valuable role in project financing. A rate savings may be realized through two avenues: the lower interest rate and related bond costs, and the extension of repayment obligation to all tax-paying properties (not just developed properties) through the authorization of an ad valorem property tax levy.

Revenue Bonds

Revenue bonds are commonly used to fund utility capital improvements. The debt is secured by the revenue of the issuing utility and the debt obligation does not extend to the City's other revenue sources. With this limited commitment, revenue bonds typically bear higher interest rates than general obligation bonds and also require security conditions related to the maintenance of dedicated reserves (a bond reserve) and financial performance (added bond debt service coverage). The City agrees to satisfy these requirements by ordinance as a condition of bond sale.

Revenue bonds can be issued in Washington State without a public vote. There is no bonding limit, except perhaps the practical limit of the utility's ability to generate sufficient revenue to repay the debt and provide coverage. In some cases, poor credit or lack of reserves might make issuing bonds problematic.

13.5 Financial Forecast

The financial forecast, or revenue requirement analysis, forecasts the amount of annual revenue necessary from sewer rates. The analysis incorporates projected operating revenues, O&M expenses, debt service payments, rate-funded capital needs and any other identified revenues or expenses related to operations. The objective of the financial forecast is to evaluate the sufficiency of the current level of rates and determine if changes to those rates are necessary.

The analysis determines the amount of revenue needed in a given year to meet the year's expected financial obligations. For this analysis, two revenue sufficiency tests have been developed to reflect the financial goals and constraints of the City: Cash needs must be met and debt coverage requirements must be realized. In order to operate successfully with respect to these goals, both tests of revenue sufficiency must be met.

- **Cash Test** - The cash flow test identifies all known cash requirements for the City in each year of the planning period. Cash requirements typically include O&M expenses, debt service payments, depreciation funding or directly funded capital outlays and any additions to specified reserve balances. The total annual cash needs of the utility are then compared to projected cash revenues using the current rate structure. Any projected revenue shortfalls are identified, and rate increases necessary to make up the short fall are established.

- **Coverage Test** - The coverage test is based on commitments made by the City when issuing revenue bonds and some other forms of long-term debt. As a security condition of issuance, the City would be required per covenant to agree that the revenue bond debt would have a higher priority for payment (a senior lien) compared to most other utility expenditures; the only outlays with a higher lien are O&M expenses. Debt service coverage is expressed as a multiplier of the annual revenue bond debt service payment. For example, a 1.00 coverage factor would imply that no additional cushion is required. A 1.25 coverage factor means revenue must be sufficient to pay O&M expenses, annual revenue bond debt service payments, plus an additional 25 percent of annual revenue bond debt service payments. The excess cash flow derived from the added coverage, if any, can be used for any utility purpose, including funding capital projects. Targeting a higher coverage factor can help the City achieve a better credit rating and provide lower interest rates for future debt issues.

In determining the annual revenue requirement, both the cash and coverage sufficiency test must be met and then the test with the greatest deficiency drives the level of needed rate increase in any given year.

13.5.1 Current Financial Structure

The City maintains a fund structure and implements financial policies that target management of a financially viable and fiscally responsible enterprise fund utility.

Fiscal Policies

In developing the financial plan underlying the recommended rate structure, the City has assumed the attainment of certain recommended fiscal policies. The purpose of establishing fiscal policies for the City's sewer utility is to promote the financial integrity and stability of the utility and help ensure the sustainability of essential utility services. A brief summary of the key financial policies employed by the City, as well as those recommended and incorporated in the financial program are discussed below.

Operating Fund - Operating reserves are designed to provide a liquidity cushion to ensure that adequate cash will be maintained to deal with significant cash balance fluctuations, such as seasonal fluctuations in billings and receipts, unanticipated cash expenses, or lower than expected revenue collections. Like other types of reserves, operating reserves also serve another purpose: they help smooth rate increases over time. Target funding levels for an operating reserve are generally expressed as a certain number of days of O&M expenses, with the minimum requirement varying with the expected revenue volatility. Industry practice for utility operating reserves ranges from 30 days to 120 days of O&M expenses, with the lower end more appropriate for utilities with stable revenue streams and the higher end more appropriate for utilities with significant seasonal or consumption-based fluctuations. The analysis assumes the City is to maintain a minimum balance in the Operating Fund equal to 90 days of O&M, ranging from $527,000 in 2019 increasing to $696,000 by 2028 as shown in Table 13-6.

Capital Fund - A capital contingency reserve is an amount of cash set aside in case of an emergency should a piece of equipment or a portion of the utility's infrastructure fail unexpectedly. The reserve also could be used for other unanticipated capital needs including capital project cost overruns. There are various industry practices in use such as maintaining a balance equal to 1.00 to 2.00 percent of fixed assets, an amount equal to a 5-year rolling average of Capital Improvement Plan costs or an amount sufficient to fund an equipment failure (other than catastrophic failure). The final target level should balance industry standards with the risk and comfort level of the City. The City currently aims to maintain a capital contingency

target of $1.5 million from 2019-2023, before the target increases to $2.0 million from 2024-2029 and finally to $2.5 million for the remainder of the forecast. This value was determined to be sufficient in event of an equipment failure.

System Reinvestment - System reinvestment funding promotes system integrity through reinvestment in the system. Target system reinvestment funding levels are commonly linked to annual depreciation expense as a measure of the decline in asset value associated with routine use of the system. Particularly for utilities that do not already have an explicit system reinvestment policy in place, implementing a funding level based on full depreciation expense could significantly impact rates. A common alternative benchmark is annual depreciation expense net of debt principal payments on outstanding debt. This approach recognizes that customers are still paying for certain assets through the debt component of their rate and intends to avoid simultaneously charging customers for an asset and its future replacement. The specific benchmark used to set system reinvestment funding targets is a matter of policy that must balance various objectives, including managing rate impacts, keeping long-term costs down, and promoting "generational equity" (i.e., not excessively burdening current customers with paying for facilities that will serve a larger group of customers in the future).

The utility is not currently funding a dedicated budget account line item for system reinvestment, nor has a specified funding amount been included for system reinvestment in this analysis. Rather, the City directs the remaining revenues after the O&M and debt service expenses have been satisfied to first fund the operating fund target and then capital needs. As a result, rate revenues do contribute to the funding of capital projects, but the level of funding is not consistent from year to year. Capital funding from rates varies from $571,000 to $2.2 million depending on the year. It is recommended that the City consider a dedicated rate funding system reinvestment strategy in the future.

Debt Management - It is prudent to consider policies related to debt management as part of a broader utility financial policy structure. Debt management policies should be evaluated and formalized, including the level of acceptable outstanding debt, debt repayment, bond coverage and total debt coverage targets. The City does not currently have any outstanding sewer revenue bonds. For any future revenue bond assumptions, coverage is tested at a factor of 1.50.

Financial Forecast

The financial forecast is developed from the 2019/2020 biennial budget documents along with other key factors and assumptions to develop a complete picture of the sewer utility annual financial obligations. The following is a list of the key revenue and expense factors and assumptions used to develop the financial forecast.

- **Revenue** - The City has two general revenue sources: 1) sewer service charges (rate revenue); and 2) miscellaneous (non-rate) revenue. In the event of a forecasted annual shortfall, rate revenue can be increased to meet the annual revenue requirement. For the purpose of this financial forecast, non-rate revenues are forecast to increase with customer growth or not escalate depending on the nature of the revenue.
- **Growth** - Rate revenue is escalated at 1.30 percent based on the growth rate within the city limits as provided in **Chapter 3** of this GSP.

- **Connection Charge and Trunk Line Charge Revenue** - The existing connection and trunk line charges are applied to the projected new connections to forecast revenue. Based on the growth assumptions described above, the connection charge will generate an average of $740,000 annually from 2019-2028. This equates to an average of 52 new connections per year. Connection charge and trunk line charge revenue is directed towards annual capital needs.
- **Expenses** - O&M expense projections are based on the 2019/2020 biennial budget and forecasted to increase with general cost inflation of 3.00 percent, labor cost inflation of 3.00 percent and benefit cost inflation of 5.00 percent. Budget 2019/2020 figures were used for 2019/2020 taxes; future taxes are calculated based on forecasted revenues and prevailing tax rates.
- **Existing Debt** - The City currently has a total of five outstanding Public Works Trust Fund loans. Annual debt service ranges from a high of $742,000 in 2019 to a low of $226,000 in 2028. All loans are expected to be repaid by 2028.
- **Future Debt** - The capital funding strategy developed for this Plan forecasts the need for three new debt issuances (2031, 2033, and 2035) totaling $23.25 million in proceeds to fund the $72.7 million capital program. New debt is forecast as revenue bonds with a twenty-year term, 5.00 percent interest rate and 1.00 percent issuance cost. New debt service for the additional loans ranges from $480,000 in 2031 to $2.03 million by 2036.
- **Transfers to Capital** - Any Operating Fund balance above the minimum requirement is assumed to be available to fund capital projects and is projected to be transferred to the Capital Fund. The 2019 Operating Fund balance is expected to end the year at 90 days of O&M expenses, or $527,000. The Capital Fund balance is expected to end the year at $8.2 M.

Although the financial plan is completed for the planning period of this Plan (through 2036), the rate strategy focuses on the shorter-term planning period of 2019 through 2028. It is recommended that the City revisit the proposed rates every 2 to 3 years to ensure that the rate projections developed remain adequate. Any significant changes should be incorporated into the financial plan and future rates adjusted as needed.

Table 13-5 summarizes the annual revenue requirement based on the forecast of revenues, expenditures, fund balances and fiscal policies.

Table 13-5
10-Year Financial Forecast

Revenue Requirement	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028
Revenues										
Rate Revenues Under Existing Rates	$ 3,886,781	$ 3,937,309	$ 3,988,494	$ 4,040,344	$ 4,092,869	$ 4,146,076	$ 4,199,975	$ 4,254,575	$ 4,309,884	$ 4,365,913
Non-Rate Revenues	255,250	55,250	10,678	10,942	11,192	11,449	11,714	11,986	12,266	12,554
Total Revenues	**$ 4,142,031**	**$ 3,992,559**	**$ 3,999,172**	**$ 4,051,286**	**$ 4,104,061**	**$ 4,157,526**	**$ 4,211,689**	**$ 4,266,561**	**$ 4,322,151**	**$ 4,378,467**
Expenses										
Cash Operating Expenses	$ 2,137,044	$ 2,173,731	$ 2,252,529	$ 2,325,677	$ 2,401,315	$ 2,479,532	$ 2,560,422	$ 2,644,079	$ 2,730,602	$ 2,821,462
Existing Debt Service	741,913	738,336	734,759	731,183	672,507	669,205	303,982	302,490	300,998	225,822
New Debt Service	-	-	-	-	-	-	-	-	-	-
Rate Funded System Reinvestment	195,000	500,000	-	-	-	-	-	-	-	-
Total Expenses	**$ 3,073,957**	**$ 3,412,067**	**$ 2,987,289**	**$ 3,056,859**	**$ 3,073,821**	**$ 3,148,738**	**$ 2,864,404**	**$ 2,946,569**	**$ 3,031,600**	**$ 3,047,284**
Total Surplus (Deficiency)	**$ 1,068,074**	**$ 580,492**	**$ 1,011,883**	**$ 994,427**	**$ 1,030,239**	**$ 1,008,788**	**$ 1,347,286**	**$ 1,319,993**	**$ 1,290,551**	**$ 1,331,184**

Notes:
1) Rate funded system reinvestment shown in 2019/2020 are budgeted transfers to capital fund

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The financial forecast indicates that the utility is currently covering all financial obligations under existing rates. In recognition of the existing surplus, the City is proposing to hold rates flat (no rate increases) for three years (2020-2022) before inflationary increases (2.00 percent) are implemented thereafter. The remaining summaries are based on this proposed rate strategy.

City Funds and Reserves

Table 13-6 shows a summary of the projected City Operating and Capital Fund balances through 2028 based on the rate forecast presented herein. The operating fund is maintained at a minimum of 90 days of O&M expenses, and the capital fund balance fluctuates depending on the level of CIP funded; however, it will never fall below the capital contingency target.

Table 13-6
Ending Cash Balance Summary

Ending Fund Balances	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028
Operating Fund	$ 526,942	$ 535,988	$ 555,418	$ 573,455	$ 592,105	$ 611,392	$ 631,337	$ 651,965	$ 673,299	$ 695,703
Capital Fund	8,152,703	7,408,047	5,942,264	4,942,481	5,590,853	2,671,239	3,769,341	4,907,841	6,087,857	6,937,844
Total	**$8,679,645**	**$7,944,036**	**$6,497,682**	**$5,515,936**	**$6,182,958**	**$3,282,630**	**$4,400,678**	**$5,559,805**	**$6,761,156**	**$7,633,546**

13.6 Current and Projected Rates

13.6.1 Current Rates

The City's current rate structure consists of a fixed monthly charge that includes an allowance as well as a variable monthly charge per hundred cubic feet (ccf) for use above the allowance. All components of the rate structure vary by meter size. Table 13-7 shows the existing rate structure for 2019.

Table 13-7
2019 Existing Rate Structure

Meter Size	Allowance	Monthly Fixed	Variable (ccf)
5/8"	2	$ 59.88	$ 4.68
1"	5	158.03	4.30
1.5"	11.5	355.51	4.30
2"	20.5	632.13	4.30
3"	46	1,422.22	4.30
4"	82	2,528.35	4.30
6"	133.5	5,688.81	4.30
8"	325	8,368.62	4.30
Discount	2	14.97	4.68

13.6.1 Projected Rates

The financial strategy discussed above indicates that the sewer utility is covering all financial obligations in the near term with surplus cash available, therefore, no rate increases are forecast from 2020-2022 before 2.00 percent increases are implemented from 2023 forward. Table 13-8 shows the projected rates with increases applied uniformly to all rate components for all meter sizes.

Table 13-8
Proposed Schedule of Rates

Meter Size	Allowance	Existing	2020	2021	2022	2023	2024	2025	2026	2027	2028
5/8"	2	$ 59.88	$ 59.88	$ 59.88	$ 59.88	$ 61.08	$ 62.30	$ 63.55	$ 64.82	$ 66.12	$ 67.44
1"	5	158.03	158.03	158.03	158.03	161.19	164.41	167.70	171.05	174.47	177.96
1.5"	11.5	355.51	355.51	355.51	355.51	362.62	369.87	377.27	384.82	392.52	400.37
2"	20.5	632.13	632.13	632.13	632.13	644.77	657.67	670.82	684.24	697.92	711.88
3"	46	1,422.22	1,422.22	1,422.22	1,422.22	1,450.66	1,479.67	1,509.26	1,539.45	1,570.24	1,601.64
4"	82	2,528.35	2,528.35	2,528.35	2,528.35	2,578.92	2,630.50	2,683.11	2,736.77	2,791.51	2,847.34
6"	133.5	5,688.81	5,688.81	5,688.81	5,688.81	5,802.59	5,918.64	6,037.01	6,157.75	6,280.91	6,406.53
8"	325	8,368.62	8,368.62	8,368.62	8,368.62	8,535.99	8,706.71	8,880.84	9,058.46	9,239.63	9,424.42
Discount	2	14.97	14.97	14.97	14.97	15.27	15.58	15.89	16.21	16.53	16.86
Volume Rate - 5/8" Meter		$ 4.68	$ 4.68	$ 4.68	$ 4.68	$ 4.77	$ 4.87	$ 4.97	$ 5.07	$ 5.17	$ 5.27
Volume Rate - All Others		$ 4.30	$ 4.30	$ 4.30	$ 4.30	$ 4.39	$ 4.48	$ 4.57	$ 4.66	$ 4.75	$ 4.85

13.7 Affordability

The Washington State Department of Health and the Department of Commerce Public Works Board use an affordability index to prioritize low-cost loan awards depending on whether rates exceed 2.00 percent of the median household income for the service area. The average median household income for the City was $59,310 between 2013 and 2017 according to the U.S. Census Bureau. The 2017 value is escalated based on the assumed 3.00 percent general cost inflation to show the median household income in future years. Table 13-9 presents the City's rates projected to 2028, tested against the 2.00 percent monthly affordability threshold.

Table 13-9
Affordability Test

Year	Inflation	Median HH Income	2% Monthly Threshold	Projected Monthly Bill	% of Median HH Income
2017		$ 59,310	$ 98.85		
2018	3.00%	61,089	101.82		
2019	3.00%	62,922	104.87	$ 78.60	1.50%
2020	3.00%	64,810	108.02	78.60	1.46%
2021	3.00%	66,754	111.26	78.60	1.41%
2022	3.00%	68,757	114.59	78.60	1.37%
2023	3.00%	70,819	118.03	80.17	1.36%
2024	3.00%	72,944	121.57	81.77	1.35%
2025	3.00%	75,132	125.22	83.41	1.33%
2026	3.00%	77,386	128.98	85.08	1.32%
2027	3.00%	79,708	132.85	86.78	1.31%
2028	3.00%	82,099	136.83	88.52	1.29%

Notes:
1) Assumes a 5/8" meter and 6 ccf of monthly usage

The affordability test indicates that the City's rates are forecasted to remain below the 2.00 percent affordability threshold through 2028.

13.8 Conclusion

The results of this analysis indicate that rates can be held flat from 2020-2022 and provide revenue sufficient to cover all utility financial obligations before 2.00 percent annual increases are implemented from 2023 forward.

It is important to remember that the analysis performed in this chapter assumes growth rates from Chapter 3 of this GSP. If the future growth rates change, the existing rate strategy may need to be updated and revised.

It is recommended that the City regularly review and update the key underlying assumptions that compose the multi-year financial plan to ensure that adequate revenues are collected to meet the City's total financial obligations.

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APPENDICES

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Appendix A

SEPA Checklist

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CITY OF SNOHOMISH

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DETERMINATION OF NON-SIGNIFICANCE

File #SEPA20-0009

Date Issued: March 11, 2020

Proposal Name: City of Snohomish Comprehensive Sewer System Plan Update (City file no. **SEPA20-0009**)

Proponent: City of Snohomish Public Works Department
116 Union Avenue
P.O. Box 1589
Snohomish, WA 98291
(360) 282-3174

Lead Agency: City of Snohomish

Description of Proposal: Non-project action to adopt the updated Comprehensive Sewer System Plan (GSP), prepared in accordance with WAC 173-240-050. The GSP was last updated it 2010; it proposes various improvements to the sewer system to plan for the projected growth of the City's wastewater system customers.

The draft document is available at https://www.snohomishwa.gov/DocumentCenter/View/5998

Threshold Determination: The lead agency for this proposal has determined that it does not have a probable significant adverse impact on the environment. The requirements for environmental analysis, protection, and mitigation measures have been adequately addressed in the development regulations and comprehensive plans adopted under chapter 36.70A RCW, and in other applicable local, state, or federal laws or rules, as provided by RCW 43.21C.240 and WAC 197-11-158. An environmental impact statement (EIS) is not required under RCW 43.21C.030(2)(c). This decision was made after review of a completed environmental checklist, applicable regulations, and other information on file with the City of Snohomish. This information is available for review at the Snohomish City Hall, 116 Union Avenue, Snohomish, WA 98290 between the hours of 8:00 a.m. and 4:00 p.m. Monday through Thursday, excluding holidays.

Written comments may be submitted to the lead agency to the attention of Brooke Eidem, at the address below. Comments must be received by 4:00 p.m. on March 26, 2020.

SEPA Responsible Official: Glen Pickus, AICP
Position/Title: Planning Director **Phone:** (360) 282-3173 **E-mail:** pickus@snohomishwa.gov
Address: City of Snohomish, P.O. Box 1589, Snohomish, WA 98291

Signature: _____
Glen Pickus, Planning Director

APPEALS:
Unless withdrawn or revised pursuant to comments received within the comment period above, this Determination of Non-Significance shall be final on March 26, 2020. Appeals of this SEPA threshold determination must be filed in writing with the City Clerk for the City of Snohomish by 4:00 p.m. on March 26, 2020. Appeals must be made in accordance with the provisions of Chapter 14.20 SMC. A $500 appeal fee must be filed with the appeal. Appeals must be in writing and received as original documents by the close of the appeal period. Fax, e-mail and similar forms of document transmission shall not be accepted and shall not be considered as meeting the filing requirements. Appeals must state the section of the SMC being appealed, the specific determination or mitigation being appealed and the form of relief requested.

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Contact: (360) 282-3174

Details

CITY OF SNOHOMISH DETERMINATION OF NON-SIGNIFICANCE File #SEPA20-0009 Date Issued:March 11, 2020 Proposal Name: City of Snohomish Comprehensive Sewer System Plan Update (City file no. SEPA20-0009) Proponent:City of Snohomish Public Works Department 116 Union Avenue P.O. Box 1589 Snohomish, WA 98291 (360) 282-3174 Lead Agency: City of Snohomish Description of Proposal: Non-project action to adopt the updated Comprehensive Sewer System Plan (GSP), prepared in accordance with WAC 173-240-050. The GSP was last updated it 2010; it proposes various improvements to the sewer system to plan for the projected growth of the City's wastewater system customers. The draft document is available at https://www.snohomishwa.gov/DocumentCenter/View/5998 Threshold Determination: The lead agency for this proposal has determined that it does not have a probable significant adverse impact on the environment. The requirements for environmental analysis, protection, and mitigation measures have been adequately addressed in the development regulations and comprehensive plans adopted under chapter 36.70A RCW, and in other applicable local, state, or federal laws or rules, as provided by RCW 43.21C.240 and WAC 197-11-158. An environmental impact statement (EIS) is not required under RCW 43.21C.030(2)(c). This decision was made after review of a completed environmental checklist, applicable regulations, and other information on file with the City of Snohomish. This information is available for review at the Snohomish City Hall, 116 Union Avenue, Snohomish, WA 98290 between the hours of 8:00 a.m. and 4:00 p.m. Monday through Thursday, excluding holidays. Written comments may be submitted to the lead agency to the attention of Brooke Eidem, at the address below. Comments must be received by 4:00 p.m. on March 26, 2020. SEPA Responsible Official: Glen Pickus, AICP Position/Title: Planning Director Phone: (360) 282-3173 E-mail: pickus@snohomishwa.gov Address:City of Snohomish, P.O. Box 1589, Snohomish, WA 98291 APPEALS: Unless withdrawn or revised pursuant to comments received within the comment period above, this Determination of Non-Significance shall be final on March 26, 2020. Appeals of this SEPA threshold determination must be filed in writing with the City Clerk for the City of Snohomish by 4:00 p.m. on March 26, 2020. Appeals must be made in accordance with the provisions of Chapter 14.20 SMC. A $500 appeal fee must be filed with the appeal. Appeals must be in writing and received as original documents by the close of the appeal period. Fax, e-mail and similar forms of document transmission shall not be accepted and shall not be considered as meeting the filing requirements. Appeals must state the section of the SMC being appealed, the specific determination or mitigation being appealed and the form of relief requested. Published: March 11, 2020.EDH893924

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CITY OF SNOHOMISH

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STATE ENVIRONMENTAL POLICY ACT (SEPA) CHECKLIST

PURPOSE OF CHECKLIST

The State Environmental Policy Act (SEPA), Chapter 43.21C RCW, requires all governmental agencies to consider the environmental impacts of a proposal before making decisions. 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 [help]

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.

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.

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PLANNING

Project #:	SEPA20-0009		
Date:	2/25/2020	**Staff Intake:**	KMH

A. BACKGROUND [help]

ADMINISTRATION COMMENTS ONLY

1. Name of proposed project, if applicable:

   City of Snohomish General Sewer Plan

2. Name of applicant:

   City of Snohomish, Public Works Department

3. Address and phone number of applicant and contact person:

   Andrew Sics, City of Snohomish, PO Box 1589, Snohomish, WA 98291
   360-282-3174

4. Date checklist prepared:

   February 24, 2020

5. Agency requesting checklist:

   City of Snohomish, Public Works Department
   Washington State Department of Ecology

6. Proposed timing or schedule (including phasing, if applicable):

   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 2020.

7. Do you have any plans for future additions, expansion, or further activity related to or connected with this proposal? If yes, explain.

   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.

   Any future non-exempt projects will be subject to a separate SEPA review process.

8. List any environmental information you know about that has been prepared, or will be prepared, directly related to this proposal.

   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.

   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.

    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.)

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.

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.

ADMINISTRATION COMMENTS ONLY

Non-project action to adopt an updated General Sewer Plan (GSP). The GSP is a planning document prepared in accordance with WAC 173-240-050.
Draft plan available on City website at: https://www.snohomishwa.gov/DocumentCenter/View/5998

B. ENVIRONMENTAL ELEMENTS

1. Earth [help]

a. General description of the site (circle one):
Flat, rolling, hilly, steep slopes, mountainous, other _____

Flat, rolling, hilly, steep slopes.

b. What is the steepest slope on the site (approximate percent slope)?

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.

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.

ADMINISTRATION COMMENTS ONLY

d. Are there surface indications or history of unstable soils in the immediate vicinity? If so, describe.

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.

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.

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)?

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:

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.

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.

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:

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.

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.

ADMINISTRATION COMMENTS ONLY

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.

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.

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.

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.

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. 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 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.

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.

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.

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.

ADMINISTRATION COMMENTS ONLY

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.

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.

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.

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.

ci. Proposed measures to reduce or control surface, ground, and runoff water, and drainage pattern impacts, if any:

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:

- [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

b. What kind and amount of vegetation will be removed or altered?

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.

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:

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.

ADMINISTRATION COMMENTS ONLY

e. List all noxious weeds and invasive species known to be on or near the site.

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. Examples include:

birds: hawk, heron, eagle, songbirds, other:
mammals: deer, bear, elk, beaver, other:
fish: bass, salmon, trout, herring, shellfish, other _____

b. List any threatened and endangered species known to be on or near the site.

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.

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:

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.

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.

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.

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.

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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:

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.

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.

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.

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.

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.

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:

No measures to reduce or control environmental health hazards are necessary as part of this non-project action.

b. Noise

1) What types of noise exist in the area which may affect your project (for example: traffic, equipment, operation, other)?

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.

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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.

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:

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.

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.

City's Land Use map can be found online at https://www.snohomishwa.gov/DocumentCenter/View/476

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?

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:

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.

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?

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?

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.

The GSP has applicability throughout City limits. Zoning information is available on City website at https://www.snohomishwa.gov/177

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f. What is the current comprehensive plan designation of the site?
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?
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.
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?
Not applicable to this non-project action.

j. Approximately how many people would the completed project displace?
None for this non-project action.

k. Proposed measures to avoid or reduce displacement impacts, if any:
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:
None applicable for this non-project action.

m. Proposed measures to ensure the proposal is compatible with nearby agricultural and forest lands of long-term commercial significance, if any:
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.
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.
Not applicable to this non-project action.

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c. Proposed measures to reduce or control housing impacts, if any:

Not applicable to 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?

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?

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.

c. Proposed measures to reduce or control aesthetic impacts, if any:

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?

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?

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?

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:

None applicable for this non-project action.

12. Recreation [help]

a. What designated and informal recreational opportunities are in the immediate vicinity?

Not applicable to this non-project action.

City of Snohomish has several existing parks, not proposed for alteration under this plan adoption

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b. Would the proposed project displace any existing recreational uses? If so, describe.

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:

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 located on or near the site? If so, specifically describe.

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.

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.

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.

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.

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.

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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?

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?

This non-project action will not impact parking. Parking requirements for subsequent project work will need to comply with city code.

Parking regulated under Chapter 14.235 SMC

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).

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.

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 nonpassenger vehicles). What data or transportation models were used to make these estimates?

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.

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:

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.

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.

ADMINISTRATION COMMENTS ONLY

b. Proposed measures to reduce or control direct impacts on public services, if any.

None applicable for this non-project action.

16. Utilities [help]

a. Circle utilities currently available at the site:

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.

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 information and answers provided in this Environmental Checklist (including Supplement for Non-project Actions, if applicable) 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: _____

Date Submitted: 02/24/2020

Agency Evaluation completed by: Brooke Eidem Date: 3/9/2020

Signature: _____

D. supplemental sheet for nonproject actions [help]

(IT IS NOT NECESSARY to use this sheet for project actions)

ADMINISTRATION COMMENTS ONLY

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.

   Current local, state, and federal development regulations will adequately address discharge to water, emissions to air, toxic or hazardous substances and production of noise.

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.

   Plan adoption is not anticipated to affect plants, animals, fish, or marine life.

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.

ADMINISTRATION COMMENTS ONLY

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.

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Appendix B

Washington State Department of Ecology Agreed Order No. 10467 and Notice of Compliance

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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

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Appendix C

Population Analysis Methodology Memo

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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

---

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 D

WWTP DMR Flow Data Summary

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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**

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Appendix E

WWTP Hydraulic Analysis

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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**

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Appendix F

WWTP Process Capacity Analysis

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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			
Contact Tank							
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			
Typical PAA Dose, mg/L	1	1	1	1			
CT (concentration-time) , mg-min/L							
at annual avg flow	40	139	129	94			60
at design max month flow	40	67	65	57			60
at peak day flow	20	22	23	20			20-30
PAA Dose							
Type	15% PAA	15% PAA	15% PAA	15% PAA			
PAA Storage							
PAA Tote Volume, gallons	300	300	300	300			
Number of Totes	2	2	2	2			
Total PAA Volume, gallons	600	600	600	600			
Average PAA Use, gpd	12	8	9	12			
PAA Storage at Average Use, days	50	75	67	50			
Peak PAA Use, gpd	59	49	48	56			
PAA Storage at Peak Use, days	10	12	13	11			
PAA Metering							
Number of Metering Pumps	2	2	2	2			
Metering Pump Capacity Each, gpd	77	77	77	77			
Metering Pump Total Capacity	154	154	154	154			
Metering Pum Firm Capacity, gpd	77	77	77	77			

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 G

WWTP Lagoon Capacity Calculations

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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)

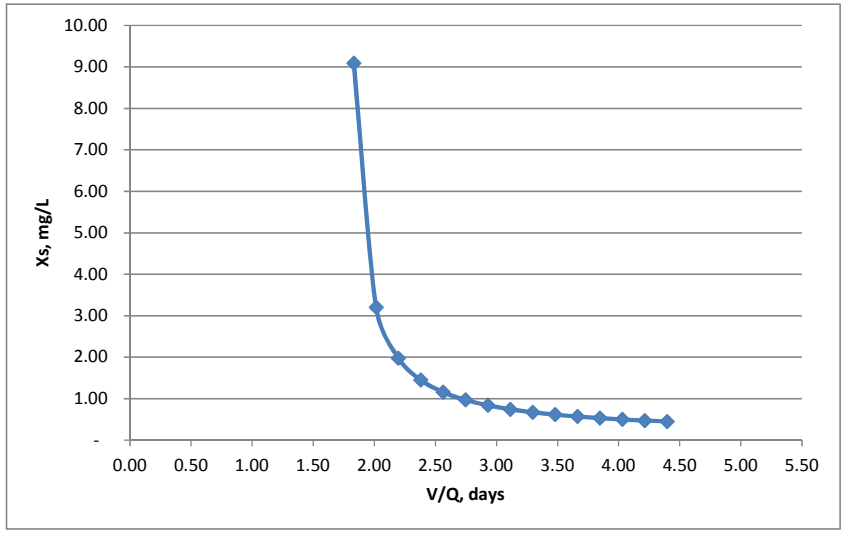
First, solve for $X_{B,H}$ based on multiple volumes using Eqn. 4:

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.

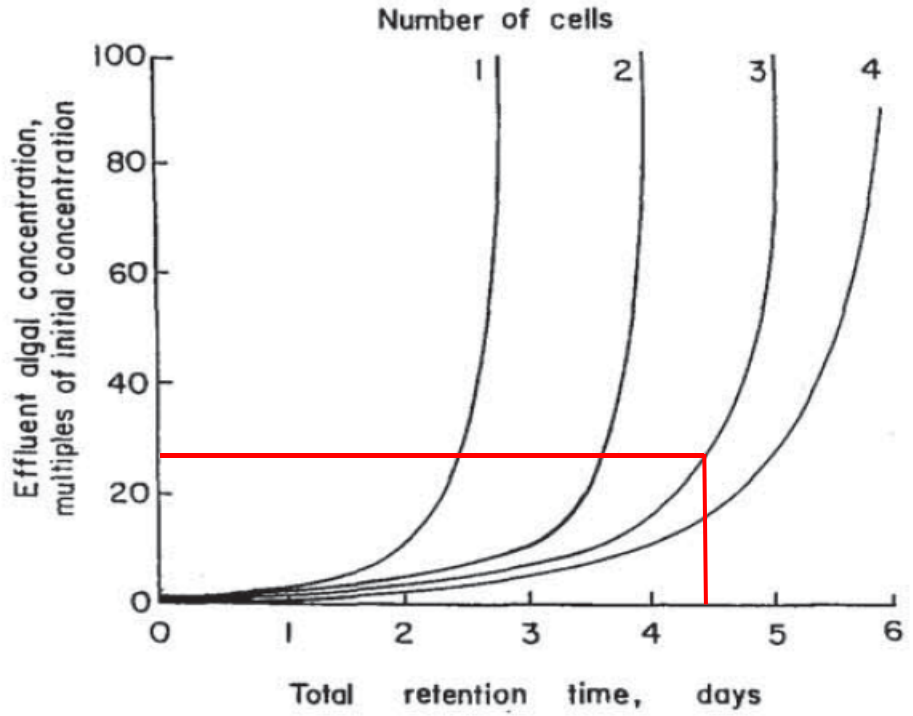


FIG. 4. Influence of Retention Time and Multicellular Configuration on Algal Growth [from Eq. (5)]

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)

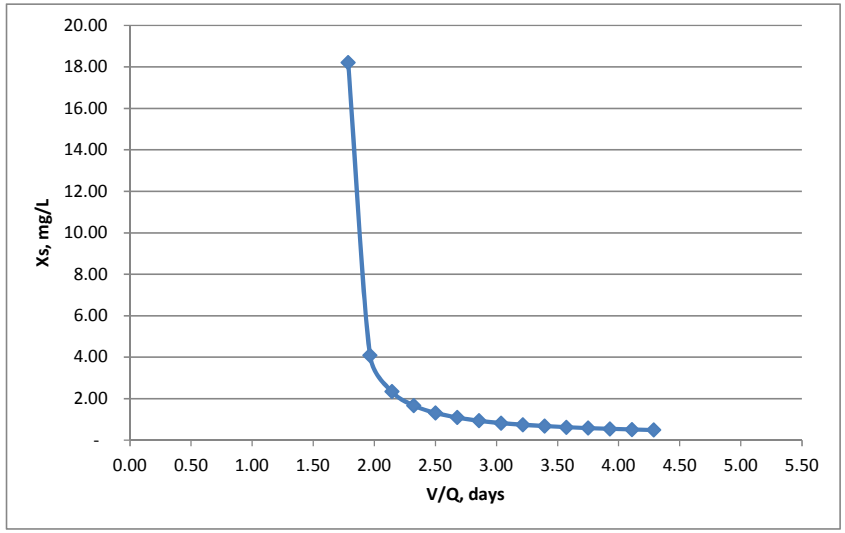
First, solve for $X_{B,H}$ based on multiple volumes using Eqn. 4:

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.

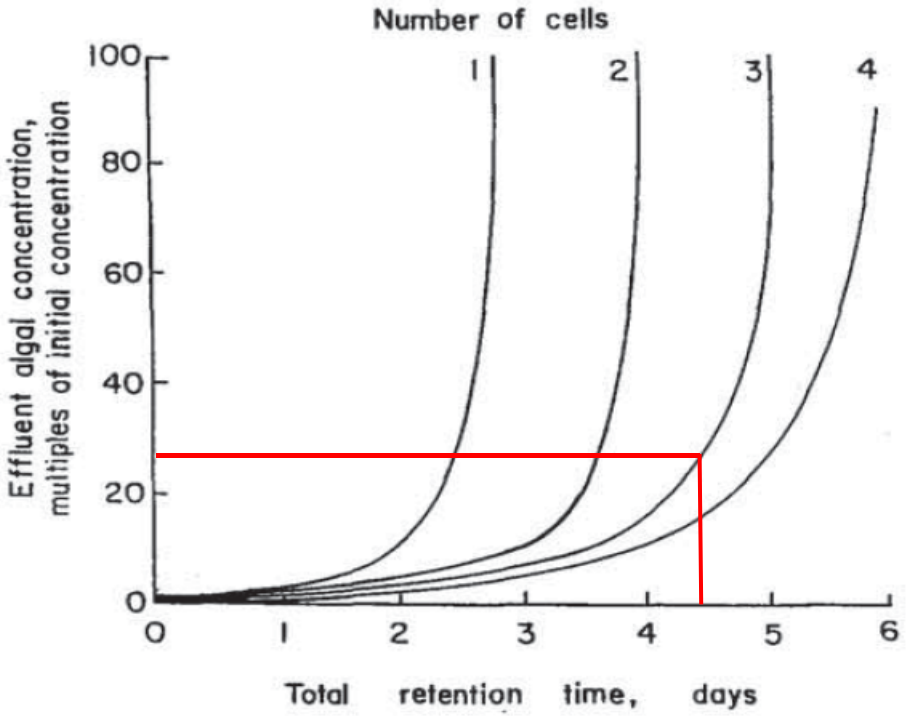


FIG. 4. Influence of Retention Time and Multicellular Configuration on Algal Growth [from Eq. (5)]

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)

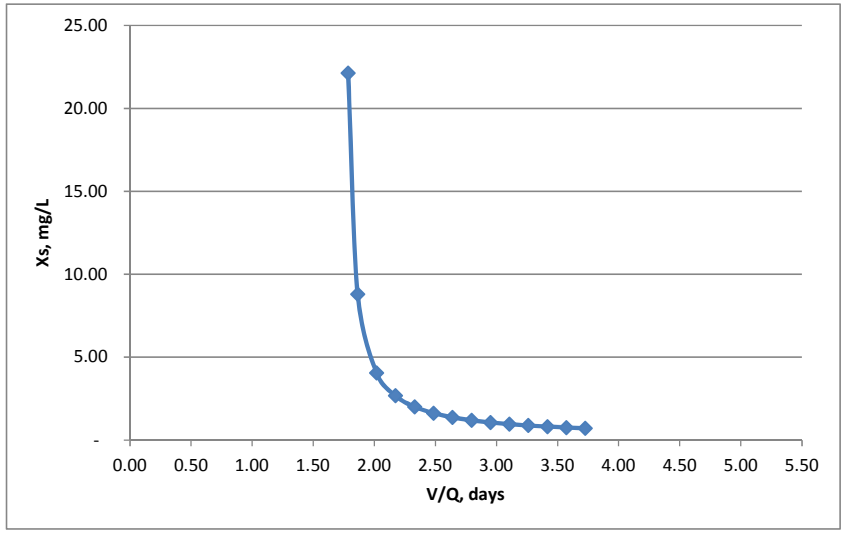
First, solve for $X_{B,H}$ based on multiple volumes using Eqn. 4:

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:
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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.

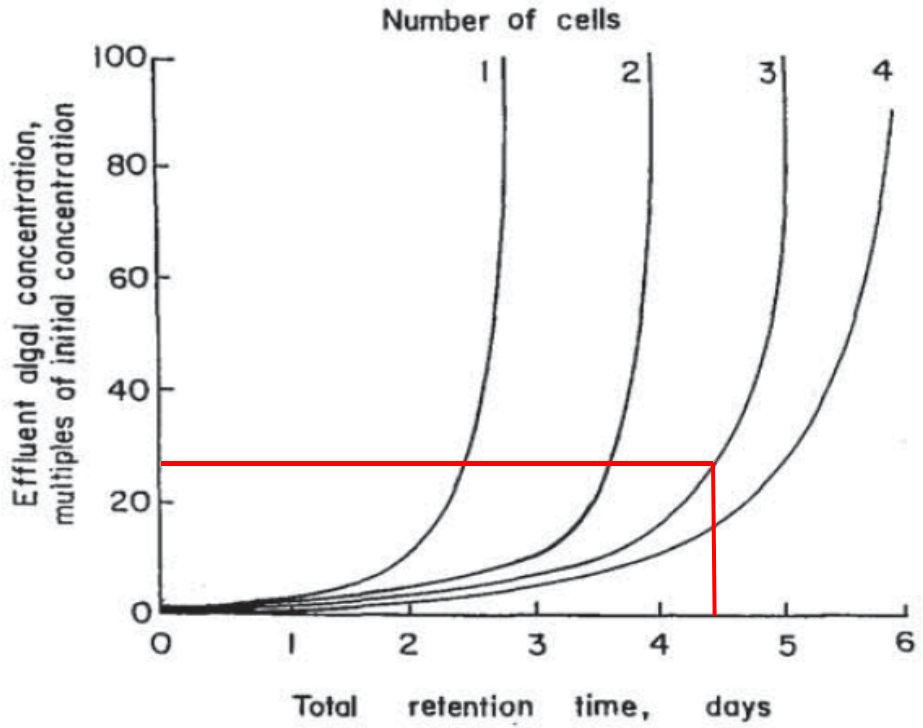

FIG. 4. Influence of Retention Time and Multicellular Configuration on Algal Growth [from Eq. (5)]

DESIGN OF DUAL-POWER MULTI-CELLULAR AERATED LAGOONS - Aeration and Mixing Sizing (2036)

This design method is based on the following articles:

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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)

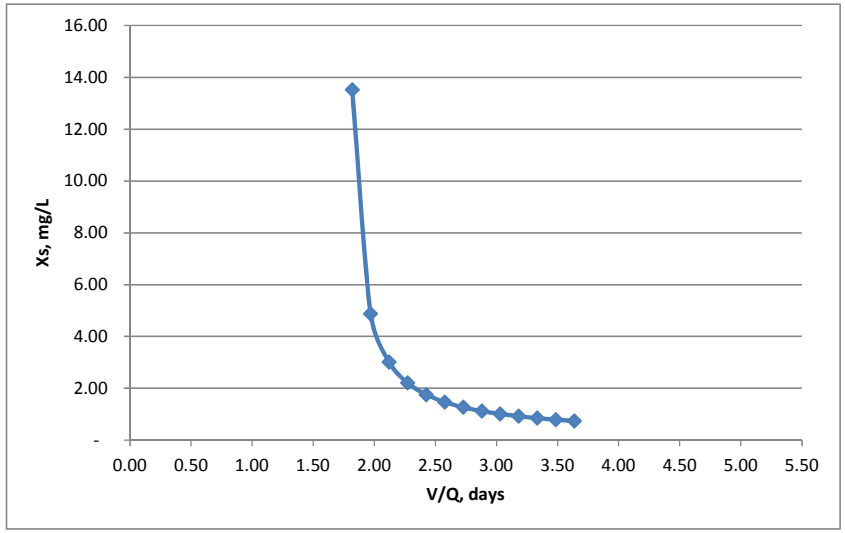
First, solve for $X_{B,H}$ based on multiple volumes using Eqn. 4:

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.

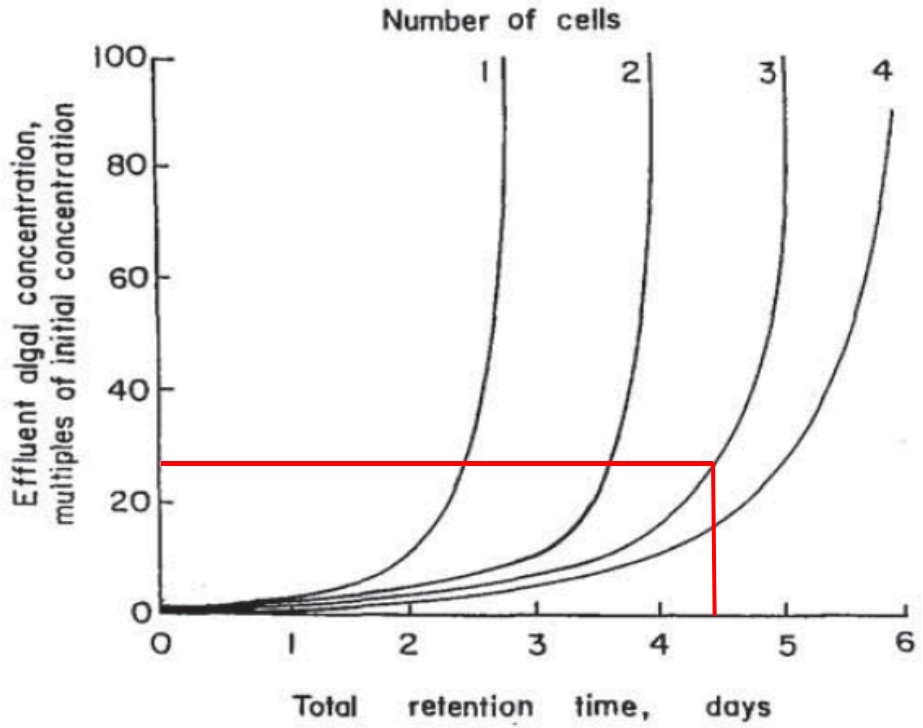


FIG. 4. Influence of Retention Time and Multicellular Configuration on Algal Growth [from Eq. (5)]

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 H

WWTP Alternative Life Cycle Costs

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Costs for Filtration Improvements

	Units	DynaSand	ECOwash	(2) 5x5 Fuzzy Filter	FlexFilter	Notes
Equipment Cost						
On-Site Package	$	259,500	338,500	329,000	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	27,600	86,000	
Fuzzy: Turbidity Monitoring				15,000		
Installation Labor	$	3,360	3,360	incl.	incl.	Labor on Dynasand quote was light. Used 4 workers, 80 hours, $50/hr
2.8 MGD Filtration Package	***$***	***363,360***	***452,360***	***371,600***	***580,000***	
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			
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**	**485,200**	**693,600**	
Structure						
Filter Cell Dimension	ft x ft	14x7	14x7	5x5		
Filter Cell Surface Area	sq. ft	100	100	25	90	
# of New Cells	#	2	2	2	3	
Total Filter Cell Surface Area	sq. ft	200	200	50	270	
Floc tank Length, width	ft			14	14	
Floc tank depth	ft			13	13	
Floc Tank Surface Area	sq. ft			196	196	
Floc tank Volume	gal			19,059	19,059	
Retention Time at 2.8 MGD	min			10	10	
Floc Tank Wall Volume	cu.ft			1,092	1,092	
Floc Tank Foundation Volume	cu.ft			490	490	
Floc Tank Concrete Volume	CY				40	
Added width around	ft			2		
Added footprint	sq.ft			28		
Added wall	cu.ft			540		
Added foundation	cu.ft			70		
Total new surface area for grating	sq. ft	200	200	246	466	
Grating unit price	$/sq.ft	58	58	58	58	

Void depth below new cell				7		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	
Special foundation	$/sq.ft			222		
Modify Outdoor Canopy Space for larger alum tank	$	10,000	10,000	10,000	10,000	
						Flex: Void fill plus other chamber modifications, use 1/2 of Dynasand new basin. For 3rd cell, use same concrete amt.
Concrete (w/ JH pricing)	$	104,791	104,791	n/a	52,395	as for new sand filter cell
Grating	$	11,598	11,598	14,265	27,023	Based on filter surface area
Void fill below filter	$			3,227	16,137	
New wall + foundation built	$			16,447		
Modifications for void space	$			20,000	0	Fuzzy: ladder, portable sump, light, etc.
Floc Tank	$			38,997	38,997	Floc: assume 10 min. HRT, 14x14x13 tank
Structures Total	***$***	***$126,400***	***$126,400***	***$103,000***	***$144,600***	
Subtotal Construction Cost	**$**	**670,300**	**711,800**	**588,200**	**838,200**	
Special Foundations	$	43,492	43,492	0	43,492	piling for new tank (filter, floc tank)
						For Fuzzy, use 10% for site work due to
Site Works (5% of Subtotal, 10% for Flex)	$	33,515	35,590	58,820	41,910	deep excavation for piping.
Electrical (15% of Subtotal)	$	100,545	106,770	88,230	125,730	
Mechanical (10% of Subtotal)	$	67,030	71,180	58,820	83,820	
Control (5% of Subtotal, instruments included in Quote)	$	33,515	35,590	29,410	41,910	
Subtotal Including Site Work, Electrical, Mechanical and Control	**$**	**948,400**	**1,004,500**	**823,500**	**1,175,100**	
Incidentals - OH&P, Mobilization, etc (15% of Structures and Equipment)	$	142,260	150,675	123,525	176,265	
Equipment, Structures, Incidentals Total	**$**	**1,091,000**	**1,156,000**	**948,000**	**1,352,000**	
Sales Tax (9.1%)	$	99,281	105,196	86,268	123,032	
OPCC Subtotal	**$**	**1,191,000**	**1,262,000**	**1,035,000**	**1,476,000**	
Contingency (35% of OPCC Subtotal)	$	381,850	404,600	331,800	473,200	
Total OPCC (2013)	**$**	**1,573,000**	**1,667,000**	**1,367,000**	**1,950,000**	
Plannning (5% of Total OPCC)	$	78,650	83,350	68,350	97,500	
Design and Permitting (10% of Total OPCC)	$	157,300	166,700	136,700	195,000	
Construction Mangmt. (15% of Total OPCC)	$	235,950	250,050	205,050	292,500	
Total Project Cost (2013)	**$**	**2,045,000**	**2,168,000**	**1,778,000**	**2,535,000**	
RSMeans Cost Index (2013)		201.2	201.2	201.2	201.2	
RSMeans Cost Index (2017)		208.5	208.5	208.5	208.5	
Adjusted Total Project Cost (2017)	**$**	**2,120,000**	**2,247,000**	**1,843,000**	**2,627,000**	

O&M Cost

Air compressor	HP	5	5	25	40	
Backwash duration	min	60	22	15	30	
Backwash #		continuous	once per hour	3 to 5 times per day	2 to 5 times per day	
Backwash #	ea	24	24	3	5	
Backwash flow	gpm	292	292	100	75	
Backwash cycle on time	hr/day	24	9	1	3	
Daily power	kWh/day	90	33	14	75	
Annual blower power consumption	kWh/year	32675	11981	5105	27229	
Backwash Volume	MG/year	153.48	56.27	1.64	4.11	
HP consumption at	gpm	280	280	100	100	
Reject HP	HP	7.60	7.60	3.40	3.40	
Annual RJ flow power consumption	kWh/year	51794	18991	694	1736	
						dyna from parkson. Based on backwash
Annual Power Consumption	kWh/year	84,469	30,972	5,800	28,965	minutes and HP of compressor
Energy Cost	$/kWh	0.075	0.075	0.075	0.075	
Total Annual Energy Cost	$/Year	6,300	2,300	400	2,200	
						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	12	10	26	40	Duane Leach says 8 hrs/mo. But they haven't done any large maint. Yet? Added 4 hrs/mo. for continuous and 2 hrs/mo. For ECO, since ECO replaces airlift less often
Labor Cost	$/hr	50	50	50	50	
Total Annual Labor Cost	$/Year	7,200	6,000	15,600	24,000	
Media	**$**			**19,000**	**12,150**	Repl. Every 10 years.
Motor and actuator	$			24,000	24,000	Repl. Every 10 years.
Sum	$			43,000	36,150	
Life	years			10	10	
Replacement NPV	$			32,000	26,900	
Yearly Replacement Conversion				***3,751***	***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	3,750	3,150	
Total Yearly O&M Cost (20 years)	**$/Year**	**82,100**	**62,700**	**71,000**	**80,600**	
Discount Rate (2010 rate for federal water projects)	%	3.000%	3.000%	3.000%	3.000%	
Period	years	20	20	20	20	
Present value factor of an equal annuity for the period		14.88	14.88	14.88	14.88	
20-Year O&M Cost	**$**	**1,221,000**	**933,000**	**1,056,000**	**1,199,000**	
20-Year NPV	**$**	**3,341,000**	**3,180,000**	**2,899,000**	**3,826,000**	

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Cost
Denitrifying Sand Filters Alternative
Prepared by: C. Bryant
Reviewed by: T. Giese
Date: 7/24/20

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	12" Flow Meter	$8,000	1	EA	$8,000
2	Flow Control Valve	$3,500	1	EA	$3,500
3	Flow Control Valve Actuator	$10,000	1	EA	$10,000
4	Vault	$10,400	1	EA	$10,400
5	Excavation for Sand Filter Basins	$14	1,944	CY	$27,611
6	Backfill for Sand Filter Basins	$11	1,019	CY	$11,102
7	Shoring	$29	4,750	SF	$135,375
8	Dewatering	$25,000	1	LS	$25,000
9	Concrete for Sand Filter Basins (includes foundation)	$1,000	260	CY	$260,276
10	Aluminum Covers for Sand Filter Basins	$105	375	SF	$39,375
11	Concrete for Chemical Storage Tank Pad	$1,000	11	CY	$10,667
12	Micro C Storage Tank	$82,980	1	LS	$82,980
13	WesTech Package	$576,700	1	LS	$576,700
14	Upsize Reject Pumps and Pipeline	$60,000	1	LS	$60,000
15	Piping Allowance	$75,000	1	LS	$75,000
16	16" Pipe (Ductile Iron)	$201	160	LF	$32,234
17	2" Chemical Piping (DR 11 HDPE)	$3	188	LF	$553
18	Pipe Excavation	$15	285	CY	$4,272
19	Pipe Bedding	$2	800	CF	$1,200
20	Pipe Backfill	$10	285	CY	$2,848
21	Equipment Installation (25%)	$144,175	1	LS	$144,175
22	Electrical, Instrumentation and Controls (14%)	$212,977	1	LS	$212,977
23	Mobilization and Division 1 Costs (10%)	$173,424	1	LS	$173,424
	Subtotal				$1,907,669
	Estimate Contingency @	35.0%			$667,684
	Subtotal				$2,575,353
	Sales Tax @	9.2%			$236,932
	Estimated Bid Price (Rounded to 1K)				**$2,812,000**
	Design, Bid Support @	15.0%			$421,800
	Construction Mgmt @	10.0%			$281,200
	Estimated Project Price (Rounded to 10K)				**$3,520,000**

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Cost
Microvi Alternative
Prepared by: C. Bryant
Reviewed by: T. Giese
Date: 7/16/20

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	16x16x12 Tee (Ductile Iron)	$4,430	1	EA	$4,430
2	16" Plug Valve	$7,025	1	EA	$7,025
3	12" Plug Valve	$2,555	1	EA	$2,555
4	Vault	$10,400	1	EA	$10,400
5	12" Flow Meter	$8,000	1	EA	$8,000
6	Flow Control Valve	$3,500	1	EA	$3,500
7	Flow Control Valve Actuator	$10,000	1	EA	$10,000
8	Concrete for Microvi Reactors Pad	$1,000	222	CY	$222,222
9	Concrete for Chemical Storage Tank Pad	$1,000	11	CY	$10,667
10	Micro C Storage Tank	$82,980	1	LS	$82,980
11	Microvi Package	$3,000,000	1	LS	$3,000,000
12	16" Pipe (Ductile Iron)	$201	187	LF	$37,673
13	12" Pipe (Ductile Iron)	$157	187	LF	$29,329
14	2" Chemical Piping (DR 11 HDPE)	$3	330	LF	$970
15	Pipe Excavation	$15	666	CY	$9,986
16	Pipe Bedding	$2	1,870	CF	$2,805
17	Pipe Backfill	$10	666	CY	$6,657
18	Equipment Installation (25%)	$750,000	1	LS	$750,000
19	Electrical, Instrumentation and Controls (14%)	$587,888	1	LS	$587,888
20	Mobilization and Division 1 Costs (10%)	$478,709	1	LS	$478,709
	Subtotal				$5,265,795
	Estimate Contingency @	35.0%			$1,843,028
	Subtotal				$7,108,824
	Sales Tax @	9.2%			$654,012
	Estimated Bid Price (Rounded to 1K)				**$7,763,000**
	Design, Bid Support @	15.0%			$1,164,450
	Construction Mgmt @	10.0%			$776,300
	Estimated Project Price (Rounded to 10K)				**$9,700,000**

City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Cost
MBBR Alternative
Prepared by: C. Bryant
Reviewed by: T. Giese
Date: 7/15/20

Bid Item No.	Bid Item Description	Unit Bid Price	Quantity	Unit	Total
1	Excavation	$14.20	2,778	CY	$39,448
2	Dewatering	$25,000	1	LS	$25,000
3	Shoring	$28.50	5,500	SF	$156,750
4	Backfill	$11	1,237	CY	$13,483
5	24x24x16 Tee (Ductile Iron)	$12,820	1	EA	$12,820
6	24" Plug Valve	$14,726	1	EA	$14,726
7	16" Plug Valve	$7,025	1	EA	$7,025
8	Vault	$10,400	1	EA	$10,400
9	16" Flow Meter	$10,667	1	EA	$10,667
10	Flow Control Valve	$3,500	1	EA	$3,500
11	Flow Control Valve Actuator	$10,000	1	EA	$10,000
12	Concrete for MBBR Tank	$1,000	434	CY	$434,444
13	Concrete for Chemical Storage Tank Pad	$1,000	11	CY	$10,667
14	Concrete for Building Slab	$1,000	33	CY	$33,333
15	Building	$275	450	SF	$123,750
16	Micro C Storage Tank	$82,980	1	LS	$82,980
17	Nexom MBBR Package	$660,000	1	LS	$660,000
18	16" Pipe (Ductile Iron)	$201	642	LF	$129,418
19	8" Air Piping (DR 11 HDPE)	$19	94	LF	$1,750
20	2" Chemical Piping (DR 11 HDPE)	$3	88	LF	$259
21	Pipe Excavation	$15	1,143	CY	$17,152
22	Pipe Bedding	$2	3,212	CF	$4,818
23	Pipe Backfill	$10	1,143	CY	$11,435
24	Catwalk	$400	160	SF	$64,000
25	Railing	$200	260	LF	$52,000
26	Equipment Installation (25%)	$165,000	1	LS	$165,000
26	Electrical, Instrumentation and Controls (14%)	$293,276	1	LS	$293,276
27	Mobilization and Division 1 Costs (10%)	$238,810	1	LS	$238,810
	Subtotal				$2,626,911
	Estimate Contingency @	35.0%			$919,419
	Subtotal				$3,546,330
	Sales Tax @	9.2%			$326,262
	Estimated Bid Price (Rounded to 1K)				**$3,873,000**
	Design, Bid Support @	15.0%			$580,950
	Construction Mgmt @	10.0%			$387,300
	Estimated Project Price (Rounded to 10K)				**$4,840,000**

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 (9.1%)	$ 24,593	$ 22,500	$ 31,274	$ 111,741
OPCC Subtotal	**$ 294,843**	**$ 269,750**	**$ 374,946**	**$ 1,339,665**
Contingency (35% of OPCC Subtotal)	$ 103,195	$ 94,412	$ 131,231	$ 468,883
Total OPCC	**$ 398,038**	**$ 364,162**	**$ 506,177**	**$ 1,808,548**
Plannning (5% of Total OPCC)	$ 19,902	$ 18,208	$ 25,309	$ 90,427
Design and Permitting (10% of Total OPCC)	$ 39,804	$ 36,416	$ 50,618	$ 180,855
Construction Mangmt. (15% of Total OPCC)	$ 59,706	$ 54,624	$ 75,927	$ 271,282
Total Project Cost	**$ 517,000**	**$ 473,000**	**$ 658,000**	**$ 2,351,000**

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	**$ 2,592,000**	**$ 2,548,000**	**$ 3,389,000**	**$ 4,036,000**

Appendix I

CIP Detailed Cost Estimates

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City of Snohomish
General Sewer Plan and Wastewater Facilities Plan
Engineer's Planning Level Opinion of Probable Project Costs
CIP C-2: 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-4: 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-5: 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-6: 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-8: 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-10: 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-11: 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-12: 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

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 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

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 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

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.

Appendix J

Snohomish WWTP NPDES Permit

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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.

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Appendix K

Snohomish CSO Management Plan

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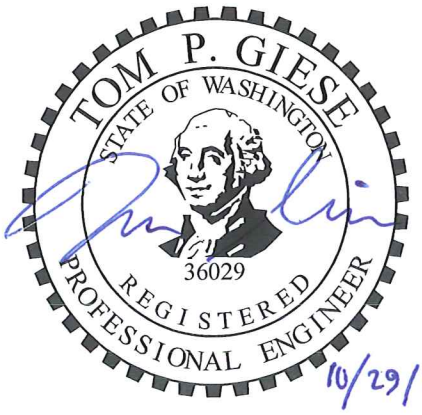
SNOHOMISH
WASHINGTON

City of Snohomish

Combined Sewer Overflow Management Plan

October 2014



10/29/14

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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

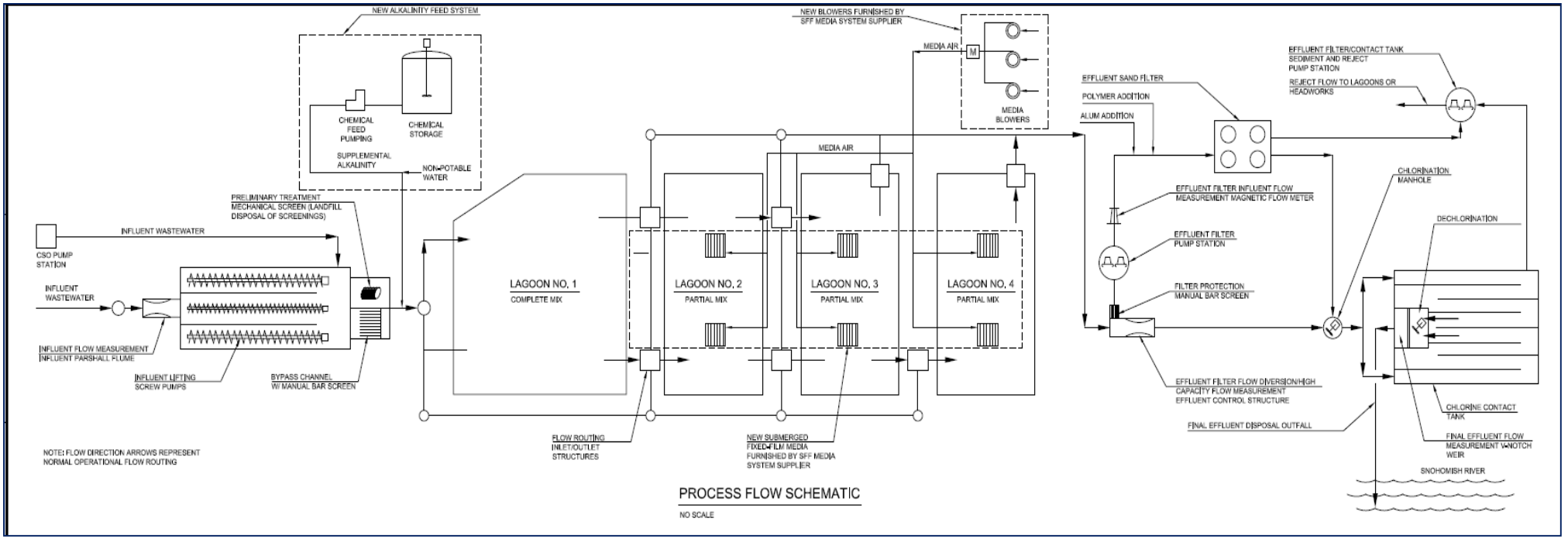
The City currently operates a four-stage lagoon WWTP, which was retrofitted with a submerged fixed-film (SFF) media system in 2012. The WWTP also includes influent pumping and flow measurement, screening, addition of supplemental alkalinity, effluent filtration, and chlorine disinfection followed by 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)

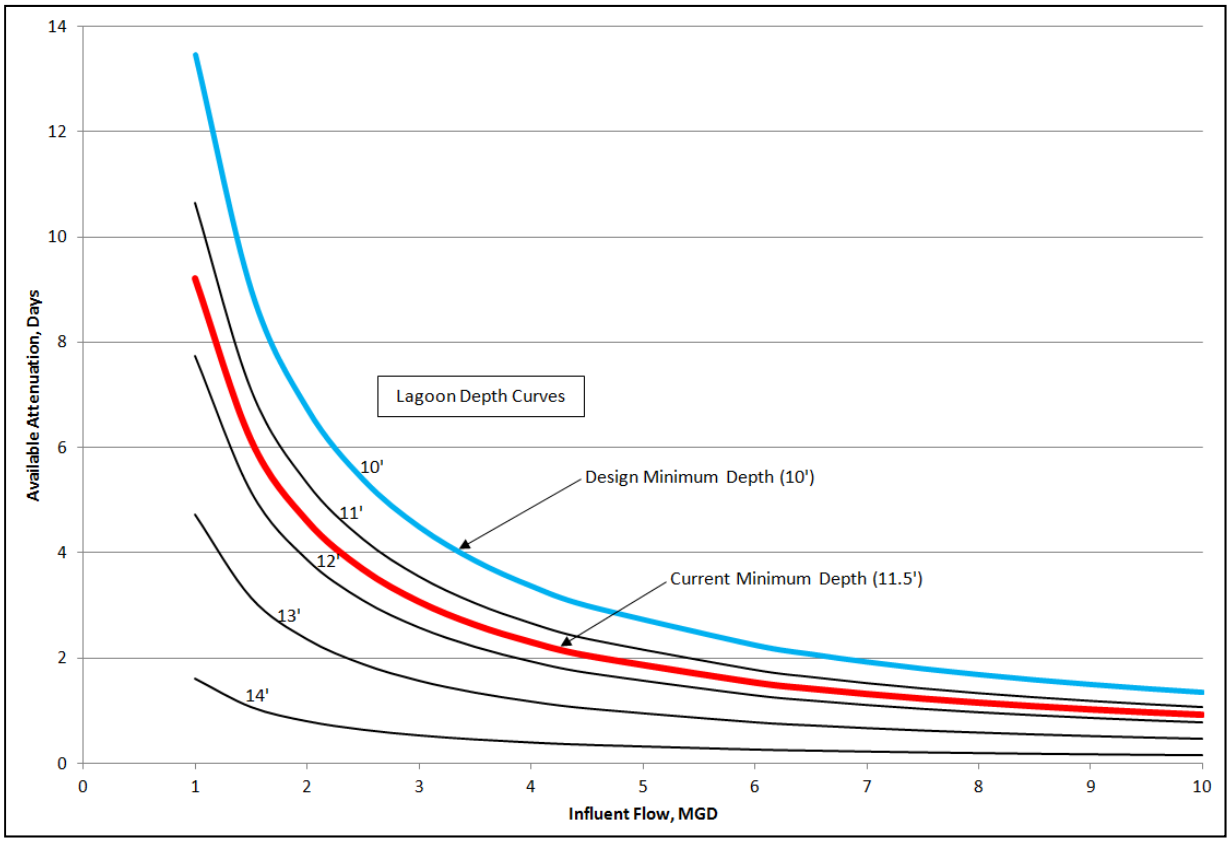
FLOW ATTENUATION

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

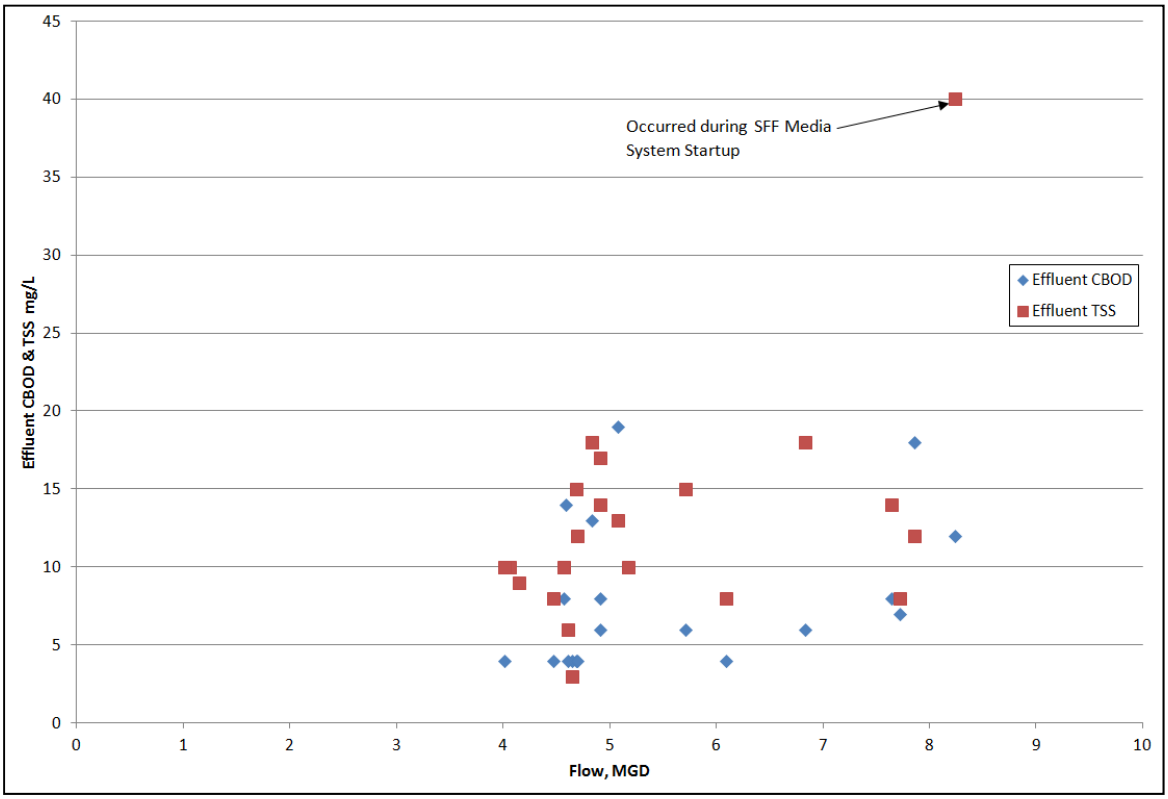
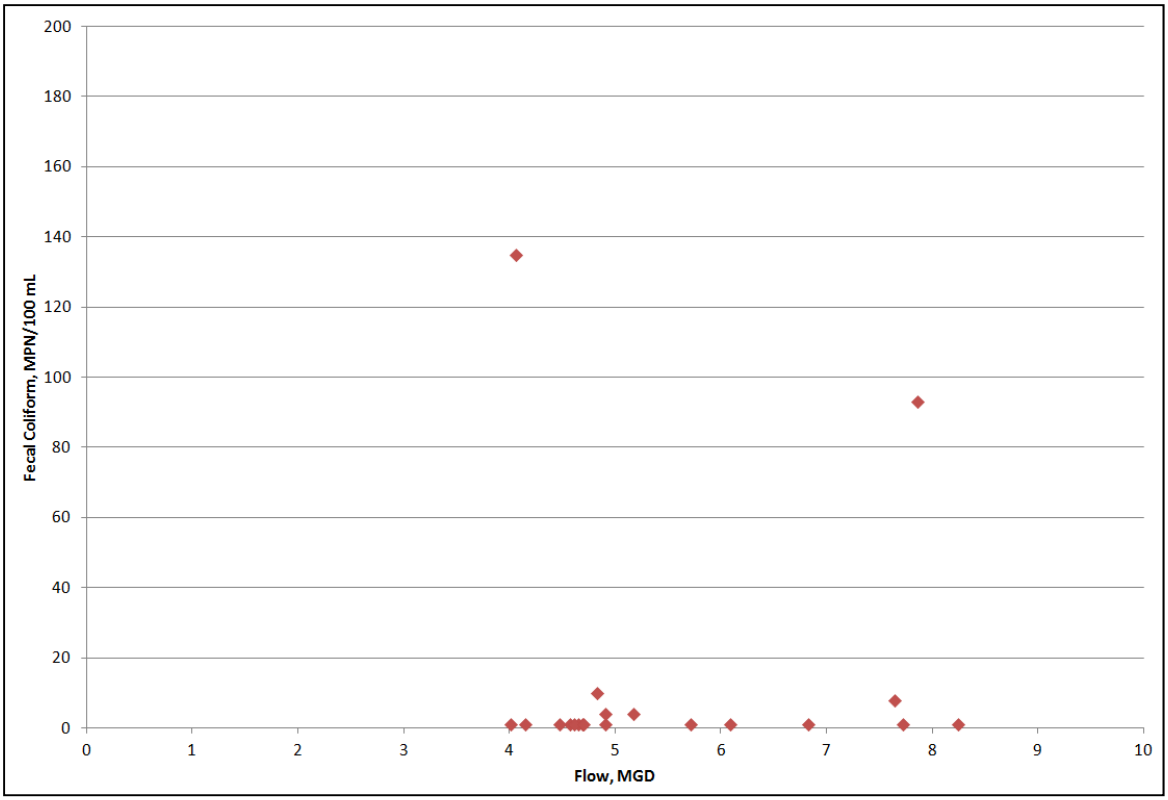


FIGURE 2-4: EFFLUENT FECAL COLIFORM AT HIGH FLOWS



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

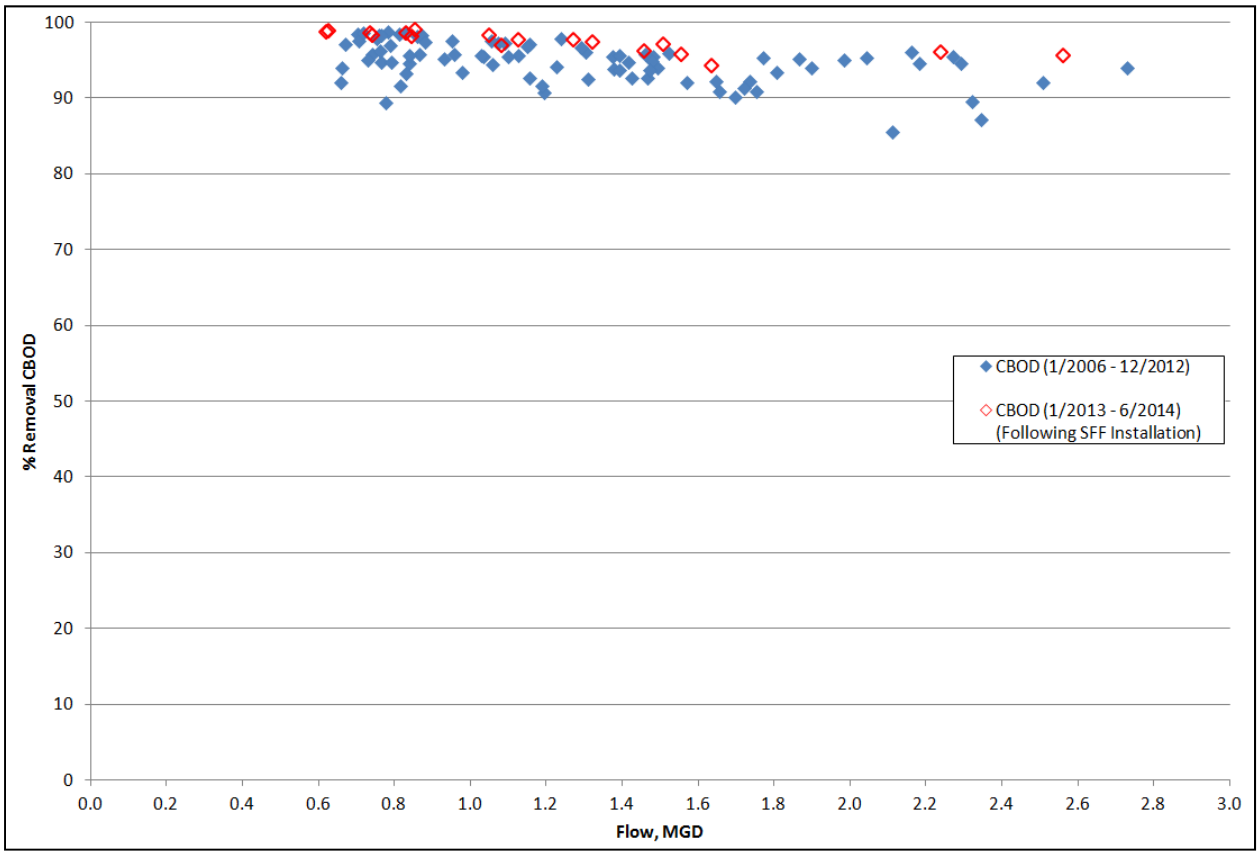
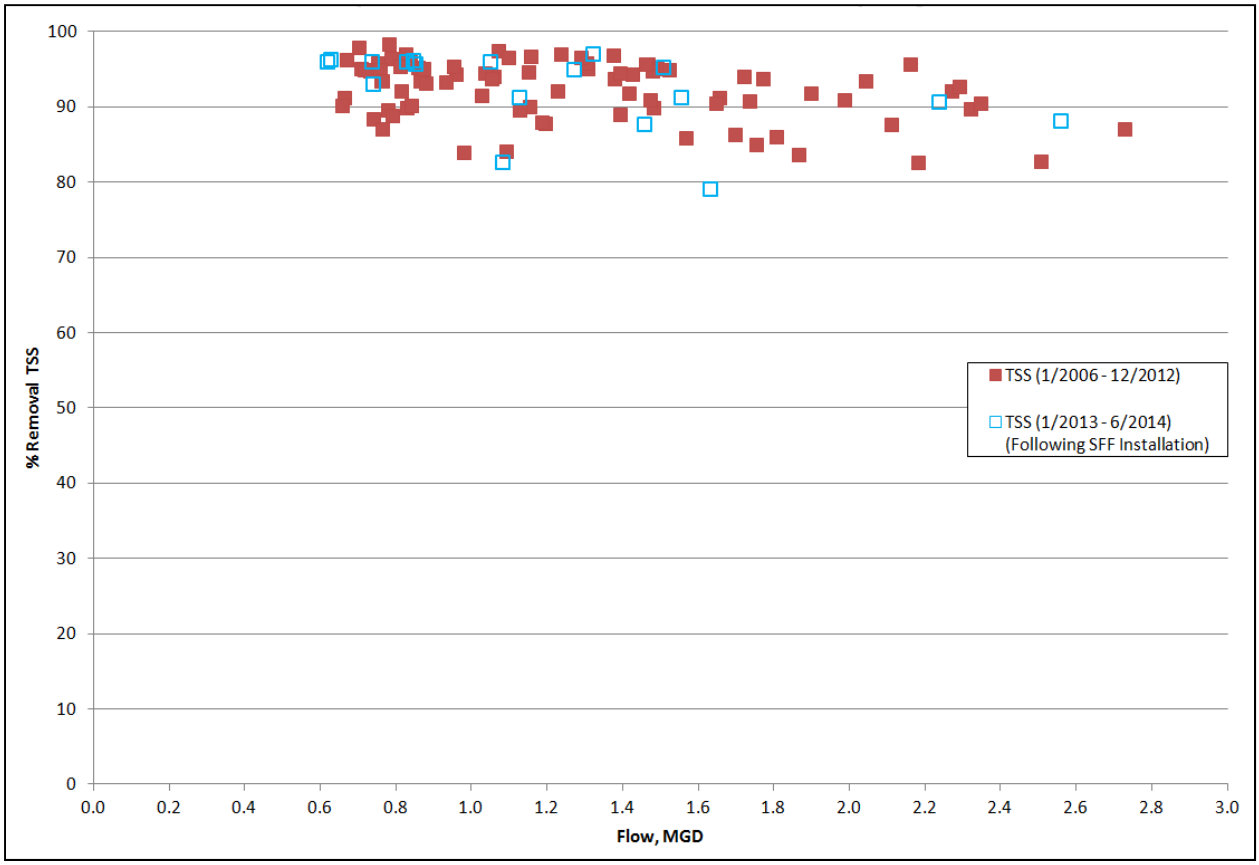


FIGURE 2-6: MONTHLY AVERAGE TSS PERCENT REMOVAL VERSUS MONTHLY AVERAGE FLOW



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

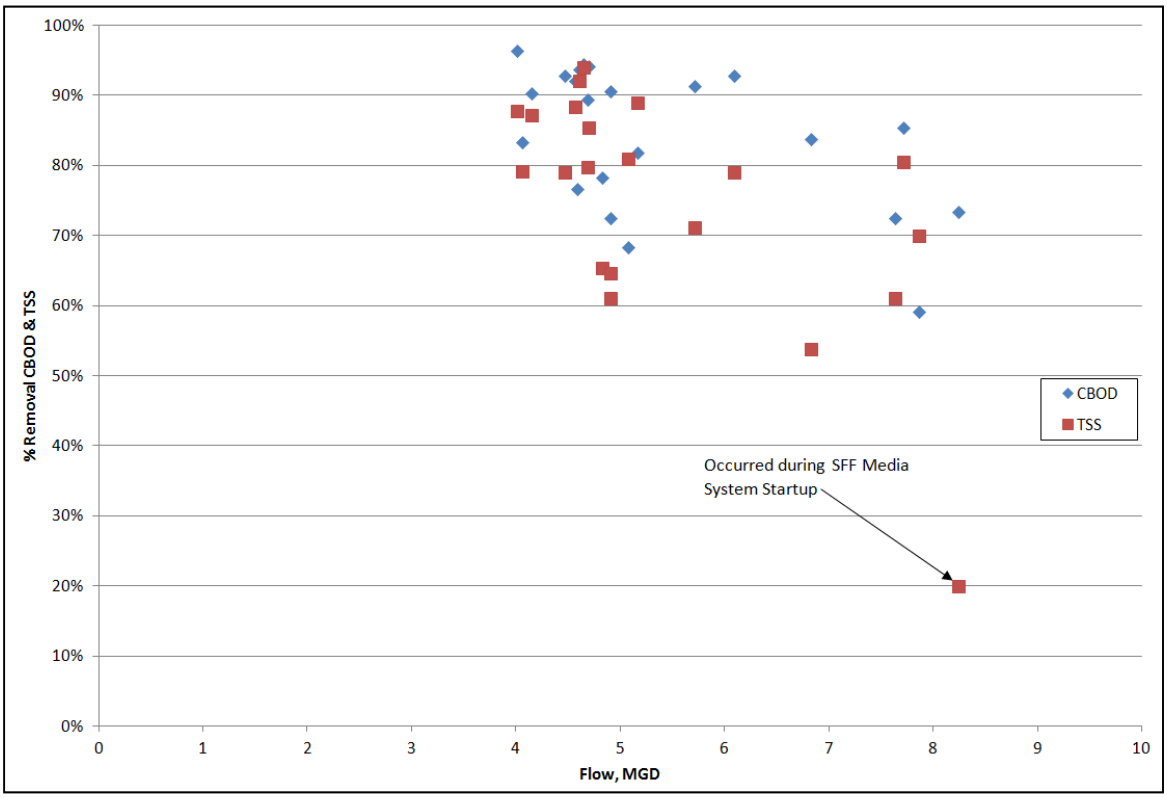


FIGURE 2-8: EFFLUENT TSS CONCENTRATION VERSUS FLOW

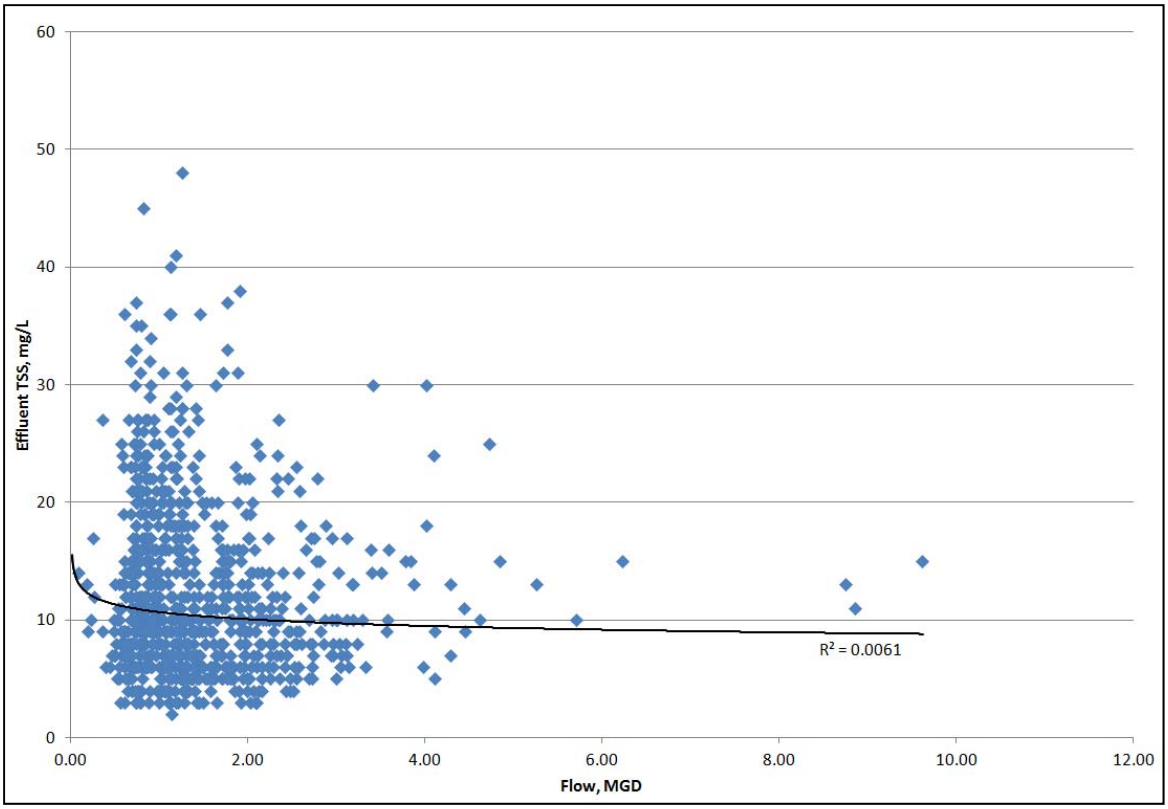
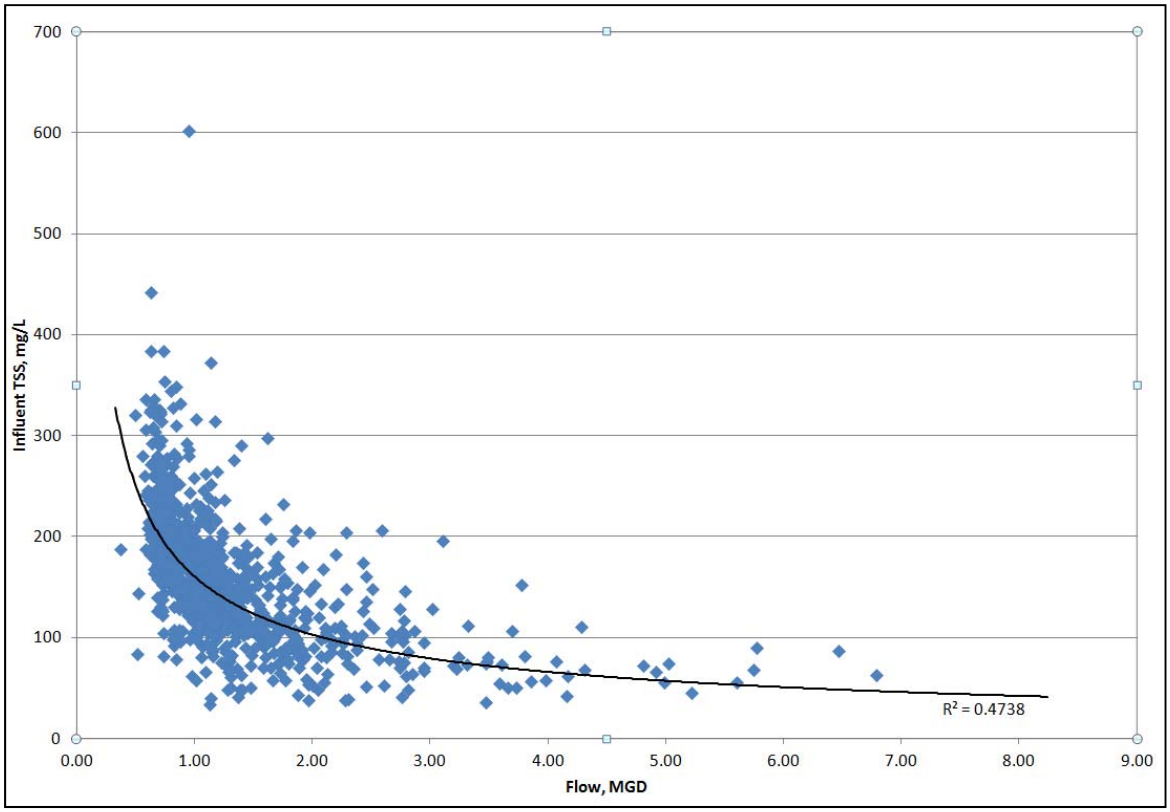


FIGURE 2-9: INFLUENT TSS CONCENTRATION VERSUS FLOW



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

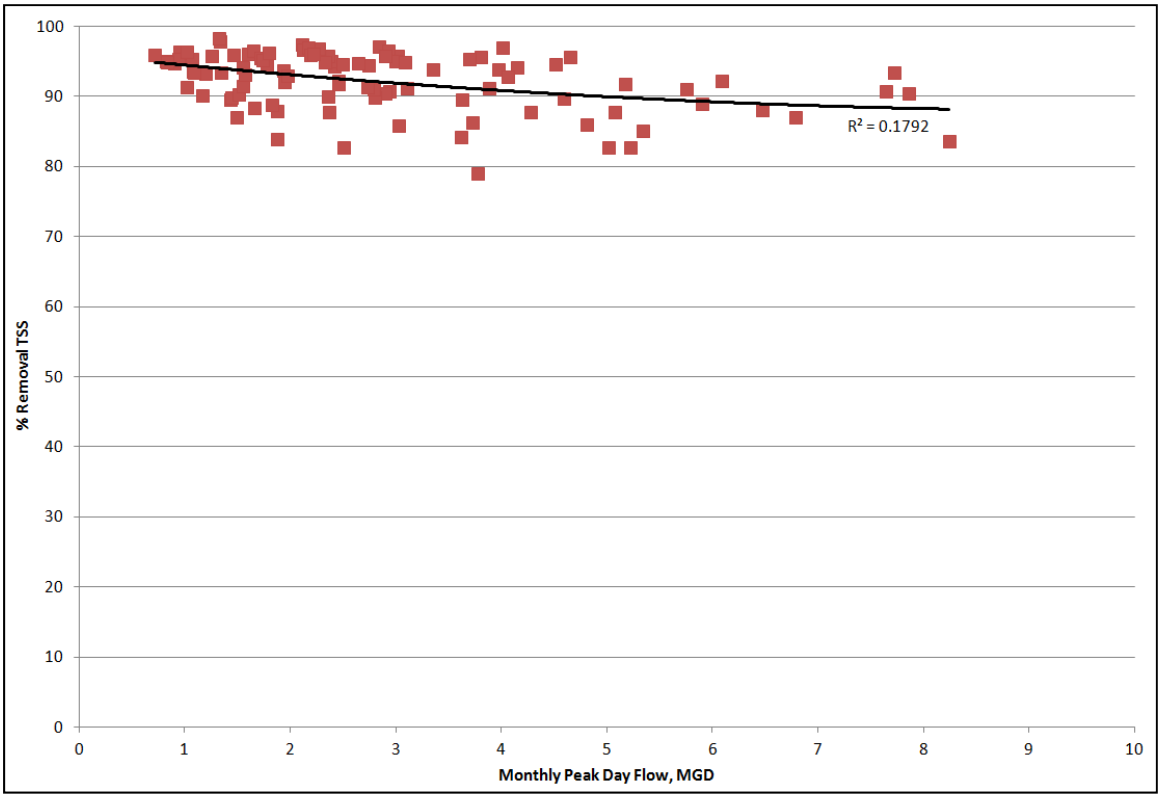
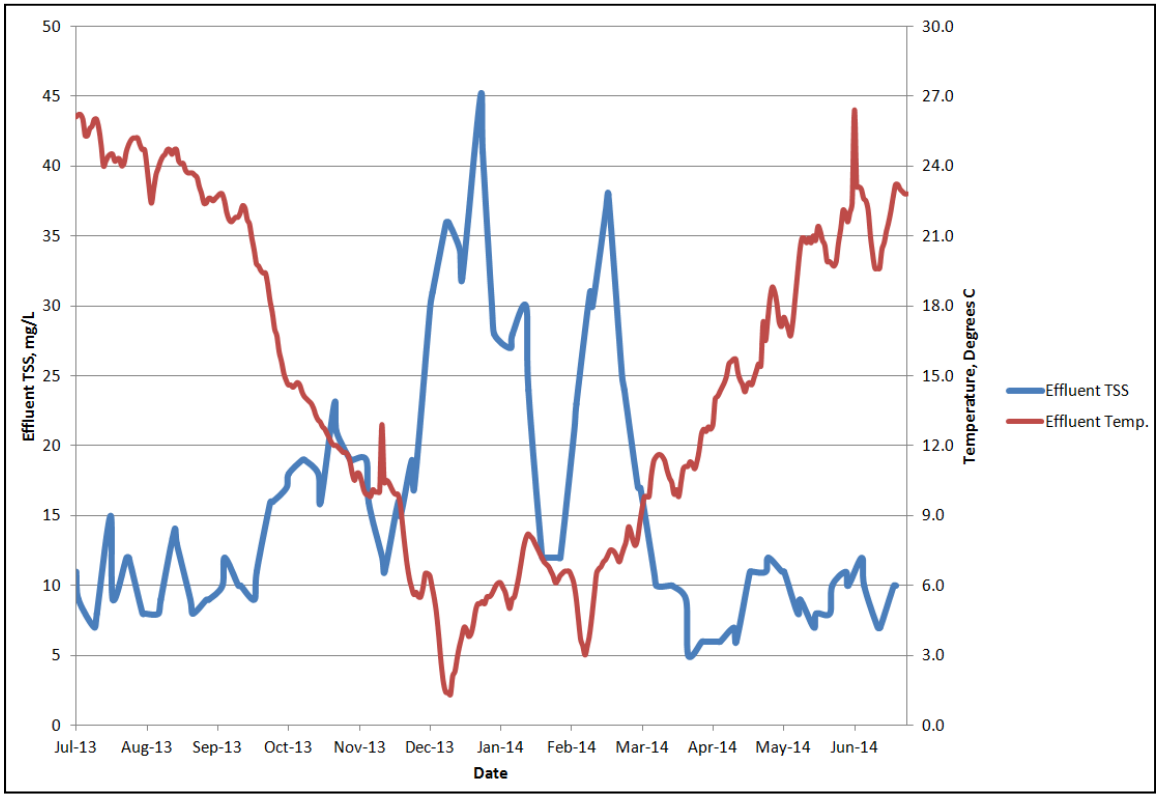
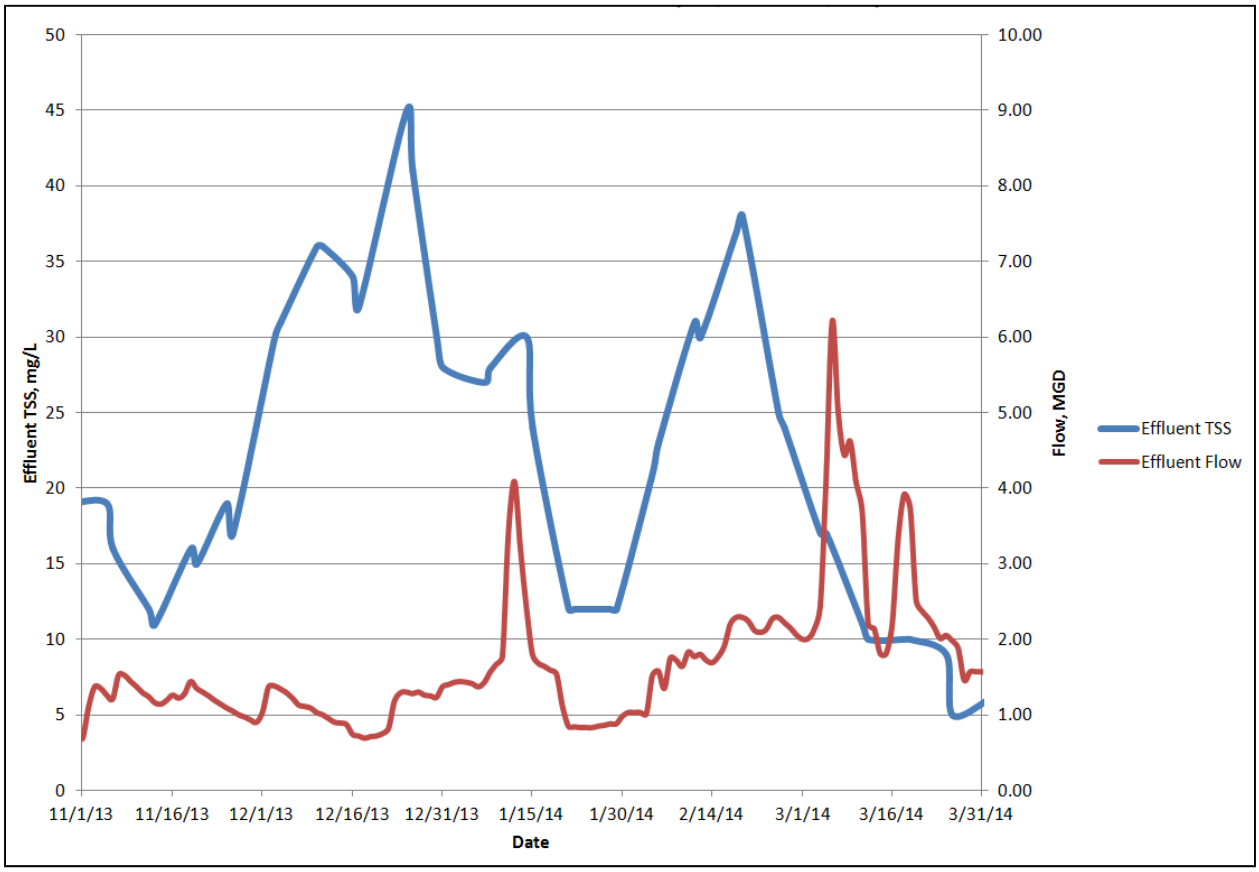


FIGURE 2-11: EFFLUENT TSS CONCENTRATION AND EFFLUENT TEMPERATURE



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.

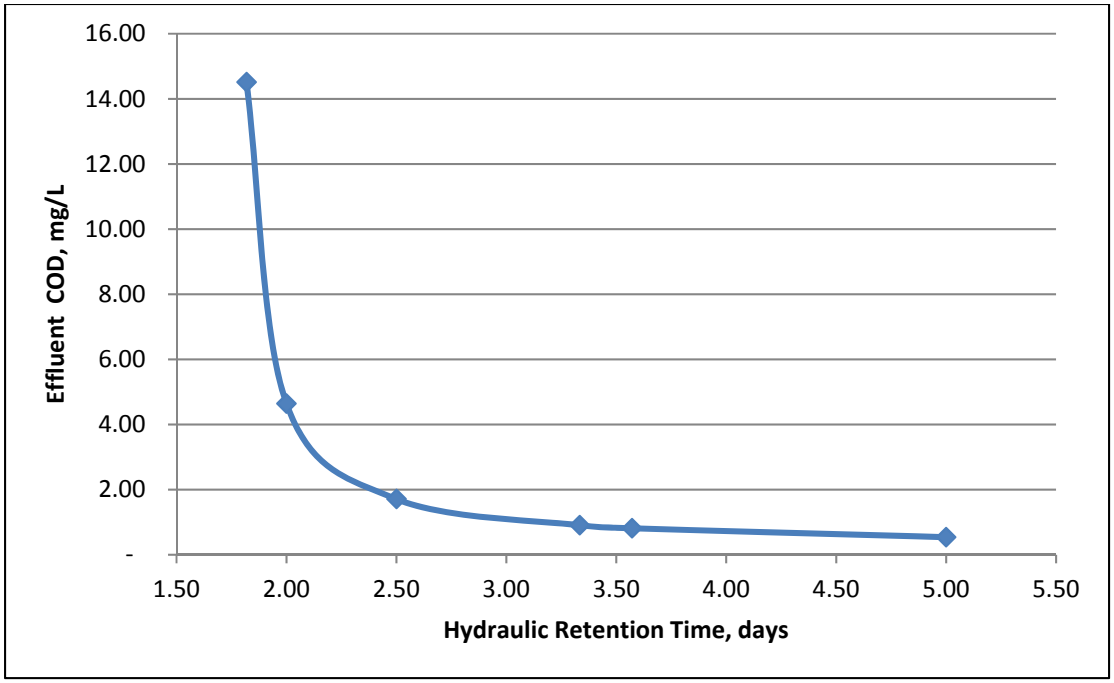
DPMC Aerated Lagoons

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	

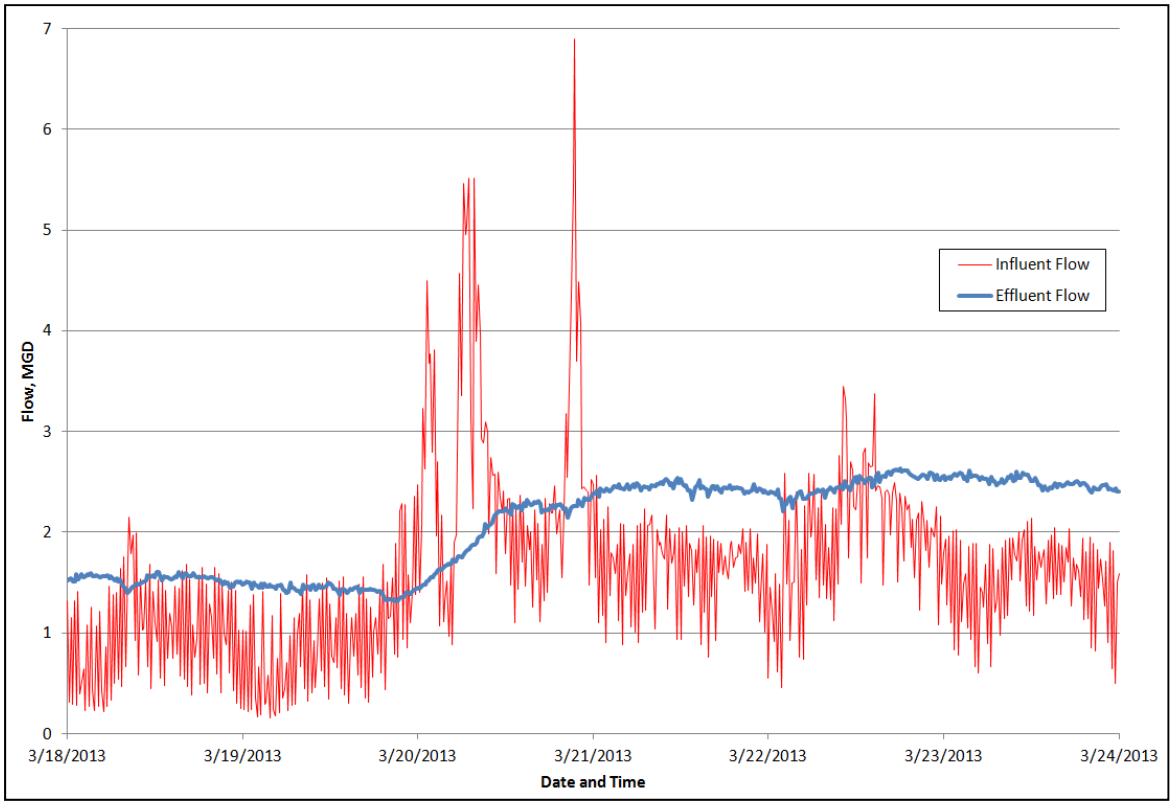
TABLE 2-3
WWTP HYDRAULIC CAPACITY SUMMARY

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

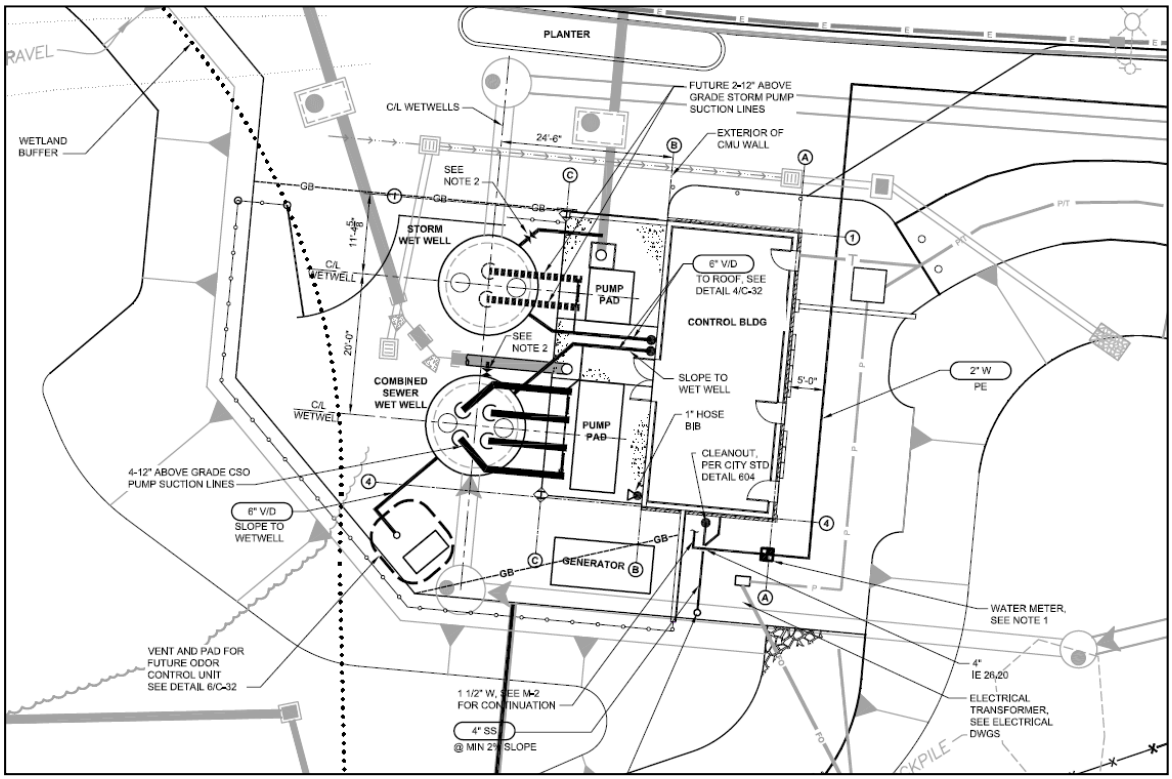
This Chapter describes the current configuration and operation of the CSO Pump Station, identifies relevant control elements of the pump station, identifies the associated storage volume and normal operating conditions of the pump station, and discusses how the storage volume and operating conditions 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

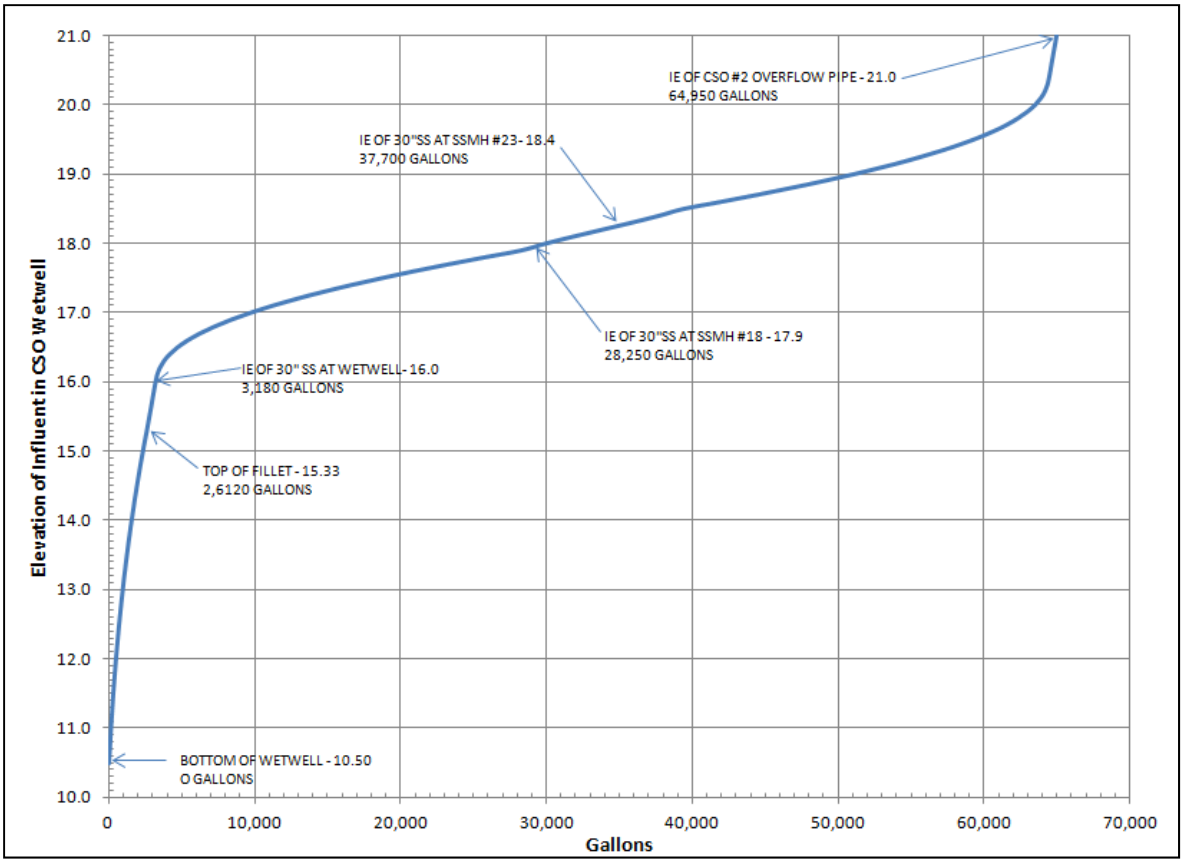
Control Set Point	Wet Well Depth	Elevation
CSO #2 Overflow	10.5 feet	21.0 feet
High-High Level Alarm and Backup Pump Start	8.0 feet	18.5 feet
High Level Alarm	7.6 feet	18.1 feet
Lag-Lag Pump Start	7.0 feet	17.5 feet
Lag Pump Start	6.5 feet	17.0 feet
Lag-Lag Pump Stop	6.0 feet	16.5 feet
Lead Pump Start	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

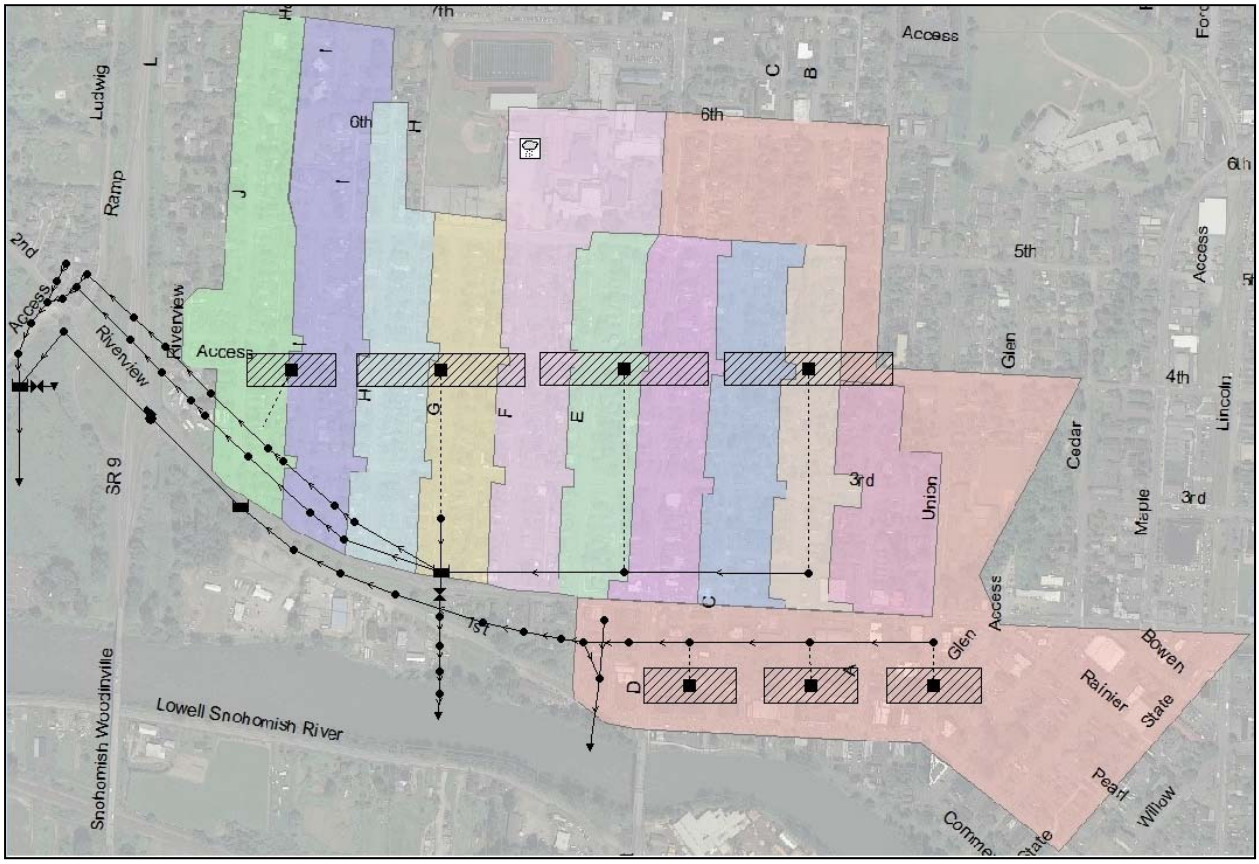
This Chapter provides an overview of model development, discusses model calibration, summarizes the modeling results based on simulating 27 years of historical rainfall and compares rainfall data used for model calibration with the original design storm.

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”.

Lenhart, J. (February 2007), “TSS vs. SSC”, Storm Water Solutions.

Rich, L. (1982), “Design Approach to Dual-Power Aerated Lagoons”, J. Environ. Eng., 108(3), pg. 532–548.

Rich, L. (1996), “Modification of Design Approach to Aerated Lagoons”, J. Environ. Eng., 122(2), pg. 149–153.

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Appendix L

Engineering and Construction Standards

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* **Revised February 2011 per City of Snohomish Ordinance No. 2211**

ENGINEERING DESIGN AND CONSTRUCTION STANDARDS

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DESIGN AND CONSTRUCTION STANDARDS
AND
SPECIFICATIONS

SECTION 1

GENERAL CONSIDERATIONS

1-0 STANDARDS

These City of Snohomish Design and Construction Standards and Specifications, hereinafter referred to as the "Standards", shall apply whenever any public or private work is performed within the street rights-of-way of the City of Snohomish including work performed by private parties at their own expense under authority granted by ordinance of the City Council or permit process. Except where these Standards provide otherwise, design, construction and materials shall conform to the appropriate standards of the current edition of the following publications produced separately by the Washington State Department of Transportation (WSDOT) or jointly by WSDOT and the Washington State Chapter of the American Public Workers Association (APWA).

A. WSDOT/APWA Standard Specifications for Road, Bridge and Municipal Construction, hereinafter referred to as the "WSDOT/APWA Standard Specifications".

B. WSDOT/APWA Standard Plans for Road, Bridge and Municipal Construction, hereinafter referred to as "WSDOT/APWA Standard Plans".

C. WSDOT Design Manual

D. WSDOT Construction Manual

1-1 SCOPE

These public works design and construction standards are intended to apply to all public improvements in City rights-of-ways and City easements and private improvements that impact the public system (see section 5.2 for City position on easements). Examples of this latter group include parking lots, on-site storm water control facilities and side sewers and water services from the meter to the user. Not included are such things as sidewalks and building projects not in the ROW (they may however still be controlled by the Uniform Building Code).

The content of each chapter is generally organized in an alphabetical manner except where it was helpful to group similar topics.

Prior to the awarding of any construction contract, the City Engineer may request a copy of the project bid documents for the purpose of determining if the bid specifications and project scope meet City Standards. The contract must contain a bid proposal form and special conditions section giving any information not found in the City Standards. Prior to any construction, the City Engineer may request a copy of the executed construction contract.

1-2 REFERENCES

These Standards are intended to be consistent with the most currently adopted provisions of the following:

A. Snohomish Municipal Code (SMC)
B. City of Snohomish Land Use Code (Title 14)
C. State of Washington Shoreline Management Act
D. State and National Environmental Policy Acts
E. City Design Standards-Washington State
F. International Building Code
G. International Electrical Code
H. Uniform Plumbing Code
I. International Mechanical Code
J. International Fire Code
K. WSDOT Design Manual
L. WSDOT Traffic Manual
M. WSDOT Utilities Manual
N. WSDOT Construction Manual
O. AWWA Standards
P. Manual on Uniform Traffic Control Devices (MUTCD)
Q. Washington State DOE Storm Water Management Manual (1992 version)
R. Washington State DOE Criteria for Sewage Design
S. Washington State Department of Health, Group A water systems

1-2.1 PRELIMINARY ENGINEERING REPORT

On large or complex projects, a preliminary engineering report and plan may be required by the City Engineer prior to preparation of final construction drawings. Generally a preliminary engineering report will be required when the proposed development is submitted to the City as a part of the requirement for pre application review (PAR) and demands a level of service higher than the current City infrastructure can provide. The report must identify and explain all potential complications with the proposed project and possible alternatives. Approval of these preliminary submittals shall constitute only a conceptual approval and shall not be construed as approval of specific design details.

1-2.2 QUALIFICATIONS OF PREPARERS OF ENGINEERING REPORTS, PLANS AND SPECIFICATIONS

Unless specified otherwise, all engineering plans and specifications prepared for the City or prepared for City approval shall be prepared by or under the direct supervision of a Professional Engineer registered and licensed to practice engineering in the State of Washington. All plans, specifications, and calculations must indicate this by being signed by the licensed engineer along with his or her registration number, seal, and expiration date.

1-3 AS-BUILT DRAWINGS

Prior to the acceptance of the work, the developer/contractor shall furnish the City Engineer one neatly and legibly marked set of reproducible, mylar drawings of significant permanent items showing any and all changes in the final locations of all items of work including, but not limited to finish elevations of building areas, curb and gutter, storm drain lines, water lines, sewer lines, catch basins, manholes, fire hydrants, valves, new and existing utilities and all other miscellaneous items included in the work. Marking of the drawings shall represent all changes, vertical and horizontal, and be done at the time the material and equipment is installed and inspected. These drawings shall be "wet stamped" and generated by the Project Engineer.

The City would require a computer file of the as-builts submitted in conjunction with the mylar. The computer file should be capable of being imported to AutoCAD™. Please refer to Section 1-10.1 of these Standards for information on plan format.

As-built drawings shall be required whether for private or public construction in accordance with the following:

Private Development

A. Plats - Final plat approval shall be withheld until after the as-builts have been submitted and approved.
B. Commercial - Final approval and installation of water meters will be withheld until the as-builts have been submitted and approved.

Public Construction

As-built drawings shall be considered an item on the contractor's punch list. Until all items on the punch list are completed, the project will not be sent to the City Council for approval. Final acceptance will be withheld until the as-built drawings are submitted and approved.

1-4 CITY PERFORMED WORK

When work is to be performed by the City for the developer, the city will provide all the material required for the said work. The cost for the material and the work performed shall be at the developer's expense. Any requests for City work should be scheduled at least one week in advance.

1-5 CONTROL OF NOISE

The city has established regulations for control of noise. For the purpose of regulating potentially disruptive or annoying noise, City of Snohomish Municipal Code Section 8.16.010 shall apply.

Under the City's Noise Ordinance, construction related noise has limitations during the hours before 7 a.m. and after 10 p.m. on weekdays and before 9 a.m. and after 10 p.m. on weekends and state recognized holidays. Due to citizen concerns about construction noise in neighborhoods, and given the city's intent to limit the occurrence of public disturbance noise, construction sites will be monitored and violators are subject to fines.

Construction contractors are responsible for notifying subcontractors of the City's noise regulations.

1-6 GUARANTEES

Performance and warranty guarantees will be required for all public works improvements, unless waived by the City Engineer. Work to be performed by any state agency or unit of local government shall be exempt from providing guarantees based on Chapter 35A.21.250 R.C.W.

Acceptable methods of guarantees will be as follows:

A. Bond
B. Assignment of Funds (Performance)
C. Set-aside Letter (Warranty)
D. Cash Deposit

Standard documents as approved by the city for the above items are available from the Public Works Department.

1-6.1 PERFORMANCE

Performance guarantees will be required for all improvements located in the public rights-of-way and as required by City ordinance as detailed on the approved plans and as noted in the following summary:

Street/Alley	Drainage (private)	Utilities (public)
Estimated cost plus 50%; construct improvements prior to building permit issuance; for a plat construct all improvements prior to final plat approval.	For properties one acre or more with drainage abatement facilities, estimated cost plus 50%.	Estimated cost plus 50%; construct improvements prior to occupancy; for a plat construct prior to final plat approval.

The initial guarantee and subsequent extensions as approved will be limited to one year increments. If time extensions are approved, the guarantee amount shall be revised to reflect inflation and/or other cost impacts.

The developer shall provide an estimate, prepared by a licensed engineer, of the improvements based on the approved plans. The estimate shall be itemized by description, quantities, and costs. The submitted data will be reviewed by public works for adequacy of quantities and comprehensiveness of estimates. The estimate shall be reviewed to reflect the City's cost to complete the improvements.

1-6.2 WARRANTY

Warranty guarantees shall be required at the time of final acceptance of the public improvements and/ or improvements required by City ordinance. The guarantee amount will be 15% of the documented final cost of the improvements in accordance with SMC 14.215.080. The warranty guarantee is required prior to release of the performance guarantee. Methods of posting warranty guarantee shall be the same as for performance guarantee and shall be for the lengths of time as listed below:

Street/Alley	Drainage (private)	Utilities (public)
One Year	One Year (will be extended for one year if City elects to assume maintenance)	One Year

1-7 PERMITS

A Right-of-Way Disturbance permit is required for all work within the public right-of-way and City utility easements, and for all sewer, water, and drainage improvements, including fill and excavation, parking lot construction, and/or paving on private property.

The approved applicant's copy of the public works permit, together with a set of plans approved by the City Engineer shall be available on the job site whenever work is being done on any portion of the project.

Any questions regarding information about permits and the approved prices should be directed to the Permitting Coordinator's Office at City Hall at 116 Union Avenue in Snohomish, WA 98290.

1-8 LEGAL RELATIONS AND RESPONSIBILITIES

The contractor, at all times, shall comply with all Federal and State laws, local laws and ordinances, and any regulations which in any manner affect the project.

The contractor shall release, indemnify and promise to defend and save harmless the City, its officers, employees, and agents from and against any and all liability, loss, damage, expense, actions and claims, including costs and reasonable attorneys fees incurred by the City in defense thereof, asserting or arising directly or indirectly on account of any violation of laws, ordinances, or regulations whether such violations are by the contractor, his subcontractors, his employees, or his agents. A Contractor's Bill of Sale (release of interest) shall be signed, notarized, and submitted to the City prior to the acceptance of all utilities and infrastructure to be dedicated to the City of Snohomish.

1-9 MODIFICATIONS OF STANDARDS

Modifications from these Standards may be granted by the City Engineer upon evidence that such modifications are in the public interest, that they are based upon sound engineering judgment, and that requirements for safety, function, appearance, and maintainability are fully met. Desired modifications must be applied for by Deviations from Standards form submittal and approved prior to construction.

1-10 PLAN REVIEW AND REQUIREMENTS

The Public Works Department has established basic standards for improvement plans so plan checking can be processed efficiently. See each section for specific requirements over and above the following standards. Plans must meet these standards before they will be accepted. Requirements for plans will be divided into two general categories.

A. Minor Projects: Work not requiring public right-of-way improvements and dedication. For example, small site projects such as a new house or duplex, garage addition, house addition, sidewalk, driveway, side sewer, or remodel.

B. Major Projects: Work involving street improvements, drainage, water, and sewer improvements. Plans for major projects must be drawn and stamped by a registered civil engineer licensed in the State of Washington.

1-10.1 FORMAT AND REQUIRED DATA

A. All public works plans for street improvements and utility systems shall be prepared in a Mylar plan/profile format either with sheets printed in half plan and half profile, or with separate sheets for plan view and profile views. Four (4) sets of copies shall be submitted for City staff review.

B. In addition to "hard copy" plan and profile sheets, it is required that all plans for major projects should be submitted in an AutoCAD™ compatible computer file format. Major projects are as defined in Section 1-10 B above or as otherwise identified by the City Engineer.

C. Plans and profiles shall be prepared by the Project Engineer with all utilities, both new and existing, shown on all sets of plans. For example, on the sanitary sewer sets, the water and storm drains shall be shown with the sanitary sewer portions including service lines being heavily highlighted. Invert elevations and rim elevations are required for all manholes. Benchmark elevations must be described and identified on the plan. Slope of sewers, storm lines, size and type of service lines, and inlets must be shown. Typical sections of open channels and ponding areas must be shown. Other utilities such as power, gas, cable, and telephone are also to be shown in profile views where crossings occur.

D. Whenever possible, in the "Construction Notes" on the drawings, use notes specifying City standard sections and drawing plate numbers for common items such as catch basins, restrictors, fire hydrant assemblies, etc. See drawing plate 102.

E. Show the existing channelization of all streets that front the proposed development. Show all curb cuts on both the adjacent properties and the properties across the streets that front on the proposed development.

F. Show complete data for curb radii, utility locations (new and existing), curb elevations, street stationing, street widths, existing adjacent improvements, elevations of existing street improvements, and utilities, etc.

G. Where required, all elevations and grades on public works plans shall be tied to the N.G.V.D 1929 Vertical Datum per City GIS and as set forth in the "Requirements For Vertical Datum On Land Development Projects Within Snohomish County " Policy 8310 Snohomish County Planning and Development Services, a copy of which is available at City Hall, 116 Union Avenue, Snohomish, WA 98290. Topography shall be shown using two foot contours. Ties to range, property or section lines including block and lot numbers shall be shown.

H. A summary of quantities for all work within the public right-of-way or in easements granted to the City shall be listed on the title sheet or on the first sheet of all plans or sets of plans. The following list can be used as a guideline for the items to be listed, but is to be supplemented as necessary for a particular project:

1. Asphalt Concrete Pavement (Roadway) S.Y.
2. Cement Concrete Pavement (Roadway) S.Y.

3.	Asphalt Concrete Pavement (Alley)	S.Y.
4.	Cement Concrete Pavement (Alley)	S.Y.
5.	Cement Concrete Curb and Gutter-Type "A-1"	L.F.
6.	Cement Concrete Sidewalk	S.F.
7.	Cement Concrete Driveway Approaches	S.F.
8.	Ductile Iron Water Main (Size)	L.F.
9.	Gate Valves (Size)	EA.
10.	Butterfly Valves (Size)	EA.
11.	Concrete or PVC Sewer Main (Size)	L.F.
12.	Concrete or PVC Side Sewer (Size)	L.F.
13.	Sewer Manholes (Type)	EA.
14.	Concrete Storm Drain Pipe (Size)	L.F.
15.	Catch Basins (Type)	EA.
16.	Traffic Regulatory Signs (Type)	EA.
17.	Street Name Signs	EA.
18.	Fire Hydrant Assemblies	EA.
19.	Water Main Blow-Offs	EA.
20.	Borrow Backfill	CY.

I. A "Driveway Schedule" which lists all of the driveways, both residential and commercial, being constructed and shall include the following information pertaining to each driveway, in tabular form:

1. Location of driveway
2. Surface type
3. Width
4. Profile grade (may require separate sketch)
5. Length

1-10.2 REQUIRED DRAWINGS

The following plans for public works improvements and utilities shall be prepared:

A. Erosion Control and Grading Plan
B. Street Improvements*****
C. Storm Drain or Drainage Plan (Drainage and Street Plans may be combined together)
D. Sanitary Sewer Plan
E. Water System Plan
F. Landscaping Plan

***** A separate cross section plan sheet is required for all street construction. Distance between stations shall be determined by the City Engineer based on site topography. Generally, cross sections shall be shown at 25 foot intervals for reconstruction and 50 foot intervals for new construction. Cross sections shall show the entire right of way width, with existing grade and features as well as improvements shown. For reconstruction and pavement widening, crown slopes shall be as directed by the City Engineer. In no case shall crown slopes be less than one percent nor more than four percent. The designed centerline shall be approved by the City Engineer.

For most minor projects, all of the above required sheets may be condensed into one plan sheet. If an extension or either a sewer main, water main, or storm drain main is required, a separate sheet will be required for that work.

1-10.3 SCALE OF DRAWINGS

All plans are to be drawn utilizing an engineer's scale.

The acceptable scale for public works improvement plans shall be 1"=40' for plan view (horizontal) and 1"=5' for profile view (vertical). A larger scale of 1"=20' for the plan view may be used on smaller projects and street improvements within the right-of-way when greater clarity can be attained.

1-10.4 SIZE OF DRAWINGS

A. Minor Projects: Plans must be drawn or printed on paper that is relatively heavy, such as blueprint quality or standard drafting paper. Plans drawn on tissue paper, poster board or cardboard will not be accepted. The minimum acceptable size is to be 8-1/2"x11", with the maximum acceptable size being 24"x36".

B. Major Projects: Plans must be drawn on polyester drafting film. Plans shall 24"x36" and shall be done in permanent ink.

1-10.5.1 TYPICAL NOTES

All construction drawing sets shall have a set of City of Snohomish standard construction notes as applicable to the project included on the appropriate sheets. These notes are available from the Engineering Division in various formats. See drawing plate 102 as an example of specifics.

1-11 PROTECTION OF PROPERTY AND UTILITIES

1-11.1 PROPERTY

The contractor shall protect and preserve from damage, interference, and destruction all private and public property on or in the vicinity of the work. If such property is damaged or destroyed or its use interfered with by the contractor or his agents, it shall be restored immediately to its former condition by the contractor at his expense and such interference terminated.

1-11.2 UTILITIES

The contractor shall protect from damage private and public utilities, including telephone and telegraph lines, power lines, sewer and water lines, railroad tracks and appurtenances, highway lighting and signal systems, and similar facilities. Before beginning any excavation, the contractor shall provide notice of commencement to all owners of underground facilities through the one number locator service, phone number 1-800-424-5555, if available; if not, he shall give notice to all individual utility owners. Such notice shall be not less than two or more than ten business days before the scheduled date of excavation.

1-12 SITE MAINTENANCE

The developer and contractor shall schedule and control his work so as to prevent all hazards to public safety, health, and welfare.

Streets shall be kept free of dirt and debris on a continuous basis.

Pedestrian facilities shall be kept free of obstruction, and continuity shall be maintained at all times unless otherwise approved by the Public Works Department.

On existing streets, two-way traffic shall be maintained at all times unless detour plans have been approved in advance by the City Traffic Engineer.

Pedestrian and vehicular access to occupied buildings shall be maintained at all times except where approval from the building owner has been obtained.

Stop Work Orders shall be posted where maintenance issues are neglected or ignored. Such orders will be left in place until the site is brought up to above standards.

1-13 TRAFFIC CONTROL

1-13.1 GENERAL Standard drawings for this section should be attached to this section

Traffic control for all projects shall comply with Chapter 6 of Manual on Uniform Traffic Control Devices (MUTCD). The contractor shall be responsible to furnish and maintain all required labor and materials as needed to the satisfaction of the City Engineer.

The contractor shall conduct his operations as to offer the least possible obstruction and inconvenience to the public, and he shall have under construction no greater length or amount of work than he can prosecute properly with regard to the rights of the public. He shall not open up sections of the work and leave them unfinished, but he shall finish the work as he goes insofar as practicable.

Unless otherwise approved in writing by the City Engineer, all public traffic shall be permitted to pass through the work with as little inconvenience and delay as possible. The contractor shall keep existing roads and streets adjacent to or within the limits of the project open to and maintained in a good and safe condition for traffic at all times. The contractor shall remove any deposits or debris and shall repair any damage resulting from his operations. Construction shall be conducted so as to cause as little inconvenience as possible to abutting property owners. Convenient access to driveways, houses, and buildings along the line of work shall be maintained.

1-13.2 DETOURS AND ROAD CLOSURES

Approval must be received from the City Traffic Engineer for all detours and road closures. A formal traffic control plan complying with the MUTCD shall be submitted to public works for review and approval by the traffic engineer prior to any work proceeding. A 72-hour notice shall be given to all emergency services and transportation agencies prior to closure.

1-13.3 FLAGGERS, BARRICADES, AND SIGNS

Flagger(s), barricades, and signs shall conform to the Standards established in the latest edition of the MUTCD. Standard Plan Numbers 103a through 103c show typical traffic control conditions. Standard Plan Number 104 shows typical traffic control devices. The contractor shall submit a traffic control plan prepared by a certified traffic Engineer showing the necessary construction signing, barricades, and flagger(s) required for the project to the public works for approval by the City Engineer 72 hours in advance of the time the signing and barricades will be required. All equipment and materials required for traffic control shall be furnished, installed, and maintained by the contractor to the satisfaction of the City Engineer.

1-13.4.1 CONSTRUCTION TRUCK ROUTES

The applicant shall submit a proposed route for review and approval by the City Engineer for all heavy hauling associated with any proposed construction activity. Routes shall consist of City of Snohomish arterials and adhere to City Truck Route SMC 12.12.170 to the greatest extent possible.

1-14 CONTROL AND INSPECTION

1-14.1 GENERAL

Work performed in construction or improvements within the City, whether by a private developer, a City contractor or City forces, shall be done in accordance with the approved plans and specifications and to the satisfaction of the City Engineer.

No work may be started until such plans are approved. Any revision to such plans shall be approved by the City Engineer prior to performance of the work.

The City Engineer will have authority to enforce these Standards as well as other referenced or pertinent specifications and will appoint project engineers, assistants, and inspectors as necessary to inspect the work for compliance.

1-14.2 MATERIALS SAMPLING & TESTING

1-14.2(1) DEVELOPMENTS

It shall be the responsibility of the developer to provide test reports certified by a professional engineer licensed in the State of Washington to verify compliance of materials used in the project (per WSDOT Standards). Sampling and/or testing shall be at a frequency and magnitude determined by the City Engineer in accordance with the WSDOT Construction Manual. Copies of all test reports shall be furnished to the City Engineer. All costs incurred for testing or sampling, as required, shall be the responsibility of the developer.

1-14.2(2) CITY FORCES & CITY CONTRACTORS

Construction work performed by City forces and City contractors shall be inspected by City inspectors. Sampling and testing shall be performed by City inspectors or by a professional laboratory.

1-14.2(3) NOTIFICATION OF INSPECTION

The developer shall notify the city of inspection needs in a timely manner. In general, a minimum of 24-hours advance notice will be required. Failure to notify in time may oblige the City to arrange appropriate sampling and testing after-the-fact, with certification by a qualified private testing laboratory. Costs of such testing and certification shall be borne by the developer.

1-15 ASBESTOS CONTROL

Asbestos containing material (ACM) may be encountered during a construction project in the form of asbestos cement pipe, pipe insulation, or as insulation in a structure that is being demolished. It can be found in pipe for water and sewer mains, electrical conduits, drainage pipe, and vent pipes, etc. Normal breakage and crushing of the material can cause an asbestos fiber release which presents a serious respiratory hazard. It is imperative that asbestos fiber release be controlled. Citations, by regulatory agencies, for an asbestos fiber release carry substantial fines.

Only employees certified by the State of Washington as a Certified Asbestos Worker may work on ACM during construction, demolition, repair, maintenance, renovation, salvage, or disposal of ACM.

The contractor shall have all asbestos removed from the site and property disposed of by a State licensed asbestos contractor in accordance with the practices specified by the State of Washington Department of Ecology, the Snohomish County Solid Waste Division, and all other pertinent State and Federal Regulations. See WAC 296-62-077.

1-16 LANDSCAPING

The development of landscaping and erosion control is to conform to the basic concepts and principles set forth in the City of Snohomish Land Use Code. Landscaping shall be required on all projects to provide visual orientation for traffic safety; to create physical delineation of parking areas and to furnish definition and scale of the entire complex by interval plantings and to insure the preservation of land values by creating an environmental quality which compliments the objectives of the respective land uses in any zone. A copy of the Land Use Code is available for review at the Permit Coordinator's office in City Hall, 116 Union Avenue in Snohomish, WA 98290.

DRAWING INDEX

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101	Sample Site Plan	4-01-04
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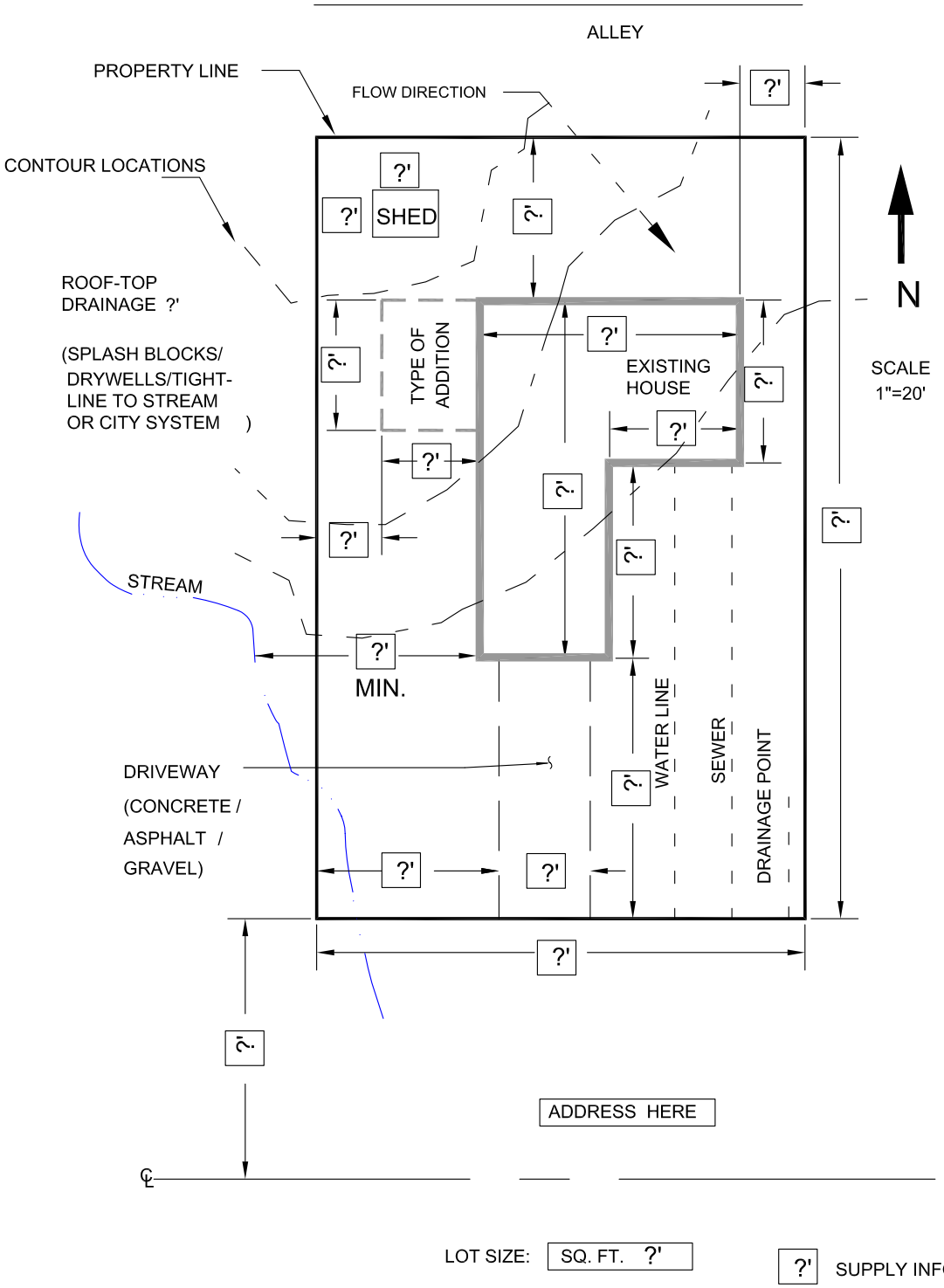
103c	Traffic Control Plan - 2 way left turn lane closure	4-01-04
104	Traffic Control Devices	4-01-04

PROVIDE LEGAL DESCRIPTION
AND TAX ID NUMBER



LOT SIZE: SQ. FT. ?'

?' SUPPLY INFORMATION

SNOHOMISH
WASHINGTON

SAMPLE SITE PLAN

Approved By:
City Engineer
Date: May 30, 2004

101
Number

City of Snohomish Public Works Department

CONSTRUCTION SEQUENCE

1. ATTEND PRE-CONSTRUCTION MEETING WITH THE CITY OF SNOHOMISH.
2. CALL 1-800-424-5555 FOR UNDERGROUND LOCATES
3. INSTALL TESC MEASURES, FLAG OR FENCE CLEARING LIMITS TESC MEASURES SHALL BE MAINTAINED UNTIL FINAL GRADING , PAVING AND HYDROSEEDING
4. DEMO EXISTING PAVEMENT AND PERFORM ROUGH GRADING AS SHOWN ON THE PLANS, INCLUDING OFFSITE DITCH IMPROVEMENTS. STABILIZE EXPOSED GROUND WITH SUITABLE GROUND COVER, PLACE ROCK CHECK DAMS IN DITCH.
5. INSTALL UTILITIES, INCLUDING STORM DRAINAGE, SANITARY SEWER, WATER, POWER, GAS, ETC.
6. PERFORM FINAL GRADING, PAVING AND SURFACING IMPROVEMENTS.
7. HYDROSEED AND MULCH ALL EXPOSED AREAS.
8. SWEEP AND CLEAN ALL PAVED AREAS, THEN CLEAN THE STORM SYSTEM.
9. REMOVE ALL CLEARING LIMITS AND TEMPORARY EROSION/SEDIMENTATION CONTROL FACILITIES ONLY AFTER EXPOSED AREAS ON SITE ARE STABILIZED.
10. CALL FOR FINAL INSPECTION OF FACILITIES.

GENERAL NOTES

1. ALL SITE GRADE WORK MUST COMPLY WITH CHAPTER 33 OF THE UNIFORM BUILDING CODE (1997 EDITION), CITY OF SNOHOMISH PUBLIC WORKS DESIGN AND CONSTRUCTION STANDARDS, AND/OR WSDOT/APWA STANDARDS AS SPECIFICATIONS APPLY.
2. IF CUT OR FILL SLOPES EXCEED A MAXIMUM OF TWO FEET HORIZONTAL TO ONE FOOT VERTICAL, A ROCK OR CONCRETE RETAINING WALL MAY BE REQUIRED. ALL ROCK RETAINING WALLS GREATER THAN FOUR (4) FEET IN HEIGHT ARE TO FOLLOW CITY SPECIFICATIONS AND TO BE DESIGNED AND CERTIFIED BY A CIVIL ENGINEER EXPERIENCED IN SOILS MECHANICS.
3. PUBLIC STREETS ARE TO BE KEPT CLEAR OF DIRT AND DEBRIS DURING EXCAVATIONS AND FILL OPERATIONS.
4. THE SITE SOILS ARE GENERALLY CONSIDERED MOISTURE SENSITIVE AND AS SUCH ARE SUSCEPTIBLE TO DISTURBANCE BY CONSTRUCTION EQUIPMENT. PARTICULARLY DURING PERIODS OF WET WEATHER. THE GRADING CONTRACTOR SHALL TAKE ALL PRECAUTIONS TO LIMIT SURFACE DISTURBANCE AND PROTECT THE SITE GRADING AREA FROM EXCESSIVE RUNOFF EROSION.
5. AREAS TO RECEIVE FILL SHALL BE CLEARED OF ALL VEGETATION AND DELETERIOUS MATTER.
6. AREAS TO RECEIVE FILL SHALL BE PROOFROLLED. ALL LOOSE AND SOFT AREAS SHALL BE REMOVED AND REPLACED WITH STRUCTURAL FILL. FILLS PLACED ON SLOPES SHALL BE BENCHED OR KEYED INTO THE EXISTING GROUND.
7. ALL FILL MATERIALS USED SHALL BE FREE OF VEGETATION AND DELETERIOUS MATTER AND SHALL NOT CONTAIN ROCKS GREATER THAN SIX INCHES IN DIAMETER.
8. STRUCTURAL FILLS SHALL BE PLACED IN MAXIMUM 8" TO 10" THICK LOOSE HORIZONTAL LIFTS AND SPREAD UNIFORMLY.
9. THE SURFACE OF ALL SLOPES SHALL BE COMPACTED BY THE END OF EACH WORK DAY.
10. STOCKPILES ARE TO BE LOCATED IN SAFE AREAS AND ADEQUATELY PROTECTED BY TEMPORARY SEEDING AND MULCHING. HYDROSEEDING IS PREFERRED.
11. PRIOR TO ANY SITE CONSTRUCTION TO INCLUDE CLEARING. LOGGING OR GRADING THE SITE/LOT CLEARING LIMITS SHALL BE LOCATED AND FIELD IDENTIFIED BY THE PROJECT SURVEYOR/ENGINEER AS REQUIRED BY THESE PLANS. THE PROJECT SURVEYOR/ENGINEER'S NAME AND TELEPHONE NUMBER ARE SHOWN ON THE PLANS.
12. THE TEMPORARY EROSION/SEDIMENTATION CONTROL FACILITY SHALL BE CONSTRUCTED PRIOR TO ANY GRADING OR EXTENSIVE LAND CLEARING IN ACCORDANCE WITH THE APPROVED TEMPORARY EROSION/SEDIMENTATION CONTROL PLAN. THESE FACILITIES MUST BE SATISFACTORILY MAINTAINED UNTIL CONSTRUCTION AND LANDSCAPING IS COMPLETED AND THE POTENTIAL FOR ON-SITE EROSION HAS PASSED.
13. IMMEDIATELY FOLLOWING FINISH GRADING, PERMANENT VEGETATION (CONSISTING OF RAPID, PERSISTENT AND LEGUME) WILL BE APPLIED (MINIMUM 80# PER ACRE). THIS IS TO INCLUDE THE FOLLOWING: 20% ANNUAL, PERENNIAL OR HYBRID RYE GRASS, 40% CREEPING RED FESCUE, 40% WHITE CLOVER.
14. FERTILIZER SHALL BE APPLIED AT 400# PER ACRE OF 10-20-20 (10# PER 1000 SQUARE FEET) OR EQUIVALENT.
15. A COPY OF THE APPROVED PLANS MUST BE ON-SITE WHENEVER CONSTRUCTION IS IN PROGRESS. IT SHALL BE THE RESPONSIBILITY OF THE PERMITTEE TO OBTAIN ANY OTHER RELATED OR REQUIRED PERMITS PRIOR TO BEGINNING OF CONSTRUCTION.
16. SUBMITTAL OF REPRODUCIBLE AS-BUILT PLANS IS REQUIRED PRIOR TO APPROVAL OF ANY CERTIFICATE OF OCCUPANCY. OR ACCEPTANCE OF CONSTRUCTION IMPROVEMENTS.
17. ALL LOCATIONS OF EXISTING UTILITIES SHOWN HEREON SHOULD BE CONSIDERED APPROXIMATE ONLY AND NOT NECESSARILY COMPLETE. IT IS THE SOLE RESPONSIBILITY OF THE CONTRACTOR TO INDEPENDENTLY VERIFY THE ACCURACY OF ALL UTILITY LOCATIONS SHOWN. AND TO FURTHER DISCOVER AND AVOID ANY OTHER UTILITIES NOT SHOWN HEREON WHICH MAY BE AFFECTED BY THE IMPLEMENTATION OF THIS PLAN.
18. THE CITY OF SNOHOMISH ENGINEERING DIVISION MUST BE NOTIFIED AT LEAST 24 HOURS PRIOR TO COMMENCING CONSTRUCTION.

T.E.S.C. NOTES

1. THE TEMPORARY EROSION CONTROL SYSTEM SHALL BE INSTALLED PRIOR TO ALL OTHER CONSTRUCTION.
2. WHERE POSSIBLE NATURAL VEGETATION WILL BE MAINTAINED FOR SILT CONTROL.
3. AS CONSTRUCTION PROGRESSES AND SEASONAL CONDITIONS DICTATE, THE EROSION CONTROL FACILITIES SHALL BE MAINTAINED AND/OR ALTERED AS REQUIRED BY THE CITY ENGINEER TO ENSURE CONTINUING EROSION/ SEDIMENTATION CONTROL.
4. TEMPORARY SILTATION PONDS AND ALL TEMPORARY SILTATION CONTROLS SHALL BE MAINTAINED IN A SATISFACTORY CONDITION UNTIL SUCH TIME THAT CONSTRUCTION IS COMPLETED. PERMANENT DRAINAGE FACILITIES ARE OPERATIONAL. VEGETATION HAS BEEN ESTABLISHED, AND THE POTENTIAL FOR FOR EROSION HAS PASSED.
5. ALL DISTURBED LAND AREAS THAT WILL BE LEFT FOR 30 DAYS OR MORE DURING THE PERIODS OF MARCH 1 TO MAY 15 AND AUGUST 15 TO OCTOBER 1 SHALL BE IMMEDIATELY SEEDED WITH A MIX AND BY A METHOD APPROVED BY THE CITY OF SNOHOMISH PUBLIC WORKS DIRECTOR AND MAINTAINED UNTIL SEED GERMINATION IS ASSURED. IN ADDITION TO SEEDING. SLOPES OF 25% OR GREATER WILL BE MULCHED WITH 2.0 TONS OF STRAW PER ACRE, OR WITH AN APPROVED EQUAL. STOCKPILED MATERIALS SHALL BE COVERED WITH AN IMPERMEABLE MEMBRANE AND SECURED TO HOLD IT IN PLACE. DURING THE REST OF THE YEAR, TEMPORARY SOIL STABILIZATION MUST BE APPLIED IMMEDIATELY TO DISTURBED AREAS THAT WILL BE EXPOSED FOR 7 DAYS OR MORE, AND IMMEDIATELY TO SLOPES GREATER THAN 25% THAT WILL BE LEFT EXPOSED FOR 3 DAYS OR MORE.
6. THE PUBLIC RIGHT-OF-WAY SHALL BE KEPT CLEAN. TRACKING OF MUD AND DEBRIS FROM THE SITE WILL NOT BE ALLOWED. FAILURE TO COMPLY WITH THIS CONDITION WILL RESULT IN ALL WORK ON THE SITE BEING STOPPED.
7. PROVIDE AND MAINTAIN TEMPORARY SEDIMENTATION COLLECTION FACILITIES TO ENSURE SEDIMENT OR OTHER HAZARDOUS MATERIALS DO NOT ENTER THE STORM DRAINAGE SYSTEM. AS CONSTRUCTION PROGRESSES AND UNEXPECTED SEASONAL CONDITIONS DICTATE. MORE EROSION CONTROL FACILITIES MAY BE REQUIRED TO ENSURE COMPLETE SILTATION CONTROL ON THE PROJECT. DURING THE COURSE OF CONSTRUCTION. IT SHALL BE THE OBLIGATION AND RESPONSIBILITY OF THE PERMITTEE OR CONTRACTOR TO ADDRESS ANY NEW CONDITIONS THAT MAY BE CREATED BY HIS ACTIVITIES AND TO PROVIDE ADDITIONAL FACILITIES. OVER AND ABOVE MINIMUM REQUIREMENTS, AS MAY BE NEEDED TO ENSURE COMPLIANCE.

STORM DRAINAGE NOTES

1. ALL WORK SHALL CONFORM TO THE CURRENT EDITION OF THE CITY OF SNOHOMISH PUBLIC WORKS DESIGN AND CONSTRUCTION STANDARDS.
2. A COPY OF THE APPROVED PLANS MUST BE ON-SITE WHENEVER CONSTRUCTION IS IN PROGRESS. IT SHALL BE THE RESPONSIBILITY OF THE PERMITTEE TO OBTAIN ANY OTHER RELATED OR REQUIRED PERMITS PRIOR TO BEGINNING THAT CONSTRUCTION.
3. SUBMITTAL OF REPRODUCIBLE AS-BUILT PLANS IS REQUIRED PRIOR TO APPROVAL OF ANY CERTIFICATE OF OCCUPANCY. OR ACCEPTANCE OF CONSTRUCTION IMPROVEMENTS.
4. ALL STRUCTURE LADDERS SHALL BE FIRMLY ATTACHED AND EXTEND TO THE BOTTOM OF THE STRUCTURE.
5. BEDDING MATERIAL SHALL BE PEA GRAVEL OR AS OTHERWISE APPROVED BY THE CITY ENGINEER. BEDDING SHALL BE A MINIMUM OF TWELVE (12) INCHES THICKNESS SURROUNDING THE PIPE TO SPRING LINE.
6. TRENCH BACKFILL SHALL BE DEPOSITED EVENLY ON BOTH SIDES OF THE PIPE IN TAMPED LAYERS NOT EXCEEDING SIX (6) INCHES IN DEPTH UNTIL THE TOP OF THE PIPE. ALL TRENCH BACKFILL SHALL BE COMPACTED TO A MINIMUM OF 95% OF MAXIMUM DRY DENSITY PER ASTM D1557-70.
7. NO PART OF THE DRAINAGE SYSTEM SHALL BE COVERED, CONCEALED OR PUT INTO USE UNTIL IT HAS BEEN TESTED. INSPECTED. AND ACCEPTED BY THE CITY.
8. ALL STORM DRAIN WORK MUST BE STAKED BY A LICENSED SURVEYOR FOR 'LINE AND GRADE' PRIOR TO STARTING CONSTRUCTION.
9. RIMS OF CATCH BASINS RECEIVING RUNOFF FROM GUTTERS SHALL BE FLUSH WITH GUTTERS.

TRAFFIC CONTROL NOTES

1. CONSTRUCTION SIGNS AND DELINEATORS SHALL BE PROVIDED AND MAINTAINED BY THE CONTRACTOR.
2. ALL FLAGGERS ARE TO BE EQUIPPED WITH ORANGE SAFETY VESTS AND HEAD GEAR APPROPRIATE FOR FLAGGING TRAFFIC WITHIN THE PUBLIC RIGHT-OF-WAY RIGHT-OF-WAY AND SHALL HAVE VALID FLAGGING CARDS FROM THE DEPARTMENT OF LABOR AND INDUSTRIES.
3. AFTER WORK WITHIN THE TRAVELLED ROADWAY IS COMPLETED AT THE END OF EACH DAY. THE ROAD SHALL BE COMPLETELY OPEN TO TRAFFIC AND ALL EQUIPMENT SHALL BE OUT OF THE TRAVELLED ROADWAY. ANY EQUIPMENT OR CONSTRUCTION MATERIALS LEFT WITHIN THE PUBLIC RIGHT-OF-WAY SHALL HAVE LIGHTED TYPE II BARRICADES AROUND THEM. ALL NONAPPLICABLE SIGNS SHALL BE REMOVED. TURNED OR COVERED DAILY WHEN CONDITION NO LONGER EXISTS.
4. ACCESS BY EMERGENCY VEHICLES SHALL BE MAINTAINED AT ALL TIMES.
5. CONTRACTOR SHALL SUPPLY THE CITY OF SNOHOMISH WITH A TRAFFIC CONTROL PLAN FOR THEIR APPROVAL.

SIGNING NOTES

1. ALL SIGNING SHALL CONFORM TO THE MOST RECENT VERSION OF THE FEET MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES.
2. SIGNS WHICH ARE WIDER THE 26 INCHES OR LARGER THAN 10 SQUARE SHOULD BE MOUNTED ON TWO POSTS.
3. WITHIN A CONSTRUCTION ZONE THERE MAY BE REGULATORY, WARNING, AND GUIDE SIGNS. WARNING SIGNS IN A CONSTRUCTION ZONE WILL BE ORANGE WITH BLACK CHARACTERS.
4. ALL SIGNS INTENDED FOR NIGHT USE SHOULD EITHER BE REFLECTORIZED OR ILLUMINATED. FOLLOWING IS A LIST OF SIGN NUMBERS AND MESSAGES. FOR A COMPLETE LIST OF SIGN MESSAGES AND NUMBERS REFER TO THE MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES (MUTCD), PART IV. TRAFFIC CONTROLS FOR STREET AND HIGHWAY MAINTENANCE.

SANITARY SEWER NOTES:

1. SANITARY SEWER PIPE TO BE PVC CONFORMING WITH ASTM D-3034, SDR 35. PIPE BEDDING TO BE CLASS B WITH ONE FOOT OF SAND OR PEA GRAVEL ABOVE PIPE.
2. 18" VERTICAL SEPARATION BETWEEN STANDARD WATER AND SEWER LINES IS REQUIRED. WHENEVER SEPARATION IS FROM 12" TO 17" , C- 900 PVC OR CL. 52 DUCTILE IRON SEWER PIPE MUST BE UTILIZED.
3. ALL SIDE SEWERS SHALL BE MINIMUM 6" DIAMETER PIPE AT A MINIMUM 2% SLOPE. SEWER STUBS SHALL BE EXTENDED 10 FEET INTO PROPERTY.
4. ALL SIDE SEWERS SHALL BE CAPPED WITH A WATERTIGHT PLUG AND SHALL BE MARKED FOR LOCATION WITH A 2"X4" BOARD, PAINTED WHITE, MARKED "SEWER" AND NOTING THE DEPTH OF THE STUB. 20 GAUGE LOCATOR WIRE SHALL BE USED AND EXTEND TO TOP OF MARKER BOARD.
5. ALL TESTING AND CONNECTIONS TO EXISTING MAINS SHALL BE DONE IN THE PRESENCE OF A REPRESENTATIVE FROM THE CITY OF SNOHOMISH.
6. ALL TRENCHES SHALL BE COMPACTED PRIOR TO TESTING SEWER LINES FOR ACCEPTANCE.
7. SIDE SEWERS SHALL BE PRESSURE TESTED AT THE SAME TIME THE MAIN LINE IS TESTED.
8. ALL SEWER MAIN EXTENSION WITHIN THE PUBLIC RIGHT-OF-WAY OR IN EASEMENTS MUST BE STAKED BY SURVEYOR FOR LINE AND GRADE PRIOR TO STARTING CONSTRUCTION.
9. THE CONTRACTOR SHALL MAINTAIN 10 FEET HORIZONTAL SEPARATION BETWEEN WATER AND SEWER MAIN LINES.
10. MINIMUM COVER OVER SEWER LINES IS FIVE FEET UNLESS APPROVED BY CITY ENGINEER.
11. WHEN REQUIRED, THE PROJECT ENGINEER SHALL PREPARE A WSDOE "DECLARATION OF CONSTRUCTION" FOR SUBMITTAL TO THE DEPARTMENT OF ECOLOGY WHEN ALL CONSTRUCTION IS COMPLETE.
12. ALL SEWER PIPE SHALL BE PLACED ON STABLE EARTH, OR IF IN THE OPINION OF THE CITY INSPECTOR, THE EXISTING FOUNDATION IS UNSATISFACTORY, THEN IT SHALL BE EXCAVATED BELOW GRADE AND BACKFILLED IN ACCORDANCE WITH THE STANDARD SPECIFICATIONS. PIPE SHALL NOT BE PLACED ON SOD, FROZEN EARTH OR LARGE BOULDERS OR ROCK. SEWER BEDDING SHALL BE CONSTRUCTED TO SMOOTH PROPER GRADE AND SHALL PROVIDE UNYIELDING CONTINUOUS SUPPORT FOR THE PIPE.

WATER NOTES:

1. ALL FIRE HYDRANTS SHALL BE MUELLER CENTURION A-423 929. THE BODY IS PAINTED WITH TWO COATS OF FAR WEST YELLOW HYDRANT PAINT. INSTALL A BLUE REFLECTOR ONE FOOT OFF CENTERLINE TOWARDS HYDRANT. A MINIMUM CLEARANCE OF 3' SHALL BE MAINTAINED FROM ANY OBSTRUCTIONS. ALL FIRE HYDRANTS INSTALLED IN UNPAVED AREAS SHALL HAVE A 3'X3'X4" CONCRETE PAD PLACED AROUND THE BASE.
2. THE WATERMAIN SHALL BE INSTALLED ONLY AFTER THE ROADWAY SUBGRADE IS BACKFILLED, GRADED AND COMPACTED IN CUT AND FILL AREAS. THE ROAD CENTERLINE SHALL BE STAKED ALONG WITH THE PROPERTY LINES.
3. ALL WATER VALVES INSTALLED IN UNPAVED AREAS SHALL HAVE A 2' X 2' X 4" CONCRETE PAD PLACED AROUND THE VALVE BOX.
4. WHERE SHACKLE RESTRAINT IS REQUIRED, TIE ROD SHALL BE 3/4" DIA., TIE BOLTS SHALL BE 3/4" STAR NATIONAL PRODUCTS NO. SST7 OR EQUAL, RETAINER CLAMP ASSEMBLIES SHALL BE STAR NATIONAL PRODUCTS NO. S11 OR EQUAL. ALL PARTS ARE TO BE CAD- PLATED.
5. ALL FITTINGS SHALL BE BLOCKED UNLESS OTHERWISE SPECIFIED.
6. DEFLECT THE WATERMAIN ABOVE OR BELOW EXISTING UTILITIES AS REQUIRED TO MAINTAIN 3 FT. MINIMUM COVER AND 18 IN. VERTICAL CLEARANCE BETWEEN UTILITIES UNLESS OTHERWISE SPECIFIED. THE WATERMAIN SHALL NOT BE JOINTED WITHIN 8' OF ALL CROSSINGS WITH LESS THAN 18" OF VERTICAL CLEARANCE. ALL CLEARANCES 8" AND LESS SHALL BE SEPARATED BY HDPE INERT FOAM BLOCKING OR APPROVED EQUAL.
7. AFTER DISINFECTING THE WATERMAIN, DISPOSE OF CHLORINATED WATER BY DISCHARGING TO THE NEAREST OPERATING SANITARY SEWER.
8. ALL PIPE AND FITTINGS NOT DISINFECTED IN PLACE SHALL BE SWABBED WITH 200 PPM CHLORINE SOLUTION PRIOR TO INSTALLATION.
9. IF THE INITIAL TAP OR EXTENSION REQUIRES SHUTTING DOWN AN EXISTING WATER MAIN, THE CITY MAY REQUIRE TWO WEEKS NOTIFICATION IN ORDER TO PLACE A PUBLIC NOTICE IN THE LOCAL PAPER. NO TAPS WILL BE ALLOWED ON MONDAY, FRIDAY, OR WEEKENDS. THE SNOHOMISH PUBLIC WORKS DEPARTMENT SHALL OPERATE ALL WATER VALVES IN RIGHT-OF-WAY.
10. CAST IRON FITTINGS SHALL BE USED IN ALL CASES WHERE THE DEFLECTION AT THE PIPE JOINTS EXCEEDS THREE DEGREES EXCEPT ON SWEEP RADIUSES. THE INSPECTOR MAY ALLOW THE CONTRACTOR TO INSTALL NORMAL OR SHORT LENGTHS OF PIPE, PROVIDING THE MAXIMUM DEFLECTION PER JOINT SHALL NOT EXCEED THREE-FOURTHS THE MAXIMUM PERMISSIBLE DEFLECTION AS OUTLINED IN THE PIPE MANUFACTURER'S DATA. DEFLECTION OF JOINTS OF PVC PIPE IS NOT ALLOWED.
11. THE CONTRACTOR SHALL FLUSH, TEST, AND CHLORINATE THE LINES IN THE PRESENCE OF THE UTILITIES INSPECTOR. LINES SHALL BE PRESSURE TESTED AT 150 PSI PLUS EXISTING LINE PRESSURE (MIN 200 PSI) FOR 15 MINUTES. WATER SAMPLES WILL BE TAKEN BY THE PUBLIC WORKS DEPARTMENT. SERVICE WILL NOT BE AVAILABLE UNTIL THE TESTS ARE RETURNED SATISFACTORY.
12. WATER MAINS SHALL BE CLASS 52 DUCTILE IRON PIPE.
13. ALL FITTINGS AND MATERIALS REMOVED FROM EXISTING MAINS SHALL BE RETURNED TO THE UTILITIES DEPARTMENT.
14. FIRE HYDRANTS SHALL STAND PLUMB, BE SET TO THE ESTABLISHED GRADE WITH THE LOWEST OUTLET OF THE HYDRANT NO LESS THAN EIGHTEEN INCHES ABOVE GRADE.
15. ALL WET TAPS SHALL BE DONE BY AN APPROVED PRIVATE CONTRACTOR.
16. ALL GATE VALVES SHALL BE RESILIENT SEATED.
17. NO CONNECTION TO THE EXISTING MAINS WILL BE ALLOWED EXCEPT BY MEANS OF AN APPROVED BACKFLOW PREVENTION DEVICE PRIOR TO SATISFACTORY FLUSHING, TESTING, DISINFECTION, AND RECIEPT OF SATISFACTORY BACTERIOLOGICAL TEST RESULTS.
18. MAXIMUM DEPTH OF THE WATERLINE SHALL BE 5 FEET.

SNOHOMISH
WASHINGTON

TYPICAL CONSTRUCTION NOTES (AS APPLIED BY PROJECT)

Approved By:
City Engineer
Date: May 30, 2004

102
Number

City of Snohomish Public Works Department

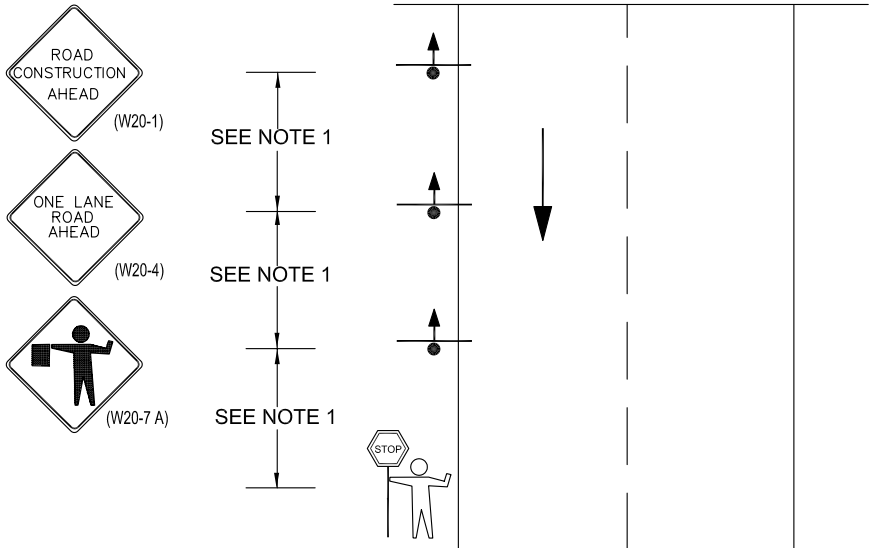


TABLE A

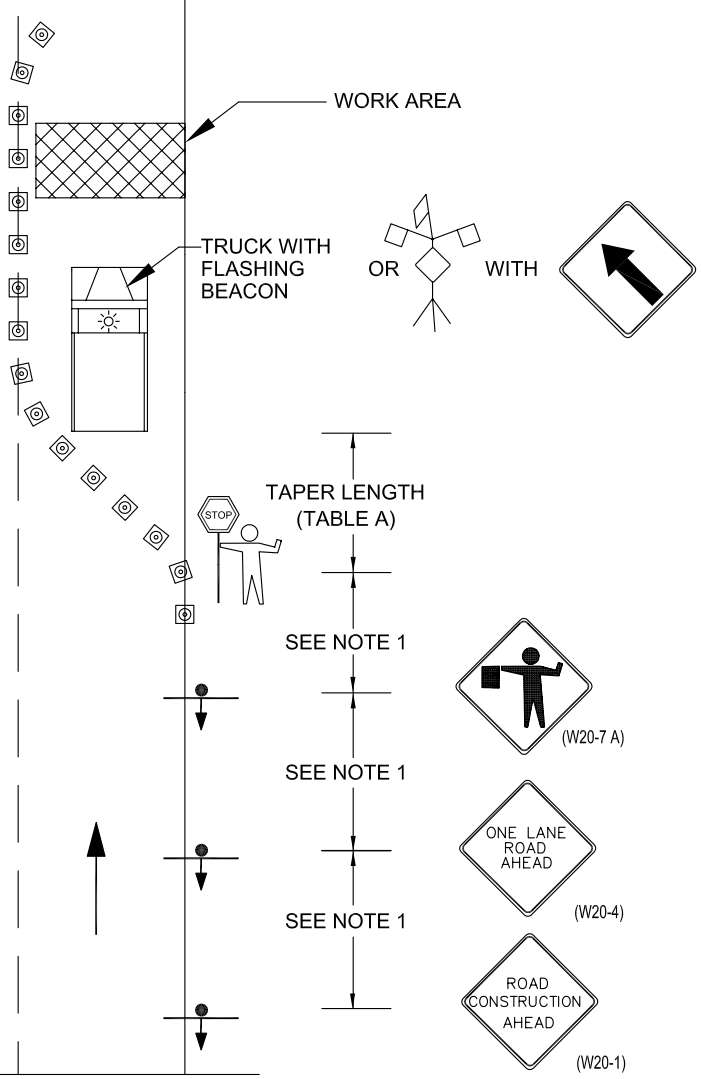
SPEED (MPH)	TAPER LENGTH (FT) OFFSET WIDTH		CONE SPACING ALONG TAPER (FT)
	10'	12'	
25	105	125	25
30	150	180	30
35	205	245	35
40	270	320	40
45	450	540	45

NOTE:

1. DISTANCE BETWEEN SIGNS SHALL BE 100' FOR RESIDENTIAL STREETS (25 MPH), AND 200' FOR ARTERIAL ROADWAYS
2. FLASHING BEACON SHALL BE INSTALLED AT EACH SIGN FOR NIGHT-TIME USE.
3. DISTANCES MAY VARY AS APPROVED BY THE ENGINEER.
4. FLAGGERS REQUIRED TO CONTROL TRAFFIC WHENEVER THE CONTRACTOR MUST INTERRUPT TRAFFIC FLOW TO ACCESS THE WORK SITE WITH MATERIALS OR EQUIPMENT.
5. SIGN SIZE PER MUTCD.

LEGEND:

◎ CONE OR CHANNELIZING DEVICE (SEE STANDARD DRAWING 104)



SNOHOMISH WASHINGTON

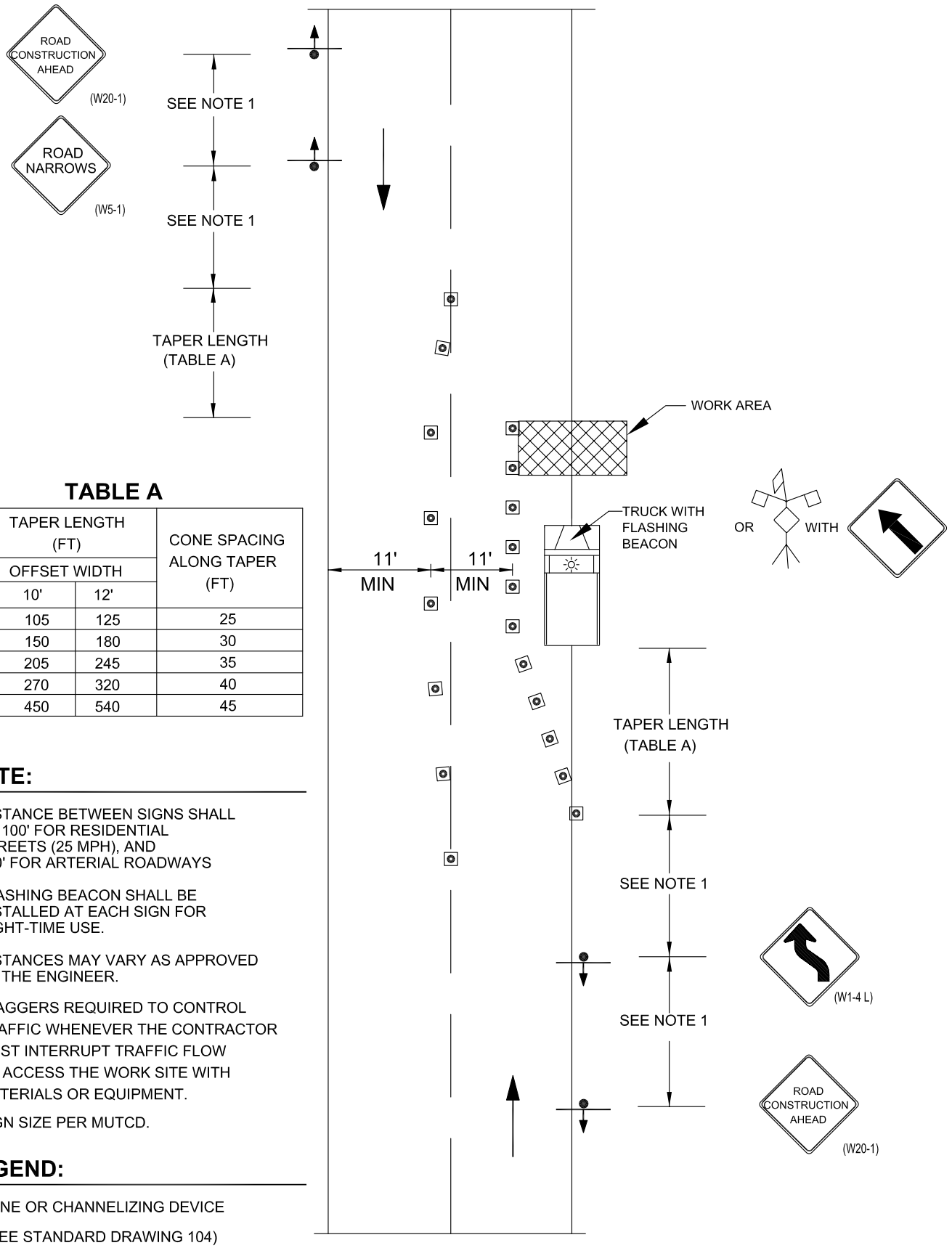
TRAFFIC CONTROL PLAN

2 LANE ROADWAY: ONE LANE CLOSED WITH ALTERNATING ONE-WAY TRAFFIC AND FLAGGERS

Approved By:
City Engineer
Date: May 30, 2004

103a
Number

City of Snohomish Public Works Department



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	OFFSET WIDTH		
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2. FLASHING BEACON SHALL BE INSTALLED AT EACH SIGN FOR NIGHT-TIME USE.
3. DISTANCES MAY VARY AS APPROVED BY THE ENGINEER.
4. FLAGGERS REQUIRED TO CONTROL TRAFFIC WHENEVER THE CONTRACTOR MUST INTERRUPT TRAFFIC FLOW TO ACCESS THE WORK SITE WITH MATERIALS OR EQUIPMENT.
5. SIGN SIZE PER MUTCD.

LEGEND:

CONE OR CHANNELIZING DEVICE

(SEE STANDARD DRAWING 104)

SNOHOMISH WASHINGTON

TRAFFIC CONTROL PLAN

2 LANE ROADWAY: PARTIAL LANE CLOSURE

Approved By:

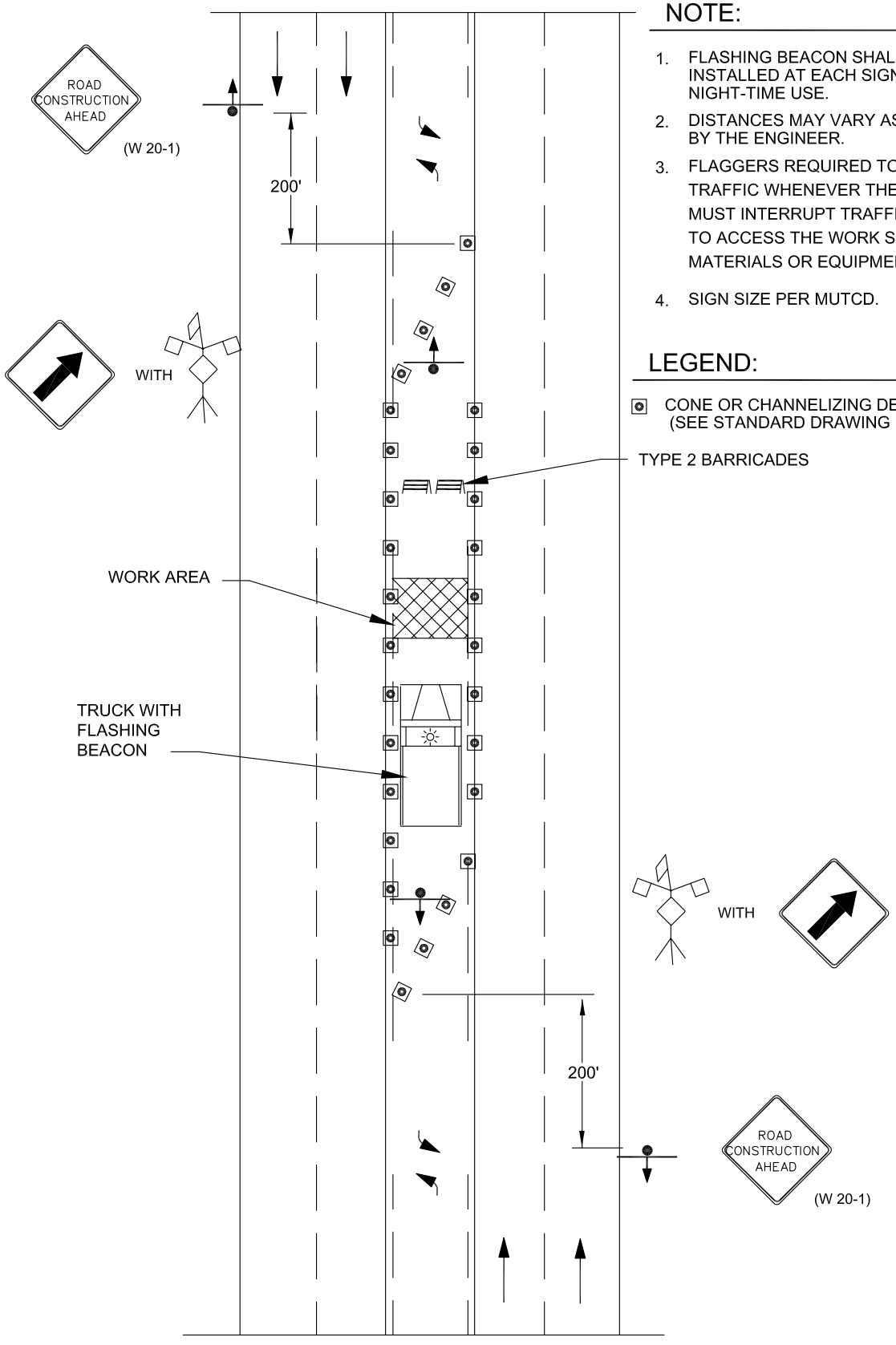
City Engineer

Date: May 30, 2004

103b

Number

City of Snohomish Public Works Department



NOTE:

1. FLASHING BEACON SHALL BE INSTALLED AT EACH SIGN FOR NIGHT-TIME USE.
2. DISTANCES MAY VARY AS APPROVED BY THE ENGINEER.
3. FLAGGERS REQUIRED TO CONTROL TRAFFIC WHENEVER THE CONTRACTOR MUST INTERRUPT TRAFFIC FLOW TO ACCESS THE WORK SITE WITH MATERIALS OR EQUIPMENT.
4. SIGN SIZE PER MUTCD.

LEGEND:

CONE OR CHANNELIZING DEVICE (SEE STANDARD DRAWING 104)

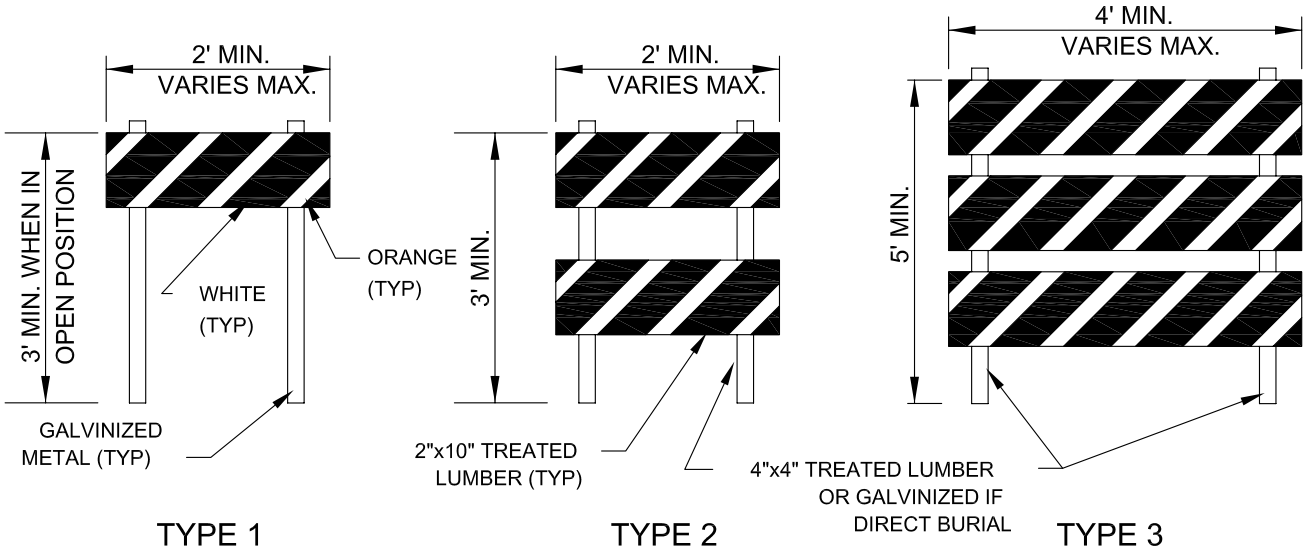
TRAFFIC CONTROL PLAN
TWO WAY LEFT TURN LANE CLOSURE

Approved By:
City Engineer
Date: May 30, 2004

103c
Number

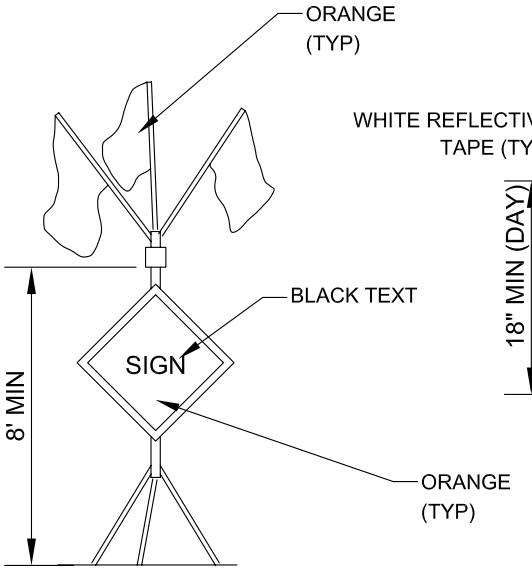
SNOHOMISH WASHINGTON

City of Snohomish Public Works Department

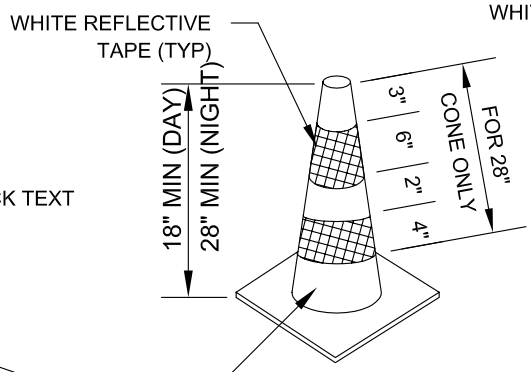


TYPE 1 TYPE 2 TYPE 3

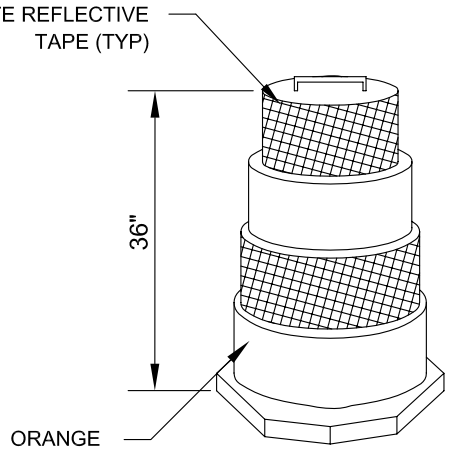
BARRICADE



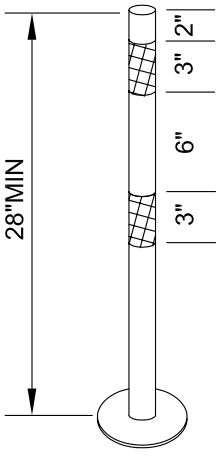
HIGH RISE
WARNING



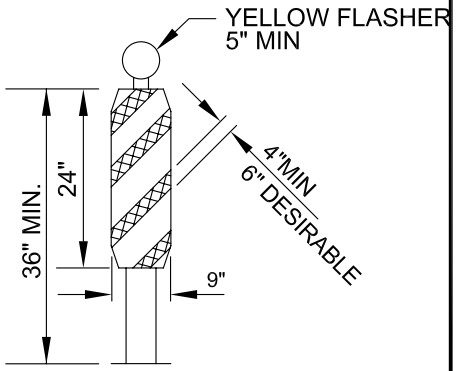
CONE



CHANNELIZING DRUM



GUIDE POST



VERTICAL PANEL

SNOHOMISH
WASHINGTON

TRAFFIC CONTROL DEVICES

Approved By:
City Engineer
Date: May 30, 2004

104
Number

City of Snohomish Public Works Department

ENGINEERING DESIGN AND CONSTRUCTION STANDARDS

TABLE OF CONTENTS

SECTION 2

EROSION AND SEDIMENT CONTROL

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DESIGN AND CONSTRUCTION STANDARDS
AND
SPECIFICATIONS

SECTION 2

EROSION AND SEDIMENTATION CONTROL

2-1 GENERAL

This section of the standards covers erosion and sediment control for small parcels, where land alteration activities will disturb less than one acre of land. Projects which will disturb one acre of land or more must refer to the most current DOE Storm water Management Manual for large parcel erosion and sediment control requirements.

Land alteration activities are those activities which are commonly referred to as clearing (the act of vegetation removal from the land surface, often referred to as land clearing); grubbing (the act of root vegetation removal from beneath the surface of the earth, usually in conjunction with clearing); excavation (the mechanical removal of earth material); filling (deposition of earth material placed by artificial means); grading (excavation of filling or combination thereof); and stockpiling (temporary deposition of earth material placed by artificial means).

2-1.1 SCOPE, DEFINITIONS AND AUTHORITY

The basis for the following standards is the Washington State Department of Ecology Best Management Practices and the Puget Sound Water Quality Management Plan. The purpose of this section of the standards is to manage water quality for the preservation of aquatic life and to protect property from the damaging effects of erosion and excessive water. Therefore, all land alteration activities within the City that have the potential for damaging off-site property or degrading water quality shall conform to the requirements of the Snohomish City Codes and these Standards.

Types of Land Alteration Regulated:

A. Clearing: the act of vegetation removal from the land surface, often referred to as land clearing (list the threshold for each);
B. Grubbing: the act of root vegetation removal from beneath the surface of the earth, usually in conjunction with clearing;
C. Grading: the mechanical removal and/or deposition of earth material; and
D. Stockpiling: temporary deposition of earth material placed by artificial means.

No land alteration, as defined above, shall occur without prior approval of an erosion and sediment control plan by the City Engineer. All clearing and grading permit applications shall be administered under Chapter 33 of the Uniform Building Code as provided in the 1997 edition
This chapter should always be reviewed in conjunction with Chapter Three: Storm and Surface Water Drainage.

2-2 GENERAL REQUIREMENT

Of primary concern are areas in:

A. Critical drainage areas;
B. Steep Slopes;
C. Sensitive soil conditions; and
D. Areas adjacent to waterways or in waterways.

These areas will require special attention in order for development to be considered, if at all. For sites not located in a critical drainage area, but generally larger than one acre, erosion control requirements will include sedimentation ponds, check dams, filter fabric fences, jute matting or plastic sheeting, and a temporary construction entrance as well as other controls specified by the City Engineer or recommended by the geotechnical/soil survey of the project site.

Developments located on steep slopes (25% or greater) shall require, in addition to the normal requirements for a comparably sized development on a slope less than 25%, the following: gradient terraces, immediate hydro-seeding and mulching, phasing of construction so as to minimize the amount of exposed soils at any given time and seasonal limitation restrictions. See Section 2.4 and 2.5. Depending upon soil and slope conditions, the proximity to a waterway and the size of the site, additional requirements may include flexible down drains, filter fabric fences, jute matting, or other approved means to secure seed and mulch in place, temporary level spreaders, and stabilized channels to convey off-site runoff through or around site.

Development adjacent to waterways may require, in addition to the normal requirements for a comparable sized development not adjacent to a waterway: a vegetative buffer between the waterway and the development, and seasonal limitation restrictions. See Section 2.4 and 2.5. Any work within the waterway will require a hydraulics permit from the Department of Fisheries and/or Game. The City Engineer may make additional requirements, including but not limited to temporary bypass culverts or channels.

After clearing and grading operations are completed, the storm drainage system has been installed, and building construction is underway, the sedimentation pond and interceptor ditches may be abandoned provided sediment traps are provided around all storm drain inlets and there is not possibility of sediment-laden water crossing property lines.

Construction equipment should only be cleaned at the contractor's yard, where methods presented in the Associated General Contractors water quality manual can be followed.

If a S.E.P.A review is required the exact combination of required erosion control measures will be clarified.

2-3 TEMPORARY EROSION/SEDIMENT CONTROL PLANS

A temporary erosion/sedimentation plan is required for the following land alteration activities: greater than 5,000 sq. ft.; on slopes 25% or greater; where cut and/or fill slopes 25% or greater will be created by the proposed work; or where work done may impact on environmentally sensitive area (stream, wetland, etc.). The plan shall clearly indicate the construction sequence for establishment of all erosion control work both temporary and permanent and shall be on a separate sheet.

2-3.1 EMERGENCY MANAGEMENT PLANS

Emergency Management Plans are to be submitted for all clearing and grading permit applications. To include at a minimum:

A. Name, address, and 24-hour telephone number(s) for the person(s) responsible for regular observation and repair or replacement of all erosion and sedimentation control measures.
B. Schedule for regular inspection, maintenance, and replacement of erosion and sedimentation control measures.
C. Location and inventory of materials required to be stockpiled on the site for emergency repair of the approved erosion and sedimentation control system.
D. Contingency plans for the use (of the above) in case of failure of the erosion and sedimentation control system, including how individual erosion control measures would be accessed during undesirable site conditions.

2-3.2 REQUIRED NOTES ON PLANS

The following are the minimum notes required on all Temporary Erosion/Sedimentation Control Plans*:*

A. The temporary erosion control system shall be installed prior to all other construction.

B. Where possible natural vegetation will be maintained for silt control.

C. As construction progresses and seasonal conditions dictate, the erosion control facilities shall be maintained and/or altered as required by the City Engineer to ensure continuing erosion/ sedimentation control.

D. Temporary siltation ponds and all temporary siltation controls shall be maintained in a satisfactory condition until such time that clearing and/or construction is implemented, permanent drainage facilities are operational, and the potential for erosion has passed.

E. All disturbed land areas that will be left for 30 days or more during the periods of March 1 to May 15 and August 15 to October 1 shall be immediately seeded with a mix and by a method approved by the City of Snohomish Public Works Department and maintained until seed germination is assured. In addition to seeding, slopes of 25% or greater will be mulched with 2.0 tons of straw per acre, or with an approved equal. During the rest of the year, temporary soil stabilization must be applied immediately to disturbed areas that will be left exposed for seven days or more, and immediately to slopes greater than 25% that will be left exposed for three days or more.

F) Approval of this plan does not constitute an approval of design, size, nor location of pipes, restrictors, or detention facilities; but is an approval of grading and sedimentation control plan only.

G) The public right-of-way shall be kept clean. Tracking of mud and debris from the site will not be allowed. Failure to comply with this condition will result in all work on the site being stopped.

2-4 SEASONAL AND WEATHER LIMITATIONS

Land alteration operations are restricted to seasonal limitations. The restrictions are site specific and are based on, but not limited to, steepness of slopes on site, distance from sediment/erosion sensitive areas, soil type, etc. These operations shall be governed by the following seasonal limitations:

A. No fill material shall be placed, spread, or rolled while either the fill material or the site surface is frozen or thawing, or during other unfavorable conditions.

B. All land alteration work is subject to stoppage by the City Engineer or his designee due to heavy rain. In these cases operations shall not be resumed until the City Engineer determines that erosion control facilities are operating satisfactorily.

C. Underground utilities and foundation installation are allowed with seasonal limitations under the following conditions:

1. Every effort is made to close utility trenches by the end of the day.

2. If unavoidable circumstances result in the inability to close a utility trench, all open areas and side spoils are covered with plastic sheeting that is staked in place or anchored by rocks, sand bags, tires, or by other approved methods at the end of the day.

3. Discharge from dewatering of utility trenches or foundation areas is directed to the nearest sedimentation pond, or to a specially created sump area, in a non-erosive fashion. Note: Large quantities of silt in the discharge water may result in the dewatering activity being stopped by the City Inspector until the source of the sediment is identified and attempts made to minimize the quantity in the discharge.

4. Utility corridors are re-stabilized by temporary soil stabilization measures immediately following the completion of utility work or if earth is to be left exposed for seven days or more on flat ground or three days or more on slopes greater than 25%,

5. Water is prevented from entering foundation work areas from surface runoff by creating small, compacted earth berms around the perimeter of the building site to divert runoff away from the working area.

2-5 METHODS OF CONTROL

The types of controls as noted in this section and the related Standard Plans for Erosion and Sedimentation Facilities are a minimum requirement, and, in certain conditions, may require much more extensive facilities. Erosion control facilities shall be periodically inspected and maintained by the developer or contractor to ensure continued intended operation.

2-5.1 CHECK DAMS AND ROCK BERMS

Check dams and/or berms constructed of earth, rock, or straw bales shall be incorporated into erosion control facilities as appropriate. . See Standard Plan 210b.

Straw bales (staked in place) may be used as energy dissipating drop structures, flow direction control structures and/or dams to create ponding. See Standard Plan 206.

Rock berms may be substituted for straw bales as filtering devices.

2-5.2 CUT-OFF TRENCHES - INTERCEPTOR DITCHES, DIKES OR BERMS

These structures are constructed to channel water away from unprotected slopes or erodible soils, to convey silt laden water to sedimentation facilities or to dissipate drainage into the natural on-site vegetation.

If the location of the trench, ditch, or dike may result in erosion of the structure itself, stabilization of the structure may be required. Riprap, temporary sodding, or a combination of filter fabric and riprap is methods of structure stabilization that may be required to prevent erosion. See Standard Plan No. 210B.

2-05.3 FILTERING DEVICES

Filtering devices, such as filter fabric fences, shall be used to filter runoff prior to discharge from site. See Standard Plan No. 205a and 205b. Approved filter fabrics are Celanese fiber, polyvinyl chloride woven cloth, reinforced chlorosulfinated polyethylene cloth, chlorinated polyethylene woven cloth, such as Mirafi 100X, Typar 3401, Stabilenka 100, or approved equal.

2-5.4 FLEXIBLE DOWN DRAINS

Flexible down drains may be utilized as temporary structures to protect open slopes and shall be constructed of flared end sections connected by plastic sheet tubing, heavy duty fabric, or non-perforated corrugated plastic pipe. See Standard Plan No. 209.

2-5.5 GRADIENT TERRACE

A gradient terrace is an earth embankment or ridge designed so that the top of the constructed ridge is no lower at any point than the design elevation of the water surface at the outlet under design flow and is installed so as to intercept surface runoff and convey it to a stable outlet at a non-erosive velocity. Gradient terraces may be useful both as a temporary and/or a permanent erosion control measure. See Standard Plan No. 210A.

2-5.6 SEDIMENT TRAPS

Sediment traps are structures of limited capacity designed to create a temporary siltation pond/filter around storm drain inlets or at points where silt-laden storm water is discharged.

Periodic Maintenance by the contractor or developer is crucial to the proper functioning of sediment traps. Examples of typical sediment trap installations are found in Standard Plan No. 211, 212 and 213. Placement of filter fabric under the grate of a catch basin is not an acceptable method of inlet protection.

2-5.7 TEMPORARY CONSTRUCTION ENTRANCE

A temporary construction entrance is a rock stabilized temporary entrance pad and shall be constructed at points where traffic will be entering or leaving a construction site from or onto public right-of-way. The pad shall be of sufficient length and width to eliminate transportation of mud and sediment from the construction area onto the public right-of-way by motor vehicles or by runoff, but under no circumstances shall it be less wide than the ingress/egress at the right-of-way nor less than 50 feet long. The stabilized construction entrance shall be a minimum thickness of eight inches and constructed of material approved by the City Engineer. The entrance shall be maintained to the satisfaction of the City Engineer. See Standard Plan No. 208). When site conditions are such that the temporary entrance pad fails to perform as required, all vehicles exiting the site shall have their tires and wheels cleaned by sweeping, brushing, or washing prior to entering public right-of-way. All washing shall be done on an area draining to an approved erosion control facility.

2-5.8 TEMPORARY SOIL STABILIZATION MEASURES

Soil stabilization measures protect soil from the erosive forces of raindrop impact and flowing water. Acceptable measures include establishing vegetation by sodding or seeding, mulching with 2.0 tons of straw per acre or approved equal, plastic or other impervious covering staked to the ground or anchored with rocks or sandbags, and the early application of gravel base on areas to be paved. The most appropriate measure should be chosen given the time of the year and the site conditions. Seeding alone is acceptable only on flat areas and slopes less than 25%, and only during the periods from March 1 to May 15 and August 15 to October 1 or as otherwise required or approved. Mulch may need to be held in place by utility mesh or netting.

2-5.9 TEMPORARY SILTATION/SEDIMENTATION PONDS

Temporary siltation/sedimentation ponds shall be required of all land alteration operations in order to detain runoff waters and trap sediment from erodible areas thus protecting properties, drainage ways, and streams below the installation from damage by excessive sedimentation and debris deposition. The dam or barrier forming the pond shall be located to provide for maximum volume capacity for trapping sediment behind the structure as well as for greatest ease of clean out. The temporary pond requirement may be waived, at the discretion of the City Engineer, for small areas of land disturbance where potential damage is minimal and pond construction impractical as long as runoff from all such areas is filtered prior to discharge from the site.

Temporary siltation/sedimentation ponds are basins created by construction of a barrier or by excavation or by a combination of both.

Interior surfaces of the sedimentation pond shall be stabilized where required to prevent erosion of the pond bottom and/or sides. Interior sides of the pond shall be no steeper than three feet horizontal to one foot vertical.

Siltation/sedimentation ponds shall provide a minimum of two feet of dead storage below the outflow elevation and will be sized to provide a minimum of one cubic foot of live storage per 100 square feet of channel area.

A stabilized access will be provided to the siltation/sedimentation pond for sediment removal and other maintenance.

2-6 VEGETATION

Addressed under the City Landscape Regulations SMC 14.41

2-6.1 RESTORATION

Areas disturbed by construction activity which are not to be covered by permanent impervious surfaces shall be landscaped or reseeded at the earliest possible time, not to exceed 15 days after final grade is reached during the allowable construction period (see section 2.5). This period may be reduced to seven days if site and weather conditions warrant the reduction. If the need to reduce the 15 day period is likely, the City will provide a verbal notice and a written one if necessary. Existing practical limitations of requesting a reduction of the allowable restoration period will be taken into account when directing a change.

If wet weather prohibits reseeding or landscaping during the given period, temporary soil stabilization measures must be used until conditions are suitable for permanent measures. When the area to be seeded is hard, compacted, or crusted, the top layer of soil shall be loosened by dicing, raking, or other acceptable means before seeding.

Although the specifications for seed, mulch, and fertilizer will depend upon the slope, soil conditions, and the planned use of the site, the following is an example which will usually be considered adequate:

Seed Mixture
- 40% Perennial Rye
- 20% Alta Fescue
- 20% Red Fescue
- 10% Chewing Fescue
- 10% Annual Blue Grass

@ 40 lbs per acre

Mulch 2,200 lbs/acre on slopes
1,000 lbs/acre on level areas

Fertilizers: Fertilizer application should depend on the results of soil testing. In general the following rates are typical:

Nitrogen: 20-40 lbs./acre
Phosphorus: 40-60 lbs./acre
Potash: 60 lbs./acre

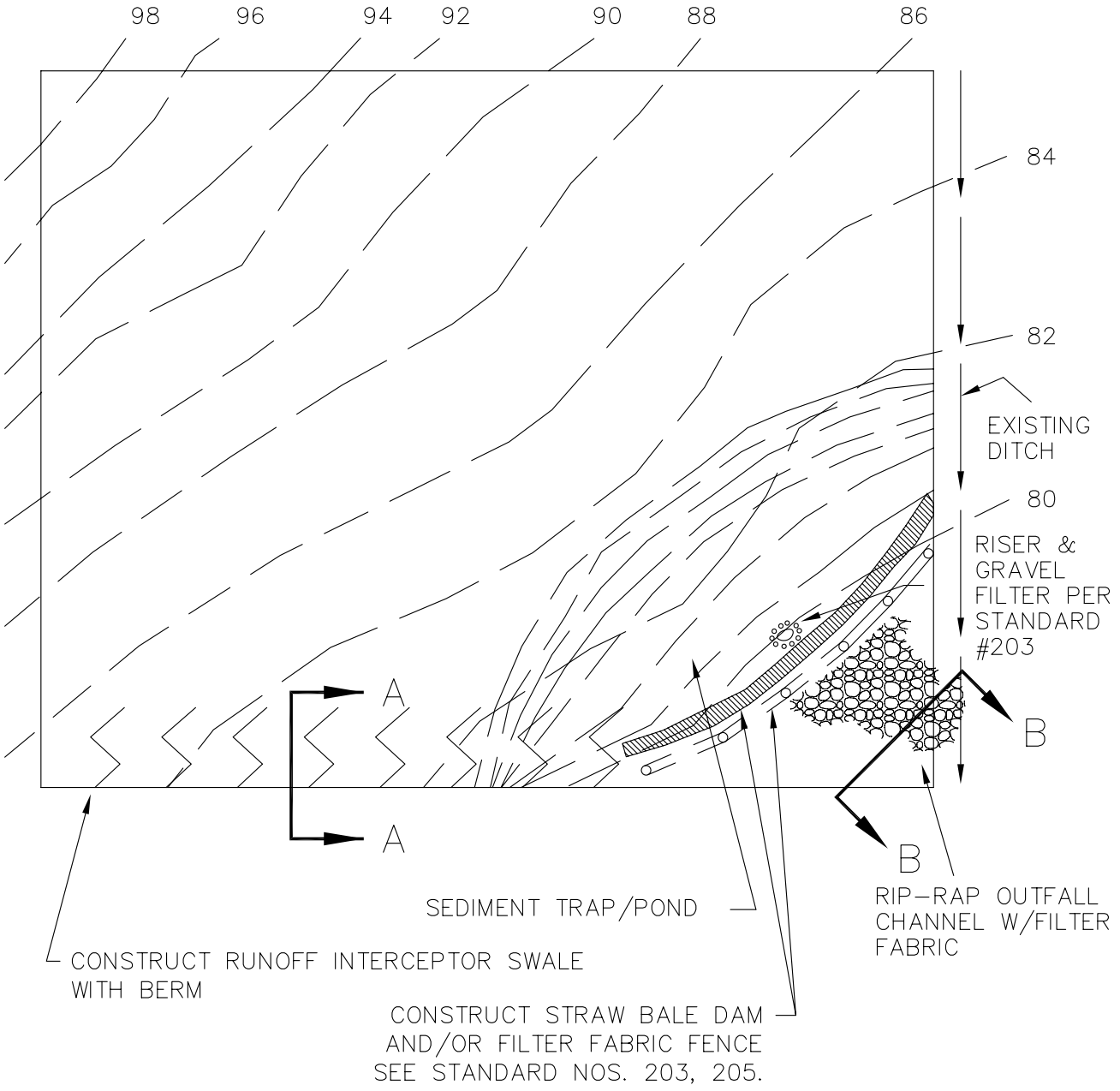
On steep slopes, hydro seeding and mulch anchoring may be required.

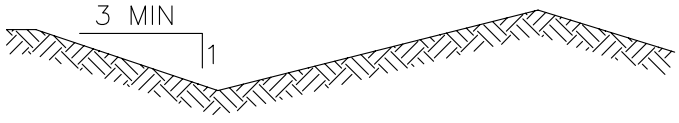
NOTE: For repair work or construction that disrupts sodded landscaped City ROW or private easements (existing developed property) restoration shall mean replacement with same-sod, not seed, unless the area is

so small as to make sod impractical. While the City and contractors will attempt to preserve to the best of our ability landscaping in the City ROW, the City will not accept responsibility for landscaping in the City ROW, nor hold contractors responsible for such landscaping.

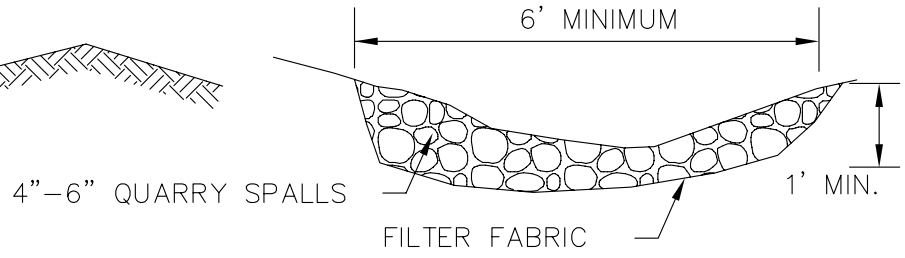
DRAWING INDEX

200 SERIES	EROSION AND SEDIMENT CONTROL	Last Revision Date
201	Typical Temporary Erosion Control	4-01-04
202	Sediment Trap Outlet	4-01-04
203	Sediment Pond Outlet	4-01-04
204	Anti-Vortex Device	4-01-04
205a	Temporary Silt Fence	4-01-04
205b	Silt Fence With Berm And Trench	4-19-96
206	Straw Bale Dam	4-01-04
207	Temporary Diversion And Perimeter Dike	4-01-04
208	Temporary Construction Entrance	4-01-04
209	Temporary Downdrain Structure	4-01-04
210a	Gradient Terrace	4-01-04
210b	Interceptor Ditch With Rock Check Dam	4-01-04
211	Catch Basin/Inlet Sedimentation Trap-Type "A"	4-01-04
212	Catch Basin/Inlet Sedimentation Trap-Type "B"	4-01-04
213	Catch Basin/Inlet Sedimentation Trap-Type "C"	4-01-04





INTERCEPTOR SWALE
DETAIL SEC. A-A



RIP-RAP OUTFALL CHANNEL SEC. B-B

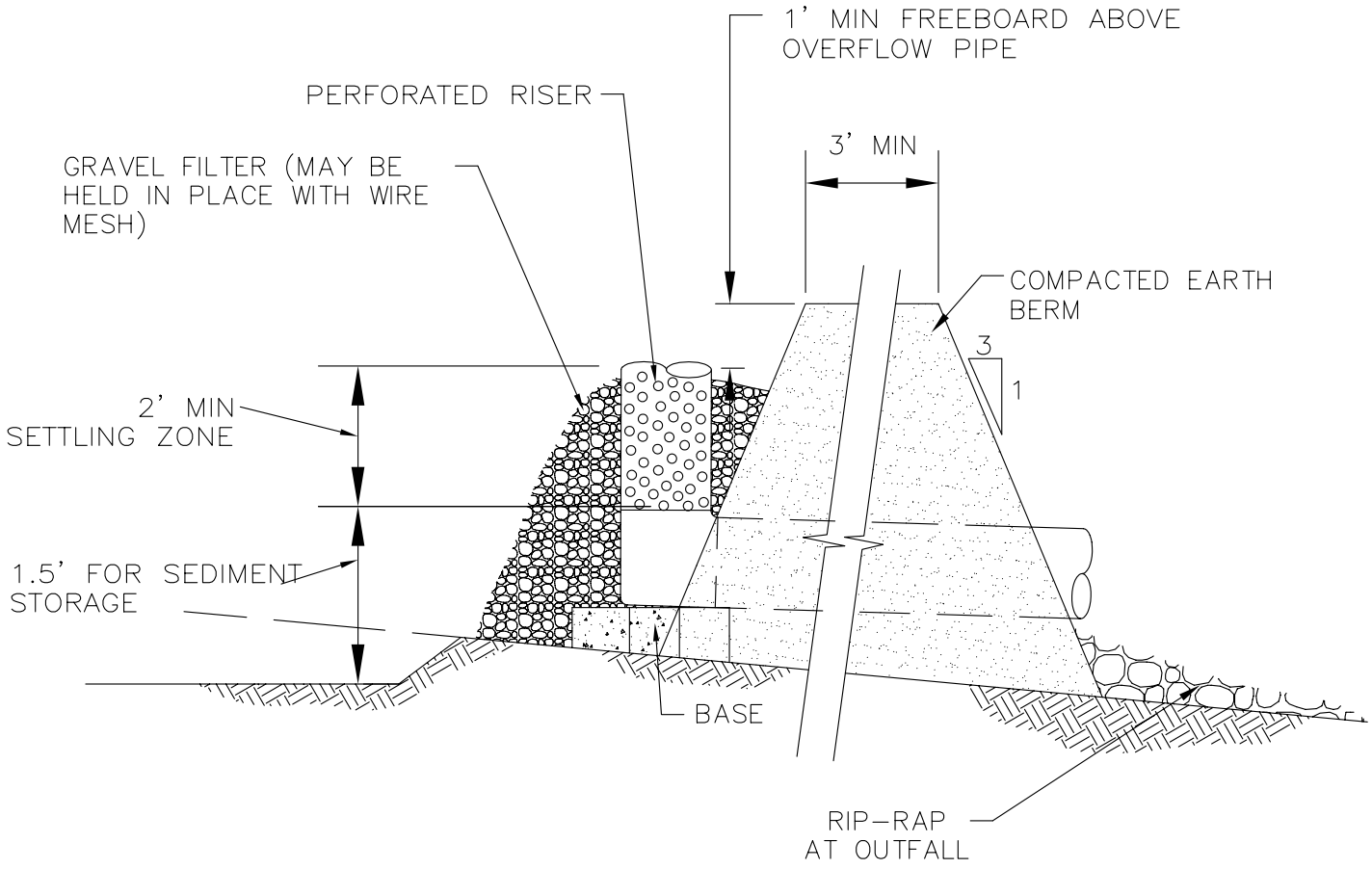
SNOHOMISH
WASHINGTON

TYPICAL TEMPORARY EROSION CONTROL

Approved By:
City Engineer
Date: May 30, 2004

201
Number

City of Snohomish Public Works Department



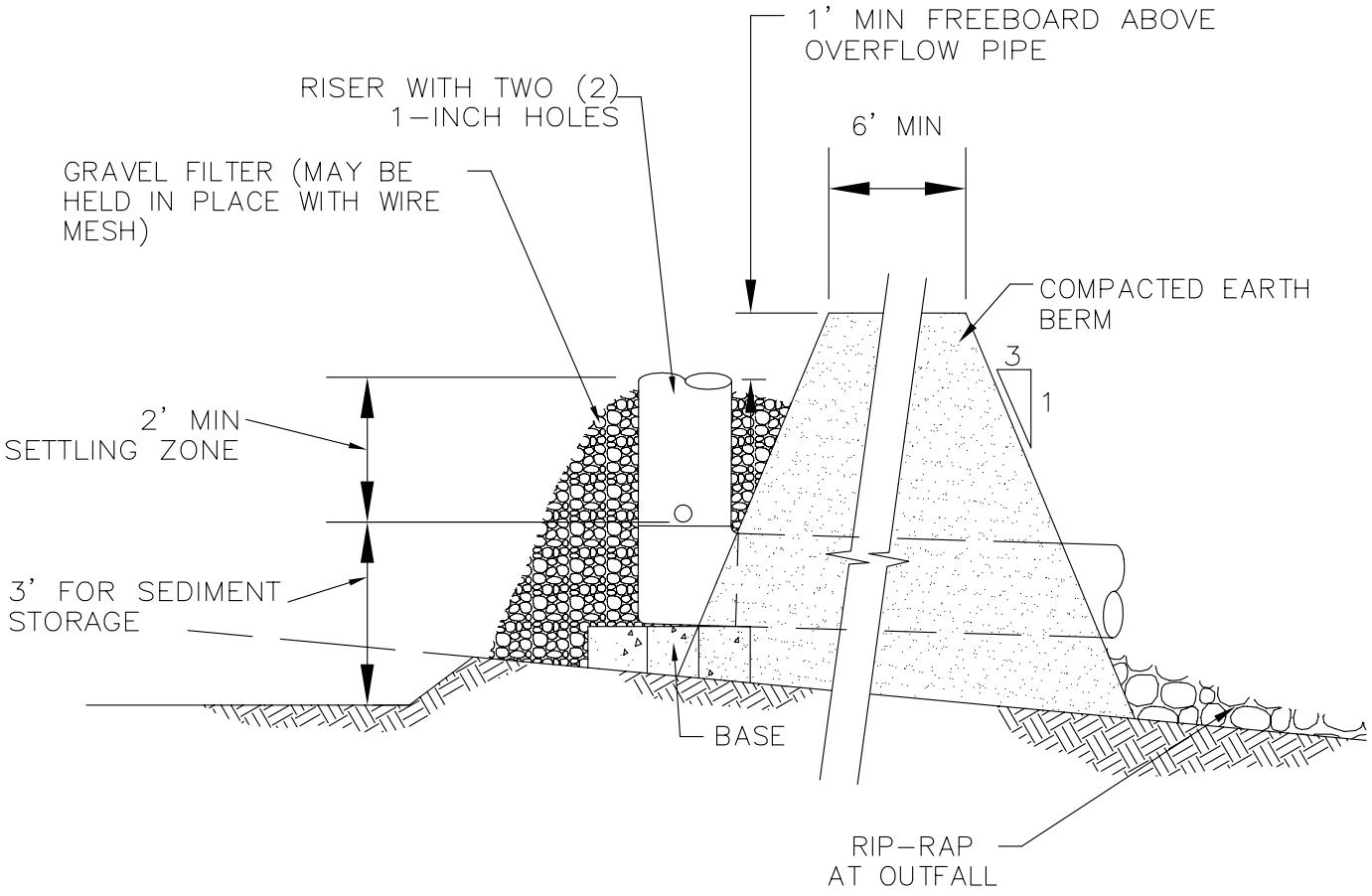
SNOHOMISH WASHINGTON

SEDIMENT TRAP OUTLET

Approved By:
City Engineer
Date: May 30, 2004

202
Number

City of Snohomish Public Works Department



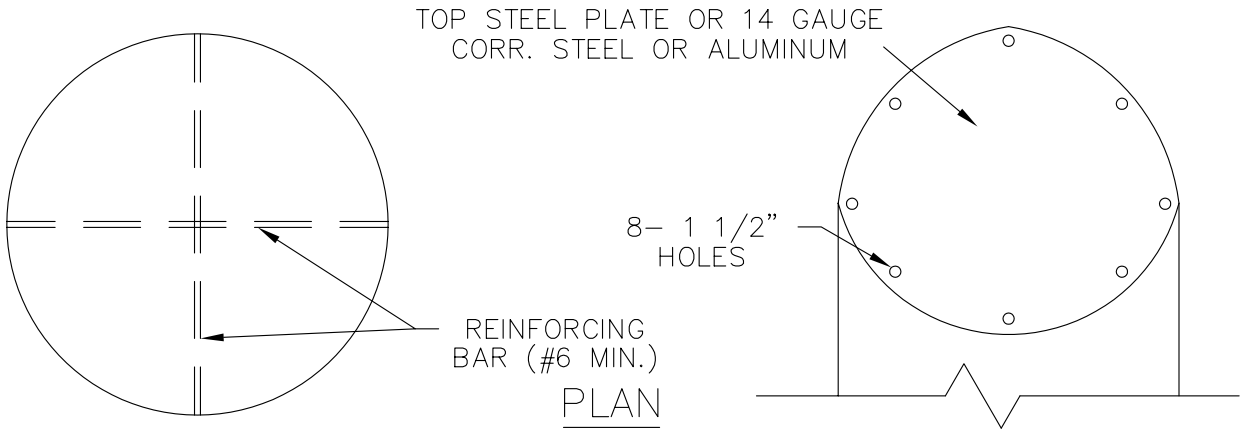
SNOHOMISH WASHINGTON

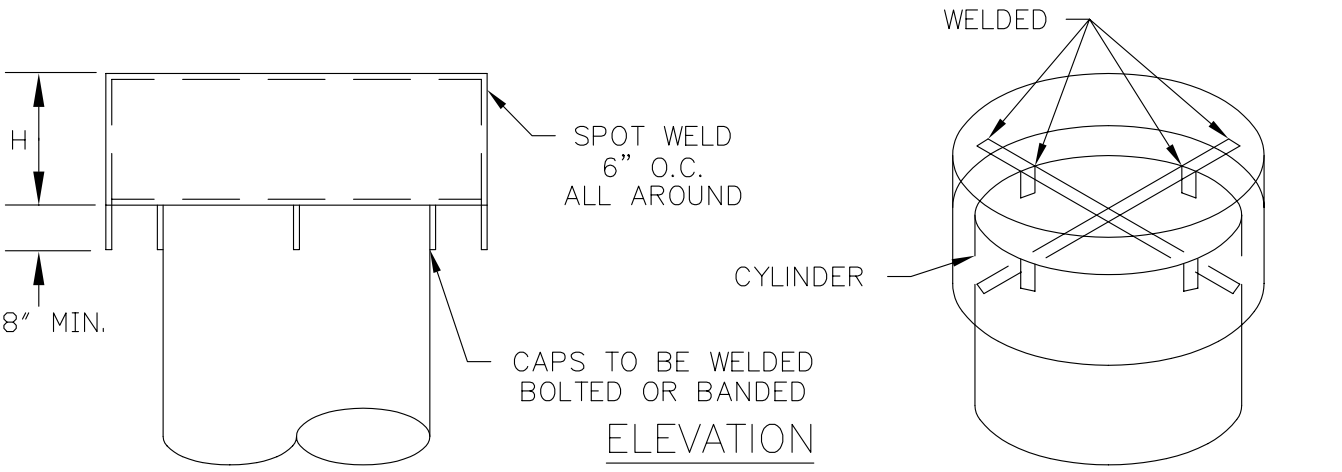
SEDIMENT POND OUTLET

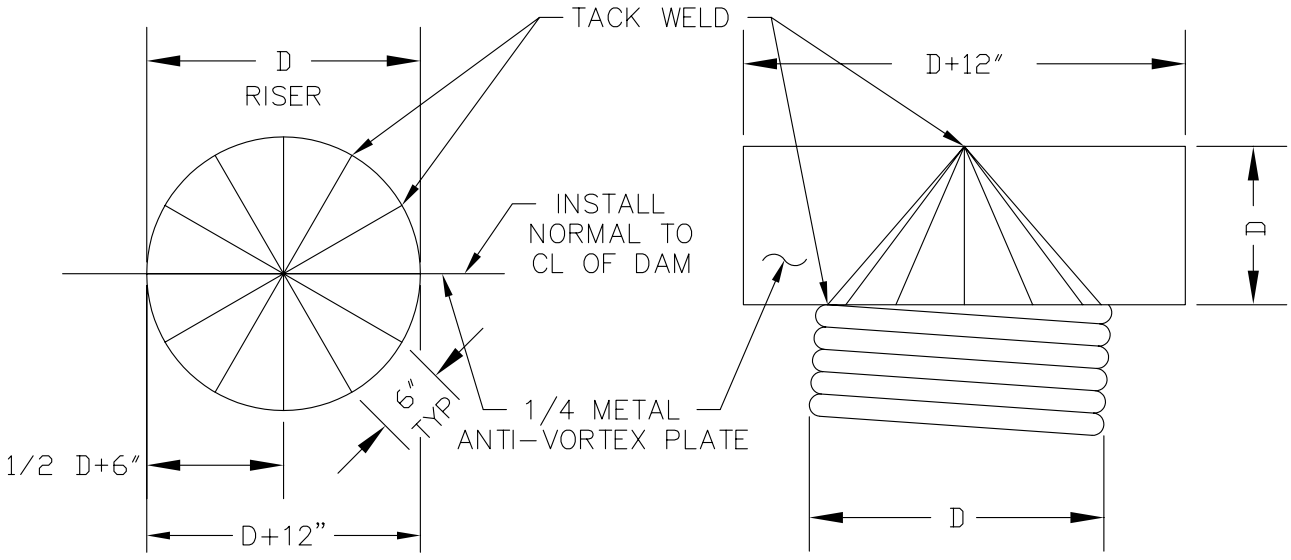
Approved By:
City Engineer
Date: May 30, 2004

203
Number

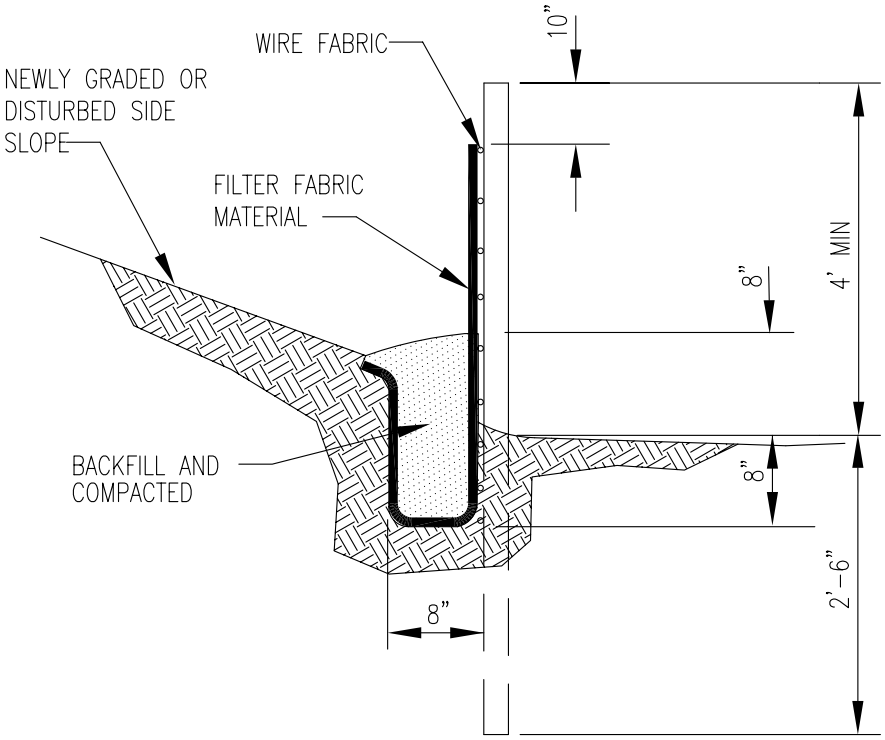
City of Snohomish Public Works Department



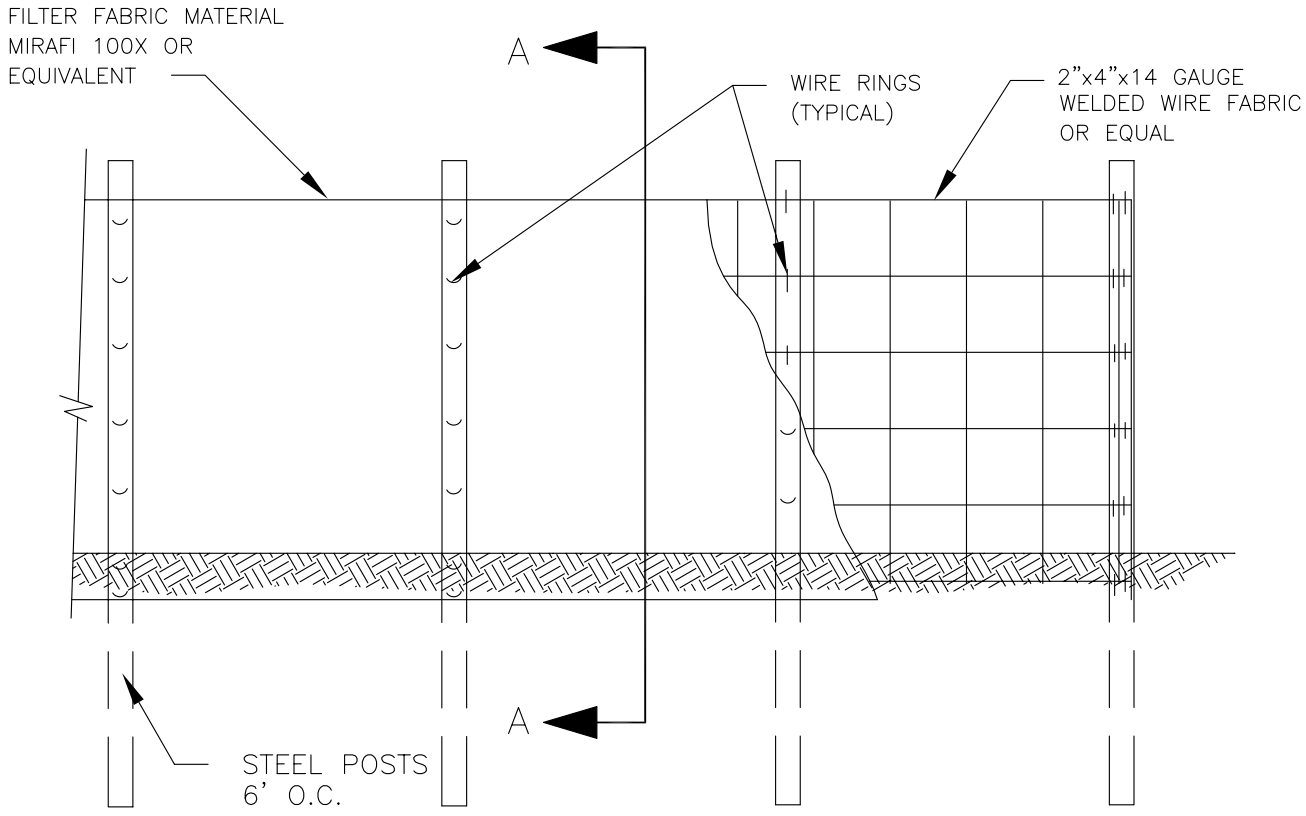




SNOHOMISH WASHINGTON	**ANTI VORTEX DEVICE**	Approved By: City Engineer Date: May 30, 2004	204 Number
	City of Snohomish Public Works Department		



SECTION A–A



ELEVATION

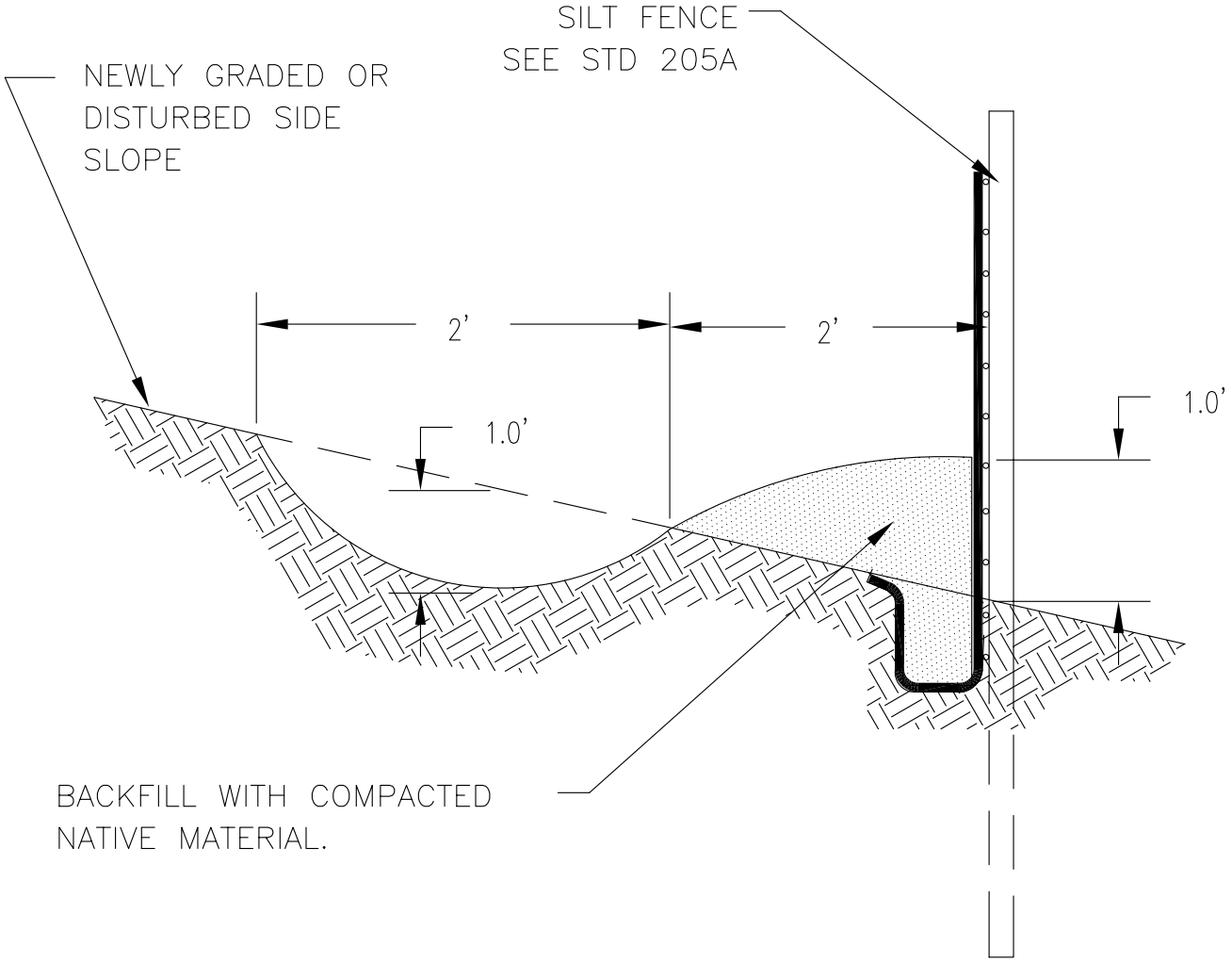
SNOHOMISH
WASHINGTON

TEMPORARY SILT FENCE

Approved By:
City Engineer
Date: May 30, 2004

205a
Number

City of Snohomish Public Works Department



NOTE:

INSTALL THE SILT FENCE FIRST, PER STD #205A. AFTER THE SILT FENCE HAS BEEN INSTALLED, CONSTRUCT BERM AND TRENCH.

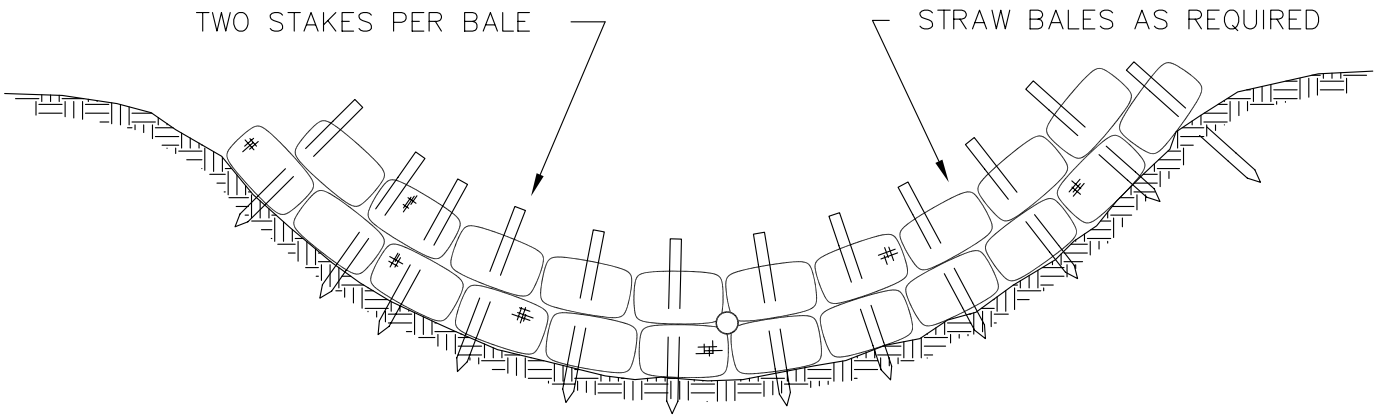
SNOHOMISH
WASHINGTON

A BERM AND TRENCH

Approved By:
City Engineer
Date: May 30, 2004

205b
Number

City of Snohomish Public Works Department



NOTES

1. WHERE POSSIBLE, MAINTAIN NATURAL VEGETATION FOR SILT CONTROL.

2. TEMPORARY SILTATION AND DETENTION PONDS TO BE CONSTRUCTED BY PLACING STRAW BALES OR FILTER FABRIC FENCES ACROSS SWALES OR EXCAVATION SILTATION PONDS UTILIZING FILTER SYSTEM PRIOR TO DISCHARGE. PONDS SHALL BE CONSTRUCTED SO AS TO PROVIDE ONE CUBIC FOOT OF SETTLING POND PER 100 SQUARE FEET OF CLEARED AREA TRIBUTARY TO POND.

3. ALL TEMPORARY SILTATION AND DETENTION PONDS SHALL BE MAINTAINED IN A SATISFACTORY CONDITION UNTIL SUCH TIME THAT CLEARING AND/OR CONSTRUCTION IS COMPLETED AND THE PERMANENT DRAINAGE FACILITIES ARE OPERATIONAL.

4. RETURN SILTATION CONTROL AREAS TO ORIGINAL GROUND CONDITIONS.

5. RIP–RAP BASE (BOTH SIDES) OF BALES OR OUTFALL CHANNEL FOR EROSION CONTROL, AS REQUIRED.

6. APPROVAL OF THIS PLAN DOES NOT CONSTITUTE AN APPROVAL OF DESIGN, SIZE NOR LOCATION OF PIPES, RESTRICTORS, CHANNELS OR RETENTION FACILITIES; BUT IS AN APPROVAL OF TEMPORARY SEDIMENTATION CONTROL PLAN ONLY.

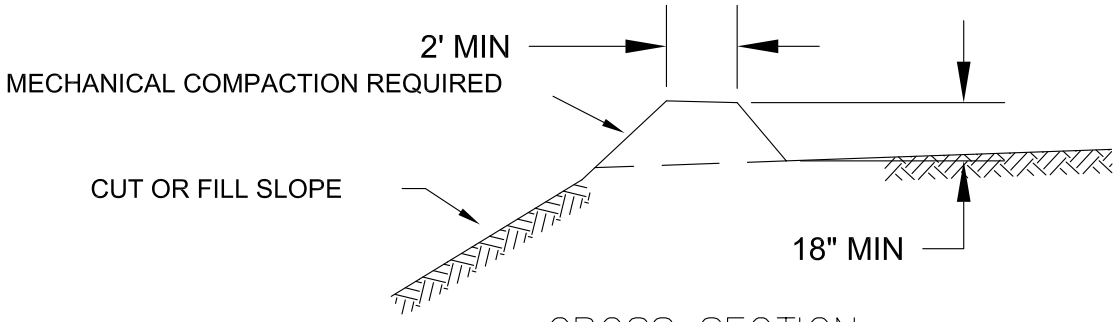
SNOHOMISH
WASHINGTON

STRAW BALE DAM

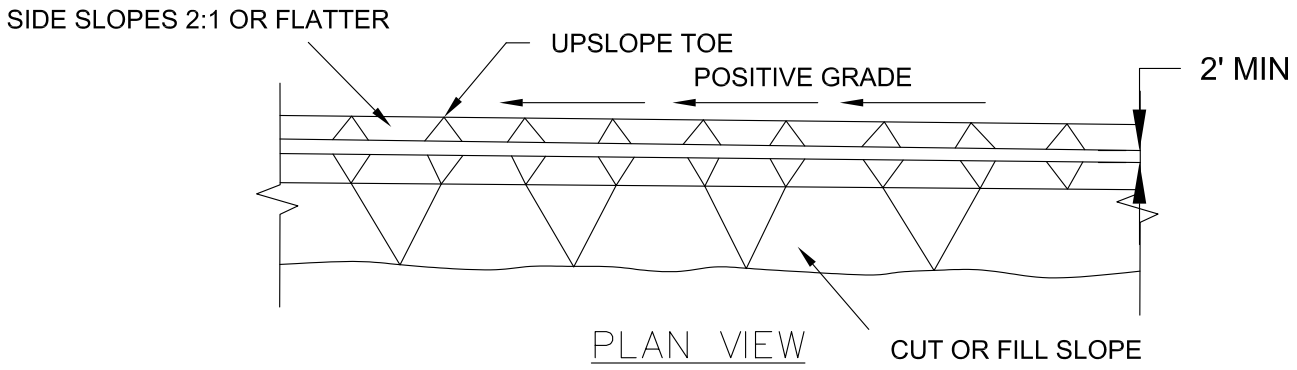
Approved By:
City Engineer
Date: May 30, 2004

206
Number

City of Snohomish Public Works Department

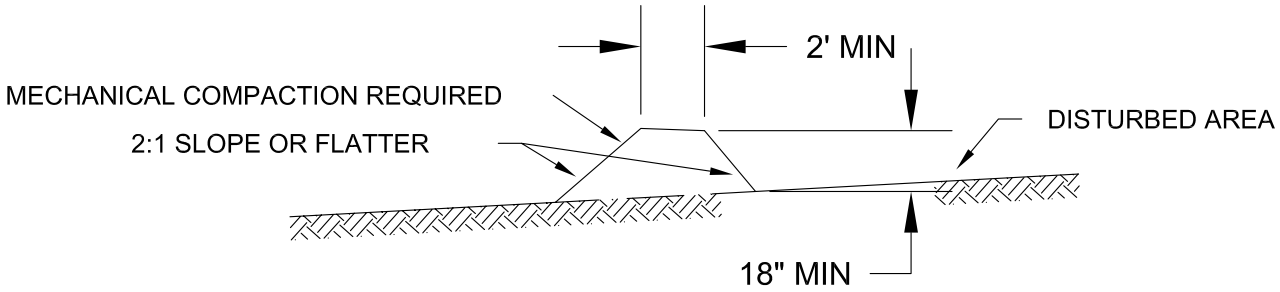


CROSS SECTION



PLAN VIEW

TEMPORARY DIVERSION DIKE



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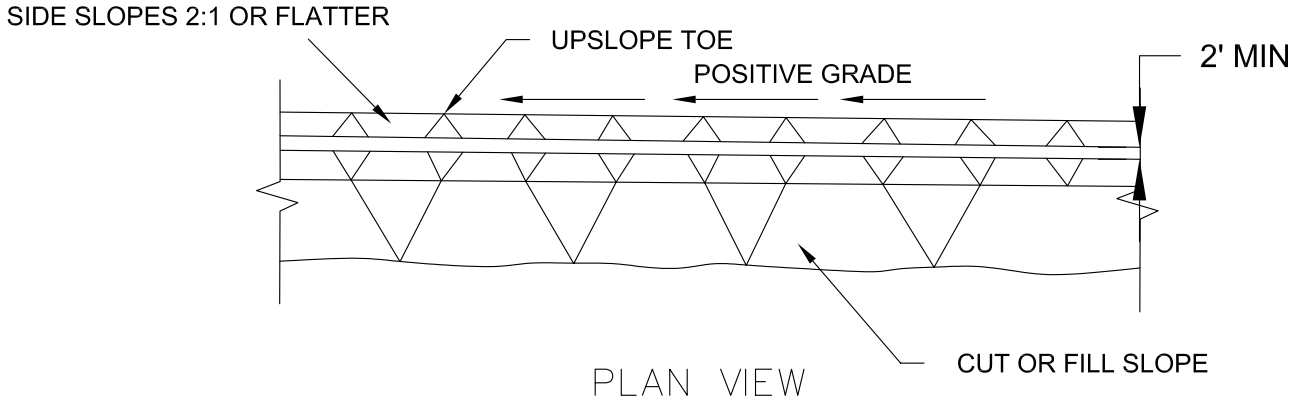


PLAN VIEW

TEMPORARY PERIMETER DIKE

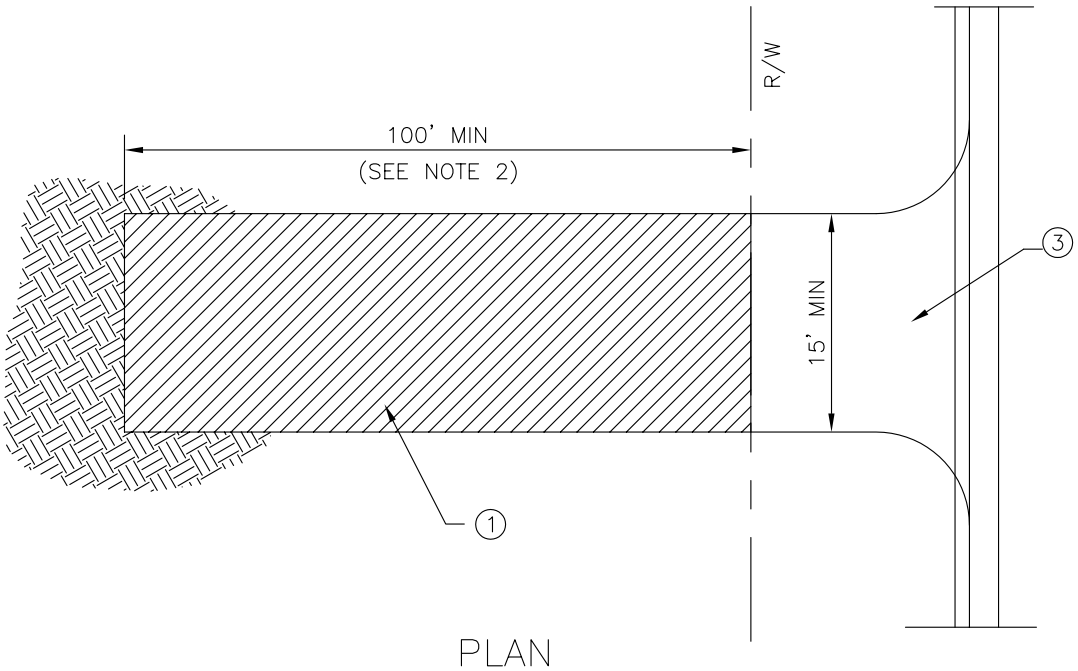
SNOHOMISH WASHINGTON

TEMPORARY DIVERSION AND PERIMETER DIKE

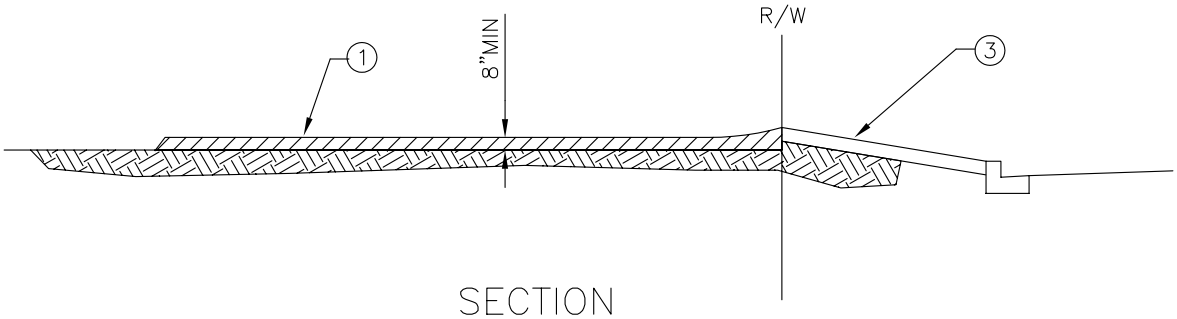
Approved By:
City Engineer
Date: May 30, 2004

207
Number

City of Snohomish Public Works Department



PLAN



SECTION

NOTES:

1. QUARRY SPALLS AS SPECIFIED IN: SECTION 9-13.6 OF THE WSDOT/APWA STANDARD SPECIFICATIONS.

GRADING REQUIREMENTS

SIEVE SIZE	% PASSING
8"	100
3"	40
3/4"	10

2. THE 100' MINIMUM LENGTH SHALL BE LENGTHENED AS NECESSARY TO ENSURE MATERIAL IS NOT TRACKED INTO THE PUBLIC RIGHT-OF-WAY. ALTERNATE CONSTRUCTION ENTRANCES WILL BE ALLOWED WITH APPROVAL OF THE CITY ENGINEER ON A CASE BY CASE BASIS, WHERE PHYSICAL SITE CONDITIONS AND SIZE DICTATE

3. EXISTING DRIVEWAY RAMP, OR SITE ACCESS ROAD 15' WIDE MIN. MATERIAL MUST BE EQUAL TO OR BETTER THAN SPECIFIED IN NOTE 1.

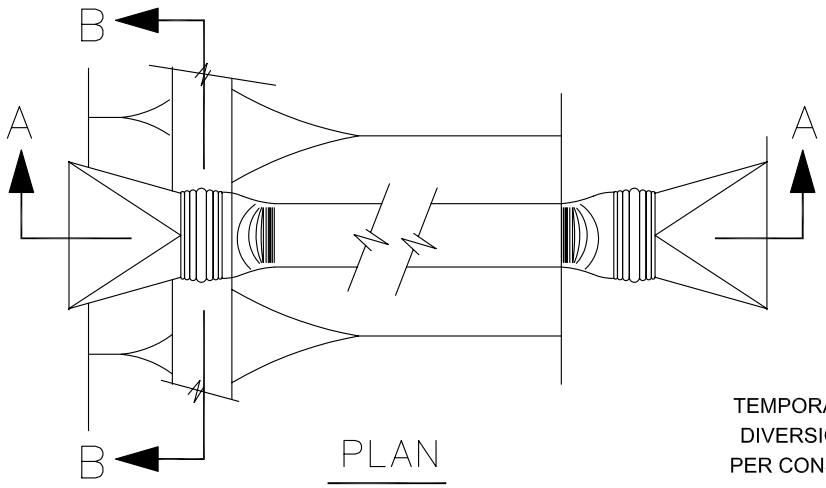
SNOHOMISH WASHINGTON

TEMPORARY CONSTRUCTION ENTRANCE

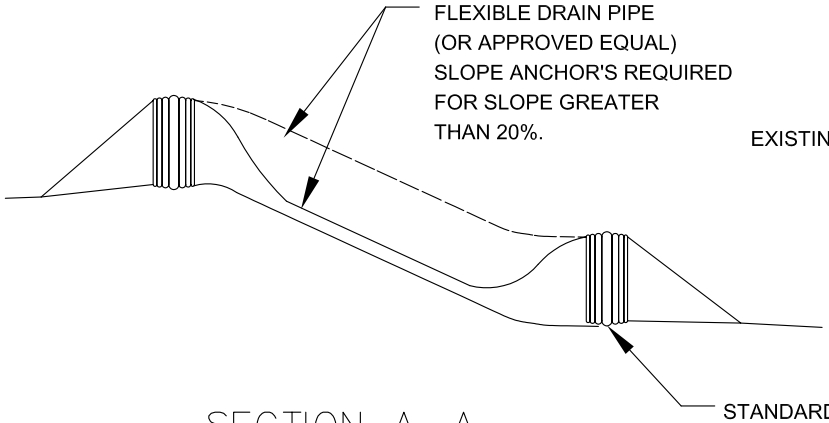
Approved By:
City Engineer
Date: May 30, 2004

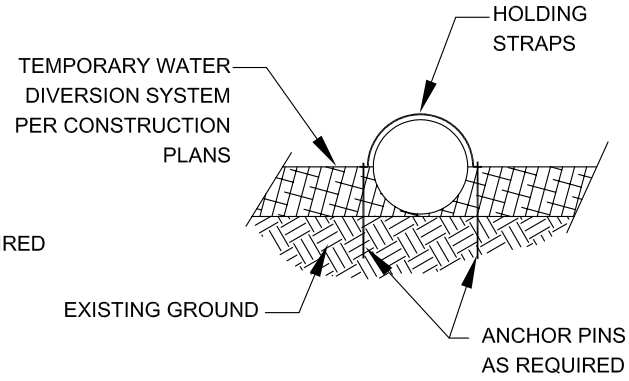
208
Number

City of Snohomish Public Works Department



PLAN

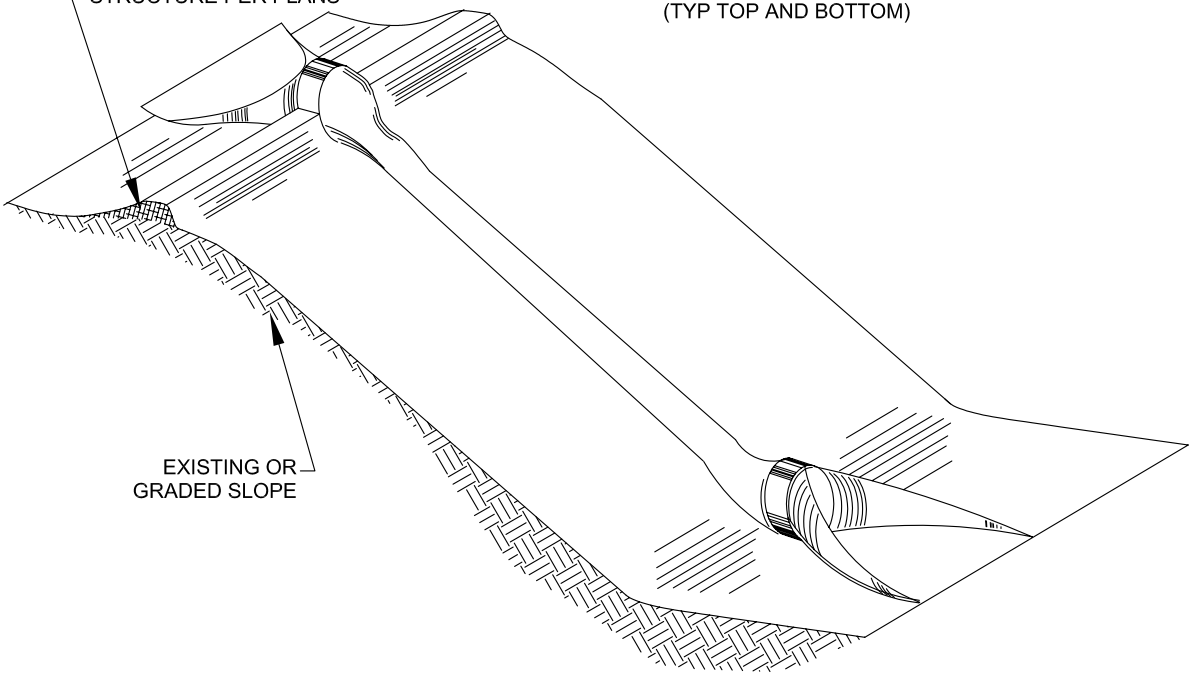




SECTION B–B

STANDARD METAL OR
HIGH DENSITY POLYETHYLENE
END SECTION WITH HOLDING
STRAPS AND ANCHOR PINS
(TYP TOP AND BOTTOM)

SECTION A–A



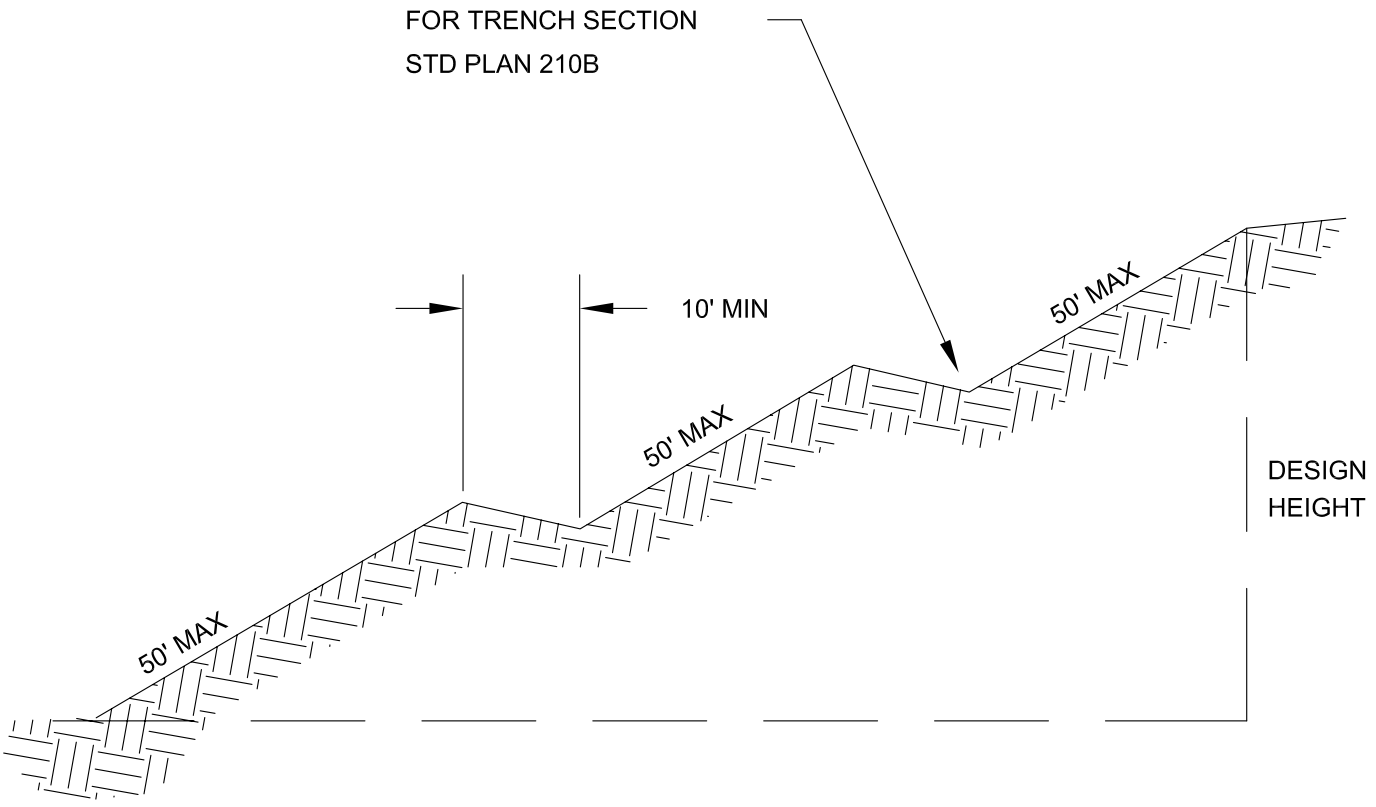
SNOHOMISH
WASHINGTON

TEMPORARY DOWNDRAIN STRUCTURE

Approved By:
City Engineer
Date: May 30, 2004

209
Number

City of Snohomish Public Works Department



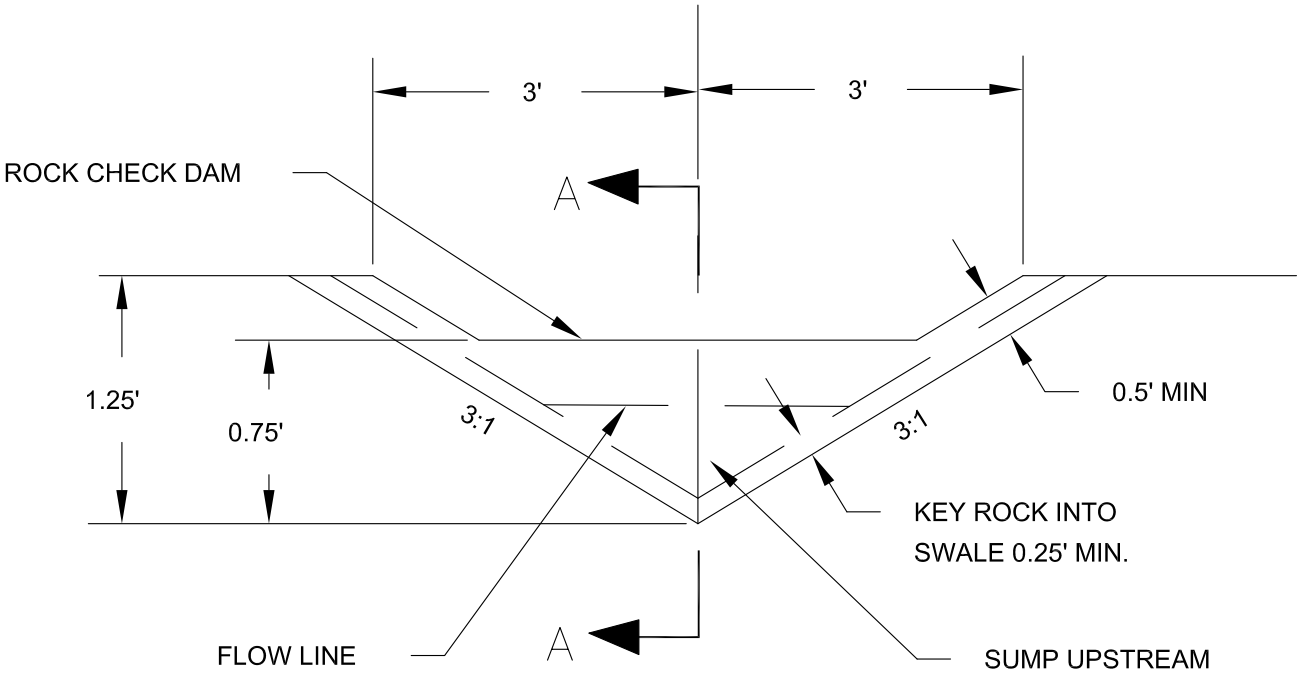
SNOHOMISH
WASHINGTON

GRADIENT TERRACE

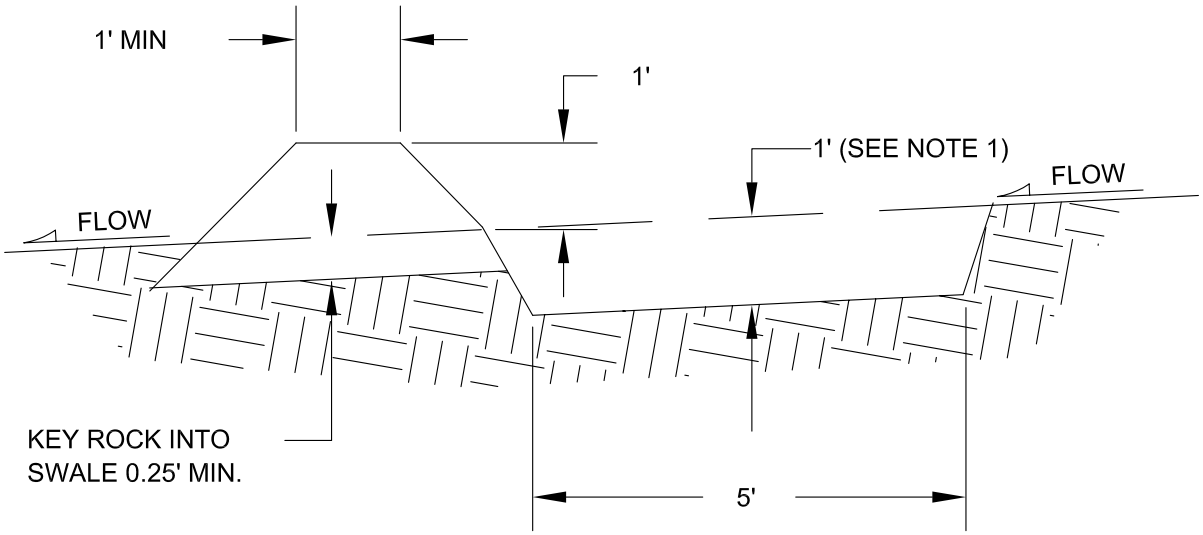
Approved By:
City Engineer
Date: May 30, 2004

210a
Number

City of Snohomish Public Works Department



TYPICAL CROSS SECTION



NOTE:

1. SUMP BEHIND ROCK CHECK DAM SHALL BE INSPECTED DAILY, AND CLEANED WHEN COLLECTED DEBRIS EXCEEDS 1/2 OF ITS DEPTH.

SECTION A–A

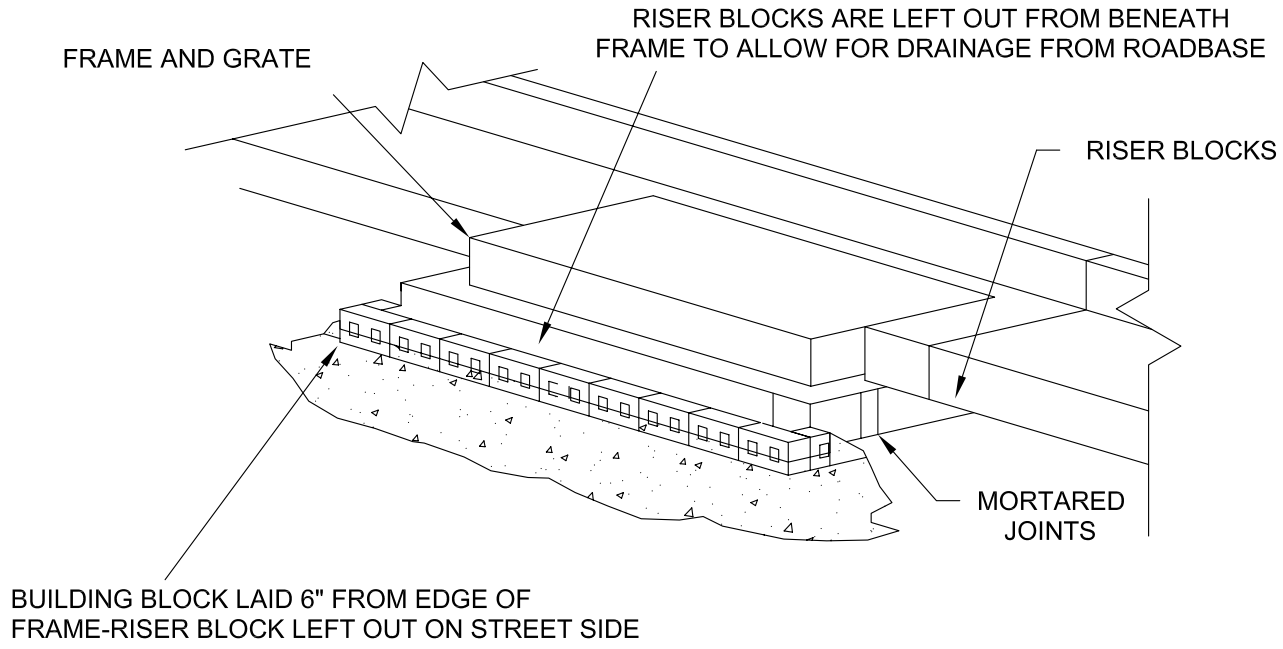
SNOHOMISH WASHINGTON

INTERCEPTOR DITCH WITH ROCK CHECK DAM

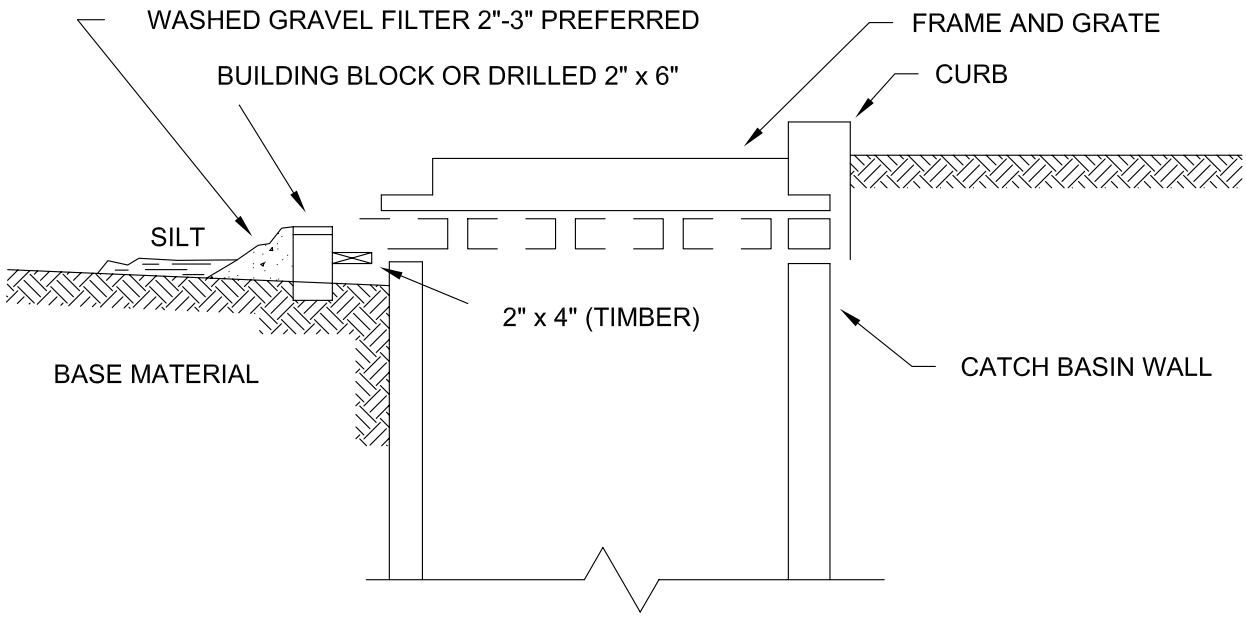
Approved By:
City Engineer
Date: May 30, 2004

210b
Number

City of Snohomish Public Works Department



ISOMETRIC



SECTION

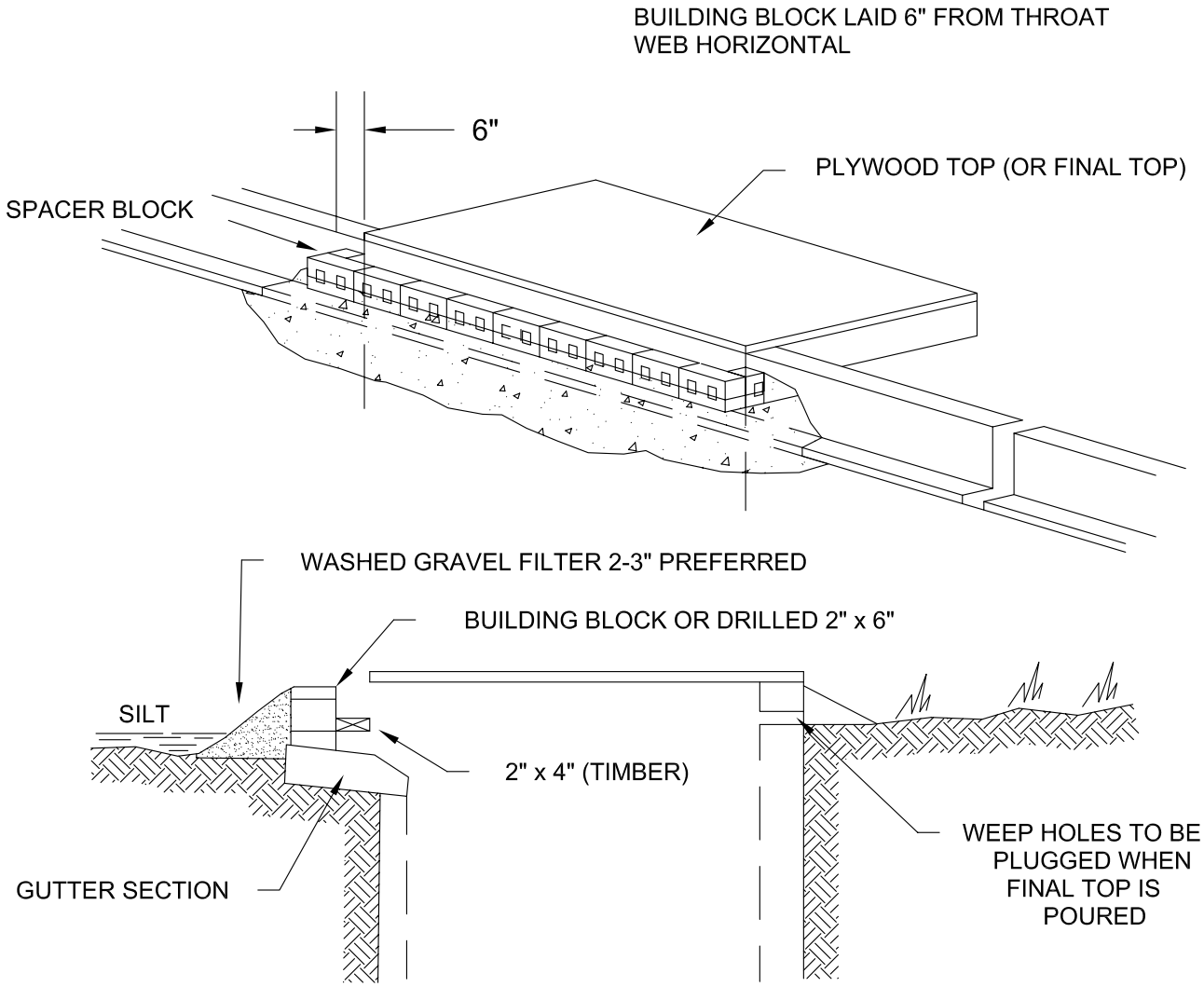
SNOHOMISH
WASHINGTON

CATCH BASIN INLET SEDIMENTATION TRAP-TYPE A

Approved By:
City Engineer
Date: May 30, 2004

211
Number

City of Snohomish Public Works Department



NOTES

1. USE DURING STREET CONSTRUCTION.
2. FULL THROAT IS AVAILABLE FOR HEAVY FLOW.
3. EXPANDED METAL OR HARDWARE CLOTH IN FRONT OF BLOCK PREVENTS GRAVEL FROM WASHING INTO STRUCTURE.
4. 2" x 4" BEHIND BLOCK AND ACROSS THROAT HELPS KEEP BLOCK IN PLACE. PLACE IN OUTER HOLE OF SPACER BLOCK.

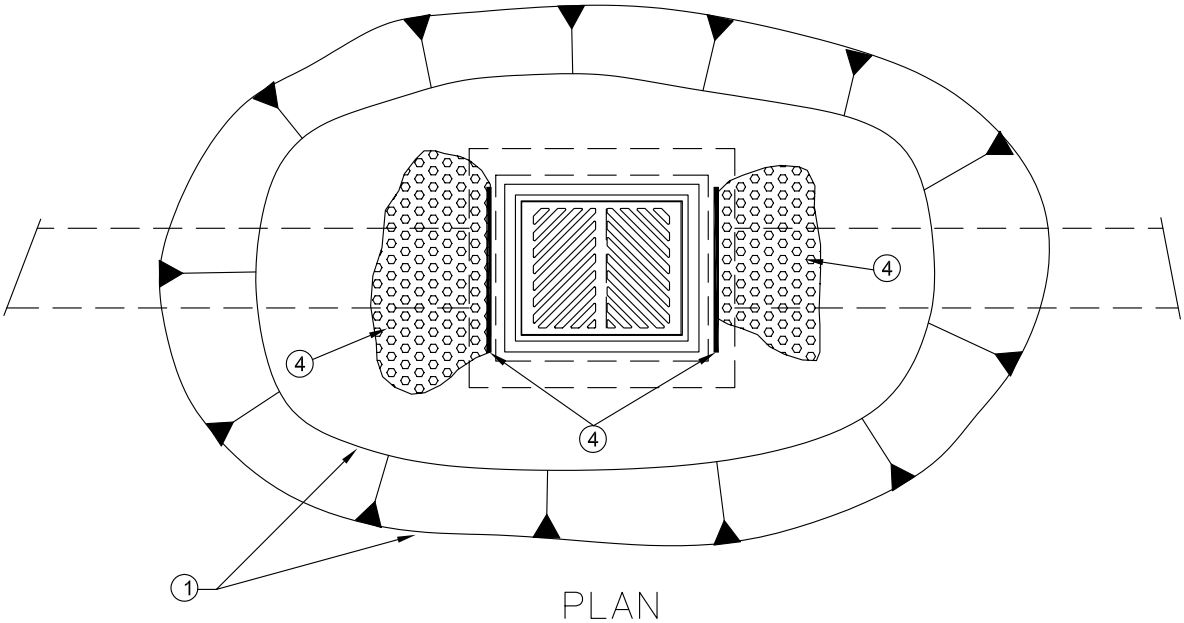
SNOHOMISH WASHINGTON

CATCH BASIN INLET SEDIMENTATION TRAP-TYPE B

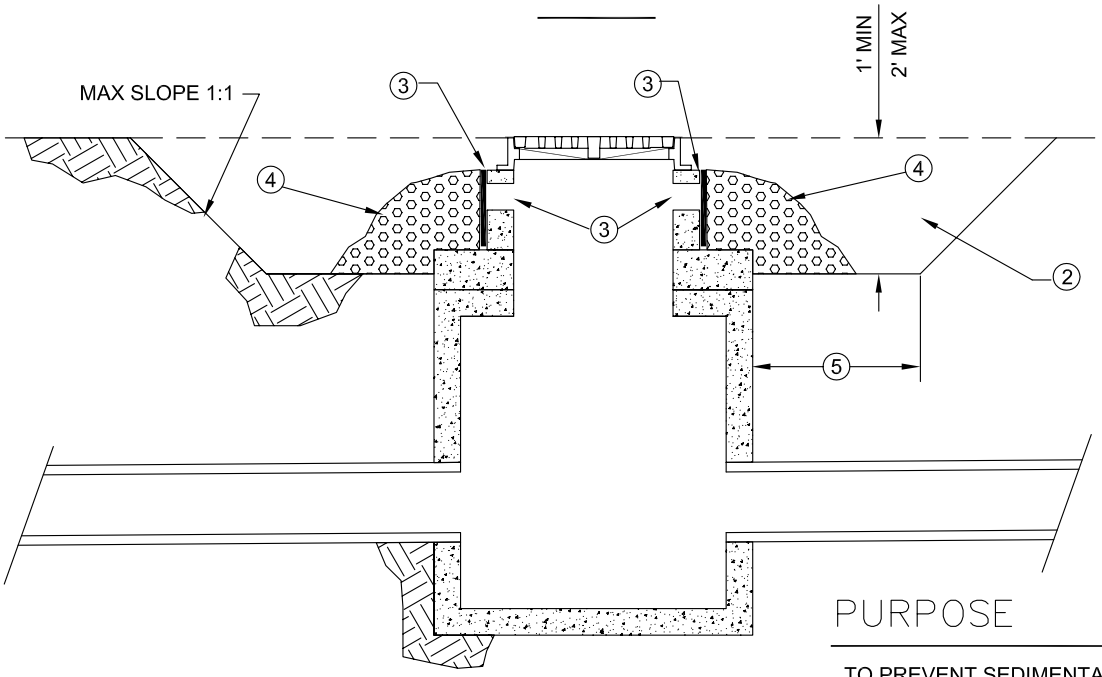
Approved By:
City Engineer
Date: May 30, 2004

212
Number

City of Snohomish Public Works Department



PLAN



SECTION

PURPOSE

TO PREVENT SEDIMENTATION FROM ENTERING STORM DRAINAGE SYSTEM AT CATCH BASIN/INLETS DURING CONSTRUCTION.

NOTES

1. SHAPE OF SEDIMENTATION POND MAY VARY TO FIT DRAINAGE AREA AND TERRAIN. MODIFY AS NECESSARY TO ENSURE SATISFACTORY TRAPPING OF SEDIMENT.HALF-CIRCLE POND MAY BE USED WHEN CURB AND GUTTER ARE INSTALLED DURING STREET CONSTRUCTION.
2. CLEAN OUT WHEN SEDIMENT REACHES 6" BELOW GRATE
3. TEMPORARILY LEAVE OUT BLOCK. COVER OPENING WITH WIRE SCREEN.
4. PLACE GRAVEL IN FRONT OF SCREEN TO FILTER SEDIMENT.
5. SIZE POND BASED ON EXPECTED FLOWS DURING CONSTRUCTION.

SNOHOMISH WASHINGTON

CATCH BASIN INLET SEDIMENTATION TRAP-TYPE C

Approved By:
City Engineer
Date: May 30, 2004

213
Number

City of Snohomish Public Works Department

ENGINEERING DESIGN AND CONSTRUCTION STANDARDS

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DESIGN AND CONSTRUCTION STANDARDS
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DESIGN AND CONSTRUCTION STANDARDS
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DESIGN AND CONSTRUCTION STANDARDS
AND
SPECIFICATIONS

SECTION 3

STREETS AND RELATED WORK

3-1 GENERAL REQUIREMENTS

All work performed in the design, preparation of plans and in the construction or improvement of city streets and all appurtenances, whether public or private shall be the responsibility of the developer or contractor and done to the satisfaction of the city engineer and in accordance with the plans and specifications approved by the city for the work.

It is emphasized that no permits will be issued to start work until plans for that work are approved. Any revisions to the plans shall be approved by the city engineer before being implemented. A set of "as-built" drawings (mylars) will be required at the completion of the project and prior to final acceptance of the work. See individual utility sections for more specific "as-built" requirements.

City Ordinances and Standards establish policy for the installation of street improvements. Specific application will be determined at the time of permit application and or issuance.

3-2 ROADWAY TYPES AND GEOMETRICS

3-2.1 GENERAL

City of Snohomish roadways are classified functionally as indicated in Standard Plan No. 300. Criteria for minimum right-of-way and roadway widths and other geometrics shall be as listed for given classifications.

Structural sections and roadway appurtenances shall be as shown on Standard Plan Nos. 301 and 302.

The City Engineer may require the second lift of asphalt to be bonded and delayed for up to one year.

Typical utility locations for design purposes are shown on Standard Plan No. 322.

3-2.2 HORIZONTAL ALIGNMENT

3-2.2(1) CURB RETURN RADII

Radii of 40 feet or more should be three centered compound curves or simple curves with tapers to fit the paths of appropriate design vehicles.

3-2.2(2) LOCAL STREETS

For the intersection of two local streets, the minimum allowable curb radius shall be 25 feet, which is to be measured from the radius point to the face of curb.

For the intersection of a local street with any collector or arterial, the minimum radius shall be 30 feet.

3-2.2(3) OTHER STREETS

On all other street intersections, the minimum allowable radii shall be 30 feet.

3-2.2(4) TRUCKS AND BUSES

Radii of 40 feet or more should be provided where large truck combinations and buses turn frequently. Larger radii are also desirable where speed reductions would cause problems.

3-2.3 INTERSECTIONS

Intersections

Angle of Intersection	80º to 90º
Minimum Centerline Radius	N/A
Minimum Curb Radius	25 feet

3-2.4 STREET ENDS

Cul-de-sacs shall be provided at all permanent street ends and on any temporary dead end location when the length of the street is more than 150 feet in length. Cul-de-sacs shall be per Standard Plan No. 304.

On temporary dead ends, when the street is less than 200 feet in length, the required turnaround area may be a hammerhead type of design per Standard Plan No. 323.

3-2.5 GRADING OF STREETS

3-2.5.1 CENTERLINE PROFILE

The centerline profile of the street and accompanying centerline elevations are approved by the Department of Public Works and developed by the developer's professional civil engineer for approval by the Director of Public Works.

When no profile has been established for the streets abutting and leading to the development site, the developer shall provide a survey of the street area by a licensed surveyor for the purpose of establishing the proposed centerline profile. The survey shall extend the full length of the block and be based on City of Snohomish datum, with City of Snohomish bench mark elevations and locations, and field book pages and numbers referenced on the survey. (Field books containing bench mark and monument information are located in the Engineering Division at City Hall.)

The profile shall have a constant slope from cross-street to cross-street, with vertical curves as needed at street intersections. Additional slope changes within the block are permitted only when a constant slope cannot be obtained.

The minimum roadway profile grade permitted is 1%.

The minimum vertical curve for roadways is 75 feet. The point of vertical curvature (pvc) shall not encroach into a cross street any further than the cross street center of pavement.

Minimum horizontal curves with super elevation are as follows:

Principal Arterials
Minor Arterials - 410 feet

Collector Arterials
Commercial Access Streets - 300 feet

Residential Access Streets - 115 feet

3-2.5.2 DESIGN CROSS SECTION

Whenever possible, street improvements shall conform to the standard street cross section. The standard roadway cross slope is 2% down from crown to gutter line. In areas where pavement width is being added to an existing street, the slopes may vary, when necessary, as follows: maximum permitted cross slope is 4% (a variance may be approved up to 5% by the Public Works Director per Section 1.9 of these standards); minimum permitted cross slope is 1%.

The standard alley cross slope is 4.7%. When necessary to provide access to existing driveways and garages along the alley, the alley cross slope may be adjusted as needed, to a maximum cross slope of 6% and a minimum of 2%.

A typical cross section with dimensions shall be shown on the street improvement plans. In addition, design cross sections are required for proposed street and alley improvements every 25 feet for the length of the improvement, and shall include a cross section at each end of the improvement, at all driveways, and at all building entrances located within ten feet of the property line.

The design cross sections shall show existing and proposed grades, with spot elevations provided at the centerline, existing edge of pavement, gutter line, top of curb or thickened edge, back of sidewalk, property lines, catch lines, and any retaining walls or rock facing. The cross sections shall be stamped and signed by the design engineer. The existing elevations shall be based on recent survey data. Elevations on design cross sections shall be consistent with elevations shown on the Building Grade Sheet.

3-2.5.3 ROADWAY

Roadway design includes:

Grading to the approved profile and cross section elevations (see previous section)
Pavement type, depth and width
Curb type and location
Roadway drainage
The transition from the improvement area to the existing roadway

3.2.5.4 COMPACTION OF SUBGRADE

The subgrade must be compacted to 95% of maximum dry density for all street and alley improvements. Subgrade materials that cannot be compacted to this density shall be over-excavated (removed) and the subgrade replaced with approved material. Soil tests will be required during construction to show that the required compaction has been obtained.

3.2.5.5 PAVEMENT REPLACEMENT

When new street improvements are required and the existing pavement is in poor condition, it shall be replaced or restored. Examples of this reconstruction are as follows:

When existing asphalt or seal coat pavement is in poor condition, the applicant shall overlay the existing pavement with a minimum two inch depth of asphalt. When the existing pavement cross slope is less than 1% or greater than 4%, the existing pavement shall be adjusted as necessary to provide a cross slope that falls within these limits.

On Portland Cement concrete streets, when the existing concrete pavement depth is less than the depth of the designed pavement, and when the panels are in poor condition, the existing pavement shall be replaced. If the existing panels are in good condition they may remain, with new pavement installed to the design depth. Tie bars for longitudinal joints and/or load transfer dowels for transverse joints may be required between new and existing pavement, depending on the condition of the existing pavement. In most cases, tie bars and dowels shall be required between new concrete panels on arterials, commercial access streets, and residential access streets used by trucks and buses

3-2.5.6.1 ROADWAY TAPER

For permanent roadway tapers, the standard taper length for the narrowing from two lanes to one lane or offsetting of a lane is $L = WS^2/60$, where L is the length of taper in feet, W is the lane offset in feet, and S is the design speed in miles per hour.

Temporary asphalt tapers are allowed for narrowing a single lane when additional street improvements are anticipated in the future. The standard taper for temporary tapers is 25:1. See Standard Drawing Number 329.

3-3 EASEMENTS

A nonexclusive easement shall be reserved for and granted to all utilities serving subject plat and their respective successors and assigns, over, under and upon the exterior 10 feet parallel with and adjoining the street frontage of all lots and common areas in which to install, lay, construct, renew, operate and maintain underground conduits, cables, pipes, and wires; together with other necessary facilities and equipment, for the purpose of serving this subdivision and other property with utility service, together with the right to enter upon the lots at all times for the purposes herein stated.

3-4 FIRE DEPARTMENT ACCESS

As required by the Fire Chief, every building constructed shall be accessible to the fire department, both during and after construction, by way of access roadways approved by the fire department. The roadway shall have at least 20 feet of unobstructed width, shall have adequate roadway turning radius, and be capable of supporting the imposed loads of fire apparatus. The minimum allowable vertical clearance shall be 13 feet 6 inches. All required fire access roads must be in service prior to commencement of construction.

When access roads cannot be installed due to topography, waterways, nonnegotiable grades, or other similar conditions, the Chief is authorized to require additional fire protection as specified in Section 10.501(b) of the Uniform Fire Code. Such devices or appliances may consist of automatic fire alarm systems, automatic sprinkler or water spray systems, standpipe and hose, fixed or portable fire extinguishers, suitable fire blankets, breathing apparatus, manual or automatic covers, carbon dioxide, foam, halogenated or dry chemical or other special fire-extinguishing systems. Where such systems are provided, they shall be designed and installed in accordance with the applicable Uniform Fire Code Standards.

The following definitions shall apply:

A. **Fire Lane:** That portion of the fire department access to areas or structures which is required by the provisions of Chapter 19.04.070 of the Snohomish Municipal Code. Generally, this access is in larger complexes and constitutes continuous loops around buildings or complexes.

B. **Fire Access Road:** That portion of the fire department access to areas or structures which is required by the provisions of the current Uniform Fire Code edition or subsequent revision. This type of access may be provided to almost any type of property.

C. **Access Easement:** That portion of a "fire access road" as defined above which is provided by the granting of a permanent easement over one or more properties in order to provide permanent access to other properties.

The required 20 feet of access shall consist of a suitable native or fill material topped with four inches of crushed surfacing top course, compacted to 95% of maximum density on both surfaces. It shall be provided with an approved all season driving surface. A permanent short plat fire access roadway shall be designed to support a minimum of a 40,000 lb vehicle

Temporary access roads in use during building construction shall be constructed for all weather driving conditions. At no time during the construction of the project should the roadway surface consist primarily of dirt, mud, sand, or other material that, in the opinion of the Fire Chief, may impair fire fighting or rescue operations. The required 20 foot width must be maintained so that the driving surface is recognizable day or night.

The required width of any fire apparatus access road shall not be obstructed in any manner, including parking of vehicles. Minimum required widths and clearances established under this section shall be maintained at all times. The required cul-de-sac turnaround for fire apparatus shall be per Standard Drawing No. 304.

The access roadway shall be extended to within 150 feet of all portions of the exterior walls of the first story of any building. Where the access roadway cannot be provided, due to topography, waterways, nonnegotiable grades, or other similar conditions, the City Engineer may request additional fire protection.

More than one fire apparatus road may be requested when it is determined by the City Engineer that access by a single road may be impaired by vehicle congestion, condition of terrain, climatic conditions, or other factors that could limit access.

The requirements of the section may be modified when, in the opinion of the City Engineer or his designee, fire fighting or rescue operation would or would not be impaired.

3-5 STREET PAVEMENT PRESERVATION

No permit shall authorize an open cut crossing of a newly paved or overlaid asphalt street for a period of five years after paving, and directional bore shall be required in such circumstances. The City Engineer or designee may grant exemptions to this requirement in order to facilitate development on adjacent properties, to provide for emergency repairs to subsurface facilities, to provide for underground service connections to adjacent properties or to allow the upgrading of underground utility facilities.

When granting exceptions to this restriction, the City Engineer may impose conditions determined appropriate to insure complete restoration of the street and the surface paving. In addition to complying with the City Engineering Design and Construction Standards, conditions may include ADA improvements, surface grinding, base and sub-base repairs, or other related work as needed, and up to full-width surface paving of the roadway. The length of restoration will be determined by the City Engineer.

In addition to a right of way permit, any person who is required to partially or fully repave a street shall be responsible for the full cost of plan review, construction inspection, material testing, bonding, other permits and all other City fees related to the work.

If the City Engineer determines that final repaving of the street is not appropriate at that particular time for reasons relating to weather or other short term problems, the City Engineer may grant a delay until proper conditions allow for repaving subject to bonding or other acceptable security as deemed appropriate by the City Engineer.

3-6 THROAT LENGTH REQUIREMENTS

The throat length is the unobstructed storage length requirement measured from the inside face of curb to the first driveway or parking stall. Distances may be reduced for multiple driveways as approved by the City Engineer. Minimum throat lengths for each particular land use are as shown on the following table:

MINIMUM THROAT LENGTHS

Land Use	Size		Min Throat Length Collector (ft)	Min Throat Length Arterial (ft)
Light Industrial	<	100,000 sq. ft.	25	50
		100,001-500,000 sq. ft	50	100
	>	500,000 sq. ft.	50	200
Discount Store	<	30,000 sq. ft.	25	50
	>	30,000 sq. ft.	25	75
Shopping Center	<	250,000 sq. ft.	25	50
		250,001-500,000 sq. ft.	50	75
		500,001-750,000 sq. ft.	75	200
	>	750,000 sq. ft.	125	250
Supermarket	<	20,000 sq. ft.	50	75
	>	20,000 sq. ft.	75	125
Apartments	<	50 units	25	25
		50-100 units	25	50
		100-200 units	50	75
	>	200 units	75	125
Quality Restaurant	<	15,000 sq. ft.	25	50
	>	15,000 sq. ft.	25	75
Drive-in Restaurant	<	2,000 sq. ft.	25	75
	>	2,000 sq. ft.	50	100
General Office	<	50,000 sq. ft.	25	50
		50,001-100,000 sq. ft.	25	75
		100,001-200,000 sq. ft.	50	100
		200,001-500,000 sq. ft.	100	150
	>	500,000 sq. ft.	125	250
Motel	<	150 rooms	25	75
	>	150 rooms	25	100

3-7 TRAFFIC CONTROL SIGNING AND STRIPING

All traffic control devices, signing, striping, and other pavement delineation shall conform to the Manual on Uniform Traffic Control Devices (MUTCD). It shall be the developer's responsibility to furnish all materials and labor as required to install all traffic control as required by the City Traffic Engineer. All required signing (traffic control and street name signs), striping, and other delineation as required, shall be shown on the street improvement plans prior to plan approval.

3-8 TRAFFIC STUDIES

A. Responsibility for Traffic Studies

Traffic studies are required by the city for all developments in the City that develop 50 or more "peak hour" trips in either the A.M. or P.M. peak hour, or deemed necessary by the City Engineer. Developments that develop less than the 50 trip threshold have the option of paying a fee of $80.00 per daily trip or preparing a traffic study to outline the impacts and mitigations. The primary responsibility for assessing the traffic impacts associated with a proposed development will rest with the developer and with the City serving in a review capacity.

The study is the responsibility of the applicant and must be prepared by a registered professional engineer with adequate experience in transportation traffic and/or transportation planning.

B. Traffic Study Format

1. Land Use:

   A brief description of the size of the land parcel, general terrain features and location within the city must be included in this section. In addition, the roadways that afford access to the site, and are included in the study area, must also be identified.

2. Existing and Proposed Site Uses:

   The existing and proposed uses of the site must be identified. The intent of the traffic study is to evaluate the traffic impacts due to the development. If the final use is not clear, the land use with the greatest overall traffic impact must be assumed for the study.

3. Project Trip Generation and Distribution:

   The project trip generation is to be determined by the latest approved edition of the ITE Trip Generation Manual, unless otherwise required or approved by the City Engineer.

   The developments peak hour trips are to be distributed through the street network to a level of ten peak hour trips.

4. Existing and Projected Volumes:

   Existing traffic volumes including turning movement counts will be provided by the City when available. Growth shall be calculated at 4% per year compounded annually.

   Separate graphics shall be provided for the existing volumes, for the site generated volumes, and for the cumulative SEPA project volumes. An additional graphic shall be provided compiling all of the information at the horizon year. Volumes including turning movements shall be shown throughout the study area for normal and peak hours.

5. The following Tables should be provided in the study:

Table A

LEVELS OF SERVICE (LOS)				
Intersection	**Existing**	**2004 w/project**	**1998 w/o projects**	**1998 w/mitigations**

Table B

VOLUME					
Roadway Section Intersection	**Project a.m. p.m.** (1)	**Existing a.m. p.m.** (2)	**2004 a.m. p.m.** (3)	**Proj. Vol. as % of Diff. 1/(3-2)**	**Proj. Vol. as % of Total 1/3**

NOTE: The horizon year is normally five years from the date of project application. A horizon year of 2004 is used here for illustrative purposes only.

6. Traffic Accidents:

   Traffic accident information may be required for effected street corridors and intersections. The study period will normally be three years. Information is available from the City.

   Estimates of increased or decreased accident potential must be evaluated for the development particularly if the proposed development might impact existing traffic safety problems in the study area. Safety mitigation measures must be included where necessary.

7. Recommendations:

   In the event that analysis indicates unacceptable Levels of Service (LOS) in the study area, a description and cost estimate of the proposed improvements to return intersection to an acceptable LOS is required. The cost estimates should be all inclusive and include any additional right-of-way as required.

8. The Study shall also analyze the interface of entrances and exits with the City street system. The study shall also make recommendations regarding site circulation for both vehicles and pedestrians including handicap access.

3-9 UNDERGROUND UTILITIES

3-9.1 GENERAL

1. The WSDOT/APWA Standard Specifications shall apply unless otherwise stated below.

2. When trenching through existing pavement, the open cut shall be a neat line made by saw cutting a continuous line. Saw cutting will be required unless the cut is made prior to reconstruction or an overlay.

3. Temporary pavement patch shall be accomplished by using cold mix (MC 250), ATB or steel plates.

4. Permanent pavement patch shall be as specified on Standard Drawing No. 316.

5. Where trench excavation equals or exceeds a depth of four feet, the developer/contractor shall provide, construct, maintain, and remove, as required, safety systems that meet the requirements of the Washington Industrial Safety and Health Act, RCW 49.17, including WAC 296-155. The trench safety systems shall be designed by a qualified person, and meet accepted engineering requirements (see WAC 296-155-660).

6. The developer/contractor shall furnish, install, and operate all necessary equipment to keep excavations above the foundation level free from water during construction, and shall dewater and dispose of the water so as not to cause injury to public or private property or nuisance to the public. Sufficient pumping equipment in good working condition shall be available at all times for all emergencies, including power outage, and shall have available at all times competent workmen for the operation of the pumping equipment.

7. Compaction tests will be required to ensure adequate compaction on all lifts. All compaction tests shall be conducted by a licensed testing laboratory at the expense of the developer/contractor. See Section 3-9.4 of these Specifications.

8. Reference to the City Engineer below means the City's representative on site.

9. Water setting of backfill in trenches is not permitted.

3-9.2 TRENCH EXCAVATION

The length of trench excavation in advance of pipe laying shall be kept to a minimum and in no case shall exceed 150 feet unless specifically authorized by the City Engineer. The maximum permissible trench width between the foundation level to the top of the pipe shall be 40 inches for pipe 15 inches or smaller inside diameter; or 1-1/2 I.D. plus 18 inches for pipe 18 inches or larger. If the maximum trench width is exceeded without written authorization of the City Engineer, the developer/contractor will be required to provide pipe of higher strength classification or to provide a higher class of bedding, as required by the City Engineer.

3-9.3 TRENCH BACKFILL

Suitable native material excavated during trenching shall be used for trench backfill unless notified by the City Engineer that the native material is unsuitable. The City Engineer or his representative will examine excavated native material at the time of excavation to determine its suitability for use as backfill. Native material will be considered suitable for trench backfill if it is:

A. Capable of attaining the degree of compaction specified in Section 3-9.4 Compactions.

B. Within reasonable tolerance of optimum moisture content.

C. Reasonably free of organic material, clay, frozen lumps, rocks, or other deleterious matter.

Unsuitable backfill material shall be removed from the site and hauled to an approved disposal site. The City Engineer shall be provided with the location of all disposal sites to be used and also copies of the permits and approvals for such disposal sites.

Imported material shall meet the requirements of gravel borrow as specified in Section 3-20.2 of these Standards or crushed surfacing top course as specified in Section 9-03.9(3) of the WSDOT/APWA Standard Specifications and Section 3-20.5 of these Standards.

3-9.4 COMPACTION

Trench backfill shall be spread in layers and compacted by mechanical tampers of the impact type approved by the City Engineer. The backfill material shall be placed in successive layers with the first layer not to exceed two feet above the pipe, and the following layers not exceeding 12 inches in loose thickness with each layer being compacted to the density specified below:

A. Improved areas such as street and sidewalks shall be compacted to 90% of maximum dry density to within three feet of subgrade. The last three feet shall be compacted to 95% of maximum dry density.

B. Unimproved area or landscape areas shall be compacted to 90% of maximum dry density.

3-9.5 TRENCHING LONGITUDINAL TO ROADWAY

Sewer, water and storm lines that are within the roadway section and longitudinal to the roadway shall be backfilled with native material or gravel borrow as approved by the City Engineer to the pavement patch level or subgrade whichever applies. All other utility cuts such as gas, telephone, power, and cable TV shall be backfilled with controlled density fill.

3-9.6 TRENCHING TRANSVERSE TO ROADWAY

Utility trenching that crosses transversely to the roadway alignment will generally not be permitted unless it can be shown that alternatives such as jacking, auguring or tunneling are not feasible or unless the utility can be installed just prior to reconstruction or an overlay of the road. Should an open cut be approved, the top four feet minimum of the trench shall be backfilled with **controlled density fill**. When high ground water levels are encountered, relief drains shall be installed at 15 feet intervals to prevent damming. The relief drains shall be three-inch PVC and placed at a minimum three feet from finished grade or as otherwise approved by the City Engineer.

3-9.7 JACKING, AUGERING, OR TUNNELING

Tunneling may be ordered by the City Engineer under pavements, buildings, railroad tracks, etc. The developer/contractor shall install the pipe by jacking, auguring or tunneling, or installing the pipe in a casing pipe by a combination of these methods.

When use of a casing pipe is required, the developer/contractor shall be responsible to select the gauge and size required, unless otherwise indicated on the drawings, and consistent with his jacking or auguring operation, and shall be set to line and grade. During jacking or auguring operations, particular care shall be

exercised to prevent caving ahead of the pipe which will cause voids outside the pipe. When the carrier pipe is installed within a casing pipe, the carrier pipe shall be skidded into position in an acceptable manner and to the line and grade as designated. The annular space between the casing and the pipe shall be filled with controlled density fill or as otherwise approved.

The faces of the jacking pit shall be constructed by driving steel sheets, or installing timber lagging as the excavation proceeds. The sheets, or lagging, shall extend a minimum of five feet below the bottom of the pit except at the entrance of the utility. Prior to jacking or auguring activities, shop drawings describing these activities, including dimensioning of pit length and size of underground borings and complete description of shoring, shall be submitted to the City Engineer for approval.

3-10 SURVEYING AND MONUMENTATION

3-10.1 DESCRIPTION

This work shall consist of all the surveying and monumentation required to construct the project as described in the plans and these Specifications.

It shall be the responsibility of the developer/contractor to furnish materials and install monuments and castings in accordance with the drawings and where directed by the City Engineer. All survey work shall be performed by or under the direct supervision, of a Professional Land Surveyor (PLS) licensed in the State of Washington and using Washington State Coordinate system NAD83 Horizontal, NGVD 1929 vertical datum, North Zone U.S. Foot Coordinate System. Monument and monument case and cover shall be supplied and installed per Standard Plan No. 313.

Surveying, as required to construct a given project per the approved plans, shall be furnished by the developer at no expense to the City. It is required that survey stakes be set for new curb and gutter construction, for both horizontal and vertical control. Additionally, any water, storm drain, or sanitary sewer mains which are to be constructed in easements are to have survey offset stakes set prior to starting that work, and any deviation from that staked line must be left uncovered and resurveyed to realign easement as required and for corrected as-built information.

3-10.2 MATERIALS

Materials for monumentation shall be Class 3000 concrete or commercial concrete per Section 6-02 of the WSDOT/APWA Standard Specifications and the monument case and covers shall be gray iron castings conforming to the requirements of AASHTO M 105, Class 30B. The cover and seat shall be machined so as to have perfect contact around the entire circumference and full width of bearing surface.

3-10.3 CONSTRUCTION REQUIREMENTS

Monuments shall be located at all centerline intersections of intersecting streets. Curved streets shall be monumented at centerline PI's (point of intersection), if it falls within the street pavement, otherwise the PC (point of curvature) and PT (point of tangency) of the curve shall be monumented.

It shall be the developer's/contractor's responsibility to provide the surveying required to establish or perpetuate land corner monumentation as may be required on the project.

All land corner surveying shall conform to the requirements of RCW 58.09. If the developer's or contractor's surveyor replaces or restores an existing or obliterated "General Land Office" (GLO) corner(s), it shall be their responsibility to file "Land Corner Records" for these monuments with the Snohomish County Auditor's Office.

When all land corners have been established, replaced or restored and monumented as described herein, the surveyor shall certify this information with a letter to the City Engineer. This certification letter shall include the location of the monumented corner(s) and that all land corner(s) have been monumented as described and required herein. All land corners and lot corners shall be in place prior to acceptance of final development as-builts and building construction.

The City reserves the right to check survey points and/or the correct locations and elevations of new construction. These spot-checks will not change the requirements for normal checking and testing as described elsewhere, and do not relieve the contractor of the responsibility of producing a finished product that is in accordance with the contract. If unacceptable errors are found due to errors or omissions by the contractor's survey activities, then the contractor shall correct these errors including removing and replacing improvements and pay all expenses incurred by the City including the re-survey.

3-11 STREET ILLUMINATION

Street light design, height and wattage shall be approved by the City Engineer per WSDOT design manual specifications chapter 840.

Typically street lights shall be provided in plats and for commercial developments. Street light poles shall be aluminum with a concrete base. Special ornamental poles may be installed with approval of the City Traffic Engineer. The luminaires are supplied and maintained by the Snohomish County Public Utilities District (PUD). Special luminaires, which are not supplied by the PUD, must be approved by the City Traffic Engineer. All street light wiring, conduit and service connections for new construction shall be located underground, unless otherwise approved by the City Engineer.

3-11.1 PLACEMENT

Street light locations must be approved by the City Engineer. Street lights shall be placed a minimum of 200 feet apart, but no more than 250 feet apart. Every intersection shall have one located as close to the intersection as is practical. In established areas light shall be placed so as to minimize interference from trees or other large objects. Where possible it is desirable to place the light on property lines, and at the edge of City right-of-way.

3-11.2 POWER

In new subdivisions, power will be supplied by the PUD under contract with the developer. All power is to be underground. In existing neighborhoods, power shall be supplied by the best method as approved by the City Engineer

3-11.3 COMMUNITY RELATIONS

In established neighborhoods, the party requesting a new light must contact impacted neighbors to determine interest in having a new street light placed. Neighborhood acceptance of the proposal, including the desired location is important. When the issue is a strong safety one consensus from the neighborhood may not be required. Final determination of the safety merits shall be the responsibility of the City Engineer.

3-11.4 EQUIPMENT

In general, all new City lights shall be mounted on either an existing utility pole or a standard fiberglass pole. Heads shall be either cobra head. Special ornamental poles may be installed with approval of the City Engineer. The luminaires are supplied and maintained by the Snohomish County PUD. Special luminaires, which are not supplied by the P.U.D., must be approved by the City Engineer. The installation of special luminaires, not provided by the P.U.D., shall be the responsibility of the developer in new subdivisions or commercial areas.

Street lights shall be provided in plats and for commercial developments.

All street light wiring, conduit, and service connections shall be typically located underground.

3-11.5 MAINTENANCE

The City is responsible for maintenance of all underground wiring while the overhead wire and the luminaires are maintained by the PUD.

3-12 GUARDRAILS

Unenclosed floor and roof openings, open and glazed sides of stairways, landings and ramps, balconies or porches, which are more than 30 inches above grade or floor below, and roofs used for other than service of the building shall be protected by a guardrail. See Sections 1712 and 3306 of the latest edition of the Uniform Building Code for specific requirements.

For Safety Rail see Section 3-16 of these Standards and Standard Drawing Nos. 325, 325A, and 326.

Roadway guardrail installations shall conform to WSDOT/APWA Standard Plan C-1, Beam Guardrail Type 1. End anchors shall conform to WSDOT/APWA Standard Plan C-6, Beam Guardrail Anchor Type 1.

3-13 MAILBOXES

New residential developments shall have mailboxes installed similar to Standard Plan No. 320, or gang box supplied by the U.S. Postal Service similar to Standard Plan No. 321.

A. When mailboxes are located adjacent to the sidewalk, the sidewalk shall be widened to provide a clear width of not less than five feet from back of curb to any portion of the mailbox structure, per Standard Plan No. 321. If the mailboxes are set adjacent to curb, a bump-out of the curb may be required to provide the required five feet of sidewalk behind the boxes, in lieu of five feet sidewalk routing behind the boxes.

B. In the case of new road construction or reconstruction requiring mailboxes to be moved back or rearranged, the builder shall coordinate with the U.S. Postal Service through the Snohomish Postmaster in the main Post Office in Snohomish, for acceptable box locations and to ensure uninterrupted mail service. Approved locations for mailboxes shall be shown on street construction plans.

3-14 PAVEMENT PATCHING

3-14.1 DESCRIPTION

This work shall consist of the patching of various types of pavement cuts, the performances of which shall be in accordance with these Specifications, the WSDOT/APWA Standard Specifications and Standard Plan No. 316.

3-14.2 MATERIALS

All materials shall conform to the requirements specified for material in other sections of the WSDOT/APWA Standard Specifications as follows:

A. Asphalt concrete pavement patch shall be Class B meeting the requirements of Section 5-04.

B. Asphalt for temporary patch shall be MC 250 meeting the requirements of Section 9-02.

C. Cement concrete pavement patch shall be Class 4000 HES meeting the requirements of Section 6-02.

D. Crushed surfacing top course shall meet the requirements of Section 9-03.9.(3).

3-14.3 CEMENT CONCRETE PAVEMENT RESURFACED WITH ASPHALT CONCRETE

Streets which have cement concrete pavements surfaced with asphalt concrete shall be patched as shown on Standard Drawing No. 316.

The cement concrete portion of the patch shall be Class 4000, HES. The thickness shall be one inch thicker than the existing concrete base or six inches, whichever is greater. The top surface of the concrete patch shall match the top surface of the existing concrete base; in no case shall the top of the concrete be higher than the top of the existing concrete base. Brush finishing will not be required. Joints shall be placed to match existing or as directed by the City Engineer.

Asphalt concrete plant mix shall not be placed until three days after the cement concrete base has been placed or otherwise permitted by the City Engineer. The asphalt concrete plant mix shall not be placed until the concrete base has received a tack coat of CRS-2 at a rate of 0.12 to 0.20 gallons per square yard. The edges of the existing asphalt and castings shall also be painted with the tack coat. The asphalt concrete pavement shall then be placed, leveled, and compacted to conform to the surface of the existing asphalt pavement. Immediately thereafter, all joints between the new and original asphalt pavement shall be painted with CSS-1 asphalt emulsion and covered with dry sand before the asphalt solidifies.

Asphalt shall be compacted to 92% of maximum density as determined by WSDOT Test Method 705.

3-14.4 ASPHALT CONCRETE ON GRANULAR BASE

After the crushed surfacing top course sub-grade has been leveled and compacted, Asphalt Concrete Pavement Class B shall be placed to a thickness of one inch greater than the existing asphalt pavement depth or to a minimum of three inches, whichever is greater. Asphalt shall be compacted to 92% of maximum density as determined by WSDOT Test Method 705.

3-14.5 UNTREATED ROADWAY SURFACES

Existing crushed rock, gravel, and oil mat streets shall be restored with 5/8" crushed surfacing top course to a compacted depth of four inches within the neat lines of the trench. Crushed surfacing shall be mixed, placed, spread, and shaped in accordance with the requirements of Section 4-04 of WSDOT/APWA Standard Specifications. Compaction shall be as specified by one of the methods shown in Section 3-14.7(1) of these Specifications.

3-14.6 TEMPORARY PAVEMENT PATCHING

The contractor shall furnish, place, and maintain temporary pavement patching at locations as directed by the City Engineer until such time as a permanent patch of permanent paving can be made.

Temporary pavement patch shall consist of a two inch thick course of cold mix asphalt (MC 250) over a four inch course of 5/8" crushed surfacing top course. The crushed surfacing shall be compacted to 95% maximum density as determined by one of the methods described in Section 3-14.7(1) of these Specifications. Asphalt shall be compacted to 90 % of maximum density as determined by WSDOT Test Method 705.

Temporary asphalt patching shall be required where roadway or walkway is needed for vehicular or pedestrian traffic, during the construction period, until permanent pavement and sidewalks can be constructed.

In the event that the temporary surface subsides after the initial placement, additional MC 250 and crushed surfacing shall be applied to maintain the surface.

3-14.7 CONSTRUCTION REQUIREMENTS

3-14.7(1) GENERAL

Pavement patching shall be scheduled to accommodate the demands of traffic and shall be performed as rapidly as possible to provide maximum safety and convenience to public traffic.

The placing and compaction of the trench backfill and the preparation and compaction of the subgrade shall be in accordance with the various applicable sections of the WSDOT/APWA Standard Specifications except as modified by these Specifications.

Before the pavement patch is to be constructed the pavement shall be saw cut so that the marginal edges of the patch will form a rectangular shape with straight edges and vertical faces.

Signs, barricades, lights, and other warning devices shall be installed per the requirements of the "Manual on Uniform Traffic Control Devices" and they shall be maintained 24-hours a day until the patching work is completed and ready for traffic.

Compaction of the sub-grade shall be completed prior to the required patching. Subgrade compaction shall be to 95% as determined by one of the following methods:

ASTM D1556	(sand cone method)
ASTM D2167	(rubber balloon method)
ASTM D2922	(nuclear method)

3-14.7(2) CEMENT CONCRETE PAVEMENT

After the crushed surfacing top course sub-grade for the pavement has been constructed and compacted to line and grade, the cement concrete pavement patch shall be placed and struck off to a thickness of one inch greater than the existing pavement or eight inch minimum, whichever is greater. All work shall be in accordance with Section 5-05 of the WSDOT/APWA Standard Specifications, except as modified by these Specifications and Standard Drawing No. 316.

Through joints and dummy joints shall be placed to match existing or as directed by the City Engineer. The surface of the concrete patch shall be finished and brushed with a fiber brush. Approved curing compound shall be placed on the finished concrete immediately after finishing.

3-15 ROCKERIES AND ROCKWALLS

3-15.1 DESCRIPTION

This work shall consist of constructing rockeries with rock facing height of eight feet or less used for erosion control or the containment of cuts and embankments. Work shall be performed in accordance with these Specifications and Standard Plan No. 324. Rockeries over eight feet in height must be designed by a civil engineer licensed in the State of Washington.

3-15.2 MATERIALS

All rock shall be sound, angular ledge rock that is resistant to weathering. The longest dimension of any individual rock should not exceed three times its shortest dimension. Acceptability of rock will be determined by laboratory tests as hereinafter specified, geologic examination and historical usage records. All rock delivered to and incorporated in the project shall meet the following minimum specifications:

A. Absorption ASTM C127 AASHTO T-85	*Not more than 2.0% for igneous and metamorphic rock types and 3.0% for sedimentary rock types.*
B. Accelerated Expansion (15 days) CRD-C-148*1, *2	*Not more than 15% breakdown.*
C. Soundness (MgS04 at 5 cycles) ASTM C88 or CRD-C-137	*Not greater than 5% loss.*
D. Unconfined Compressive Strength *ASTM D 2938*	*Intact strength of 6,000 psi, or greater.*

E. Bulk Specific Gravity (155pcf) ASTM C127 or AASHTO T-85 — *Greater than 2.48*.

*1. The test sample will be prepared and tested in accordance with Corps of Engineers Testing Procedure CRD-C-148, "Method of Testing Stone for Expansive Breakdown on Soaking in Ethylene Glycol".

*2. Accelerated expansion tests should also include analyses of the fractures and veins found in the rock.

The density of the rock shall be equal to or greater than one hundred fifty-five (155) pcf. Typically, rocks used for rock wall construction shall be sized approximately as shown on Standard Plan 324.
Rockery caps will be required on all rockeries higher than four feet in the public right-of-way and optional on private property. The cement concrete cap shall be a minimum of two inches thick. Concrete for rockery cap shall be Class 3000 or Commercial. Lamp black coloring agent to match the color of the rockery shall be added to the cement concrete during mixing in an amount not to exceed 1½ pounds per cubic yard of concrete. Where a pedestrian or ornamental handrail is required, the rockery cap shall be deepened to a minimum of twelve inches for a section six inches either side of each pipe sleeve. Dummy joints shall be constructed at twelve foot intervals. The depth of the dummy joint shall be one-third the depth of the cap.

3-15.3 GENERAL

Surfaces reasonably accessible to pedestrians above and adjacent to rockeries over 30" in height shall be protected by a guardrail conforming to Section 1712 of the Uniform Building Code and to Section 3-16 of these Specifications.

A Public Works permit is required for all rock walls within the public right-of-way and for all those exceeding four
feet in height on private property.

3-15.4 CONSTRUCTION REQUIREMENTS

The first step in rock wall construction, after general excavation, is to construct a keyway of at least twelve inches in depth, extending for the full length of the rock wall. The keyway shall be slightly inclined back towards the face being protected. Once the competency of the keyway subgrade to support the rock wall is verified, a shallow ditch or trench, approximately twelve inches wide and deep, shall be dug along the read edge of the keyway. A four-inch diameter perforated or slotted high-density polyethylene (HDPE) smooth interior pipe shall be placed in the trench. This drain pipe shall be installed with sufficient slope to initiate flow and the outfall connected to a positive and permanent discharge.

The contractor shall have sufficient space available so that he can select from among a number of stockpiled rocks for each space in the rock wall to be filled. Rocks which have shapes which do not match the spaces offered by the previous course of rock should be placed elsewhere to obtain a better fit. Rocks shall be of a generally cubical, tubular, or rectangular shape. Any rocks of basically rounded or tetrahedral form shall be rejected or used for filling large void spaces.

The first course of rock shall be placed on firm unyielding soil. There shall be full contract between the rock and soil, which may require shaping of the ground surface or slamming or dropping the rocks into place

so that the soil foundation conforms to the rock face bearing on it. The bottom of the first course of rock shall be a minimum of twelve inches below the lowest adjacent site grade.

As the rock wall is constructed, the rocks shall be placed so that there are no continuous joint planes in either the vertical or lateral direction. Whenever possible, each rock shall bear on at least two rocks below it. Rocks should be placed so that there is some bearing between flat rock faces rather than on joints. Joints between courses (the top surface or rock) shall slope back towards the cutface and away from the rock wall.

Because of the nature of the product used to construct a rock wall, it is virtually impossible to avoid creating void spaces between individual rocks. Where voids of greater than six inches in dimension exist in the face of a rock wall, they shall be visually examined to determine if contact between the rocks exists within the thickness of the rock wall. If there is no rock contact within the rock wall thickness, the void shall be chinked with a smaller piece of rock.

A rock drainage filter shall be installed between the rear face of the rock wall and the soil face being protected. This drain rock layer shall be at least twelve inches thick. For rock walls eight feet in height or higher, it shall be at least eighteen inches thick. The material for the drainage filter shall be quarry rock, as specified in Section 3-20.7 of these Specifications.

3-16 METAL HAND RAILINGS

3-16.1 DESCRIPTION

This section applies to providing and building metal hand railings that meet the requirements of the Plans, these Specifications and the City Engineer.

3-16.2 MATERIALS

Materials shall meet the requirements of the following:

Ornamental Handrail

Ornamental handrail shall be constructed in accordance with Standard Plan No. 326 and these Specifications. Horizontal rails and vertical support posts shall be 1½ inches by 1½ inches by 1/8 inch tubular steel conforming to ASTM A120. Balusters shall be 1/2 inch by 1/2 inch and the horizontal bottom rail 1-1/2 inches by 1/2 inch by 1/8 inch channel steel (ASTM A120). Vertical support posts shall be a maximum eight feet on center and balusters a maximum four inches clear space. The center of the bottom rail shall be a maximum four inches above finished grade. Finished height of the railing shall be 42 inches above the pedestrian walking surface. Provide slip joints at stairway expansion joints and at 24 feet on center maximum.

Pedestrian Handrail (Galvanized Steel and Aluminum)

Galvanized steel and aluminum pedestrian handrail shall be constructed in accordance with Standard Plan Nos. 325 and 325A and these Specifications. Horizontal rails and vertical support posts shall be 1-1/2 inch diameter Schedule 40 Standard Pipe and balusters shall be 3/4 inch diameter Schedule 40 Standard Pipe. Vertical support posts shall be on eight foot centers and balusters on four inches clear space.

Finished height of the railing shall be 42 inches above the pedestrian walking surface. Provide slip joints at stairway expansion joints and at 24 feet on center maximum.

3-16.3 FABRICATION

Before fabricating the railing, the contractor shall submit six copies of the shop plans for the City Engineer's approval. The contractor may substitute other rail connection details for those shown in the plans if details of these changes show in the shop plans and if the City Engineer approves. In approving shop plans, the City Engineer indicates only that they are adequate and complete enough. Approval does not indicate a check on dimensions.

Welding shall conform to the requirements of the "Structural Welding Code" AWS D1.1 for steel, and to the requirements of the "Specifications for Aluminum Structures" of the Aluminum Association for aluminum alloys. All exposed welds shall be ground flush with adjacent surfaces.

Railing panels shall be straight and true to dimensions. Adjacent railing panels shall align with each other with a variation not to exceed 1/16 inch. Joints shall be match marked.

For structures on curves, either horizontal or vertical, the railing shall conform closely to the curvature of the structure by means of series of short chords. The lengths of the chords shall be the distance center to center of rail posts.

Steel railing units shall be galvanized after fabrication. Zinc used for galvanizing shall be grade Prime Western conforming to ASTM B6 with a minimum two ounces per square foot.

Completed aluminum railing units shall be anodized after fabrication conforming to the requirements of the Aluminum Class 1 Anodic Coating, AA-C22-A41.

Ornamental railing shall be painted with a rust-proof metal primer and one coat of black ornamental iron metal paint.

3-16.4 INSTALLATION

The railing shall be erected in accordance with the plans on anchor bolts, or in holes formed by inserts provided in the concrete railing base to receive the railing posts. Sheet metal inserts shall be removed before the erection of the railing.

No railing shall be erected on the structure until the sidewalk to which it is to be attached is completed and all falsework supporting the system is released.

The railing shall be carefully erected, true to line and grade. Posts and balusters shall be vertical with the direction from the vertical for the full height of the panel not exceeding 1/8 inch.

Slip joints shall be as shown on Standard Drawing Nos. 325, 325A and 326. Railing installed without slip joints will be rejected, and the contractor shall install new railing at his own expense.

3-17 CEMENT CONCRETE SIDEWALKS

3-17.1 DESCRIPTION

This work shall consist of constructing cement concrete sidewalks, thickened edge for sidewalks, curb ramps, and bus shelter pads, including excavation for the depth of the sidewalk and subgrade preparation, in accordance with these Specifications, the WSDOT/APWA Standard Specifications and Standard Drawings Nos. 306, 306A, 310A thru 310C, and 311. Porous concrete sidewalks may be used subject to approval of the City Engineer.

3-17.2 MATERIALS

Materials shall meet the requirements of the following section of the WSDOT/APWA Standard Specifications:

Cement Concrete Class 3000	6-02
Portland Cement	9-01
Aggregates	9-03
Premolded Joint Filler	9-04
Concrete Curing Materials and Admixtures	9-23

Slump of the concrete mix shall not exceed 3-1/2 inches. Lamp black coloring agent for matching the color of newly constructed cement concrete sidewalks to the color of adjacent existing cement concrete sidewalks shall be added to the concrete during mixing in an amount not to exceed 1-1/2 pounds per cubic yard of concrete. No lamp black shall be used in curb ramps.

The use of Calcium Chloride as an admixture is prohibited.

3-17.3 CONSTRUCTION REQUIREMENTS

3-17.3(1) GENERAL

The curb and gutter section shall be placed prior to the placement of the sidewalk section unless otherwise directed by the engineer.

Subgrade shall be approved by the Public Works Inspector prior to concrete being placed. Generally, 1/4 inch V-grooves deep are to be placed on five feet centers, but at the discretion of the inspector, this may be changed to make for a better match with the surrounding area. Expansion joints shall be placed to match those placed in existing curbs if new sidewalk is poured adjacent a curb and gutter, in all other cases the maximum spacing on expansion joints shall be 30 feet center to center. Dummy joints shall be 1/2 inch by 1-1/2 inch on 15 foot centers. Through joints shall be 1/2 inch by four inches.

A minimum distance of three feet is required from the face of curb to any obstruction on or within the sidewalk unless otherwise noted.

It is expected there will be sufficient suitable native material excavated from various portions of the improvement to fill low areas in the sidewalk sub-grade and planting strip area when needed.

Where there is insufficient suitable native material on the project site, the contractor shall furnish, place, and compact gravel borrow. All sidewalks shall be constructed over a minimum two inches of crushed surfacing top course meeting the requirements of Section 3-20.5 of these Specifications and Section 9-03.9(3) of the WSDOT/APWA Standard Specifications compacted to 95% of maximum density.

3-17.3(2) FORMS AND FINE GRADING

Wood forms shall be 2"x4" (nominal) in lengths of not less than ten feet. Steel forms may be used upon approval of the City Engineer. Forms shall be staked to a true line and grade. A sub-grade template shall then be set upon the forms and the fine grading completed so that the sub-grade will be a minimum of 3-5/8 inches below the top of the forms. Forms shall be provided around all street name sign posts and traffic sign posts that are placed in concrete areas. Forms used for this purpose shall be two foot square or two foot minimum diameter cutout, as approved by the City Engineer.

3-17.3(3) PLACING AND FINISHING CONCRETE

The concrete shall be spread uniformly between the forms and thoroughly compacted with a steel shod strike board. Through joints and dummy joints shall be located and constructed in accordance with the Standard Plans. In construction of through joints, the premolded joint filler shall be adequately supported until the concrete is placed on both sides of the joint.

Whenever castings are located in the sidewalk area, joints shall be installed at the casting location to control cracking of the sidewalk. If spacing of joints or scoring is such that installation of joint material would be unsuitable, the contractor shall install rebar to strengthen the sidewalk section.

Dummy joints shall be formed by first cutting a groove in the concrete with a tee bar of a depth equal to, but not greater than the joint filler material, and then working the premolded joint filler into the groove. Premolded joint filler for both through and dummy joints shall be positioned in true alignment at right angles to the line of the sidewalk and be normal to and flush with the surface. Where the sidewalk will be contiguous with the curb, it shall be constructed with a thickened edge as shown on Standard Plan No. 306A.
After the concrete has been thoroughly compacted and leveled, it shall be floated with wood floats and finished at the proper time with a metal float.

The surface shall be brushed with a fiber hair brush of an approved type in a transverse direction except that at driveway and alley crossings it shall be brushed longitudinally. The placing and finishing of all sidewalk shall be performed under the control of the City Engineer, and the tools used shall meet with his approval. After brush finish, the edges of the sidewalk and all joints shall be lightly edged again with an edging tool to give it a finished appearance.

The surface finish and joint pattern may vary at the direction of the City Engineer in order to match existing sidewalk.

3-17.3(4) CURING AND PROTECTION

The curing materials and procedures specified in Section 5-05.3(13) of the WSDOT/APWA Standard Specifications shall prevail, except that white pigment curing compounds shall not be used on sidewalks.

The contractor shall have readily available sufficient protective covering, such as waterproof paper or plastic membrane, to cover the pour of an entire day in event of rain or other unsuitable weather.

The sidewalk shall be protected against damage or defacement of any kind until it has been accepted by the City Engineer. Sidewalk which is not acceptable to the City Engineer because of damage or defacement, shall be removed and replaced by the contractor.

3-17.3(5) CURING AND HOT WEATHER

In periods of low humidity, drying winds, or high temperatures, a fog spray shall be applied to concrete as soon after placement as conditions warrant in order to prevent the formation of shrinkage cracks. The spray shall be continued until conditions permit the application of a liquid curing membrane or other curing media. The City Engineer shall make the decision when the use of a fog spray is necessary.

3-17.3(6) COLD WEATHER WORK

When the air temperature is expected to reach the freezing point during the day or night, the concrete shall be protected from freezing. The contractor shall provide a sufficient supply of straw, hay, grass, earth, blankets, or other suitable blanketing material and spread it over the pavement to a sufficient depth to prevent freezing of the concrete. The contractor shall be responsible for the quality and strength of the concrete thus cured. Any concrete injured by frost action or freezing shall be removed and replaced at the contractor's expense in accordance with these Specifications.

3-17.3(7) THROUGH AND CONTRACTION JOINTS

Standard locations for through joints for sidewalks are:

a) At street margins produced and at 30 foot intervals.

b) To separate concrete driveways, stairways, curb ramps and their landings from sidewalks.

c) Around the vertical barrel of fire hydrants, around utility poles, and large diameter underground utility cover castings when located in the sidewalk area.

d) Longitudinally between concrete walks, curbs, paved planting strips, and solid masonry or concrete walls where they abut.

e) To match as nearly as possible, the through joints in the adjacent pavement and curb when sidewalk abuts curb.

Transverse contraction joints (dummy joints) shall be constructed with premolded material 1/2 inch wide by 2 inches depth, and set at 15 foot intervals, or as decided by the engineer. At no time will dummy joint spacing exceed 15 feet.

Transverse and longitudinal through joints as shown on Standard Plan No. 307 shall be 1/2 inch thickness premolded non-extruding joint material, cut to a width equal to the full depth of the concrete where located, plus 1/2 inch. When installed, they shall be placed with top edge 1/8 inch below the finished surface of the

concrete, in a perpendicular plane to the surface and with the bottom edge embedded in the subgrade. All joints shall be in straight alignment, except where placed in curved locations.

Construction joints for sidewalks shall conform to the applicable requirements for through joints. The top edge shall be 1/8 inch below the finished surface of the sidewalk. At no time will joint spacing exceed 15 feet.

3-17.3(8) CURB RAMPS

In accordance with State law and the American Disability Act Agency Guidelines, curb ramps shall be provided at all pedestrian crossings with curb sections. It is required that when a ramp is constructed giving handicap access to the roadway area, the corresponding ramp at the opposite side of the roadway will also be required. Exact locations at each curb return will be approved in the field during construction.

Curb ramps shall be constructed in accordance with the Standard Plan Nos. 310, 310A, 310B, 310C, and 311. Curb ramps shall be constructed where shown on the plans or as directed by the City Engineer. This work shall include curb ramps installed in new sidewalks and curb ramps to be installed in existing sidewalks. Existing sidewalks shall be neatly saw-cut full depth prior to construction of curb ramps.

Curb ramps shall be constructed separately from the sidewalk to produce a definite break line between the ramp and the sidewalk. A 1/2 inch non-extruded through joint material shall be installed between the curb ramp and the sidewalk with edging.

Ramp texturing is to be done with broom finish. Truncated dome warning devices shall be installed 6” from the curb, 24” deep and painted “safety yellow” per ADAAG 2.9.2, current version and as shown on the above standard plans.

Curb ramps will not be poured integral with sidewalk and shall be isolated by expansion joint material on all sides, but not at end of ramp adjacent to the roadway.

3-18 CURB AND GUTTER

3-18.1 DESCRIPTION

The standard curb and gutter section used in Snohomish shall by Type A-1 per Standard Plan No. 305A. No new curb and gutter is to be placed until forms have been checked and approved for line, grade, and compaction by the Public Works Inspector. Curb cuts or "invisible" curbs may be used subject to approval of the City Engineer.

3-18.2 MATERIALS

Materials shall meet the requirements of the following Sections of the WSDOT/APWA Standard Specifications:

Portland Cement	9-01
Concrete Aggregate	9-03
Reinforcing Steel	9-07
Premolded Joint Filler	9-04
Curing Compounds	9-23

The Portland Cement Concrete shall meet the requirements of Section 5-05 of the WSDOT/APWA Standard Specifications. Concrete mix for curbs shall be Class 3000. Slump of the concrete shall not exceed 3 1/2 inches.

All new curb and gutter shall be placed over not less than two inches of crushed surfacing top course compacted to 95% maximum density.

Forms may be of wood or metal at the option of the contractor, provided that the forms as set will result in a curb, or curb and gutter of the specified thickness, cross section, grade, and alignment shown on the drawings and Standard Plan No. 305A.

3-18.3 PLACING CONCRETE

The subgrade shall be properly compacted and brought to specified grade before placing concrete. The subgrade shall be thoroughly dampened immediately prior to the placement of concrete. Concrete shall be spaded and tamped thoroughly into the forms to provide a dense, compacted concrete free of rock pockets. The exposed surfaces shall be floated, finished, and brushed longitudinally with a fiber hair brush approved by the City Engineer.

The rate of concrete placement shall not exceed the rate at which the various placing and finishing operations can be performed in accordance with these Specifications.

If concrete is to be placed by the extruded method, the contractor shall demonstrate to the satisfaction of the City Engineer that the machine is capable of placing a dense, uniformly compacted concrete to exact section, line, and grade.

3-18.4 CURING

Transparent curing compounds shall be applied to all exposed surfaces immediately after finishing. Transparent curing compounds shall contain a color dye of sufficient strength to render the film distinctly visible on the concrete for a minimum period of four hours after application.

The contractor shall have readily available sufficient protective covering, such as waterproof paper or plastic membrane, to cover the pour of an entire day in event of rain or other unsuitable weather.

Additional requirements for curing in hot weather shall be as specified in Section 3-17.8 of these Specifications. Additional requirements for curing in cold weather may be found in Section 3-17.9 of these Specifications.

3-19 CEMENT CONCRETE DRIVEWAY

3-19.1 DESCRIPTION

This work shall consist of cement concrete driveway and alley returns constructed at the locations shown on the drawings and where directed by the City Engineer, and shall be in accordance with these Specifications, the WSDOT/APWA Standard Specifications and Standard Plan Nos. 307, 308, and 309.

Type 1 driveways should only be used where there is likely to be limited storm water runoff in the gutter line or adequate drainage facilities exist to prevent street storm water from flowing onto adjacent properties.

3-19.2 MATERIALS

Materials shall meet the requirements of the following sections of WSDOT/APWA Standard Specifications:

Portland Cement	9-01
Fine Aggregate	9-03
Coarse Aggregate	9-03
Joint Materials	9-04
Curing and Admixtures	9-23

The concrete mix shall be as specified for Class 3000 and the slump of the concrete shall not exceed three inches. Porous asphalt or porous concrete satisfying the requirements of the adopted Stormwater Management Manual may be used subject to approval of the City Engineer.

A minimum of two inches of crushed surfacing top course shall be compacted to 95% maximum density prior to any placement of concrete.

3-19.3 CONSTRUCTION REQUIREMENTS

3-19.3(1) GENERAL

No driveway approach shall project beyond the extension of the side property line to the curb, unless the owner of the adjacent property is a co-signer of the driveway permit.

There must be at least 20 feet of full height curb between driveways serving any one property frontage.

There must be at least 6 feet of full height curb between driveways on adjacent lots.

Driveway aprons shall be constructed per Standard Plan Nos. 307, 308, or 309 as applicable. The minimum thickness of the driveway apron shall be six inches, placed over a minimum of two inches of crushed surfacing top course compacted to 95% maximum density over a compacted sub-grade. In all cases, subgrade and rock grade shall be approved by the Public Works Inspector prior to concrete being placed. Driveway aprons over 15 feet wide shall have an expansion joint placed in the center of the apron.

In locations where a new driveway is to be constructed and sidewalk and curb and gutter is already existing, it must be totally removed and replaced to driveway standards. It is not permissible to "knock-off" existing curb and install driveway apron, the total curb and gutter section must be removed, either by saw cutting or to the nearest expansion joint, and replaced to driveway standards.

New driveways installed in areas where curb and gutter improvements are not existing, and not required to be installed, shall be paved from the existing edge of pavement to the property line regardless of whether the remainder of the driveway on the private property is paved.

In areas not fully improved with curbs and sidewalks, the elevation of the driveway at the point where it crosses the property line shall not be more than 3 inches higher than the elevation of the centerline of the

existing paved street if the driveway is rising on the private property side and no lower than level with the elevation of the centerline of the existing street if the driveway is going down on the private property side.

3-19.3(2) DRIVEWAY LOCATION

Whenever possible, developments which have frontage on more than one roadway will locate the driveway on the lowest classified roadway, considering sight distance and separation from roadway intersections.

Access to corner lots shall be located on the minor street whenever possible and as close as practicable to the property line most distant from the intersection.

Driveways should be placed directly opposite from each other whenever possible. (More details on this in the county document.)

When allowed, driveways located closer than 100 feet from the approach to an arterial intersection shall be signed and marked "Right Turn Only" unless otherwise approved by the City Engineer.

The City Engineer shall have the authority to restrict the number, size, and location of access driveways. In general, driveways will not be allowed closer than 50 feet to the intersection in residential streets and 100 feet on arterials. The distance of measurement is from the intersection of the straight lines formed by the extension of the curb lines.

3-19.3(3) DRIVEWAY SEPARATION OR SPACING

The desirable driveway spacing for arterial streets is 200 feet or more. The following table shows minimum acceptable driveway spacing.

Roadway Speed(1) (in mph)	Min. Separation(2) Arterial(in feet)	Local
25	105	35
30	125	40
35	150	45
40	185	50
45	230	50
50	275	50

(1) Refers to posted speed.
(2) Between two-way driveways. Distances between adjacent, one-way driveways (with the inbound drive upstream) can be one-half the distances shown above.

3-19.3(4) FREQUENCY OF DRIVEWAYS

The allowable number of driveways for a development is determined based on the following criteria:

- Noncommercial developments receive a maximum of one driveway per lot. The City Engineer may consider requests for more than one driveway based on the following criteria:
  a. The lot is not a corner lot;
  b. No intersection is within 75 feet;

c. The extra access will not cause parking or other complications for the lot or neighboring one; and
d. The lot is located on a local access street with low traffic volume (generally less than 300 vehicles per day);

- Commercial developments receive one driveway per 500 feet of total property frontage, unless approved otherwise by the City Engineer.
- For additional driveways information see County document.

3-19.3(5) DRIVEWAY DESIGN CONSIDERATIONS

The following topics shall be considered when designing new driveways or modifying existing ones and are addressed in detail in the Snohomish County Manual of Standards and Guidelines for Access Management.

A. Location of Bus Pullouts.
B. Entrance Geometrics
   1. Width and Curb Radius
   2. Horizontal Alignment (angle and skew)
   3. Direction (one-way or two-way)
   4. Sight Distance
   5. Channelization
   6. Acceleration and Deceleration Lanes and Tapers.
C. Internal Circulation
   1. Joint Driveways
   2. Exiting Storage
   3. Connecting Roadways and Vehicle Aisles
   4. Parking Coordination
D. On-Street Left Turn Storage
   1. Median Treatment Warrants
   2. Median Structures
   3. Median Barrier Openings
E. Vehicle Guidance
   1. Driveway Curb Marking
   2. Pavement Markings
   3. Signs

Driveway approaches shall be constructed of Portland cement unless otherwise authorized by the City Engineer.

No driveway approach shall project beyond the extension of the side property line to the curb, unless the owner of the adjacent property is a co-signer of the driveway permit.

There must be at least 20 feet of full height curb between driveways serving any one property frontage.

There must be at least 6 feet of full height curb between driveways on adjacent lots.

Egress and ingress shall be from an alley where reasonable. All driveways for commercial purposes must provide sufficient space for vehicles, twenty feet in length to turn around before returning to the

street, unless an exit to an alley or secondary street is provided. In critical on-street parking areas, additional off-street parking space(s) are required for the on-street spaces eliminated by any driveway(s).

3-19.3(6) COMMERCIAL DRIVEWAYS

When allowed, driveways located closer than 100 feet from the approach to an arterial intersection shall be signed and marked "Right Turn Only" unless otherwise approved by the City Traffic Engineer.

A. Commercial Driveway Width:

Street Posted Speed MPH	Driveway Max Width* feet
25	30
26-45	35
Over 45	40

*Dimension "1" on Standard Plan Nos., 307, 308, and 309.

The minimum width of driveway shall be 20 feet.

B. Grade:

The maximum recommended grade is 8%. Vertical curves should be used for smooth transitions at significant grade differentials.

3-19.3(7) RESIDENTIAL DRIVEWAYS

A. **Width:** The maximum width shall be 20 feet at dimension "1" on Standard Plan Nos. 307, 308, & 309. The minimum width shall be 8 feet per SMC 14.235.130(B).

B. **Grade:** The maximum recommended grade is 15%. Grade changes that exceed 16% shall require vertical curves to connect tangents.

3-19.3(8) INTERSECTION TYPE CRITERIA

Private intersection-type driveway openings will be considered in lieu of conventional driveways in commercial areas where the criteria A through D below are met. Meeting the criteria is not a guarantee that an intersection-type driveway will be allowed.

A. Projected driveway usage is greater than 1,000 vehicles per day.
B. The opening is at least 160 feet from any other intersection.
C. The opening is at least 160 feet from any other driveway on the property frontage under control of the applicant.
D. A minimum 100 feet storage area is provided between the curb line on the street and any turning or parking maneuvers within the development.

3-19.3(9) EXCAVATION AND SUBGRADE

Where directed by the City Engineer, unsuitable material in the sub-grade shall be removed to a specific depth and backfilled with select material such as gravel borrow conforming to Section 3-20.2 of these Specifications.

Before any concrete is placed, the contractor shall bring the sub-grade to the required line, grade, and cross-section. The contractor shall maintain the sub-grade in the required condition until the concrete is placed. Compaction shall be to 95% standard density.

3-19.3(10) FORMS AND FINE GRADING

Forms for the straight sections of the driveway or alley return shall have a minimum thickness of two inches and be equal to the nominal depth of the concrete. Plywood or one inch lumber may be used on radii. All forms shall be securely staked and blocked to true line and grade.

A template shall be set upon the forms and the sub-grade shall be fine graded to conform to the required section. The sub-grade shall then be compacted to the approval of the City Engineer. Prior to placement of the concrete, the sub-grade shall be thoroughly dampened.

3-19.3(11) PLACING AND FINISHING

The concrete shall be spread uniformly between the forms and thoroughly compacted with an approved type of strike board. Through joints and contraction joints shall be located and constructed in accordance with the Standard Plans. In the construction of through joints, the premolded joint filler shall be adequately supported until the concrete is placed on both sides of the joint.

Contraction joints (dummy joints) shall be formed with a tee bar by first cutting a groove in the concrete to a depth equal to, but not greater than the joint filler material and then working the premolded joint filler into the groove. Premolded joint filler for both through and dummy joints shall be positioned in true alignment and at right angles to the center line of the driveway or alley return.

After the concrete has been thoroughly compacted and leveled, it shall be floated with wood floats and finished at the proper time with a metal float. Joints shall be edged with 1/4 inch radius edger and the driveway or alley return edges shall be tooled with 1/2 inch radius edger.

The surface shall be brushed in a transverse direction in relation to the center line of the driveway or alley return with a fiber hair brush of approved type.

3-19.3(12) CURING AND PROTECTION

The curing materials and procedures specified in Sections 5-05 and 9-23 of the WSDOT/APWA Standard Specifications and Section 3-17.7 of these Specifications shall be used. The driveway and the alley return shall be protected against damage or defacement of any kind until acceptance by the owner. Any driveway or alley return not acceptable, in the opinion of the City Engineer because of damage or defacement, shall be removed and be replaced by the contractor.

Before placing any concrete, the contractor shall have on the job site enough protective paper to cover the pour of an entire day, in event of rain or other unsuitable weather conditions.

3-20 PROPORTIONING OF MATERIALS

3-20.1 CONTROLLED DENSITY FILL (CDF)

CDF shall conform to the following specifications:

1. Portland Cement: Type I-II ASSHTO M85.

2. Mineral Filler Admixtures: pozzolans or fly ash (ASTM C-618, Class F).

3. Aggregate: Snohomish Washed Coarse Sand No. 2.

CDF shall be used in the following proportions for 1 cubic yard. Batch weights may vary depending on specific weights of aggregates.

Portland Cement	50 lbs/yd^3
Fly Ash	250 lbs/yd^3
Everett No. 2 Washed Coarse Sand(SSD)	3,200 lbs/yd^3
Water 50 gals/yd^3 (Max)	

Add sufficient water to provide a 6 inch to 8 inch slump delivered in place at the job site.

3-20.2 GRAVEL BORROW

The gradation for Gravel Borrow in Section 9-03.14 of the WSDOT/APWA Standard Specifications is superseded by the following:

U.S. Standard Sieve Size	Percent Passing By Dry Weight
3 inch	100
1-1/4 inch	80-100
No. 4	20-70
No. 40	0-25
No. 200	0-5
Sand Equivalent	50 Min.

3-20.3 EVERETT NO. 2 WASHED COARSE SAND

Everett No. 2 Washed Coarse Sand shall be a clean mixture free from organic matter and conforming to the following gradation:

U.S. Standard Sieve Size	Percent Passing By Weight
1/2 inch	100
#4	65-100
#50	0-10
#200	0-3

All percentages are by weight.

3-20.4 SPAWNING GRAVEL

Spawning Gravel shall be clean, well-rounded, uniformly graded, and shall conform to the following gradation:

Sieve Size	Percent Passing
4" Square	100
3" Square	85 - 95
1-1/2" Square	65 - 75
1/2" Square	0 - 50
1/4" Square	2 Max.

All percentages are by weight.

3-20.5 CRUSHED SURFACING

Crushed Surfacing Top Course and Crushed Surfacing Base Course shall meet the requirements of Section 9-03.9(3) of the WSDOT/APWA Standard Specifications including the grading and quality shown.

	Percent Passing	
Sieve Size	Base Course	Top Course
1-1/4" Square	100	
5/8" Square	50 - 80	100
1/4" Square	30 - 50	55 - 75
U.S. No. 40	3 - 18	8 - 24
U.S. No. 200	7.5 Max.	10.0 Max.
% Fracture	75 Min.	75 Min.
Sand Equivalent	40 Min.	40 Min.

All percentages are by weight.

The fracture requirements shall be at least one fractured face and will apply to material retained on each sieve size U.S. No. 10 and above if that sieve retains more than 5% of the total sample.

The portion of Crushed Surfacing retained on a 1/4 inch square sieve shall not contain more than 0.15% wood waste.

3-20.6 FOUNDATION MATERIAL CLASS A

Foundation Material Class A shall meet the requirements of Section 9-03.7 of the WSDOT/APWA Standard Specifications as shown herein:

Sieve Size	Percent Passing
2-1/2" Square	98 - 100
2" Square	92 - 100
1-1/2" Square	72 - 87
1-1/4" Square	58 - 75
3/4" Square	27 - 47
3/8" Square	3 - 14
U.S. No. 4	0 - 1

All percentages are by weight.

3-20.7 QUARRY ROCK

Quarry Rock shall meet the following requirements:

U.S. Standard Sieve Size	Percent Passing By Weight
4"	100
2"	40 maximum
3/4"	10 maximum

All percentages are by weight.

3-20.8 NON-SHRINK CEMENT SAND GROUT

Non-shrink cement sand grout shall be proportioned as follows:

One part high early strength (H.E.S.) cement.
Two parts clean fine-grained sand by weight and well-mixed with sufficient water to obtain a stiff consistency.

Unpolished aluminum powder shall be added to the dry cement in the proportion of one heaping teaspoonful per sack of cement no more than 30-minutes before the grout mixture reaches its final in-place position.

The required strength of the non-shrink concrete or grout shall be fc=4,000 psi and be verified by the cube strength test. The strength shall be confirmed by schmidt hammering of the pads.

Prior to placing the grout, the contact surface shall be thoroughly cleaned, roughened and wetted with water. The grout shall be covered with burlap sacks after the initial concrete set and wetted at regular intervals until the required strength is obtained.

3-20.9 NO. 2 COARSE AGGREGATE

No. 2 Coarse Aggregate shall conform to the following grading [Section 9-3.1(3)C of the WSDOT/APWA Standard Specifications]:

	Percent Passing	
Sieve Size	Minimum	Maximum
1 1/2" Square	100	---
1 1/4" Square	95	100
1" Square	---	---
3/4" Square	40	70
1/2" Square	---	---
3/8" Square	5	20
U.S. No. 4	0	2
U.S. No. 200	0	0.5

DRAWING INDEX

STREETS AND RELATED WORK		Last Revision Date
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301	Typical Roadway Section-Arterials	4-01-04
301a	Bickford Avenue Roadway Section1	4-01-04
301b	Bickford Avenue Roadway Section2	4-01-04
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302	Typical Roadway Section-Local Access Streets	4-01-04
303	Typical Roadway Section-Alley	4-01-04
304	Typical Cul-de-Sac	4-01-04
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	Detached single family, duplex, tri-plex, and four-plex residential.					
PUBLIC STREET CLASSIFICATION	SHORT SUBDIVISION ACCESS (2)	LOCAL ACCESS "A"	LOCAL ACCESS "B"	COLLECTOR ARTERIAL	MINOR ARTERIAL	PRINCIPAL ARTERIAL
Maximum Number of dwelling units serviced (3)	9	40	100	OVER 100	N.A	N.A
Minimum R.O.W	40'	50' (6)	50' (6)	60'	60'	80'
Minimum Pavement Width Curb to Curb	24' (4)	28' (4)	32' (4)	36'	44'	48'
Sidewalks (5)	1 to 4 D.U.-OPTIONAL 5 to 9 D.U.-REQUIRED	Required	Required	Required	Required	Required
Geometrics & Structural Section	Std. Plan # 302	St. Plan # 302	St. Plan # 302	St. Plan # 301	St. Plan # 301	St. Plan # 301
Max. Allowable Grade ***(%) (1)	15%	15%	15%	12%	9%	8%
Utility Easement Beyond R.O.W Req'd	10' Each Side of Public R.O.W.			As Required By City Engineer		

NOTES:

(1) Maximum grade may be exceeded subject to approval by the City Engineer, such approval may be conditional upon the following:

a) No practical alternative exists.
b) Any grade over 15% will be review by the city on a case by case basis.

(2) Can only be used on short plats and cannot be part of a larger development. Must be a permanent dead end.

(3) Maximum potential number of dwelling units served, will include FORECASTED future development of adjacent areas.

(4) 36' wide street section required if less than four(4) off-street parking spaces provided per dwelling unit.
One (1) driveway allowed per lot on "access" streets.

(5) City Engineer may allow sidewalk on one side only in areas of extensive cuts and/or fills and if projected pedestrian volumes are less than normal.

(6) City Engineer may allow variance per Section 1.9

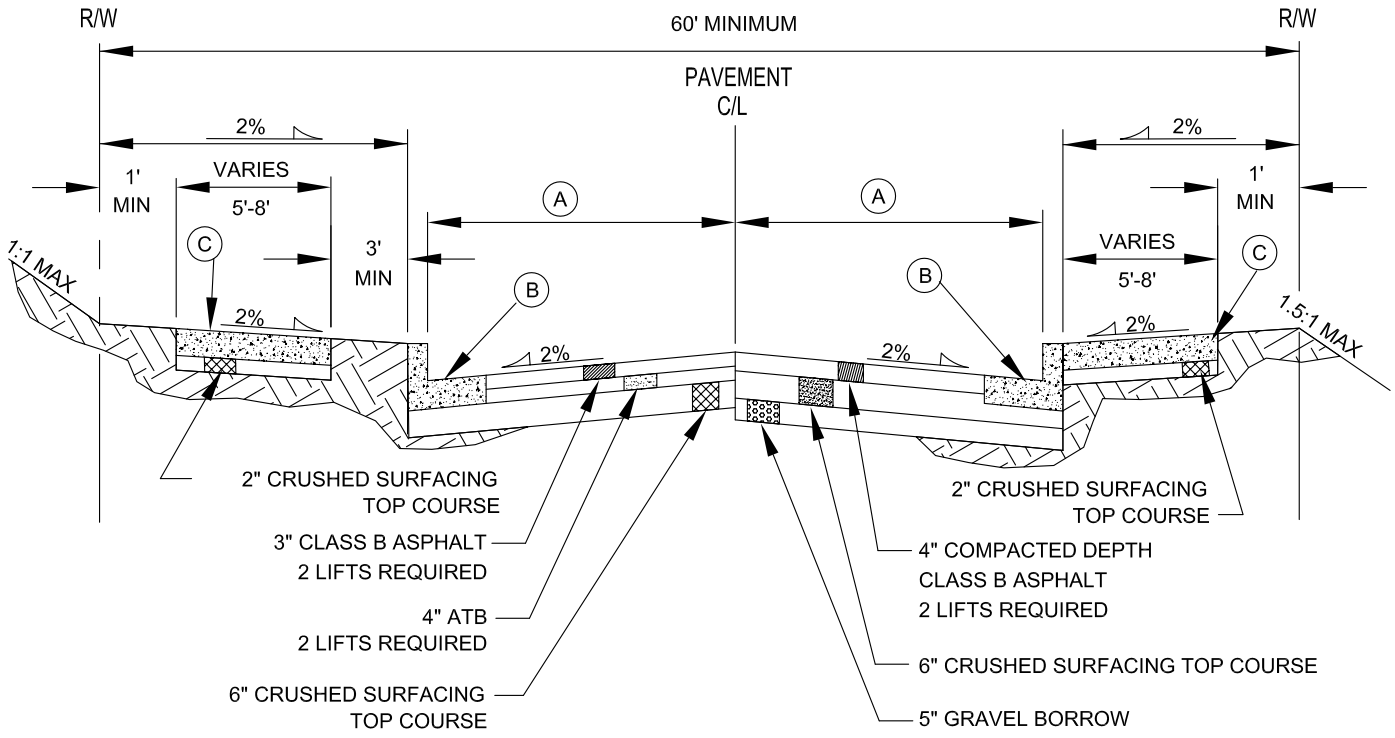
SNOHOMISH WASHINGTON

ROADWAY FUNCTIONAL CLASSIFICATIONS

Approved By:
City Engineer
Date: May 30, 2004

300
Number

City of Snohomish Public Works Department



ALTERNATE ROADWAY SECTION

STANDARD ROADWAY SECTION

(A) PAVEMENT WIDTH

COLLECTOR ARTERIAL = 18'
MINOR ARTERIAL = 22'
PRINCIPAL ARTERIAL = 24+'

(B) CONCRETE CURB AND GUTTER TYPE A-1
SEE STD DWG 305A

(C) CEMENT CONCRETE SIDEWALK
SEE STD DWG 306

NOTES

IN WIDENING AREAS, THE EXISTING PAVEMENT EDGE SHALL BE SAW-CUT TO LEAVE A JOIN POINT. ANY TRAFFIC STRIPING REMOVED OR DAMAGED DURING WIDENING WORK SHALL BE REPLACED IN KIND OR AS DIRECTED BY THE CITY ENGINEER.

COMPACTION TESTS ON SUBGRADE AND TOP OF ROCK WILL BE REQUIRED. THE NUMBER OF TESTS SHALL BE AT THE DISCRETION OF THE CITY INSPECTOR. ALL TESTING SHALL BE THROUGH A LICENSED TESTING LABORATORY. THE MINIMUM COMPACTION SHALL BE 95% OF MAXIMUM DENSITY ON BOTH SUBGRADE AND TOP OF ROCK.

ADJUSTMENT OF CATCH BASIN LIDS OR GRATES, MONUMENTS CASES, VALVE BOXES, ETC SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR OR DEVELOPER.

ROADWAY SECTION MAY BE ADJUSTED WITH THE APPROVAL OF THE CITY ENGINEER UPON SUBMISSION OF SUBSTANTIATING ENGINEERING DATA (CBR, ETC) TO SUPPORT THE ADJUSTMENT. FOR DESIGN PURPOSES, THE MINIMUM THICKNESS OF CLASS B ASPHALT SHALL BE 3" COMPACTED DEPTH. COMPACTION SHALL BE AN AVERAGE OF 91% OF RICE DENSITY.

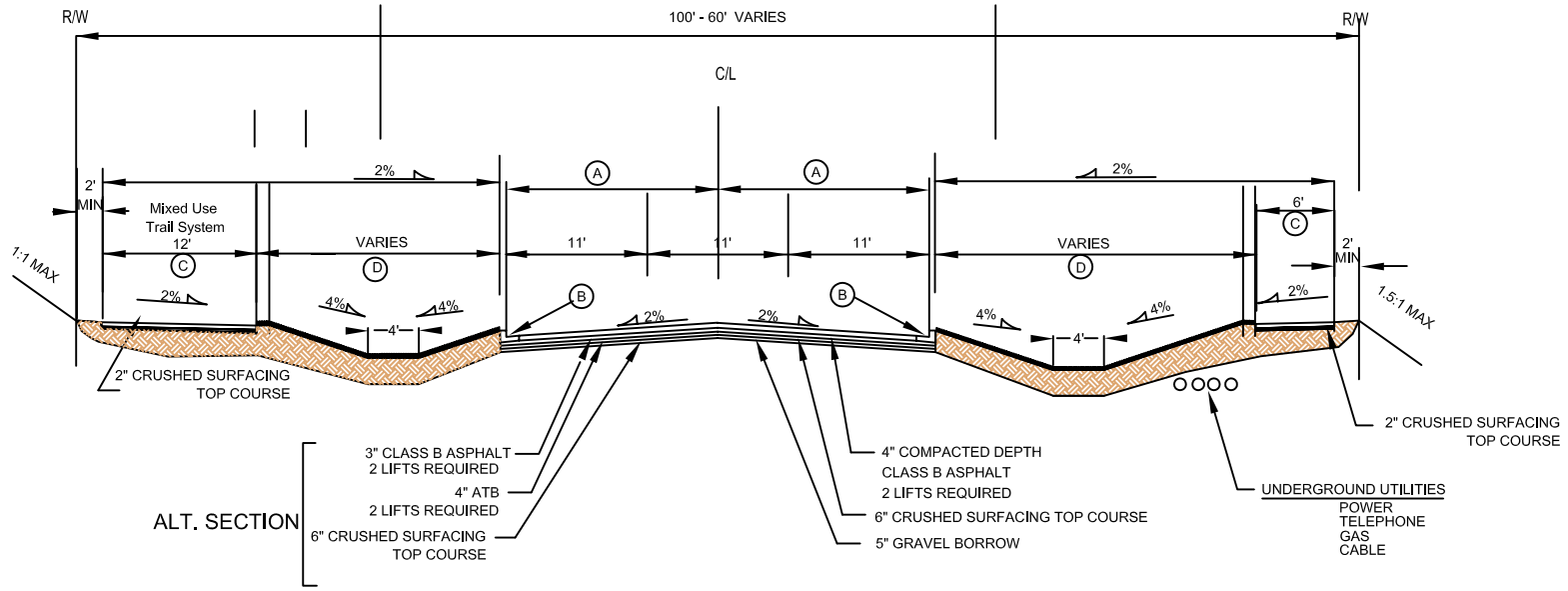
SNOHOMISH
WASHINGTON

TYPICAL ROADWAY SECTION ARTERIALS

Approved By:
City Engineer
Date: May 30, 2004

301
Number

City of Snohomish Public Works Department



BICKFORD AVENUE STANDARD ROADWAY SECTION 1
FACING NORTH

0 5 10

(A) PAVEMENT WIDTH - 16.5'
TRAVELED LANE WIDTH - 11'
CENTER TURN LANE WIDTH - 11'

(B) CONCRETE CURB AND GUTTER TYPE A-1
SEE STD DWG 305A

(C) CEMENT CONCRETE SIDEWALK
SEE STD DWG 306

(D) GRASS SWALED CLEANSING AND INFILTRATION
DRAINAGE AREA

NOTES

IN WIDENING AREAS, THE EXISTING PAVEMENT EDGE SHALL BE SAW-CUT TO LEAVE A JOIN POINT.

ANY TRAFFIC STRIPING REMOVED OR DAMAGED DURING WIDENING WORK SHALL BE REPLACED IN KIND OR AS DIRECTED BY THE CITY ENGINEER.

COMPACTION TESTS ON SUBGRADE AND TOP OF ROCK WILL BE REQUIRED. THE NUMBER OF TESTS SHALL BE AT THE DISCRETION OF THE CITY INSPECTOR. ALL TESTING SHALL BE THROUGH A LICENSED TESTING LABORATORY. THE MINIMUM COMPACTION SHALL BE 95% OF MAXIMUM DENSITY ON BOTH SUBGRADE AND TOP OF ROCK.

ADJUSTMENT OF CATCH BASIN LIDS OR GRATES, MONUMENTS CASES, VALVE BOXES, ETC SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR OR DEVELOPER.

ROADWAY SECTION MAY BE ADJUSTED WITH THE APPROVAL OF THE CITY ENGINEER UPON SUBMISSION OF SUBSTANTIATING ENGINEERING DATA (CBR, ETC) TO SUPPORT THE ADJUSTMENT. FOR DESIGN PURPOSES, THE MINIMUM THICKNESS OF CLASS B ASPHALT SHALL BE 3" COMPACTED DEPTH.

ALL UTIITIES SHALL BE UNDERGROUND AS PART OF DEVELOPMENT FRONTAGE IMPROVEMENTS

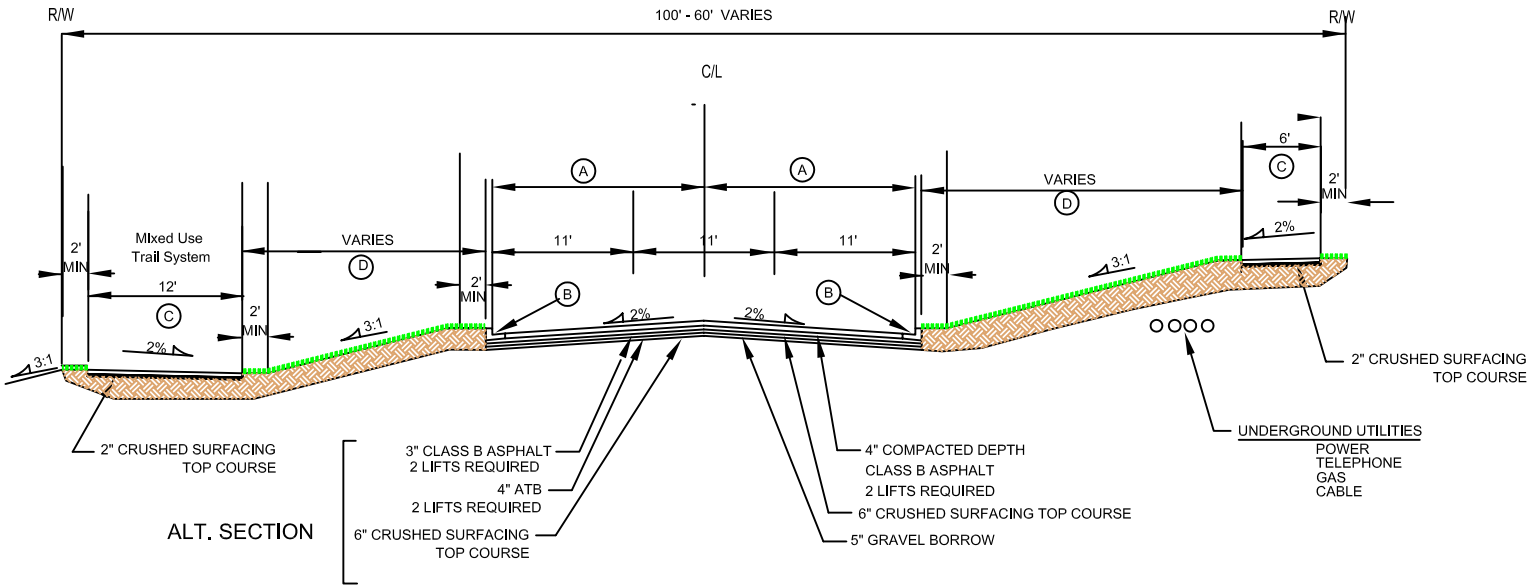
SNOHOMISH
WASHINGTON

Bickford Avenue Roadway Section 1

Approved By:
City Engineer
Date: May 30, 2004

301a
Number

City of Snohomish Public Works Department



BICKFORD AVENUE STANDARD ROADWAY SECTION 2
FACING NORTH

0 5 10

(A) PAVEMENT WIDTH - 16.5'
TRAVELED LANE WIDTH - 11'
CENTER TURN LANE WIDTH - 11'

(B) CONCRETE CURB AND GUTTER TYPE A-1
SEE STD DWG 305A

(C) CEMENT CONCRETE SIDEWALK
SEE STD DWG 306

(D) GRASS SWALED CLEANSING AND INFILTRATION
DRAINAGE AREA

NOTES

IN WIDENING AREAS, THE EXISTING PAVEMENT EDGE SHALL BE SAW-CUT TO LEAVE A JOIN POINT.

ANY TRAFFIC STRIPING REMOVED OR DAMAGED DURING WIDENING WORK SHALL BE REPLACED IN KIND OR AS DIRECTED BY THE CITY ENGINEER.

COMPACTION TESTS ON SUBGRADE AND TOP OF ROCK WILL BE REQUIRED. THE NUMBER OF TESTS SHALL BE AT THE DISCRETION OF THE CITY INSPECTOR. ALL TESTING SHALL BE THROUGH A LICENSED TESTING LABORATORY. THE MINIMUM COMPACTION SHALL BE 95% OF MAXIMUM DENSITY ON BOTH SUBGRADE AND TOP OF ROCK.

ADJUSTMENT OF CATCH BASIN LIDS OR GRATES, MONUMENTS CASES, VALVE BOXES, ETC SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR OR DEVELOPER.

ROADWAY SECTION MAY BE ADJUSTED WITH THE APPROVAL OF THE CITY ENGINEER UPON SUBMISSION OF SUBSTANTIATING ENGINEERING DATA (CBR, ETC) TO SUPPORT THE ADJUSTMENT. FOR DESIGN PURPOSES, THE MINIMUM THICKNESS OF CLASS B ASPHALT SHALL BE 3" COMPACTED DEPTH.

ALL UTILITIES SHALL BE UNDERGROUND AS PART OF DEVELOPMENT FRONTAGE IMPROVEMENTS

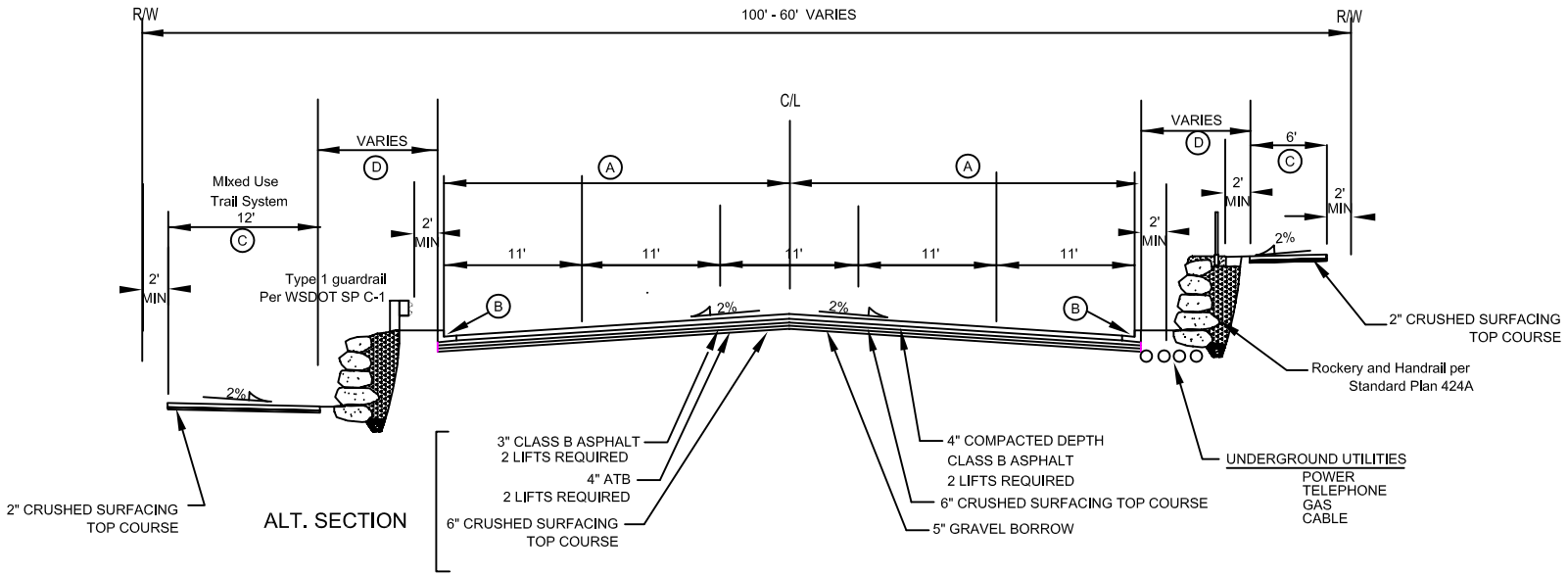
SNOHOMISH
WASHINGTON

Bickford Avenue Roadway Section 2

Approved By:
City Engineer
Date: May 30, 2004

301b
Number

City of Snohomish Public Works Department



BICKFORD AVENUE STANDARD ROADWAY SECTION 3
FACING NORTH

0 5 10

(A) PAVEMENT WIDTH - 27.5'
TRAVELED LANE WIDTH - 11'
CENTER TURN LANE WIDTH - 11'

(B) CONCRETE CURB AND GUTTER TYPE A-1
SEE STD DWG 305A

(C) CEMENT CONCRETE SIDEWALK
SEE STD DWG 306

(D) GRASS SWALED CLEANSING AND INFILTRATION
DRAINAGE AREA

NOTES

IN WIDENING AREAS, THE EXISTING PAVEMENT EDGE SHALL BE SAW-CUT TO LEAVE A JOIN POINT. ANY TRAFFIC STRIPING REMOVED OR DAMAGED DURING WIDENING WORK SHALL BE REPLACED IN KIND OR AS DIRECTED BY THE CITY ENGINEER.

COMPACTION TESTS ON SUBGRADE AND TOP OF ROCK WILL BE REQUIRED. THE NUMBER OF TESTS SHALL BE AT THE DISCRETION OF THE CITY INSPECTOR. ALL TESTING SHALL BE THROUGH A LICENSED TESTING LABORATORY. THE MINIMUM COMPACTION SHALL BE 95% OF MAXIMUM DENSITY ON BOTH SUBGRADE AND TOP OF ROCK.

ADJUSTMENT OF CATCH BASIN LIDS OR GRATES, MONUMENTS CASES, VALVE BOXES, ETC SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR OR DEVELOPER.

ROADWAY SECTION MAY BE ADJUSTED WITH THE APPROVAL OF THE CITY ENGINEER UPON SUBMISSION OF SUBSTANTIATING ENGINEERING DATA (CBR, ETC) TO SUPPORT THE ADJUSTMENT. FOR DESIGN PURPOSES, THE MINIMUM THICKNESS OF CLASS B ASPHALT SHALL BE 3" COMPACTED DEPTH.

ALL UTILITIES SHALL BE UNDERGROUND AS PART OF DEVELOPMENT FRONTAGE IMPROVEMENTS

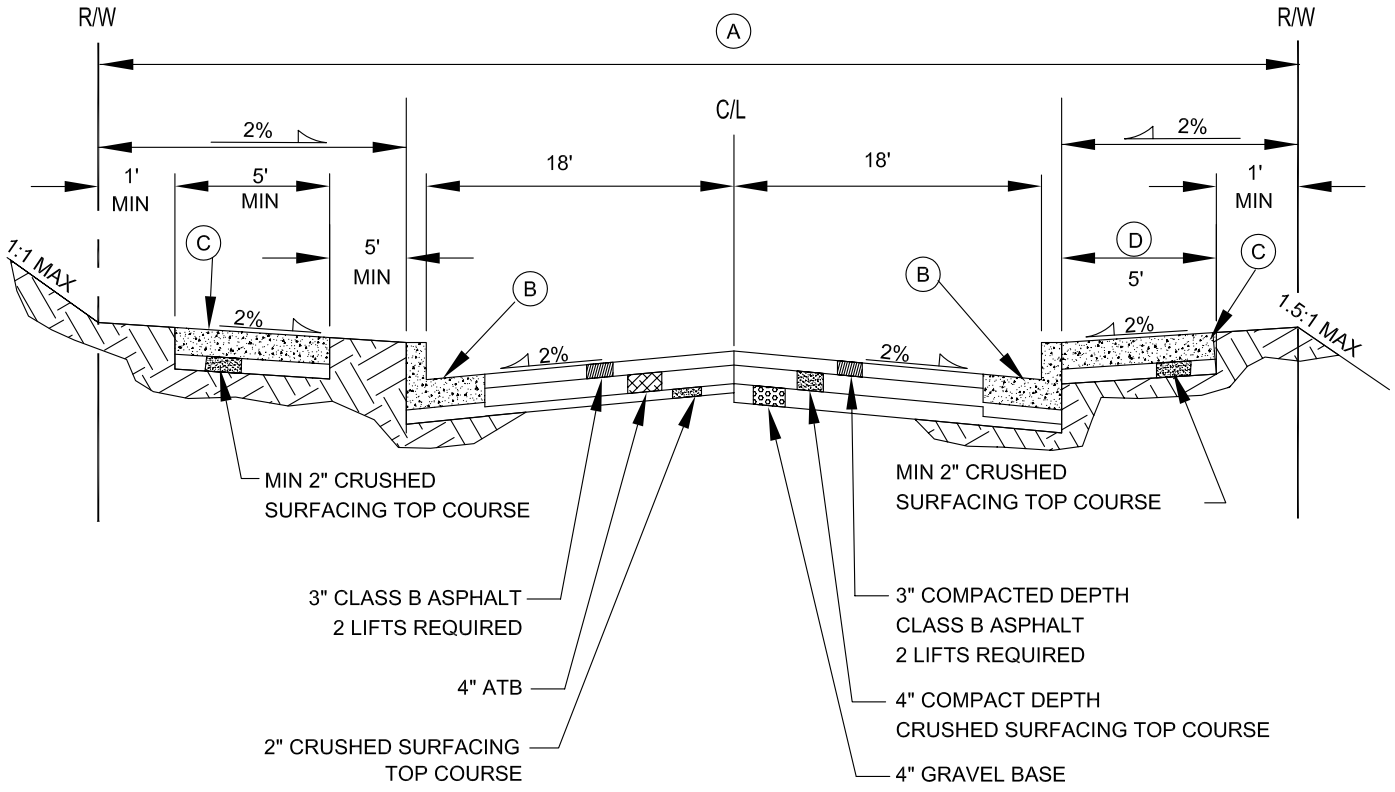
SNOHOMISH
WASHINGTON

Bickford Avenue Roadway Section 3

Approved By:
City Engineer
Date: May 30, 2004

301c
Number

City of Snohomish Public Works Department



ALTERNATE ROADWAY SECTION

(D) VARIANCE MAY BE ADJUSTED FOR A NARROWER RIGHT OF WAY AND THE SIDEWALK LOCATED ADJACENT TO CURB PER SECTION 1.9 IF EXISTING SITE CONSTRAINTS CAUSE EXTREME HARDSHIPS FOR THE WIDER PAVEMENT SECTION AS DETERMINED BY THE PUBLIC WORKS DIRECTOR.

STANDARD ROADWAY SECTION

(A) 60' STANDARD RIGHT OF WAY

(B) CONCRETE CURB AND GUTTER TYPE A-1 SEE STD DWG 305A

(C) CEMENT CONCRETE SIDEWALK SEE STD DWG 306

NOTES:

1 IN WIDENING AREAS, THE EXISTING PAVEMENT EDGE SHALL BE SAW-CUT TO LEAVE A JOIN POINT. ANY TRAFFIC STRIPING REMOVED OR DAMAGED DURING WIDENING WORK SHALL BE REPLACED IN KIND OR AS DIRECTED BY THE CITY ENGINEER.

2 COMPACTION TESTS ON SUBGRADE AND SURFACING SHALL BE REQUIRED. THE NUMBER OF TESTS SHALL BE AT THE DISCRETION OF THE CITY INSPECTOR. ALL TESTING SHALL BE THROUGH A LICENSED TESTING LABORATORY. THE MINIMUM COMPACTION SHALL BE 95% OF MAXIMUM DENSITY ON BOTH SUBGRADE AND SURFACING.

3 ADJUSTMENT OF CATCH BASIN LIDS OR GRATES, MONUMENTS CASES, VALVE BOXES, ETC SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR OR DEVELOPER.

4 ROADWAY SECTION MAY BE ADJUSTED WITH THE APPROVAL OF THE CITY ENGINEER UPON SUBMISSION OF SUBSTANTIATING ENGINEERING DATA (CBR, ETC) TO SUPPORT THE ADJUSTMENT. FOR DESIGN PURPOSES, THE MINIMUM THICKNESS OF CLASS B ASPHALT SHALL BE 3" COMPACTED DEPTH. COMPACTION SHALL BE AN AVERAGE OF 92% OF RICE DENSITY.

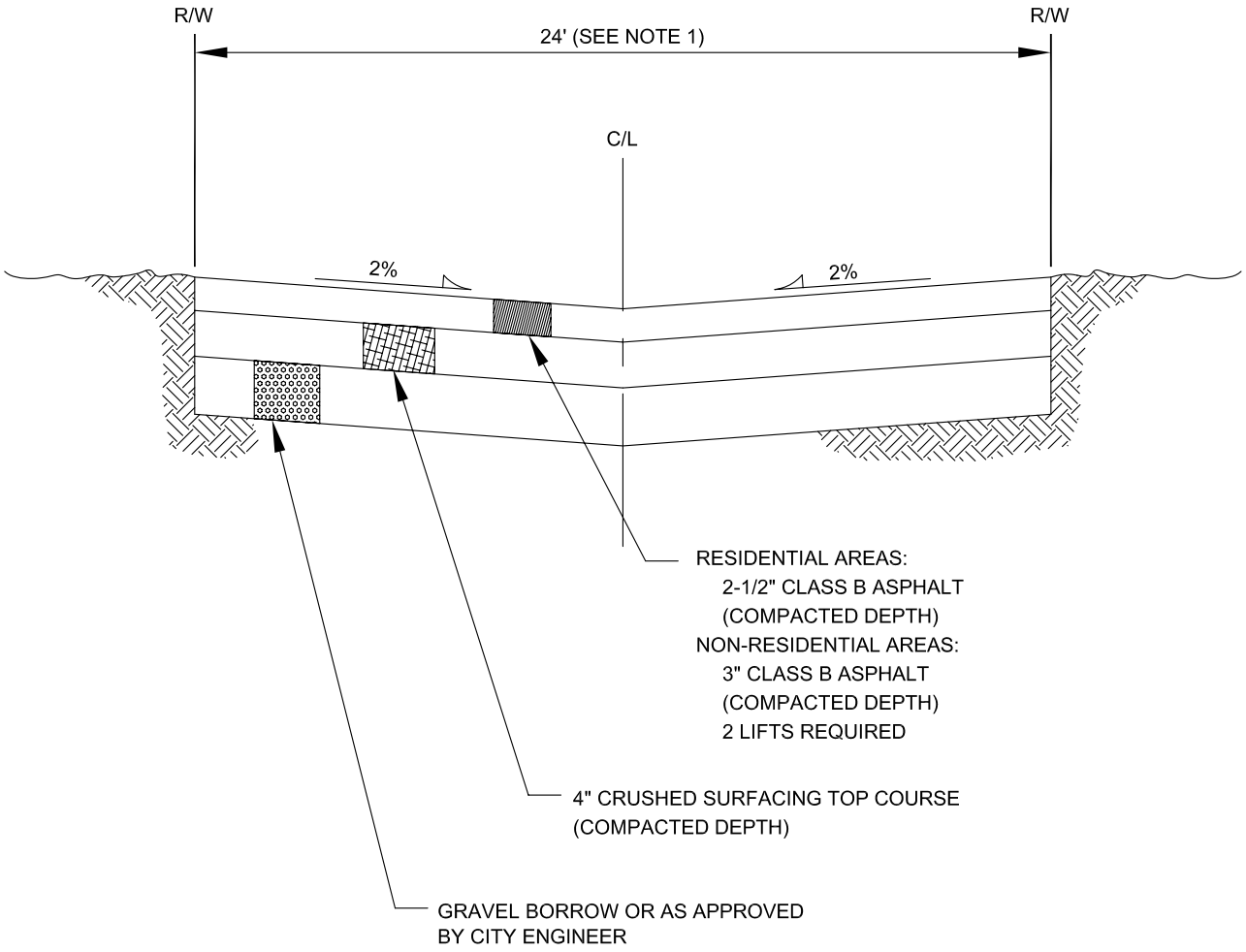
SNOHOMISH WASHINGTON

TYPICAL ROADWAY SECTIONS LOCAL ACCESS STREETS

Approved By:
City Engineer
Date: May 30, 2004

302
Number

City of Snohomish Public Works Department



NOTES

1 ALL NEW ALLEYS SHALL HAVE A MINIMUM WIDTH OF 24'. EXISTING ALLEY RIGHT-OF-WAYS MAY VARY FROM 12' TO 24'.

2 DRAINAGE TO BE COLLECTED AT LOW END OF IMPROVED SECTION WITH CATCH BASIN CONNECTED TO STORM DRAINAGE SYSTEM.

3 COMPACTION TESTS ON SUBGRADE AND TOP OF ROCK WILL BE REQUIRED. THE NUMBER OF TESTS SHALL BE AT THE DISCRETION OF THE CITY ENGINEER. ALL TESTING SHALL BE THROUGH A LICENSED TESTING LABORATORY. THE MINIMUM COMPACTION SHALL BE 95% OF MAXIMUM DENSITY OF BOTH SUBGRADE AND TOP OF ROCK.

4 ADJUSTMENT OF CATCH BASIN LIDS OR GRATES, MONUMENT CASES, VALVE BOXES, ETC SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR OR DEVELOPER AS REQUIRED.

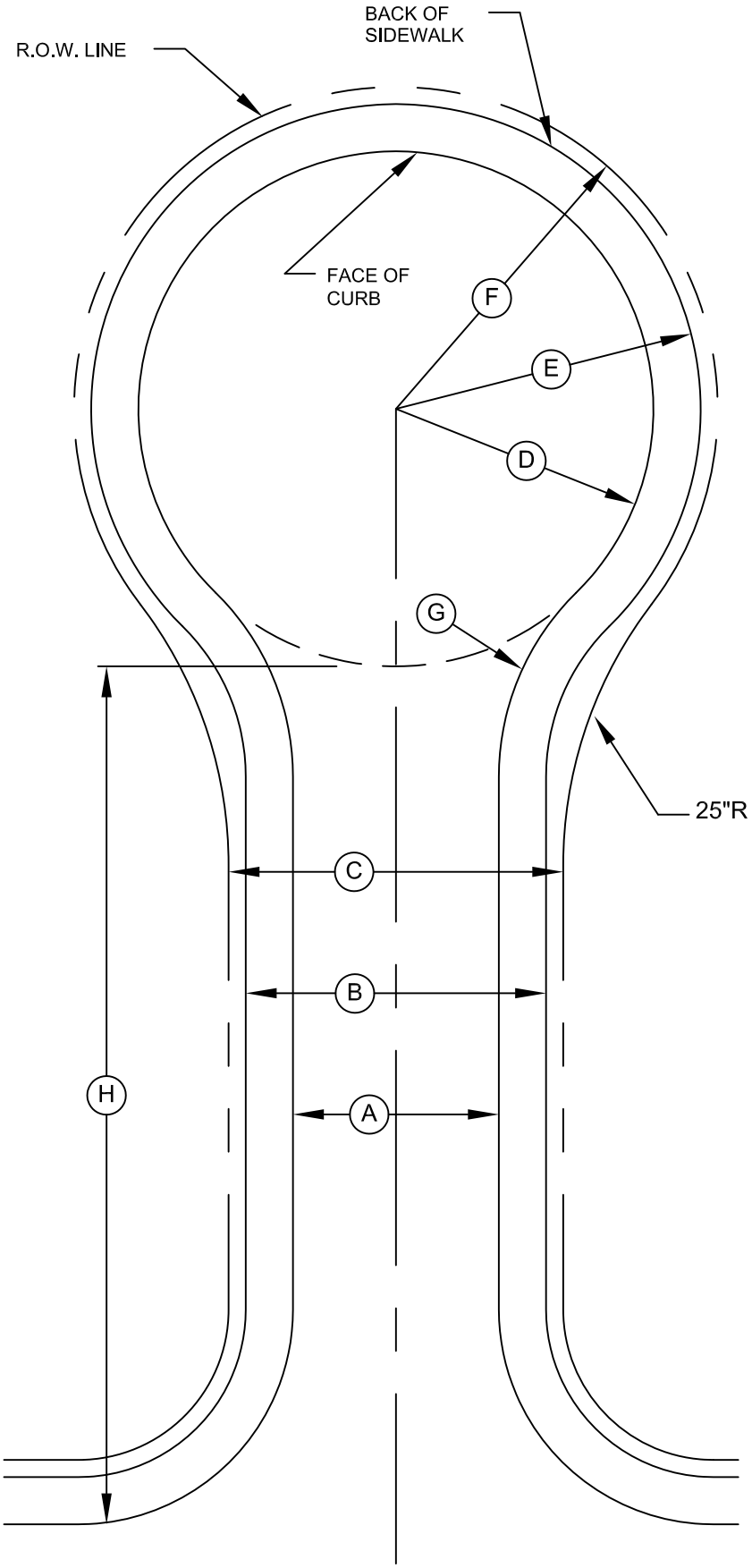
SNOHOMISH
WASHINGTON

TYPICAL ROADWAY SECTION
ALLEY

Approved By:
City Engineer
Date: May 30, 2004

303
Number

City of Snohomish Public Works Department



(A) VARIES 24' TO 48'+
SEE STD PLANS 300, 301 & 302

(B) VARIES 35' TO 59'+
SEE STD PLANS 300, 301 & 302

(C) VARIES 40' TO 80'+
SEE STD PLANS 300, 301 & 302

(D) VARIES 30' TO 45'
PER BELOW:

(H) STREET LENGTH	(D) MIN RADIUS
0'-150'	30'
151'-300'	35'
301'-500'	40'
501'-750'	45'
OVER 750'	SPECIAL APPROVAL REQUIRED

(E) 35.5' MIN. - LOCAL ACCESS STREETS AND SHORT SUBDIVISION STREETS

51.5' MIN. - ARTERIAL CLASSIFICATIONS

(F) 40' MIN. - LOCAL ACCESS STREETS AND SHORT SUBDIVISION STREETS

55' MIN. - ARTERIAL CLASSIFICATIONS

(G) CURB FACE RADIUS TO BE SAME AS RADIUS (D)

NOTE: CENTER LANDSCAPE ISLAND MAY BE ALLOWED. CITY ENGINEER APPROVAL REQUIRED.

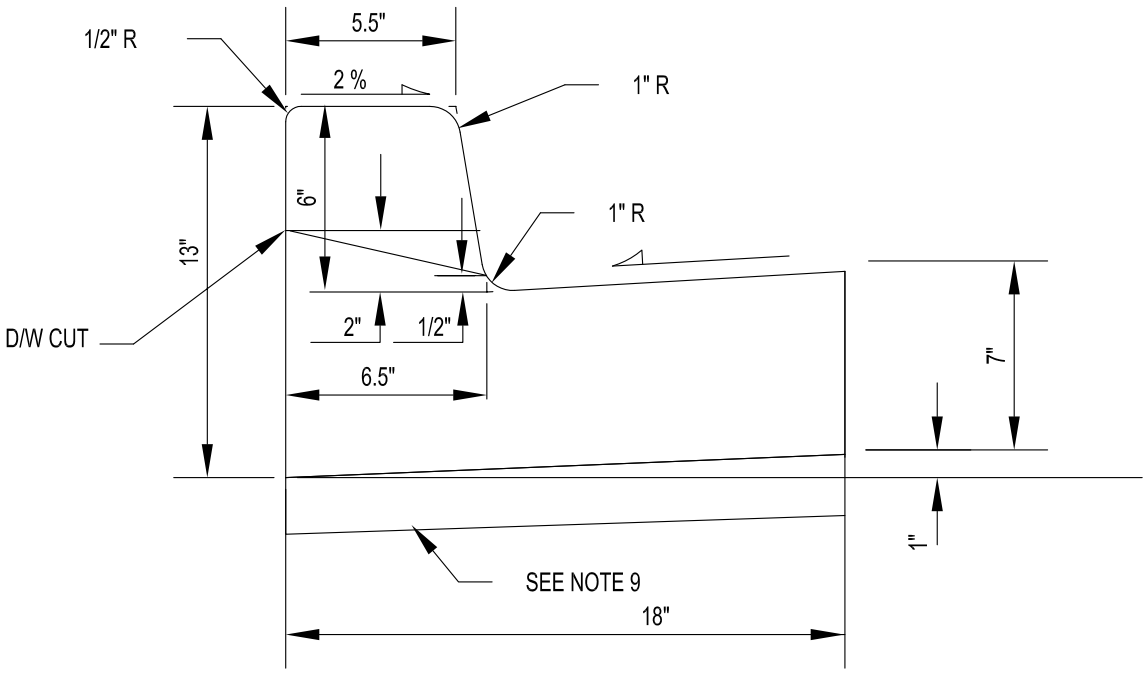
SNOHOMISH WASHINGTON

TYPICAL CUL-DE-SAC

Approved By: Yosh Monzaki
City Engineer
Date: Aug 15, 2016

304
Number

City of Snohomish Public Works Department



TYPICAL SECTION

NOTES

1 FORMS SHALL BE TRUE TO LINE AND GRADE AND SECURELY STAKED.

2 DUMMY JOINTS SHALL BE PLACED ON 15 FOOT CENTERS. DUMMY JOINTS SHALL BE 1/2" x 1-1/2".

3 THRU JOINTS SHALL BE PLACED ADJACENT TO CATCH BASINS, INLETS AND AT POINTS OF TANGENCY ON STREETS, ALLEY AND DRIVEWAY RETURNS. MAXIMUM SPACING SHALL BE 30 FT. PRE-MOLDED JOINT FILLER SHALL BE 1/2" WIIDE AND CONFORM TO AASHTO DESIGN M213.

4 ALL JOINTS SHALL BE CLEAN AND EDGED W/ 1/2" EDGING TOOL.

5 CONCRETE SHALL BE CEMENT CONCRETE, CLASS 3000.

6 STEEL FORMS ONLY SHALL BE USED ON TANGENT SECTIONS. WOOD FORMS MAY BE USED ON CURVED SECTIONS.

7 FINISH SHALL BE LIGHT BROOM FINISH.

8 THE FINISHED CURB SHALL BE SPRAYED WITH A TRANSPARENT CURING COMPOUND AND COVERED BY WATERPROOF PAPER OR PLASTIC MEMBRANE IN THE EVENT OF RAIN OR OTHER UNSUITABLE WEATHER. CURING TIME SHALL BE A MINIMUM OF 72 HOURS.

9 ALL CURB AND GUTTER SHALL BE PLACED ON A MIN OF 2" OF CRUSHED SURFACING TOP COURSE.

10 DUMMY JOINT 1/2" x 1 1/2" BETWEEN A-1 CURB AND GUTTER AND THE SIDEWALK.

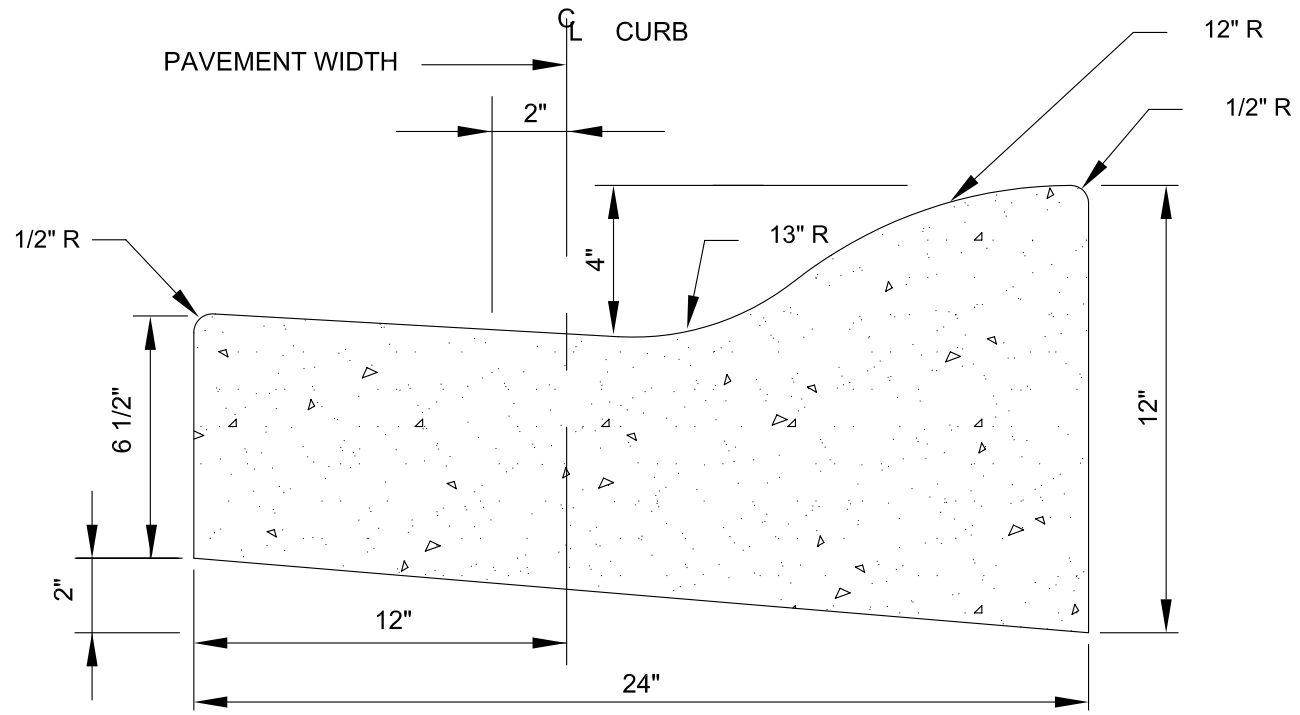
SNOHOMISH
WASHINGTON

CEMENT CONCRETE CURB AND GUTTER TYPE "A"

Approved By:
City Engineer
Date: May 30, 2004

305a
Number

City of Snohomish Public Works Department



INDUSTRIAL USE ONLY

NOTES

1. ROLLED CURB AND GUTTER MAY ONLY BE USED IN HIGHLY INDUSTRIALIZED AREAS AND ONLY WITH WRITTEN APPROVAL OF THE CITY ENGINEER.
2. FORMS SHALL BE TRUE TO LINE AND GRADE AND SECURELY STAKED.
3. THRU JOINTS SHALL BE PLACED ADJACENT TO CATCH BASINS, INLETS, CURB RETURNS, ALLEYS, OR A MAXIMUM SPACING OF 30 FEET.
4. DUMMY JOINTS SHALL BE PLACED EVERY 15 FEET. DUMMY JOINTS SHALL BE 1/2" x 1 1/2".
5. THRU JOINTS SHALL BE 1/2" WIDE PRE-MOLDED JOINT FILLER.
6. ALL JOINTS SHALL BE CLEANED AND EDGED.
7. CONCRETE SHALL BE CEMENT CONCRETE CLASS 3000.
8. STEEL FORMS ONLY MAY BE USED ON TANGENT SECTIONS, WOOD FORMS MAY BE USED ON CURVED SECTIONS.
9. FINISH SHALL BE LIGHT BROOM.
10. CURB IS TO BE SPRAYED WITH TRANSPARENT CURING COMPOUND.
11. ALL SIDEWALKS POURED BEHIND ROLL CURB IN INDUSTRIAL APPLICATIONS SHALL BE SIX INCHES (6") THICK OVER 2" OF CRUSHED SURFACING TOP COURSE WITH TOP OF ROCK COMPACTED TO 95% OF MAXIMUM DENSITY.

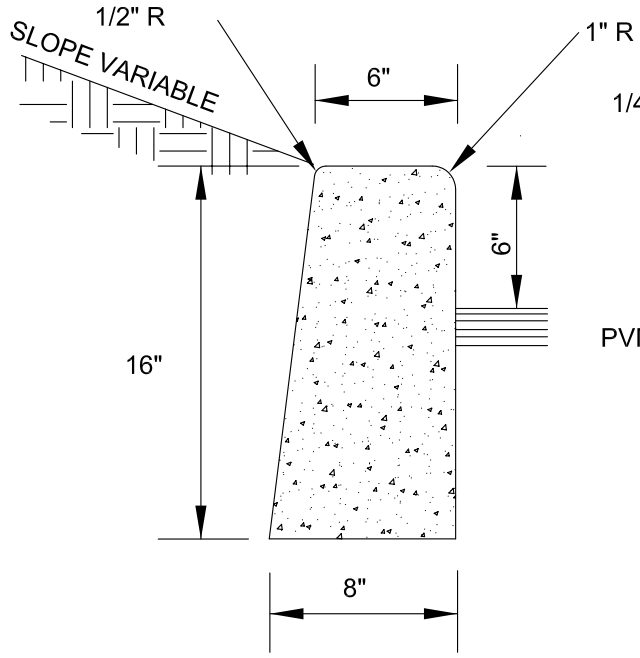
SNOHOMISH WASHINGTON

CEMENT CONCRETE ROLLED CURB AND GUTTER

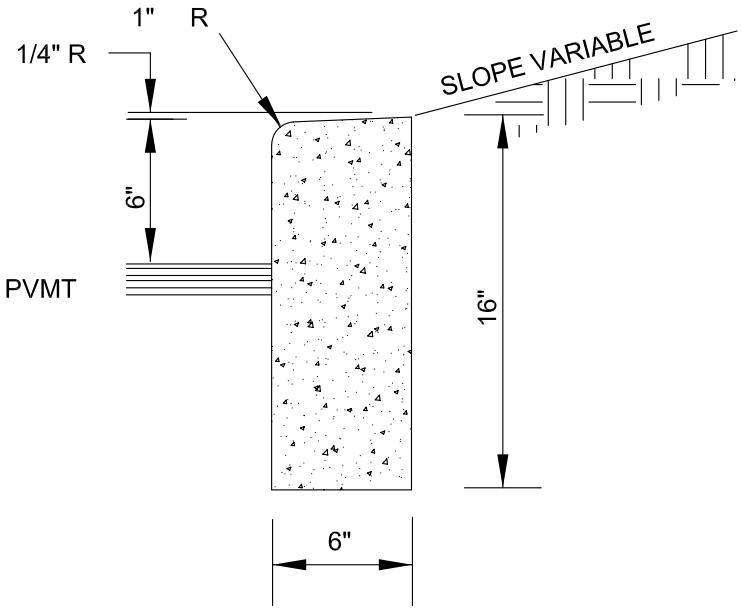
Approved By:
City Engineer
Date: May 30, 2004

305b
Number

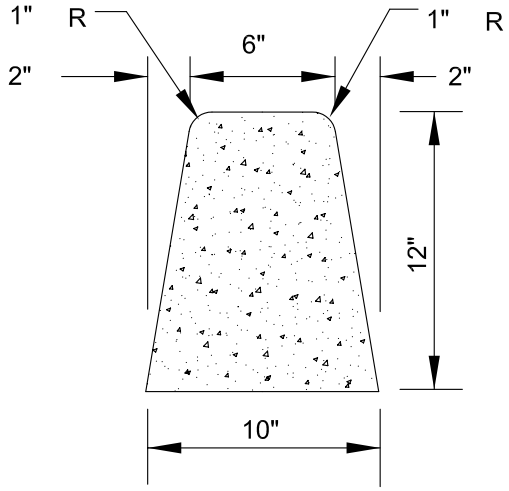
City of Snohomish Public Works Department



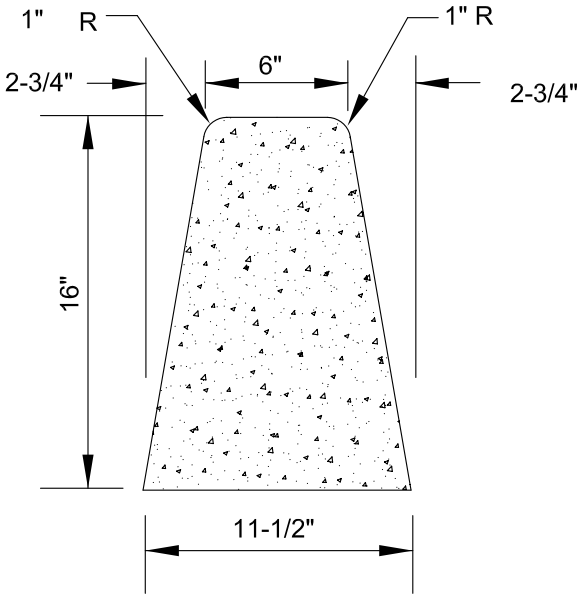
TYPE E-1 CURB



TYPE E-2 CURB



TYPE E-3 CURB



TYPE E-4 CURB

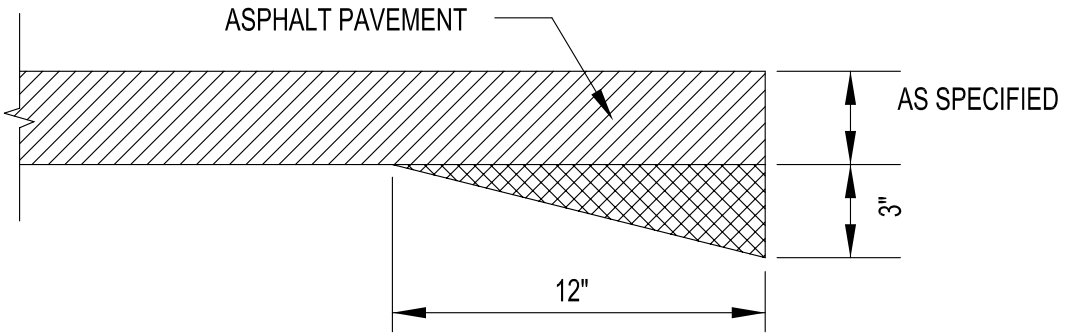
SNOHOMISH
WASHINGTON

CEMENT CONCRETE CURB
TYPE: E1, E1,E3 AND E4

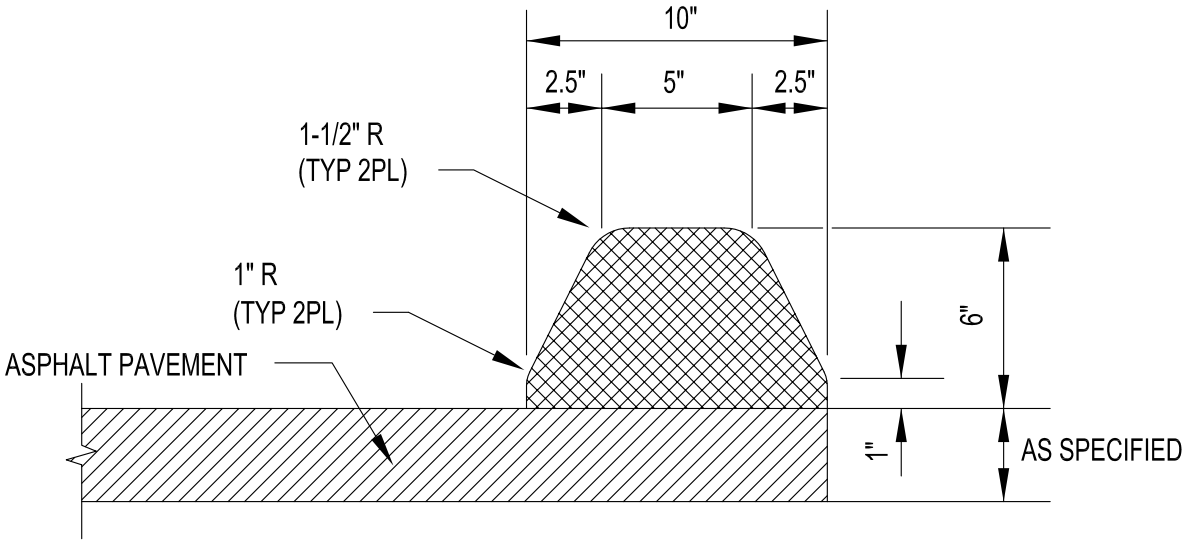
Approved By:
City Engineer
Date: May 30, 2004

305c
Number

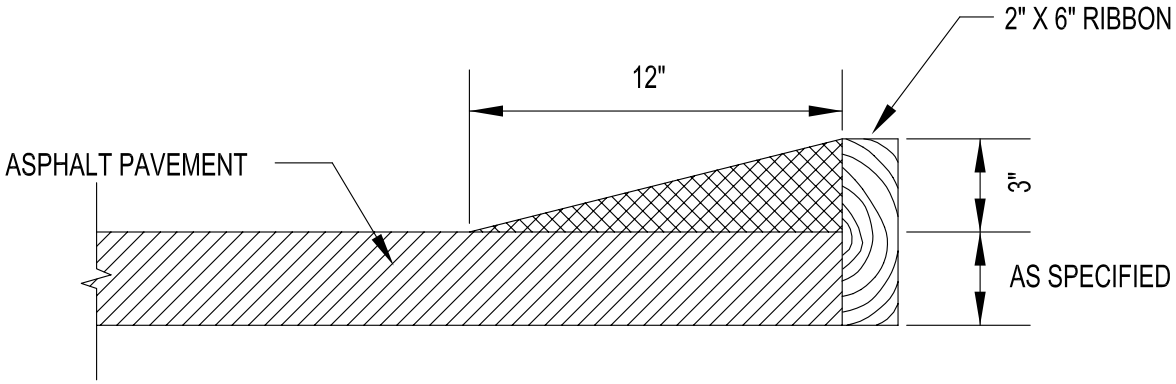
City of Snohomish Public Works Department



THICKER EDGE FOR ASPHALT PAVEMENT



EXTRUDED ASPHALT CONCRETE CURB



ASPHALT WEDGE CURB

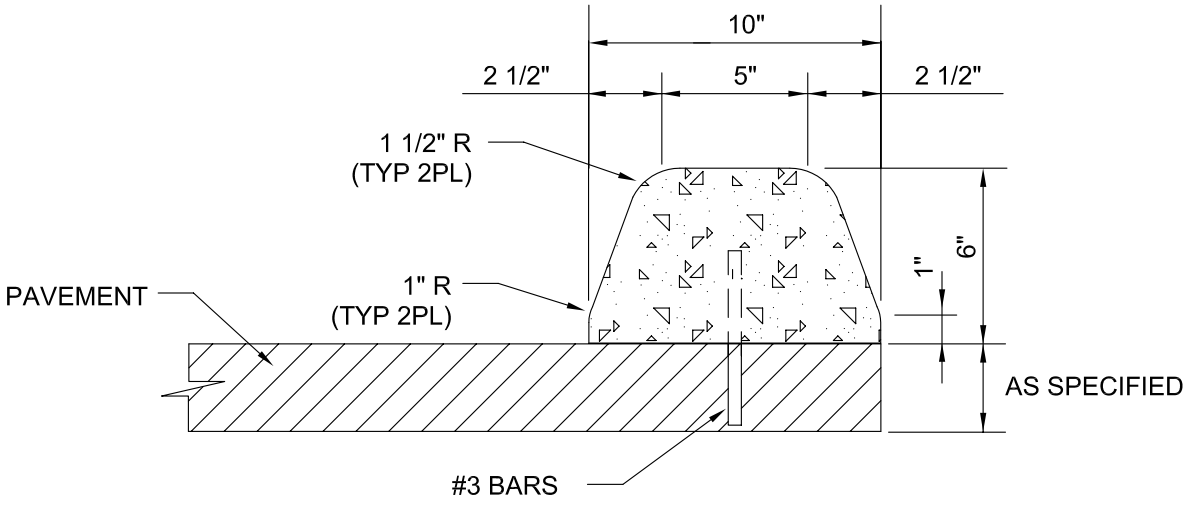
SNOHOMISH
WASHINGTON

EXTRUDED ASPHALT CONCRETE SECTION

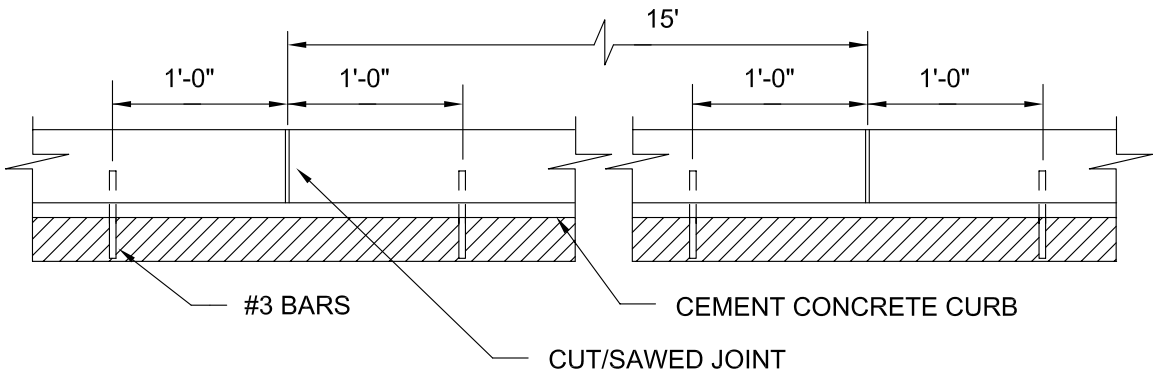
Approved By:
City Engineer
Date: May 30, 2004

305d
Number

City of Snohomish Public Works Department



EXTRUDED CEMENT CONCRETE CURB



SPACING OF ANCHOR BARS

NOTES:

1. DUMMY JOINTS SHALL BE PLACED NOT TO EXCEED 15'CLS. THRU JOINTS SHALL BE PLACED ONLY AT POINTS OF TANGENCY ON STREET ALLEY AND DRIVEWAY RETURNS AND WHERE THRU JOINTS OCCUR IN THE PAVEMENT SLAB.

2. CONCRETE SHALL BE CLASS 3000 OR COMMERCIAL WITH AIR-ENTRAINMENT.

3. AT THE CONTRACTOR'S OPTION CONCRETE CURBS MAY BE ANCHORED TO THE EXISTING PAVEMENT EITHER BY PLACING STEEL TIE BARS 1 FOOT ON EACH SIDE OF EVERY JOINT, OR BY USING AN ADHESIVE. THE ADHESIVE SHALL MEET THE REQUIREMENTS OF SECTION 9-20 OF THE WSDOT/APWA STANDARD SPECIFICATIONS FOR TYPE II EPOXY RESIN.

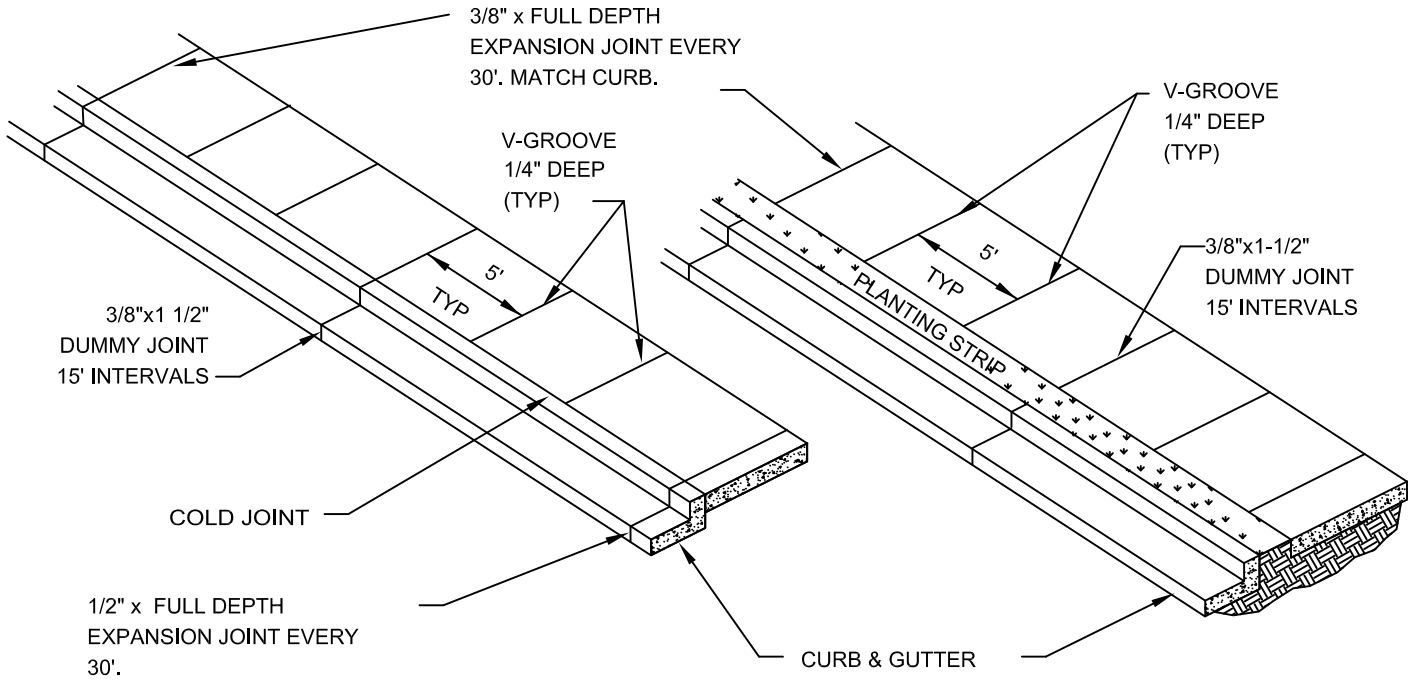
SNOHOMISH
WASHINGTON

EXTRUDED CEMENT CONCRETE CURB

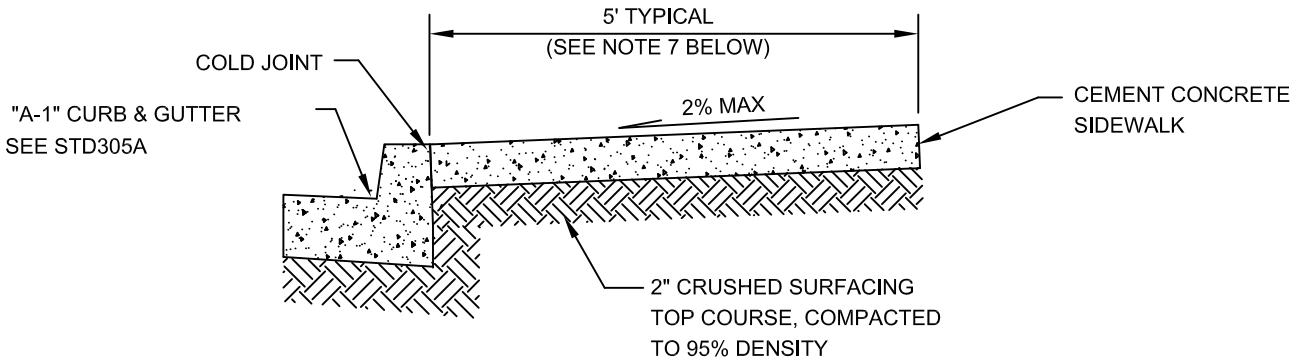
Approved By:
City Engineer
Date: May 30, 2004

305e
Number

City of Snohomish Public Works Department



PLAN VIEWS



TYPICAL SECTION

NOTES

1. SIDEWALKS SHALL BE A MINIMUM OF 4" THICK, AND SHALL BE CLASS 3000 CEMENT CONCRETE, WITH AIR ENTRAINMENT (MIN 4.5 %, MAX 6.5 %).
2. FULL EXPANSION JOINTS SHALL GENERALLY BE PLACED TO MATCH THOSE PLACED IN ADJACENT CURB & GUTTER, WITH MAXIMUM SPACING OF 30 FEET, FINAL SPACING DETERMINATION SHALL BE DECIDED BY THE INSPECTOR IN THE FIELD.
3. SUBGRADE SHALL BE COMPACTED TO NOT LESS THAN 95% OF MAXIMUM DENSITY.
4. SIDEWALK SHALL BE AT LEAST 6" THICK IN DRIVEWAYS AND BEHIND ROLL-CURB.
5. THE FINISHED SIDEWALK SHALL BE SPRAYED WITH A TRANSPARENT CURING COMPOUND COVERED BY WATERPROOF PAPER OR PLASTIC SHEETING IN THE EVENT OF RAIN OR OTHER INCLEMENT WEATHER. CURING TIME SHALL BE FOR A MINIMUM OF 72 HOURS.
6. ALL JOINTS SHALL BE CLEANED AND EDGED WITH AN EDGER HAVING A 1/4" RADIUS.
7. SIDEWALKS ARE TYPICALLY 5' WIDE, EXCEPT 6' IN COMMERCIAL AREAS, OR AS APPROVED BY THE CITY ENGINEER.
8. POROUS CONCRETE SIDEWALK MAY BE USED SUBJECT TO APPROVAL OF THE CITY ENGINEER.

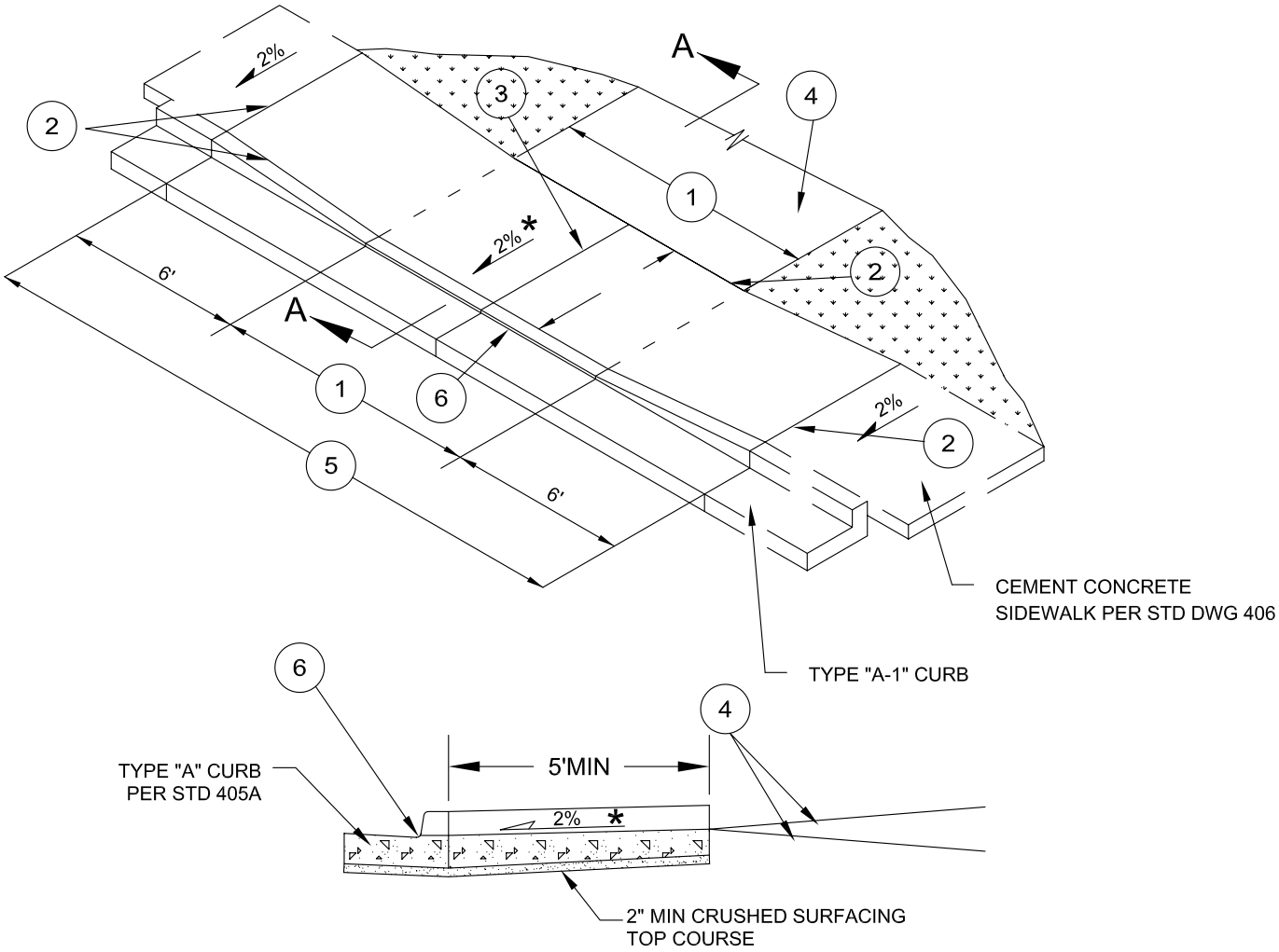
SNOHOMISH
WASHINGTON

CEMENT CONCRETE SIDEWALK DETAIL

Approved By:
Yosh Monzaki
City Engineer
Date: Aug 15, 2016

306
Number

City of Snohomish Public Works Department



SECTION A-A

* Maximum 8% slope in commercial
Maximum 15% slope in residential

NOTES

1. EQUALS WIDTH OF DRIVEWAY AT PROPERTY LINE.
2. 3/8" WIDE FULL DEPTH EXPANSION JOINT.
3. 3/8" WIDE FULL DEPTH EXPANSION JOINT IF (1) IS 15' OR GREATER.
4. DRIVEWAY TO BE SURFACED WITH ASPHALT OR CONCRETE.
5. DRIVEWAY CEMENT CONCRETE SHALL BE A MIN OF 6" THICK INCLUDING WING RAMPS AND PLACED ON A MINIMUM OF 2" CRUSHED SURFACING TOP COURSE COMPACTED TO 95% MAXIMUM DENSITY.
6. SEE DWG. 405A

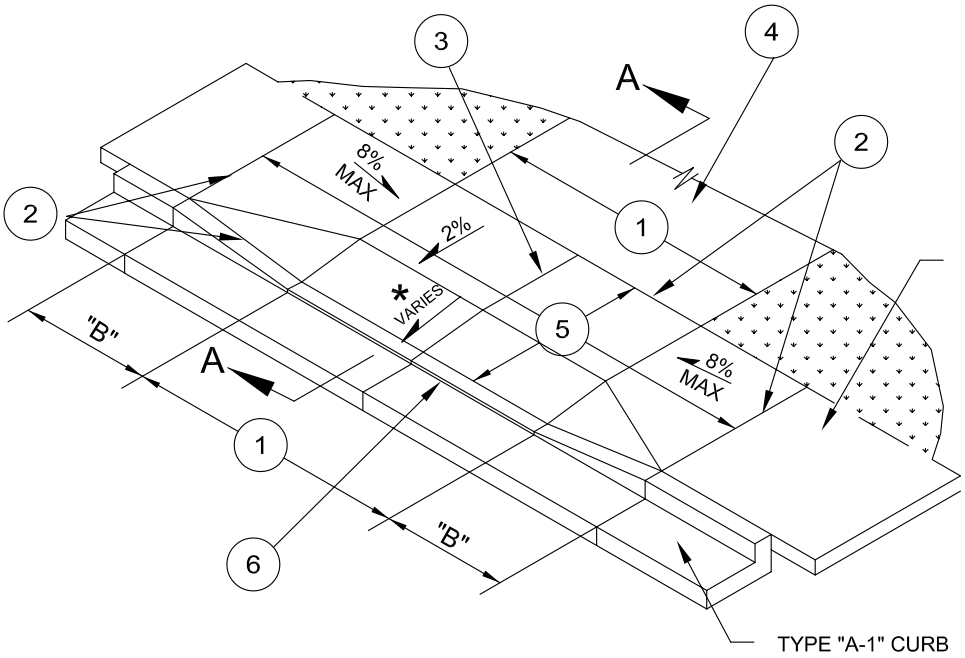
SNOHOMISH
WASHINGTON

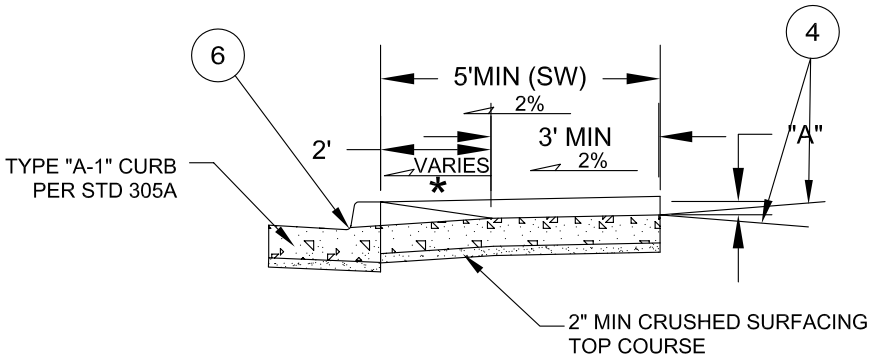
CEMENT CONCRETE DRIVEWAY TYPE 1

Approved By:
City Engineer
Date: May 30, 2004

307
Number

City of Snohomish Public Works Department





SECTION A-A

LOWERING BACK OF SIDEWALK "A" (IN)	LENGTH OF TRANSITION "B" (FT)
3	3
4	4
5	5

* Maximum 8% slope in commercial
Maximum 15% slope in residential

NOTES

1. EQUALS WIDTH OF DRIVEWAY AT PROPERTY LINE.
2. 3/8" WIDE FULL DEPTH EXPANSION JOINT.
3. 3/8" WIDE FULL DEPTH EXPANSION JOINT IF (1) IS 15' OR GREATER.
4. DRIVEWAY TO BE SURFACED WITH ASPHALT OR CONCRETE.
5. DRIVEWAY CEMENT CONCRETE SHALL BE A MIN OF 6" THICK INCLUDING WING RAMPS AND PLACED ON A MINIMUM OF 2" CRUSHED SURFACING TOP COURSE COMPACTED TO 95% MAXIMUM DENSITY.
6. MAINTAIN 1/2" LIP AT GUTTER.

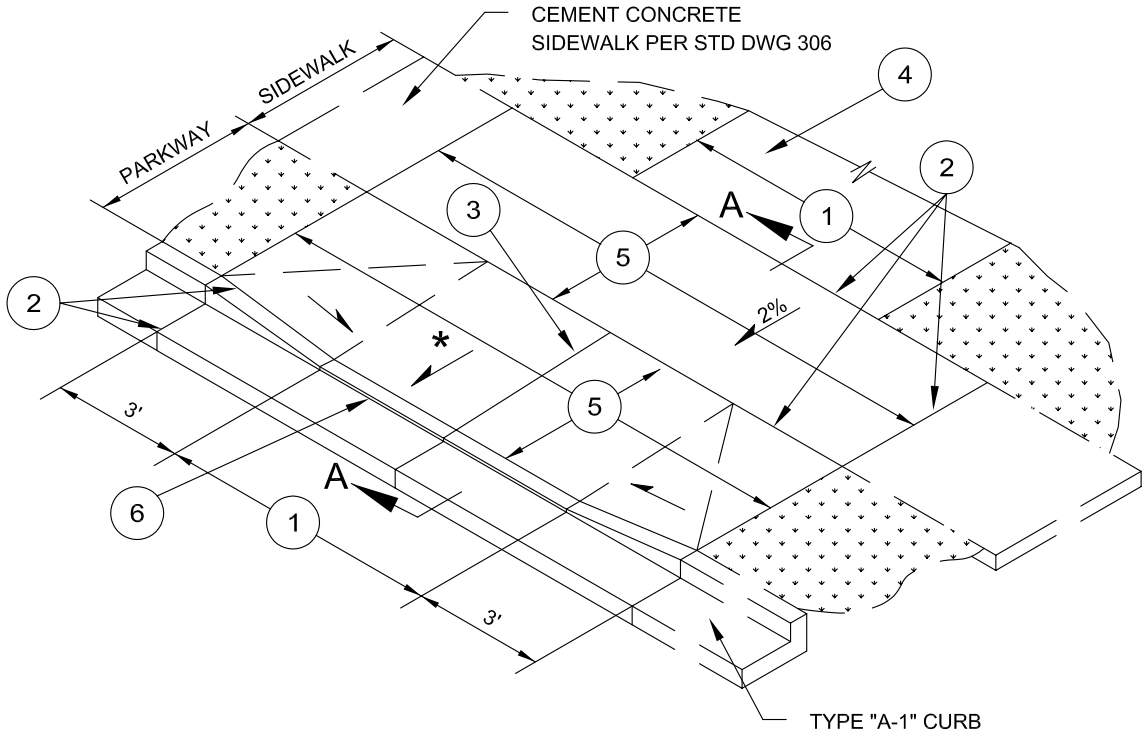
SNOHOMISH
WASHINGTON

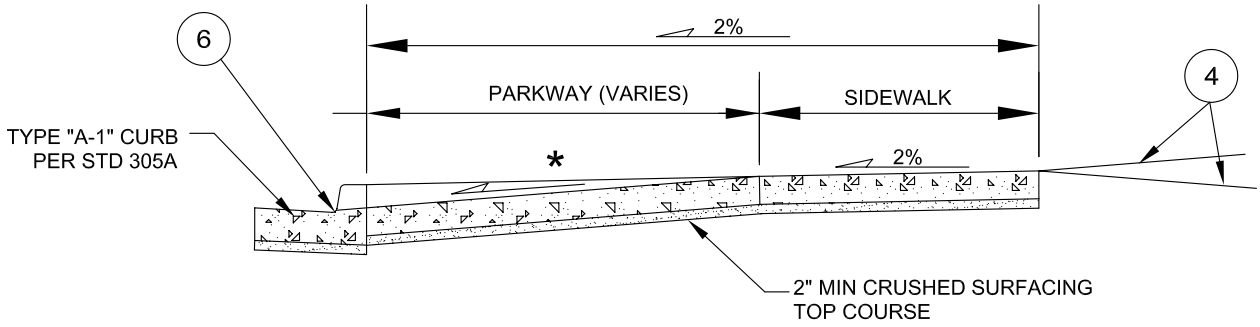
CEMENT CONCRETE DRIVEWAY TYPE 2

Approved By:
City Engineer
Date: May 30, 2004

308
Number

City of Snohomish Public Works Department





SECTION A-A

* Maximum 8% slope in commercial
Maximum 15% slope in residential

NOTES

1. EQUALS WIDTH OF DRIVEWAY AT PROPERTY LINE.
2. 3/8" WIDE FULL DEPTH EXPANSION JOINT.
3. 3/8" WIDE FULL DEPTH EXPANSION JOINT IF (1) IS 15' OR GREATER.
4. DRIVEWAY TO BE SURFACED WITH ASPHALT OR CONCRETE.
5. DRIVEWAY CEMENT CONCRETE SHALL BE A MIN OF 6" THICK INCLUDING WING RAMPS AND PLACED ON A MINIMUM OF 2" CRUSHED SURFACING TOP COURSE COMPACTED TO 95% MAXIMUM DENSITY.
6. MAINTAIN 1/2" LIP AT GUTTER

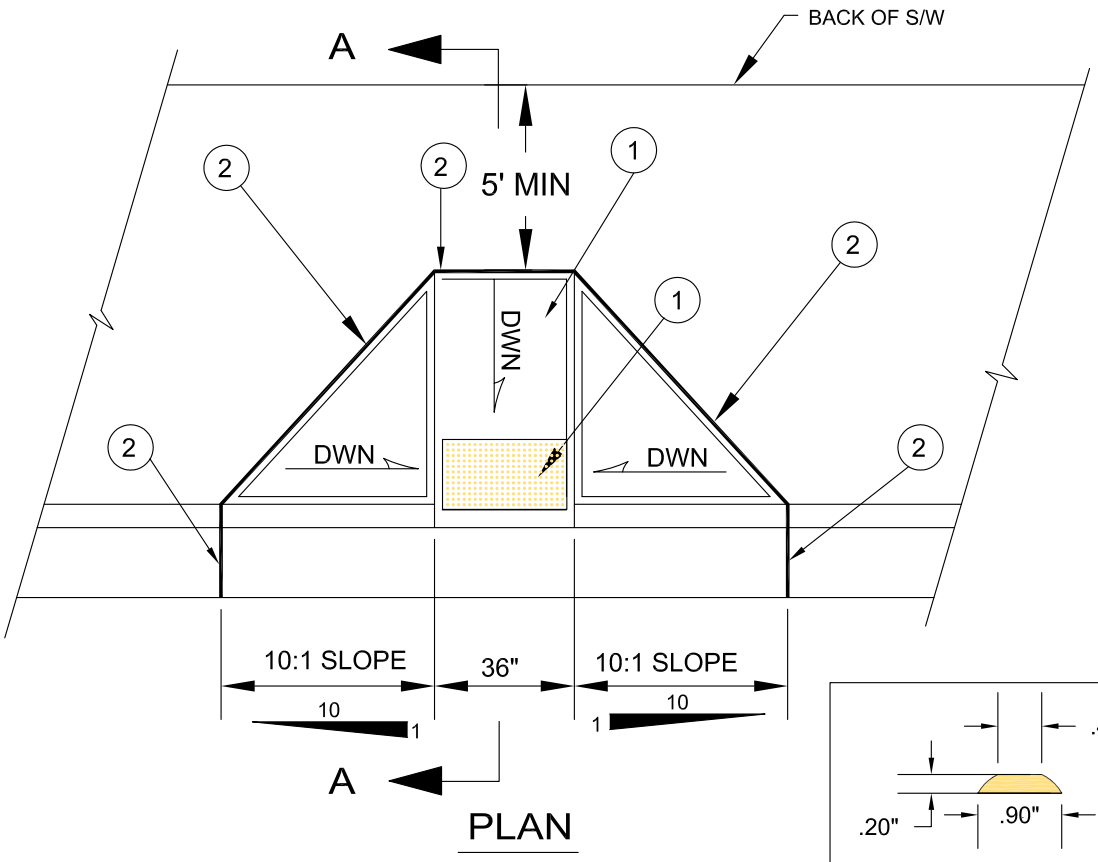
SNOHOMISH
WASHINGTON

CEMENT CONCRETE DRIVEWAY TYPE 3

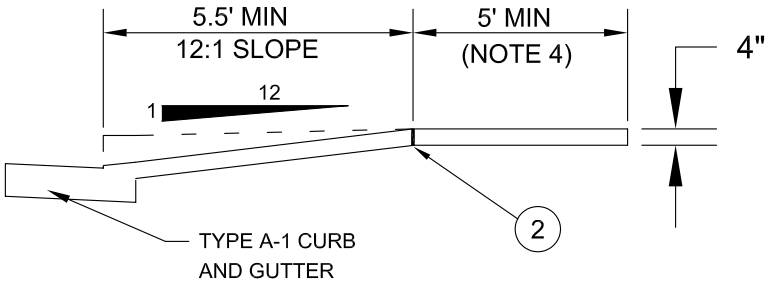
Approved By:
City Engineer
Date: May 30, 2004

309
Number

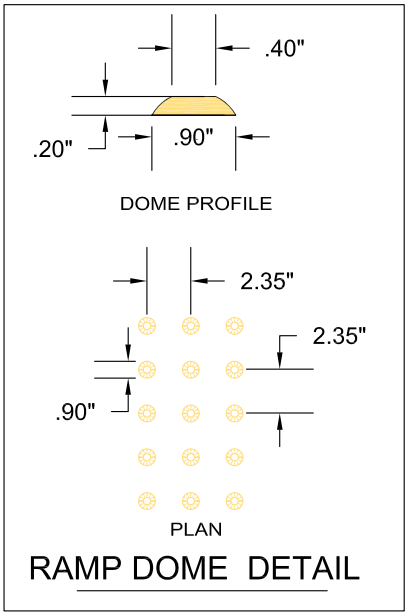
City of Snohomish Public Works Department



PLAN



SECTION A-A



RAMP DOME DETAIL

NOTES:

1. RAMP TEXTURE IS TO BE BROOM FINISHED. TRUNCATED DOME WARNING DEVICES WILL BE INSTALLED 6" FROM CURB AND 24" DEEP AND PAINTED "SAFETY YELLOW". SPACING AND SIZING AS SHOWN IN DETAIL. PER ADAAG.2.9.2, CURRENT VERSION.
2. 3/8" EXPANSION JOINT.
3. IF LANDING AREA IS LESS THAN 5' USE TYPE C CURB RAMP (STD DWG 310C)
4. CURB RAMPS WILL NOT BE POURED INTEGRAL WITH SIDEWALK AND SHALL BE ISOLATED BY EXPANSION JOINT MATERIAL ON ALL SIDES, BUT NOT AT END OF RAMP ADJACENT TO ROADWAY.

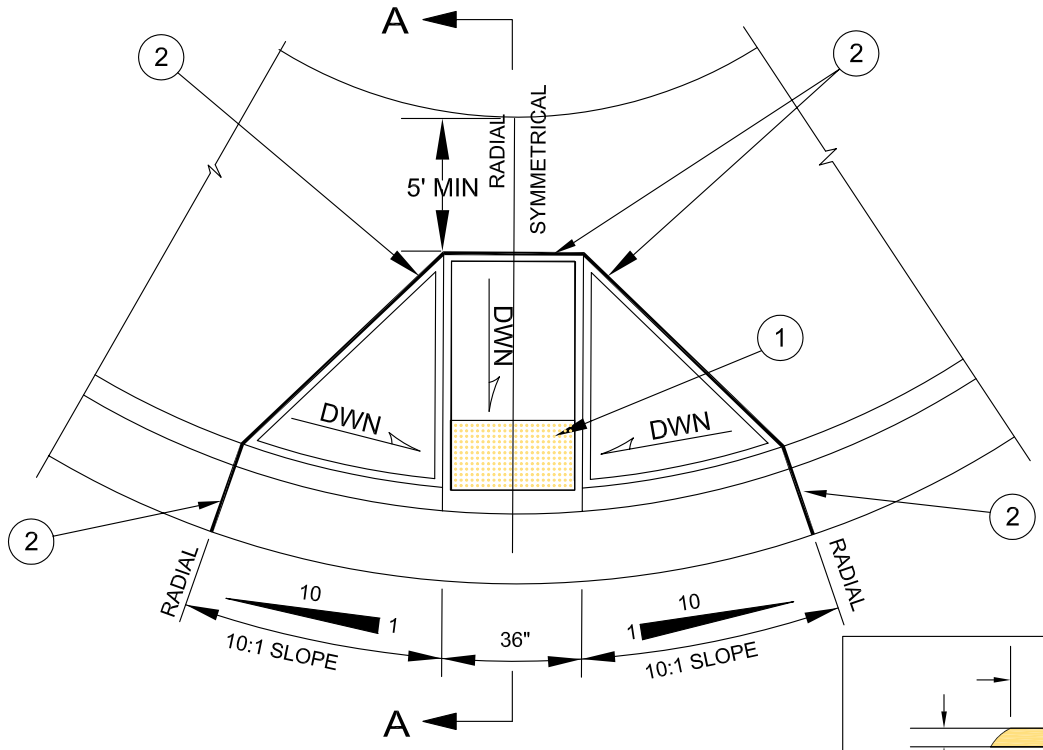
SNOHOMISH
WASHINGTON

TYPE A CURB RAMP

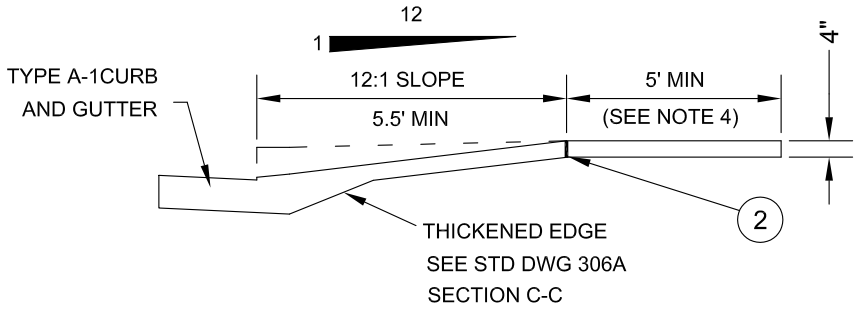
Approved By:
City Engineer
Date: May 30, 2004

310a
Number

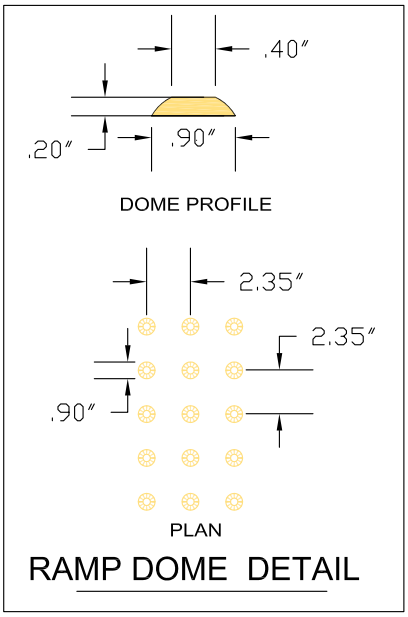
City of Snohomish Public Works Department



PLAN



SECTION A-A



RAMP DOME DETAIL

NOTES:

1. RAMP TEXTURE IS TO BE BROOM FINISHED. TRUNCATED DOME WARNING DEVICES WILL BE INSTALLED 6" FROM CURB AND 24" DEEP AND PAINTED "SAFETY YELLOW". SPACING AND SIZING AS SHOWN IN DETAIL. PER ADAAG.2.9.2, CURRENT VERSION.
2. 3/8" EXPANSION JOINT.
3. IF LANDING AREA IS LESS THAN 5' USE TYPE D CURB RAMP (STD DWG 310D)
4. CURB RAMPS WILL NOT BE POURED INTEGRAL WITH SIDEWALK AND SHALL BE ISOLATED BY EXPANSION JOINT MATERIAL ON ALL SIDES, BUT NOT AT END OF RAMP ADJACENT TO ROADWAY.

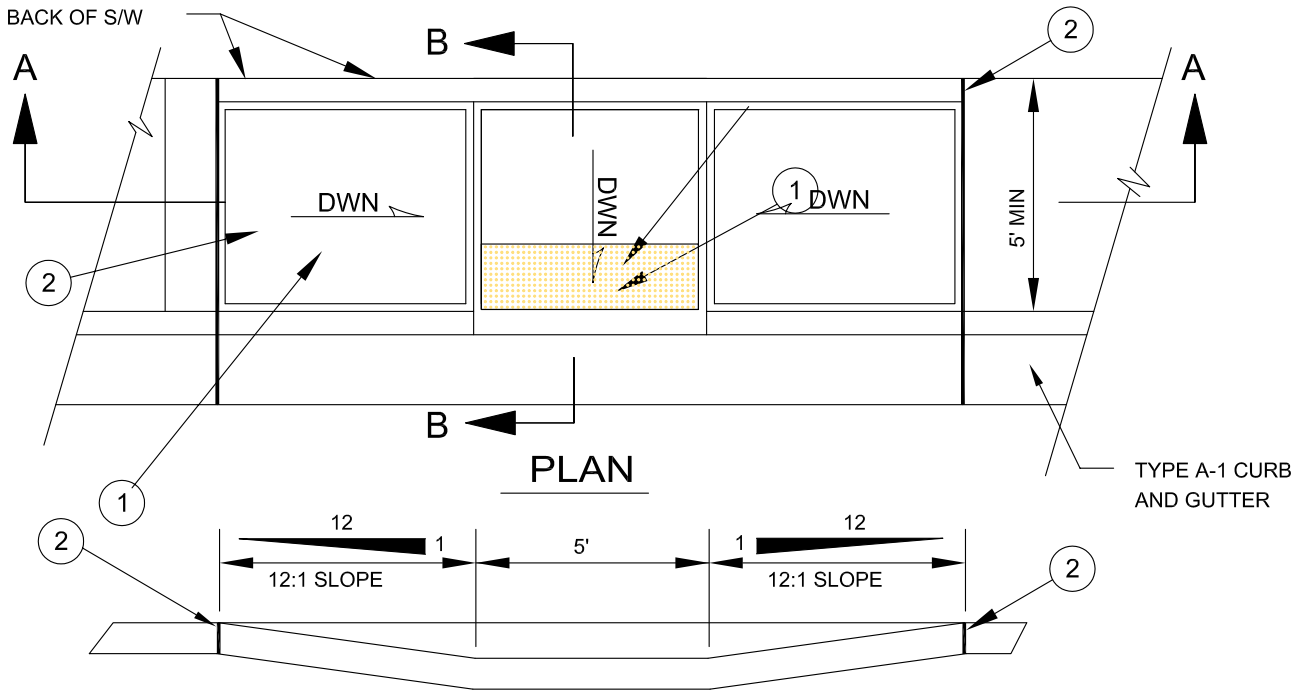
SNOHOMISH WASHINGTON

TYPE "B" CURB RAMP

Approved By:
City Engineer
Date: May 30, 2004

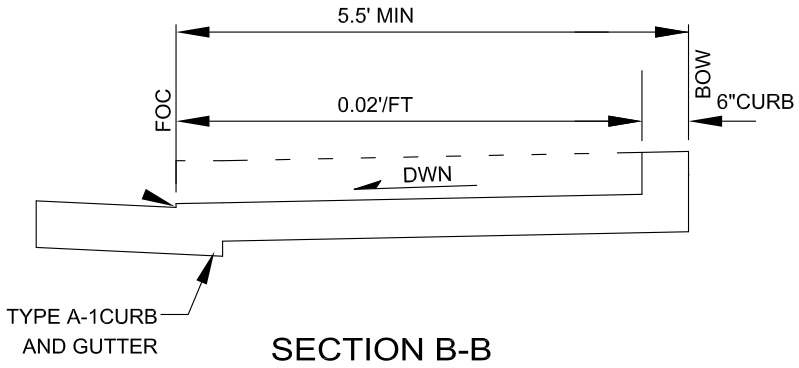
310b
Number

City of Snohomish Public Works Department

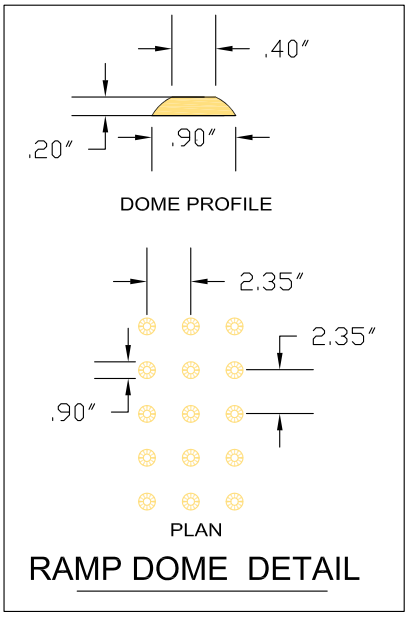


PLAN

SECTION A-A



SECTION B-B



RAMP DOME DETAIL

NOTES:

1. RAMP TEXTURE IS TO BE BROOM FINISHED. TRUNCATED DOME WARNING DEVICES WILL BE INSTALLED 6" FROM CURB AND 24" DEEP AND PAINTED "SAFETY YELLOW". SPACING AND SIZING AS SHOWN IN DETAIL. PER ADAAG.2.9.2, CURRENT VERSION.

2. 3/8" EXPANSION JOINT.

3. CURB RAMPS WILL NOT BE POURED INTEGRAL WITH SIDEWALK AND SHALL BE ISOLATED BY EXPANSION JOINT MATERIAL ON ALL SIDES, BUT NOT AT END OF RAMP ADJACENT TO ROADWAY.

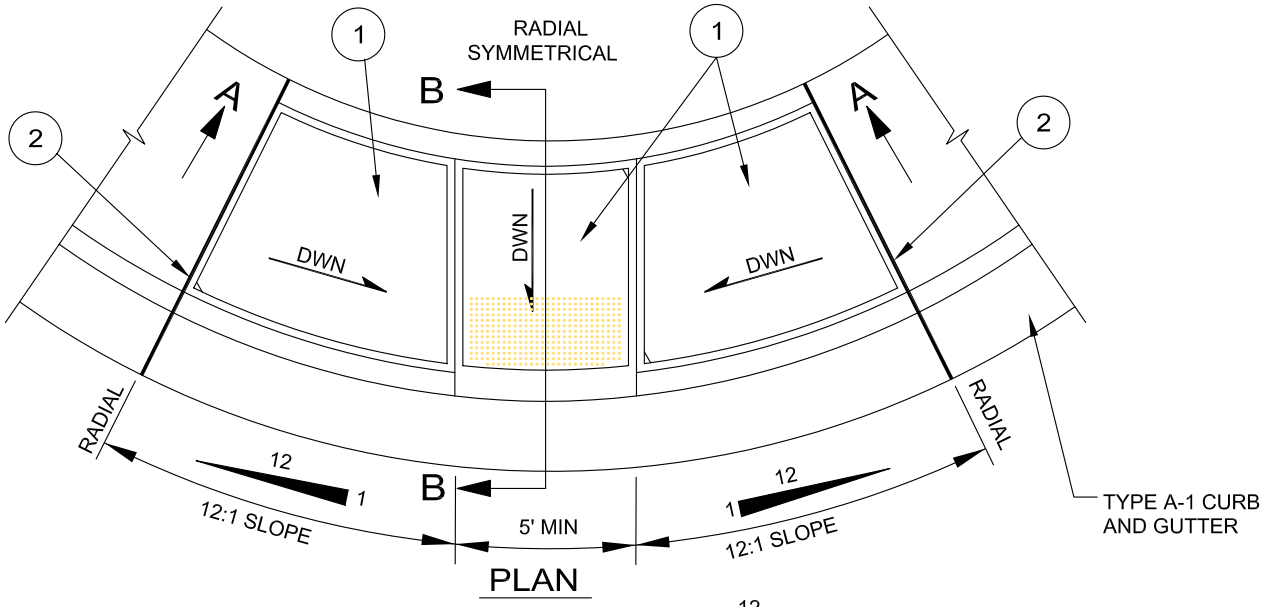
SNOHOMISH
WASHINGTON

TYPE "C" CURB RAMP

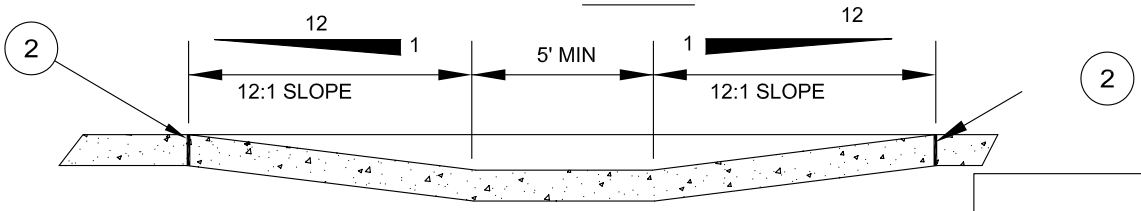
Approved By:
City Engineer
Date: May 30, 2004

310c
Number

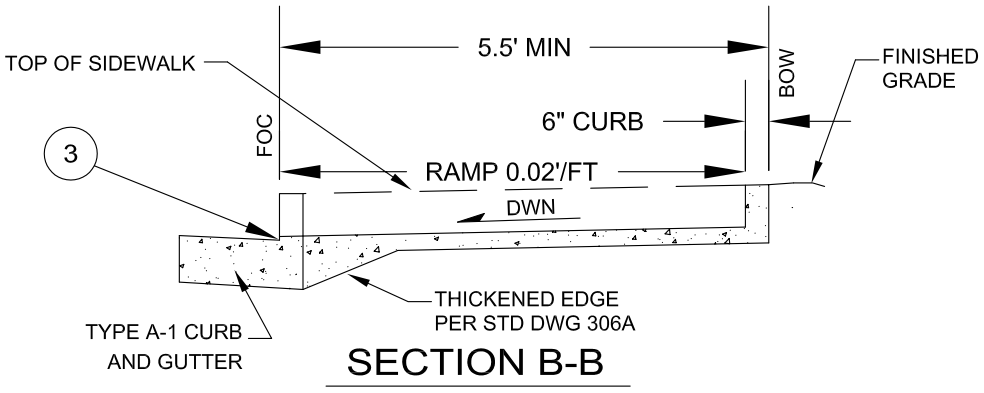
City of Snohomish Public Works Department



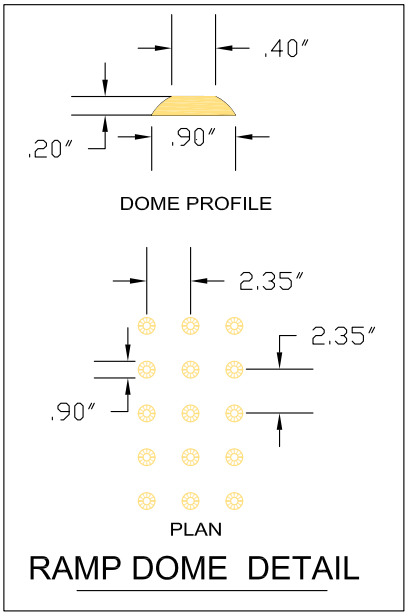
PLAN



SECTION A-A



SECTION B-B



RAMP DOME DETAIL

NOTES:

1. RAMP TEXTURE IS TO BE BROOM FINISHED. TRUNCATED DOME WARNING DEVICES WILL BE INSTALLED 6" FROM CURB AND 24" DEEP AND PAINTED "SAFETY YELLOW". SPACING AND SIZING AS SHOWN IN DETAIL. PER ADAAG.2.9.2, CURRENT VERSION.
2. 3/8" EXPANSION JOINT.
3. 1/2" MAXIMUM LIP AT GUTTER LINE.
4. CURB RAMPS WILL NOT BE POURED INTEGRAL WITH SIDEWALK AND SHALL BE ISOLATED BY EXPANSION JOINT MATERIAL ON ALL SIDES, BUT NOT AT END OF RAMP ADJACENT TO ROADWAY.

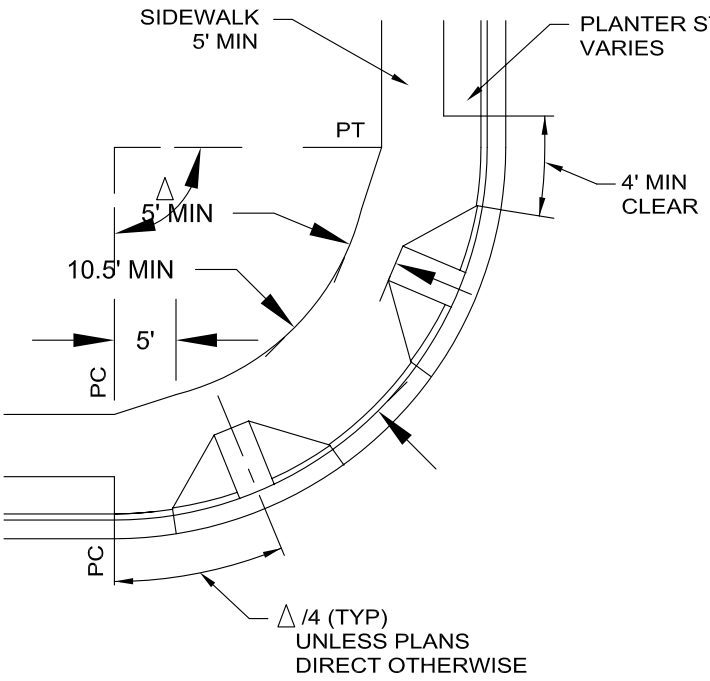
SNOHOMISH
WASHINGTON

TYPE "D" CURB RAMP

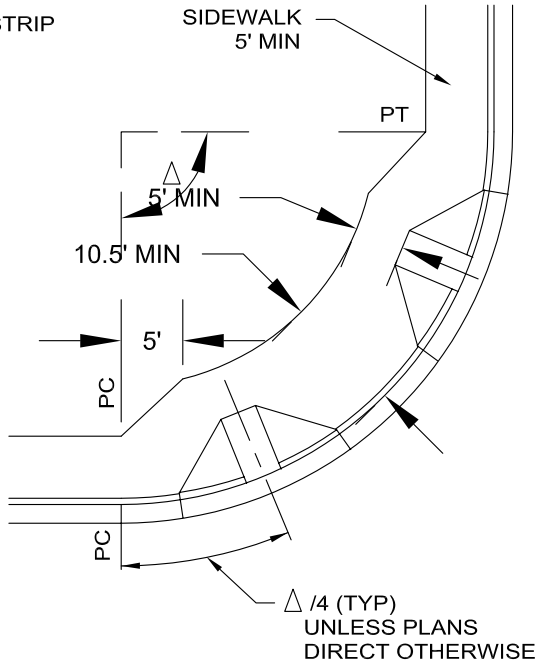
Approved By:
City Engineer
Date: May 30, 2004

310d
Number

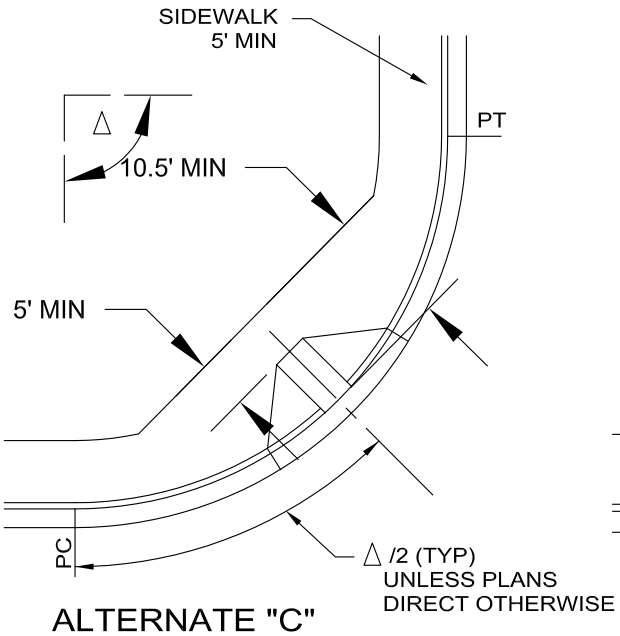
City of Snohomish Public Works Department



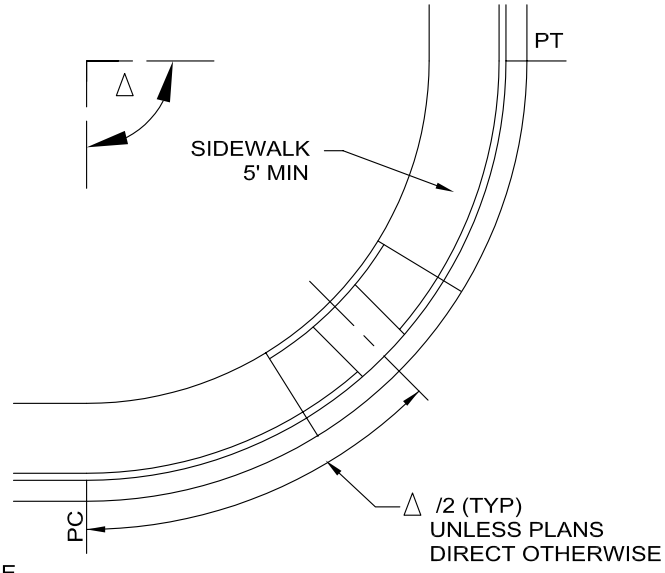
ALTERNATE "A"



ALTERNATE "B"



ALTERNATE "C"



ALTERNATE "D"

NOTES:

1. ALTERNATES "A" & "B" FOR USE AT ARTERIAL/ARTERIAL AND ARTERIAL/LOCAL ACCESS INTERSECTIONS.
2. ALTERNATES "C" & "D" FOR USE AT LOCAL ACCESS/LOCAL ACCESS INTERSECTIONS OR AS APPROVED BY CITY ENGINEER.
3. FOR ALTERNATE "A", "B" AND "C" USE CURB RAMP PER STD DWGS 306A AND 310B.
4. FOR ALTERNATE "D" USE CURB RAMP PER STD DWGS 306A AND 310D.

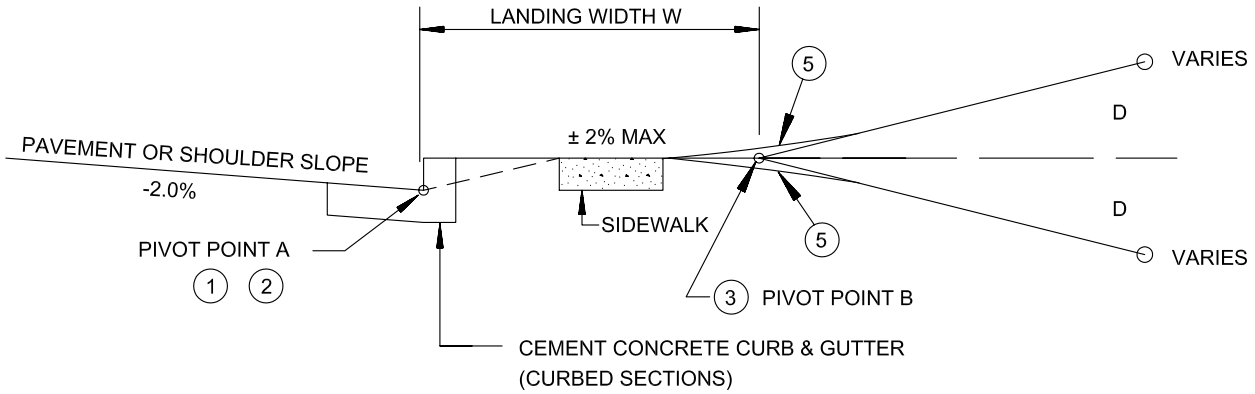
SNOHOMISH
WASHINGTON

TYPICAL CURB RAMP LOCATIONS

Approved By:
City Engineer
Date: May 30, 2004

311
Number

City of Snohomish Public Works Department



NOTES:

1. SEE STD DWG 4-140 FOR CURB DETAILS.

2. WHEN ACCESSING SHOULDERED ROADWAYS, MAINTAIN SHOULDER SLOPE TO PIVOT POINT A.

3. ACCESS POINT GRADE SHALL BE MEASURED FROM PIVOT POINT B.

4. LANDING WIDTH W MAY BE REDUCED SUBJECT TO APPROVAL OF THE ENGINEER IN ACCORDANCE WITH SECTION 1-05 OF THESE STANDARDS.

5. A VERTICAL CURVE SHALL BE CONSTRUCTED TO TRANSITION THE LANDING TO THE ACCESS APPROACH. THE VERTICAL SEPARATION BETWEEN THE CURVE AND A 10-FOOT CHORD OF THE CURVE SHALL NOT EXCEED 3.25 INCHES (WHERE D IS POSITIVE) OR 2.00 INCHES (WHERE D IS NEGATIVE).

TYPE OF ACCESS	ACCESSING	LANDING WIDTH W ④	ACCESS GRADE D
RESIDENTIAL (URBAN)	NON-ARTERIAL	15'	± 15% MAX.
RESIDENTIAL (URBAN)	ARTERIAL	15'	± 7% MAX.
COMMERCIAL/INDUSTRIAL	NON-ARTERIAL	30'	± 8% MAX.
COMMERCIAL/INDUSTRIAL	ARTERIAL	30'	± 5% MAX.

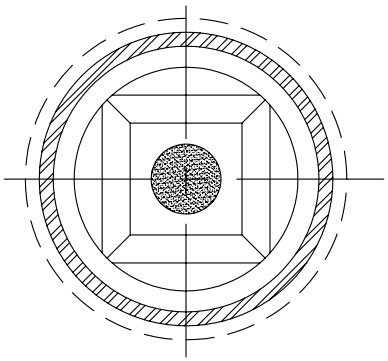
SNOHOMISH
WASHINGTON

ACCESS GRADE POINTS

Approved By:
City Engineer
Date: May 30, 2004

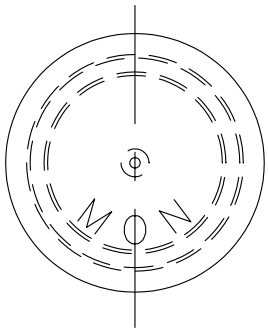
312
Number

City of Snohomish Public Works Department

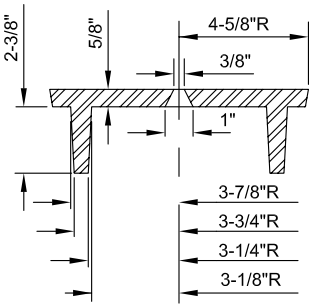


PLAN

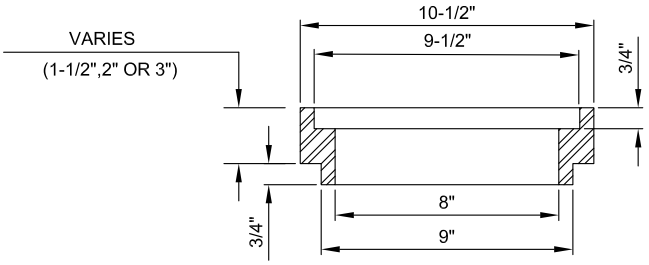
COVER REMOVED



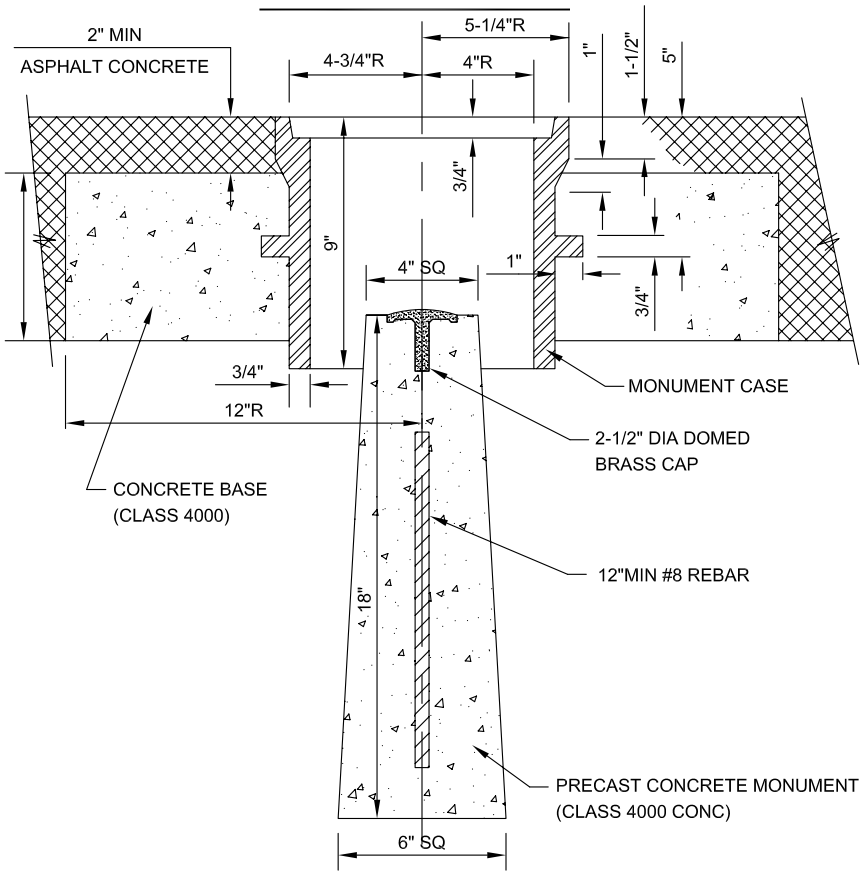
COVER PLAN



COVER SECTION



EXTENSION SECTION



SECTION

NOTES

1. MONUMENTS IN NON PAVED AREAS SHALL BE 3" ABOVE GRADE.
2. ALL MONUMENTS SHALL BE PRECAST CONC WITH REBAR AND 2-1/2" DIA BRASS CAP.
3. MONUMENT CASE AND RISER SECTION SHALL BE CAST IRON PER ASTM-A48, CLASS 30, WITH BITUMINOUS COATING.
4. COVER SHALL BE DUCTILE IRON PER ASTM-A536, GRADE 80-55-06. WITH BITUMINOUS COATING.
5. LEGEND ON COVER SHALL BE 1/8" RAISED INTEGRALLY CAST LETTERS 1" HIGH WITH A MIN FACE WIDTH OF 3/16".

SNOHOMISH
WASHINGTON

SURVEY MONUMENT

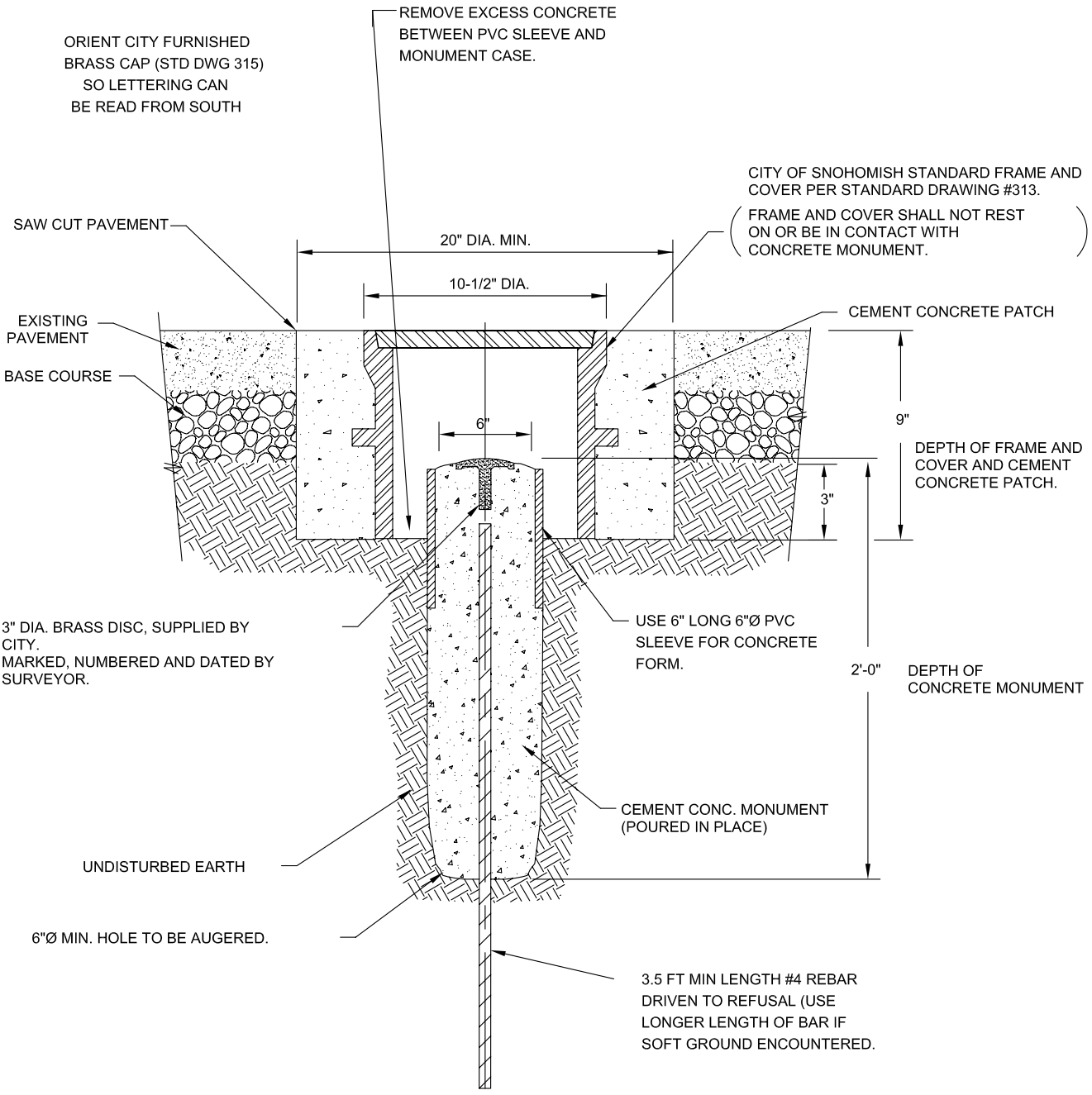
Approved By:
City Engineer
Date: May 30, 2004

313
Number

City of Snohomish Public Works Department

NOTES

THIS MONUMENT SHALL BE USED ONLY FOR CONTROL MONUMENTATION SURVEYS AT LOCATIONS APPROVED BY THE CITY ENGINEER AS DESIGNATED BY LICENSED SURVEYOR.



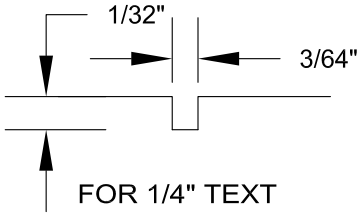
SNOHOMISH WASHINGTON

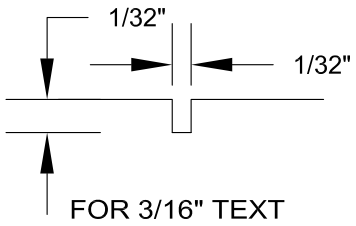
SURVEY CONTROL MONUMENT

Approved By:
City Engineer
Date: May 30, 2004

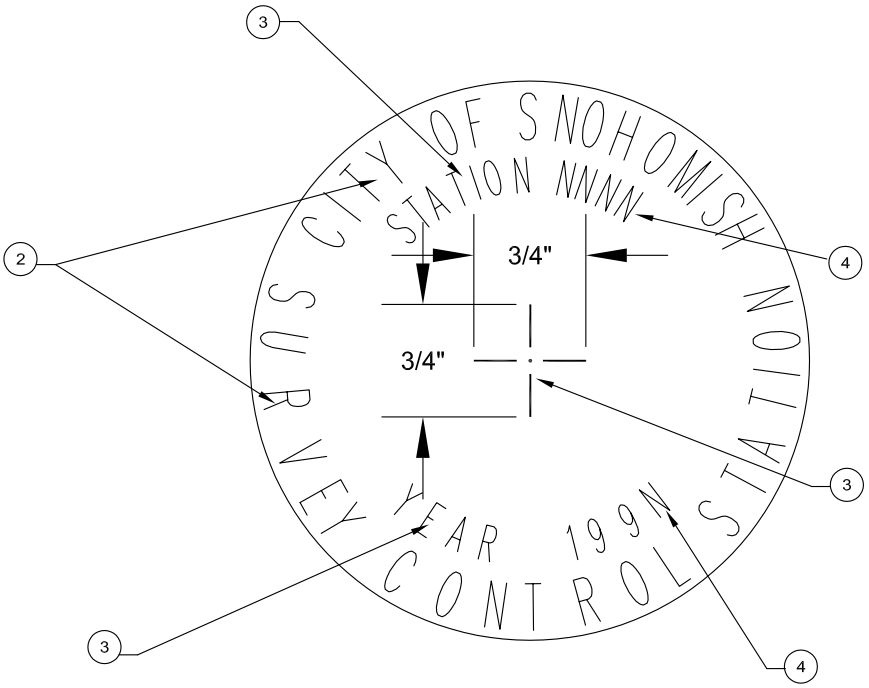
314
Number

City of Snohomish Public Works Department

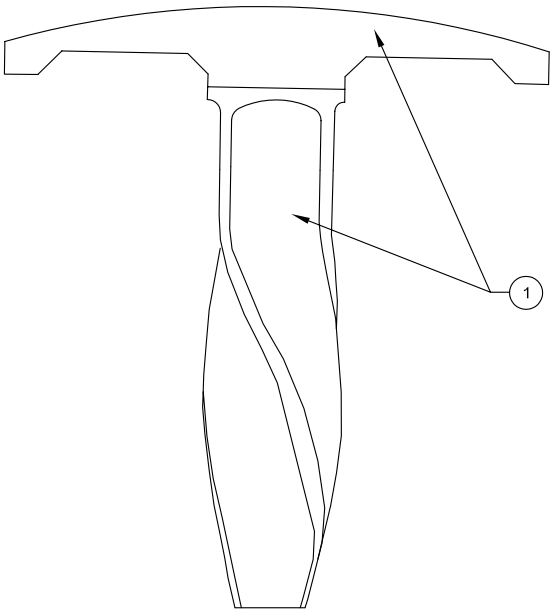




GROOVE DETAIL



PLAN



ELEVATION

NOTES:

1. DIMENSIONS OF CASTING BASE & CAP PER WSDOT/APWA STANDARD PLAN H-6.
2. GROOVE FOR 1/4" HIGH CAST LETTERING ON CAP SHALL BE 1/32 IN DEEP BY 3/64 IN WIDE.
3. GROOVE FOR 3/16" HIGH CAST LETTERING AND LINES ON CAP SHALL BE 1/32 IN DEEP BY 1/32 IN WIDE.
4. FIELD STAMPED LETTERS AND NUMBERS SHALL BE OF SUFFICIENT DEPTH AND WIDTH SO AS TO BE CLEARLY READABLE AND SHALL BE A MIN. OF 3/16 IN. HIGH.
5. THIS BRASS DISC SHALL ONLY BE USED FOR CONTROL MONUMENTATION PER STD DWG 314

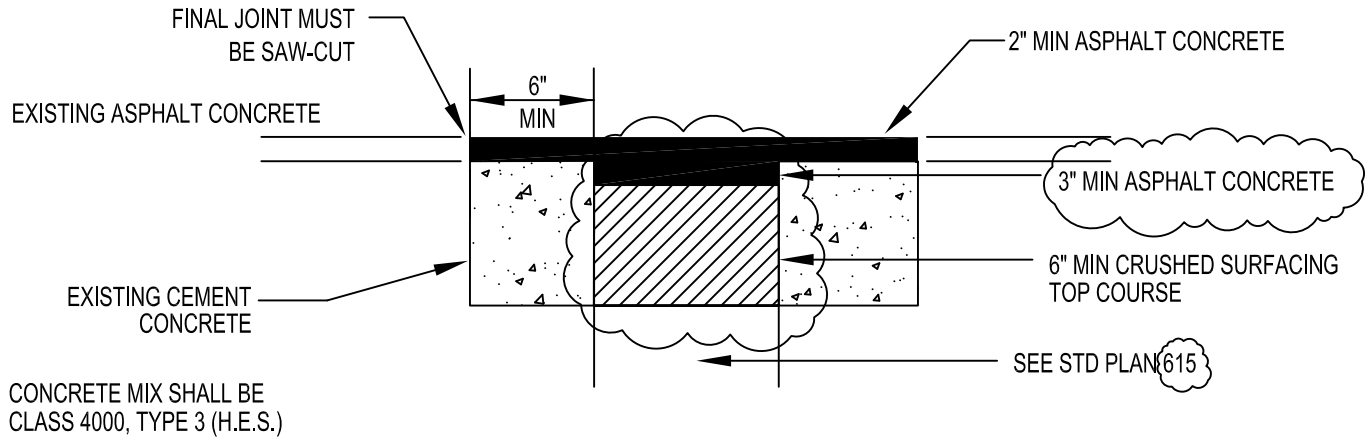
SNOHOMISH
WASHINGTON

SURVEY CONTROL MONUMENT
3" BRASS DISC

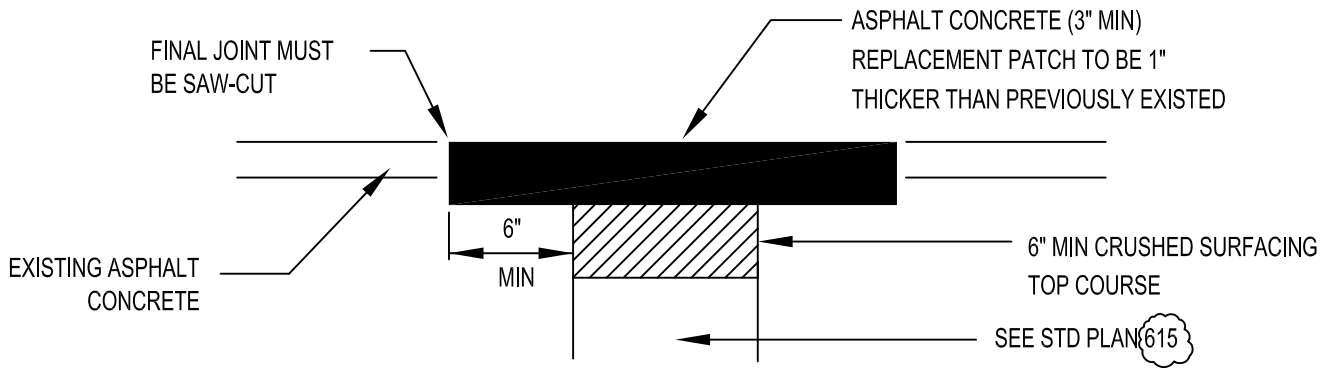
Approved By:
City Engineer
Date: May 30, 2004

315
Number

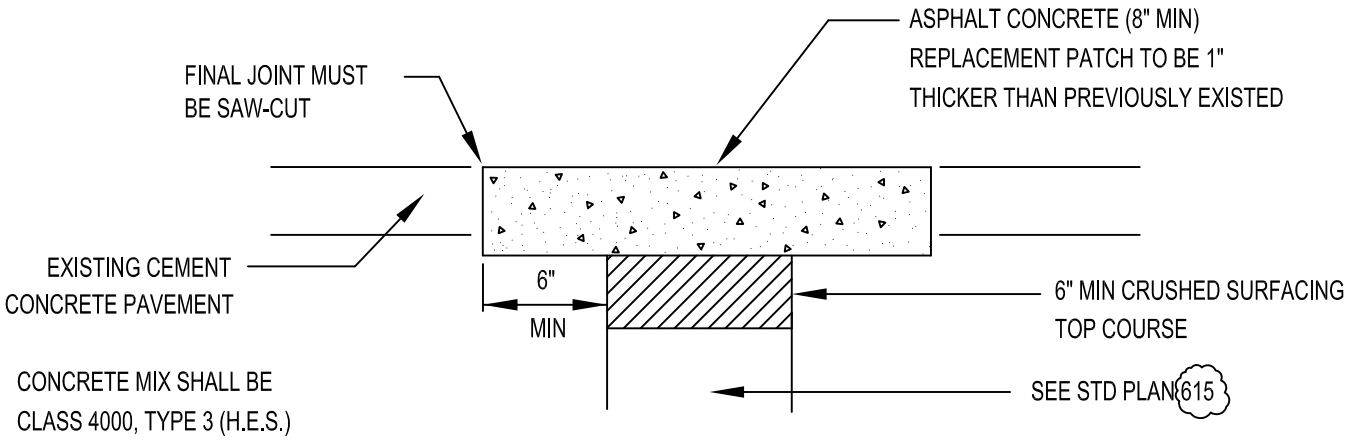
City of Snohomish Public Works Department



EXISTING ASPHALT CONCRETE OVER CEMENT CONCRETE



EXISTING ASPHALT CONCRETE OVER PREPARED GRADE



EXISTING CEMENT CONCRETE OVER PREPARED GRADE

NOTES:

1. ALL TRENCHES IN ROADWAY AREAS SHALL BE BACKFILLED AND PATCHED WITH TEMPORARY ASPHALT AT THE END OF EACH WORK DAY, UNLESS PERMISSION IS GRANTED TO DO OTHERWISE BY THE CITY ENGINEER,
2. ALL TEMPORARY PATCHES ON TRENCHES SHALL BE PERMANENTLY PATCHED WITHIN 2 WEEKS OF COMPLETION OF WORK WITHIN THE ROADWAY AREA.

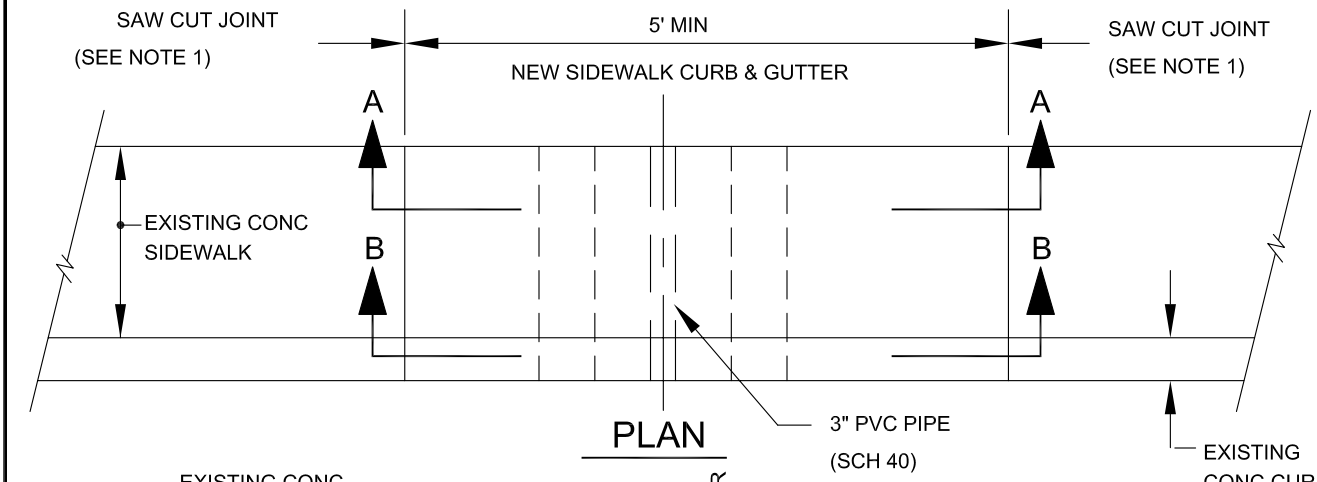
SNOHOMISH
WASHINGTON

PAVEMENT PATCHING DETAILS

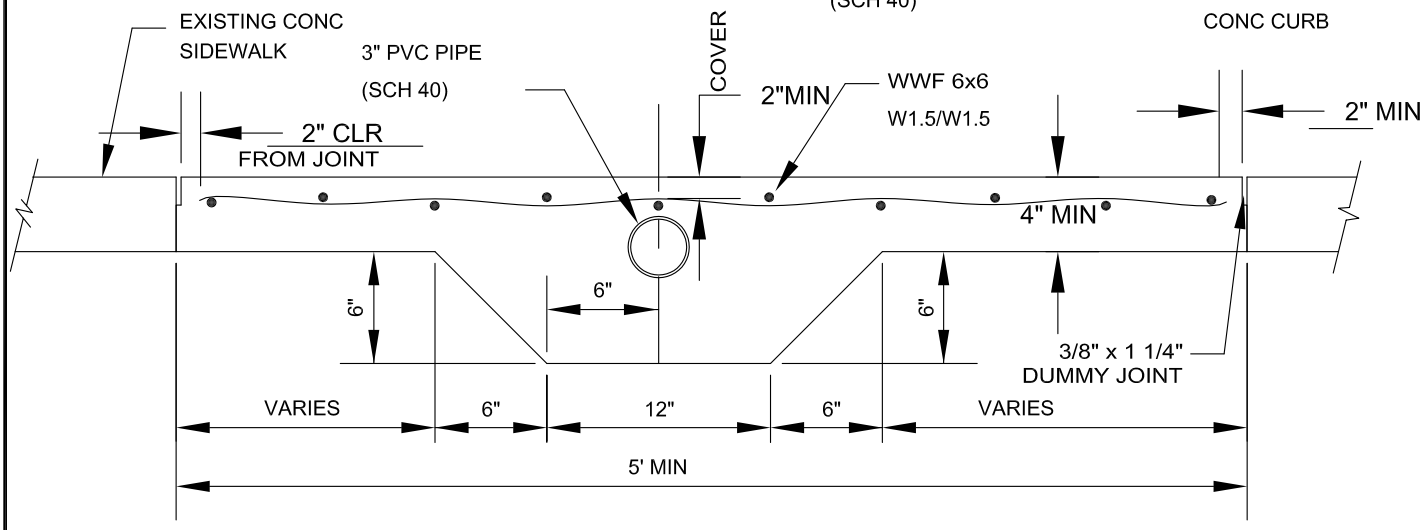
Approved By:
Yosh Monzaki
City Engineer
Date: Aug 15, 2016

316
Number

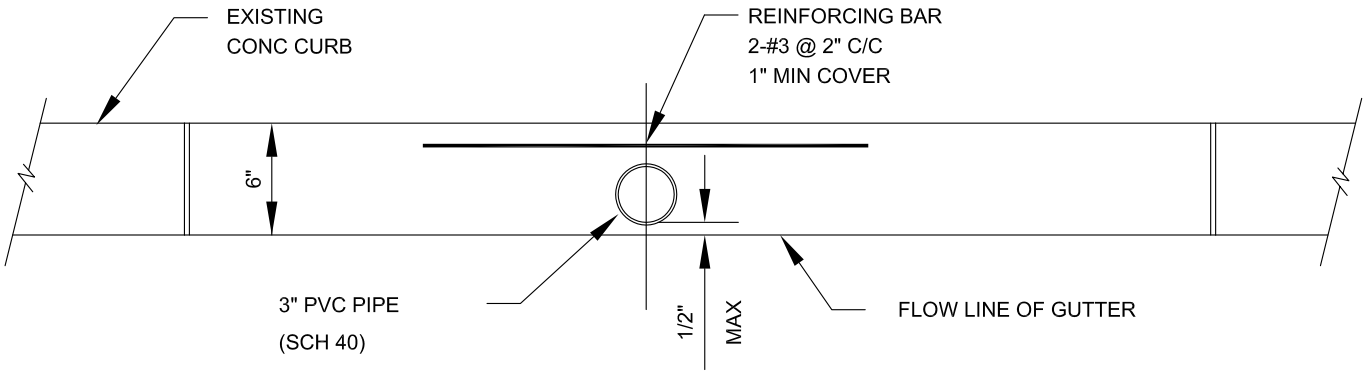
City of Snohomish Public Works Department



PLAN



SIDEWALK SECTION A-A



CURB SECTION B-B

NOTES:

1 SIDEWALK AND CURBING MUST BE SAW-CUT AT A DUMMY JOINT OR FULL EXPANSION JOINT.

2 FULL DEPTH OF CURB AND GUTTER MUST BE REMOVED AND REPLACED.

3 ALL NEW CURB, GUTTER AND SIDEWALK SHALL BE CLASS 3000 CEMENT CONCRETE.

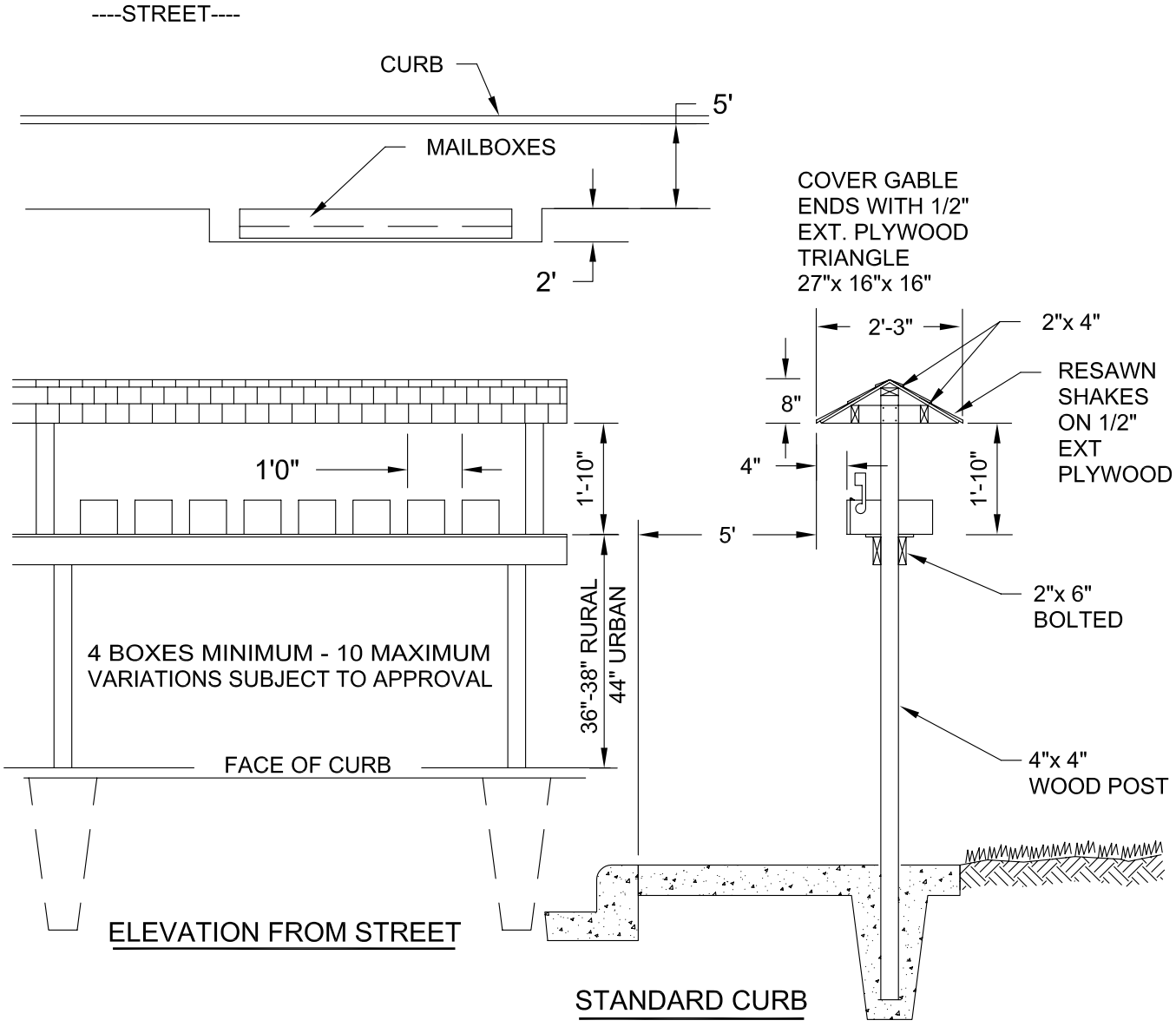
SNOHOMISH
WASHINGTON

RESIDENTIAL SIDEWALK DRAIN

Approved By:
City Engineer
Date: May 30, 2004

319
Number

City of Snohomish Public Works Department



NOTES:

1. MAILBOX MUST BE TYPE "APPROVED BY THE POSTMASTER GENERAL" WITH A UNIFORM BOX STYLE AND METHOD OF ADDRESS IDENTIFICATION PER EACH STANDARD.

2. LOCATION IS SUBJECT TO APPROVAL BY THE CITY FOR PROTECTION OF VIEWS AND ACCESS AND IS TO BE SHOWN ON STREET IMPROVEMENT PLANS.

3. THE SKETCH DEPICTS A MINIMUM STRUCTURAL AND DIMENSIONAL STANDARD. INNOVATIVE DESIGNS MEETING THE MINIMUM DIMENSIONAL AND STRUCTURAL REQUIREMENTS ARE ACCEPTABLE.

4. ALL WOOD TO BE PRESSURE TREATED FIR OR HEMLOCK.

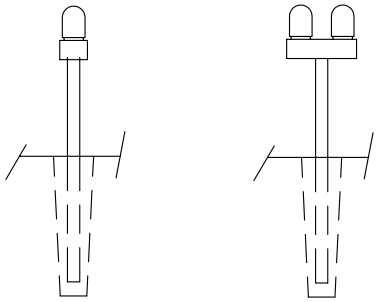
SNOHOMISH WASHINGTON

MAILBOX STRUCTURE INSTALLATION

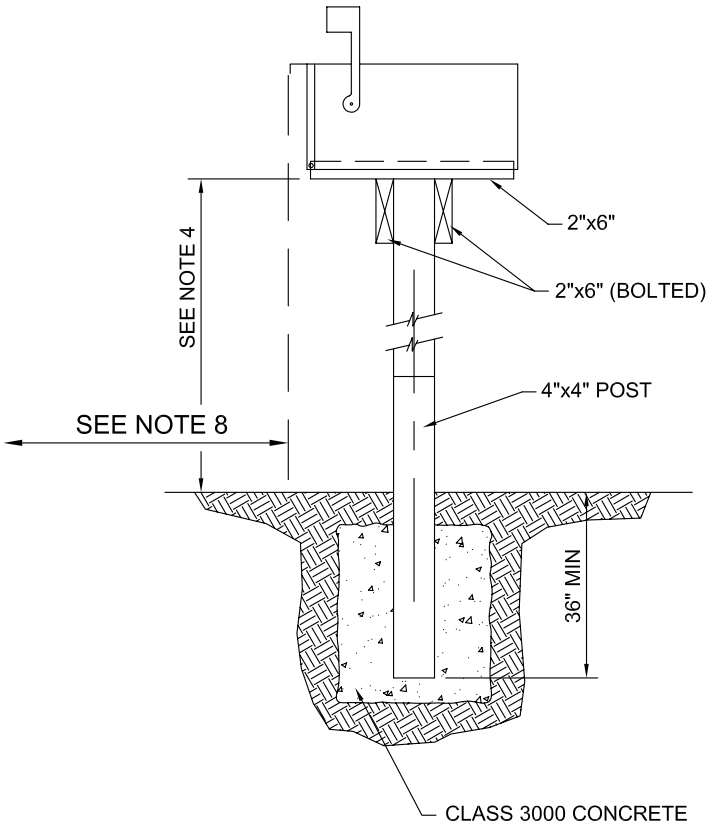
Approved By:
City Engineer
Date: May 30, 2004

320
Number

City of Snohomish Public Works Department



TYPICAL CONFIGURATIONS



TYPICAL SECTION

NOTES

1. FOR 1 OR 2 MAILBOXES PER STRUCTURE USE SINGLE 4"x4" POST. FOR 3 OR MORE MAILBOXES SEE STD DWG 320 OR 321.
2. ALL WOOD TO BE PRESSURE TREATED FIR OR HEMLOCK.
3. FOR ALTERNATE MAILBOX MOUNTINGS SEE STANDARD DWG 320.
4. MAILBOX HEIGHT VARIES ACCORDING TO THE TYPE OF DELIVERY VEHICLE. WHERE MAIL DELIVERY IS ACCOMPLISHED BY MAIL TRUCKS ("MOUNTED" ROUTES) THE MAILBOX HEIGHTS SHALL BE 44". WHERE MAIL DELIVERY IS ACCOMPLISHED BY PASSENGER VEHICLE ("RURAL" ROUTES) THE MAILBOX HEIGHT SHALL BE 36" TO 38".
5. MAILBOXES MUST BE POSTMASTER APPROVED WITH A UNIFORM BOX STYLE AND METHOD OF ADDRESS IDENTIFICATION.
6. LOCATIONS OF MAILBOXES ARE SUBJECT TO APPROVAL BY THE CITY ENGINEER FOR PROTECTION OF VIEWS AND ACCESS.
7. THIS DRAWING DEPICTS A MINIMUM STRUCTURAL AND DIMENSIONAL STANDARD. INNOVATIVE DESIGNS MEETING OR EXCEEDING THIS MINIMUM STANDARD MUST BE APPROVED BY THE CITY ENGINEER.
8. ALL MAILBOX STRUCTURES SHALL BE PLACED BACK OF SIDEWALK WITH NO PORTION OF THE BOX OR STRUCTURE PROTRUDING INTO THE SIDEWALK. IF NO SIDEWALK EXISTS SETBACK WILL BE SET BY THE CITY ENGINEER.

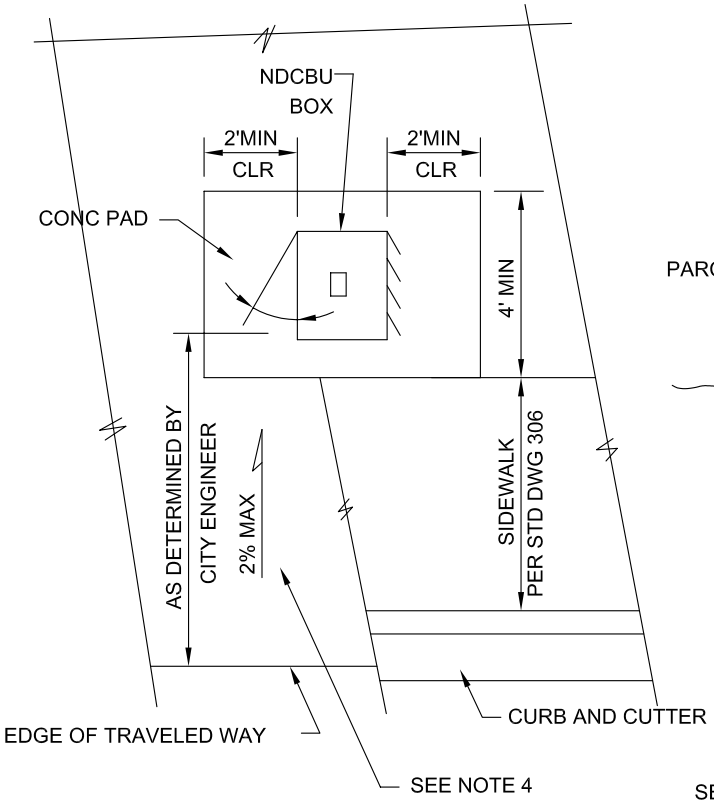
SNOHOMISH WASHINGTON

MAILBOX STRUCTURE FOR ONE OR TWO BOXES

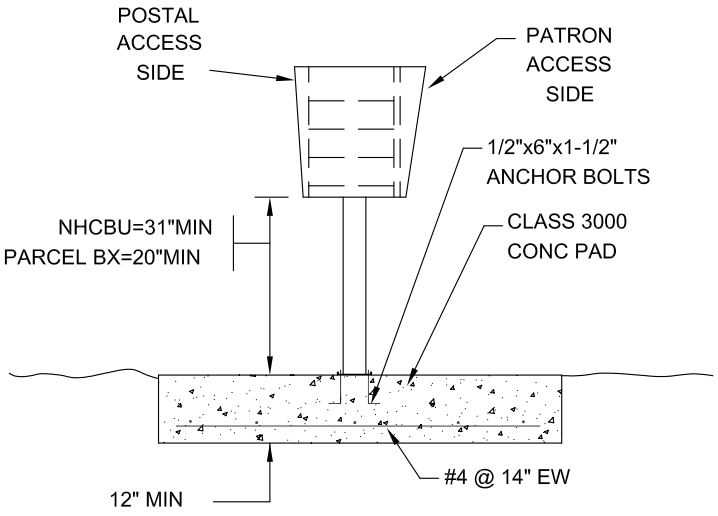
Approved By:
City Engineer
Date: May 30, 2004

320a
Number

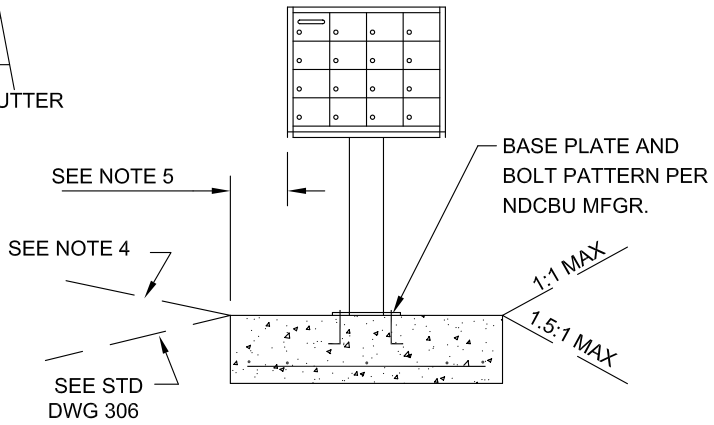
City of Snohomish Public Works Department



SETBACK PLAN



FRONT ELEVATION



SIDE ELEVATION

NOTES:

1. THIS DRAWING DEPICTS A MINIMUM STRUCTURAL AND DIMENSIONAL STANDARD FOR NEIGHBORHOOD DELIVERY & COLLECTION BOX UNIT (NDCBU) AND PADS FOR SPECIFIC POSTAL REQUIREMENTS CONTACT THE POSTMASTER.
2. MAILBOXES MUST BE POSTMASTER APPROVED WITH A UNIFORM BOX STYLE AND METHOD OF ADDRESS IDENTIFICATION.
3. LOCATIONS OF MAILBOXES ARE SUBJECT TO APPROVAL BY THE CITY ENGINEER FOR PROTECTION OF VIEWS AND ACCESS.
4. INSTALLATION OF DRAINAGE CULVERT MAY BE NECESSARY IN AREAS WHERE THERE IS NO CONCRETE SIDEWALK AND THE REQUIRED SETBACK SPANS A ROADSIDE DITCH. ACCESS TO SUCH STRUCTURES WILL HAVE A MAX. SLOPE OF 2%. AND SHALL HAVE A PAD CONSISTING OF A MINIMUM OF 2" OF CRUSHED SURFACING TOP COURSE COMPACTED TO 95% MAXIMUM DENSITY.
5. ALL MAILBOX STRUCTURES SHALL BE PLACED BACK OF SIDEWALK WITH NO PORTION OF BOX OR STRUCTURE PROTRUDING INTO THE SIDEWALK. IF NO SIDEWALK EXISTS SETBACK WILL BE SET BY THE CITY ENGINEER.
6. SUGGESTED SOURCE SECURITY MFG CORP (800) 762-6937, 8000 SERIES PEDESTAL BOXES, SALSBURY INDUSTRIES (800) 323-3003 OR POSTAL APPROVED EQUAL.
7. PLACEMENT LOCATION OF PEDESTAL PARCEL LOCKER WILL BE APPROVED BY THE CITY ENGINEER AND THE POSTAL SERVICE.

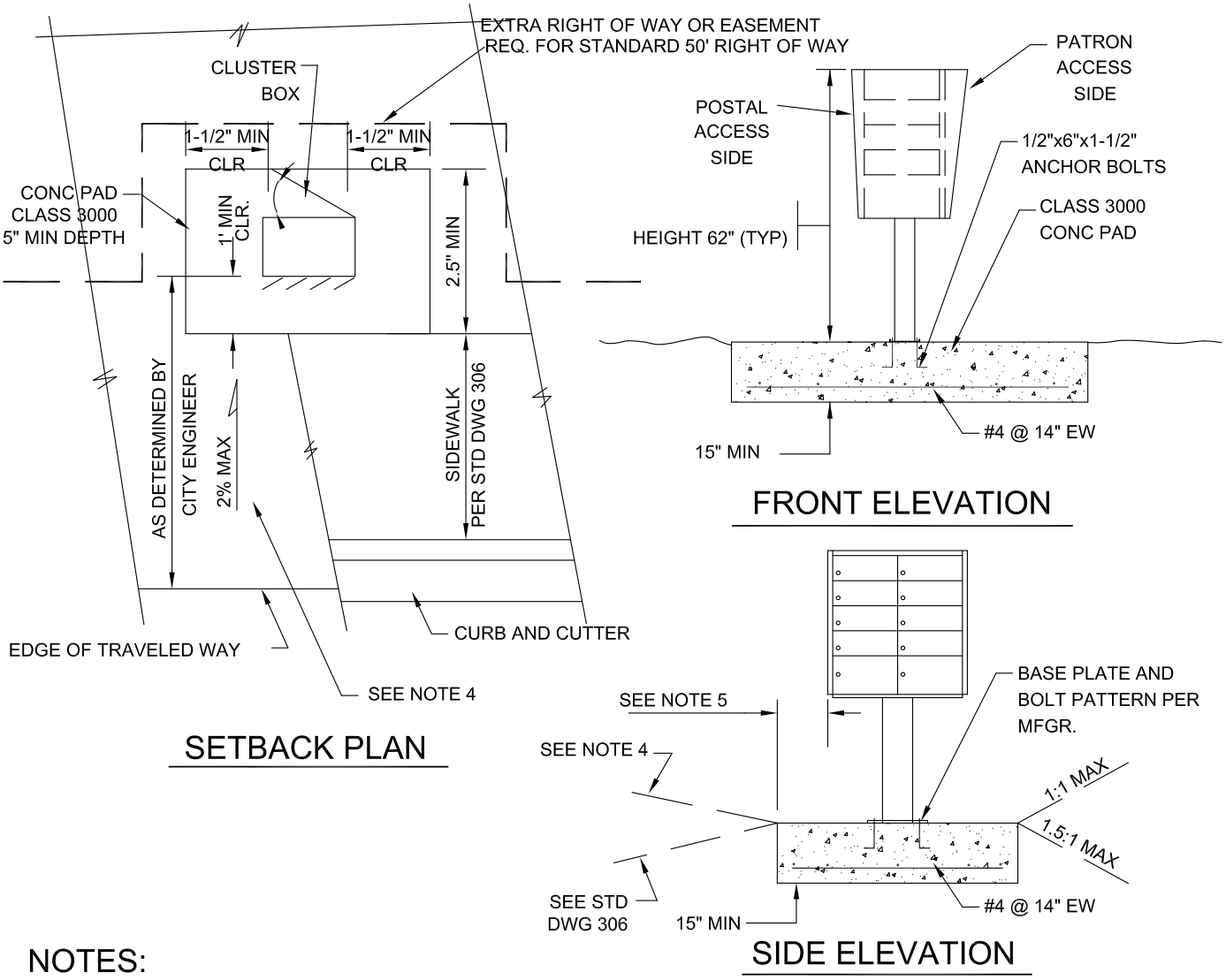
SNOHOMISH
WASHINGTON

NDCBU MAILBOX CLUSTER

Approved By:
City Engineer
Date: May 30, 2004

321a
Number

City of Snohomish Public Works Department



NOTES:

1. THIS DRAWING DEPICTS A MINIMUM STRUCTURAL AND DIMENSIONAL STANDARD FOR NEIGHBORHOOD CLUSTER MAILBOX UNIT AND PADS. FOR SPECIFIC POSTAL REQUIREMENTS CONTACT THE POSTMASTER.
2. MAILBOXES MUST BE POSTMASTER APPROVED WITH A UNIFORM BOX STYLE AND METHOD OF ADDRESS IDENTIFICATION.
3. LOCATIONS OF MAILBOXES ARE SUBJECT TO APPROVAL BY THE CITY ENGINEER FOR PROTECTION OF VIEWS AND ACCESS.
4. INSTALLATION OF DRAINAGE CULVERT MAY BE NECESSARY IN AREAS WHERE THERE IS NO CONCRETE SIDEWALK AND THE REQUIRED SETBACK SPANS A ROADSIDE DITCH. ACCESS TO SUCH STRUCTURES WILL HAVE A MAX. SLOPE OF 2%. AND SHALL HAVE A PAD CONSISTING OF A MINIMUM OF 2" OF CRUSHED SURFACING TOP COURSE COMPACTED TO 95% MAXIMUM DENSITY.
5. ALL MAILBOX STRUCTURES SHALL BE PLACED BACK OF SIDEWALK WITH NO PORTION OF BOX OR STRUCTURE PROTRUDING INTO THE SIDEWALK. IF NO SIDEWALK EXISTS SETBACK WILL BE SET BY THE CITY ENGINEER.
6. SUGGESTED SOURCE SECURITY MFG CORP (800) 762-6937, 800 SERIES CLUSTER BOX UNITS, SALSBURY INDUSTRIES (800) 323-3003 OR POSTAL APPROVED EQUAL.
7. PLACEMENT LOCATION OF PEDESTAL PARCEL LOCKER WILL BE APPROVED BY THE CITY ENGINEER AND THE POSTAL SERVICE.

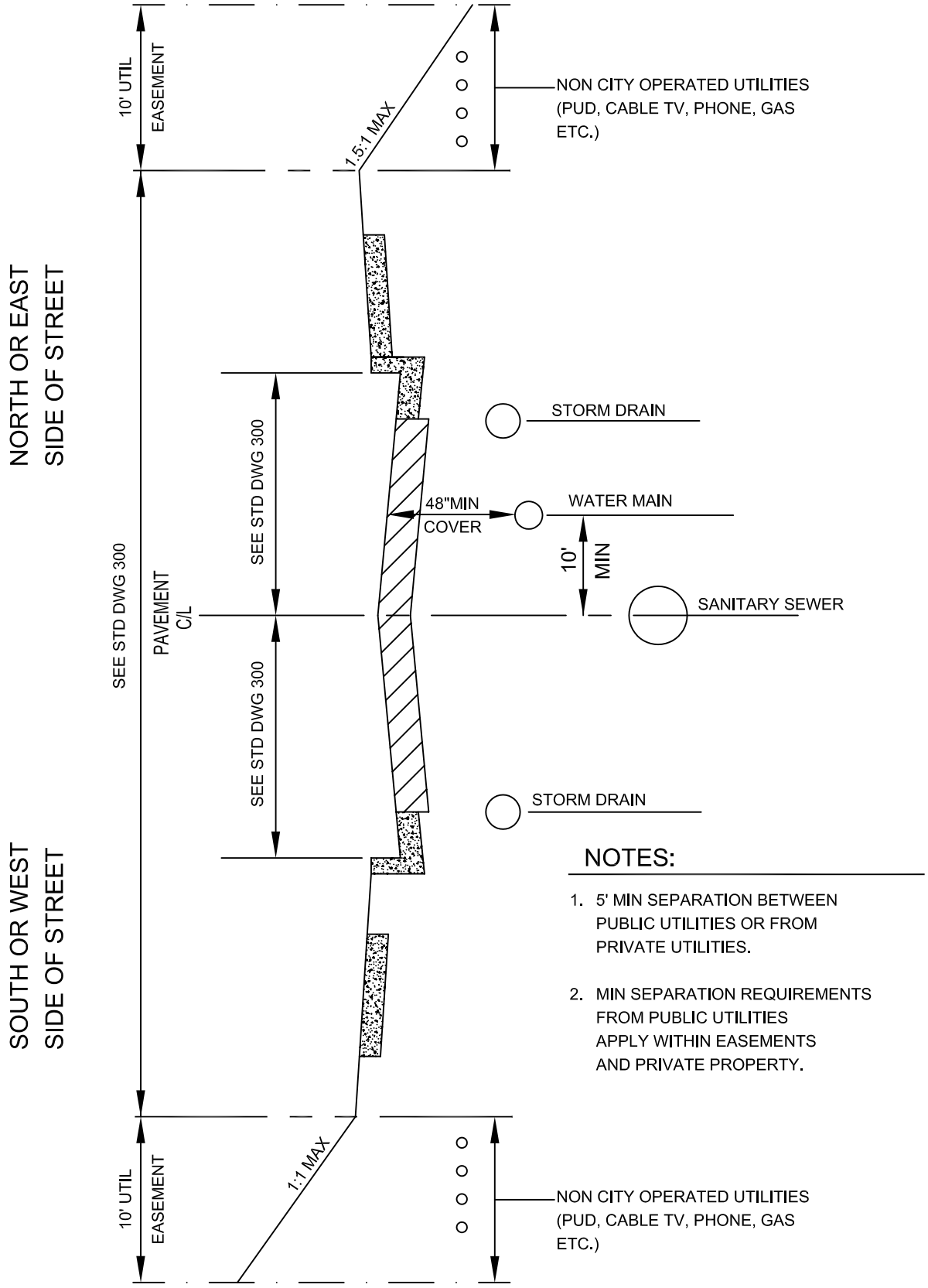
SNOHOMISH
WASHINGTON

CLUSTER MAILBOX UNITS FRONT OPENING ONLY

Approved By:
City Engineer
Date: May 30, 2004

321b
Number

City of Snohomish Public Works Department



NOTES:

1. 5' MIN SEPARATION BETWEEN PUBLIC UTILITIES OR FROM PRIVATE UTILITIES.
2. MIN SEPARATION REQUIREMENTS FROM PUBLIC UTILITIES APPLY WITHIN EASEMENTS AND PRIVATE PROPERTY.

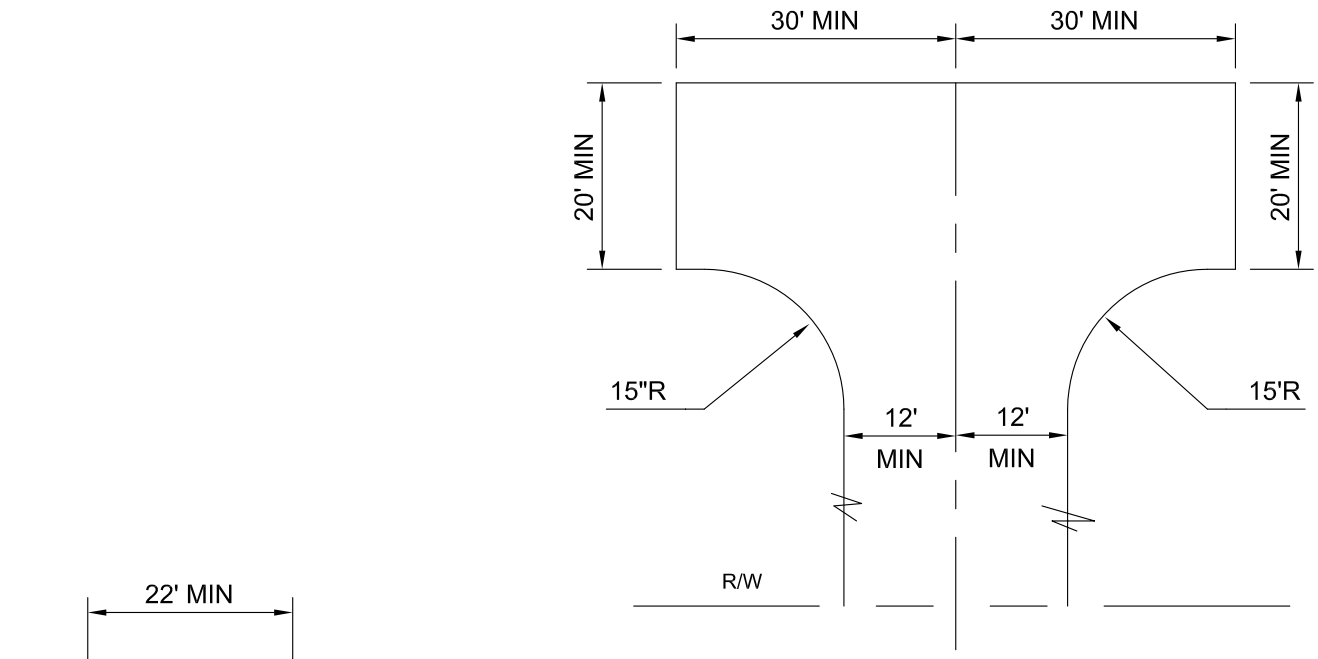
SNOHOMISH WASHINGTON

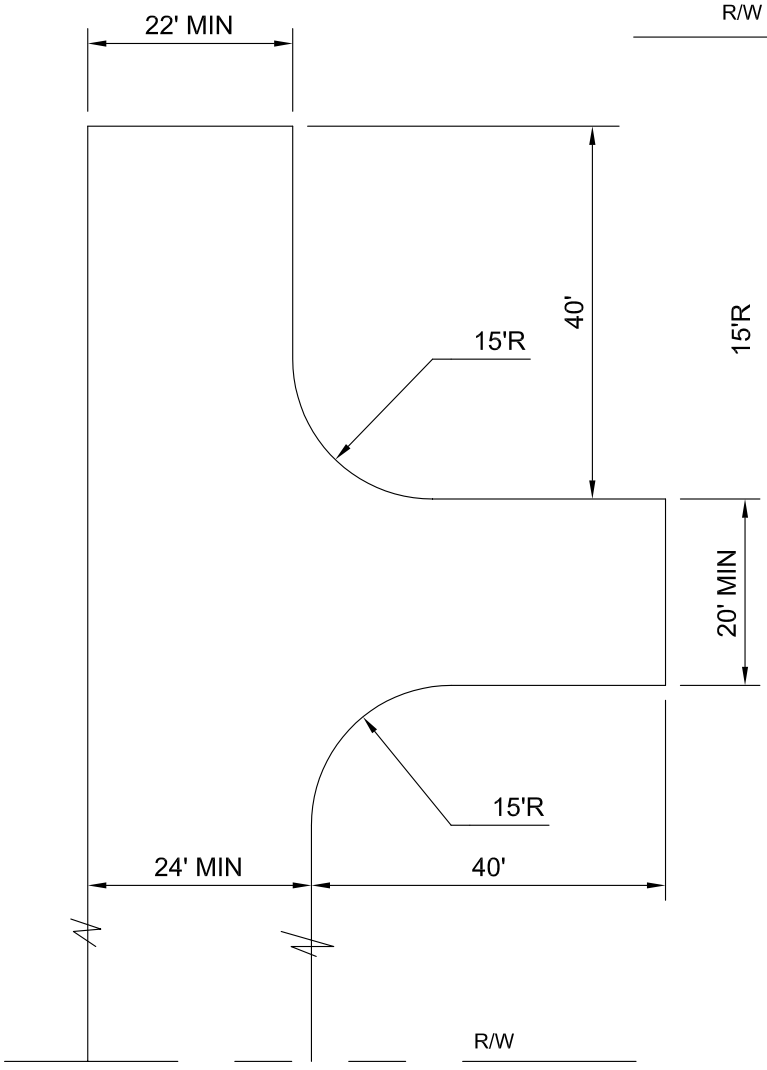
TYPICAL UTILITY LOCATIONS

Approved By: City Engineer
Date: May 30, 2004

322
Number

City of Snohomish Public Works Department





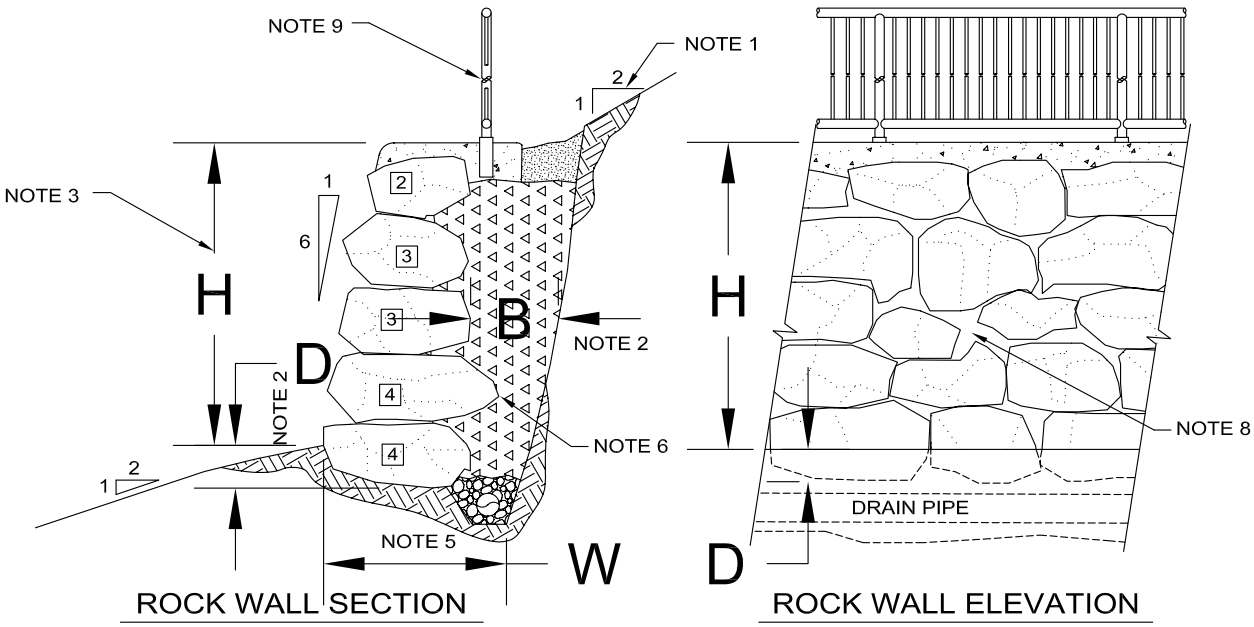
SNOHOMISH
WASHINGTON

TEMPORARY TURNAROUNDS

Approved By:
City Engineer
Date: May 30, 2004

323
Number

City of Snohomish Public Works Department



ROCK WALL SECTION

ROCK WALL ELEVATION

NOTES

1. MAXIMUM INCLINATION OF THE SLOPES ABOVE AND BEHIND ROCK WALL SHALL BE 2:1 (HORIZONTAL:VERTICAL)
2. MINIMUM THICKNESS OF ROCK FILTER LAYER B=12 INCHES. MINIMUM EMBEDMENT D=12 INCHES.
3. MAXIMUM ROCK WALL HEIGHT H=8 FEET. ROCK WALLS GREATER THAN 8 FEET IN HEIGHT SHALL BE DESIGNED BY A CIVIL ENGINEER LICENSED IN THE STATE OF WASHINGTON.
4. ROCK SHALL BE PLACED TO GRADUALLY DECREASE IN SIZE WITH INCREASING WALL HEIGHT.
5. MINIMUM WIDTH OF KEYWAY EXCAVATION W, SHALL BE EQUAL TO THE THICKNESS OF THE BASE ROCK PLUS B (ROCK FILTER)
6. THE LONG DIMENSION OF THE ROCKS SHALL EXTEND BACK TOWARDS THE CUT OR FILL FACE TO PROVIDE MAXIMUM STABILITY.
7. WHENEVER POSSIBLE EACH ROCK SHALL BEAR ON TWO OR MORE ROCKS BELOW IT, WITH GOOD FLAT-TO-FLAT CONTACT.
8. WHERE VOIDS OF GREATER THAN 6 INCHES IN DIMENSIONS EXIST IN THE ROCK FACE AND THERE IS NO ROCK CONTACT WITHIN THE ROCK WALL THICKNESS, THE VOID SHALL BE CHINKED WITH SMALL PIECES OF ROCK.
9. ROCKERIES WHICH ARE MORE THAN 30 INCHES ABOVE GRADE OR FLOOR BELOW SHALL BE PROTECTED BY GUARDRAIL SUCH AS A ORNAMENTAL OR PEDESTRIAN RAIL. TYPE TO BE DETERMINED BY THE CITY ENGINEER, SEE DWGS 325 & 326
10. FOR DESIGN LOCATION AND UNDERGROUND UTILITY LIMITATIONS REFEER TO STD DWG 324B.

NOTES (CONT)

11. THE DENSITY OF ROCK MATERIAL SHALL BE A MINIMUM OF 155 PCF. THE SIZE CATEGORIES FOR ROCK SHALL BE AS FOLLOWS:

SIZE	APPROXIMATE WEIGHT - LBS.	APPROXIMATE DIAMETER-INCHES
1 MAN	50 -200	12 - 18
2 MAN	200 -700	18 - 28
3 MAN	700 - 2000	28 - 36
4 MAN	2000 - 4000	36 - 48
5 MAN	4000 - 6000	48 - 54
6 MAN	6000 - 8000	54 - 60

LEGEND

- DRAINAGE MATERIALS TO CONSIST OF CLEAN 4"-2" ANGULAR SPALLS.
- NO. 2 COARSE AGGREGATE PER WSDOT/APWA 9-03.1(3)C (6" ABOVE PIPE MIN)
- CONCRETE ROCKERY CAP. REQUIRED IN R.O.W., OPTIONAL ON PRIVATE PROPERTY.
- UNDISTURBED FIRM NATIVE SOIL
- SEED OR SOD ON 12" OF TOPSOIL WITH UNDERLAYER OF FILTER FABRIC.
- 4 INCH DIAMETER, HDPE OR SDR35 PVC, PERFORATED OR SLOTTED, WITH SMOOTH INTERIOR PIPE. SET SLIGHTLY LOWER THAN THE BASE ROCK TO PREVENT DAMAGE. LAY WITH A POSITIVE SLOPE TO DISCHARGE AWAY FROM ROCKERY.
- [4] DESIGNATES SIZE OF ROCK, I.E. 4 MAN. SEE NOTE 11.

SNOHOMISH WASHINGTON

ROCKERY

(DESIGN AND CONSTRUCTION REQUIREMENTS)

Approved By:

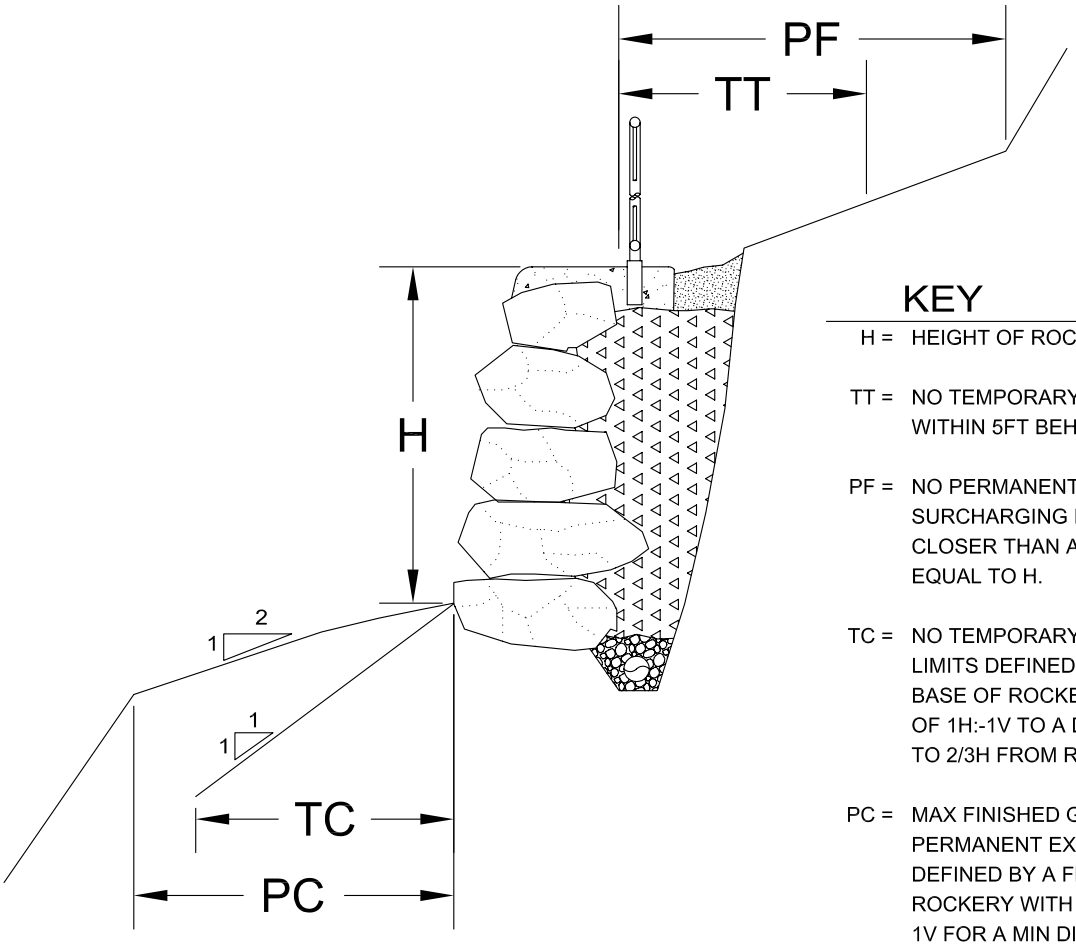
City Engineer

Date: May 30, 2004

324a

Number

City of Snohomish Public Works Department



KEY

H = HEIGHT OF ROCK WALL.

TT = NO TEMPORARY EXCAVATION WITHIN 5FT BEHIND ROCKERY.

PF = NO PERMANENT EXCAVATION OR SURCHARGING BEHIND ROCKERY CLOSER THAN A DISTANCE EQUAL TO H.

TC = NO TEMPORARY EXCAVATION BELOW LIMITS DEFINED BY A LINE FROM BASE OF ROCKERY WITH A SLOPE OF 1H:-1V TO A DISTANCE EQUAL TO 2/3H FROM ROCKERY BASE.

PC = MAX FINISHED GRADE OR PERMANENT EXCAVATION DEFINED BY A FROM BASE OF ROCKERY WITH A SLOPE OF 2H:-1V FOR A MIN DISTANCE EQUAL TO H FROM ROCKERY BASE.

DESIGN AND POST CONSTRUCTION LIMITATIONS

NOTES

1. ALL NEW ROCKERY DESIGN AND PLACEMENT WILL FOLLOW TO CONSTRUCTION LIMITATIONS DESCRIBE ABOVE, AND FOLLOW THE GUIDELINES ESTABLISHED BY THE ASSOCIATED ROCKERY CONTRACTORS "STANDARD ROCK WALL CONSTRUCTION GUIDELINES" DATED 12/2/92 INCLUDING ANY AND ALL REVISIONS.

2. MODIFICATIONS TO OR PLACEMENT OF SUBSEQUENT UNDERGROUND UTILITIES WILL ALSO FOLLOW LIMITATIONS DESCRIBED ABOVE.

SNOHOMISH WASHINGTON

ROCKERY

(PLACEMENT & POST CONSTRUCTION LIMITS)

Approved By:

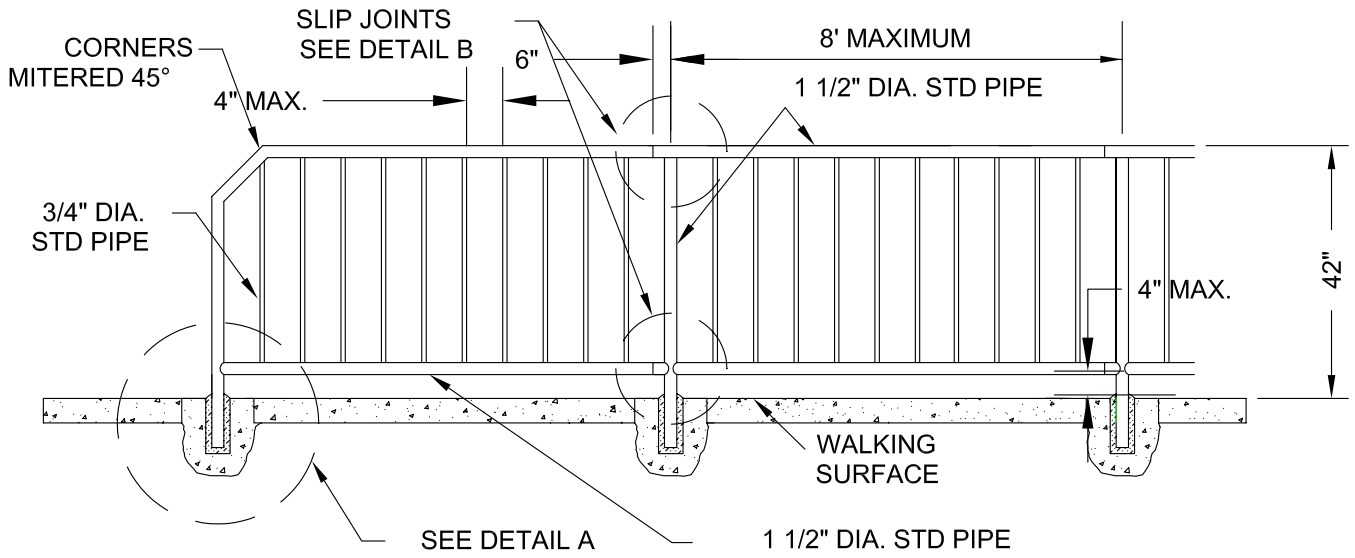
City Engineer

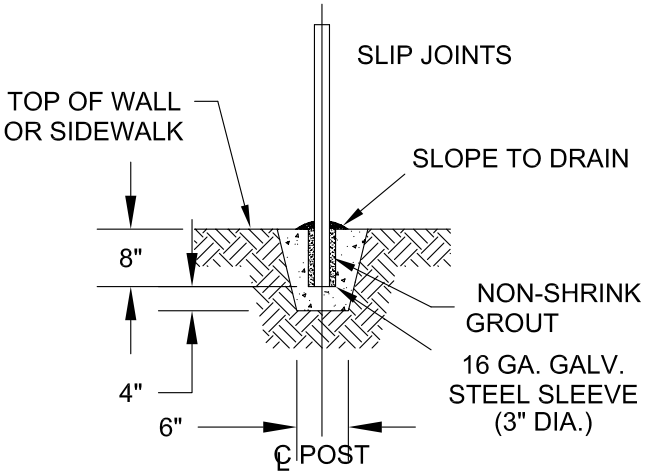
Date: May 30, 2004

324b

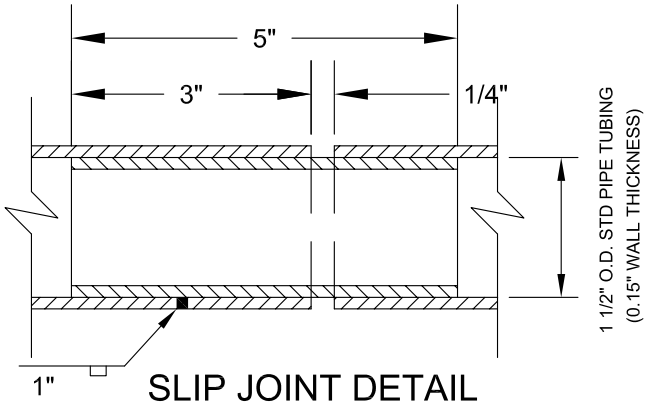
Number

City of Snohomish Public Works Department



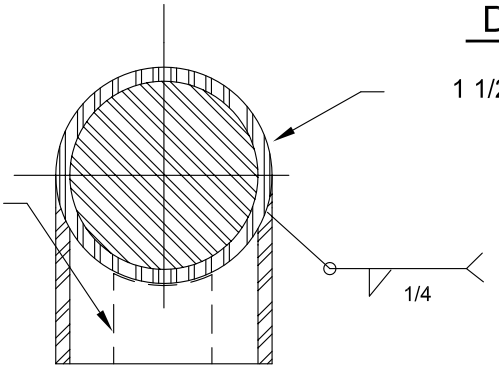


DETAIL A



SLIP JOINT DETAIL

DETAIL B



3/4" SCH 40 (STD PIPE) BALUSTERS INSERTED IN HOLE AND TACK WELDED OPPOSITE TRAFFIC

TYP. @ EACH POST FOR ALL RAIL SECTIONS

NOTES:

1. MATERIAL FOR PEDESTRIAN HANDRAIL SHALL BE ALUMINUM (ASTM B-429) OR GALVINIZED STEEL (ASTM 120) AS APPROVED BY THE CITY ENGINEER.
2. SEE STANDARD DRAWING No. 325A FOR ADDITIONAL FABRICATION AND SPECIFICATION REQUIREMENTS.
3. PROVIDE SLIP JOINTS AT STAIRWAY EXPANSION JOINTS AND AT EVERY 24 FEET ON CENTER MAXIMUM.

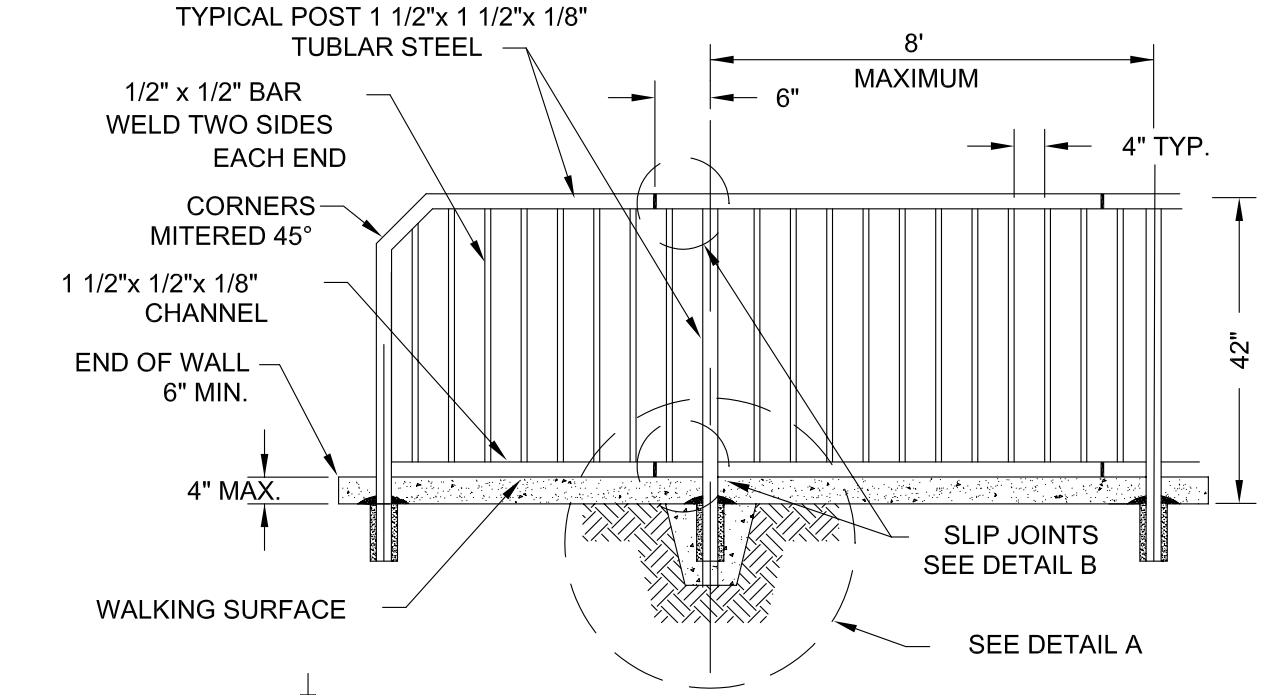
SNOHOMISH WASHINGTON

PEDESTRIAN HANDRAIL

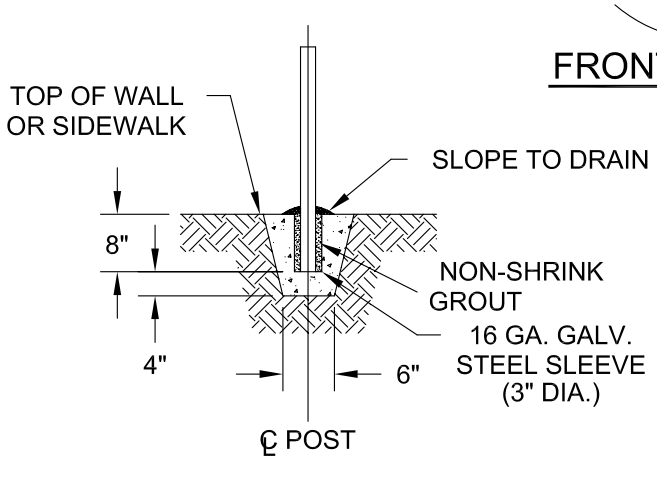
Approved By:
City Engineer
Date: May 30, 2004

325
Number

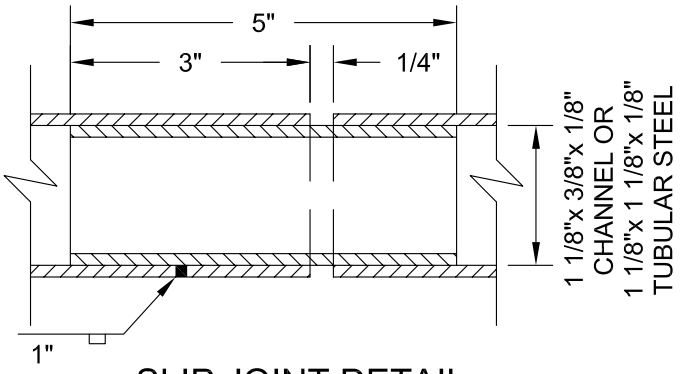
City of Snohomish Public Works Department



FRONT VIEW



DETAIL A



SLIP JOINT DETAIL

DETAIL B

NOTES:

1. ORNAMENTAL RAILING SHALL BE CONSTRUCTED OF STEEL CONFORMING TO ASTM A-120.
2. WELDING SHALL CONFORM TO THE REQUIREMENTS OF THE "STRUCTURAL WELDING CODE" AWS D 1.1.
3. PROVIDE SLIP JOINTS AT STAIRWAY EXPANSION JOINTS AND AT EVERY 24 FEET ON CENTER MAXIMUM.
4. MAXIMUM SPACING OF POSTS SHALL BE 8 FEET ON STRAIGHT ALIGNMENT AND 6 FEET ON CURVED ALIGNMENT LESS THAN 30 FEET RADIUS.
5. AFTER FABRICATION, ALL BURRS AND SHARP EDGES SHALL BE REMOVED.
6. APPLY RUST PROOF METAL PRIME AND ONE COAT OF BLACK ORNAMENTAL IRON METAL PAINT.

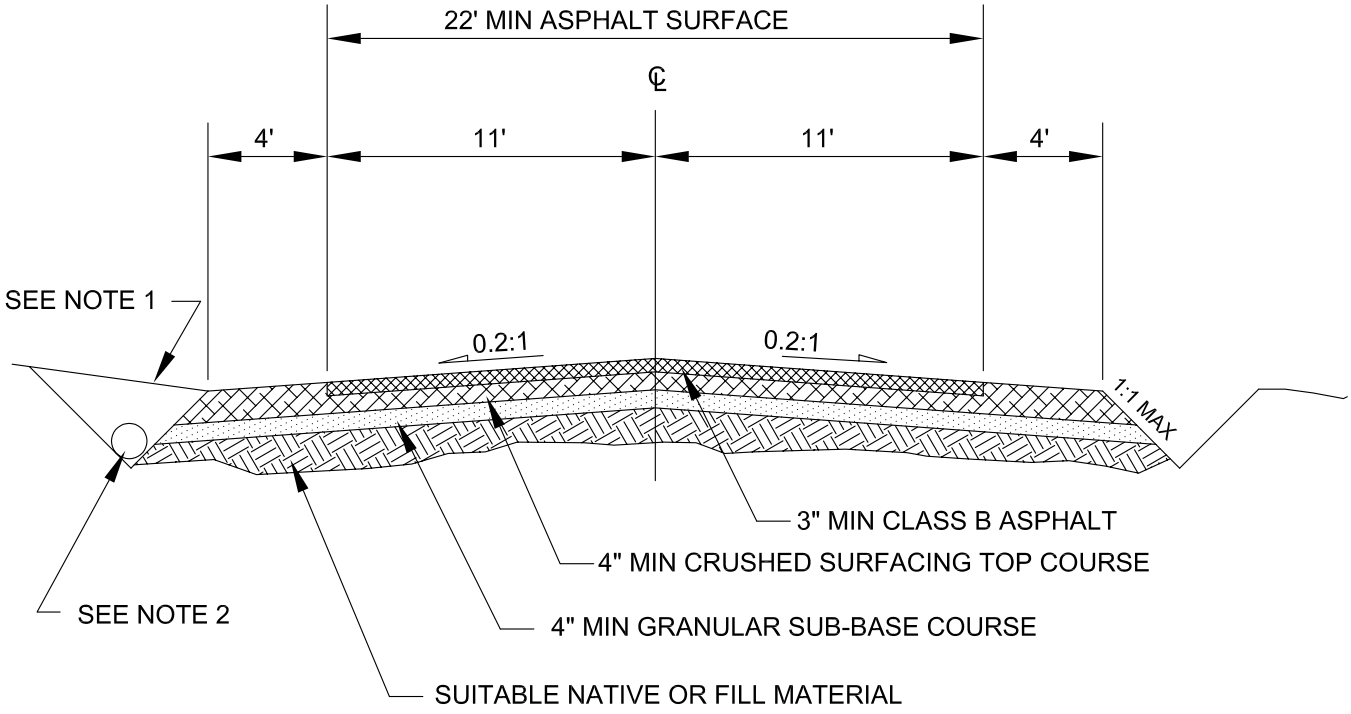
SNOHOMISH
WASHINGTON

ORNAMENATL HANDRAIL

Approved By:
City Engineer
Date: May 30, 2004

326
Number

City of Snohomish Public Works Department



NOTES:

1. DRIVE GRADE AT RIGHT-OF-WAY LINE SHALL CONFORM TO SECTION 3 UNLESS OTHERWISE APPROVED BY CITY ENGINEER.

2. A 12 INCH MINIMUM CONCRETE OR CORRUGATED POLYETHYLENE SMOOTH INTERIOR PIPE IS REQUIRED UNDER ALL DRIVEWAYS.

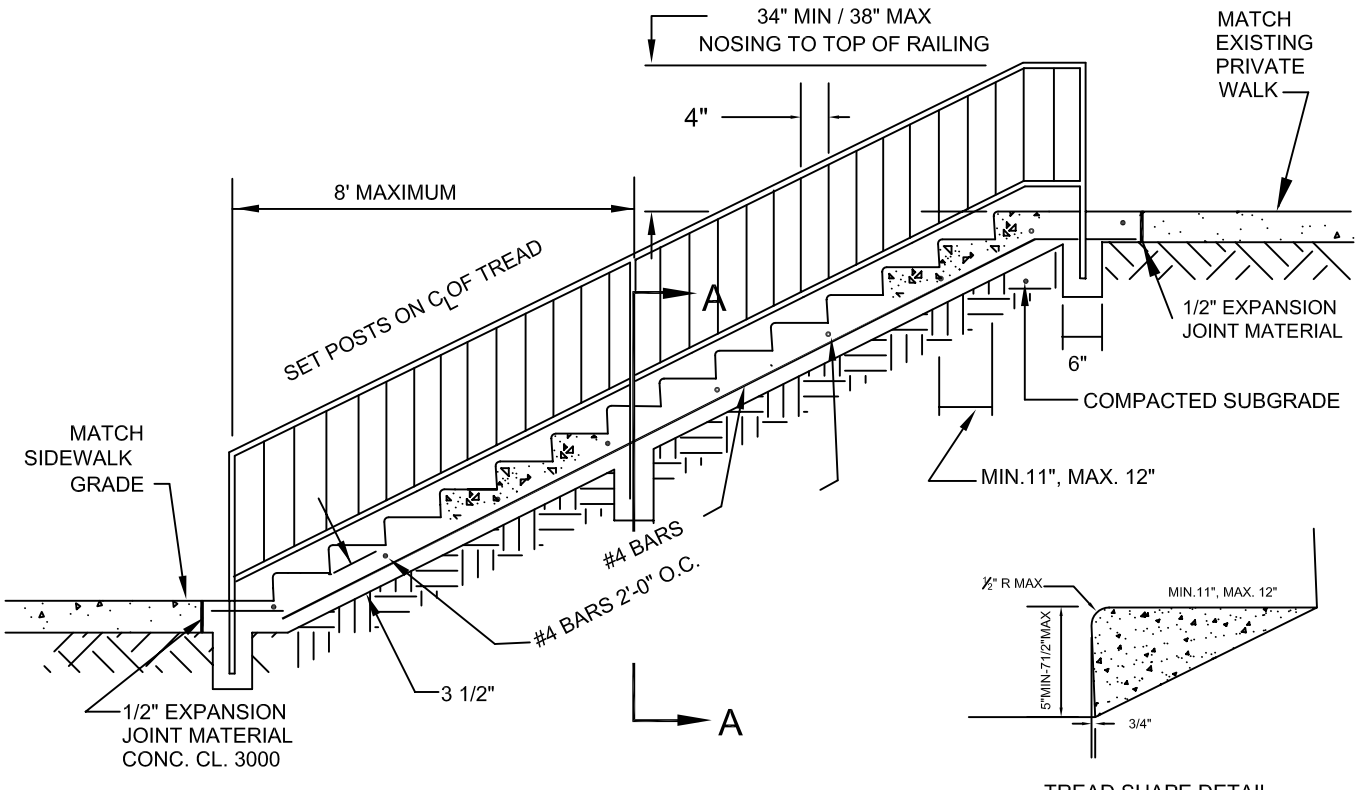
SNOHOMISH WASHINGTON

TYPICAL ROADWAY SECTION SPECIAL INTERIM STREET

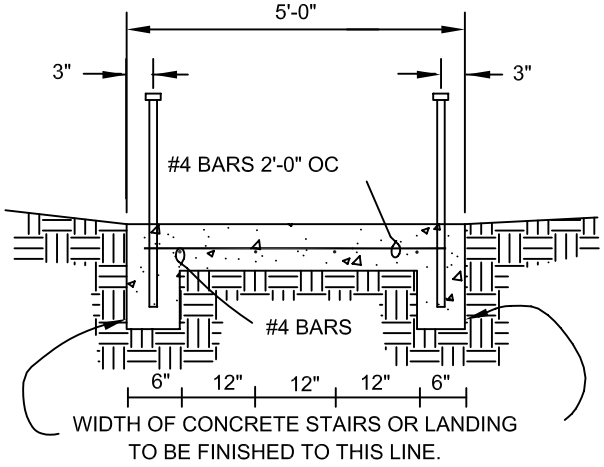
Approved By:
City Engineer
Date: May 30, 2004

327
Number

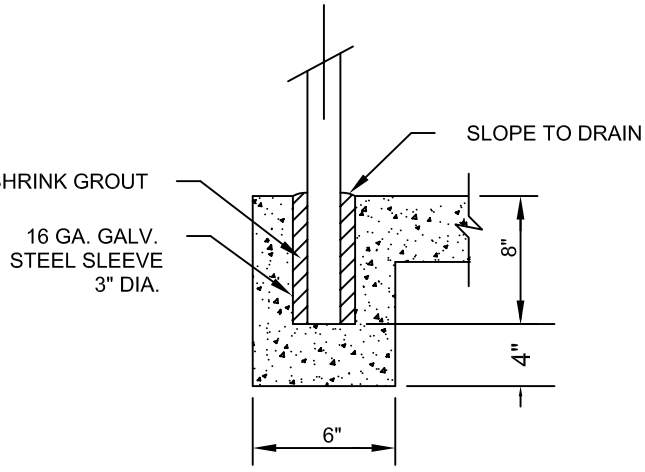
City of Snohomish Public Works Department



TREAD SHAPE DETAIL



SECTION A-A



POST DETAIL

NOTES:

1. CEMENT CONCRETE STEPS SHALL BE 4' WIDE, RISERS SHALL BE 5" MIN., 7" MAX. TREAD SHALL BE 11" MIN. 12" MAX.

2. HEIGHT OF RAILING SHALL BE 34" MINIMUM, 38" MAXIMUM TOP OF NOSING TO TOP OF RAILING.

3. USE PEDESTRIAN OR ORNAMENTAL HANDRAIL AS DIRECTED BY THE CITY ENGINEER. SEE STANDARD DRAWING NOS. 325, 325A, AND 326.

4. CLEAR SPACE BETWEEN BALUSTERS SHALL BE A MAXIMUM OF 4".

5. STEPS WITH MORE THAN 4 RISERS SHALL HAVE HANDRAIL ON BOTH SIDES.

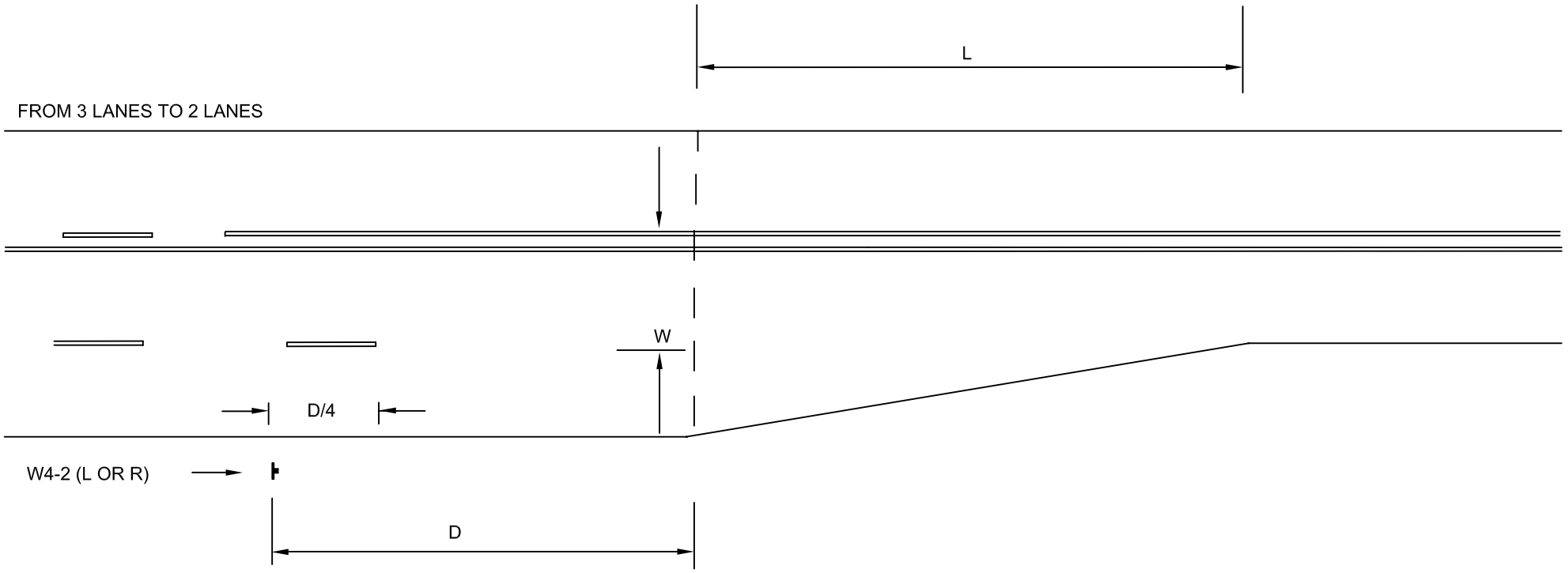
SNOHOMISH
WASHINGTON

CEMENT CONCRETE STAIRWAY CONSTRUCTION DETAILS

Approved By:
City Engineer
Date: May 30, 2004

328
Number

City of Snohomish Public Works Department



STANDARD:

THE MINIMUM TAPER LENGTH SHALL BE 100 FEET IN URBAN AREAS AND 200 FEET IN RURAL AREAS.

FORMULA WHEN POSTED SPEED IS:

>45 MPH, L=WS.

<45 MPH, L=WS²/60.

VARIABLE LEGEND:

L=LENGTH IN FEET

S=POSTED SPEED OR 85th-PERCENTILE SPEED, WHICHEVER IS GREATER.

W=OFFSET IN FEET

D=ADVANCE WARNING DISTANCE. SEE SECTION 2C.05 OF MUTCD FOR PLACEMENT.

SNOHOMISH
WASHINGTON

TYPICAL LANE REDUCTION TRANSITION

Approved By:

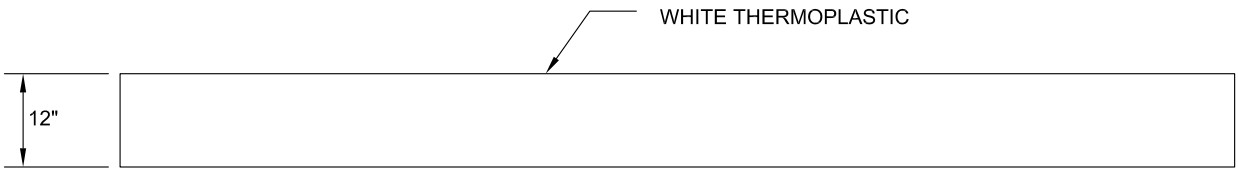
City Engineer

Date: May 30, 2004

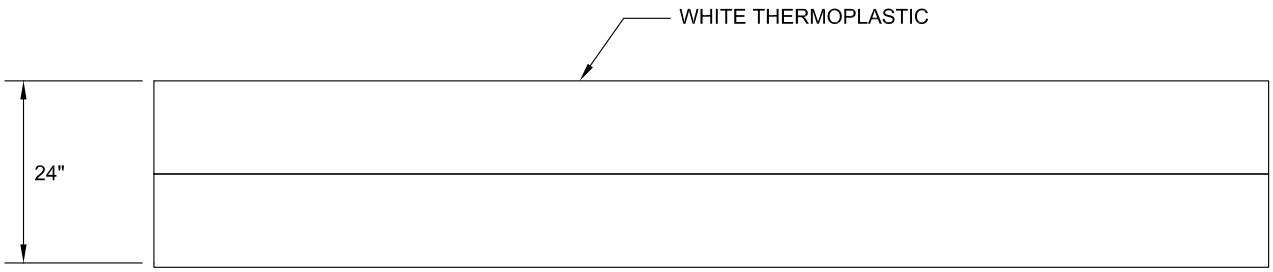
329

Number

City of Snohomish Public Works Department



NON-ARTERIAL



ARTERIAL

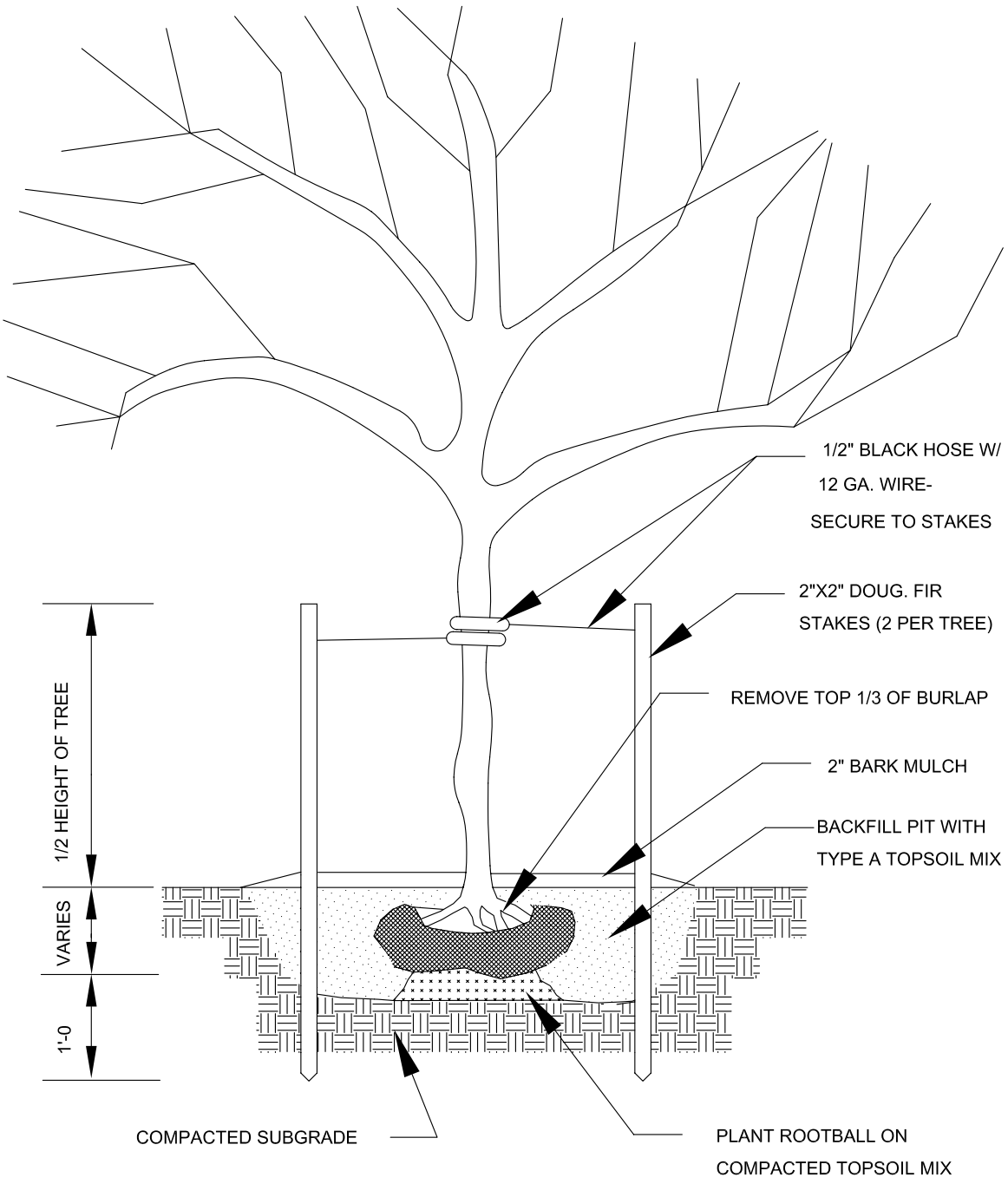
SNOHOMISH
WASHINGTON

STOP BARS

Approved By:
City Engineer
Date: May 30, 2004

330
Number

City of Snohomish Public Works Department



TREE PLANTING AND STAKING DETAIL

NTS

TREE PLANTING SHALL CONFORM TO THE
REQUIREMENTS OF SECTION 8-02 OF
THE WSDOT / APWA STANDARD SPECIFICATIONS.

SNOHOMISH
WASHINGTON

TYPICAL TREE PLANTING

Approved By:
City Engineer
Date: May 30, 2004

333
Number

City of Snohomish Public Works Department

SECTION 4 STORM AND SURFACE WATER

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4-2.2	Design Flow and Route Requirements	4-2
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DESIGN AND CONSTRUCTION STANDARDS
AND
SPECIFICATIONS

SECTION 4

STORM AND SURFACE WATER

4-1 GENERAL

This section of the Standards provides criteria for the design of storm and surface water drainage systems, including storm pipes, culverts, and catch basins. Each of the sections in this chapter contains the design criteria and references standard details for the various systems or facilities.

Drainage control and storm water treatment shall be provided for all property improvements within the City of Snohomish per these Standards and current version of the Department of Ecology (DOE) Storm Water Management Manual and the City's Land Use Code.

Projects which will disturb one acre or more of land, or create 5,000 square feet or more of impervious surface, paved or otherwise, must refer to the said DOE Storm Water Management Manual for drainage plan and runoff control requirements.

The use of Low-Impact Development concepts are strongly encouraged where they do not conflict with the requirements of these Standards. Low-Impact Development concepts include such practices as preserving native vegetation for stormwater dispersal, soil amendments to promote stormwater infiltration, pervious pavements, and vegetated swales.

4-2 STORM DRAINAGE CONVEYANCE SYSTEM DESIGN CRITERIA

4-2.1 OVERVIEW

For the purposes of this section, the conveyance system includes all portions of the surface water system that transport storm and surface water runoff, either natural or man-made, except those features protected as environmentally sensitive areas under the City's land use code. Environmentally sensitive areas may only be modified as allowed under the City of Snohomish Land use Code. Storm water must generally be treated and detained prior to discharge to an environmentally sensitive area, including those features created for mitigation.

This section covers the following components of the conveyance system:

Pipe systems
Culverts
Outfalls
Open channels

4-2.2 DESIGN FLOW AND ROUTE REQUIREMENTS

Design Flow

All existing and proposed conveyance systems shall be designed to convey runoff from the 25-year storm. The method used to determine the design flow will depend on the characteristics of the drainage area and the type of conveyance. Tight-lined conveyance systems receiving runoff from relatively small and/or highly impervious drainage areas shall be sized using flows determined using the rational method. Flows for culverts or channels receiving runoff from large and/or relatively pervious drainage areas shall be sized using flows determined using hydrograph methods.

A backwater analysis may be required for a proposed or existing pipe system if the ability of the pipe system to convey the peak rate of runoff from the 25-year design storm event may be effected by tail water conditions (outlet control) anywhere in the pipe system.

Structures such as catch basins and manholes for proposed pipe systems, such as catch basins and manholes, must provide a minimum of 0.5 feet of freeboard between the headwater surface (hydraulic grade line) and the top of the structure for flow from a 25-year storm. However, structures may overtop for flow resulting from a 100-year storm. When overtopping occurs for the 100-year storm, the additional flow over the ground surface is analyzed using the methods described in **Section 1-6 and is added to the flow capacity of the pipe system.

Easements and Building Setback Lines

All public and privately maintained conveyance systems shall be located in drainage easements in accordance with **Section 4-1.3 and **Section 4-2.1.3.8. The exceptions are roof downspout and minor yard and footing drains, unless they serve other adjacent properties.

4-2.3 PIPE SYSTEMS

Pipe systems are networks of storm drain pipes, catch basins, manholes, inlets, and outfalls designed and constructed to convey storm and surface water. The hydraulic analysis of flow in storm drain pipes is typically limited to "gravity flow". The following subsections give design criteria for different components and aspects of pipe systems.

4-2.3(1) PIPE MATERIALS

1. Pipe material, joints, and protective treatment shall conform to the requirements set forth in Section 9-05 of the WSDOT/APWA Standard Specifications.

   The following pipe materials are allowed for use in pipe systems in the City of Snohomish; other pipe materials may be approved on a case-by-case basis:

   (a) Plain concrete pipe (12 inches in diameter, used only for driveway culvert)
   (b) Reinforced concrete pipe
   (c) Ductile iron pipe

(d) PVC pipe (SDR35, ASTM D3034 with 3 feet of cover, minimum)
(e) Corrugated high density polyethylene pipe, with smooth interior
(f) Profile wall PVC, culvert pipe, profile wall PVC storm sewer pipe (AASHTO M304, or ASTM F794 Series 46).

2. Coupling bands shall be of the same material as the pipe.

3. Materials for concrete, rubber gaskets, metal castings, reinforcing steel, and masonry units shall meet the requirements of the appropriate sections of the WSDOT/APWA Standard Specifications.

4-2.3(2) PIPE SIZES, SLOPES, AND VELOCITIES

1. No storm drain pipe between catch basins or manholes in the public right-of-way shall be less than 12 inches in diameter; with the exception that 8-inch pipe may be used between inlets and catch basins in runs of 50 feet or less.

2. The minimum velocity in any pipe or culvert carrying the design storm flow shall be two feet per second.

3. The maximum allowable velocity in concrete pipe shall be 30 feet per second.

4. Changes of pipe size are allowed only at junctions, and structures must be located at all junctions.

5. Downstream decrease in pipe size is not a recommended practice and will only be allowed under special conditions.

4-2.3(3) STRUCTURES

1. Manholes, catch basins, and inlets shall be constructed of pre-cast units in accordance with the following Standard Plans:

Item	Section
Manholes	605A-C
Manhole ring and cover	405
Catch Basins	402 through 404
Inlets	401
Steps and Ladder	606, 606A
Frames and Grates	405A through 409

2. Catch basin (or manhole) diameter shall be determined by pipe size and orientation at the junction structure. A plan view of the junction structure, drawn to scale, will be required when more than four pipes enter the structure on the same plane, or if angles of approach and clearance between pipes is of concern. The plan view (and sections if necessary) must insure a minimum solid concrete wall distance between pipe openings of 8 inches for 48-inch and 54-inch catch basins and 12 inches for 72-inch and 96-inch catch basins.

3. Catch basin evaluation of structural integrity for H-20 loading may be required for multiple junction catch basins and other structures.

4. Catch basins shall be provided within 50 feet of the entrance to a pipe system to provide for silt and debris removal.

5. HDPE pipe systems longer than 100 feet must be secured at the upstream end and the downstream end must be placed in a four foot section of the next larger pipe size. This sliding sleeve connection accounts for the high thermal expansion/contraction coefficient of this pipe material.

6. The maximum slope of the ground surface for a radius of five feet around a catch basin grate shall be 3:1.

7. A Type II catch basin or a manhole shall be required when the depth to the flow-line exceeds 5.5 feet, regardless of the pipe size.

8. All Type II catch basins and all manholes shall be equipped with ladders per Standard Plan No. 606 or 606A.

9. A Type II catch basin shall be installed as the last collector in the public right-of-way prior to discharge to the combined sanitary sewer.

10. Concrete inlets shall not be used where the discharge goes directly into the main storm drain system.

4-2.3(4) PIPE ALIGNMENT/CONNECTIONS/COVER

1. Pipes must be laid true to line and grade with no curves, bends, or deflections in any direction, except for HDPE and ductile iron with flanged restrained mechanical joint bends (not greater than 30 degrees) on steep slopes.

2. A catch basin or manhole will be required at all changes in storm drain diameter and changes in grade or alignment.

3. Connections to a pipe system shall be made only at catch basins or manholes. No wyes or tees are allowed except on roof, footing, or yard drain systems on pipes eight inches in diameter or less, with clean-outs upstream of each wye or tee.

4. Six inches minimum vertical and three feet minimum horizontal clearance (between outside surfaces) shall be provided between storm drain pipes and other utility pipes and conduits.

5. Any closed storm drainage system collecting runoff from paved areas in the public right-of-way or private property shall provide for floatable material separation (see Standard Plan No. 410) prior to discharge to the main storm drainage system in the public right-of-way, unless otherwise approved by the City Engineer.

6. All PVC connections to catch basins or manholes shall be made by grouting in an approved manhole adapter into which the PVC pipe is inserted.

7. Activities such as trench excavation, tunneling or boring, pipe embedment, backfilling, compaction, safety and pavement patching, whether for public or private utilities, shall conform to the requirements set forth in other sections of these Standards. For all the above, except pavement patching, see Section 3-9 Underground Utilities and Standard Plan Nos. 610, 611, and 615. For pavement patching see Section 3-14 and Standard Plan No. 316.

4-2.3(5) FRAMES/LIDS/GRATES/COVERS

1. In general, frames and grates shall be furnished and installed per Standard Plan Nos. 405 through 409.

2. The cover or grating of a manhole or catch basin shall not be grouted to final grade until the final elevation of the pavement, gutter, ditch, or sidewalk in which it is to be placed has been established, and until permission thereafter is given by the City Inspector to grout the cover or grating in place.

3. Lids, grates, and covers shall be seated properly to prevent rocking.

4. All catch basins and manholes in unpaved areas shall be equipped with locking frames and lids or grates per Standard Plan Nos. 405 or 607.

5. All Type II catch basins and all manholes with catches shall be supplied with locking lids or grates per standard plan No. 405.

6. Type II catch basins and manholes functioning exclusively as access structures shall be equipped with round 24-inch covers and frames per Standard Plan No. 607.

7. Round lids on all storm drain structures shall have “Drain” cast into the lid.

8. In conditions when the effectiveness of a normal grate installation would be limited, an open curb face frame and grate shall be furnished and installed per Standard Plan Nos. 408 and 409. These conditions include high likelihood of clogging from leaf fall, especially in sag vertical curves; when the inlet is a surface drainage end point, such as a cul-de-sac; and when normal inlet grates may be passed over due to the road grade. The use of open curb face frame and grates must be approved by the City Engineer.

4-2.3(6) RESTRICTOR DEVICES

1. The minimum orifice size diameter allowed for use in the City of Snohomish is one inch.

2. Restrictor devices shall be constructed and installed in accordance with Standard Plan numbers 412A through 412C.

4-2.3(7) DEBRIS BARRIERS

Debris barriers (trash racks) are required on all pipes entering or leaving a closed pipe system, including pipes entering or leaving a control/restrictor manhole or catch basin from a surface-type BMP (detention pond, infiltration basin, wet pond, biofiltration swale, etc.).

4-2.3(8) DRAINAGE EASEMENTS

1. Public drainage easements a minimum of 15 feet wide are required for maintenance and access of pipes located outside of the public right-of-way which convey runoff from public streets or other facilities.

2. Where possible, pipes shall be located within the center of the drainage easement, but in no case shall a pipe be closer than five feet to a property line.

3. Where possible, drainage easements shall be contained on one lot, and not bisected by a lot line.

4. Shared private pipes must be located within private drainage easements.

5. Easements for downspout roof drains, yard drains, and footing drains are not required unless these systems are shared by more than one property owner.

4-2.4 CULVERT DESIGN CRITERIA

4-2.4(1) Headwater

1. For new culverts 18 inches in diameter or less, the maximum allowable 25-year design storm headwater elevation (measured from the inlet invert) shall not exceed two times the pipe diameter/arch culvert height used and shall be one foot or more below the road or parking lot sub-grade.

2. For new culverts larger than 18 inches in diameter, the maximum 25-year design storm headwater elevation for the new culvert shall be one foot or more below the road or parking lot sub-grade.

4-2.4(2) INLET

1. For culverts 18 inches in diameter and larger, the embankment around the culvert inlet shall be protected from erosion by rock lining or riprap as specified in **Table 4-2.1, except the length shall be a minimum of five feet (upstream of the culvert), and the height shall be at the design headwater elevation.

2. Trash racks/debris barriers are required on culverts that are over 60 feet in length and that are 12 inches to 36 inches in diameter. Exceptions are culverts on fish-bearing streams.

3. In order to maintain the stability of roadway embankments, concrete headwalls, wing walls, or tapered inlets and outlets may be required if right-of-way and/or easement constraints prohibit the culvert from extending to the toe of the embankment slope. Normally concrete inlet structures/headwalls installed in or near roadway embankments must be flush with and conform to the slope of the embankment.

4-2.4(3) OUTLETS

The receiving channel at the outlet shall be protected from erosion by rock lining, as specified in **Table 4-2.1, except the height shall be one foot above the maximum tail water elevation or one foot above the crown of the pipe, whichever is higher.

4-2.4(4) FISH PASSAGE

Guidance for designing culverts for fish passage must be obtained from the Washington State Department of Fish and Wildlife.

4-2.5 OUTFALL DESIGN CRITERIA

4-2.5(1) GENERAL

1. All outfalls (at a minimum) shall be provided with rock protection per **Table 4-2.1. For outfalls with a velocity at the design flow greater than 10 fps, a gabion dissipater or engineered energy dissipater shall be required.

2. Mechanisms which reduce velocity prior to discharge from an outfall are encouraged.

3. Engineered energy dissipaters, including stilling basins, drop pools, hydraulic jump basins, baffled aprons, and bucket aprons, are required for outfalls with velocity at design flow greater than 20 fps.

4. Inlet control will usually dictate outfall pipe system capacity. The inlet conditions should be carefully examined, as well as the consequences should the inlet to the pipe system become plugged or capacity exceeded.

4-2.5(2) OUTFALL SYSTEMS TRAVERSING STEEP SLOPES

1. Outfall systems constructed of pipe segments which are banded and/or gasketed are not acceptable for traversing steep slopes. Failure of the system will result from leaks which develop at the joints.

2. Continuously fused, welded or flange bolted mechanical joint pipe systems (such as high density polyethylene pipe (HDPEP) or ductile iron pipe with flange-bolted mechanical joints) with proper anchoring shall be used for outfall systems traversing steep slopes.

3. In general, outfall pipes systems shall be installed in trenches with standard bedding on slopes up to 20 percent. On slopes greater than 20 percent, outfall pipe systems shall be placed on the ground surface with proper pipe anchorage.

4. HDPEP outfall systems must be designed to address the material limitations as specified by the manufacturer, in particular thermal expansion/contraction and pressure design. Sliding sleeve connections to address thermal expansion and contraction shall be used. These sleeve connections consist of a section of the appropriate length of the next larger size diameter of pipe into which the outfall pipe is fitted. These sleeve connections must be located as close to the discharge end of the outfall system as is practical.

5. Flows of very high energy will require a specifically engineered energy dissipation structure as described above.

4-2.6 OPEN CHANNEL DESIGN CRITERIA

Open channels, either natural or artificial, may be used to convey storm water on and from a site. In general, however, natural channels are protected as environmentally sensitive areas under the City's Land Use Code and may not be used to convey untreated, undetained storm water. Alteration of these channels, including bank stabilization projects, requires special permits. Artificial channels are those constructed from upland areas specifically to convey storm and surface water. In general, the criteria in this section do not apply to biofiltration swales, which are primarily designed to treat storm water runoff.

When constructing artificial channels, vegetation-lined channels are preferred when properly designed and constructed. Rock-lining may be necessary along the length of channels or at specific locations (such as bends and outfalls) when a vegetative lining will not provide adequate protection from erosive velocities.

4-2.6 (1) ARTIFICIAL CHANNELS

1. Channel section geometry shall be trapezoidal. Side slopes shall not be steeper than 3H:1V for vegetation-lined channels and 2H:1V for rock-lined channels, unless the channel is engineered specifically for steeper slopes.

2. A minimum 0.5 ft freeboard above design flows must be provided.

3. Vegetation-lined channels shall have bottom slope gradients of five percent or less and a maximum average velocity at the design flow of five fps.

4. Rock-lined channels shall be used when design flow velocities exceed five fps. Rock lining shall be in accordance with **Table 4-2.2.

5. A maintenance access easement 15 feet wide (minimum) is required along all publicly maintained constructed channels located on private property. However, required easement widths and building setback lines may vary with channel top width. A minimum 15 feet setback must be provided between any structures and the top of the bank of the channel.

4-2.6 (2) ROCK-LINING

In rock-lined channels, stone (riprap) is placed on the channel sides and bottom to protect the underlying material from erosion. Proper riprap design requires the determination of the median size of stone, the thickness of the riprap layer, the gradation of stone sizes, and the selection of angular stone which will interlock when placed.

Research by the U.S. Army Corps of Engineers has provided criteria for selecting the median stone size, W_{50} (Figure 4-2.1). If the riprap is to be used in a highly turbulent zone, such as a culvert outfall, downstream of a stilling basin, at sharp changes in channel geometry, etc., the median stone (W_{50}) should be increased from 200 percent to 600 percent depending on the severity of the locally high turbulence. The thickness of the riprap layer should generally be twice the median stone diameter (D_{50}) or at least that of

the maximum stone. The riprap should have a reasonably well-graded assortment of stone sizes within the following gradation:

$1.25^3 (D_{max}/D_{50})^3$ 1.50, $(D_{15}/D_{50}) = 0.50$, $(D_{min}/D_{50}) = 0.25$

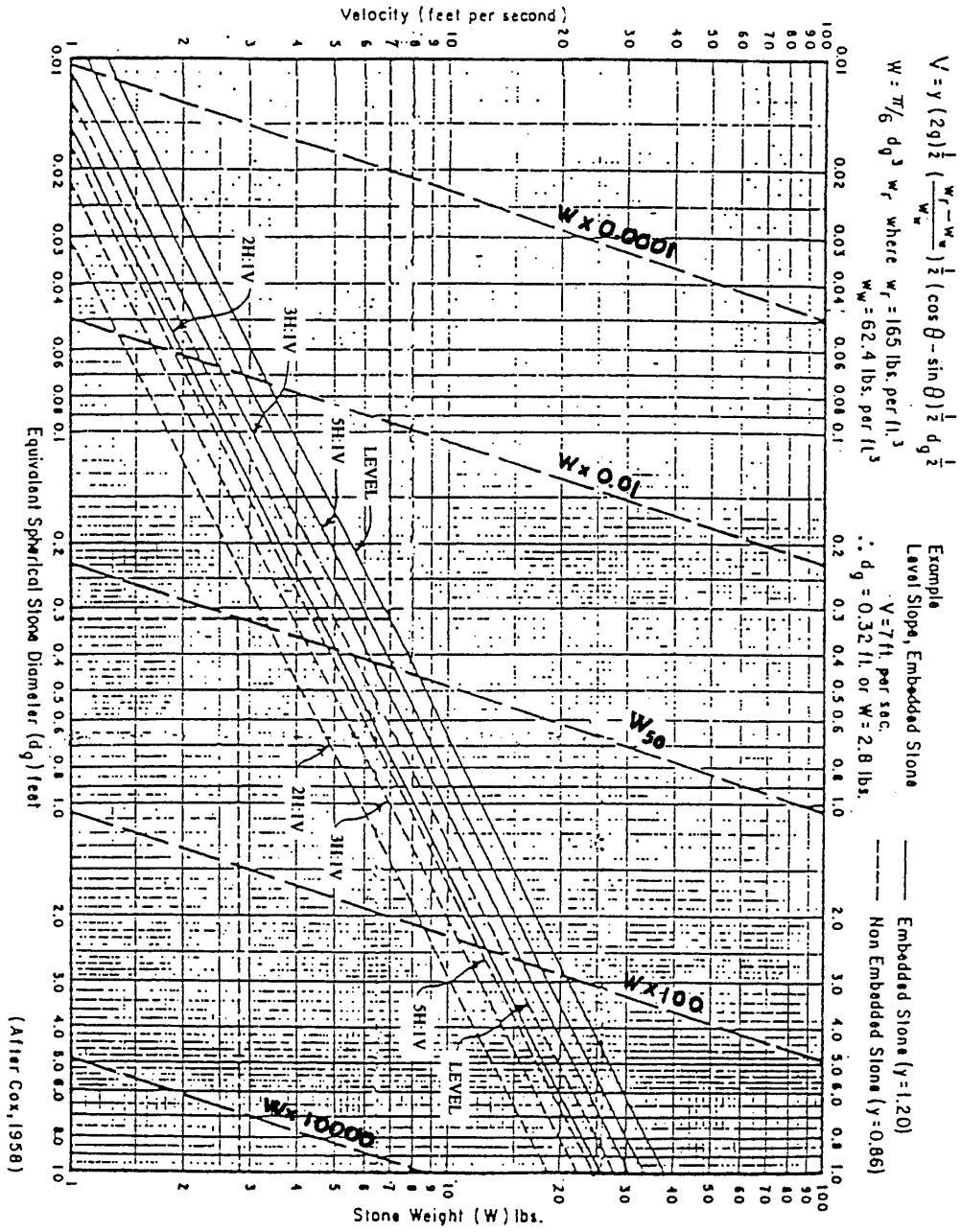
4-2.6 (3) RIPRAP FILTERS

Riprap should be underlain by a sand and gravel filter (or filter fabric) to keep the fine materials in the natural or artificial channel from being washed through the voids in the riprap. Likewise, the filter material must be selected so that it is not washed through the voids in the riprap. Adequate filters can usually be provided by a reasonably well-graded sand and gravel material with $D_{15} < 5d_{85}$, where d refers to the sieve opening through which 85 percent of the material being protected will pass and D_{15} has the same interpretation for the filter material. A filter with a D_{50} of 0.5 mm will protect any finer material including clay. Where very large riprap is used, it is sometimes necessary to use two filter layers between the material being protected and the riprap.

For additional information and procedures for specifying filters for riprap and general guidance, refer to the Army Corps of Engineers Manual EM 1110-2-1601 (1970) "Hydraulic Design of Flood Control Channels", paragraph 14, Riprap Protection.

Figure 4-2. 1 Median Stone Size Selection for Riprap Design

Figure 4-2. 1 Median Stone Size Selection for Riprap Design



Rev. 4/02

Table 4-2.1 Outlet Protection

Design Flow Discharge Velocity (fps)	REQUIRED PROTECTION (Minimum Dimensions)				
	Type	Thickness	Width	Length	Height
0 - 5	Riprap*	1 ft	Diameter + 6 ft	8 ft or 4x diameter, whichever is greater	Crown + 1 ft
>5 - 10	Riprap**	1 ft	Diameter + 6 ft or 3x diameter, whichever is greater	12 ft of 3X diameter, whichever is greater	Crown + 1 ft
>10 - 20	Gabion	1 ft	As required	As required	Crown + 1 ft

Note: The stone for riprap shall be hard, sound, and durable and free of rock fines, soil, or other extraneous material. It shall also be free of segregation, seams, cracks, and other defects which would tend to destroy its resistance to weather.

* Riprap for these velocities shall be reasonably well-graded with rock gradation as follows:

100 % passing an 8 inch square sieve (maximum stone size = 8 inches)
40 - 60% passing a 6 inch square sieve (median stone size = 6 inches)
0 - 10% passing a 2 inch square sieve (minimum stone size = 2 inches)

** Riprap for these velocities shall be reasonably well-graded with rock gradation as follows:

100% passing a 24 inch square sieve (maximum stone size = 24 inches)
40 - 60% passing a 16 inch square sieve (median stone size = 16 inches)
0 - 10% passing a 4 inch square sieve (minimum stone size = 4 inches)

Note: The above riprap sizing assumes 3H:1V side slopes on the outlet channel.

Table 4-2. 2 Channel Protection

Table 4-2.2 Channel Protection

Velocity at Design Flow	Required Protection	Thickness	Min. Height Above Design Water Surface
0 - 5 fps	Grass Lining/Bioengineering	Not applicable	0.5 feet
>5 - 8 fps	Riprap*/Bioengineering	1 ft	1 ft
>8 - 12 fps	Riprap**	2 ft	2 ft
>12 - 20 fps	Slope Mattress	varies	2 ft
>20 fps	Gabion, etc.	varies	varies

Note: The stone for riprap shall be hard, sound, and durable and free of rock fines, soil, or other extraneous material. It shall also be free of segregation, seams, cracks, and other defects which would tend to destroy its resistance to weather.

* Riprap for these velocities shall be reasonably well-graded with rock gradation as follows:

100 % passing an 8 inch square sieve (maximum stone size = 8 inches)
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** Riprap for these velocities shall be reasonably well-graded with rock gradation as follows:

100% passing a 24 inch square sieve (maximum stone size = 24 inches)
40 - 60% passing a 16 inch square sieve (median stone size = 16 inches)
0 - 10% passing a 4 inch square sieve (minimum stone size = 4 inches)

DRAWING INDEX

STORM AND SURFACE WATER		Last Revision Date
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402	Catch Basin Type-A	4-01-04
403	Catch Basin Type-B	4-01-04
404	Catch Basin Type-2 (48" or 54")	4-01-04
405	Storm Drain Manhole Ring and Cover	4-01-04
405a	Frame For Grate Or Solid Cover	4-01-04
406a	Grate For Catch Basin Or Inlet	4-01-04
406b	Solid Cover For Catch Basin Or Inlet	4-01-04
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410	Floatable Material Separator And/or Gas Trap (6" or 8")	4-01-04
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414	Typical Parking Lot Detention	4-01-04
415	Typical Parking Lot Detention	4-01-04
416	Typical Closed Underground Detention System	4-01-04
417	Typical Dry Type Detention Pond	4-01-04
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420	Typical Wet Pond Layout	4-01-04
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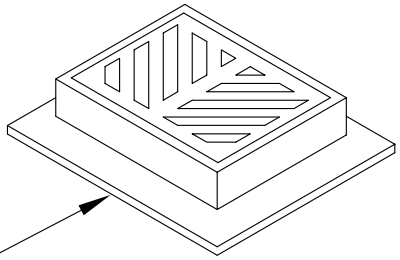
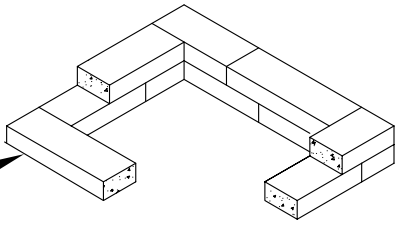
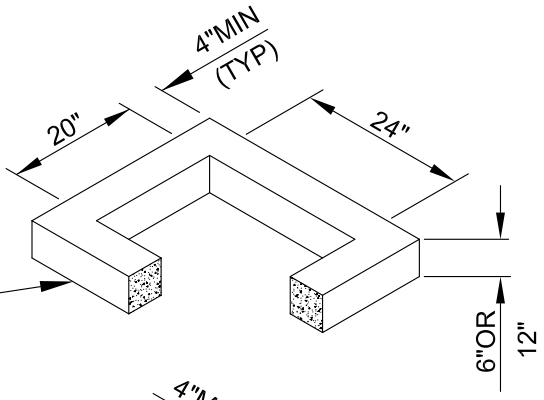
422b	Bypass Structure Type 2	4-01-04
422c	Bypass Structure Type 3	4-01-04
422d	Bypass Structure Type 4	4-01-04
422e	Multiple Restrictor Structure	4-01-04



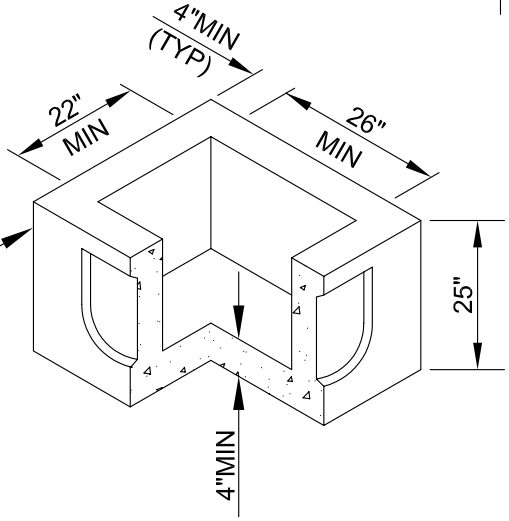
FRAME AND OPEN GRATE PER STD 405
VANED GRATE PER STD 406A



2"x4"x8" SOLID BRICK
USED FOR FINAL ADJUSTMENT
TO GRADE. 6" HIGH MAX



6" OR 12" CONCRETE
RISER SECTION
CLASS 4000 CONC



PRECAST CONC BASE SECTION
W/MAX OF ONE 17"Ø KNOCKOUT
PER SIDE. ENTRANCE ANGLE
TO BE LIMITED BY KNOCKOUTS
CLASS 4000 CONC
PIPE OUTER DIA PLUS
INLET WALL THICKNESS
SHALL NOT EXCEED 17".

INLET TO CONFORM TO WSDOT STANDARD PLAN B-26

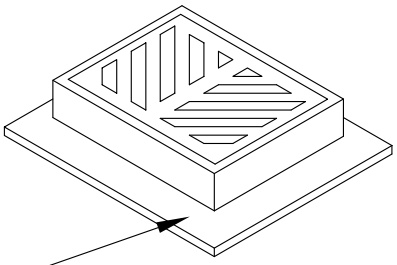
SNOHOMISH
WASHINGTON

PRECAST CONCRETE INLET

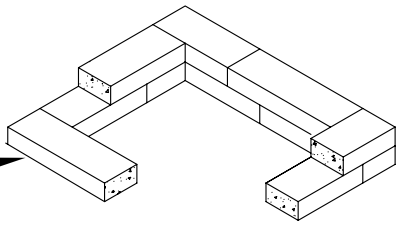
Approved By:
City Engineer
Date: May 30, 2004

401
Number

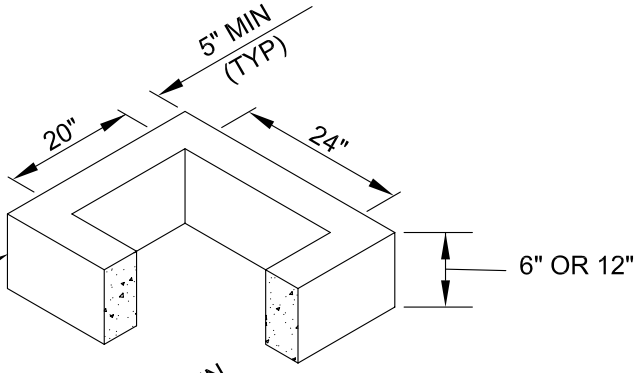
City of Snohomish Public Works Department



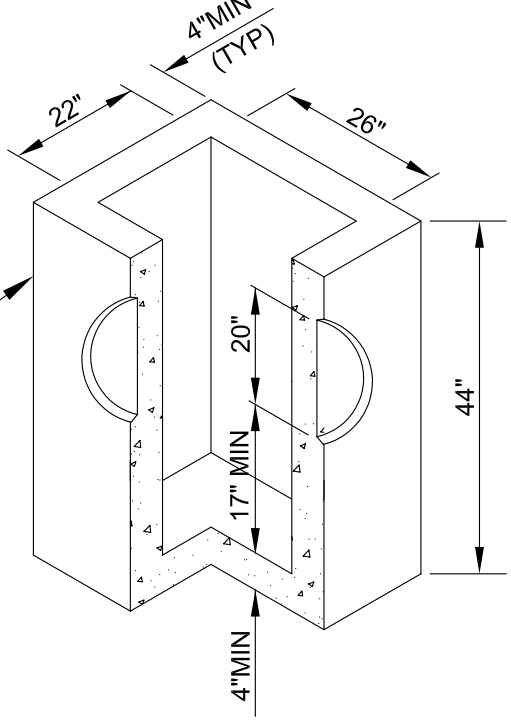
FRAME AND OPEN GRATE PER STD 405
VANED GRATE PER STD 406A
SOLID COVER PER STD 406



2"x4"x8" SOLID BRICK
USED FOR FINAL ADJUSTMENT
TO GRADE. 6" HIGH MAX



6" OR 12" CONCRETE
RISER SECTION
CLASS 4000 CONC



PRECAST CONC BASE SECTION
W/MAX OF ONE 20"Ø KNOCKOUT
PER SIDE, ENTRANCE ANGLE
TO BE LIMITED BY KNOCKOUTS
CLASS 4000 CONC
PIPE OUTER DIA PLUS CB
WALL THICKNESS SHALL
NOT EXCEED 20".

CATCH BASIN TO CONFORM TO WSDOT STANDARD PLAN B-1

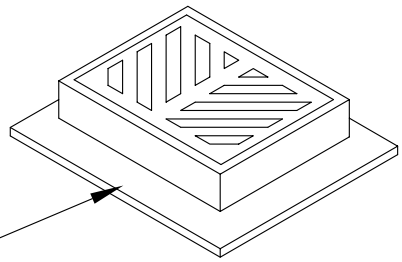
SNOHOMISH
WASHINGTON

CATCH BASIN TYPE A

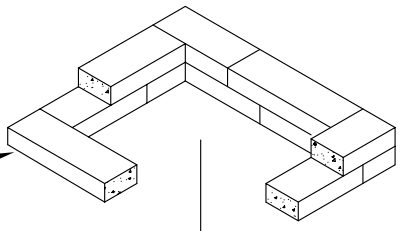
Approved By:
City Engineer
Date: May 30, 2004

402
Number

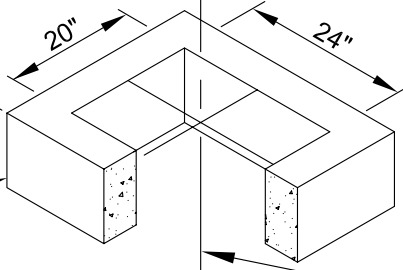
City of Snohomish Public Works Department



FRAME AND OPEN GRATE PER STD 405
VANED GRATE PER STD 406A
SOLID COVER PER STD 406

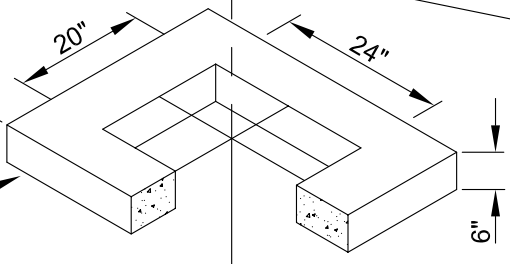


2"x4"x8" SOLID BRICK
USED FOR FINAL ADJUSTMENT
TO GRADE. 6" HIGH MAX

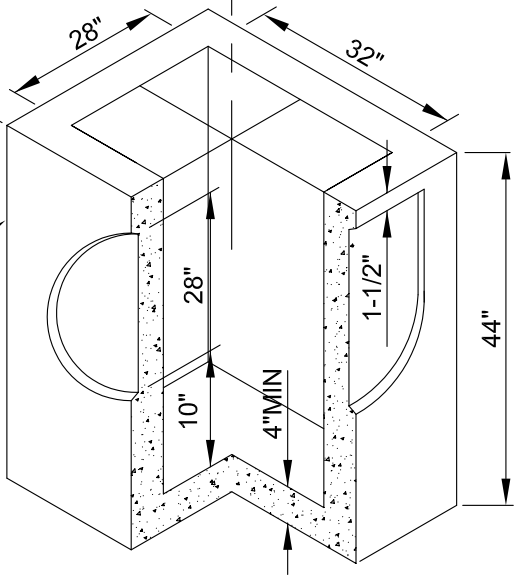


6" OR 12" CONCRETE
RISER SECTION
CLASS 4000 CONC

CB CL



6" CONCRETE
REDUCING TOP SLAB
CLASS 4000 CONC



PRECAST CONC BASE SECTION
W/MAX OF ONE 28"Ø KNOCKOUT
PER SID. ENTRANCE ANGLE
TO BE LIMITED BY KNOCKOUTS
CLASS 4000 CONC
PIPE OUTER DIA PLUS CB
WALL THICKNESS SHALL
NOT EXCEED 28"

CATCH BASIN TO CONFORM TO WSDOT STANDARD PLAN B-1A

SNOHOMISH
WASHINGTON

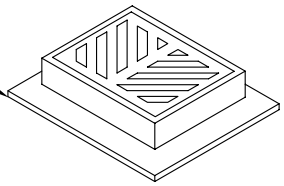
CATCH BASIN TYPE B

Approved By:
City Engineer
Date: May 30, 2004

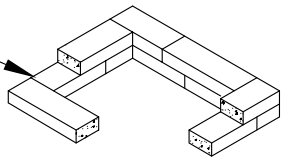
403
Number

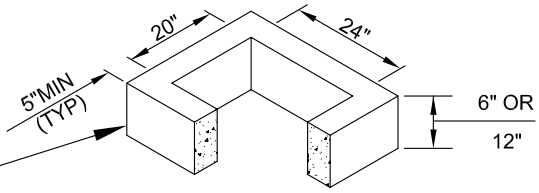
City of Snohomish Public Works Department

FRAME AND OPEN GRATE PER STD 405
VANED GRATE PER STD 406A
SOLID COVER PER STD 406



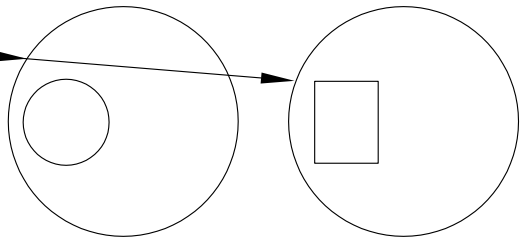
2"x4"x8" SOLID BRICK
USED FOR FINAL ADJUSTMENT
TO GRADE. 6" HIGH MAX





6" OR 12" CONCRETE
RISER SECTION
CLASS 4000 CONC

TOP SLAP WITH
24" ROUND ACCESS OR
20"x24" RECTANGULAR ACCESS



NOTES:

1 ALL TYPE 2 CB'S EQUIPPED WITH OPEN GRATES SHALL BE LOCKING TYPE.

2 ALL TYPE 2 CB'S NOT IN PAVED AREAS SHALL BE EQUIPPED WITH LOCKING LIDS,

3 CB'S SHALL BE SET SO THAT STEPS ARE DIRECTLY UNDER OPENING.

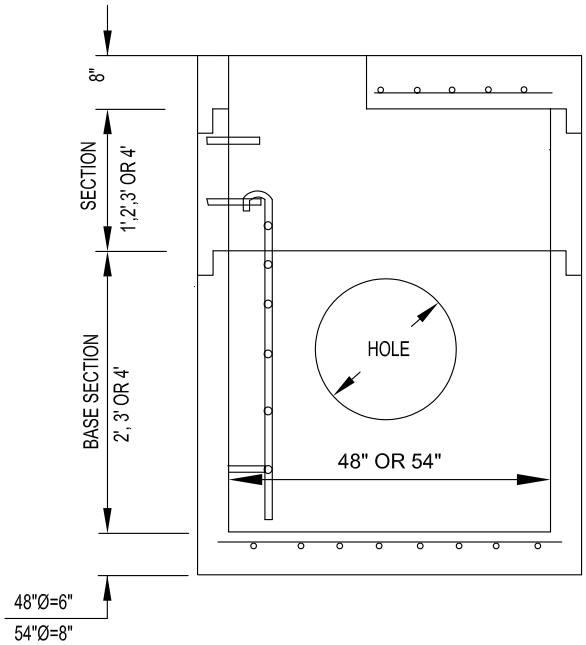
4 ALL TYPE 2 CB'S SHALL BE EQUIPPED WITH STEPS AND LADDER PER STD 606.

5 CONCRETE SHALL BE CLASS 4000.

6 MIN DISTANCE FROM INVERT TO CB BOTTOM SHALL BE 2.0'.

7 CB SECTIONS AND LID WILL BE HS20 TRAFFIC LOAD CERTIFIED BY MFG.

8 HOLE SIZE = PIPE OD + 5". MAX
MAX HOLE SIZE = 36" (48"° CB)
MAX HOLE SIZE = 42" (54"° CB)



CATCH BASIN TO CONFORM TO WSDOT STANDARD PLAN B-1E

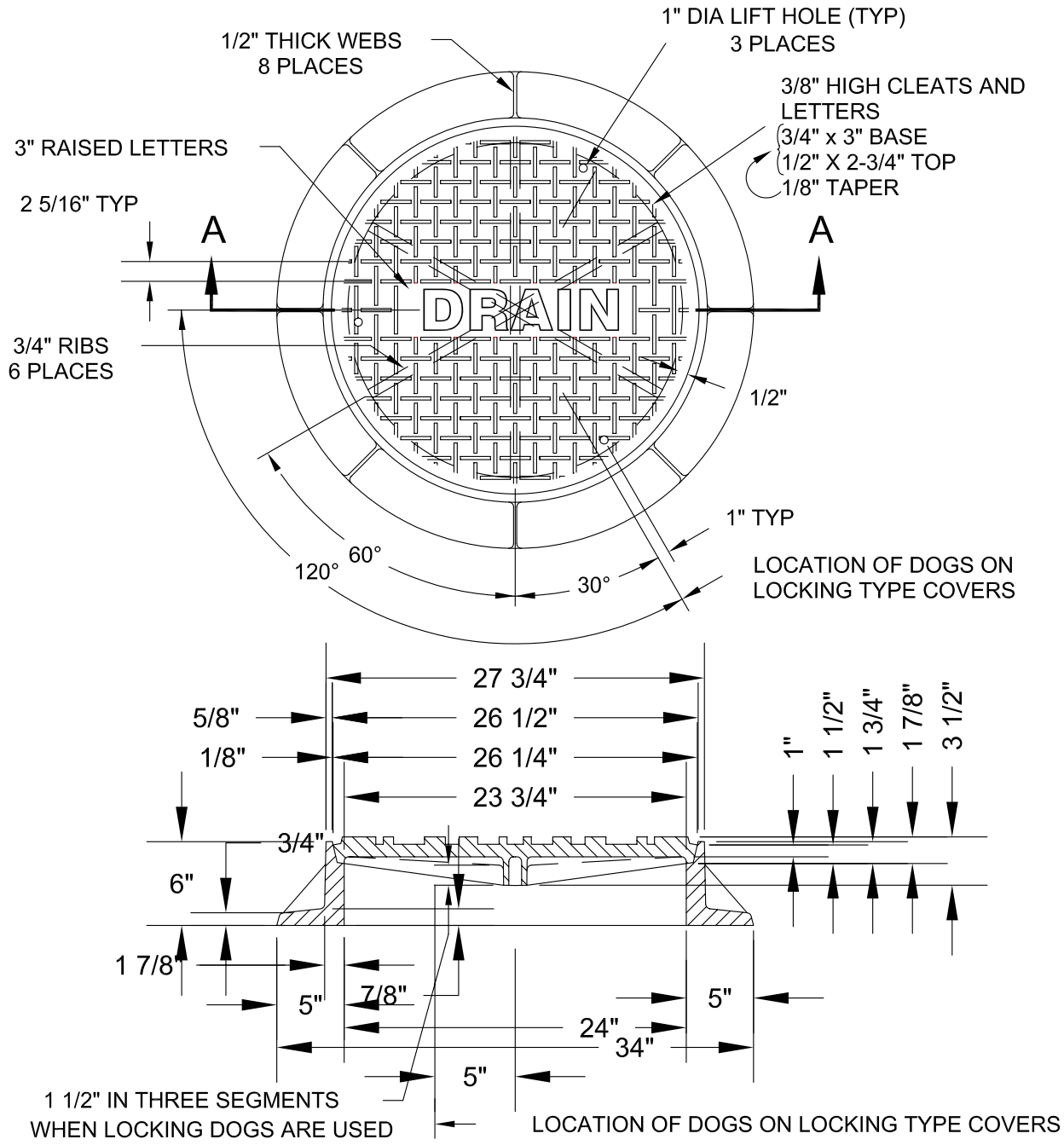
SNOHOMISH
WASHINGTON

CATCH BASIN TYPE 2 (48" OR 54")

Approved By:
City Engineer
Date: May 30, 2004

404
Number

City of Snohomish Public Works Department



SECTION A-A

NOTES:

1. MANHOLE RINGS SHALL BE GRAY IRON CONFORMING TO THE REQUIREMENTS OF AASHTO M 105, GRADE 30B.
2. MANHOLE COVER TO BE DUCTILE IRON CONFORMING TO ASTM A536, GR 80-55-06
3. LOCKING COVER TO BE USED AT OFF-STREET LOCATIONS AND OTHER LOCATIONS AS DIRECTED. THE LID SHALL BE LOCKED DOWN WITH 3-5/8" S.S SOCKET HEAD CAP SCREWS.
4. RING AND COVER SHALL BE TESTED FOR ACCURACY OF FIT AND SHALL BE MARKED IN SETS FOR DELIVERY.
5. ALL CASTINGS TO HAVE A BITUMINOUS COATING.

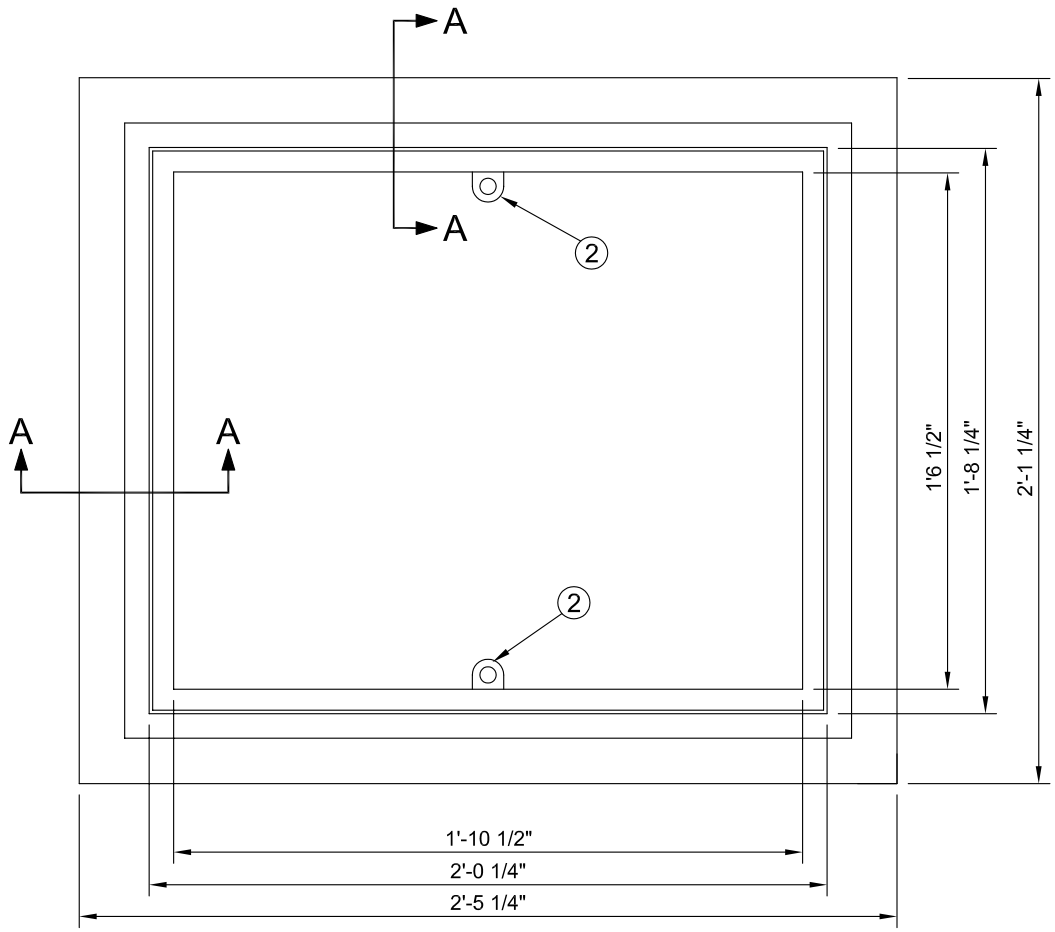
SNOHOMISH
WASHINGTON

STORM DRAIN MANHOLE RING AND COVER

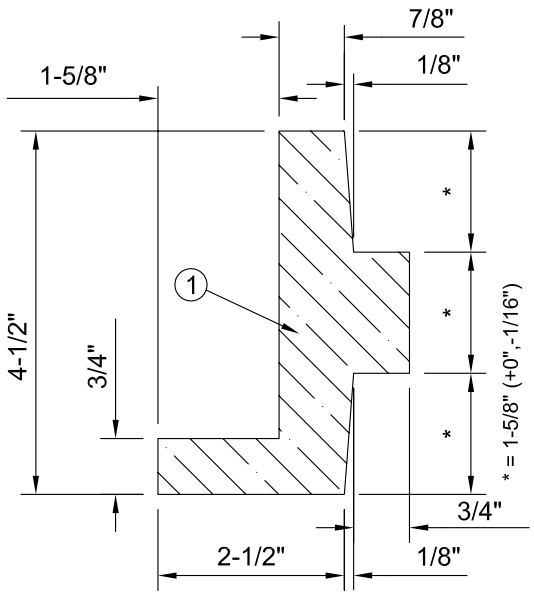
Approved By:
City Engineer
Date: May 30, 2004

405
Number

City of Snohomish Public Works Department



TOP VIEW



SECTION A-A

NOTES:

1. MATERIAL USED SHALL BE CAST IRON PER ASTM-A48, CLASS 30. FRAME SHALL HAVE A BITUMINOUS COATING APPLIED.
2. WHEN A LOCKING GRATE OR COVER IS REQUIRED THE FRAME SHALL BE FABRICATED SO AS TO ACCEPT TWO (2) 5/8" STAINLESS STEEL SOCKET HEAD CAP SCREWS OF GRATE OR COVER.
3. FOR APPROVED GRATES AND SOLID MANHOLE COVER SEE STD DWG'S 405A, 406A, 406B & 406C

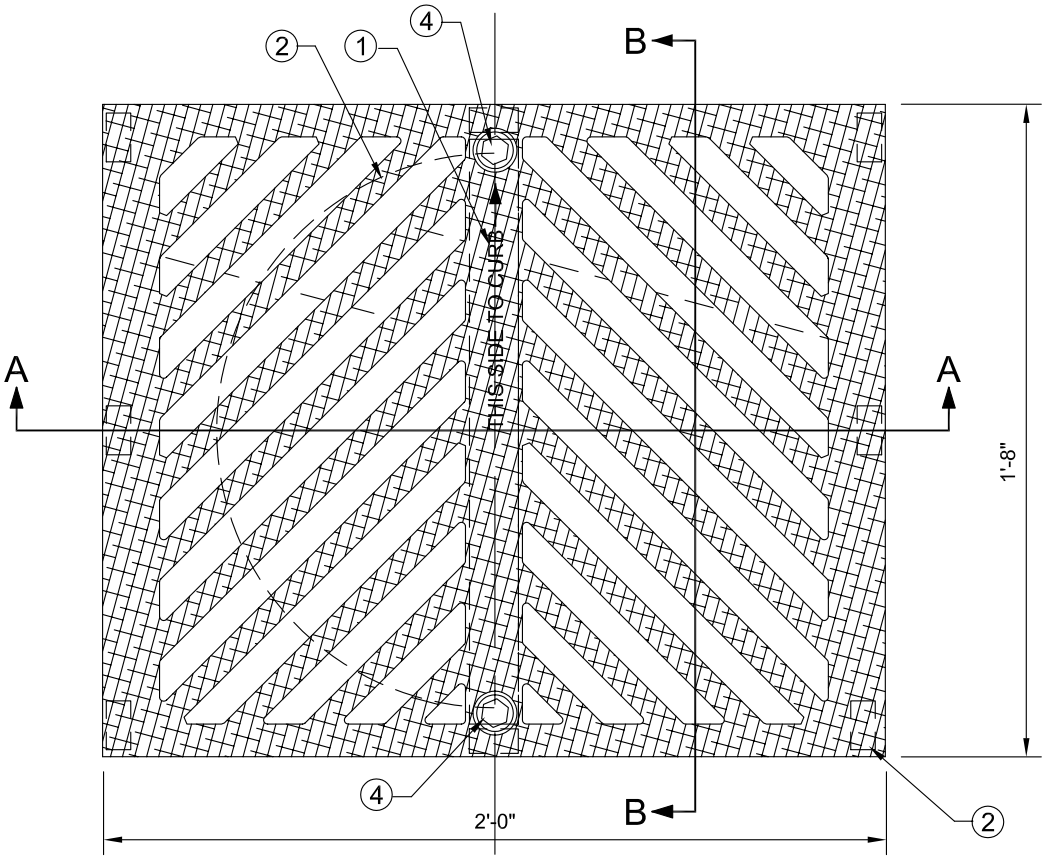
SNOHOMISH WASHINGTON

FRAME FOR GRATE OR SOLID COVER

Approved By:
City Engineer
Date: May 30, 2004

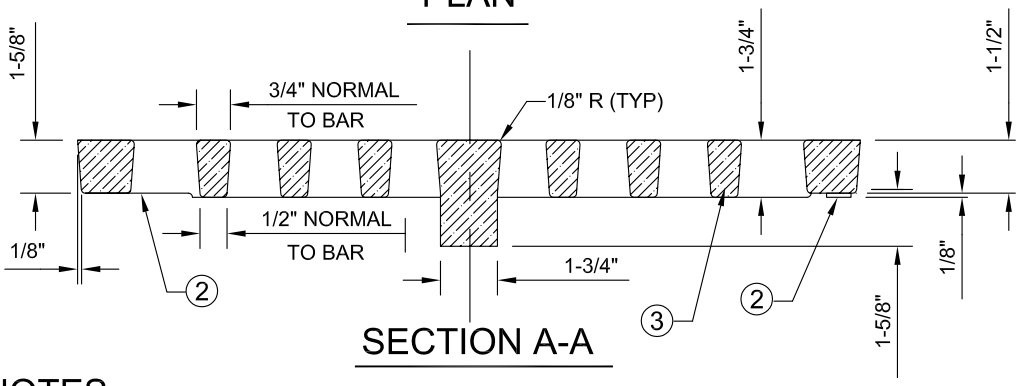
405a
Number

City of Snohomish Public Works Department



PLAN

SECTION B-B



SECTION A-A

NOTES:

1. FOUNDRY NAME, THIS SIDE TO CURB W/ARROW AND (DI) FOR DUCTILE IRON SHALL BE EMBOSSED ON TOP OF GRATE WITH 1/16" RECESSED LETTERS.
2. SEATING OF GRATE SHALL BE ACCOMPLISHED BY ONE OF THE FOLLOWING:
   A.8 INTEGRALLY CAST PADS (1-1/2"x3/4"x1/8").
   B. MACHINE BOTTOM SURFACE OUTSIDE A 17" DIA
3. MATERIAL USED SHALL BE DUCTILE IRON PER ASTM-A536, GRADE 80-55-06. ALL CASTINGS SHALL HAVE A BITUMINOUS COATING.
4. WHEN LOCKING GRATE REQUIRED HOLES WILL BE PROVIDED IN CASTING TO ALLOW FOR TWO 5/8" DIA STAINLESS STEEL SOCKET HEAD CAP SCREWS SO THAT NO PART OF HEAD PROTRUDES ABOVE TOP OF CASTING.
5. GRATE TO BE USED WITH FRAME SHOWN IN STANDARD DWG 405.

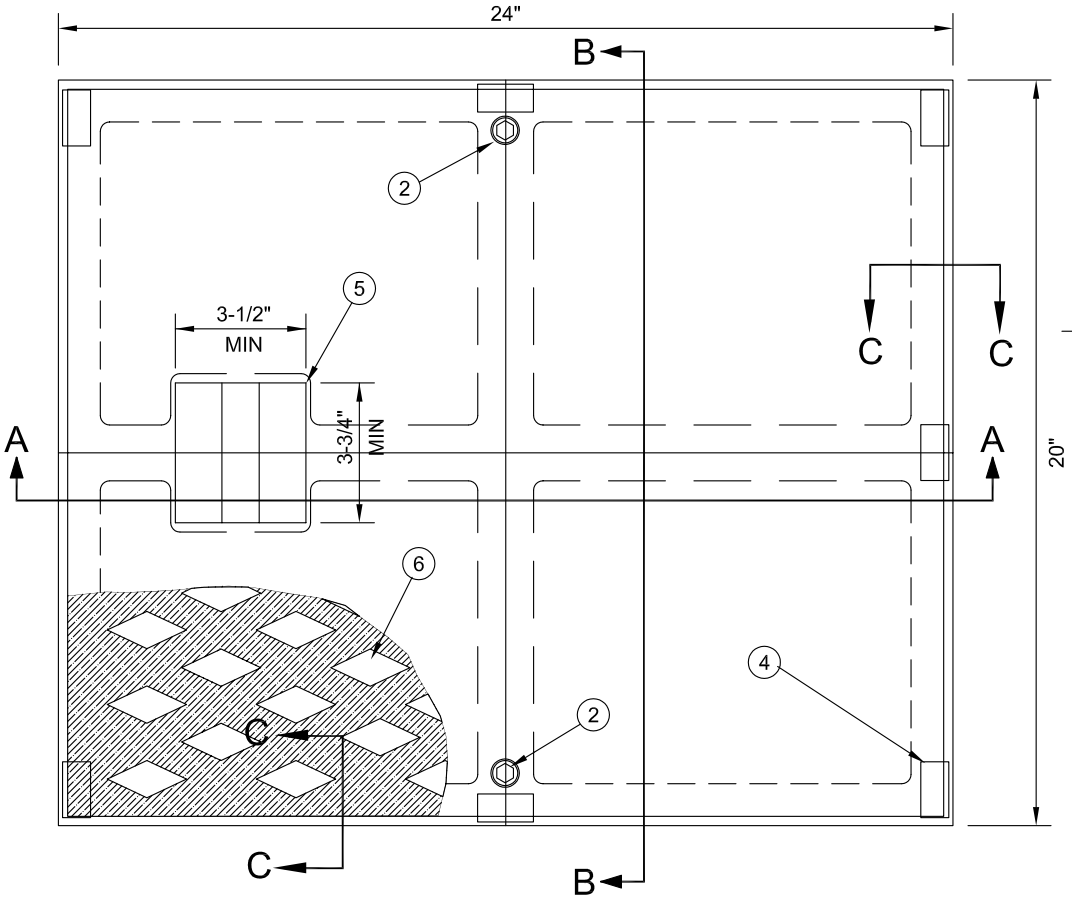
SNOHOMISH
WASHINGTON

GRATE FOR CATCH BASIN OR INLET

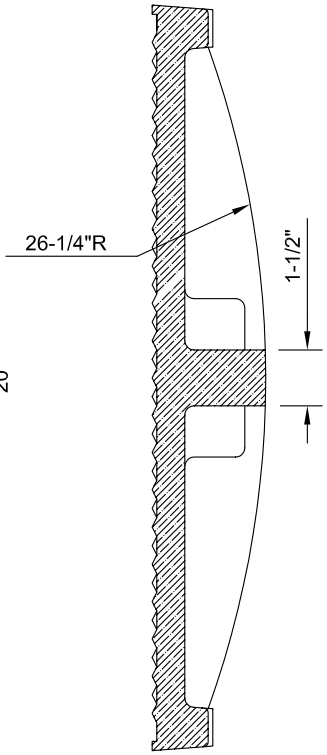
Approved By:
City Engineer
Date: May 30, 2004

406a
Number

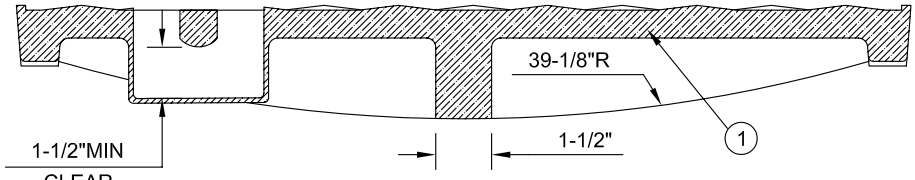
City of Snohomish Public Works Department



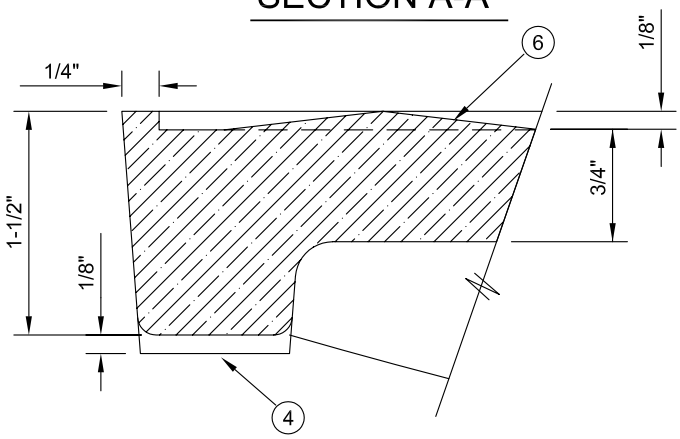
TOP VIEW



SECTION B-B



SECTION A-A



SECTION C-C

NOTE:

1. MATERIAL USED SHALL BE DUCTILE IRON PER ASTM-A536, GRADE 80-55-06, WITH BITUMINOUS COATING.
2. WHEN LOCKING GRATE REQUIRED HOLES WILL BE PROVIDED IN CASTING TO ALLOW FOR TWO 5/8" DIA STAINLESS STEEL, SOCKET HEAD CAP SCREWS. NO PART OF SCREW WILL PROTRUDE ABOVE GRATE.
3. GRATE TO BE USED WITH FRAME SHOWN IN STD DWG 405.
4. GRATE SEATING: 8 INTEGRALLY CAST PADS.
5. CAST POCKET LIFT HANDLE.
6. NON-SKID DIAMOND PATTERN APPROX 2-1/2"x1"x1/8"HIGH

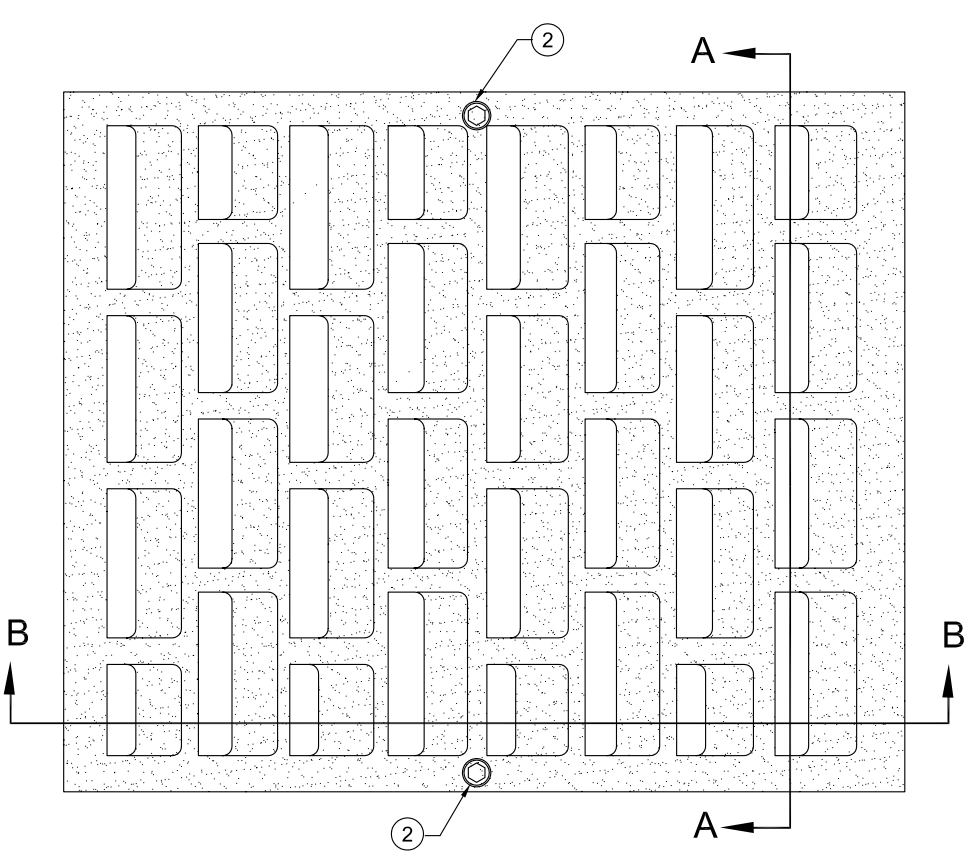
SNOHOMISH WASHINGTON

SOLID LID COVER FOR CATCH BASIN OR INLET

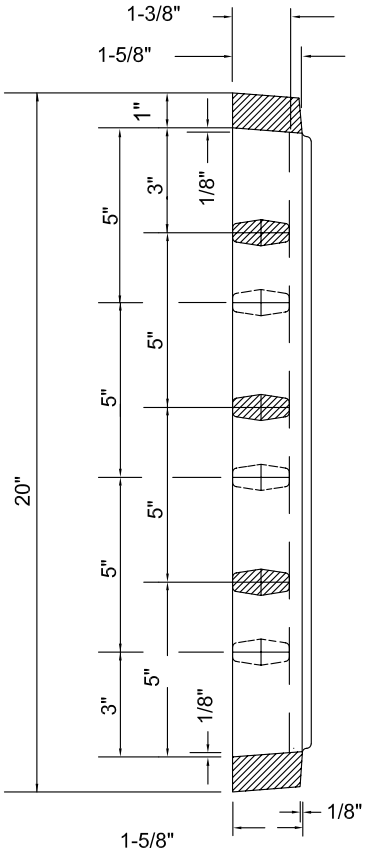
Approved By:
City Engineer
Date: May 30, 2004

406b
Number

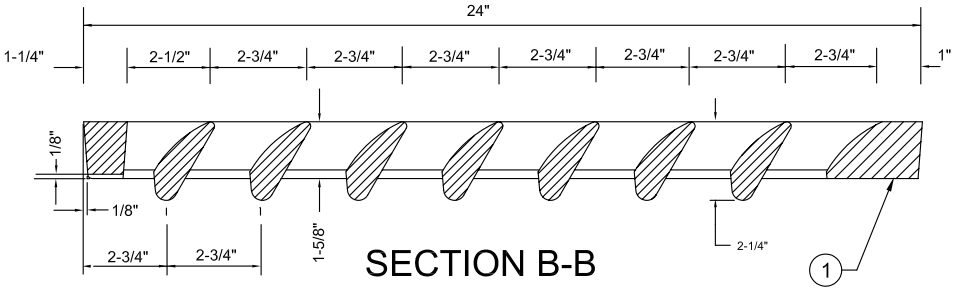
City of Snohomish Public Works Department



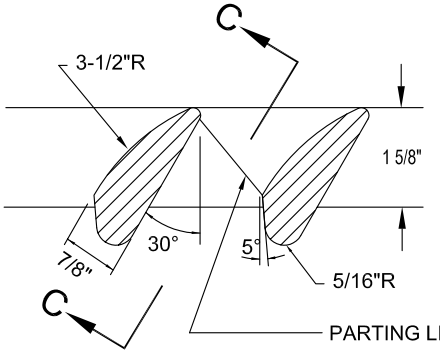
TOP VIEW



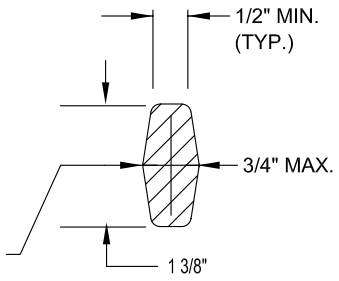
SECTION A-A



SECTION B-B



VANE DETAIL



SECTION C-C

NOTE:

1. MATERIAL USED SHALL BE DUCTILE IRON PER ASTM-A536, GRADE 80-55-06, WITH BITUMINOUS COATING.
2. WHEN LOCKING GRATE REQUIRED HOLES WILL BE PROVIDED IN CASTING TO ALLOW FOR TWO 5/8" DIA STAINLESS STEEL, SOCKET HEAD CAP SCREWS. NO PART OF SCREW WILL PROTRUDE ABOVE GRATE.
3. GRATE TO BE USED WITH FRAME SHOWN IN STD DWG 405.

W.S.D.O.T. STD PLAN B-2B

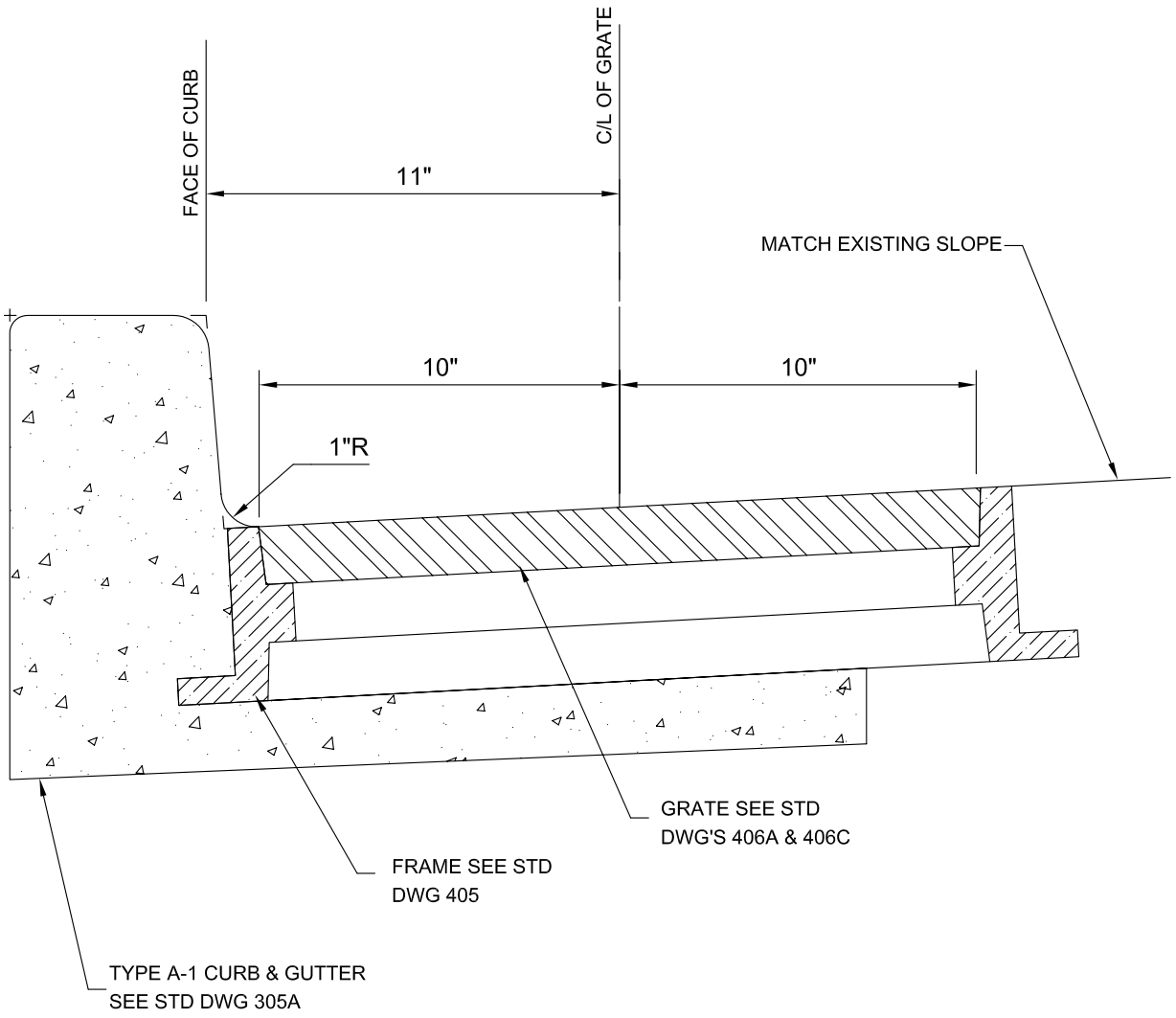
SNOHOMISH WASHINGTON

VANED GRATE FOR CATCH BASIN OR INLET

Approved By:
City Engineer
Date: May 30, 2004

406c
Number

City of Snohomish Public Works Department



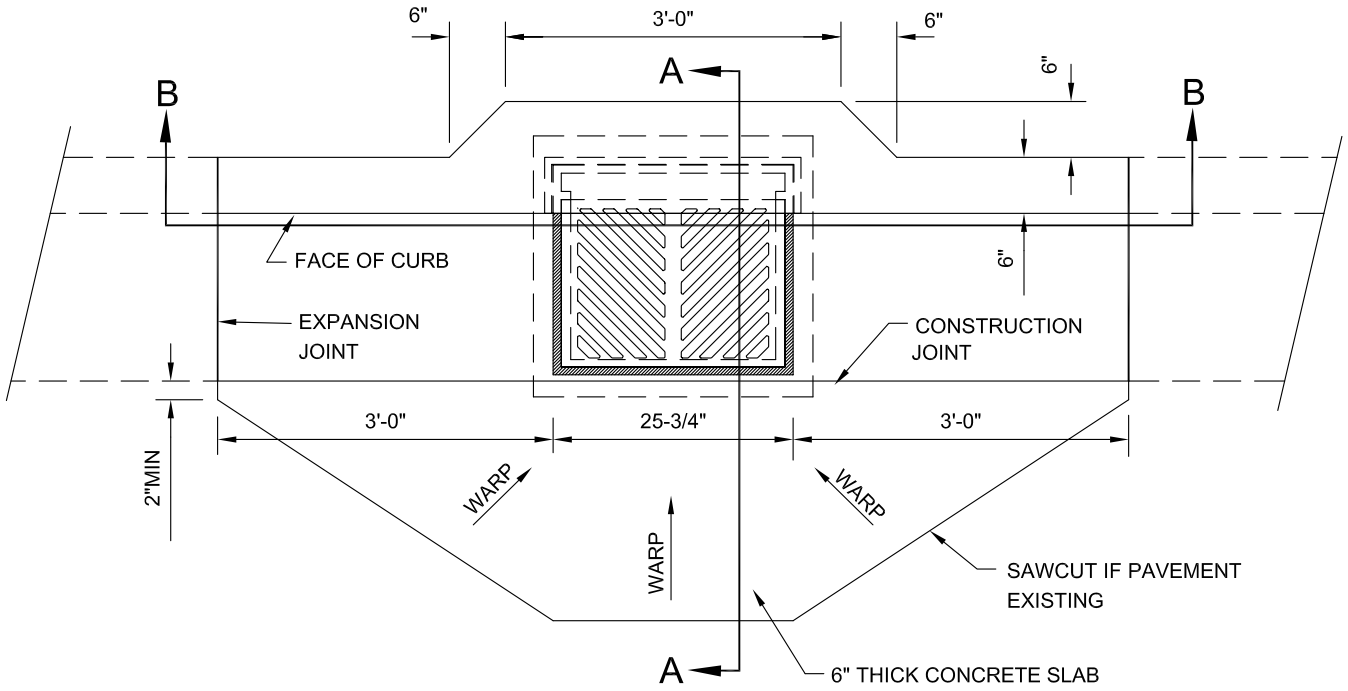
SNOHOMISH
WASHINGTON

TYPICAL FRAME AND GRATE INSTALLATION

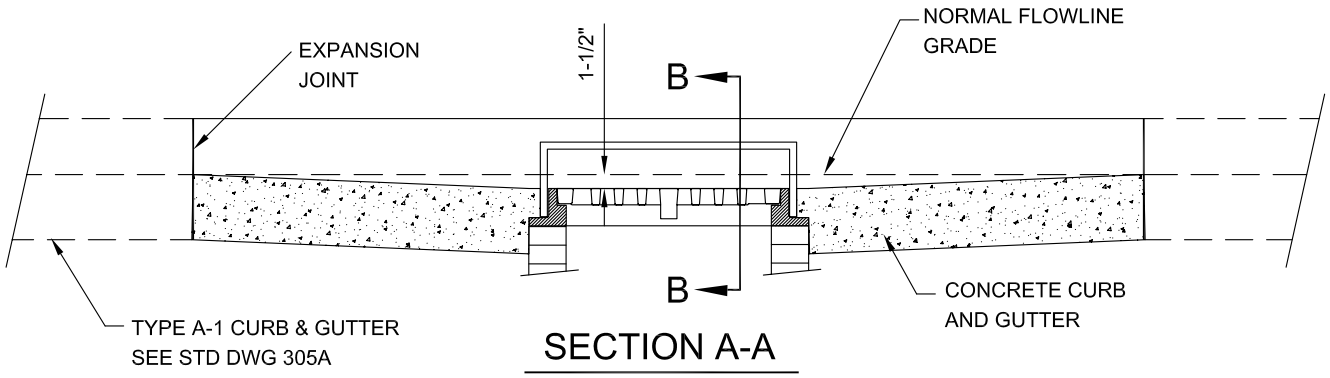
Approved By:
City Engineer
Date: May 30, 2004

407
Number

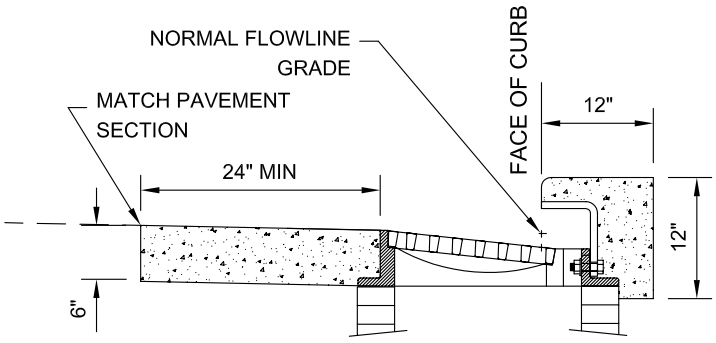
City of Snohomish Public Works Department



TOP VIEW



SECTION A-A



SECTION B-B

NOTES:

1. INSTALLATION OF OPEN CURB FACE FRAME AND GRATE MUST BE APPROVED BY CITY ENGINEER ON A CASE BY CASE BASIS.

   PROBLEMS OF FLOATABLE MATERIAL ENTERING STORM DRAINAGE SYSTEM BY USE OF THIS INSTALLATION MUST BE ADDRESSED.

2. SEE STD DWG 409 FOR FRAME AND GRATE DETAILS.

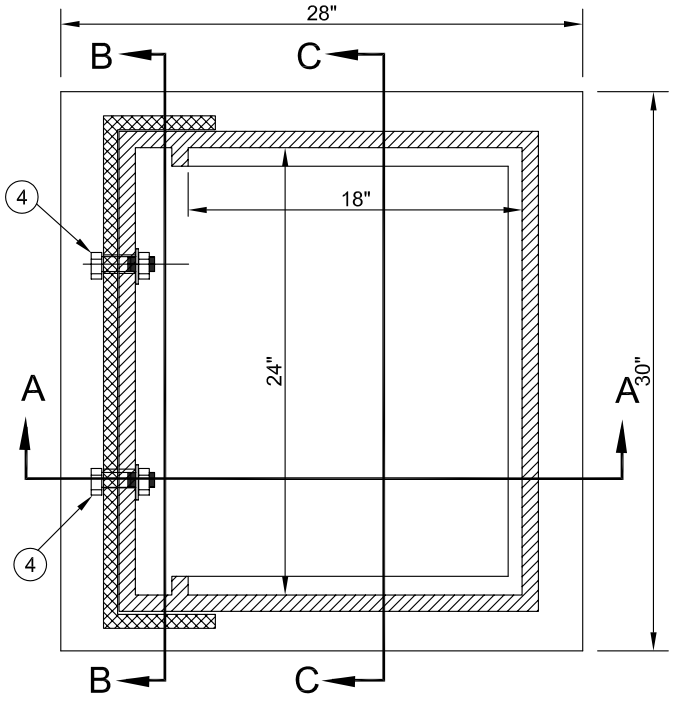
SNOHOMISH
WASHINGTON

OPEN CURB FACE FRAME AND GRATE INSTALLATION

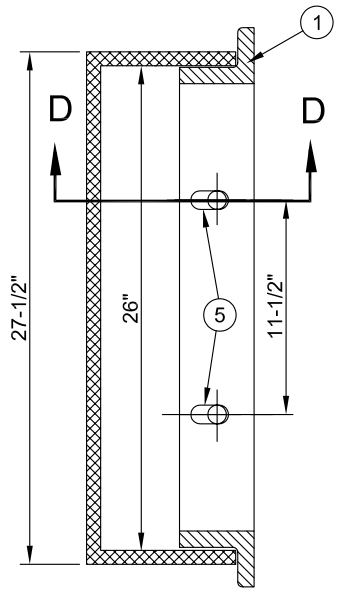
Approved By:
City Engineer
Date: May 30, 2004

408
Number

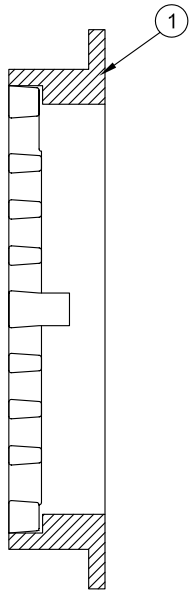
City of Snohomish Public Works Department



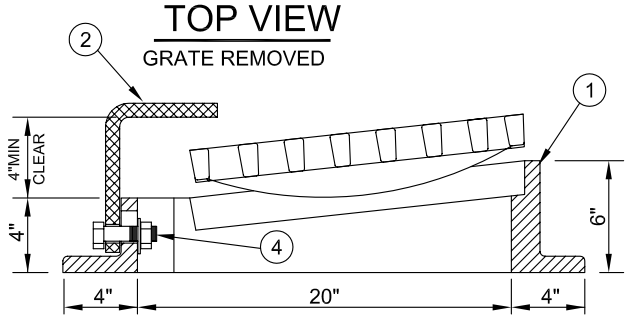
TOP VIEW



SECTION B-B

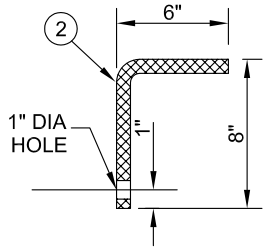


SECTION C-C

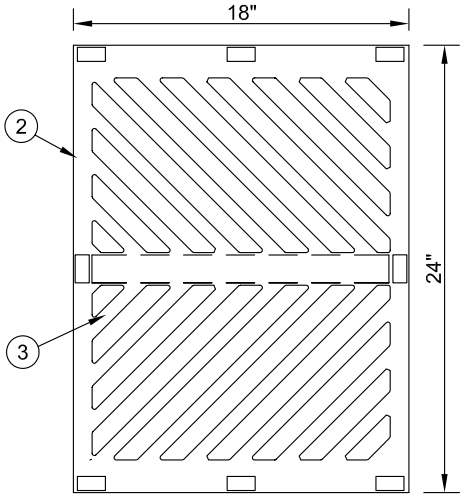


SECTION A-A

GRATE RAISED



SECTION D-D



GRATE TOP VIEW

NOTES:

1. FRAME MATERIAL SHALL BE CAST IRON ASTM A48, CLASS 30, WITH A BITUMINOUS COATING.
2. GRATE AND COVER MATERIAL SHALL BE DUCTILE IRON ASTM A536, GRADE 80-55-06 WITH BITUMINOUS COATING.
3. GRATE TO BE FABRICATED PER STANDARD DWG 406A WITH OVER ALL DIMENSIONS OF 24"x18".
4. PROVIDE 3/4" BOLT, NUT AND WASHERS
5. ADJUSTMENT SLOT IN FRAME CASTING APPROXIMATELY 1"x2". VERTICAL PLACEMENT TO PROVIDE MIN CLEARANCE BETWEEN GRATE AND COVER PLATE.
6. SEE STANDARD DWG 408 FOR INSTALLATION DETAILS.

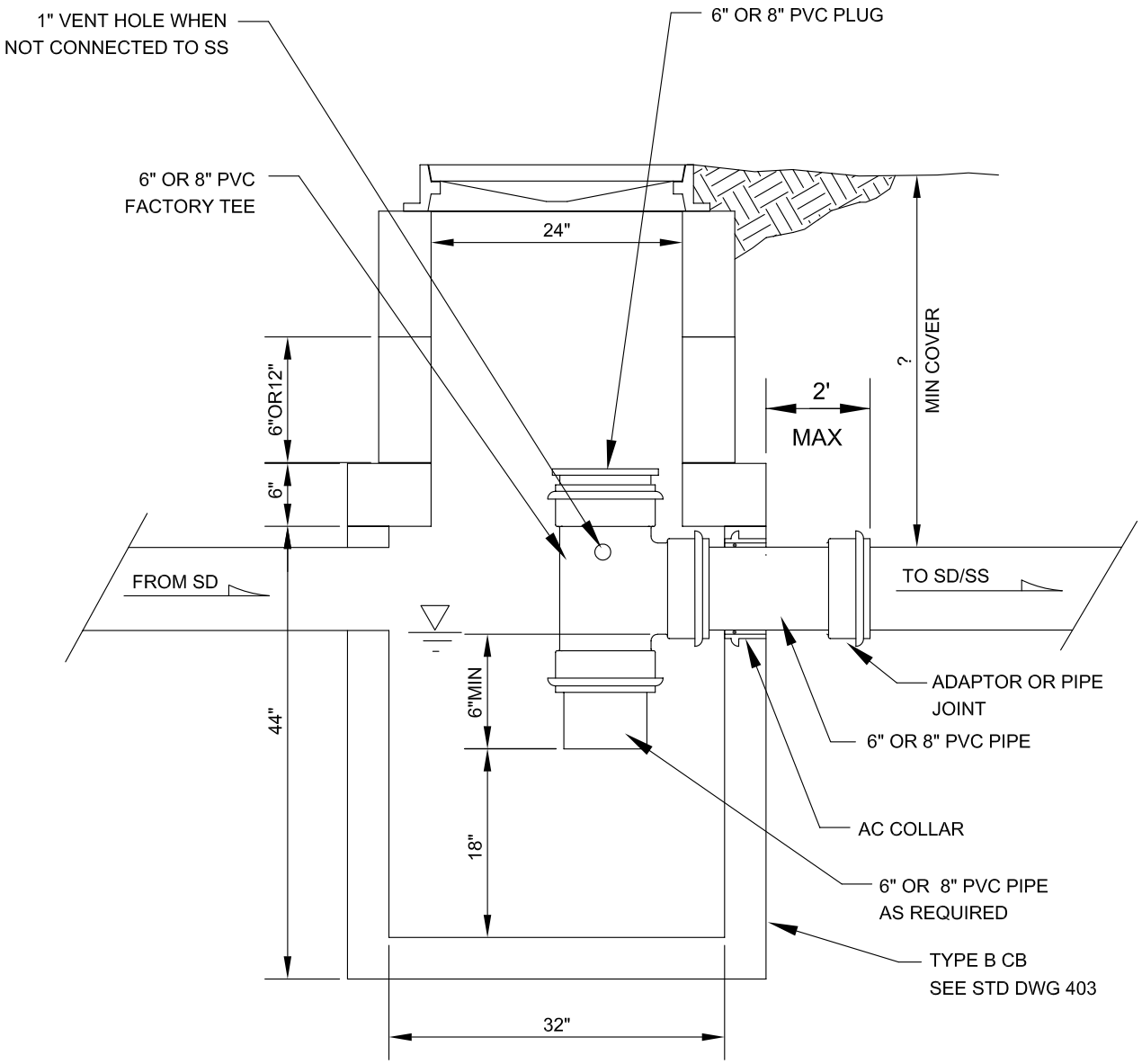
SNOHOMISH WASHINGTON

OPEN CURB FACE FRAME AND GRATE INSTALLATION

Approved By:
City Engineer
Date: May 30, 2004

409
Number

City of Snohomish Public Works Department



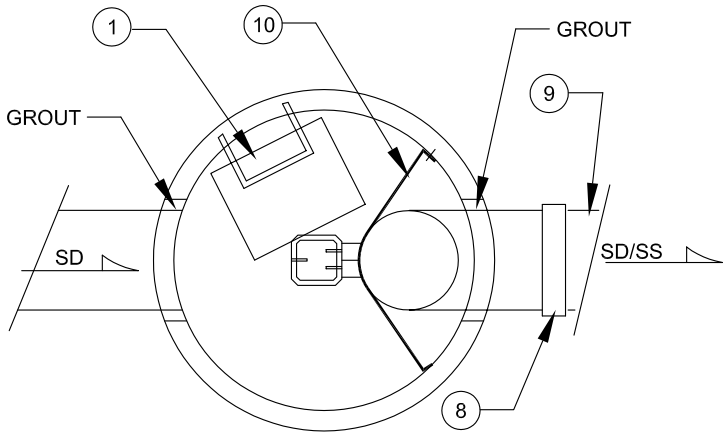
SNOHOMISH
WASHINGTON

FLOATABLE MATERIAL SEPARATOR AND/OR GAS TRAP (6" OR 8")

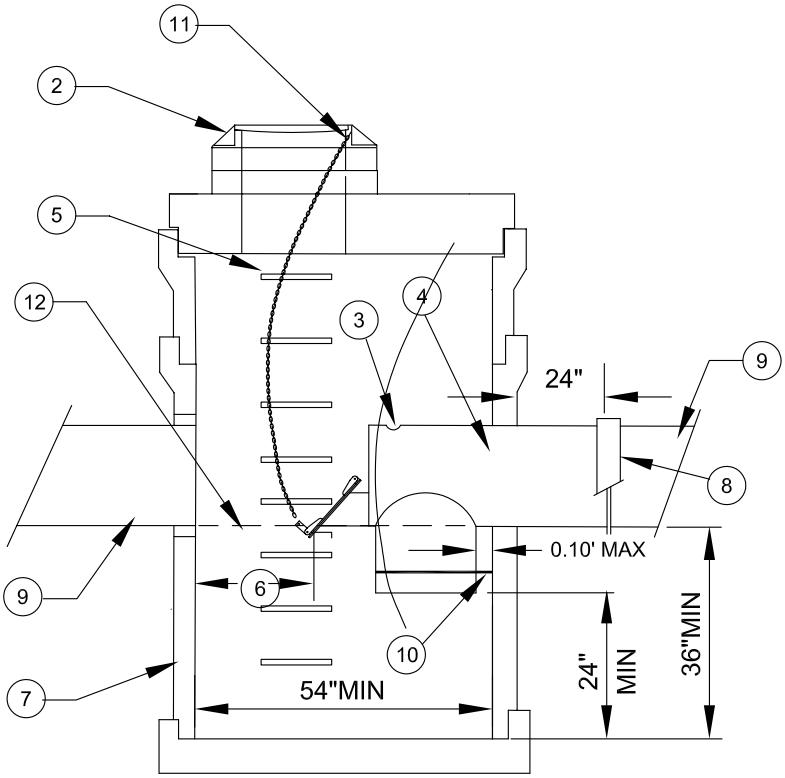
Approved By:
City Engineer
Date: May 30, 2004

410
Number

City of Snohomish Public Works Department



PLAN



ELEVATION

NOTES:

1. INSTALL CB TOP, FRAME, GRATE AND SECTIONS SO THAT LIFT GATE IS VISIBLE THROUGH OPENING AND STEPS CLEAR INLET AND RESTRICTOR UNIT.
2. INSTALL LOCKING FRAME & GRATE OR LID PER STD DWG 405 . FRAME AND LID PER STD DWG 405 IS REQUIRED IF INSTALLATION IN NOT IN PAVED AREA OR IS NOT TO FUNCTION AS A CB.
3. 1" VENT HOLE WHEN NOT CONNECTED TO TO COMBINED SANITARY SEWER SYSTEM.
4. SEPARATOR ASSEMBLY SEE STD DWG 412B & 412C AS APPLICABLE.
5. STEPS PER STD DWG 606.
6. MIN CLEARANCE: 36"FOR OUTLETS OF 24" AND LARGER 18" FOR OUTLETS OF 18" AND SMALLER
7. 54" TYPE 2 CB OR LARGER.
8. BAND STRAP WITH GASKET
9. SEE PLAN AND SPECIFICATIONS FOR SIZE AND TYPE OF PIPE ENTERING AND EXITING CB.
10. SECURE SEPARATOR TO CB WITH 8 GA ALUMINUM STRAP. BOLT TO CB WALL WITH STAINLESS STEEL ANCHOR BOLTS AND TACK WELD TO SEPARATOR UNIT.
11. BOLT OR WELD CHAIN TO FRAME.
12. INVERT ELEVATION: SEE PLANS AND SPECIFICATIONS.

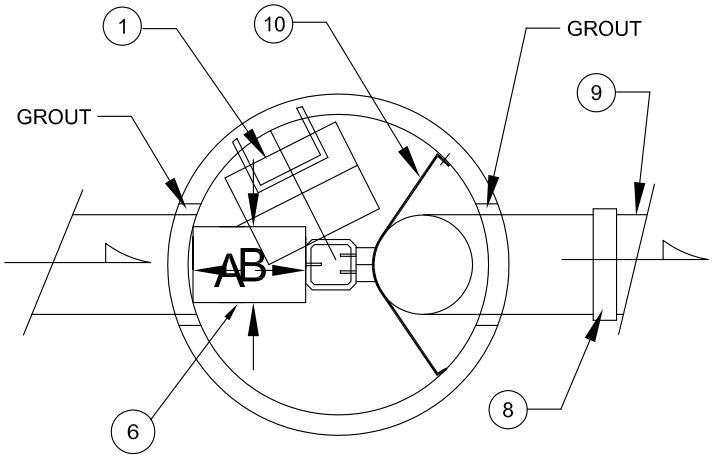
SNOHOMISH WASHINGTON

FLOATABLE MATERIAL SEPARATOR AND/OR GAS TRAP (12" AND LARGER)

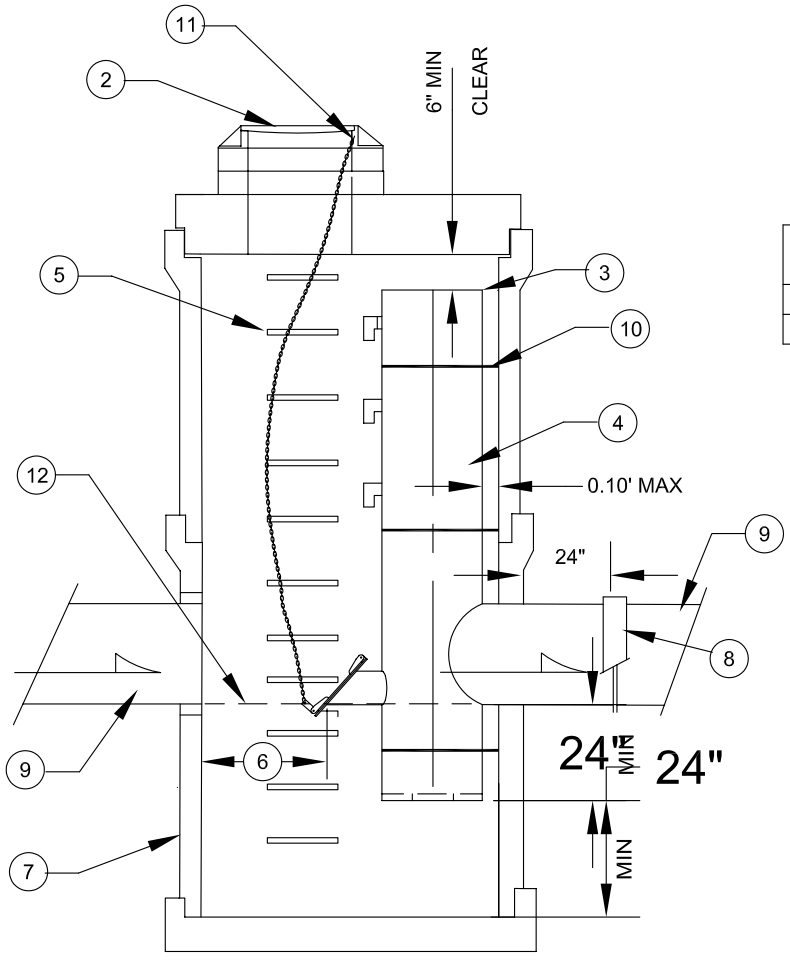
Approved By:
City Engineer
Date: May 30, 2004

411
Number

City of Snohomish Public Works Department



PLAN



ELEVATION

NOTES:

1. INSTALL CB TOP, FRAME, GRATE AND SECTIONS SO THAT LIFT GATE IS VISIBLE THROUGH OPENING AND STEPS CLEAR INLET AND RESTRICTOR UNIT.
2. INSTALL LOCKING FRAME & GRATE OR LID PER STD DWG 405. FRAME AND LID PER STD DWG 405 IS REQUIRED IF INSTALLATION IS NOT IN PAVED AREA OR IS NOT TO FUNCTION AS A CB.
3. OVERFLOW ELEVATION.
4. RESTRICTOR ASSEMBLY SEE STD DWG 412B & 412C.
5. STEPS PER STD DWG 606.
6. MIN CLEARANCE:
   36"FOR OUTLETS OF 24" AND LARGER
   18" FOR OUTLETS OF 18" AND SMALLER
6. MAINTENANCE CLEAR ZONE:
   THIS AREA IS REQUIRED FOR CLEANING AND WILL BE FREE OF OBSTRUCTIONS FROM GATE TO TOP OF STRUCTURE.

DIM	OUTLET SIZE	
	18" AND SMALLER	24" AND LARGER
A	18"	36"
B	18"	24"

7. 54" TYPE 2 CB OR LARGER.
8. BAND STRAP WITH GASKET
9. SEE PLAN AND SPECIFICATIONS FOR SIZE AND TYPE OF PIPE ENTERING AND EXITING CB.
10. SECURE RESTRICTOR TO CB WITH 8 GA ALUMINUM STRAPS BOLT TO CB WALL WITH STAINLESS STEEL ANCHOR BOLTS AND TACK WELD TO RESTRICTOR UNIT. ONE STRAP ABOVE AND BELOW OUTLET REQUIRED, INTERMEDIATE STRAPS REQUIRED FOR RESTRICTOR RISERS GREATER THAN 12' ABOVE OUTLET.
11. BOLT OR WELD CHAIN TO FRAME.
12. INVERT ELEVATION: SEE PLANS AND SPECIFICATIONS.

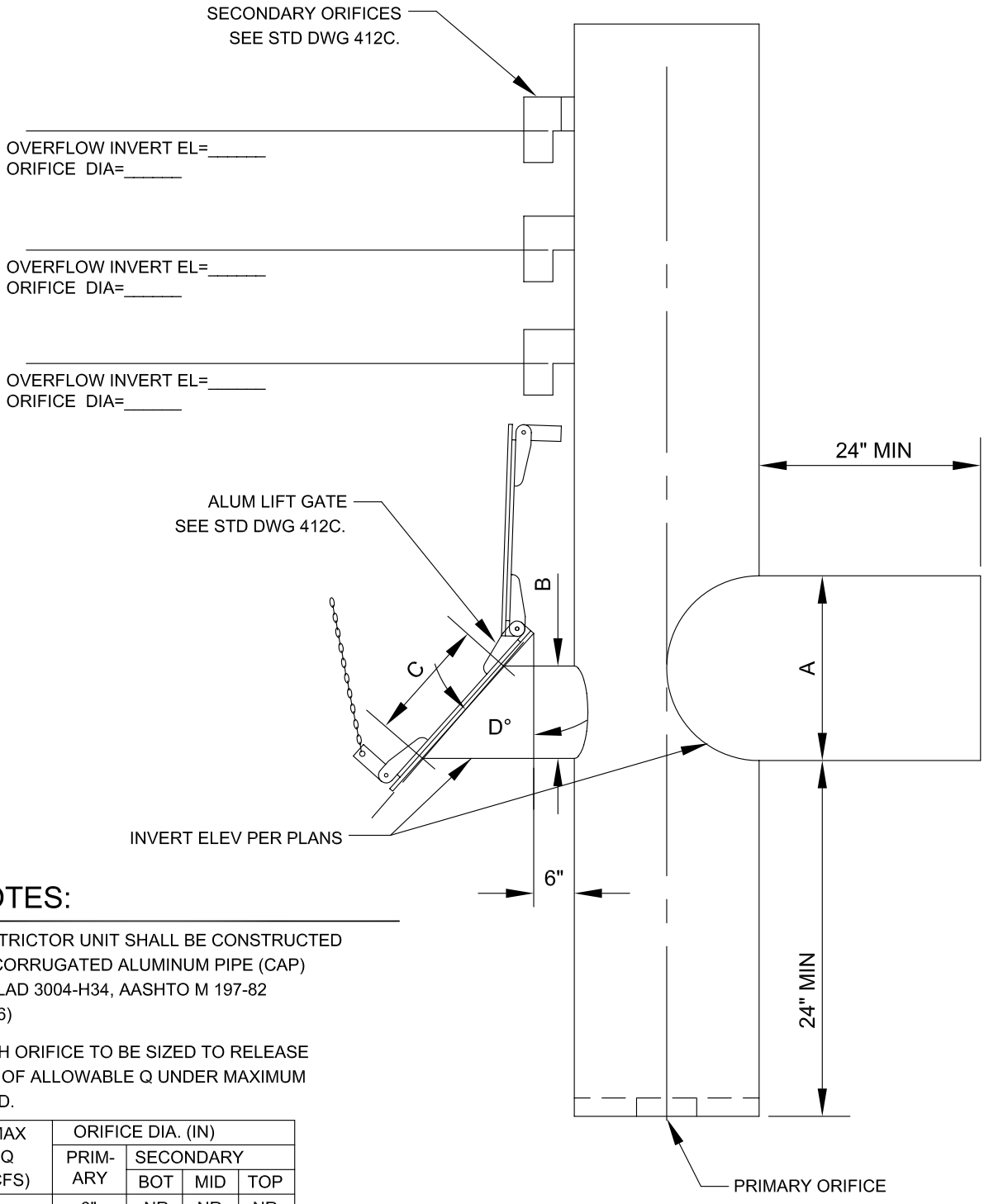
SNOHOMISH WASHINGTON

TYPICAL RESTRICTOR INSTALLATION

Approved By:
City Engineer
Date: May 30, 2004

412a
Number

City of Snohomish Public Works Department



NOTES:

1 RESTRICTOR UNIT SHALL BE CONSTRUCTED OF CORRUGATED ALUMINUM PIPE (CAP) ALCLAD 3004-H34, AASHTO M 197-82 (1986)

2 EACH ORIFICE TO BE SIZED TO RELEASE 25% OF ALLOWABLE Q UNDER MAXIMUM HEAD.

MAX Q (CFS)	ORIFICE DIA. (IN)			
	PRIM-ARY	SECONDARY		
		BOT	MID	TOP
0-5	3"	NR	NR	NR
6-10	6"	NR	NR	NR
11-25	3"	3"	NR	NR
	8"	4"	3"	NR
	9"	3"	3"	3"

ITEM	DESCRIPTION	SIZE	
A	OUTLET	18" AND SMALLER	24" AND LARGER
B	CLEANING	6" ID	10" ID
C	GATE SIZE	8" OPENING	12" OPENING
D	ANGLE	42°±	34°±

SNOHOMISH WASHINGTON

TYPICAL RESTRICTOR INSTALLATION

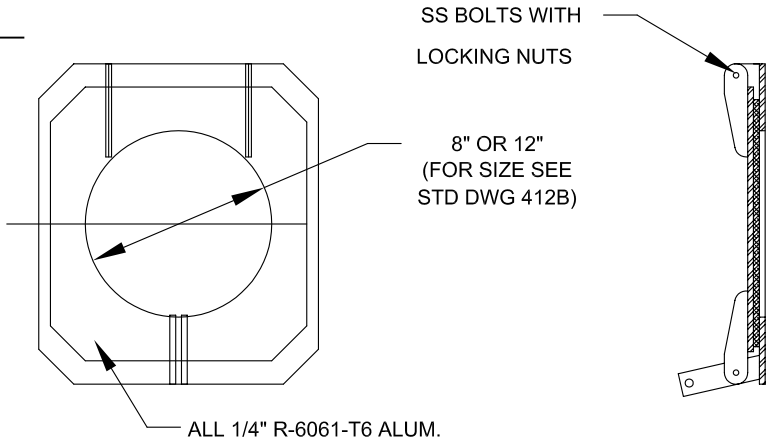
Approved By:
City Engineer
Date: May 30, 2004

412b
Number

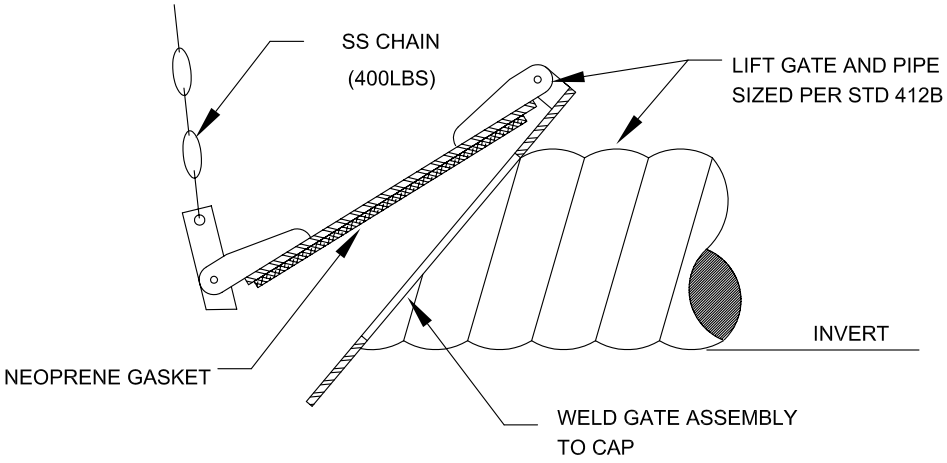
City of Snohomish Public Works Department

NOTES:

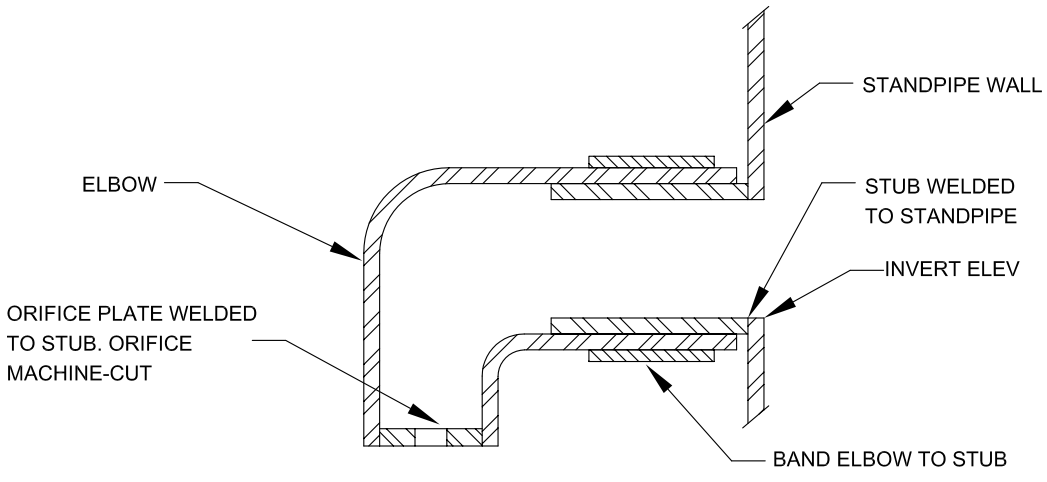
LIFT GATE SHALL BE CONSTRUCTED OF: 1/4" R-6061-T6 ALUM. W/CLOSED CELL NEOPRENE PER ASTM 1056-67 CHEMICAL RESISTANT (OIL & GREASE), OZONE RESISTANT, 67° TO +250° F SERVICE TEMP.



LIFT GATE DETAIL



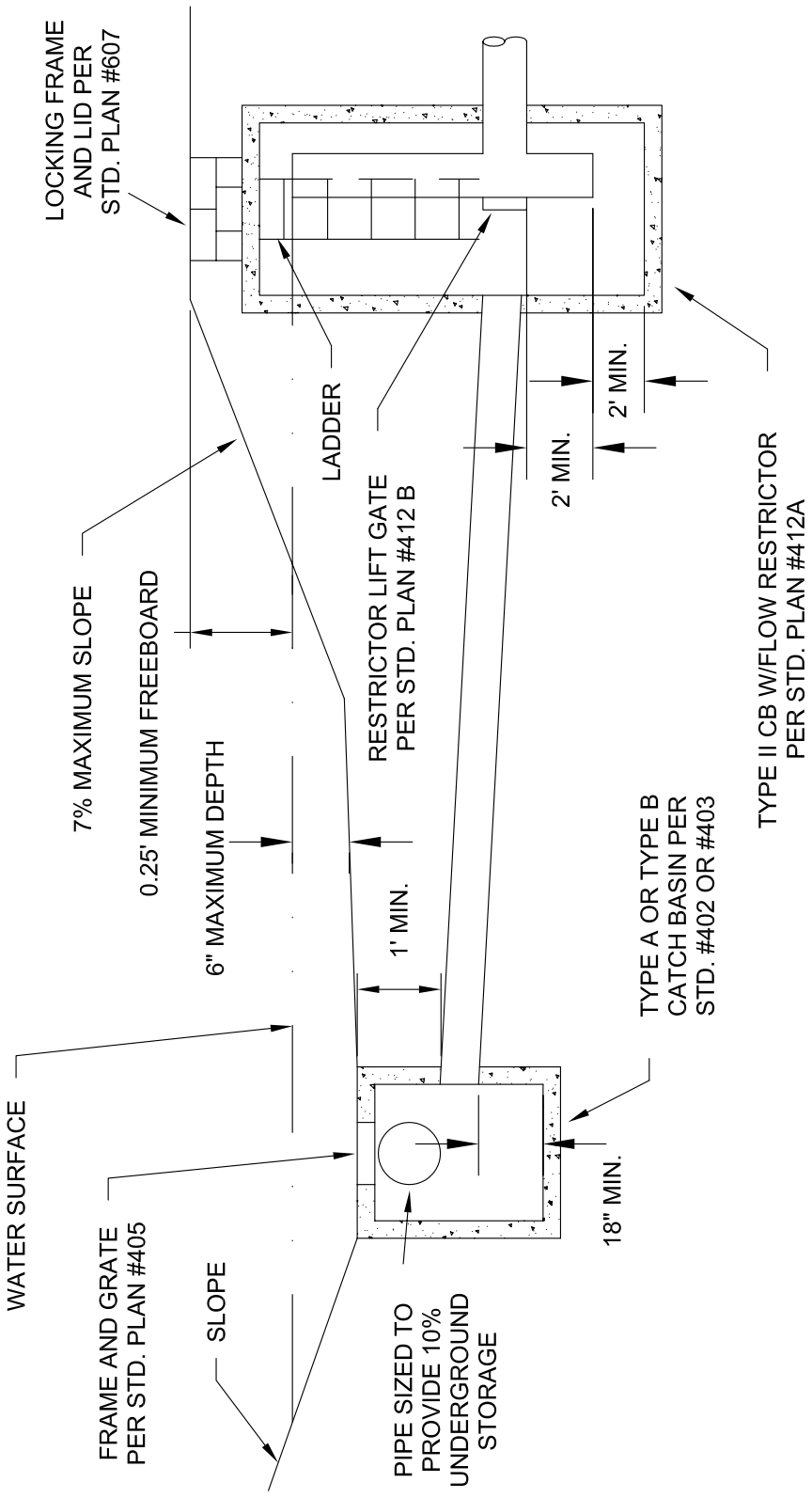
LIFT GATE ASSEMBLY



ALL PARTS TO BE R-6061-T6 ALUM

SECONDARY ORIFICE DETAIL

SNOHOMISH WASHINGTON	**LIFT GATE ASSEMBLY AND SECONDARY ORIFICE DETAIL**	Approved By: City Engineer Date: May 30, 2004	412c Number
	City of Snohomish Public Works Department		



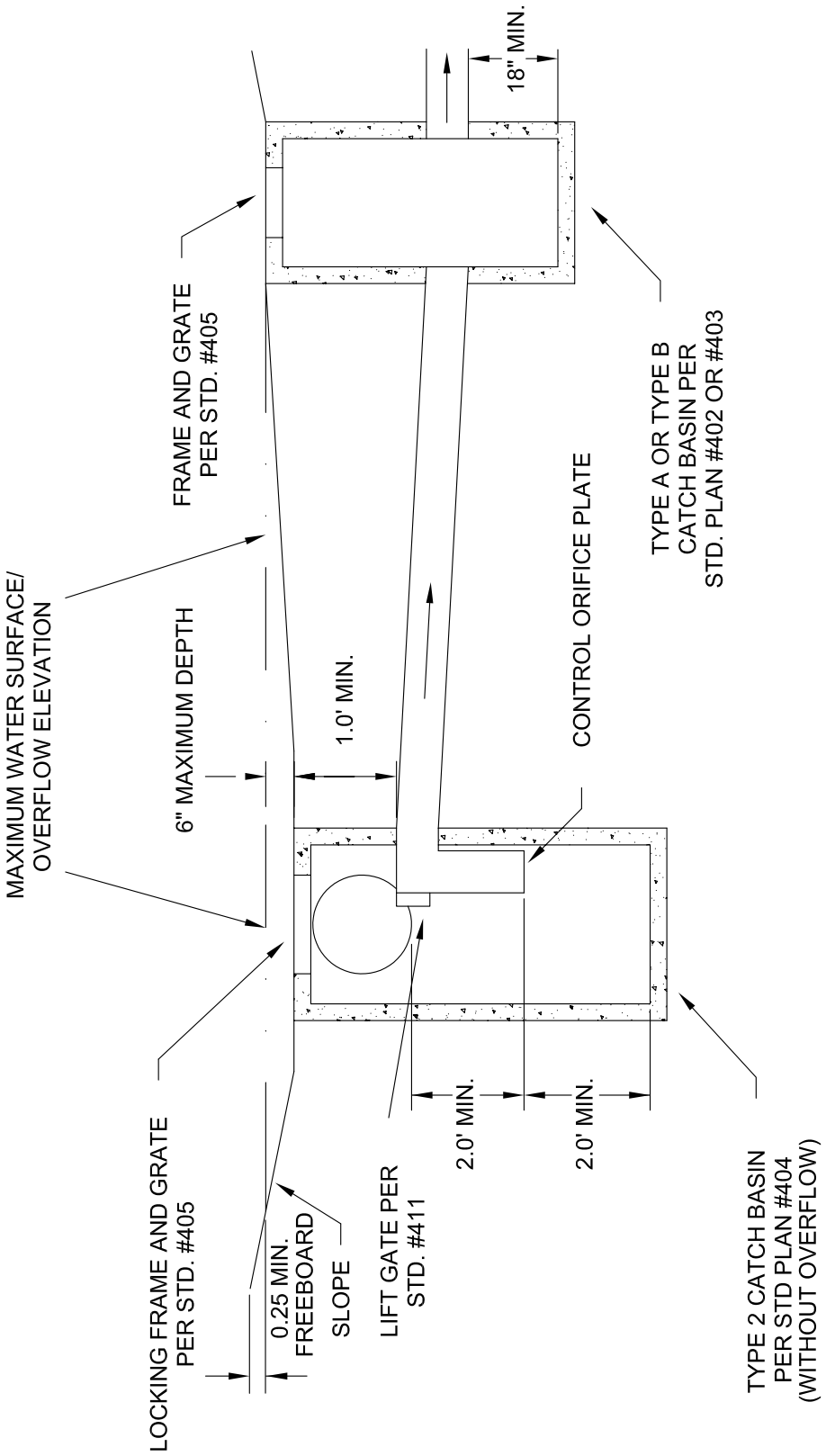
SNOHOMISH WASHINGTON

TYPICAL PARKING LOT DETENTION

Approved By:
City Engineer
Date: May 30, 2004

414
Number

City of Snohomish Public Works Department



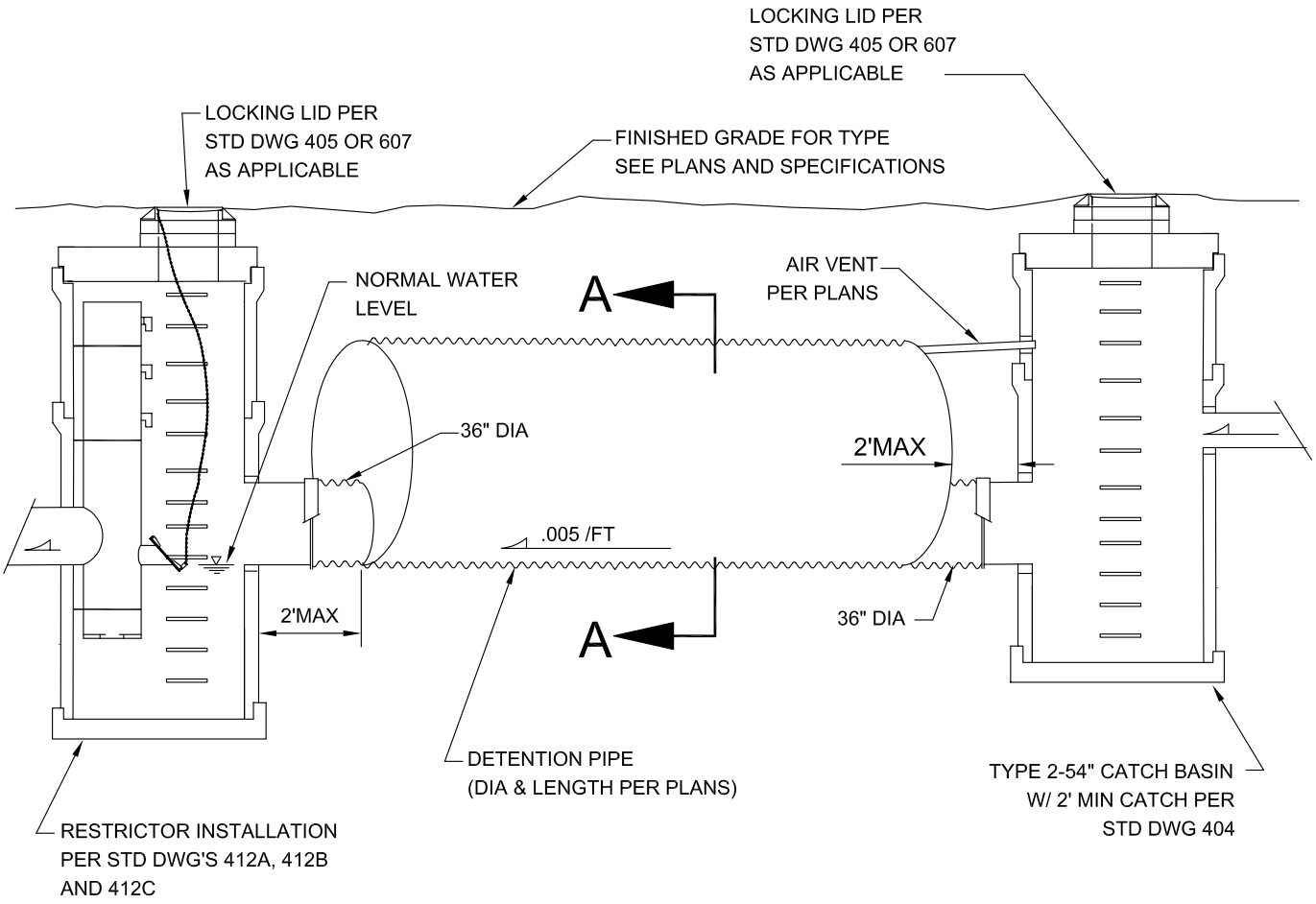
SNOHOMISH
WASHINGTON

TYPICAL PARKING LOT DETENTION

Approved By:
City Engineer
Date: May 30, 2004

415
Number

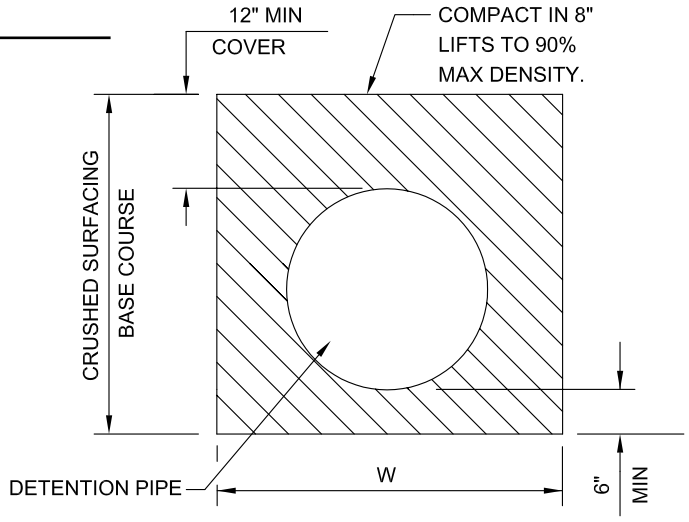
City of Snohomish Public Works Department



ELEVATION

NOTES:

1. DETENTION PIPE SHALL BE FABRICATED FROM ONE OF THE FOLLOWING:
   A. CORRUGATED STEEL PIPE
      14 GAGE MIN, WITH TREATMENT #1.
   B. CORRUGATED ALUMINIUM PIPE 12 GAGE MIN.
   C. HIGH DENSITY POLYETHYLENE PIPE.
2. ANNUAL INSPECTIONS AND CLEANING REQUIRED BY OWNER TO INSURE PROPER OPERATION OF DETENTION SYSTEM.
3. W = MAXIMUM WIDTH OF TRENCH FOR PIPES 15" OR LESS IN DIAMETER W = 40". FOR PIPES 18" OR GREATER W = 1 1/2" x I.D. + 18".
4. COMPACT IN 8" LIFTS TO 90% MAX DENSITY.



SECTION A-A

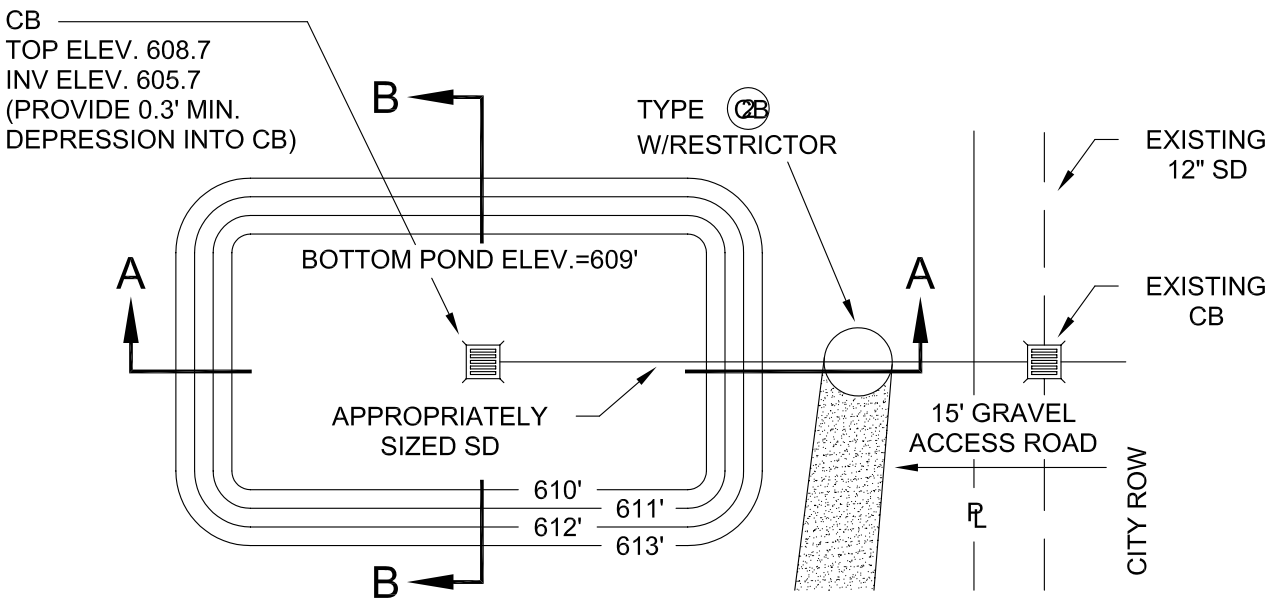
SNOHOMISH WASHINGTON

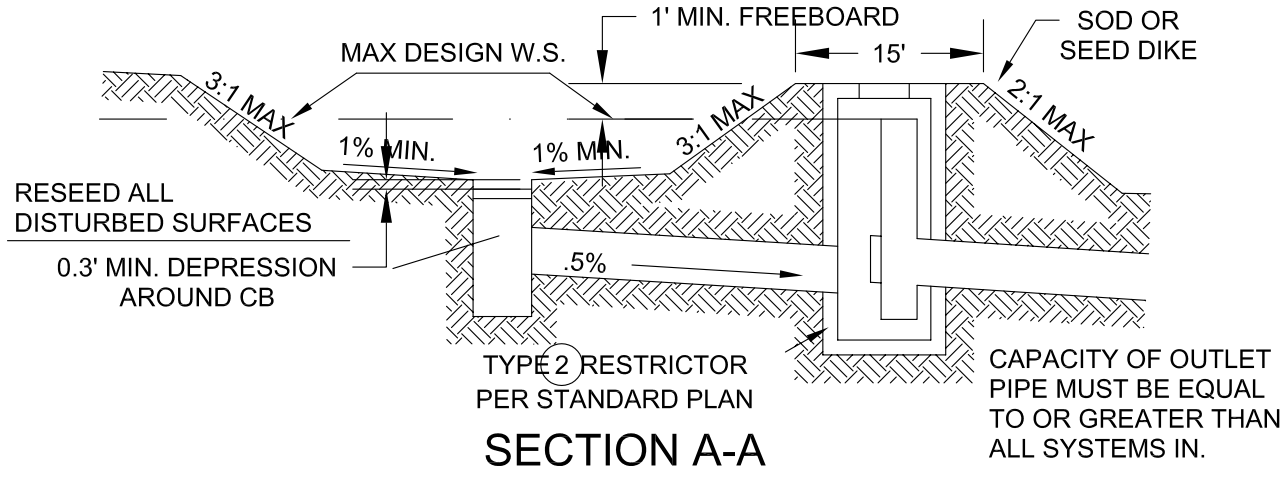
TYPICAL CLOSED UNDERGROUND DETENTION SYSTEM

Approved By:
City Engineer
Date: May 30, 2004

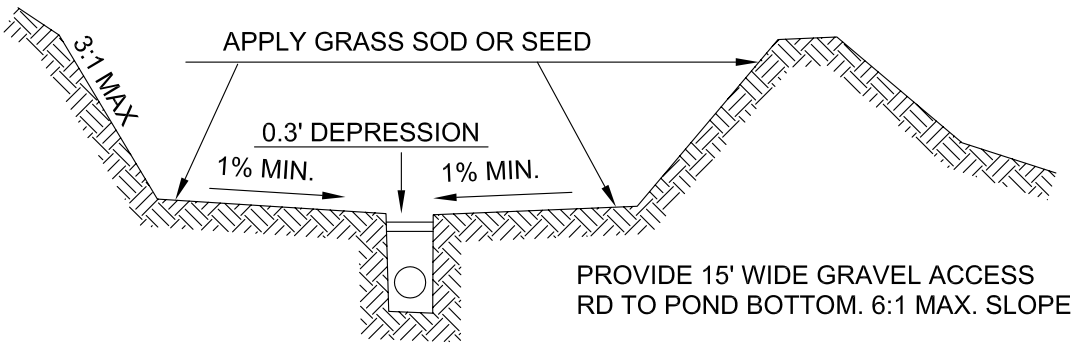
416
Number

City of Snohomish Public Works Department





SECTION A-A



SECTION B-B

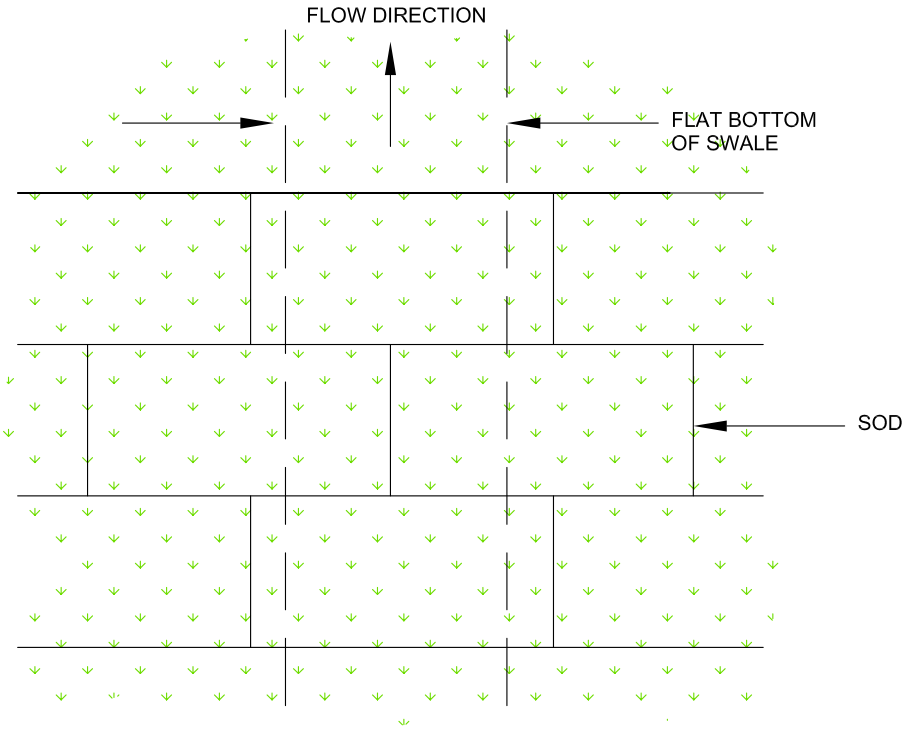
SNOHOMISH
WASHINGTON

TYPICAL DRY TYPE DETENTION POND

Approved By:
City Engineer
Date: May 30, 2004

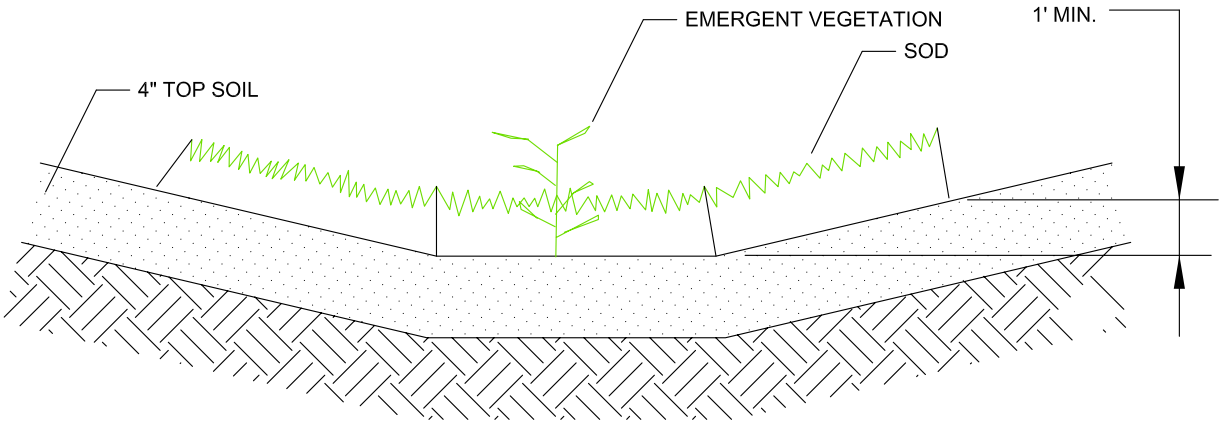
417
Number

City of Snohomish Public Works Department



BIOFILTRATION SWALE PLAN VIEW

NTS



BIOFILTRATION SWALE CROSS SECTION

NTS

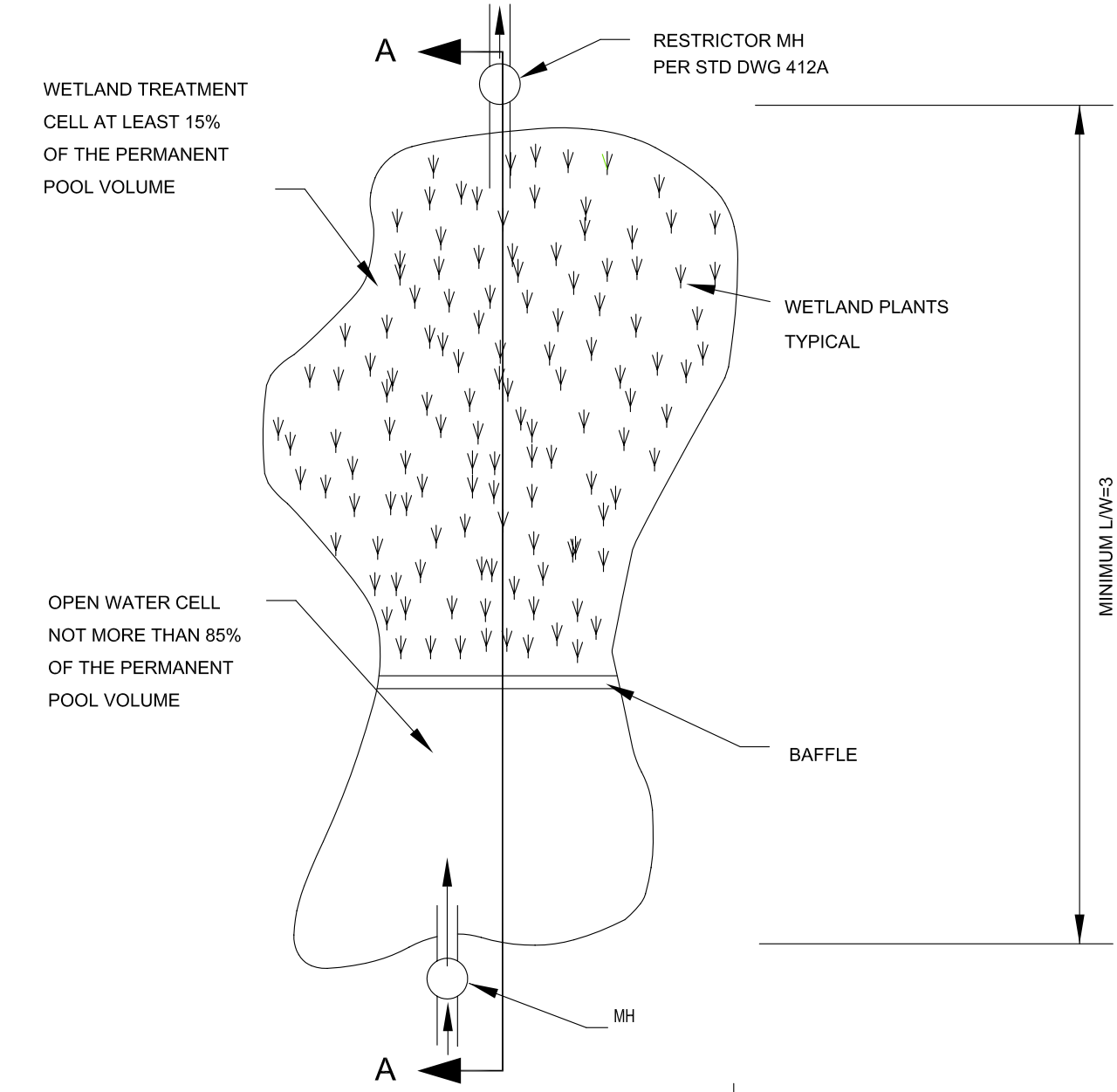
SNOHOMISH
WASHINGTON

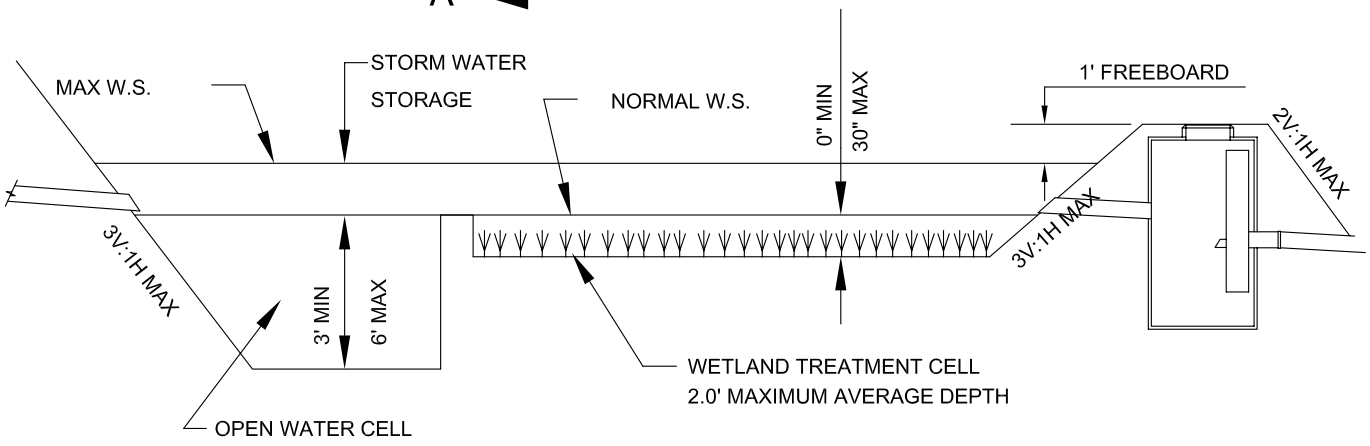
BIOFILTRATION SWALE

Approved By:
City Engineer
Date: May 30, 2004

419
Number

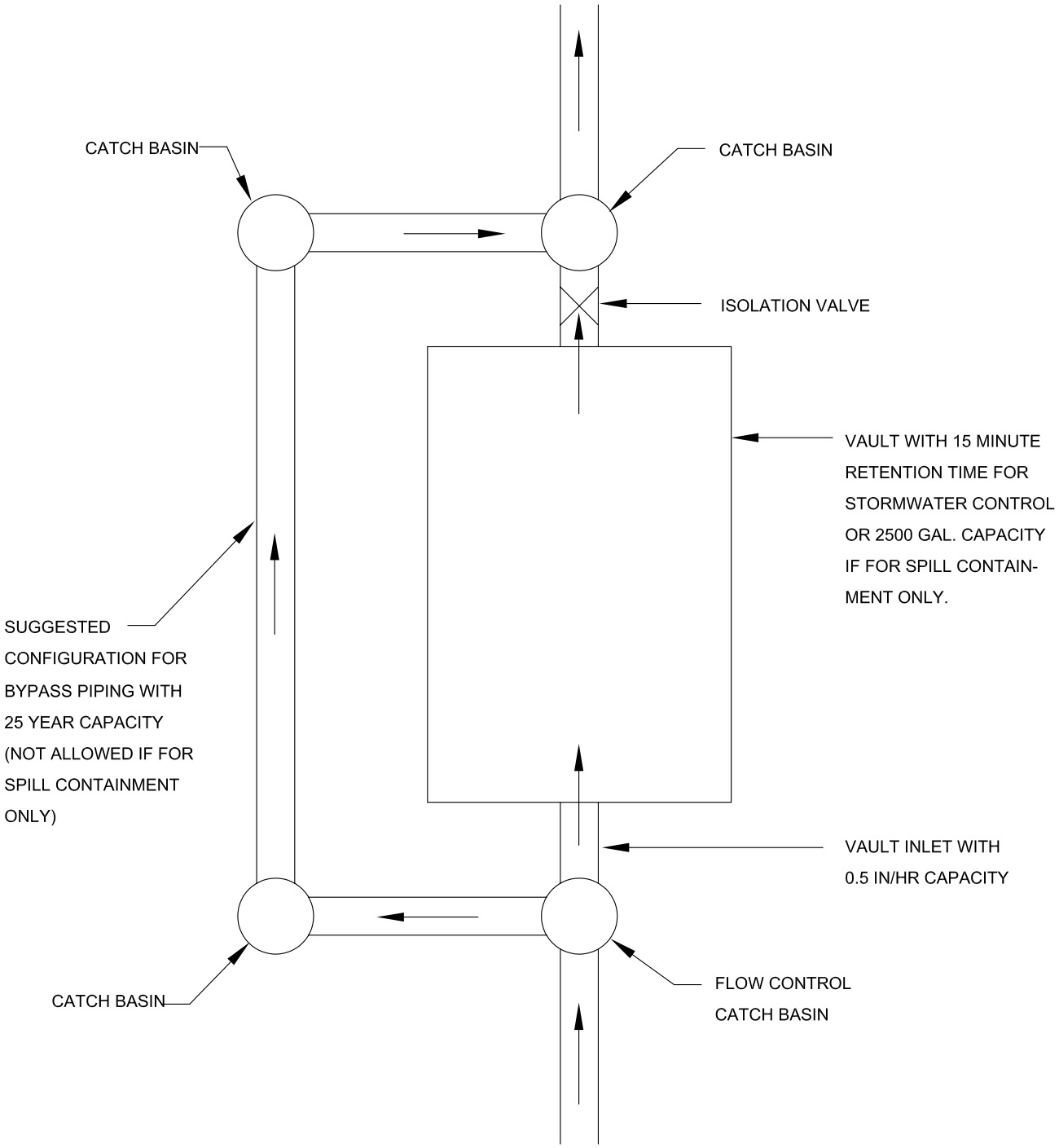
City of Snohomish Public Works Department





SECTION A-A

SNOHOMISH WASHINGTON	**TYPICAL WET POND LAYOUT**	Approved By: City Engineer Date: May 30, 2004	420 Number
	City of Snohomish Public Works Department		



PLAN VIEW

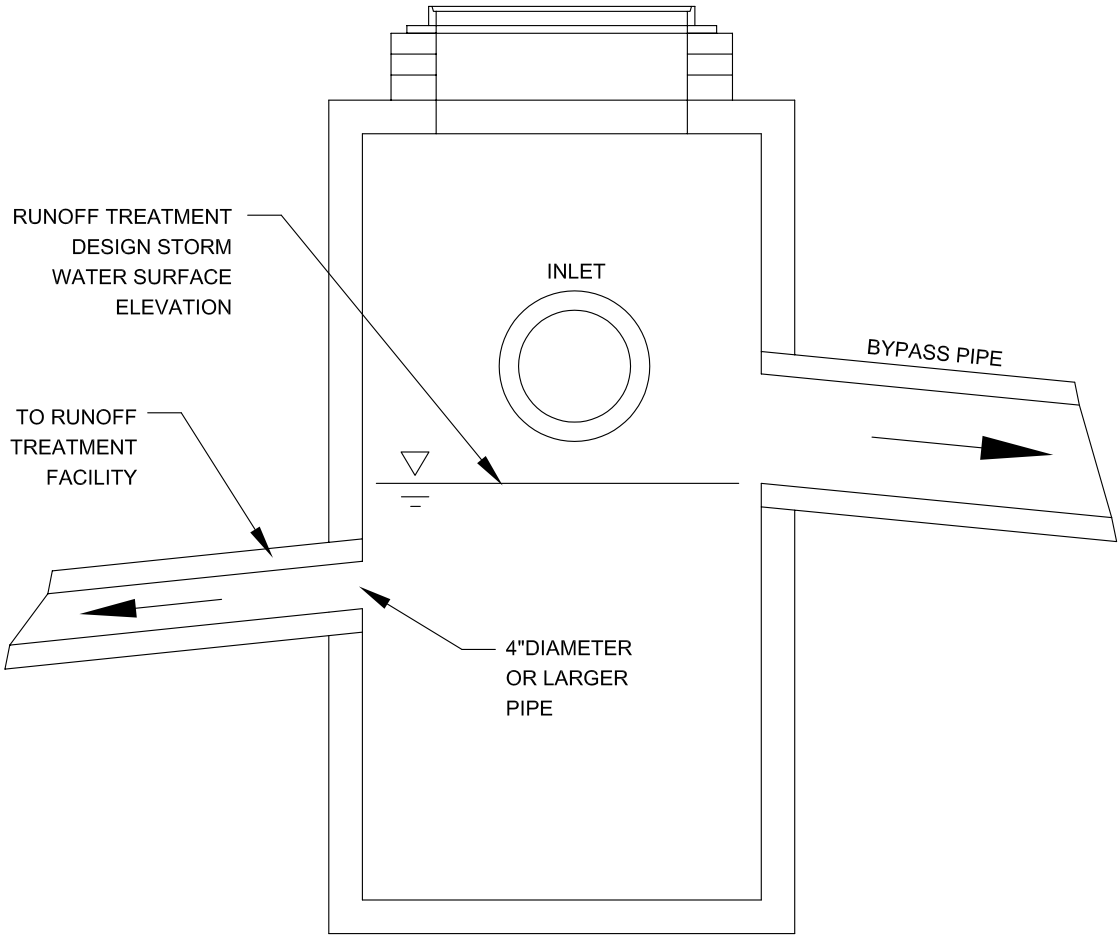
SNOHOMISH
WASHINGTON

OIL/WATER SEPARATOR AND GRIT REMOVAL VAULT

Approved By:
City Engineer
Date: May 30, 2004

421
Number

City of Snohomish Public Works Department



SECTION

NTS

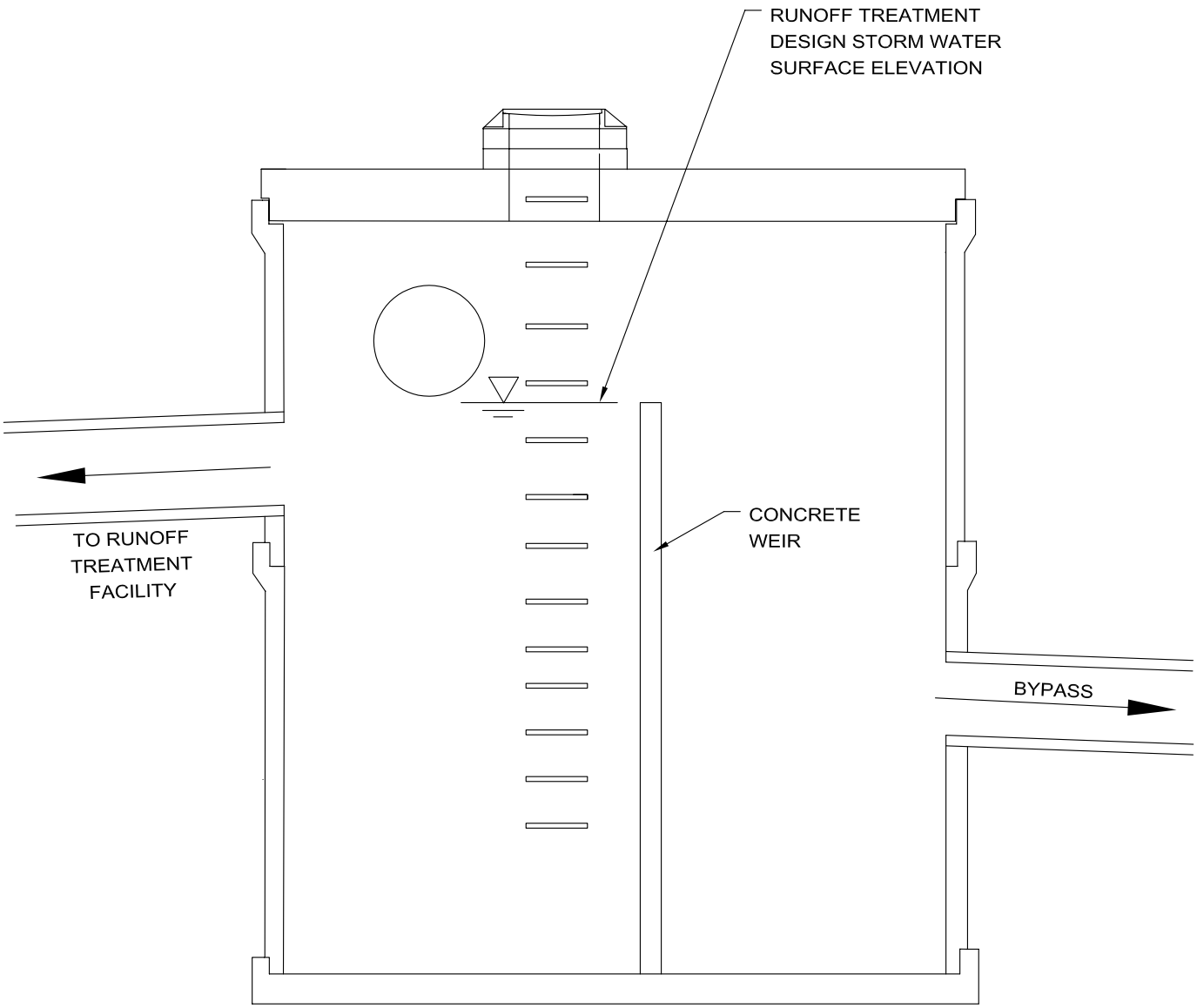
SNOHOMISH
WASHINGTON

BYPASS STRUCTURE TYPE 1

Approved By:
City Engineer
Date: May 30, 2004

422a
Number

City of Snohomish Public Works Department



CONCRETE VAULT OR TYPE 2 CATCH BASIN
(SIZE DETERMINED BY CLEARANCE REQUIREMENTS)

ELEVATION

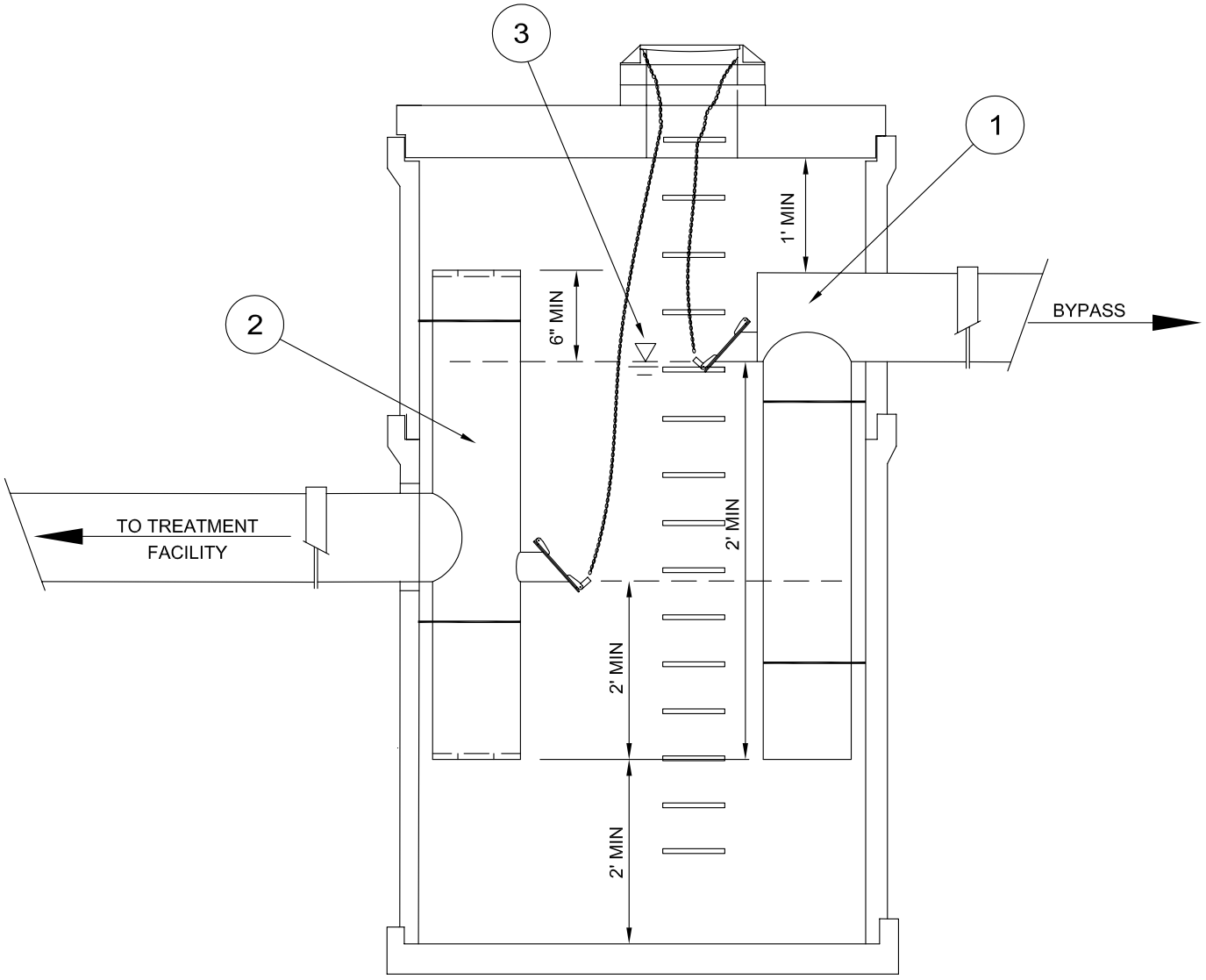
SNOHOMISH
WASHINGTON

BYPASS STRUCTURE TYPE 2

Approved By:
City Engineer
Date: May 30, 2004

422b
Number

City of Snohomish Public Works Department



ELEVATION

N.T.S.

NOTES:

1. FLOATABLE MATERIAL SEPARATOR PER CITY STD DWG 411, WITHOUT VENT HOLE.
2. RESTRICTOR STANDPIPE WITHOUT SECONDARY OVERFLOW ORIFICES AND WITH PRIMARY ORIFICE AND A CAP OR PLATE WITH 1" DIAMETER (MAX) VENT HOLE. INSTALL PER STD DWG 412A.
3. RUNOFF TREATMENT DESIGN STORM WATER SURFACE ELEVATION.

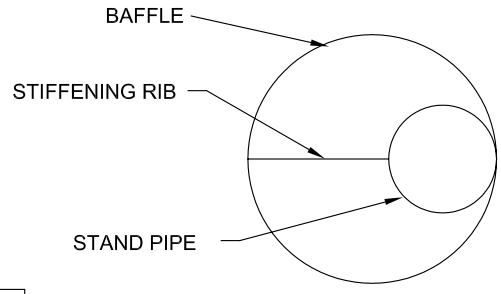
SNOHOMISH
WASHINGTON

BYPASS STRUCTURE TYPE 3

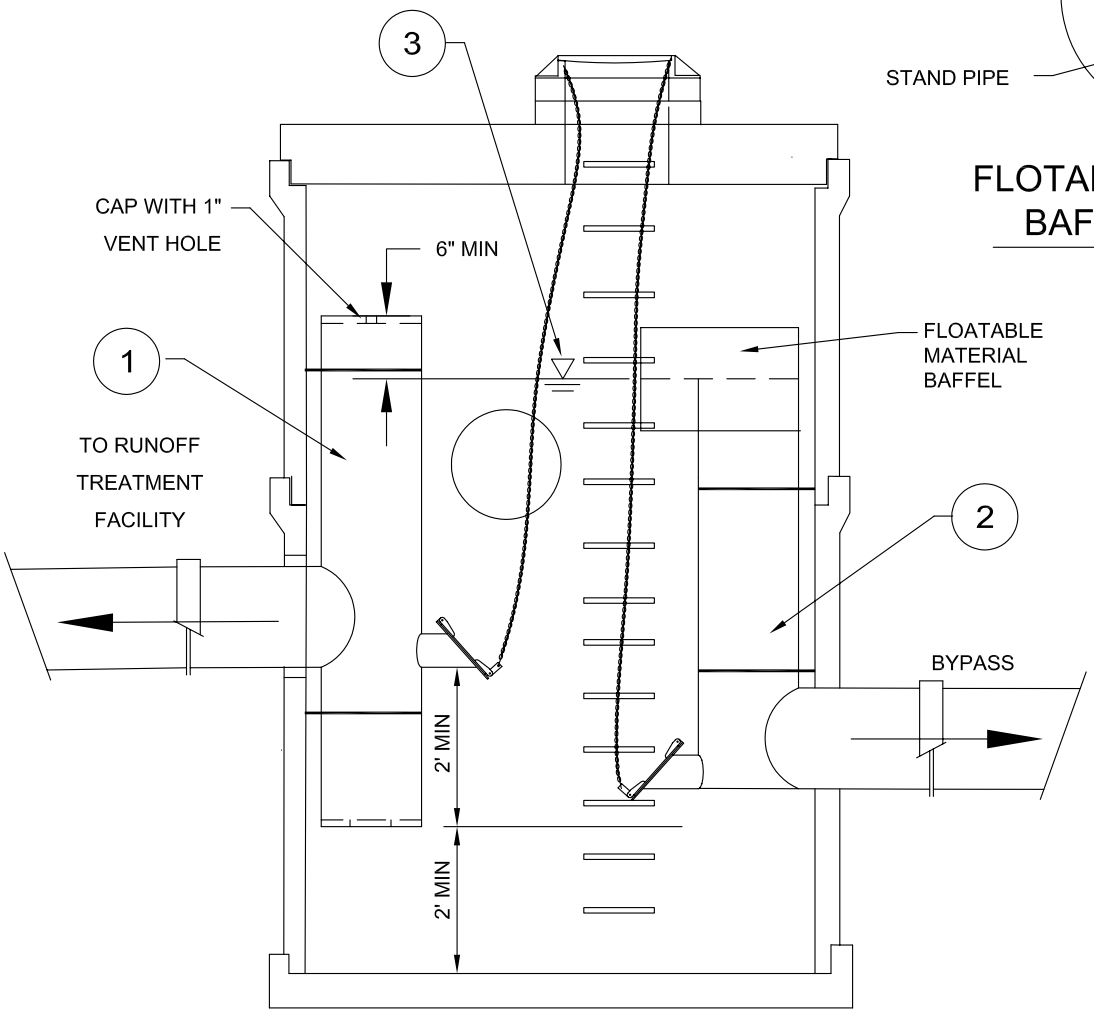
Approved By:
City Engineer
Date: May 30, 2004

422c
Number

City of Snohomish Public Works Department



FLOTABLE MATERIAL
BAFFLE DETAIL



ELEVATION

NOTES:

1. RESTRICTOR STANDPIPE WITHOUT SECONDARY OVERFLOW ORIFICE AND WITH PRIMARY ORIFICE AND CAP OR PLATE WITH 1"DIAMETER (MAX) VENT HOLE. INSTALL PER STD DWG 412A.
2. RESTRICTOR STANDPIPE WITHOUT ANY PRIMARY OR SECONDARY ORIFICES AND WITH FLOATABLE MATERIAL BAFFLE. INSTALL PER CITY STANDARD DWG 412A.
3. RUNOFF TREATMENT DESIGN STORM WATER SURFACE ELEVATION.

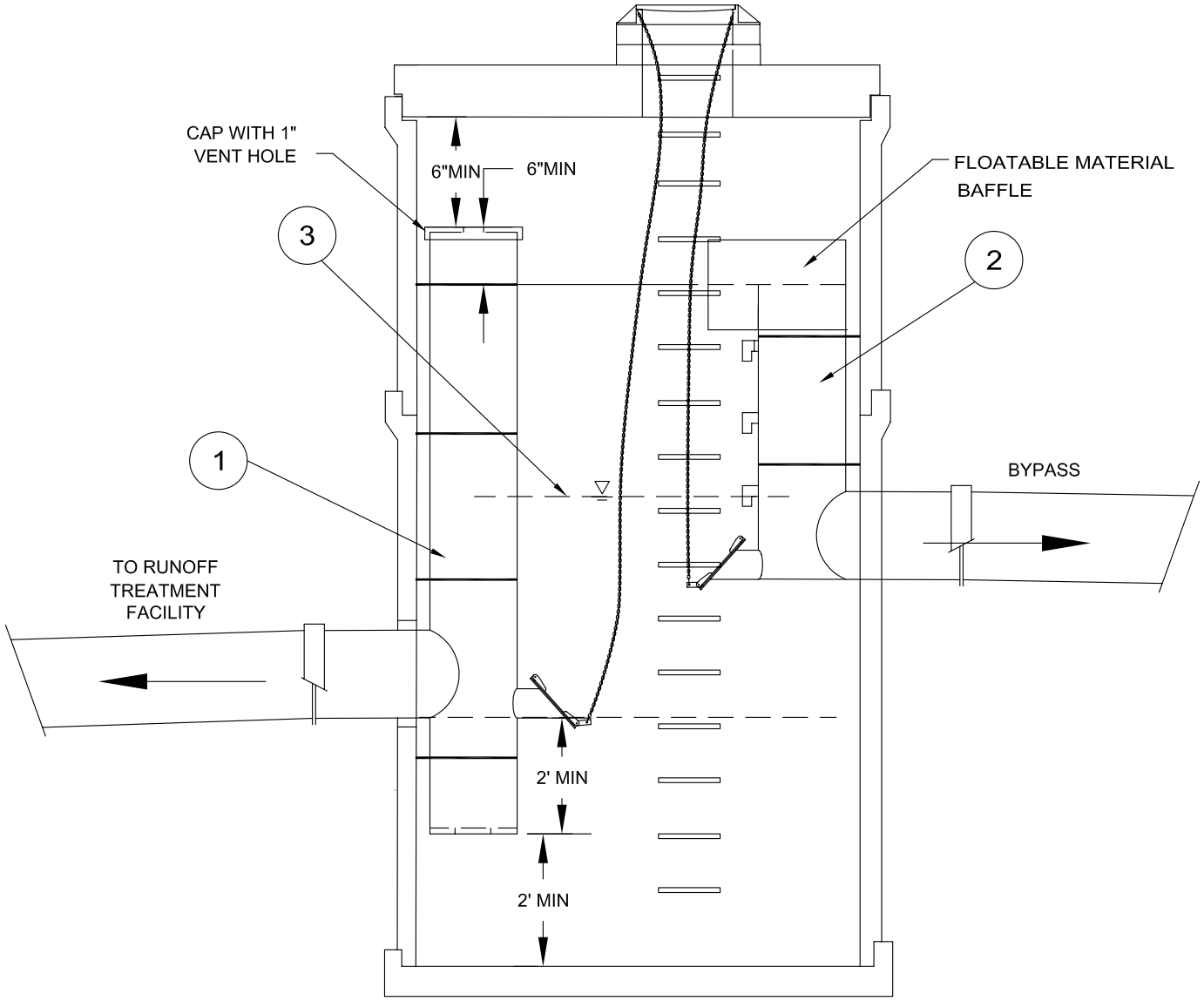
SNOHOMISH
WASHINGTON

BYPASS STRUCTURE TYPE 4

Approved By:
City Engineer
Date: May 30, 2004

422d
Number

City of Snohomish Public Works Department



ELEVATION

N.T.S.

NOTES:

1. RESTRICTOR STANDPIPE WITHOUT SECONDARY ORIFICES AND WITH PRIMARY ORIFICE AND A CAP OR PLATE WITH 1" DIAMETER (MAX) VENT HOLE. INSTALL PER STD DWG 412A.
2. RESTRICTOR STANDPIPE WITH SECONDARY ORIFICES AND A FLOATABLE MATERIAL BAFFLE (PER STD DWG 422D). INSTALL PER STD DWG 412A.
3. RUNOFF TREATMENT DESIGN STORM WATER SURFACE ELEVATION.

SNOHOMISH
WASHINGTON

MULTIPLE RESTRICTOR STRUCTURE

Approved By:
City Engineer
Date: May 30, 2004

422e
Number

City of Snohomish Public Works Department

ENGINEERING STANDARDS

SECTION 5

WATER DISTRIBUTION

5-1 DESIGN

5-1.01 GENERAL

Extension or modification to the City's water system shall be made strictly in accordance with the construction plans prepared in accordance with these Standards, the Snohomish Municipal Code, and the City's Water Comprehensive Plan, and approved by the City Engineer. Additionally, all construction of water system extensions shall conform to these Standards, applicable American Water Works Association (AWWA) Specifications and the current WSDOT/APWA Standard Specifications.

These Standards do not include design of the City's general facilities such as wells, pump stations, in-ground reservoirs, storage tanks, or treatment plant. The City's general facilities require special design and will be reviewed and approved by the City Engineer on a case-by-case basis.

5-1.02 HYDRAULIC REQUIREMENTS

Distribution System Pressure

All water mains shall be sized following a hydraulic analysis based on flow demands and pressure requirements. Per the DOH design manual current edition, during fire suppression events, the water system must be able to provide 20-psi minimum pressure at ground level at all points throughout the distribution system. The water system must be able to provide this minimum pressure under fire-flow conditions plus the MDD rate when all equalizing and fire flow storage is depleted (WAC 246-290-230(6)). The required working pressure in the City of Snohomish distribution system should be approximately 40 to 80 psi, not less than 30 psi when all equalizing storage is depleted, and not greater than 90 psi due to the age and condition of the current infrastructure. Higher pressures in unusual circumstances shall be submitted to the City Engineer for review. Individual pressure reducing valves (PRV) are required when the static pressure at the service level exceeds 60 psi.

Distribution System Velocities

Per the DOH design manual current edition, "DOH recommends a maximum velocity of no more than 8-feet per second (fps) under PHD conditions, unless the pipe manufacturer specifies otherwise. Maximum velocities greater than 8 fps may occur under fire flow conditions, for short main sections, or piping in pump and valve station facilities.

Engineers should conduct a hydraulic transient (water hammer) analysis for distribution piping designed to exceed 10 fps during PHD or fire flow conditions (Walski et al. 2003; AWWA 2004)."

Distribution System Fire Flow

Fire flow requirements shall be determined by the City Fire Marshall. The available fire flow will be determined by the City Engineer using the water system hydraulic model.

5-1.03 WATER MAIN EXTENSION

Residential

Water main extensions shall be required when a parcel does not front an existing water main. All residential water main extensions shall be extended to a point 10 feet beyond the farthest property corner where practical as determined by the City Engineer. In addition, water mains shall be installed through all internal streets and looped to all adjacent mains that will, in the City Engineer's opinion, extend past or through the property in the future; and stub to the property line where it is likely that they will be needed to connect to future mains. Depending on the property size, shape and the Water Comprehensive Plan, the City may require mains to be constructed on more than one, and up to all, sides of the property. The City Engineer may also require that extra service lines be installed to be used for sampling stations.

If a proposed water main extension is located in 2 or more pressure zones, the City Engineer may require installation of pressure reducing stations, isolation valves, check valves, and/or booster pump stations.

Dead-end mains shall be kept to a minimum by making appropriate looping and tie-ins whenever practical in order to provide increased reliability of service, assist with water quality and reduce head loss.

If a water main extension directly benefits multiple property owners, a latecomer reimbursement agreement in accordance with SMC Chapter 15.17 may be appropriate.

Non-Residential Properties

Water main extensions shall be required when a parcel does not front an existing water main. All non-residential water main extensions shall be extended to a point 10 feet beyond the farthest property corner where practical as determined by the City Engineer, and/or stub or connect to present and future mains or as directed by the City Engineer. The City Engineer may also require that extra service lines be installed to be used for sampling stations.

If the proposed water main extension is located in 2 or more pressure zones, the City Engineer may require installation of pressure reducing stations, isolation valves, check valves, and/or booster pump stations.

Dead-end mains shall be kept to a minimum by making appropriate looping and tie-ins whenever practical in order to provide increased reliability of service, assist with water quality and reduce head loss.

If a water main extension directly benefits multiple property owners, a latecomer reimbursement agreement in accordance with SMC Chapter 15.17 may be appropriate.

5-1.04 WATER MAIN LOCATION

Water mains and appurtenances should be installed within the right-of-way of public streets and roads. Water mains may be installed within City easements across private properties. Water mains within public right-of-way shall be located on the north and east sides of the centerlines. Water mains shall be in the shoulder of the roadway for rural roads, and approximately 6 feet from the street centerline for urban streets. See Standard Plan 322.

The City Engineer may approve exceptions to this requirement in order to minimize the cutting and replacing of pavement, to avoid conflicts with other underground utilities, to permit sanitary sewers to be installed on the "low side" of streets, or for other appropriate reasons. As nearly as practical, mains shall be installed on a particular street with the distance from the property line and/or centerline varied as little as possible. Water mains shall not be located under or behind parking lanes, curbs, gutters, or sidewalks. Valve boxes shall be located outside the normal wheel track whenever possible.

If there is an easement across a paved area on private property, the water main shall be installed in the driving lanes (not under parking stalls).

Water mains may be laid along road/street curves by using bends. Pipe joint deflection may be allowed upon approval by the City Engineer but the deflection shall not exceed one half of the manufacturer's recommended deflection.

5-1.05 HORIZONTAL SEPARATION

Water mains shall be laid at least 10 feet horizontally from any existing or proposed sanitary sewer, storm drain, and septic tank and/or absorption field. The distance shall be measured edge to edge. If the City Engineer determines it is not practical to maintain a 10 foot separation, the City Engineer may approve deviation on a case-by-case basis using criteria from the Washington State Department of Ecology (DOE). Such deviation may include installation of the water main closer to a sanitary sewer provided that the water main is laid in a separate trench or on undisturbed earth shelf located on one side of the sanitary sewer at such an elevation that the bottom of the water main is at least 18

inches above the top of the sewer. Water service connections and side sewers shall have minimum horizontal clearance of 10 feet unless otherwise approved by the City Engineer.

Minimum horizontal clearances from water mains:

Utility	Minimum Horizontal Clearance in feet
Cable	5
Gas	5
Power	5
Telephone, Fiber Optic	5
Storm Drain	10
Sanitary Sewer	10
Septic Tanks/Drainfields	10

5-1.06 VERTICAL SEPARATION

Wherever practical, water mains shall cross other utilities at right angles. If this is not practical as determined by the City Engineer, the crossing angle shall be maintained between 45 and 90 degrees. Water mains crossing sanitary and storm sewers shall be laid to provide a minimum vertical clearance of 18 inches between the outside of the water main and the outside of the sewer. Where a water main crosses sanitary and storm sewers, one full length of water pipe shall be used with the pipe centered over the sewer for maximum joint separation. When the above conditions cannot be met and in accordance with applicable DOE criteria, the City Engineer may approve a variance, but shall require that the sewer be constructed of ductile iron pipe and be pressure tested before being activated, and/or be encased.

Minimum vertical clearances from water mains:

Utility	Minimum Vertical Clearance in feet
Cable	1
Gas	1
Power	1
Telephone, Fiber Optic	1
Storm Drain	1.5
Sanitary Sewer	1.5

When it is not practical to maintain this minimum separation between the water main and sanitary and storm sewers, the vertical clearance may be less than 6 inches and Ethafoam pads are required. The water main may also be encased per Section 5.1.11 Steel Casing.

5-1.07 SETBACK DISTANCE FROM BUILDINGS

Water mains shall be located a minimum of 5 feet from covered parking, 10 feet minimum from building and retaining walls. A 20 foot wide minimum easement shall be provided for a water main between buildings.

5-1.08 WATER MAIN SIZING

Transmission mains and specific areas outlined in the City's Water Comprehensive Plan require 12 inch or larger water mains.

When serving fire hydrants and for local distribution mains in residential areas, 8 inch or larger pipe is required.

6 inch pipe shall not be used as part of the distribution system except as fire hydrant runs not longer than 50 feet.

5-1.09 PIPE MATERIAL

Water mains shall be cement mortar lined ductile iron pipe (DIP) Class 52 unless otherwise approved by the City Engineer.

5-1.10 PIPE FITTINGS

Pipe shall be furnished with mechanical joints or rubber gasket push-on joints (Tyton joint only) unless flanged joints or restrained joints are required. Horizontal or vertical bends shall be used when joint deflection would exceed one-half of the pipe manufacturer's recommended maximum deflection. Restrained joints are required on all hydrant runs, steep slopes, non-bearing soils and in other conditions as determined necessary by the City Engineer.

5-1.11 STEEL CASING

Ductile iron pipe shall be encased in a steel casing when crossing: (1) under rockeries over 5 feet high; (2) under retaining wall footings over 5 feet wide; and (3) under reinforced earth retaining walls (both wall and reinforcing material). Casings shall extend a minimum of 5 feet past each edge of the structure, or a distance equal to the depth of pipe, whichever is greater. Minimum vertical clearance between the bottom of the wall or footing and top of the pipe or casing shall be 2 feet. The pipe trench at the casing shall be backfilled with gravel backfill material when the vertical clearance is less than 3 feet.

Ductile iron pipe shall be encased in a steel casing when crossing under a railroad or State/County highway. Casings and carrier pipe shall be installed in accordance with applicable other local, State, and/or Federal laws and/or regulations. In the case of

railroad crossings, the project shall also comply with regulations established by the railroad company.

The carrier pipe shall be supported by casing spacers. Casing spacers shall be placed under the carrier pipe to ensure approximate centering within the casing pipe and to prevent damage during installation. See Standard Plan 522.

Steel casings may also be required when water mains cross creeks or wetlands.

5-1.12 COVER DEPTH

A cover depth of 3 feet above the top of water mains shall be maintained if possible. The cover depth shall not be less than 3 feet or more than 5 feet without the approval of the City Engineer.

If the water main is within the State or County right-of-way, the cover depths shall be in accordance with the State or County requirements.

5-1.13 SLOPES

Where the longitudinal slopes are 20% or greater, all pipe joints shall be restrained. Anchor blocks shall be used in conjunction with joint restraint where slopes are 20% or greater. Timber baffle/hill holders shall be required on unpaved slopes that exceed 20%. Maximum spacing between the holders shall be 18-foot on center with a minimum of 1 holder for each pipe length.

5-1.14 POLYETHYLENE ENCASEMENT

The City Engineer may require that ductile iron pipe and fittings be protected by 8-mil polyethylene encasement in areas of severely corrosive soils or in accordance with AWWA/ANSI C105/A21.5.

5-1.15 CONCRETE BLOCKING

When using horizontal and vertical concrete blocking, show locations and type of blocking on the plans as shown on Standard Plans 505 and 506. Concrete blocking is required on all fittings including restrained joint fittings.

An 8 inch pipe at a vertical bend shall be restrained a minimum of 36 feet (2 joints) from each side of a bend. A 12 inch or larger pipe at a vertical bend shall be restrained a minimum of 54 feet (3 joints) from each side of a bend. No change in horizontal direction or diameter shall occur within 36 feet of the vertical bend. Special blocking or joint restraint designs may be required for poor soil, conflicting utility, etc.

5-1.16 ASBESTOS CONCRETE PIPE CROSSING

When a proposed water main crosses existing asbestos concrete (A.C.) pipe, the City shall require removal and replacement of the A.C. pipe with ductile iron pipe at the expense of the person or entity responsible for the water main project. The A.C. pipe removal and disposal shall be performed in accordance with all applicable Puget Sound Clean Air Agency (PSCAA) requirements, including but not limited to obtaining a permit from PSCAA for the removal work, and other applicable State and/or Federal laws and regulations. A permit from PSCAA to perform the A.C. pipe removal is required prior to construction drawing approval. Ductile iron pipe crossings shall be connected to the existing A.C. main with Romac extended range transition couplings.

5-1.17 VALVES

Sufficient valves shall be provided on water mains so that interrupted service and sanitary hazards will be minimized during repairs. Valves shall be located at no more than 300 foot intervals in commercial, industrial and multi-family areas and at no more than one block or 600 foot intervals in other areas. At water main intersections, valves shall be placed on 4 legs at each cross and 3 legs at each tee (unless tapping an existing water main). The valves shall be spaced so that no more than one fire hydrant is removed from service with any separate main shut down.

An auxiliary valve shall be installed on each hydrant run at the tee. A valve shall be installed on a water main at each end of an easement for the main. Additional valves may be required for area isolation and unidirectional flushing. Valves on water mains shall, where practical, be located within paved area of the street. A valve box or chamber shall be provided for every valve.

Generally valve sizes shall be the same as the water main. All valves 12 inch and smaller shall be resilient seated gate valves unless minimum cover cannot be achieved. If minimum cover cannot be achieved a butterfly valve shall be installed. All valves larger than 12 inch shall be butterfly type if approved by the City Engineer. When butterfly valves are installed, the operation nuts shall be on the north or east sides of the water mains. If a valve is installed in gravel or unpaved area, a concrete pad shall be set around each valve box at finished grade per Standard Plan 512.

5-1.18 FIRE HYDRANTS

Fire hydrants shall be installed for buildings where water is served by the City. The final number of hydrants and their locations shall be approved by the City Fire Marshal.

The maximum spacing of fire hydrants serving single-family dwellings or duplex dwellings on individual lots shall be 600 feet and not more than 300 feet from the front property line of the main body of a lot. Required distances shall be measured along the normal fire department hose laying route.

Fire hydrants serving multi-family and commercial lots shall be located not more than 300 feet on center and shall be located so that at least one hydrant is located within 150 feet of all structures or uses. Fire hydrants shall not be closer than 50 feet from multi-family or commercial buildings. On arterial streets without residential access, maximum hydrant spacing shall be 600 feet.

Any hydrant run exceeding 50 feet in length shall be 8 inches in diameter unless otherwise approved by the City Engineer. The joints of hydrant runs shall be restrained. No domestic or fire sprinkler service shall be tapped on any hydrant run. Fire hydrants shall be installed at the ends of each dead end line more than 300 feet in length. Said fire hydrants may be removed to conform to standard spacing requirements when the main is again extended with the City's approval. Hydrant valves shall be no more than one pipe length and no closer than three feet from the hydrant it serves. Fire hydrant installation is shown in Standard Plan 507.

5-1.19 COMBINATION AIR VACUUM RELIEF VALVES

Combination air vacuum relief valves as shown in Standard Plan 511 shall be installed on high points of new water mains where the elevation difference between the high point and the next low point exceeds one (1) pipe diameter, or as otherwise required by the City Engineer. The air valves shall be located outside the traveled portion of the roadway, preferably behind the curb or sidewalk and within the public right-of-way and the public utility easement. If possible, the water main profile shall be adjusted to eliminate the use of the air valves.

5-1.20 BLOW-OFFS

Each dead-end main shall be provided with a fire hydrant if flow and pressure are sufficient or with an approved flushing hydrant or a blow-off assembly shown in Standard Plan 510 for flushing purposes. Flushing devices shall be sized to provide flows that will give a velocity of at least 2.5 feet per second in the water main being flushed. No flushing device shall be directly connected to any sewer. Blow-off assemblies shall be located outside the traveled portion of the roadway, behind the curb or sidewalk, and within the public right-of-way or public utility easement.

A 2 inch blow-off assembly shall be required for testing and disinfection of new water mains, where hydrants are not available.

5-1.21 CONNECTION TO EXISTING SYSTEM

Connections to existing water mains 8 inches and larger shall be via a wet tap shown in Standard Plan 513 unless cut-in is required by the City Engineer in order to install additional valves. Connections to existing water mains smaller than 8 inch diameter shall be made by cutting in a tee, unless otherwise approved by the City Engineer.

5-1.22 EASEMENTS

All water mains, valves, fire hydrants, meters, sampling stations, air vacuum relief valves, blow-offs, and other appurtenances not in public right-of-way shall be within public utility easements designated on submitted plans to provide the City with permanent access to these mains and appurtenances, as well as easements for future line connections, as required. Unless otherwise approved by the City Engineer, the easement for the water mains shall be 20 feet in width and 10 feet on all sides of the pipe lines, a minimum of 5 feet on each side of fire hydrants and other appurtenances (such as meters, sampling stations, air/vacuum valves, blow-offs, etc). Easements shall be fully executed and recorded with the Snohomish County Auditor prior to project acceptance by the City. The form of easement documents shall be subject to the approval of the City Attorney. Easement drawings and legal description shall be included as exhibits to each easement.

If off-site easements are required on properties not owned by the party responsible for the project, the responsible party shall acquire the easements at his/her expense before construction plans will approved by the City Engineer.

5-1.23 SERVICE CONNECTIONS

Service connections, including saddle, service line, meter box and appurtenances, shall be installed as part of the construction of all new water system extensions. A fire sprinkler meter per City Standards shall be provided if required by the City Fire Marshal. All service connections must be protected by a Washington State Department of Health (DOH) approved backflow prevention assembly in accordance with WAC 246-290-490. See Standard Plans 501 through 504a.

For residential developments, meter boxes shall be located in front of the lot to be served unless otherwise approved by the City Engineer. Meter boxes shall be close to the property line, behind the sidewalk, in a landscape area within public right-of-way or public utility easement, but not in paved areas such as sidewalk or driveway. An easement shall be established and recorded for placement of the water service if the required location is outside of the right-of-way or established public utility easement. Meters for two neighboring lots shall be installed near the common lot line to ease meter reading. Meters located close to driveways shall use boxes with traffic rating. The distance from the water main to the meter box shall not exceed 50 feet unless otherwise approved by the City Engineer. Meters shall be located in or as close to the public right-of-way as possible. Service lines shall be perpendicular to the water main if possible. See Standard Plan 502 for single family services and 503 and 504 for multi-family or commercial.

For commercial and multi-family developments, meters shall be located behind the back of a curb or sidewalk and not behind parking space or other obstructions. Meters shall be located for ease of reading.

Minimum allowable service lines from mains to meters shall be 1 inch for a single family residential buildings and 2 inch for multi-family or commercial buildings. All duplexes,

triplexes and fourplexes must have separate services and meters for each unit. Multifamily buildings with five or more units must have separate services and meters for each building. Irrigation and fire sprinkler systems shall also be served by separate services and meters unless otherwise approved by the City Engineer. Each irrigation and fire sprinkler system shall be equipped with an approved backflow preventer. A minimum pressure of 30 psi at the meter shall be maintained when service is flowing at anticipated maximum flow rates. If friction losses will cause the pressure at the building to drop below the minimum, the service line size shall be increased to an appropriate size to maintain the required minimum pressure level.

The standard meter size is ⅝ inch x ¾ inch for a single family residential house. Non-residential services and meter sizes (minimum ⅝ inch x ¾ inch) shall be determined by the engineer or architect per the current Uniform Plumbing Code and subject to the approval of the City Building Official, and the plans shall show the locations and sizes of the services and meters.

Static service pressures at ground floor elevation shall be determined at all lots/buildings to ensure compliance with system pressure standards. Plans shall identify lots/buildings where the builder/owner will be required to install individual pressure reducing valve (PRV) when service pressures exceed 60 psi. A PRV shall be located on the customer side of the meter, outside of the public right of way, and a minimum of 3 feet from the water meter box.

5-1.24 EXISTING WATER MAIN ABANDONMENT

Existing water mains that are out of service shall be removed or abandoned as required by the City Engineer. If water mains are to be abandoned, the ends of the abandoned water mains shall be plugged by filling with Class 2500 concrete for a minimum length of 12 inches.

5-1.25 PRESSURE REDUCING STATIONS

If the proposed water project shall be located in two or more pressure zones, pressure reducing stations may be required by the City Engineer.

5-1.26 CROSS-CONNECTION CONTROL

The City strictly prohibits interconnection of other water supplies with the City's water system.

A backflow prevention device is required for all irrigation systems, fire sprinkler systems, commercial service connections and other water uses which may cause contamination of the City water system. Approved backflow prevention assemblies shall meet the requirements of the WAC 246-290-490 ("Cross Connection Control Regulation in Washington State"), and the recommendations of the PNWS-AWWA Cross Connection Control Manual and the City of Snohomish Cross Connection Control Program. The

types of backflow prevention devices to be used for a specific project shall be determined by the City's Cross Connection Specialist.

Fire sprinkler system connections to the City's water system shall be owned and maintained by the property owner, beginning immediately at the valve where the fire sprinkler system connects to the City's water main.

The backflow prevention assembly on fire sprinkler system connections shall be located as close to the serving water main as possible, either on the owner's property or in an easement.

5-1.27 PRIVATE WELLS

To receive water services from the City, the property owner shall either:

(1) decommission the existing well(s) in accordance with WAC 173-160-381. The owner shall provide a copy of the decommission report to the City Water Division.

OR

(2) physically separate all domestic water supply piping from well(s) piping and permanently cap all terminations so that the two systems cannot be intertied. A double check valve (DCV) assembly shall be installed at the water meter and annual DCV testing requirements shall be meet and reports submitted to the City's Water Division.

However, if the property is undergoing development such as, but not limited to, platting or subdivision, all well(s) shall be decommissioned in accordance with WAC 173-160-381 and a copy of the decommission report shall be delivered to the City Water Division.

5-2 MATERIAL

5-2.01 GENERAL

All materials shall be new and undamaged. The same manufacturer of each item shall be used throughout the work. All materials not specifically referenced shall comply with applicable sections of ANSI, ASTM, AWWA or the current WSDOT/APWA Standard Specifications and approved by the City Engineer.

When specific manufacturers or models are specified in these Standards, no substitutions will be allowed without prior approval by the City Engineer. If required by the City Engineer, the Contractor shall furnish certification from the manufacturer of the materials being supplied that the inspection and all of the specified tests have been made and the results thereof comply with the requirements of the reference Standards.

5-2.02 DUCTILE IRON PIPE

Ductile iron pipe shall be Class 52 (Tyton joint only) and cement mortar lined unless otherwise specified and shall conform to AWWA/ANSI C151/A21.51. Standard thickness of cement mortar lining shall be in accordance with AWWA/ANSI C104/A21.4.

5-2.03 HIGH DENSITY POLYETHYLENE PIPE

High density polyethylene (HDPE) pipe shall conform to AWWA C900.

5-2.04 WATER SERVICE PIPE

Water service pipe shall be Driscopipe IPS Class 200 Hi-Mol Poly pipe for 1-inch and Driscopipe CTS Class 200 Hi-Mol Poly pipe for 2-inch Driscopipe shall conform to ASTM D-27370SDR9 (PE3408). CTS 110 SS liners for polypipe shall be used.

5-2.05 FITTINGS AND JOINTS

All fittings for ductile iron pipe shall be ductile iron compact (short body) fittings conforming to AWWA/ANSI C153/A21.53 and shall be cement mortar lined conforming to AWWA/ANSI C104/A21.4.

Joints shall be mechanical joints or rubber gasket push-on joints (Tyton joints only) unless flanged joints or restrained joints are required and shown in the plans. Pipe with push-on joints shall be suitable for use with mechanical joint fittings. Ductile iron pipe fittings shall have a pressure rating of 350 psi for push-on or mechanical joint fittings and 250 psi for flange joint fittings drilled in accordance with AWWA/ANSI C111/A21.11, unless otherwise noted.

Rubber gaskets for push-on joints (Tyton joint only) or mechanical joints shall be in accordance with AWWA/ANSI C111/A21.11. Gasket material for flanges shall be neoprene, Buna N, chlorinated butyl, or cloth inserted rubber.

All Mechanical Joints, unless otherwise noted on the plans, shall use Mechanical thrust restraint followers. Mechanical thrust restraint shall be EBAA Iron MegaLug/Flange, Romac, RomaGrip, Star Pipe StarGrip, or as approved by the City Engineer.

5-2.06 RESTRAINED JOINTS

Where restrained joints are required, they shall be either bolted or boltless design, flexible after assembly, and can be disassembled without special tools. Any device utilizing round point set screws shall not be permitted. All couplings installed underground to connect ductile iron pipe shall be manufactured of ductile iron.

Restrained joints shall be Meg-A-Lug Series 1100, TR Flex, Grinnell 595 shackle clamp, or approved equal.

5-2.07 COUPLINGS

Flexible coupling and transition coupling cast components shall be ductile iron. Bolts and nuts shall be in accordance with ASTM A536-80, Grade 65-45-12. Bolts shall be high strength, low alloy steel track head bolts with national course rolled thread and heavy hex nuts. Gaskets shall meet AWWA/ANSI C111/A21.11 composition specifications.

5-2.08 BOLTS AND NUTS

Bolts, nuts and washers used for securing fittings shall be of similar materials. Steel bolts shall meet the requirements of ASTM A307 or ASTM F568 for carbon steel or ASTM F593 or ASTM F738 for stainless steel. Nuts shall meet the requirements of ASTM A563 for carbon steel or ASTM F594 or ASTM F836 for stainless steel. Iron bolts and nuts shall meet the requirements of ASTM A536, grade 65-45-12.

5-2.09 GATE VALVES

All gate valves shall be resilient seated gated valves conforming to the latest revision of AWWA C509 or C515. All gate valves shall be epoxy coated and turn counter clockwise to open. All gate valves shall have ANSI flanges or mechanical joints ends.

Buried gate valves shall be non-rising stem suitable for installation with the type and class of pipe being installed. Operating stems shall be equipped with standard 2 inch operation nut, and O-ring stem seals.

5-2.10 BUTTERFLY VALVES

Butterfly valves shall be used only when adequate cover cannot be achieved with gate valves.

5-2.11 VALVE MARKER POSTS

Valve marker posts shall be Carsonite blue plastic markers and labeled “WATER” or approved equal. See Standard Plan 509.

5-2.12 VALVE BOXES

Valve boxes shall be installed on all buried valves. Ears (lugs) on all valve boxes shall be placed in alignment with the pipe. The box and lid shall be cast iron, two piece slip type. The cover shall have the word “WATER” cast in the upper surface. Valve boxes, lids and

extensions shall be East Jordan Ironworks deep style lid. All castings shall be coated with asphaltic varnish.

A valve operating nut extension shall be furnished and installed on all valves where the finished grade is more than 36 inches above the valve operating nut. Extensions are to be a minimum of 12 inches long with only one extension per valve. The operating nut extension shall extend into the top section of the valve box. See Standard Plan 512.

5-2.13 VALVE VAULTS

Valve vaults shall be dimensioned and sized for valve removal and replacement. Vaults shall be furnished in pre-cast concrete sections with sufficient strength to withstand H-20 traffic loading together with access frames and covers.

5-2.14 COMBINATION AIR VALVES

Combination air valves shall be designed to operate with potable water under pressure to permit discharging a surge of air from an empty line when filling and relieve the vacuum when draining the system. The air valves shall also release an accumulation of air when the system is under pressure. This shall be accomplished in a single valve body designed to withstand a pressure of 300 psi.

The body and cover shall be cast iron conforming to ASTM A48, Class 30. Floats shall be stainless steel conforming to ASTM A 240 and designed to withstand 1,000 psi. Seats shall be Buna N rubber. Internal parts shall be stainless steel or bronze. Combination air valves shall conform to AWWA C512. Air valves shall have double concrete meter boxes Fogtite 2T with solid steel lid. See City of Snohomish Standard Plan 511.

5-2.15 BLOW-OFF ASSEMBLIES

Blow-off assemblies shall be as shown in Standard Plan 510.

5-2.16 FIRE HYDRANTS

Fire hydrants shall be 5 ¼ inch MVO "Traffic Model" type with approved breakaway features, and meet or exceed the requirements of AWWA C502 as well as have the following:

1) A standard 4 inch Storz pumper port and two 2½ inch NST side ports, all opening by turning counter clockwise with 1½ inch operating nut;

2) Be painted with two coats of hi-gloss equipment yellow enamel paint, with the distance from the foot valve stenciled on the hydrant; and

3) A 4 inch Storz adaptor.

Fire hydrants shall be M&H 929 Reliant, Mueller Super Centurion 250 only or East Jordan Iron Model No. 5CD250 with operating cap.

5-2.17 TAPPING SLEEVES

Tapping sleeves shall be used in lieu of cut-in tees except as otherwise approved by the City Engineer. Tapping sleeve valves shall be epoxy coated and resilient seat. Acceptable sleeves include:

Pipe Material	**Type of Tapping Sleeve**
Ductile Iron or Cast Iron Pipe	Epoxy Coated Fabricated Steel
Asbestos Cement	Fabricated Stainless Steel Full

5-2.18 SADDLES AND CORPORATION STOPS

Service saddles shall be ROMAC, Ford or equal and shall have stainless steel double straps. See Standard Plans 501 and 502.

Corporation stops shall be the ball valve type and shall be Ford or Mueller. Corporation stops for use with the saddle shall be of bronze in accordance with AWWA Standard C800 with AWWA IP inlet by compression outlet.

5-2.19 METER BOXES

Meter boxes used for meters, sampling stations, and blow-offs shall be high density polyethylene meter boxes with solid ductile iron hatches with 2 inch touch read hole manufactured by Raven, Inc., or an equivalent as approved by the City Engineer. Refer to Standard Plans 501 and 502 for sizes and part numbers.

5-2.20 PRESSURE REDUCING STATION

Any pressure reducing station for a specific project shall be designed by a qualified, licensed engineer. The design shall be submitted to the City Engineer for approval. A typical pressure reducing station is shown in Standard Plan 520.

5-2.21 REDUCED PRESSURE BACKFLOW ASSEMBLY

All reduced pressure backflow assemblies (RPBA) shall be as listed on the most current edition of the "Approved Backflow Prevention Assemblies" published by DOH. The assembly shall include a tightly closing resilient seated shut-off valve on each end of the body and each assembly shall be fitted with four properly located resilient seated test cocks. The RPBA shall be installed in an above ground enclosure. The enclosure shall be

Hot Box, or an equivalent as approved by the City Engineer. See Standard Plans 517 through 519.

5-2.22 DOUBLE CHECK VALVE ASSEMBLY

All double check valve assemblies (DCVA) shall be as listed on the most current edition of the "Washington State Approved Backflow Prevention Assemblies" published by DOH. The assembly shall include a tightly closing resilient seated shut-off valve on each end of the body and each assembly shall be fitted with four properly located resilient seated test cocks. See Standard Plans 514 through 516.

5-2.23 STEEL CASING

Steel casing shall be black steel pipe conforming to ASTM A53. Casing shall be as specified in Standard Plan 522.

5-2.24 CASING SPACERS AND END SEALS

Casing spacers and end seals shall be sized for pipe being installed and shall be manufactured by Advance Products & Systems, Cascade Waterworks, Pipeline Seal and Insulators Co., or an equivalent as approved by the City Engineer. See Standard Plan 522.

5-2.25 CONCRETE

Thrust blocking, encasement, or slope anchor concrete shall be mixed from materials acceptable to the City Engineer and shall have a 30-day compressive strength of not less than 2,500 psi. See Standard Plans 505 and 506.

The mix shall contain five (5) sacks of cement per cubic yard and shall be of such consistency that the slump is between 1 inch and 5 inches. All concrete shall be mechanically mixed.

5-2.26 BEDDING MATERIAL

Bedding material shall be as specified in Section 9-03.12(3) “Gravel backfill for Pipe Zone Bedding” of the WSDOT/APWA Standard Specifications. See Standard Plan 524.

5-2.27 CONTROLLED DENSITY FILL

Controlled Density Fill (CDF) shall be a mixture of Portland Cement, admixture (optional), FlyAsh, aggregates and water. It shall be proportioned to provide a grouty, non-segregating, free flowing, self-consolidating and excavatable material that will result in a non-settling fill which has measurable unconfined compressive strength.

Controlled Density Fill (CDF) shall conform to the requirements of Section 2-09.3(1)E of the current WSDOT/APWA Standard Specifications.

5-3 CONSTRUCTION STANDARDS

5-3.01 GENERAL REQUIREMENTS

All work shall be constructed as shown in the plans and in accordance with these Standards.

Materials shall be installed in compliance with the manufacturer's instructions and specifications, except where a higher quality of workmanship is required by the plans and these Standards.

All work shall also be done in accordance with all applicable Federal, State, County and other local laws and regulations. The Contractor shall arrange for inspection by these agencies and shall submit evidence of their approval if requested by the City.

5-3.02 MATERIAL SUBMITTALS

Material submittals shall be submitted to the City Engineer for approval after the plans are approved for construction.

Five (5) sets of material submittals are required. The City Engineer shall either approve or otherwise indicate the reasons for disapproval. Disapproved submittals shall be resubmitted to the City Engineer for approval.

The City Engineer's review of material submittals covers only general conformance to the plans and these Standards and not for quantity determination. No material quantities shall be verified by the City.

Each "Material Submittal" section shall follow a cover page and state the category of the materials that are submitted for review. Each submittal must have the specific part number(s) checked or highlighted along with its specific purpose. The following shows the preferred order to list the material categories:

1) Pipe, Fittings, Pipe Restraints and Casing.

2) Valves (Gate Valves, Air Valves, Blow-off, and Valve Boxes).

3) Hydrants and Attachments.

4) Service Fittings, Service Pipe, Saddles, Ball Valves, Corps, Sleeves, etc.

5) Boxes for Meters, Sampling Stations, Blow-offs, and Air Valve Assemblies.

6) Cross Connection Control Assemblies (DCDA, RPBA, RPDA, DCVA).

7) Bedding Material with Sieve Analysis.

8) Other items if required.

5-3.03 PRE-CONSTRUCTION CONFERENCE

The party responsible for the project shall contact the City Public Works Inspector (360-282-3193) to schedule a pre-construction conference after the plans, material submittals, grading, and right-of-way permits are approved. The conference shall include the party responsible for the project, design engineer, and contractor, representatives from the permit agencies, other utility companies, and City staff. An on-site tailgate meeting between the contractor and the Public Works Inspector shall be arranged by the contractor at least 48 hours prior to commencing construction.

5-3.04 CONSTRUCTION SCHEDULE

The party responsible for the work shall provide the City Engineer with the water system extension construction schedule a minimum of five (5) business days prior to start of water system extension construction to arrange staking inspection and to give permitting agencies and customers two (2) business days notice.

5-3.05 EASEMENT

Prior to start of water system extension construction, the party responsible for the project must acquire and record all necessary public utility easements for construction, installation and maintenance of any water mains or facilities not located in the public right-of-way, in City easements, or on the party's property.

5-3.06 PERMITS

The party responsible for a water system extension project shall obtain at their expense all necessary permits from the City and other appropriate State and Local agencies and entities. The party responsible for the work shall provide the traffic control plan prepared by themselves, the design engineer or the Contractor.

5-3.07 HANDLING OF PIPE

All types of pipe shall be handled in a manner that prevents damage to the pipe, pipe lining or coating. Pipe shall be bagged or plugged from the manufacturer or supplier before unloading at the site. Pipe and fittings shall be loaded and unloaded using forks or cable choker in a manner to avoid shock or damage, and under no circumstances shall they be dropped, skidded, or rolled against other pipe. Damaged pipe will be rejected, and all damaged pipe shall be placed apart from the undamaged pipe and shall be removed from the project site within 24 hours.

Pipe shall be stacked in such a manner as to prevent damage to the pipe, to prevent dirt and debris from entering the pipe, and to prevent any movement of the pipe. The bottom tiers of the stack shall be kept off the ground on timbers, rails or other similar supports. Pipe on succeeding tiers shall be alternated by bell and plain end. Timbers of 4"×4" dimensions shall be placed between tiers and chocks shall be placed at each end to prevent movement. Each size of pipe shall be stacked separately.

Threaded pipe ends shall be protected by couplings or other means until the pipe is installed. Dirt or other foreign material shall be prevented from entering the pipe or pipe joints during handling and installation. When pipe installation is not in progress, the open ends of the pipe shall be closed by a watertight plug or by other means approved by the City Engineer to ensure cleanliness inside the pipe.

5-3.08 STAKING

Staking shall be performed by or under the direct supervision of a land surveyor licensed in the State of Washington. Two (2) business days notice shall be provided to the City to inspect construction staking before construction begins.

Staking shall be placed in 50 foot intervals and at all fittings on base line or edge of easement with stationing, hub elevations, and cuts to top of pipe.

5-3.09 DEVIATION FROM PLANS

No deviations from the approved plans and these Standards shall be allowed without the City Engineer's approval. Minor changes may be approved by the Public Works Inspector. If major changes are required, the design engineer shall revise, sign and submit the plans for the City Engineer's approval prior to restart of construction.

5-3.10 INSPECTION AND TESTING

The City Public Works Inspector shall have access to the project site for the purpose of inspections and testing at all times. Proper facilities shall be provided for such access, inspection, and testing.

If any work is covered without approval or consent of the City Public Works Inspector, it must be uncovered for inspection if required by the City Public Works Inspector.

Before a pressure test is to be observed by the City Public Works Inspector, the party responsible for the work shall make whatever preliminary tests are necessary to ensure that the material and/or equipment are in accordance with the plans and these Standards. Written and/or verbal notices of deficiency shall be given to the party responsible for the work. The party responsible for the work shall correct such deficiencies before final project inspection by the City Public Works Inspector.

5-3.11 WATER QUALITY

Water pollution controls shall be implemented and maintained until the project is accepted by the City Engineer. The party responsible for the project shall familiarize themselves with the requirements of DOE and other regulatory agencies having jurisdiction over such matters.

Water with chlorine residual must be dechlorinated using City-approved means and discharged into the environment when the water has zero chlorine residual. Dechlorinated water discharged into the environment shall be done so without causing erosion or impact to the environment. With the approval of the City Engineer, water with chlorine residual may be discharged into the City's sanitary sewer system. Water containing chlorine residual shall not be discharged into the storm drainage system or any waterway.

The oil and chemical storage site for the project shall be approved by the City Engineer and the area shall be diked. There shall be no disposal of waste oil or oil products on the project site. A waste oil disposal tank shall be provided if deemed necessary by the City Engineer.

5-3.12 CONSTRUCTION ON EXISTING EASEMENTS

All work within public utility easements shall be performed in accordance with terms and conditions of the respective easement. Each easement area shall be restored to equal to or better than the condition of the easement area that existed prior to the work. Work shall not be performed within any public utility easement area unless such work is specifically authorized by the City Engineer. The party responsible for the project shall provide advance written notification to and shall coordinate the authorized work with the persons and/or entities owning property that is adjacent to the easement area.

5-3.13 PRE-CONSTRUCTION PHOTOS

Prior to commencement of work, photographs shall be provided to the City that clearly show the conditions of the project site immediately before the anticipated start of the work. Photographs will be obtained as follows:

1) 50 foot interval in easements up station and down station.

2) Any other locations as directed by the Public Works Inspector.

The photographs shall be 4 inch x 6 inch, color prints, contained in albums, catalogued, and cross-referenced. A digital copy of each photograph shall also be provided to the City Engineer.

5-3.14 UNDERGROUND UTILITIES

The plans shall show the approximate locations of various existing utilities known to the design engineer such as gas lines, water mains, storm drainage, power lines, telephone lines, TV cables, fiber optics, and other obstructions based on information obtained from

various sources. The party responsible for the project is responsible to check for interferences and obstructions by inquiry from the different utilities and by underground exploration before commencing excavation.
The party responsible for the project shall request field locating and notify the owners of underground utilities about the scheduled commencement of excavation through the one-call system (1-800-424-5555).

Notice shall be made to owners of underground utilities not less than two (2) business days or more than ten (10) business days prior to scheduled date of commencement of excavation. Test pits, for the purpose of locating underground utilities or structures in advance of the construction, shall be excavated and backfilled. Test pits shall be backfilled immediately after their purpose has been satisfied and the surface restored and maintained in a manner satisfactory to the City or applicable agency.

Excavation around and under active utilities shall be performed with special care to ensure that utility service is not interrupted. Where it is necessary to cut, move or reconnect any service lines, arrangements shall be made with the respective utility owners.

The party responsible for the work shall coordinate with all utility owners and arrange for the movement or adjustment, either temporary or permanent, of utility conflicts and shall also notify the City, in advance, of any conflicts affecting the work.

The party responsible for the project shall indemnify, defend and hold harmless the City from any claim for damage of utilities and/or disruption to services resulting from project activities/operations. The form of the indemnification agreement shall be subject to the approval of the City Attorney.

5-3.15 TRENCH EXCAVATION

Trench excavation and backfill operations within State right-of-way: All excavation and backfill within the State right-of-way shall adhere to the current *WSDOT/APWA Standard Specifications*.

Trench excavation and backfill operations within County right-of-way: Excavation within Snohomish County right-of-way shall conform first to Snohomish County Road Standards, and secondly to current *WSDOT/APWA Standard Specifications*.

Trench excavation and backfill operations within City right-of-way: Excavation within the City right-of-way shall conform to these Standards.

Clearing and grubbing limits may be established by the City or governing agency for certain areas. Debris resulting from the clearing and grubbing shall be appropriately disposed of in accordance with applicable local and State laws and regulations.

Trenches shall be excavated to the line and grade shown in the plans or as designated by the City Engineer. Higher pressure class pipe or special bedding may be required because of excess trench width.
Unsuitable material below the depth of the bedding shall be removed to the extent approved by the Public Works Inspector and replaced with materials approved by the City Engineer.

The length of trench excavation in advance of pipe laying shall be kept to a minimum and shall not exceed more than 100 feet without prior written approval of the City Engineer.

When trenching operations take place in public right-of-way, the pavement and all other improvements shall be restored as required by the right-of-way permit.

When excavation of rock is encountered, all rock shall be removed to provide a clearance below and on each side of all pipe, valves, and fittings of at least 6 inches for pipe sizes 24 inches or smaller and 9 inches for pipe sizes 30 inches and larger. Material removed shall be replaced with appropriate backfill material, which shall be compacted to 95% standard proctor. See Standard Plan 525.

5-3.16 SHEETING AND SHORING

Sheeting and shoring shall be provided and installed as necessary to protect workers, the work and existing utilities and other properties in compliance with OSHA and WISHA requirements. All sheeting and shoring above the pipe shall be removed prior to backfilling. Sheeting below the top of the pipe may be cut off and left in place. Removal of the sheeting and shoring shall be accomplished in such a manner that there will be no damage to the work or to other properties.

5-3.17 TRENCH DEWATERING

Sufficient pumping equipment shall be provided and maintained on the project site to keep the trench free from standing water. Surface runoff shall not be allowed to flow into the trench. The trench water or other deleterious materials shall not be allowed to enter the pipe at any time. If water is found to be entering the new water main at any time, the water main shall be plugged and all work shall stop until the trench water is completely pumped out or otherwise controlled to the satisfaction of the Public Works Inspector. Any dewatering method used shall be in accordance with the specifications and requirements of the City and DOE.

5-3.18 PIPE BEDDING

Bedding material, when required by the City, shall be as specified in these Standards and the current WSDOT/APWA Standard Specifications. For the type of pipe (rigid or flexible) to be installed, pipe zone bedding is defined as 6 inches below the pipe and around the pipe, and 12 inches above the pipe. Native material may be used for bedding of ductile iron pipe if judged to be suitable by the City Engineer. Gravel backfill for pipe

zone bedding shall be select granular material free from wood waste, organic material, and other extraneous or objectionable materials and shall have a maximum dimension of 1½ inches. Pipe zone bedding up to 12 inches over the top of the pipe shall be evenly and carefully placed. Gravel backfill for pipe zone bedding shall be compacted to 95% maximum dry density per ASTM D1557 by approved methods (hand-held tools), so as to provide firm and uniform support for the full length of the pipe, valves, and fittings. See Standard Plan 524.

5-3.19 CONCRETE THRUST BLOCKING

Bends, tees, plugs, reducers, and caps, unless otherwise specified, shall be blocked in accordance with Standard Plans 505 and 506. All poured in place blocking shall have a minimum measurement of twelve inches 12 inches between the pipe and the undisturbed soil. Concrete blocking shall have a minimum of ¼ square foot bearing against the fitting. Blocking shall be adequate to withstand full test pressure as well as to continuously withstand operating pressure under all conditions of service. All concrete shall be 2,500 psi minimum and mechanically mixed.

Blocking shall, unless otherwise shown or directed, be placed so that pipe and fittings will be accessible for repair. Eight-mil polyethylene sheets shall be installed around all fittings and all bolts, nuts, and glands for future dismantling.

In the event of a shut down where time does not permit the proper setting of the concrete blocking, ecology blocks shall be installed with concrete poured around the connection point of the fitting and the blocks with the approval of the Public Works Inspector.

5-3.20 TRENCH BACKFILL AND COMPACTION

Trench backfill shall be done above the pipe zone bedding. All backfill material shall be free from cinders, ashes, refuse, vegetable or organic material, boulders, rocks or stones, frozen soil, or other unsuitable material. Trench backfill material shall be as specified in these Standards and the current WSDOT/APWA Standard Specifications.

Compaction of the backfill shall, at the minimum, be accomplished by mechanical tamper, by vibrating, by rolling, or by a combination of these methods, as approved by the City Engineer. Water settling is not permitted. A testing laboratory acceptable to the City Engineer shall be used to perform on-site density tests to show that the specified density has been obtained. The approval of the compaction method and the achievement of the specified density shall in no way relieve the Contractor of responsibility for all repairs caused by settlement of the backfill prior to acceptance and during the two year period after acceptance of the project.

Prior to backfilling, form lumber and debris shall be removed from the trench. Sheeting used by the Contractor shall be removed just ahead of the backfilling. Backfill shall not be deposited in the trench in any manner which will damage or disturb the pipe or the initial backfill. Care shall be taken to prevent any damage to the pipe or its protective

coating. After the initial backfill is placed, the remaining backfill material shall be placed in successive layers not exceeding 1 foot, (12 inches) in loose thickness, and each layer shall be compacted to the density specified below.
Asphalt pavement restoration shall either be by a patch or overlay method as required and noted on the right-of-way permit. When a patch method is used, the trench limits shall be saw cut prior to the final patch.

All pavement cuts shall be made by saw cuts. The saw cuts shall be 1 foot outside the trench width. If the permit requires an overlay, then the Contractor may use a jackhammer for cutting the existing pavement. Grinding may be required.

All trenching shall be backfilled in accordance with Standard Plan 525. All trench backfill materials shall be compacted to 95% maximum dry density, as determined by ASTM D1557.

Native material may be used for backfill upon approval of the City Engineer or the permitting agency.

When the trench is perpendicular to the traveled lane or any driveways, the full depth shall be backfilled with crushed surfacing top course material. When the trench is parallel, the top 4 feet must be backfilled with crushed surfacing top course material. Controlled Density Fill (CDF) may be required by the City Engineer or the permitting agency.

Backfill compaction shall be performed in 8 to 12 inch lifts. Compaction tests shall be performed in 4 foot vertical increments maximum. The test results shall be given to the City Engineer for review and approval prior to paving. Tests shall be performed at maximum intervals of 50 feet along the length of the trench.

If the area is unpaved and not subject to vehicle traffic, the backfill shall be compacted to a minimum of 90% of maximum dry density as determined by ASTM D-1557.

5-3.21 LAYING DUCTILE IRON PIPE

Work shall be accomplished in accordance with AWWA C600 and the manufacturer's recommendations.

Depths of pipe shall conform to approved plans. The typical cover depth of pipe is 36 inches measured from finished grade to top of pipe.

The bottom of the trench shall be finished to grade in such a manner that the pipe will have bearing along the entire length of the barrel. Bolts on mechanical pipe and fittings shall be tightened uniformly with a "Torque" wrench which measures the torque for mechanical joints as follows:

2 inch to 3 inch pipe sizes ⅝ inch bolts 40 to 60 foot pounds

4 inch to 24 inch pipe size ¾ inch bolts 60 to 90 foot pounds

Except where restrained joint systems are required, mechanical or push-on Tyton joints shall be used. Installation of push-on joint pipe shall be in accordance with the manufacturer's instructions.

When it is necessary to deflect pipe from a straight line in either the horizontal or the vertical plane, the amount of joint deflection shall not exceed one half (½) of the maximum deflection recommended by the pipe manufacturer. The pipe manufacturer's joint deflection recommendations shall be provided to the City Engineer prior to pipe installation as a part of the Material Submittals.

Where field conditions require deflection or curves not anticipated on the plans, an appropriate plan revision shall be prepared by the design engineer and submitted to the City Engineer for approval.

Whenever it becomes necessary to cut a length of pipe, the cut shall be done in conformance with all safety recommendations of the cutting equipment manufacturer. Cutting shall be done in a safe manner without creating damage to the pipe or cement mortar lining. The cut shall be made by an abrasive pipe saw or an approved pipe cutter.

The outside of slip joint pipes shall be beveled and smoothed so that good connections can be made without gasket damage.

All parts of the pipe ends, couplings, fittings and appurtenances shall be cleaned to remove oil, grit, or other foreign matters from the joints. Care shall be taken to keep the joints from contacting the soil.

5-3.22 POLYETHYLENE ENCASEMENT

Installation of polyethylene encasement shall be in accordance with the latest AWWA Standard C105. All ductile iron pipe and fittings installed in highly corrosive soils shall be wrapped in a manner approved by the City Engineer except as specifically excluded in the approved plans or in these Standards.

5-3.23 FIRE HYDRANT INSTALLATION

Fire hydrants shall be set as shown in Standard Plans 507 through 509 and AWWA Standard C600. The portion of the hydrants above the ground shall be painted with 2 coats of high gloss equipment yellow paint. The entire hydrant run shall be restrained joint.

All hydrants shall stand plumb and shall have their nozzles parallel with or at right angles to the curb or at the City Fire Marshall's discretion, with pumper nozzle facing the curb.

Hydrants shall be set to the established grade. Hydrants shall be installed so that the breakaway flange is 2" to 6" inches above finished grade per Standard Plan 507.

When a dry barrel hydrant is set, drainage shall be provided at the base of the hydrant by placing 1½ inches of washed drain rock from the bottom of the trench to at least 12 inches above the drain port opening in the hydrant and to a distance of 2 foot around the elbow. Fire hydrants shall not be located within 10 feet horizontally of a sanitary sewer main or side sewer.

When a hydrant is installed in an unpaved area, a concrete fire hydrant pad shall be installer per Standard Plan 508.

Additional information regarding placement of hydrants can be found in AWWA Manual M17.

When fire hydrants are located in parking lots, or other areas where vehicles may have access, hydrant guard posts shall be installed. Guard posts shall be installed according to the minimum dimensions shown in Standard Plan 509.

Reflective pavement markers for fire hydrants are required.

5-3.24 VALVE INSTALLATION

Prior to installation, valves shall be inspected for approved part/manufacturer's numbers; cleanliness of valve ports, especially seating surfaces, handling damage, and cracks.

When butterfly valves are installed, the operation nuts must be on the north or east sides of the water mains or as directed by the Public Works Inspector.

The valve and valve box shall be set plumb and centered on the valve. Valves 12 inches or larger shall be supported by a concrete block (16 inches x 16 inches x 4 inch solid concrete) on a sufficiently tamped trench bottom so that the pipe will not be required to support the weight of the valve. In no case shall valves be used to bring misaligned pipe into alignment during installation. Pipe shall be supported in such a manner as to prevent stress on the valve.

Valves shall be installed in the closed position. Where the valve operating nut is more than 3 feet below finished grade, a valve stem extension conforming to the Standard Plans must be installed. See Standard Plan 512. Tapping valves shall be water tested prior to tapping water main.

A valve box or vault shall be provided for every valve. Valve box top sections shall be adjusted flush with the finished pavement, and in those areas to be excavated for future roadway grades, enough adjustment shall be provided in the valve box to allow the top of the box to be adjusted to the required grade.

Backfill around valves shall be carefully tamped in 6 inch lifts for the full depth of the trench with the valve box in place. A minimum of 2 feet x 2 feet x 4 inch concrete pad shall be provided for a single valve box and a minimum of 4 feet x 4 feet x 4 inch concrete pad shall be provided for multiple valve boxes installed in gravel or unpaved areas as shown in Standard Plan 512.

5-3.25 COMBINATION AIR AND VACUUM RELEASE VALVE

Location of the air/vac shown in the plans is approximate. Air valves shall be set at the high points of the water main. The water main profile may need adjustment so that the high point and air/vacuum valve is installed in a convenient location with the Public Works Inspector's approval. Installation shall be as shown in Standard Plan 511.

5-3.26 BLOW-OFF ASSEMBLIES

Blow-off assemblies shall be installed as shown in Standard Plan 510. A valve marker post shall be installed when the gate valve is installed in unpaved area or as directed by the Public Works Inspector.

5-3.27 VALVE MARKER INSTALLATION

Marker posts shall be set for all valves located in unpaved areas and as directed by the City Engineer except auxiliary hydrant valves. Installation shall be as shown in Standard Plan 509.

5-3.28 SERVICE LINES

New Service Installations

Generally, corporation stops are located at ten o'clock or two o'clock positions on the circumference of the pipe. Taps shall be accomplished by saddle tap only. No direct taps shall be allowed. Taps shall be installed with double strap stainless steel saddles. When more than one tap in an existing cast iron pipe is necessary to deliver the required flow, the taps should be staggered around the circumference at least 12 inches apart (not in line). The service line shall be pressure tested before placing in service. Service installation shall be as shown in Standard Plans 501 through 504a.

Reconnecting Existing Services

Service connections shall be installed as shown in the approved plans and Standard Plans. Service lines shall be installed in paved areas by boring, and under sidewalks and curbs by boring and tunneling. Damages shall be repaired by the Contractor. A 30 inch minimum cover shall be provided on service lines. Service lines shall be installed 90 degrees horizontally to the main to intercept the existing meters. Installed service lines shall be flushed prior to connection to the meter.

Angle ball meter valves, check valves and setters (if required) and boxes shall be installed as shown in the Standard Plans or as directed by the City Engineer.

Existing service connections shall not be transferred to the new mainline until the new mainline has been successfully flushed, disinfected, tested and approved by the Public Works Inspector. When transferring services from the existing mainline to the new mainline, sanitary precautions shall be taken as necessary to protect the potable water supply in both the existing and new mains.

5-3.29 PRESSURE REDUCING STATION

Pressure reducing stations shall be installed as shown in Standard Plan 520, in approved plans, and in accordance with the manufacturer's recommendations. The pressure reducing valves, strainers, pressure relief, pipe and fittings shall be constructed in accordance with the applicable AWWA and Uniform Plumbing Code requirements. Pressure reducing valves 6 inches or larger shall be supported by pipe supports. Supports shall be bolted to the vault floor.

Pressure relief discharge pipe shall be placed in a location that will not be subject to damage or erosion during discharge of water. The pressure reducing valve manufacturer's representative(s) shall be present during the start up of the pressure reducing station.

5-3.30 CONNECTION TO EXISTING WATER MAIN

Points of connection to existing water mains shall be exposed prior to trenching of the new mains, and not less than 48 hours prior to the anticipated connection time. Unless specifically provided for elsewhere in these Standards, at least five (5) business days notice shall be given to the City Engineer prior to the anticipated connection time. The City Engineer shall be responsible for notifying the City Fire Chief and customers affected by the shut-off. Water main shut-off shall not be scheduled to take place on Fridays, or on the day before a holiday, unless otherwise approved by the City Engineer.

The party responsible for the project shall ensure that existing fittings are in accordance with the approved plans and that the connection will be made in accordance with the plans. If the connection cannot be made in accordance with the plans, the City Engineer shall be so notified. The design engineer shall submit revised plans that provide appropriate connection to the City Engineer for approval.

Connection to the existing water system shall be done only after the new mains are flushed and have passed pressure and purity tests. All connections to the existing water system must be approved by the City Engineer and shall be made in the presence of the City Public Works Inspector. Only authorized City representatives shall operate the valves in the existing water system.

Connections to existing water system may be made under pressure with a tapping machine by determining the size and type of pipe and installing a tapping tee with a tapping gate valve. Tapping tees shall be installed as shown in Standard Plan 513. Work shall not start until all materials, equipment, and labor are ready. The tapping tee and valve shall be installed in a horizontal position so that the valve stem is vertical. Where cut-ins are required in existing pipes, the work shall be conducted so as to minimize the interruption of service. Necessary pipe, fittings and gate valves shall be assembled at the site ready for installation prior to the shut-off of water in the existing main. Once the water main has been shut off, the work shall be promptly completed and shall not be halted until the water main is back in service.

The interiors of all pipe and fittings, particularly couplings and sleeves, to be used in final cut-in connection shall be swabbed or sprayed with a 200 mg/L hypochlorite solution prior to installation.

Flushing shall start as soon as repairs or connections are completed and shall be continued until discolored water is eliminated. Flushing shall be done in the presence of the City Water Quality Control Specialist.

5-3.31 SCHEDULE OF TESTS

The Public Works Inspector and the Water Quality Control Specialist shall be notified at least two (2) business days before a section of water main is ready for inspection and test. The Water Quality Control Specialist shall inspect and observe the hydrostatic test. The City Engineer shall be notified at least two (2) business days prior to purity test and flushing. The party responsible for the project shall be present at the project site when the Water Quality Control Specialist takes water samples. Sufficient manpower and resources shall be provided by the responsible party to accomplish the work in a timely manner. Flushing shall be done under direct supervision of the Water Quality Control Specialist.

5-3.32 HYDROSTATIC PRESSURE TESTS

Water main, appurtenances and service connections shall be tested in sections of convenient lengths under a hydrostatic pressure of 220 psi for 15 minutes.

Pumps, gauges, plugs, saddles, corporation stops, miscellaneous hose, piping, and measuring equipment necessary for performing the test shall be furnished and operated by the party responsible for the project.

Sections to be tested shall normally be limited to a maximum of 1,000 feet. The pipe shall be backfilled sufficiently to prevent movement of the pipe under pressure. Thrust blocking shall be in place for an adequate time for concrete to cure before testing. Where permanent blocking is not required, temporary blocking shall be furnished and installed prior to and removed after testing.

An approved double check valve assembly (DCVA) shall be provided to fill the new water mains for testing and flushing. The new water mains shall be filled and remain under 200 psi to 210 psi pressure for 24 to 48 hours to allow air to escape and the lining of the pipe to absorb water.

The pressure test shall be accomplished by pumping the main up to 200 psi to 210 psi, stopping the pump for 15 minutes, and pumping the main up to the test pressure again. During the test, the section being tested shall be observed to detect any visible leakage.

A clean container shall be used for holding water for pumping up pressure on the main being tested.

The quantity of water required to restore the pressure shall be accurately determined by pumping through a positive displacement water meter. The meter shall be approved by the City Water Quality Control Specialist. Acceptability of the test will be determined as follows: The quantity of water lost from the main shall not exceed the number of gallons per hour as determined by the formula:

$$L = \frac{SD\sqrt{P}}{266{,}400}$$

in which,

L	=	allowable leakage (gallons/hour)
D	=	nominal diameter of the pipe (inches)
P	=	test pressure during the leakage test (psi)
S	=	gross length of pipe tested (feet)

There shall not be an appreciable or abrupt loss in pressure during the 15 minute test period. Any visible leakage shall be corrected regardless of the allowable leakage specified above. Should the tested section fail to meet the pressure test successfully as specified, the defects shall be located and repaired and the pipeline retested.

Tests shall be made with the hydrant auxiliary gate valves open and the hydrant valve in the closed position. Once the new line is successfully tested, each valve shall be tested by closing each in turn and relieving the pressure behind it. The mains shall be tested between valves. As possible, no hydrostatic pressure shall be placed against the opposite side of the valve being tested. This test of the valve will be acceptable if there is no immediate loss of pressure on the gauge when the pressure comes against the valve being checked. The pressure differential across the valve shall not exceed the rated working pressure of the valve.

All service lines shall be flushed prior to contacting the Water Quality Control Specialist for a pressure test. All necessary equipment shall be ready and available for set up but not connected until the Water Quality Control Specialist is present for operation.

Defective materials or workmanship discovered as a result of hydrostatic field test shall be replaced or repaired and the hydrostatic test shall be repeated to the satisfaction of the City Engineer.

5-3.33 DISINFECTION AND FLUSHING OF WATER MAINS

Before being placed into service, new water mains and repaired portions of existing mains shall be chlorinated and a satisfactory bacteriological report obtained. Disinfection of water mains shall be accomplished in accordance with the requirements of the Washington State Department of Health, AWWA Standard C651, and in a manner satisfactory to the City Engineer. Sections shall be disinfected between adjacent valves unless otherwise approved by the City Engineer. All filling and flushing shall be done through a meter with a DCVA provided by the party responsible for the project. Valves shall only be operated by authorized City employees.

Flushing

Where dry calcium hypochlorite is used for disinfection of the pipe, flushing shall be done after disinfection. If a hydrant is not installed at the end of the water main, the Contractor shall provide a tap large enough to develop a flow velocity of at least 2.5 feet per second in the water main.

The flushing period must be approved by the City Engineer. The source water used for disinfection and pressure testing shall be flushed prior to its use to ensure that contaminants or debris are not introduced into the new pipe.

Taps for temporary or permanent release of air, chlorination or flushing purposes shall be provided as a part of the construction of the water mains.

Chlorination

The section to be tested shall be chlorinated so that a chlorine residual of no less than 25 mg/l (parts per million or ppm) remains in the water after standing 24 hours in the pipe. The initial chlorine content of the water shall not be less than 50 mg/l. The forms of chlorine that may be used in the disinfection operations are liquid chlorine and calcium hypochlorite granules.

Liquid Chlorine: Chlorine shall be applied by solution fed at one end of the section with a valve or hydrant at the opposite end open sufficiently to permit a flow through during chlorine application. The chlorine solution shall be fed into the pipeline already mixed by an automatically proportioning applicator to provide a steady application rate no less than 50 mg/l (ppm) chlorine. Hydrants along the chlorinated section shall be open during application until the presence of chlorine has definitely been detected in each hydrant run. When a chlorine concentration of no less than 50 ppm has been established throughout the line, the valves shall be closed and the line left undisturbed for 24 hours.

Dry Calcium Hypochlorite: Calcium hypochlorite conforming to AWWA B300 is available in granular tablet form and must contain approximately 65% available chlorine by weight. This procedure is allowed only when the extension has been kept clean and dry. If piping has been submerged, or is unclean, refer to AWWA Standard C651.

Granulated chlorine: Dry calcium hypochlorite at 65% - 70% chlorine shall be placed in the pipe to yield a dosage no less than 50 mg/l. The number of ounces of 65% test calcium hypochlorite required for a 20 foot length of pipe equals 0.00843ld, in which "D" is the diameter in inches (see current WSDOT Standards and Specifications). The main shall be filled with water at a rate to ensure that the water within the main will flow at a velocity no greater than 1 foot per second. Precautions shall be taken to ensure that air pockets are eliminated. When a chlorine concentration of not less than 50 ppm has been established throughout the line, the valves shall be closed and the line left undisturbed for 24 hours. If the water temperature is less than 41 degrees F, the water shall remain in the pipe for at least 48 hours.

After chlorination in accordance with the above, the line shall be thoroughly flushed and water samples taken. Flushing shall be done in a manner that avoids damage to surrounding property and that conforms to these Standards.

5-3.34 FINAL FLUSHING AND TESTING

Following chlorination, chlorinated water shall be flushed from the new water main until the replacement water throughout its length shows an absence of chlorine. In the event chlorine is normally used in the source of supply, the tests shall show a residual not in excess of that normally carried in the water supply system (never to exceed 2 mg/l).

After final flushing and before the new water main is connected to the distribution system, a sample collection test shall be scheduled with the City Engineer a minimum of two (2) business days in advance of test. The number of samples from the source and the number of representative sample points required will be determined by the Water Quality Control Specialist. Appropriate sample taps shall be furnished by the party responsible for the project. No hose or fire hydrant shall be used in the collection of samples.

At least one set of samples shall be collected from every 1,200 feet of the new water main, plus one set from the end of the line and at least one set from each branch. All samples shall be tested for total coliform bacteria and for heterotrophic bacteria by the heterotrophic plate count (HPC) analysis. The maximum allowable coliform content of the flushed sample shall be zero. The maximum allowable HPC population count in all source samples shall be 80/ml. Any source sample that exceeds a count of 80/ml shall be ruled as an indeterminate test and a new set of source and construction samples for analysis shall be required. The maximum allowable HPC population count from any construction sample shall be no greater than twenty (20) counts above the highest source HPC population count.

Before placing the lines into service, a satisfactory report shall be provided to the Water Quality Control Specialist from the certified laboratory evidencing successful tests on samples collected from representative points in the system extension.

Should the initial test result in an unsatisfactory bacteriological test, additional chlorination using the above procedure shall be repeated until satisfactory results are

obtained. Treated water flushed from the mains shall be disposed of in accordance with applicable State and local laws and regulations.

Chlorinated water shall never be flushed into the storm drain or a body of water. This includes lakes, rivers, streams, stormwater drainage systems, and any waters where fish or other natural aquatic life can be expected to be present.

5-3.35 ADJUST EXISTING STRUCTURE TO GRADE

Existing Structure Adjustment

Existing structures, including valve box covers affected by a pavement overlay, or adjustment in surface grade, shall normally be adjusted to grade within three (3) business days after the pavement overlay. The City Engineer may require immediate adjustment if the structure is critical to operations.

Valve Box Adjustment in Paved Areas

Raising the existing valve box shall be accomplished by adjusting the existing top section of the valve box or replacing the valve box. If the valve box base section needs to be extended, a 4 inch diameter cast iron soil pipe shall be installed with the bell end of the soil pipe inserted over the top of the existing valve box base section. The spigot end of the soil pipe shall be located a minimum of 6 inches and a maximum of 9 inches below finished grade. The valve box top section shall be slipped over the soil pipe and adjusted to the final grade. A polyethylene sheet, 8-mil thick, shall be placed between the valve box and soil pipe to prevent metal to metal contact where the sections overlap. In asphalt concrete pavement and overlay areas, excavation of the valve box to be raised shall be accomplished by saw cutting or neat line jack hammering the pavement a minimum of 12 inches around the perimeter of the valve box. The final adjustment of valve boxes shall be done within 24 hours after paving. Paving, repaving, and patching shall be completed within 72 hours.

Valve Box Adjustment in Unimproved Areas

Adjustment of valve box covers located in unpaved areas shall be the same as in paved areas.

5-3.36 ABANDONING FACILITIES

Abandonment of Water Main

Water mains no longer in service shall be removed and disposed of by the party responsible for the project. The water main may be abandoned in place with the approval of the City Engineer. When water mains are abandoned, the ends of the pipe and fittings shall be plugged by filling with Class 2500 concrete a minimum length of 12 inches on each pipe end. The City Engineer may require that the abandoned water mains be filled with sand or cement grout depending on the size, material, and location of the water main.

Removal of Service Lines

Service lines that are no longer in service and associated corporate stops shall be removed and disposed of by the party responsible for the project. Saddles shall be plugged with MIPT brass plugs.

Abandonment of Structures

Abandonment of structures shall be completed only after water facilities have been properly removed and/or abandoned. All valves and valve boxes shall be removed on abandoned valves.

5-3.37 LANDSCAPING AND LAWN REMOVAL AND REPLACEMENT

During the construction and installation of a water distribution system, the party responsible for the project shall minimize the disturbance and damage to any landscaping and lawn within the project area and shall restore the landscaping and lawn area to conditions prior to construction and installation.

5-3.38 BORING UNDER ROOTS

Boring under the root systems of trees that cannot be removed shall be accomplished by excavating a trench or pit on each side of the tree, being careful to avoid root injury, and then hand digging or pushing the pipe through the soil under the tree. The pit walls shall be a minimum of 7 feet from the center of the tree and shall have sufficient depth to lay the pipe at the grade shown on the plan and profile. Trees shall be removed unless otherwise directed by the City Engineer.

5-3.39 BORING AND JACKING

The vertical and horizontal location of existing utilities shall be verified by the party responsible for the project. If required to avoid conflicts and maintain minimum clearances, adjustment shall be made to the grade of the casing.

The pipe shall be bored and jacked where shown on the plans. All obstructions encountered shall be removed or penetrated. If groundwater is found to be a problem during boring operations, all necessary measures shall be taken to control the flow sufficiently to protect the excavation, pipe and equipment so that the work is not impaired. Any pipe damaged during the boring and jacking operation shall be repaired in a manner approved by the City Engineer.

The installation of the bored and jacked pipe shall be done in manner to ensure that no settlement or caving is caused to the above surface. Any such caving caused by the placement of the pipe shall be repaired by the party responsible for the project as directed by the City Engineer.

During the jacking operations, particular care shall be exercised to prevent caving ahead of the pipe which will cause voids outside of the pipe. If voids exist, the party responsible for the project shall drill through the wall of the pipe and fill the voids by pumping cement grout. All voids shall be filled to the satisfaction of the Public Works Inspector.

The carrier pipe shall be installed in the casing as shown in the City Standard Plans. The carrier pipe shall be supported with casing spacers as shown in the Standard Plans. The casing spacers shall conform to these Standards. The material shall be resistant to abrasion and sliding wear. There shall be a minimum of two spacers per length of pipe, and the spacing between spacers shall be as shown in Standard Plan 522. Spacers shall be installed per manufacturer's instructions.

Boring and receiving pits shall be backfilled with select native material approved by the City Engineer and compacted to 95% maximum dry density as determined by ASTM D-1557. A sufficient amount of select backfill material shall be provided to make up for the rejected material.

All disturbed ground shall be restored to its original condition or better.

5-3.40 WORKING WITH ASBESTOS CEMENT PIPE

When working with asbestos cement pipe, the work shall be done in manner that minimizes workers' exposure to asbestos material at or below the exposure limit as prescribed in WAC 296-62- 07705 State and Federal Guidelines and Certification and PSAPCA requirements.

5-3.41 ASBESTOS CEMENT WATER MAIN CROSSINGS

Where a new utility line crosses below an existing asbestos cement (A.C.) water main, the A.C. water main shall be replaced with ductile iron pipe to a minimum of 8 feet on either side of the pipe crossing. Where directed by the City Engineer, the trench shall be backfilled with CDF from the bottom of the trench to the bottom of the A.C. main. All DIP crossings shall be constructed using ROMAC extended range transition couplings.

5-3.42 CONTROLLED DENSITY FILL

Controlled Density Fill (CDF) shall be used only by approval of the City Engineer. CDF can be proportioned to be flowable, non-segregating, or excavatable by hand or machine. Desired flowability shall be achieved with the following guidelines:

Low Flowability	below 6 inch slump
Normal Flowability	6 – 8 inch slump
High Flowability	8 inch slump or greater

CDF shall be placed by any reasonable means into the area to be filled. CDF patching, mixing and placing may be started if weather conditions are favorable, when the

temperature is at 34 degrees F and rising. At the time of placement, CDF must have a temperature of at least 40 degrees F. Mixing and placing shall stop when the temperature is 38 degrees F or less and falling. Each filling stage shall be as continuous as possible. CDF shall not be placed on frozen ground.

Trench section to be filled with CDF shall be contained at either end of the trench section by bulkhead or earth fill.

When used to support existing A.C. pipe, the flowable CDF shall be brought up uniformly to the bottom of the A.C. pipe, as shown on the plans, or as directed by the City Engineer. Steel plates shall be provided and installed to span the utility trenches and prevent traffic contact with CDF for at least 24 hours after placement or until CDF is compacted or hardened to prevent rutting by construction equipment or traffic.

If CDF is used for trench backfill on ductile iron pipe it shall be encased in ⅝ inch minus crushed rock and services shall be encased in sand.

5-3.43 VAULT INSTALLATION

Vaults for water facilities (pressure reducing stations, valves, water service, flow meters, backflow prevention assemblies, etc.) shall be installed at the locations shown on the plan and as staked. Vaults shall be constructed as shown on the plans, Standard Plans and as directed by the City Engineer.

The excavation shall have a minimum of one (1) foot clearance between the vault outer surface and the earth bank. Foundation gravel or bedding concrete shall be used on top of undisturbed soil to support the vault. The vault shall be plumb and watertight. The access cover shall be seated properly to prevent rocking and shall be adjusted to match the finished grade.

The vault floor shall drain to daylight, or to a location specified on the plans. Gravity drain pipe shall be a minimum of 3 inches in diameter.

Where knockout locations for the pipe do not coincide with the locations of pipe penetrations into the vault, the party responsible for the project shall core drill openings for pipe.

A sump pump shall be required if directed by the City Engineer.

5-3.44 UTILITY CROSSING

If the minimum vertical distance between utility pipes is less than 6 inches and such installations are approved by the permitting agency, a rigid foam pad shall be placed between the pipes. The pad shall be outside diameter O.D. × O.D. × 2.5 inches thick minimum or as required to protect the pipes and O.D. is equal to the outside diameter of the larger pipe. The pad shall be a polyethylene foam plank (Dow Plastics Ethafoam 220),

or approved equal. Additional measures may be necessary to ensure system integrity and may be required as evaluated by the permitting agency on a case by case basis.

END OF SECTION

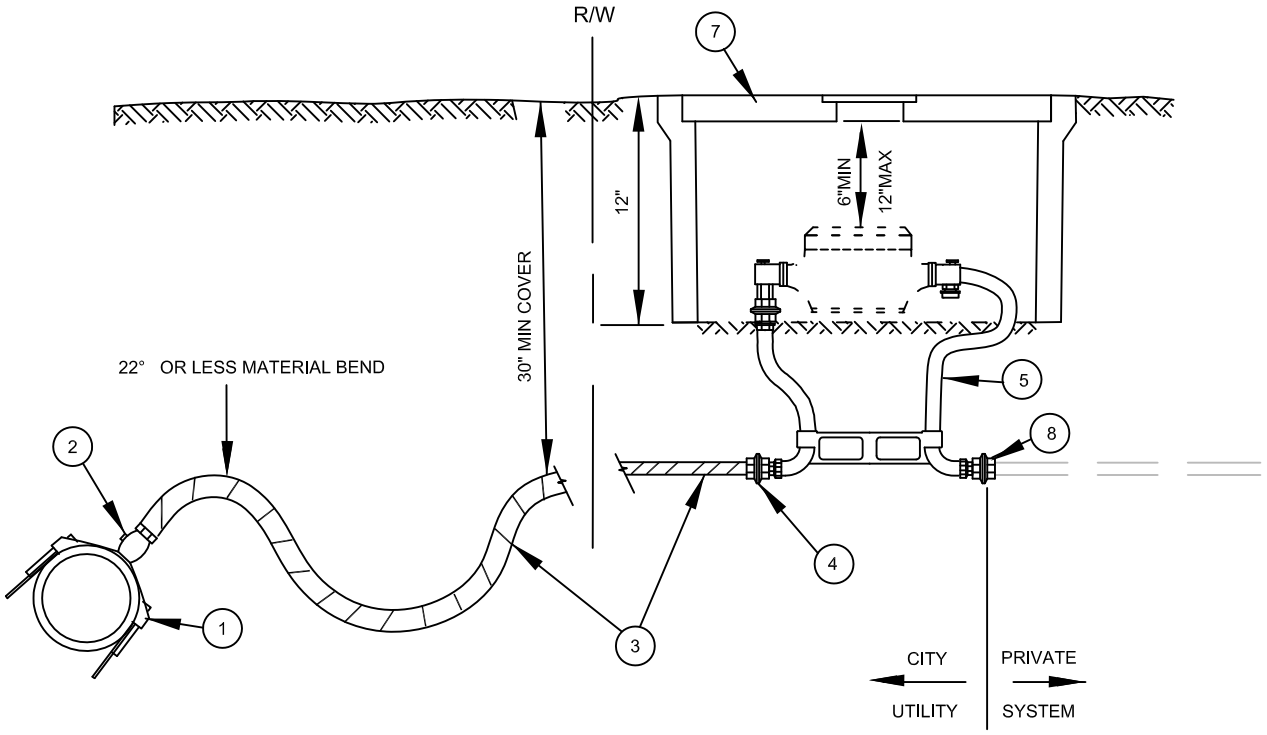
CITY OF SNOHOMISH

ENGINEERING DESIGN AND CONSTRUCTION STANDARDS

WATER DISTRIBUTION

DRAWING INDEX

STANDARD PLAN NO.	PLAN DESCRIPTION	REVISION DATE
501	5/8" – 1" Metered Water Service	April 2010
502	2" Metered Water Service	April 2010
503	Meter and Vault For 3 " Meter	April 2010
503a	Meter and Vault For 3" Meter Notes	April 2010
504	Meter and Vault For 4" & 6" Meter	April 2010
504a	Meter and Vault For 4" & 6" Meter Notes	April 2010
505	Horizontal Thrust Blocks	April 2010
506	Vertical Thrust Blocks	April 2010
507	Fire Hydrant Assembly	April 2010
508	Concrete Fire Hydrant Pad	April 2010
509	Fire Hydrant Guard Post And Valve Marker Post	April 2010
510	Blow-off Assembly	April 2010
511	1" Air-vacuum Valve Assembly	April 2010
512	Water Valve Box and Extension	April 2010
513	Tapping Tees	April 2010
514	Double Check Valve Assembly (DCVA) For 2" and Smaller	April 2010
515	Double Check Valve Assembly (DCVA) For 2 ½" and Larger	April 2010
516	Double Check Detector Valve Assembly (DCDA) 3 "And Larger	April 2010
517	Reduced Pressure Backflow Assembly (RPBA) 2" and Smaller	April 2010
518	Reduced Pressure Backflow Assembly (RPBA) 2-1/2"	April 2010
519	Reduced Pressure Detector Assembly (RPDA) 2 1/2" and Larger	April 2010
520	Typical PRV Installation	April 2010
521	Air Gap for Makeup Tank	April 2010
522	Steel Casing for Water Main	April 2010
523	Typical Water Trench Section	April 2010
524	Bedding for Water Pipe in Trenches	April 2010
525	Typical Water Trench Compaction	April 2010



NTS

NOTE:

1. ROMAC, FORD OR EQUAL DOUBLE STRAP SERVICE SADDLE WITH I.P. THREAD TO BE USED ON ALL MAINS 3" DIA AND LARGER AND ON ALL A.C. MAINS. ALL NEW TAPS ON EXISTING WATER MAINS SHALL BE DONE BY THE CITY OF SNOHOMISH WATER DIVISION AT THE DEVELOPER'S OR CONTRACTOR'S EXPENSE.

2. CORPORATION STOP: 1" MUELLER I.P. OR APPROVED EQUAL, SET AT 45° ON UPPER QUADRANT OF PIPE.

3. 1" WATER SERVICE LINE SHALL BE POLYETHYLENE TUBING (IPS).
   POLYETHYLENE SHALL:
   1. MEET THE REQUIREMENTS OF AWWA C901
   2. BE HIGH MOLECULAR MASS WITH AT LEAST 200 PSI RATING
   3. HAVE A #10 COPPER TRACE WIRE WRAPPED ALONG ITS ENTIRE LENGTH (ONE WRAP PER FT)
   4. ALL TUBING SHALL MEET THE APPLICABLE ASTM AND AWWA STANDARDS.

4. ORISEAL CURB STOP H10283 (TYP) MUELLER OR FORD.

5. 5/8" x 3/4" METER SETTER SHALL BE MUELLER B-24046A SERIES WITH CHECK VALVE.

6. METERS SHALL BE PLACED AT THE BACK OF SIDEWALK WHERE POSSIBLE AT DEVELOPER'S AND CONTRACTOR'S EXPENSE. OWNER SHALL PROVIDE CITY WITH UTILITY EASEMENT IF REQUIRED.

7. METER BOX SHALL BE RAVEN RMB 17-30-12 1 1/2" AND 2" SERVICES; RAVEN RMB 11-18-12 FOR 3/4" SERVICES AND 1" SERVICES. METER BOX LID SHALL BE SIGMA DWG LC 1730T-D DUCTILE IRON WITH READER FOR 1 1/2" AND 2" SERVICE AND SIGMA DWG LC 1118T-D DUCTILE IRON WITH READER BFC11RL FOR 3/4" AND 1" SERVICE. METER BOX SHALL HAVE A 2" HOLE FOR TOUCH READ SENSOR.

8. MUELLER BEND WITH PLUG.

Updated 01/14/2016 MMS

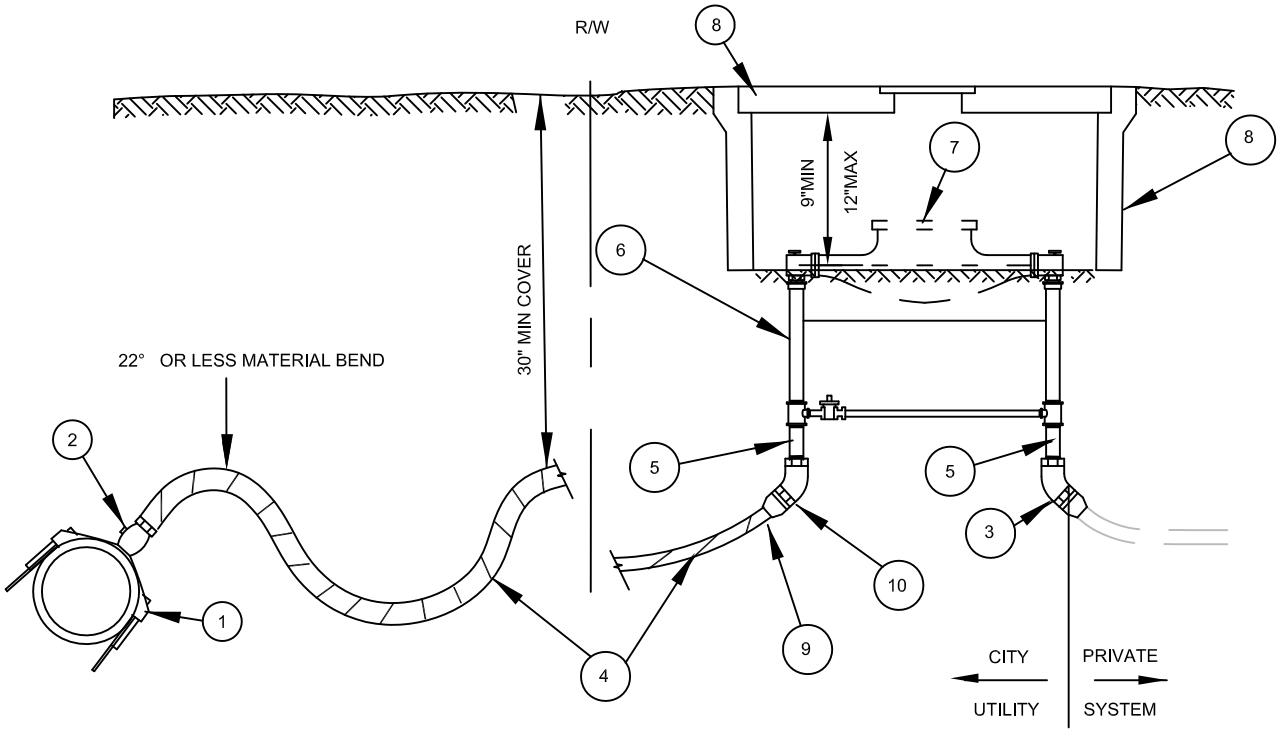
SNOHOMISH
WASHINGTON

5/8" TO 1" METERED WATER SERVICE

Approved By:
S L S
City Engineer
Date: April 2010

501
Number

City of Snohomish Public Works Department



NTS

NOTES:

1. ROMAC, FORD OR EQUAL DOUBLE STRAP SERVICE SADDLE WITH I.P. THREAD TO BE USED ON ALL SERVICE TAPS ALL NEW TAPS ON EXISTING WATER MAINS SHALL BE DONE BY THE CITY OF SNOHOMISH WATER DIVISION AT THE DEVELOPER'S OR CONTRACTOR'S EXPENSE.

2. 2" BRASS CORP STOP MUELLER ORI-CORP H9969 OR FORD.

3. MUELLER BEND WITH PLUG.

4. 2" WATER SERVICE LINE SHALL BE POLYETHYLENE TUBING :
   POLYETHYLENE TUBING SHALL BE 2" COPPER TUBE SIZE (CTS) ASTM D-2737 - SDR9 (PE3408)
   POLYETHYLENE SHALL:
   1. MEET THE REQUIREMENTS OF AWWA C901
   2. BE HIGH MOLECULAR MASS WITH AT LEAST 200 PSI RATING
   3. HAVE A #10 COPPER TRACE WIRE WRAPPED ALONG ITS ENTIRE LENGTH (ONE WRAP PER FT)
   4. ALL TUBING SHALL MEET THE APPLICABLE ASTM AND AWWA STANDARDS.

5. BRASS FITTINGS AS NEEDED.

6. METER SETTERS SHALL BE MUELLER SERIES COPPER SETTER B2427-2.012 WITH HORIZONTAL INLET AND OUTLET WITH CHECK VALVE OR APPROVED EQUAL.

7. METER SHALL BE INSTALLED BY CITY WATER DIVISION AT DEVLOPER'S OR CONTRACTOR'S EXPENSE.

8. METER BOX SHALL BE RAVEN RMB 17-30-12 FOR 1-1/2 AND 2" SERVICES;
   METER BOX SHALL HAVE A 2" HOLE FOR TOUCH READ SENSOR.
   METER BOX LID SHALL BE SIGMA DWG LC 1730T-D DUCTILE IRON WITH READER (17.5" X30.375") FOR 1-1/2 AND 2" SERVICES.

9. 1-1/2" SERVICE SHALL BE REDUCED AT THE SETTER.

10. ORISEAL CURB STOP H10283 (TYP) MUELLER OR FORD.

Updated 01/14/2016 MMS

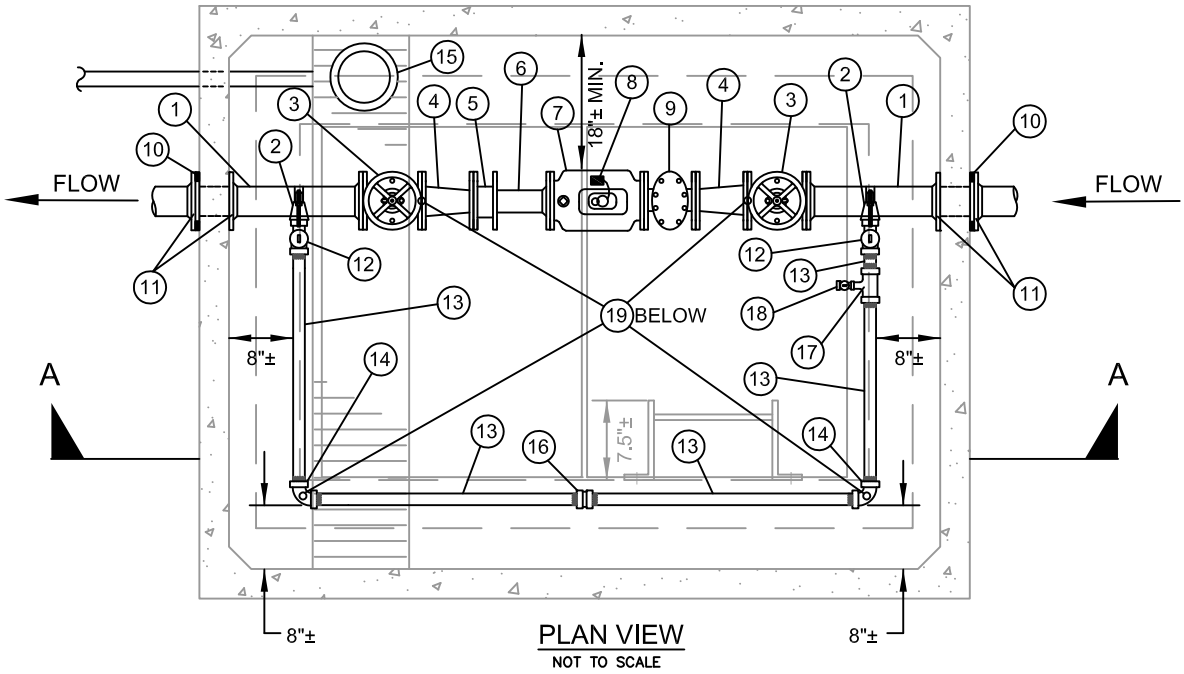
SNOHOMISH
WASHINGTON

2" METERED WATER SERVICE

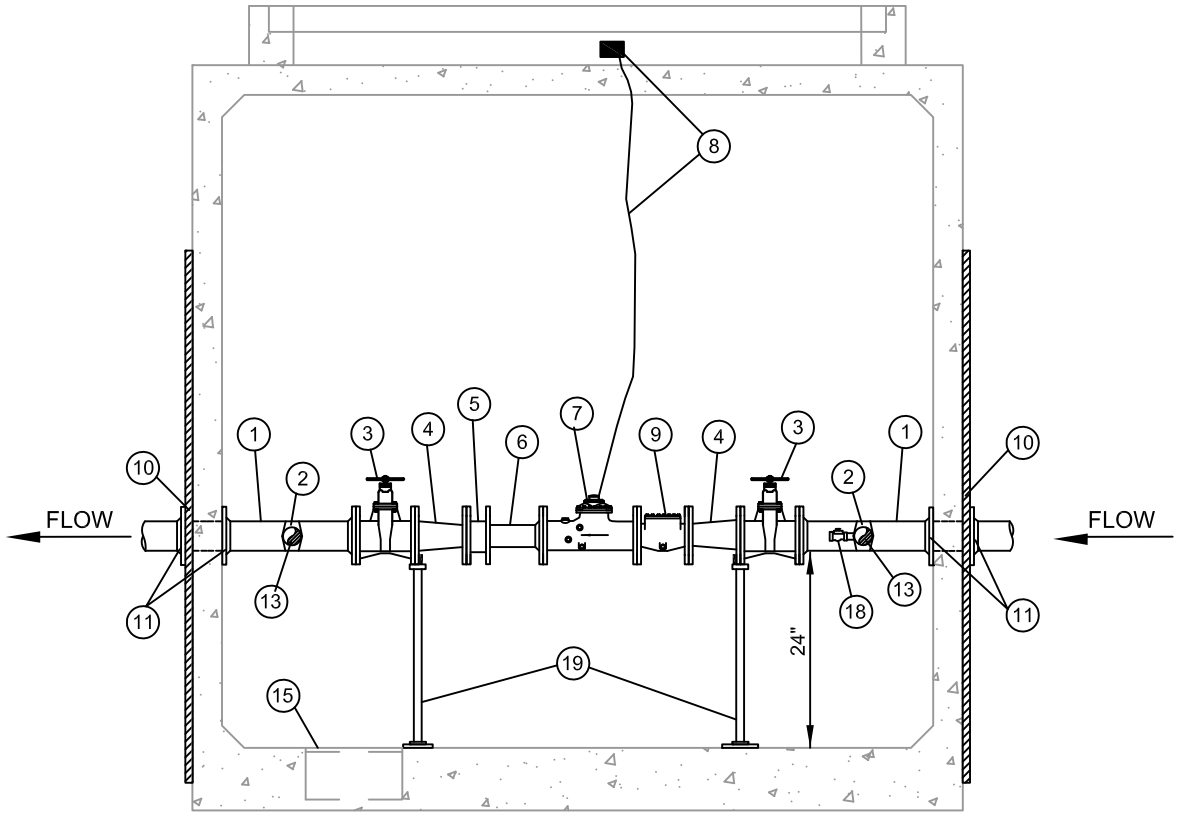
Approved By:
S L S
City Engineer
Date: April 2010

502
Number

City of Snohomish Public Works Department



PLAN VIEW
NOT TO SCALE



SECTION A-A
NOT TO SCALE

3" WATER METER INSTALLATION

PIPING DETAIL

SNOHOMISH
WASHINGTON

METER/VAULT FOR 3" METER

Approved By:
S L S
City Engineer
Date: April 2010

503
Number

City of Snohomish Public Works Department

MATERIAL LIST:

ITEM	QTY.	DESCRIPTION
1	2	4" D.I. ADAPTER, FL x PE, 5'± LONG.
2	2	SADDLE WITH STAINLESS STEEL STRAP, 2" IP TAP, FORD#FS202 SADDLE WITH IP7 TAP OR EQUAL
3	2	4" RESILIENT SEAT GATE VALVE, NRS, FL, O-RING PACKING WITH HAND WHEEL. VALVE TO BE AMERICAN, WATEROUS, MUELLER, M&H, OR CLOW MEETING AWWA C-509.
4	2	4" X 3" D.I. REDUCER, FL.
5	1	3" FLANGED COUPLING ADAPTER TO BE FORD FLANGED COUPLING ADAPTER, STYLE FFCA W/STAINLESS STEEL ANCHOR STUDS, OR EQUAL.
6	1	3" D.I. ADAPTER, FL x PE, LENGTH TO FIT.
7	1	3" FLANGED SINGLE REGISTER HIGH PERFORMANCE COMPOUND METER, TO BE PROVIDED BY THE CITY AND INSTALLED BY CONTRACTOR.
8	1	METER TRANSCEIVER UNIT (MXU) AND MOUNTING BRACKET, TO BE PROVIDED BY THE CITY AND INSTALLED BY CONTRACTOR ON HATCH RISER.
9	1	3" FLANGED STRAINER.
10	4	2" X 2" X 1/4" MIN. WALL THICKNESS SQUARE STEEL TUBING, 6' MIN. LENGTH. BOLT TO VAULT WALL IN FOUR PLACES WITH STAINLESS STEEL ANCHOR BOLTS.
11	4	UNI-FLANGE ADAPTER.
12	2	2" MIP X FIP LOCKABLE BALL VALVE.
13	5	2" BRASS NIPPLE, LENGTH TO FIT.
14	2	2" BRASS 90° BEND.
15	1	SUMP PUMP OR GRAVITY DRAIN AS REQUIRED BY SITE CONDITIONS, TO BE DETERMINED BY THE DEVELOPER AND CONTRACTOR AND REVIEWED AND APPROVED BY THE CITY. FOR GRAVITY: SUMP DRAIN, ZURN Z-551, THREADED, WITH ZURN Z-1099 BACKWATER VALVE, THREADED x NO-HUB, WITH D.I. PIPE TO 12" MIN. BEYOND VAULT WALL AND 4" PVC D3034 SDR35, MIN. 1% SLOPE TO DAYLIGHT OR C.B. FOR SUMP PUMP: 16" DIAM. x 6" DEEP SUMP WITH ZOELLER MODEL 57 SUMP PUMP. PROVIDE ELECTRICAL POWER PERALL APPLICABLE CODES. CONSTRUCT 1-1/4" DISCHARGE PIPING ENCASED IN G.I.P. TO NEAREST CATCH BASIN OR APPROVED LOCATION.
16	1	2" BRASS UNION.
17	1	2" X 1" BRASS TEE.
18	1	1" MIP LOCKABLE BALL VALVE WITH CAP.
19	4	ADJUSTABLE PIPING SUPPORT, STANDON MODEL #S89, OR EQUAL.

NOTES:

1. PROVIDE SHOP DRAWINGS AND SUBMITTALS FOR ALL MATERIALS TO THE CITY FOR REVIEW AND APPROVAL PRIOR TO CONSTRUCTION.
2. VAULT SHALL BE UTILITY VAULT CO PRECAST VAULT OR EQUAL.
3. PIPE AND CONDUIT PENETRATIONS SHALL BE CORE-DRILLED ON-SITE AND SHALL BE SEALED WATERTIGHT. PROVIDE "LINK-SEAL" FITTINGS AT ALL PENETRATIONS.
4. ALL PIPING AND APPURTENANCES OUTSIDE THE VAULT SHALL BE RESTRAINED JOINT. IN ADDITION TO THE RESTRAINED JOINTS, THRUST BLOCKING AT ALL TEES AND BENDS PER STANDARD PLANS 505 AND 506 SHALL BE REQUIRED.
5. THE CONTRACTOR IS RESPONSIBLE FOR ELECTRICAL PERMITS AND APPROVALS FOR SUMP PUMP INSTALLATIONS.

3" WATER METER INSTALLATION
PIPING DETAIL NOTES

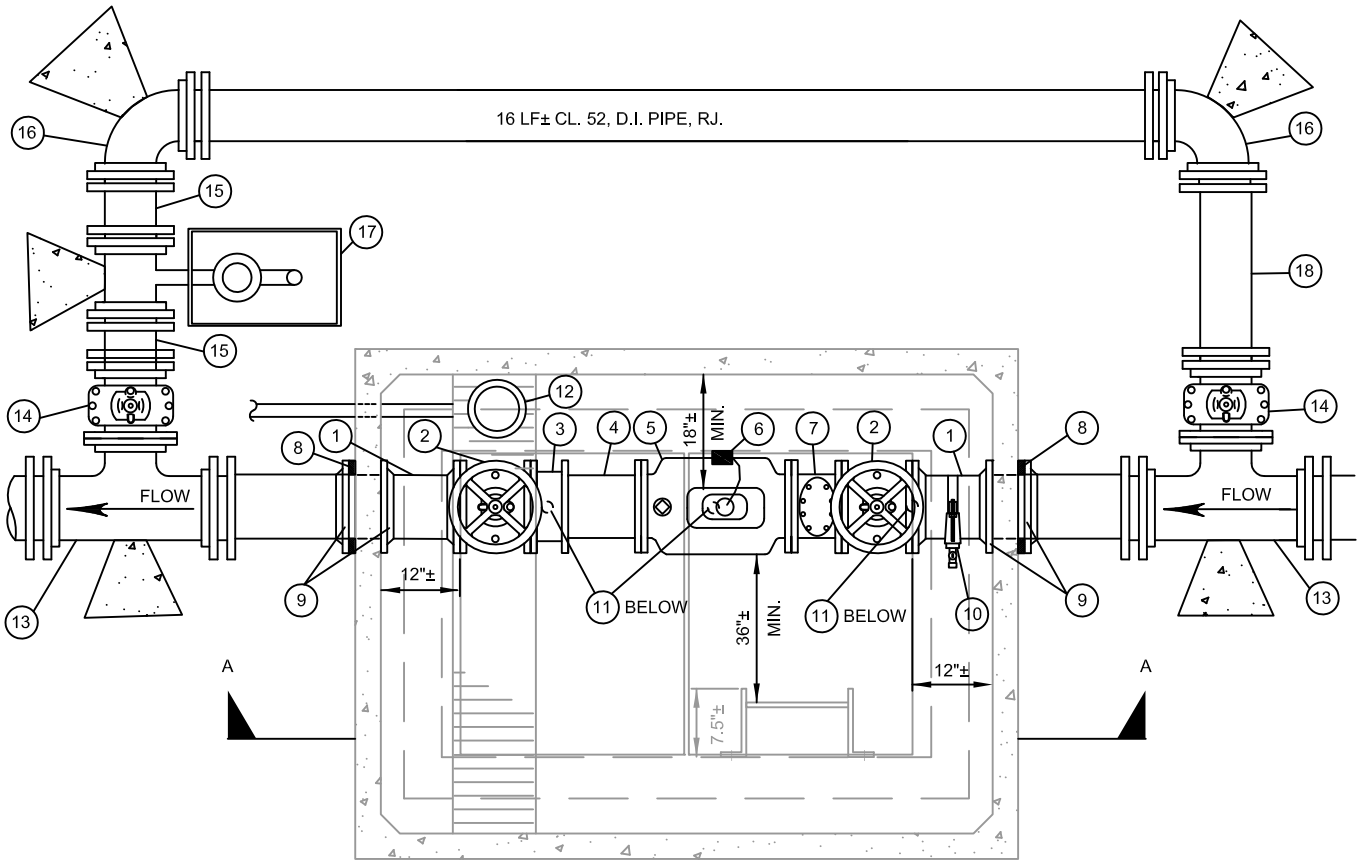
SNOHOMISH
WASHINGTON

NOTES FOR 3" METER/VAULT

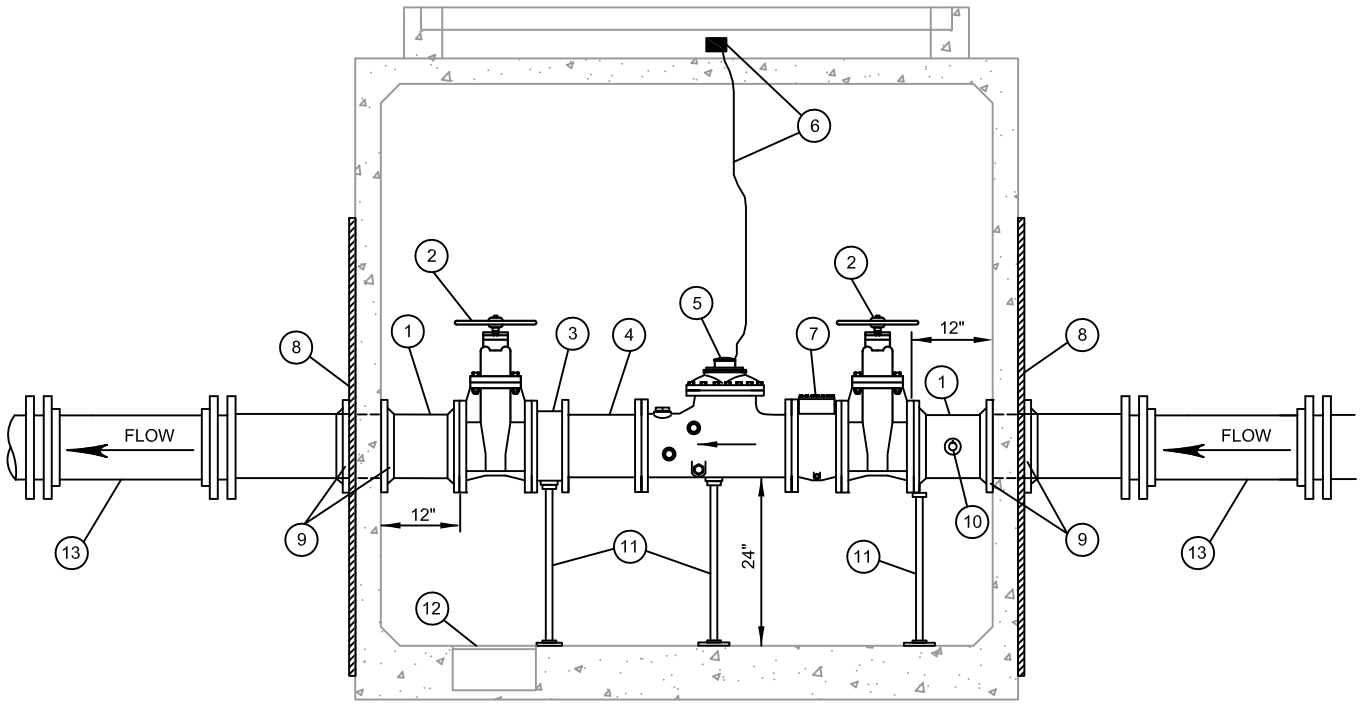
Approved By:
S L S
City Engineer
Date: April 2010

503a
Number

City of Snohomish Public Works Department



PLAN VIEW
NOT TO SCALE



SECTION A-A
NOT TO SCALE

4" & 6" WATER METER INSTALLATION
PIPING DETAIL

SNOHOMISH
WASHINGTON

METER/VAULT FOR 4" & 6" METER

Approved By:
S L S
City Engineer
Date: April 2010

504
Number

City of Snohomish Public Works Department

MATERIAL LIST: ALL PIPING & APPURTENANCES TO MATCH METER SIZE

ITEM	QTY.	DESCRIPTION
1	2	D.I. ADAPTER, FL x PE, 5'± LONG.
2	2	RESILIENT SEAT GATE VALVE, NRS, FL, O-RING PACKING WITH HAND WHEEL. VALVE TO BE AMERICAN, WATEROUS, MUELLER OR CLOW MEETING AWWA C-509.
3	1	FLANGED COUPLING ADAPTER TO BE FORD FLANGED COUPLING ADAPTER, STYLE FFCA W/STAINLESS STEEL ANCHOR STUDS, OR EQUAL.
4	1	D.I. ADAPTER, FL x PE, LENGTH TO FIT.
5	1	FLANGED SINGLE REGISTER HIGH PERFORMANCE COMPOUND METER, TO BE PROVIDED BY THE CITY AND INSTALLED BY CONTRACTOR.
6	1	METER TRANSCEIVER UNIT (MXU) AND MOUNTING BRACKET, TO BE PROVIDED BY THE CITY AND INSTALLED BY CONTRACTOR ON HATCH RISER.
7	1	FLANGED STRAINER.
8	4	2" X 2" X 1/4" MIN. WALL THICKNESS SQUARE STEEL TUBING, 6' MIN. LENGTH. BOLT TO VAULT WALL IN FOUR PLACES WITH STAINLESS STEEL ANCHOR BOLTS.
9	4	UNI-FLANGE ADAPTER.
10	1	SADDLE WITH STAINLESS STEEL STRAP, 1" IP TAP AND 1" MIP LOCKABLE BALL VALVE WITH CAP.
11	3	ADJUSTABLE PIPING SUPPORT, STANDON MODEL #S89, OR EQUAL.
12	1	SUMP PUMP OR GRAVITY DRAIN AS REQUIRED BY SITE CONDITIONS, TO BE DETERMINED BY THE DEVELOPER AND CONTRACTOR AND REVIEWED AND APPROVED BY THE CITY. FOR GRAVITY: SUMP DRAIN, ZURN Z-551, THREADED, WITH ZURN Z-1099 BACKWATER VALVE, THREADED x NO-HUB, WITH D.I. PIPE TO 12" MIN. BEYOND VAULT WALL AND 4" PVC D3034 SDR35, MIN. 1% SLOPE TO DAYLIGHT OR C.B. FOR SUMP PUMP: 16" DIAM. x 6" DEEP SUMP WITH ZOELLER MODEL 57 SUMP PUMP. PROVIDE ELECTRICAL POWER PER ALL APPLICABLE CODES. CONSTRUCT 1-1/4" DISCHARGE PIPING ENCASED IN G.I.P. TO NEAREST CATCH BASIN OR APPROVED LOCATION.
13	2	D.I. TEE, FL x RJ.
14	2	RESILIENT SEAT GATE VALVE, NRS, FL x RJ. VALVE TO BE AMERICAN, WATEROUS, MUELLER, M&H, OR CLOW MEETING AWWA C-509. VALVE BOX TO BE PER STANDARD PLAN 512
15	2	D.I. SPOOLS, 1'± LONG.
16	2	90° D.I. BENDS, RJ.
17	1	2" BLOW-OFF ASSEMBLY, STANDARD PLAN 510.
18	1	D.I. SPOOL, 3'± LONG.

NOTES:

1. PROVIDE SHOP DRAWINGS AND SUBMITTALS FOR ALL MATERIALS TO THE CITY FOR REVIEW AND APPROVAL PRIOR TO CONSTRUCTION.
2. VAULT SHALL BE UTILITY VAULT CO PRECAST VAULT OR EQUAL.
3. PIPE AND CONDUIT PENETRATIONS SHALL BE CORE-DRILLED ON-SITE AND SHALL BE SEALED WATERTIGHT. PROVIDE "LINK-SEAL" FITTINGS AT ALL PENETRATIONS.
4. ALL PIPING AND APPURTENANCES OUTSIDE THE VAULT SHALL BE RESTRAINED JOINT. IN ADDITION TO THE RESTRAINED JOINTS, THRUST BLOCKING AT ALL TEES AND BENDS PER STANDARD PLANS 505 AND 506 SHALL BE REQUIRED.
5. THE CONTRACTOR IS RESPONSIBLE FOR ELECTRICAL PERMITS AND APPROVALS FOR SUMP PUMP INSTALLATIONS.

4" & 6" WATER METER INSTALLATION DETAIL NOTES

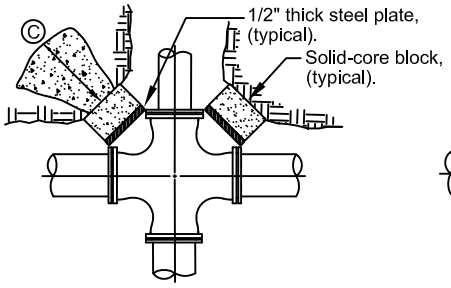
SNOHOMISH WASHINGTON

NOTES FOR 4" & 6" METER/VAULT

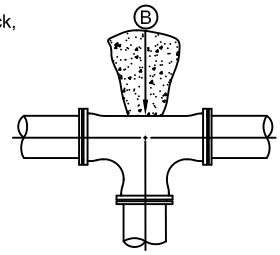
Approved By: S L S
City Engineer
Date: April 2010

504a
Number

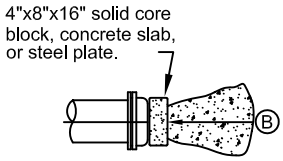
City of Snohomish Public Works Department



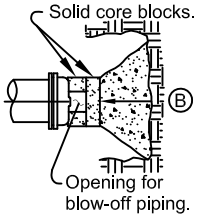
UNBALANCED CROSS



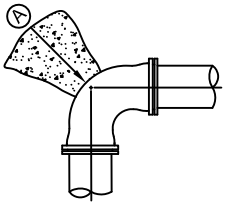
TEE



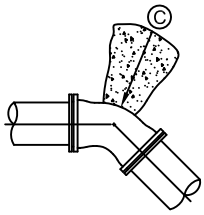
CAP



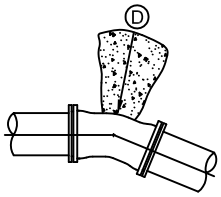
TAPPED CAP/PLUG



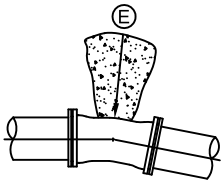
90° BEND



45° BEND



22½° BEND



11¼° BEND

NOT TO SCALE

THRUST BLOCK SIZING FOR 250 PSI PRESSURE

MIN. BEARING AREA AGAINST UNDISTRUBED SOIL-SQ. FT.

PIPE SIZE	Ⓐ	Ⓑ	Ⓒ	Ⓓ	Ⓔ
4"	3/(2)	2/(1)	2/(1)	1/(1)	1/(1)
6"	6/(4)	4/(3)	3/(2)	2/(1)	1/(1)
8"	10/(7)	7/(5)	5/(4)	3/(2)	2/(1)
10"	15/(10)	11/(7)	8/(5)	4/(3)	2/(2)
12"	22/(14)	15/(10)	12/(8)	6/(4)	3/(2)
14"	29/(20)	21/(14)	16/(11)	8/(5)	4/(3)
16"	38/(26)	27/(18)	21/(14)	11/(7)	5/(4)
18"	48/(32)	34/(23)	26/(18)	13/(9)	7/(5)
20"	60/(40)	42/(28)	32/(22)	17/(11)	8/(6)
24"	86/(58)	61/(41)	47/(31)	24/(16)	12/(8)

SAFE BEARING LOADS IN LBS./SQ. FT.

THE SAFE BEARING LOADS GIVEN IN THE FOLLOWING TABLE ARE FOR HORIZONTAL THRUSTS WHEN THE DEPTH OF COVER OVER THE PIPE EXCEEDS 2 FEET.

SOIL	SAFE BEARING LOAD
*MUCK, PEAT, ETC. (SEE NOTE 7)	0
SOFT CLAY	1,000
SAND	2,000
SAND AND GRAVEL	3,000
SAND AND GRAVEL CEMENTED W/CLAY	4,000
HARD SHALE	10,000

GENERAL NOTES

1. CONCRETE THRUST BLOCK AREA BASED UPON A SAFE BEARING LOAD OF 2000/(3000) LBS. PER SQ. FT.
2. AREAS MUST BE ADJUSTED FOR OTHER SIZE PIPE, PRESSURES AND SOIL CONDITIONS.
3. CONCRETE BLOCKING SHALL BE CAST-IN-PLACE AND HAVE MINIMUM OF 1/2 SQUARE FOOT CONTACT BEARING AGAINST THE FITTING.
4. BLOCK SHALL BEAR AGAINST FITTINGS ONLY AND SHALL BE CLEAR OF JOINTS TO PERMIT TAKING UP OR DISMANTLING JOINT.
5. CONTRACTOR SHALL INSTALL BLOCKING ADEQUATE TO WITHSTAND TEST PRESSURE AS WELL AS TO CONTINUOUSLY WITHSTAND OPERATING PRESSURE UNDER ALL CONDITIONS OF SERVICE.
6. POLYETHYLENE WRAP NOT SHOWN FOR CLARITY.
7. IN MUCK OR PEAT, ALL THRUSTS SHALL BE RESTRAINED BY PILES OR TIE RODS TO SOLID FOUNDATIONS OR BY REMOVAL OF MUCK OR PEAT AND REPLACEMENT WITH BALLAST OF SUFFICIENT STABILITY TO RESIST THRUSTS.

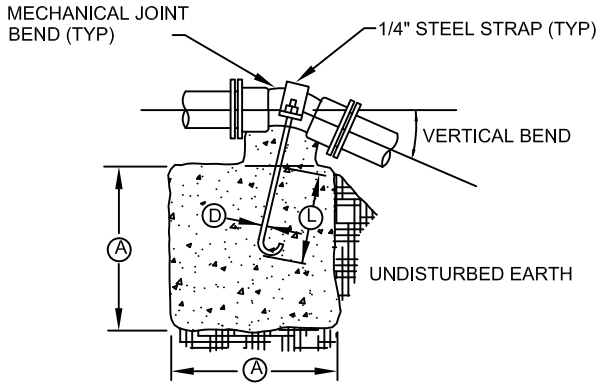
SNOHOMISH WASHINGTON

HORIZONTAL THRUST BLOCKS

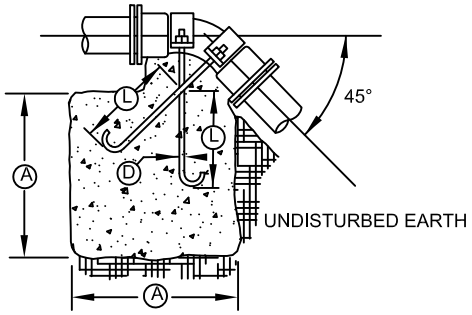
Approved By: S L S
City Engineer
Date: April 2010

505
Number

City of Snohomish Public Works Department



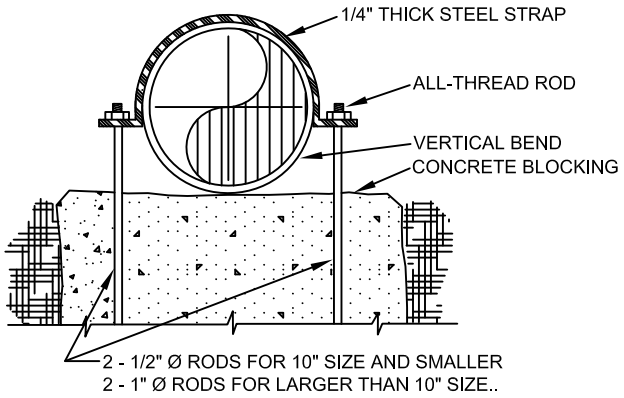
BLOCKING FOR 11-1/4°, 22-1/2° & 30° VERTICAL BENDS



BLOCKING FOR 45° VERTICAL BENDS

VERTICAL BLOCKING

PIPE SIZE	VERT. BEND	CU.FT.	(A)	(D)	(L)
4"	11-1/4°	8	2.0'	3/4"	1.5'
	22-1/2°	11	2.2'	3/4"	2.0'
	30°	17	2.6'	3/4"	2.0'
	45°	30	3.1'	3/4"	2.0'
6"	11-1/4°	11	2.2'	3/4"	2.0'
	22-1/2°	25	2.9'	3/4"	2.0'
	30°	41	3.5'	3/4"	2.0'
	45°	68	4.1'	3/4"	2.0'
8"	11-1/4°	16	2.5'	3/4"	2.0'
	22-1/2°	47	3.6'	3/4"	2.5'
	30°	70	4.1'	3/4"	2.5'
	45°	123	5.0'	3/4"	2.0'
12"	11-1/4°	32	3.2'	3/4"	2.0'
	22-1/2°	88	4.5'	7/8"	3.0'
	30°	132	5.1'	7/8"	2.5'
	45°	232	6.1'	3/4"	2.5'
16"	11-1/4°	70	4.1'	7/8"	3.0'
	22-1/2°	184	5.7'	1-1/8"	4.0'
	30°	275	6.5'	1-1/4"	4.0'
	45°	478	7.8'	1-1/8"	4.0'
20"	11-1/4°	91	4.5'	7/8"	3.0'
	22-1/2°	225	6.1'	1-1/4"	4.0'
	30°	330	6.9'	1-3/8"	4.5'
	45°	560	8.2'	1-1/4"	4.0'
24"	11-1/4°	128	5.0'	1"	3.5'
	22-1/2°	320	6.8'	1-3/8"	4.5'
	30°	480	7.9'	1-5/8"	5.5'
	45°	820	9.4'	1-3/8"	4.5'



TYPICAL CROSS-SECTION

NOTES:

1. CONCRETE BLOCKING IS BASED ON 250psi WATER PRESSURE AND 2500 psi CONCRETE STRENGTH.
2. BOLTS AND NUTS NOT EMBEDDED IN CONCRETE SHALL BE CLEANED AND COATED WITH COAL TAR EPOXY.
3. POLYETHYLENE WRAP NOT SHOWN FOR CLARITY.

NOT TO SCALE

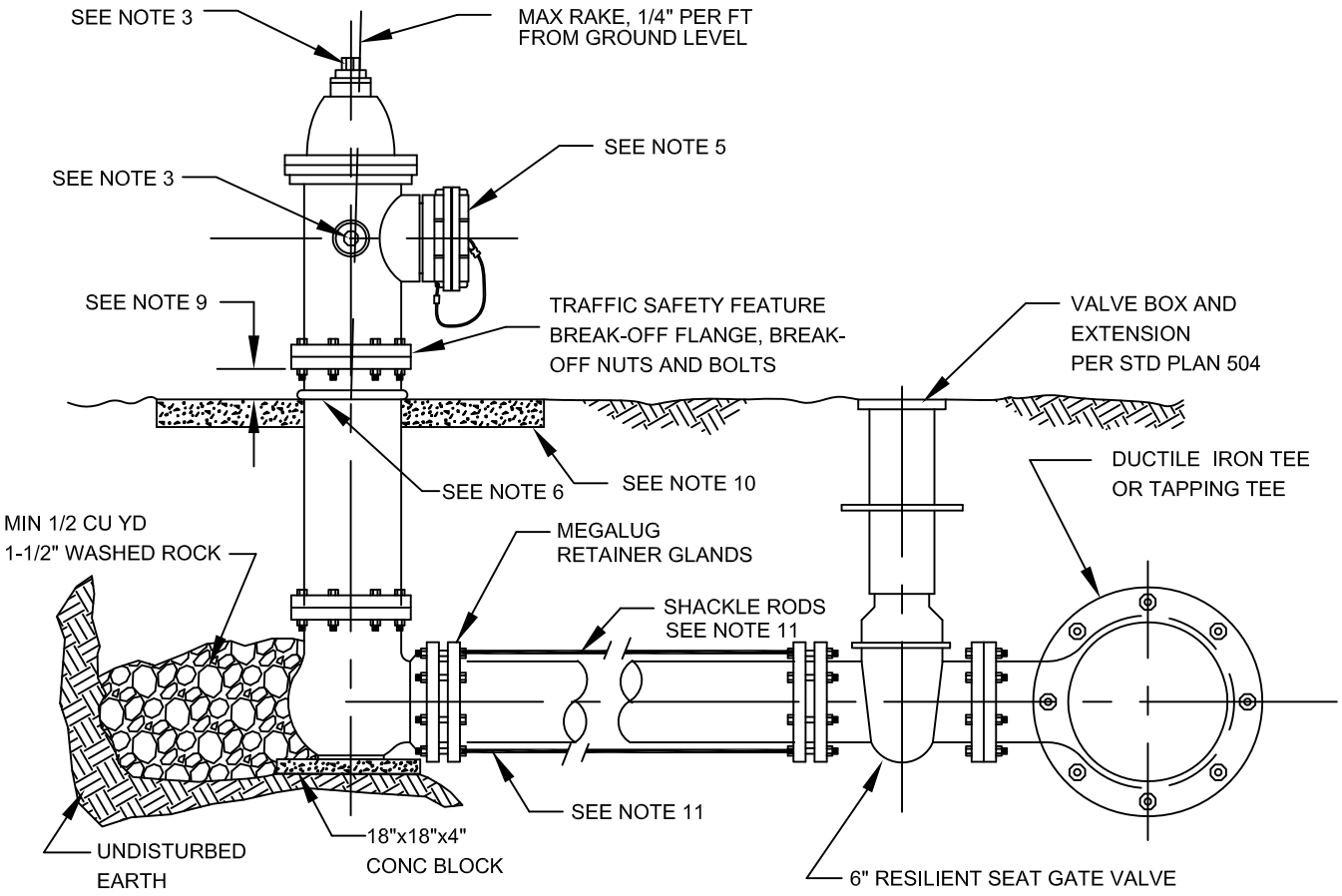
SNOHOMISH WASHINGTON

VERTICAL THRUST BLOCKS

Approved By:
S L S
City Engineer
Date: April 2010

506
Number

City of Snohomish Public Works Department



NOTES:

1. HYDRANTS AND ALL MATERIALS SHALL CONFORM TO AWWA STANDARDS AND SHALL BE OF STANDARD MANUFACTURE - MUELLER SUPER CENTURION, M&H 929, OR EAST JORDAN IRON MODEL NO. 5CD250.
2. 5-1/4" VALVE MINIMUM.
3. 1-1/2" OPERATING NUT AND CAP NUT FOR 2-1/2" PORTS
4. N.S.T. THREAD ON 2-1/2" PORTS.
5. 5" STORZ FITTING WITH NATIONAL STANDARD THREAD ON THE 4-1/2" PORT.
6. IF HYDRANT RISES THROUGH CONCRETE, USE EXPANSION STRIP AROUND HYDRANT BARREL, PER STD PLAN 509.
7. PROVIDE FOR VEHICULAR TRAFFIC PROTECTION WHEN NECESSARY PER STD. PLAN 509.
8. STEAMER PORT TO BE FACING STREET OR ROADWAY FOR FIRE ENGINE ACCESS.
9. BREAK-OFF FLANGE SHALL BE MINIMUM 2", MAXIMUM 6" ABOVE CONCRETE PAD AND/OR BACK OF SIDEWALK.
10. INSTALL CONCRETE PAD AROUND HYDRANT IN UNPAVED, SOD AND ASPHALT AREAS PER STD. PLAN 508.
11. HYDRANT CONNECTION PIPE TO BE DUCTILE IRON CLASS 52. MINIMUM 3' MAXIMUM 18' LENGTH BETWEEN HYDRANT AND FOOT VALVE. JOINTS SHALL BE MJ WITH RETAINER GLANDS OR FIELD LOCK GASKETS. SHACKLE RODS SHALL BE PART OF ASSEMBLY.
12. FIRE HYDRANTS SHALL BE PAINTED WITH TWO COATS OF EQUIPMENT YELLOW HYDRANT PAINT "RUST-OLEUM" TYPE PAINT.

Updated 01/14/2016 MMS

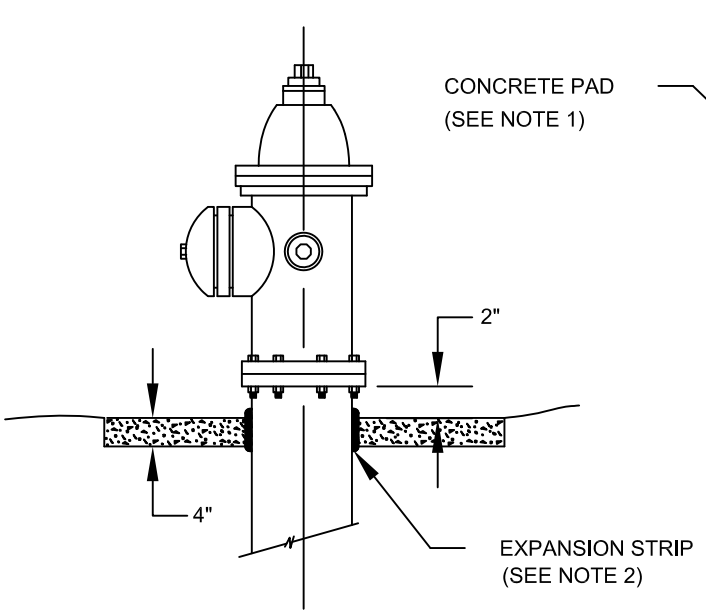
SNOHOMISH
WASHINGTON

FIRE HYDRANT ASSEMBLY

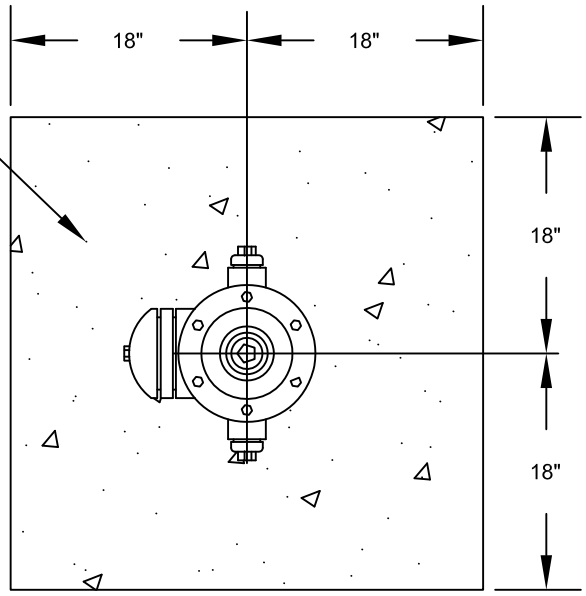
Approved By:
S L S
City Engineer
Date: April 2010

507
Number

City of Snohomish Public Works Department



ELEVATION



PLAN

NOTES:

1. CONCRETE SHALL BE CLASS 3000.
2. INSTALL 1/2"x 4" EXPANSION STRIP AROUND HYDRANT.

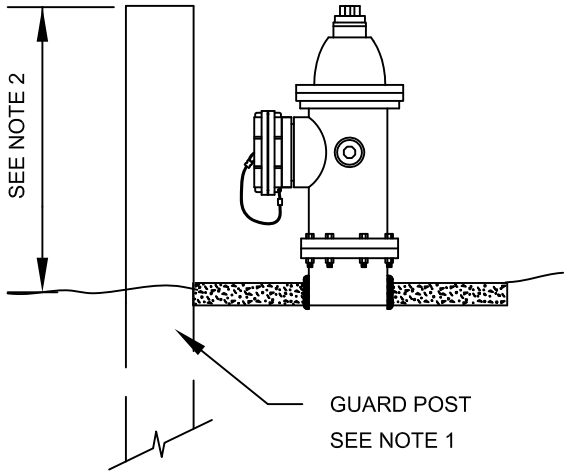
SNOHOMISH
WASHINGTON

CONCRETE FIRE HYDRANT PAD

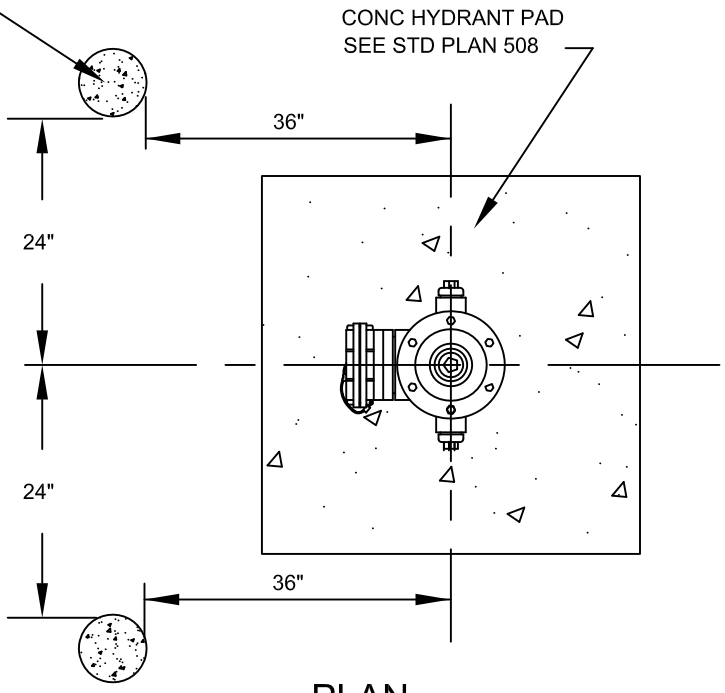
Approved By:
S L S
City Engineer
Date: April 2010

508
Number

City of Snohomish Public Works Department



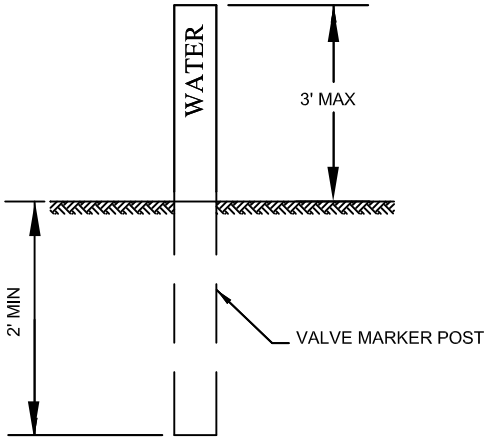
ELEVATION



PLAN

FIRE HYDRANT GUARD POST

(SEE NOTES 1 AND 2)



VALVE MARKER POST

(SEE NOTES 3, 4 AND 5)

NOTES:

1. GUARD POSTS SHALL BE 6' LONG, 6" DIAMETER CONCRETE CLASS 52 DI. PIPE OR 6' LONG, 8" DIAMETER REINFORCED CONCRETE. PAINTED WITH TWO COATS OF HIGH GLOSS EQUIPMENT YELLOW (RUST-OLEUM) TYPE PAINT.
2. TOP OF GUARD POST SHALL BE LEVEL WITH THE TOP OF THE OPERATING NUT.
3. CARSONITE PLASTIC POST MARKER, BLUE, LABELED "WATER".
4. VALVE MARKER POSTS SHALL BE INSTALLED FOR ALL VALVES LOCATED IN UNPAVED AREAS. VALVE MARKER POSTS SHALL BE SET AS DIRECTED BY THE CITY ENGINEER IN A SAFE AND REASONABLY CONSPICUOUS LOCATION.
5. VALVE MARKER POSTS MAY NOT BE REQUIRED FOR AUXILIARY HYDRANT VALVES.

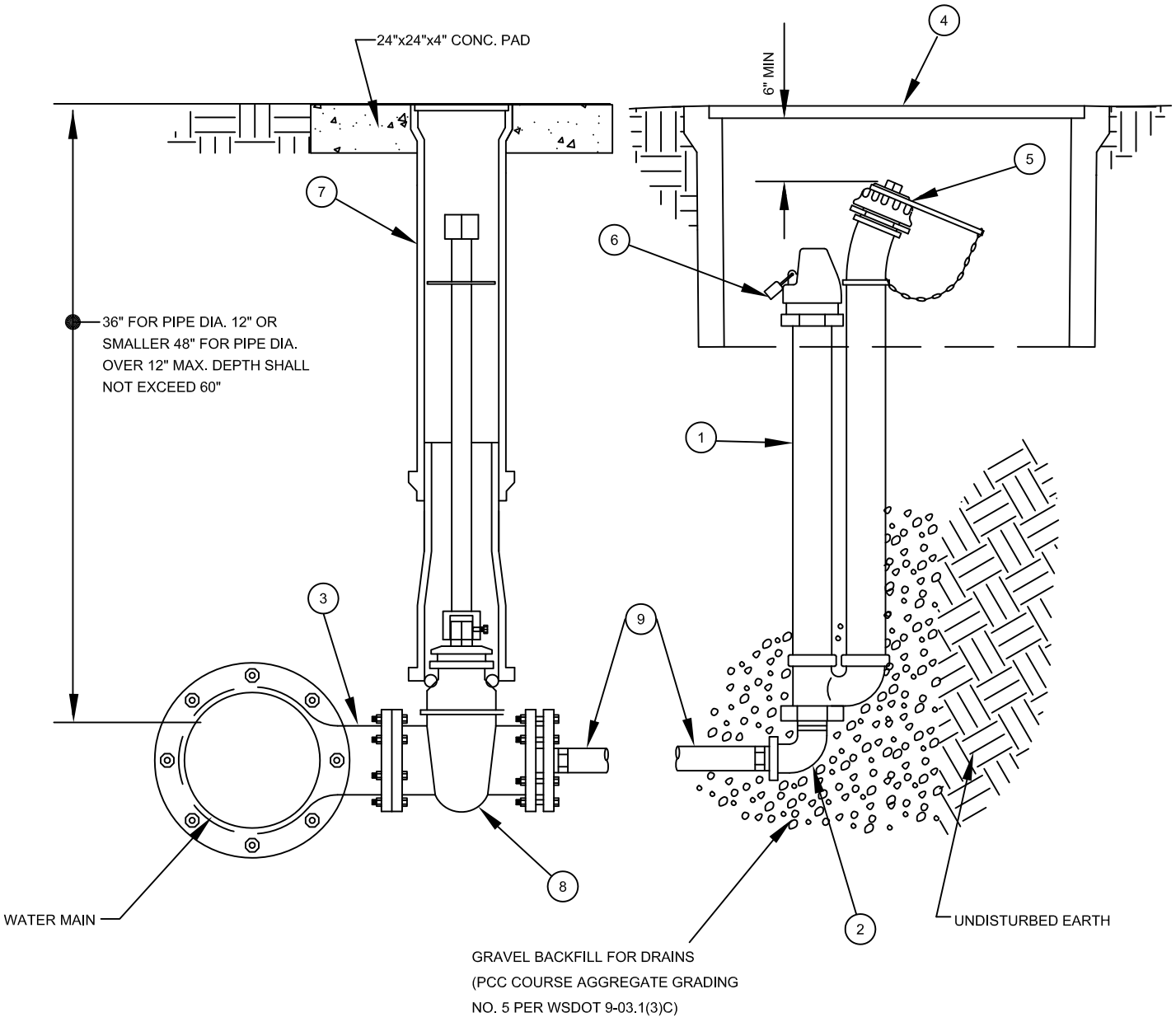
SNOHOMISH
WASHINGTON

FIRE HYDRANT GUARD POST AND VALVE MARKER POST

Approved By:
S L S
City Engineer
Date: April 2010

509
Number

City of Snohomish Public Works Department



MATERIAL LIST:

1. BLOWOFF HYDRANT KUPFERLE FOUNDRY #78 BRONZE TO BRONZE DESIGN SERVICEABLE FROM ABOVE WITH OUTLET EXPOSED. 2-1/2" NST OUTLET LOCKING CAP ON OPERATOR.
2. 2" BRASS 90° BEND
3. MAINLINE SIZE TEE WITH 6" FLANGE
4. MID-STATES PLASTICS METER BOX MSBCF 1730-18/W DI LID.
5. 2-1/2" CAP NATIONAL STANDARD THREAD.
6. LOCK TO BE SUPPLIED BY CITY OF SNOHOMISH.
7. CAST IRON VALVE BOX AND EXTENSION PER STD DETAIL W-190
8. 6" GATE VALVE WITH RESILIENT SEAT (MUELLER, M&H OR APPROVED EQUAL) WITH A 6" FLANGE X 2" COMPANION FLANGE
9. 2" HDPE HI MOL CL 200 CTS POLY PIPE W/2 2" MIP COMPRESSION ADAPTERS

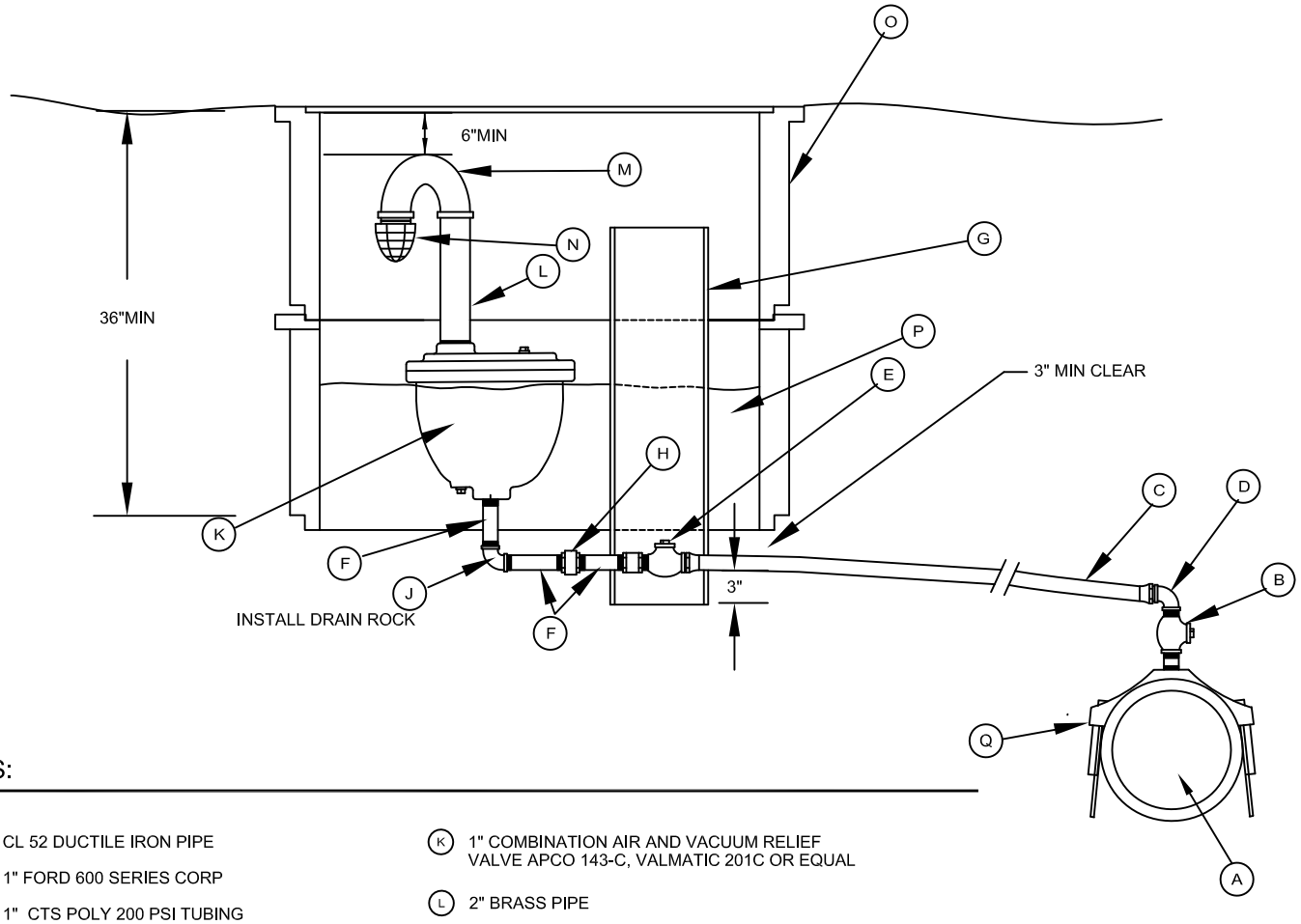
SNOHOMISH WASHINGTON

BLOW OFF ASSEMBLY

Approved By: S L S
City Engineer
Date: April 2010

510
Number

City of Snohomish Public Works Department



PARTS:

- (A) CL 52 DUCTILE IRON PIPE
- (B) 1" FORD 600 SERIES CORP
- (C) 1" CTS POLY 200 PSI TUBING
- (D) 1" FORD 602-44 ANGLE COUPLING
- (E) 1" FORD B21-444 CURB STOP
- (F) 1" BRASS NIPPLE
- (G) 6" PVC PIPE
- (H) 1" BRASS UNION
- (I) AIR VAC SHALL BE PAINTED APPROVED BLUE
- (J) 1" BRASS 90° ELL
- (K) 1" COMBINATION AIR AND VACUUM RELIEF VALVE APCO 143-C, VALMATIC 201C OR EQUAL
- (L) 2" BRASS PIPE
- (M) 2" BRASS RETURN BEND
- (N) BRASS BEEHIVE STRAINER GREENBURG P-24-08, FOR 2" PIPE
- (O) METER BOX SHALL BE MID STATES PLASTICS BFC111812BXL METER BOX LID SHALL BE DUCTILE IRON WITH READER BFC11RL
- (P) BACKFILL WITH CRUSHED ROCK TO BONNET.
- (Q) DOUBLE STRAP SADDLE.

NOTES

1. AIR-VAC UNIT AND BOX TO BE INSTALLED IN NON-TRAFFIC AREA.
2. USE MUELLER DOUBLE STRAP SERVICE CLAMP OR APPROVED EQUAL ON ALL MAINS PER STANDARD PLAN 501
3. ALL PIPE FITTINGS BETWEEN MAIN AND UNION, AFTER AIR/VACUUM RELIEF VALVE, SHALL BE BRASS.
4. INSTALLATIONS FOR OTHER SIZE AIR/VACUUM RELIEF VALVES SHALL BE INDIVIDUALLY DESIGNED AND WILL REQUIRE APPROVAL BY THE CITY ENGINEER.
5. PAINT METER BOX LID AND RISER ASSEMBLY (2) COATS BLUE, OIL BASE ENAMEL- HAND BRUSH APPLIED. STENCIL RISER ASSEMBLY WITH "AV" AND SIZE OF AIR/VAC ASSEMBLY ON SIDE FACING ROADWAY IN 2" BLACK LETTERS.
6. AIR/VAC RELEASE VALVE ASSEMBLY SHALL BE INSTALLED AT HIGH POINT ON LINE. IF HIGH POINT FALLS IN LOCATION WHERE ASSEMBLY CANNOT BE INSTALLED, PROVIDE ADDITIONAL DEPTH TO CREATE NEW HIGH POINT.
7. CONCRETE VAULT PENETRATIONS SHALL BE CORE DRILLED AND GROUTED.

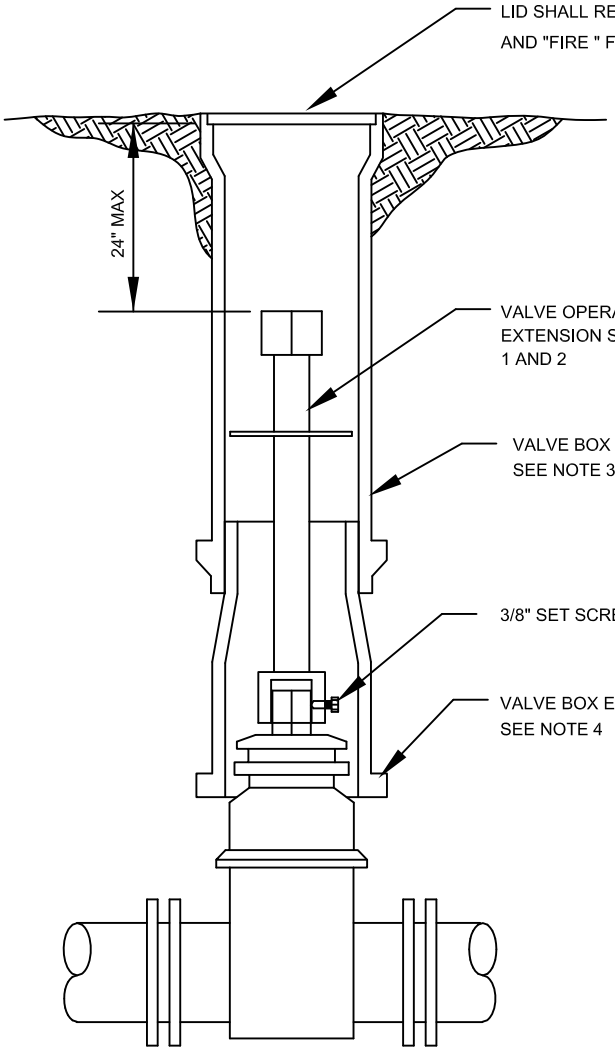
SNOHOMISH WASHINGTON

1" AIR VACUUM VALVE ASSEMBLY

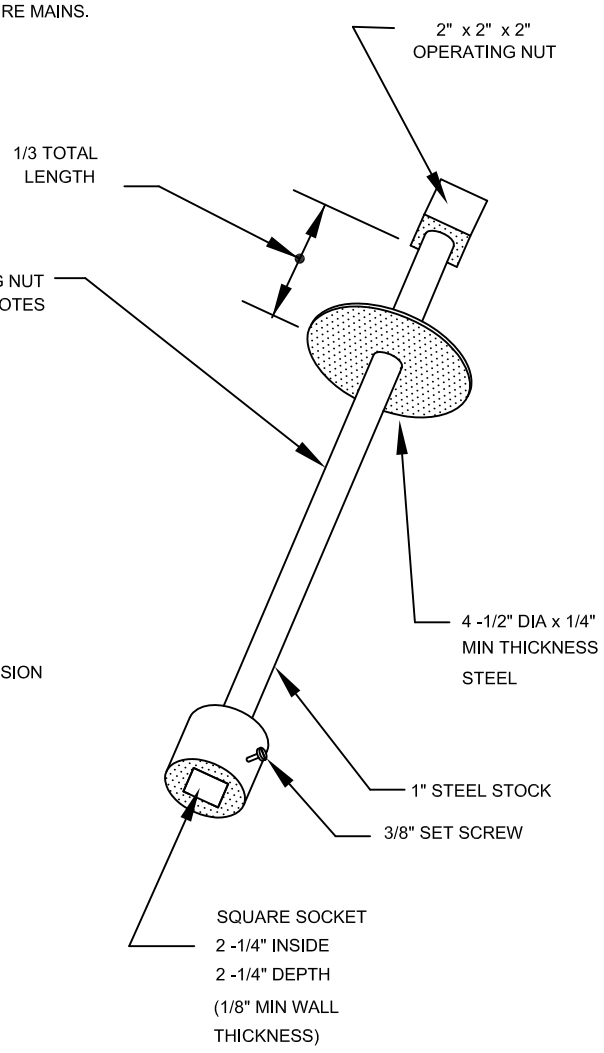
Approved By: S L S
City Engineer
Date: April 2010

511
Number

City of Snohomish Public Works Department



VALVE BOX AND EXTENSION



VALVE OPERATING NUT EXTENSION

NOTES:

1. VALVE OPERATING NUT EXTENSIONS ARE REQUIRED WHEN THE VALVE NUT IS MORE THAN THREE (3) FEET BELOW FINISHED GRADE. EXTENSIONS ARE TO BE A MINIMUM OF ONE (1) FOOT LONG. ONLY ONE EXTENSION WILL BE ALLOWED PER VALVE.
2. ALL VALVE OPERATING NUT EXTENSIONS ARE TO BE MADE OF STEEL,SIZED AS NOTED AND PAINTED WITH TWO (2) COATS OF METAL PAINT.
3. VALVE BOXES IN PAVED AREAS SHALL BE CAST IRON, TWO PIECE UNITS, DESIGNED WITH (LUGS) ON COVER, AS MANUFACTURED BY EAST JORDAN IRON WORKS OR APPROVED EQUAL.
4. APPROVED VALVE BOX EXTENSIONS SHALL BE INSTALLED AT THE TOP OF THE BONNET.
5. ALIGN TABS OR PICK HOLES WITH THE DIRECTION OF THE WATER MAIN.

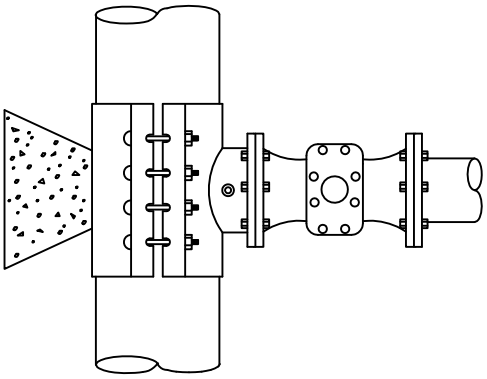
SNOHOMISH
WASHINGTON

WATER VALVE BOX AND EXTENSION

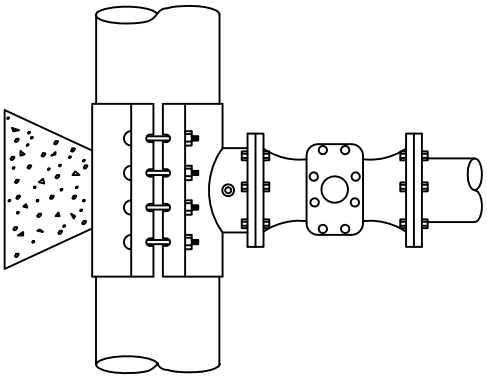
Approved By:
S L S
City Engineer
Date: April 2010

512
Number

City of Snohomish Public Works Department



PLAN



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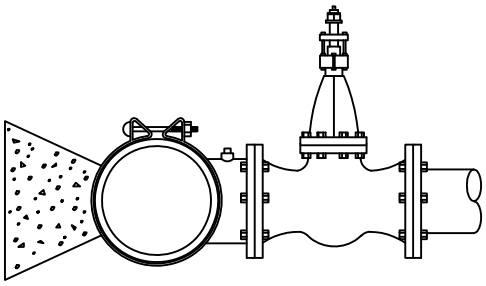
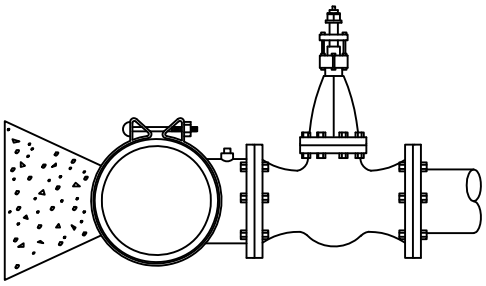


ELEVATION



ELEVATION

STEEL FABRICATED EPOXY COATED TAPPING SLEEVE

INSTALLED ON DI PIPE

FABRICATED STAINLESS STEEL TAPPING SLEEVE

INSTALLED ON AC PIPE

NOTES:

1. STAINLESS STEEL TAPPING TEES SHALL HAVE FULL CIRCLE SEAL.
2. STEEL TAPPING TEES SHALL BE EPOXY COATED.
3. ALL TEES AND VALVES TO BE WATER TESTED BEFORE TAPPING.
4. NO SIZE ON SIZE TAPS. TAP SHALL BE AT LEAST 2" SMALLER THAN THE EXISTING MAIN.

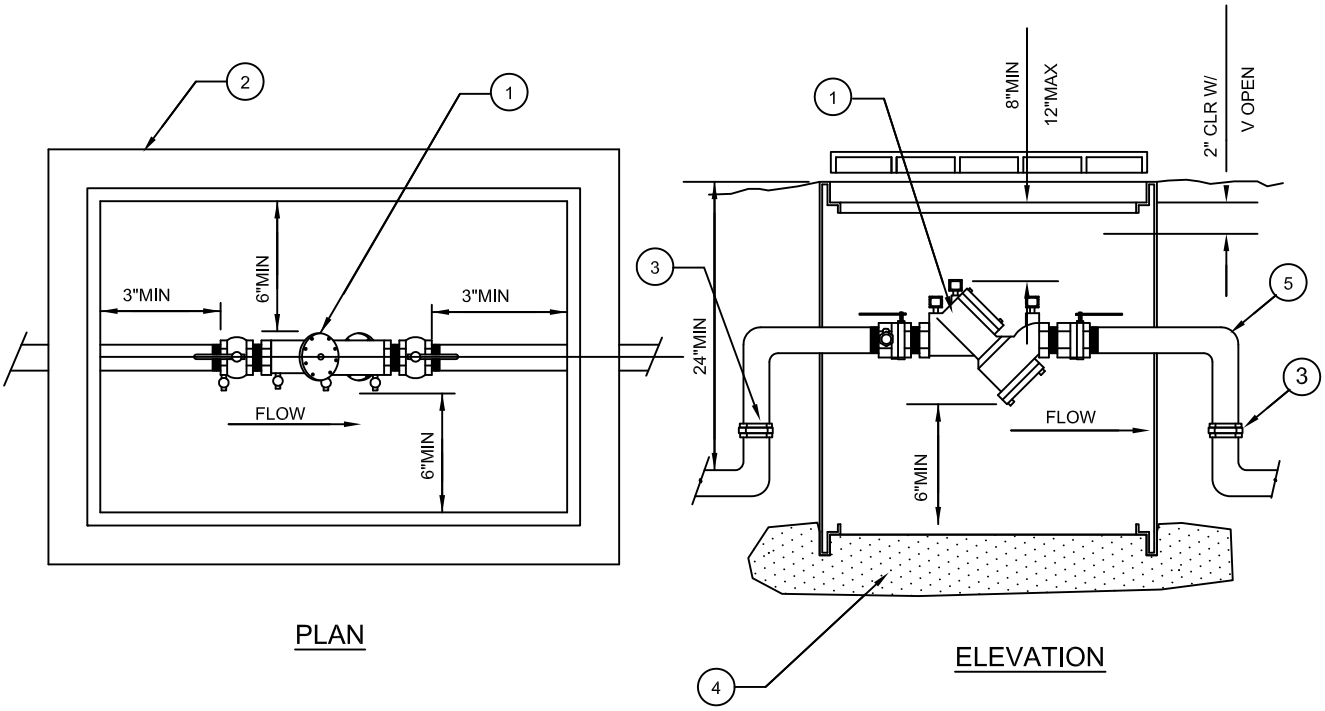
SNOHOMISH WASHINGTON

TAPPING TEES

Approved By:
S L S
City Engineer
Date: April 2010

513
Number

City of Snohomish Public Works Department



MATERIAL LIST:

1. WASHINGTON STATE APPROVED DOUBLE CHECK VALVE ASSEMBLY.
2. IN NON-TRAFFIC AREAS USE:

   FOR 3/4" TO 1" ASSEMBLIES, USE A MID STATE MSBCF 1324-12.

   FOR 1-1/4" TO 2" ASSEMBLIES, USE A MID STATE MSBCF 1730-18.

   IN TRAFFIC AREAS:

   A TRAFFIC LOADED BOX MUST BE USED AND LOCATION APPROVED BY THE CITY OF SNOHOMSIH PRIOR TO INSTALLATION
3. PROVIDE TWO UNIONS.
4. IF A DAYLIGHT DRAIN CANNOT BE PROVIDED THERE MUST BE A 12" MIN LAYER OF FREE DRAINING GRAVEL AT THE BOTTOM OF BOX.
5. ANGLES MAY BE IN OR OUT OF BOX SO LONG AS SUFFICIENT ROOM IS ALLOWED AT EACH END FOR VALVE OPERATION AND DCVA REPAIR OR MAINTENANCE.

NOTES:

1. INSTALL ACCORDING TO MANUFACTURER'S SPECIFICATIONS.
2. INSTALL ONLY IN HORIZONTAL CONFIGURATION, UNLESS APPROVED OTHERWISE BY CITY ENGINEER.
3. INSTALL TEST COCKS FACE UP OR TO ONE SIDE.
4. INSTALL BRASS PLUGS IN ALL TEST COCKS.
5. FOR 3/4" TO 1" ASSEMBLIES, USE A MIDSTATES 1324-12.
6. FOR 1-1/4" TO 2" ASSEMBLIES, USE A MID STATE MSBCF 1730-18.
7. ASSEMBLY REQUIRES CERTIFICATION UPON INSTALLATION AND RECERTIFICATION ANNUALLY.
8. ALL MINIMUM CLEARANCES MUST BE MET.
9. ALL BRANCH CONNECTIONS SHALL BE LOCATED DOWN STREAM OF THE ASSEMBLY. ANY BRANCH CONNECTION ON THE SUPPLY SIDE OF THE ASSEMBLY MUST BE PROTECTED BY A SEPARATE APPROVED BACKFLOW PREVENTION ASSEMBLY.

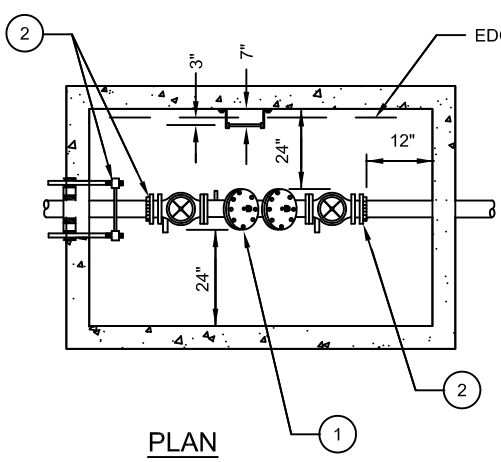
SNOHOMISH
WASHINGTON

DOUBLE CHECK VALVE ASSEMBLY (DCVA) FOR 2" AND SMALLER

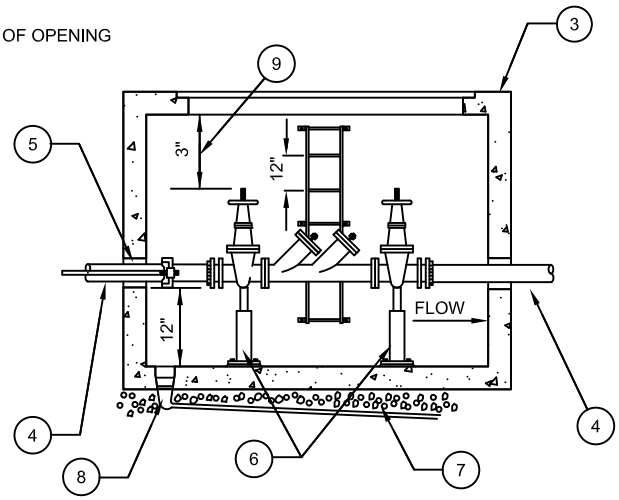
Approved By:
S L S
City Engineer
Date: April 2010

514
Number

City of Snohomish Public Works Department



PLAN



ELEVATION

MATERIAL LIST:

1. WASHINGTON STATE APPROVED DOUBLE CHECK VALVE ASSEMBLY INCLUDING: 2 EACH 0.S.& Y RESILIENT SEATED GATE VALVES, AND FOUR RESILIENT SEATED TEST COCKS.
2. UNI-FLANGE WITH SET SCREWS OR MJ x FL ADAPTER WITH MEGALUG OR GALVANIZED SHACKLE TO MAIN WITH TWO 3/4" RODS, OR MJ RETAINER GLANDS.
3. PRECAST CONCRETE VAULT WITH STEEL ACCESS HATCH (AS MANUFACTURED BY UTILITY VAULT CO OR AN APPROVED EQUAL). COVER TO READ "WATER". PROVIDE OSHA APPROVED HOT DIPPED GALVANIZED STEEL LADDER. INSTALL LADDER IN SUCH A WAY AS TO PROVIDE VAULT ACCESS THAT DOES NOT INTERFERE WITH INSTALLED EQUIPMENT OR MAINTENANCE THEREOF. PROVIDE A SPRING LOADED TRAFFIC LOAD RATED DOUBLE DOOR.
4. DUCTILE IRON PIPE (SIZED AS REQUIRED) CLASS 52.
5. WATER TIGHT GROUT SHALL BE USED IN ALL VAULT PENETRATIONS.
6. 2 EACH GALVANIZED OR STAINLESS STEEL ADJUSTABLE PIPE SUPPORTS FOR 2-1/2" AND LARGER PIPE.
7. GRAVEL FOUNDATION AS REQUIRED.
8. DRAIN, SLOPE TO DAYLIGHT WHEN POSSIBLE. PROVIDE A SUMP PER STANDARD PLAN 514 IF DAYLIGHT DRAIN CANNOT BE PROVIDED.
9. 3" MIN CLEARANCE FROM UNDERSIDE OF VAULT LID TO STEM AND OS&Y WHEN FULLY OPEN.

NOTES:

1. TEE AND GATE VALVE REQUIRED ON MAIN.
2. ASSEMBLY REQUIRES CERTIFICATION UPON INSTALLATION AND RECERTIFICATION ANNUALLY.
3. TEST COCKS ARE REQUIRED TO BE PLUGGED.
4. MAXIMUM HEIGHT OF ASSEMBLY IS FIVE FEET UNLESS AN OSHA APPROVED PLATFORM IS PROVIDED.
5. MINIMUM INSIDE VAULT HEIGHT IS 78", OR AS APPROVED BY THE ENGINEER, FOR 2-1/2" SERVICE AND LARGER.
6. METER SHALL BE INSTALLED SUCH THAT IT CAN BE READ WITHOUT ENTERING VAULT WITH ACCESS HATCH OPEN.
7. ALL DIMENSIONS ARE MINIMUM CLEARANCE REQUIREMENTS.

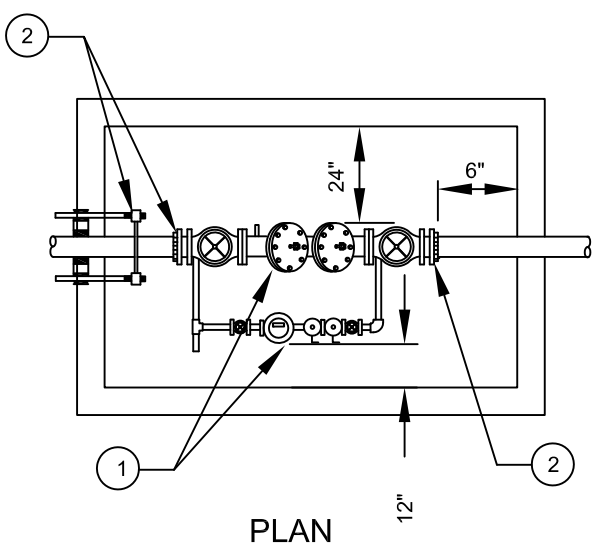
SNOHOMISH
WASHINGTON

DOUBLE CHECK VALVE ASSEMBLY (DCVA) FOR 2-1/2" AND LARGER

Approved By:
S L S
City Engineer
Date: April 2010

515
Number

City of Snohomish Public Works Department



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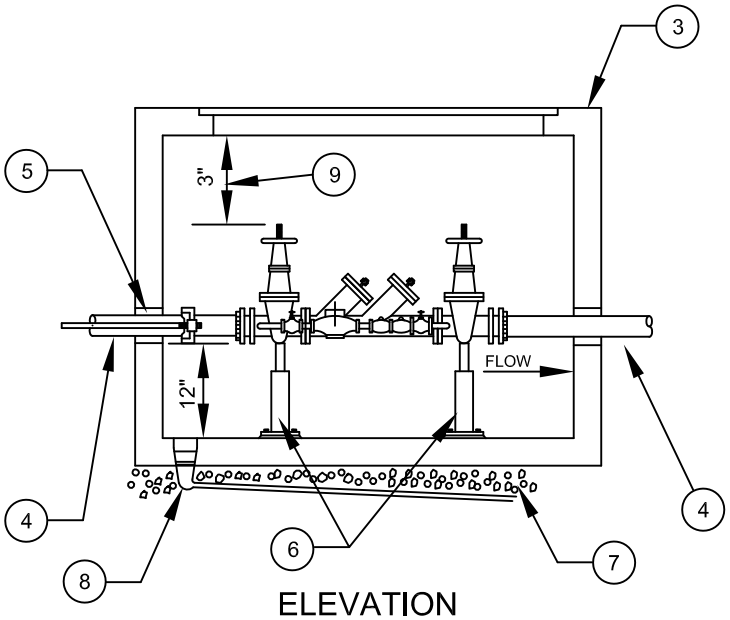


ELEVATION

LEGEND

1. WASHIINGTON STATE APPROVED DOUBLE CHECK DETECTOR VALVE ASSEMBLY METER TO READ IN CUBIC FEET.
2. UNI-FLANGE WITH SET SCREWS OR MJ x FL ADAPTOR WITH MEGALUG OR GALVANIZED SHACKLE TO MAIN WITH 2-3/4" RODS, OR MJ RETAINER GLANDS.
3. PRECAST CONCRETE VAULT WITH STEEL ACCESS HATCH (AS MANUFACTURED BY UTILITY VAULT CO OR AN APPROVED EQUAL). PROVIDE OSHA APPROVED HOT DIPPED GALVANIZED STEEL LADDER INSTALL IN SUCH A WAY THAT VAULT ACCESS DOES NOT INTERFERE WITH INSTALLED EQUIPMENT MAINTENANCE. PROVIDE SPRING LOADED, TRAFFIC LOAD RATED DOUBLE DOOR.
4. DUCTILE IRON PIPE (SIZED AS REQUIRED) CLASS 52.
5. WATER TIGHT GROUT SHALL BE USED IN ALL VAULT PENETRATIONS.
6. 2 - GALVANIZED ADJUSTABLE PIPE SUPPORTS FOR 2 1/2" DIAM AND LARGER PIPE.
7. GRAVEL FOUNDATION AS REQUIRED.
8. DRAIN, SLOPE TO DAYLIGHT WHEN POSSIBLE. INSTALL SUMP PER STANDRD PLAN 518.
9. 3" MIN CLEARANCE FROM UNDERSIDE OF VAULT LID TO STEM OF OS&Y WHEN FULLY OPEN.

NOTES

1. TEE AND GATE VALVE REQUIRED ON MAIN.
2. SINGLE DETECTOR CHECKS ARE NOT APPROVED BACKFLOW PREVENTION DEVICES.
3. ASSEMBLY REQUIRES TESTING UPON INSTALLATION, RETROFITTING, MOVING OR REPAIR AND TESTED ANNUALLY.
4. TEST COCKS ARE REQUIRED TO BE PLUGGED.
5. MAXIMUM HEIGHT OF ASSEMBLY IS FIVE FEET UNLESS AN OSHA APPROVED PLATFORM IS PROVIDED.
6. MINIMUM INSIDE VAULT HEIGHT IS 78" OR AS APPROVED BY THE CITY ENGINEER FOR 3" SERVICE AND LARGER.
7. METER SHALL BE INSTALLED SUCH THAT IT CAN BE READ WITHOUT ENTERING VAULT WITH ACCESS HATCH OPEN.
8. ALL DIMENSIONS ARE MINIMUM CLEARANCE REQUIREMENTS.

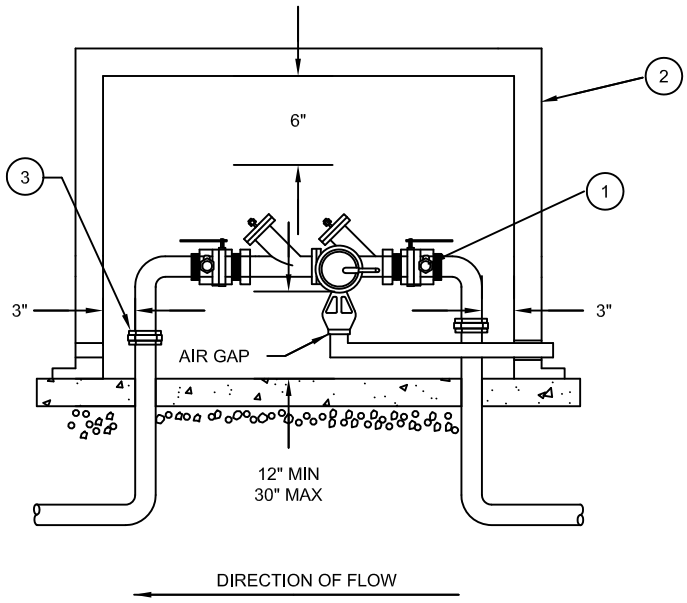
SNOHOMISH WASHINGTON

DOUBLE CHECK DETECTOR VALVE ASSEMBLY (DCDA) 2 1/2" AND LARGER

Approved By:
S L S
City Engineer
Date: April 2010

516
Number

City of Snohomish Public Works Department



ELEVATION

MATERIAL LIST:

1. WASHINGTON STATE APPROVED REDUCED PRESSURE BACKFLOW ASSEMBLY
2. INSULATED PROTECTIVE ENCLOSURE (HOT BOX) REQUIRED FOR OUTSIDE INSTALLATIONS. THE PROTECTIVE ENCLOSURE MUST BE PROVIDED WITH DRAINS AT BOTH ENDS OF THE BOTTOM SUFFICIENTLY SIZED TO PROVIDE FREE GRAVITY DRAINAGE OF MAXIMUM DISCHARGE OF RELIEF VALVE PORT (2" MIN).
3. 90° ELBOW WITH A CLOSE NIPPLE AND UNION ON VERTICAL.

NOTES:

1. ASSEMBLY REQUIRES CERTIFICATION UPON INSTALLATION AND RECERTIFICATION ANNUALLY, BY OWNER.
2. THE ENCLOSURE MUST BE INSTALLED ON A 4" THICK CONCRETE PAD.
3. AN ELECTRICAL OUTLET MUST BE PROVIDED.
4. GUARD POSTS SHALL BE INSTALLED IF LOCATED IN A TRAFFIC AREA.
5. ALL BRANCH CONNECTIONS SHALL BE LOCATED ON THE DOWNSTREAM SIDE OF THE ASSEMBLY.

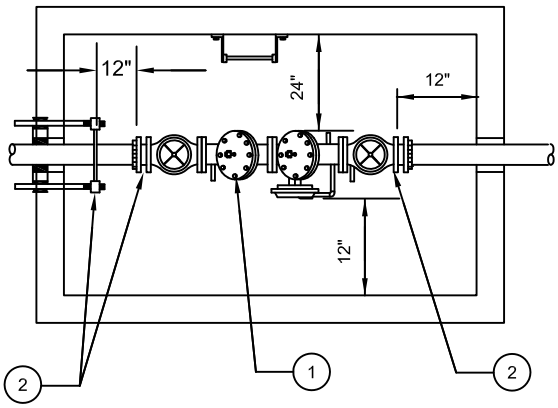
SNOHOMISH
WASHINGTON

REDUCED PRESSURE BACKFLOW ASSEMBLY (RPBA) 2" AND SMALLER

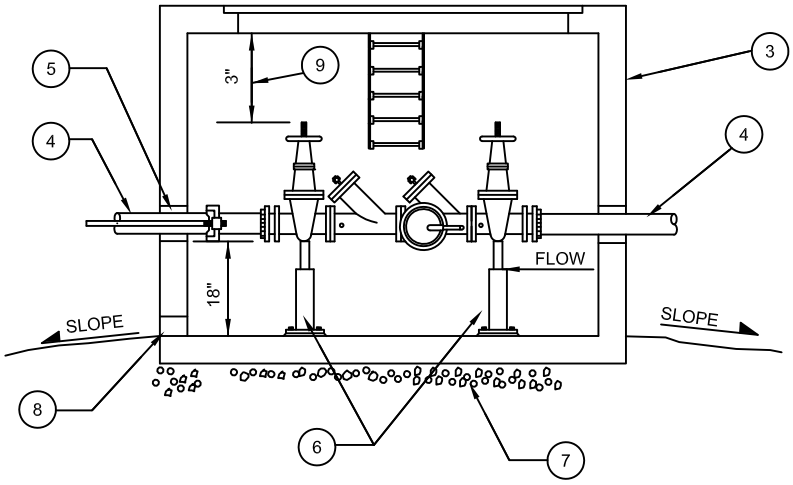
Approved By:
S L S
City Engineer
Date: April 2010

517
Number

City of Snohomish Public Works Department



PLAN



ELEVATION

MATERIAL LIST:

1. WASHINGTON STATE APPROVED REDUCED PRESSURE BACKFLOW ASSEMBLY
2. UNI-FLANGE WITH SET SCREWS OR MJ x FL ADAPTOR WITH MEGALUG OR GALVANIZED SHACKLE TO MAIN WITH TWO 3/4" RODS, OR MJ RETAINER GLANDS.
3. INSULATED PROTECTIVE ENCLOSURE (HOT BOX) REQUIRED FOR OUTSIDE INSTALLATIONS. THE PROTECTIVE ENCLOSURE MUST BE PROVIDED WITH DRAINS AT BOTH ENDS OF THE BOTTOM AND SUFFICIENTLY SIZED TO PROVIDE FREE GRAVITY DRAINAGE OF MAXIMUM DISCHARGE OF RELIEF VALVE PORT (2" MIN.).
4. DUCTILE IRON PIPE (SIZED AS REQUIRED) CLASS 52.
5. WATER TIGHT GROUT SHALL BE USED IN ALL VAULT PENETRATIONS.
6. 2 EACH GALVANIZED ADJUSTABLE PIPE SUPPORTS.
7. 5/8" CRUSHED ROCK FOUNDATION AS REQUIRED.
8. DRAIN SHALL BE INSTALLED WITH APPROVED AIR GAP, BE ABLE TO BE BORE SIGHTED TO DAYLIGHT WHICH MUST BE ABOVE 100 YEAR FLOOD LEVEL. DRAIN WILL BE SIZED SO AS TO PROVIDE FREE GRAVITY DRAINAGE OF MAXIMUM DISCHARGE OF RELIEF VALVE PORT.
9. 3" MIN CLEARANCE FROM UNDERSIDE OF VAULT LID TO STEM OF OS&Y WHEN FULLY OPEN.

NOTES:

1. TEE AND GATE VALVE REQUIRED AT WATER MAIN.
2. TEST COCKS ARE REQUIRED TO BE PLUGGED.
3. MAXIMUM HEIGHT OF ASSEMBLY IS FIVE FEET UNLESS AN OSHA APPROVED PLATFORM IS PROVIDED.
4. MINIMUM INSIDE VAULT HEIGHT IS 78", OR AS APPROVED BY CITY ENGINEER, FOR 2-1/2" SERVICE AND LARGER.
5. INSIDE INSTALLATIONS SEE STD. DETAIL NO. W-120.
6. ALL DIMENSIONS ARE MINIMUM CLEARANCE REQUIREMENTS.
7. ASSEMBLY REQUIRES CERTIFICATION UPON INSTALLATION AND RECERTIFICATION ANNUALLY.

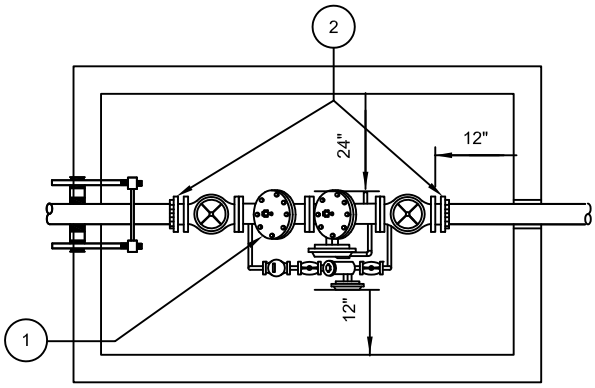
SNOHOMISH
WASHINGTON

REDUCED PRESSURE BACKFLOW ASSEMBLY (RPBA) 2-1/2" AND LARGER

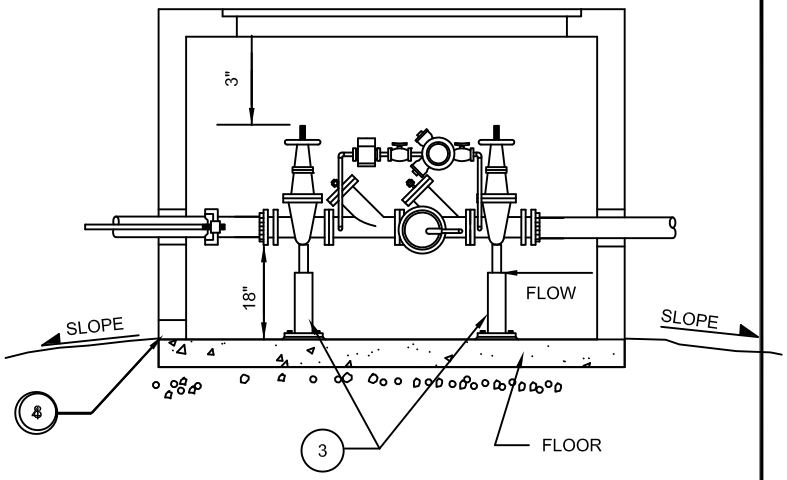
Approved By:
S L S
City Engineer
Date: April 2010

518
Number

City of Snohomish Public Works Department



PLAN



ELEVATION

MATERIAL LIST:

1. WASHINGTON STATE APPROVED REDUCED PRESSURE DETECTOR ASSEMBLY I (METER TO READ IN CUBIC FEET),
2. UNI-FLANGE WITH SET SCREWS OR MJ x FL ADAPTER WITH MEGALUG OR GALVANIZED SHACKLE TO MAIN WITH 2 EACH 3/4" RODS, OR MJ RETAINER GLANDS.
3. TWO - GALVANIZED ADJUSTABLE PIPE SUPPORTS FOR 2-1/2" DIAMETER AND LARGER PIPE.
4. DRAIN SHALL BE INSTALLED WITH APPROVED AIR GAP, BE ABLE TO BE BORE SIGHTED TO DAYLIGHT WHICH MUST BE ABOVE 100 YEAR FLOOD LEVEL. DRAIN WILL BE SIZED SO AS TO PROVIDE FREE GRAVITY DRAINAGE OF MAXIMUM DISCHARGE OF RELIEF VALVE PORT.

NOTES:

1. TEE AND GATE VALVE REQUIRED AT WATER MAIN.
2. TEST CLOCKS ARE REQUIRED TO BE PLUGGED IF ASSEMBLY IS INSTALLED UNDERGROUND.
3. ASSEMBLY REQUIRES CERTIFICATION UPON INSTALLATION AND RECERTIFICATION ANNUALLY, BY OWNER.
4. 2 1/2" AND LARGER RPDAS REQUIRE EITHER AN APPROVED BUILDING ENCLOSURE.
5. INSULATED PROTECTIVE ENCLOSURE (HOT BOX) REQUIRED FOR OUTSIDE INSTALLATIONS. THE PROTECTIVE ENCLOSURE MUST BE PROVIDED WITH DRAINS AT BOTH ENDS OF THE BOTTOM AND SUFFICIENTLY SIZED TO PROVIDE FREE GRAVITY DRAINAGE OF MAXIMUM DISCHARGE OF RELIEF VALVE PORT (2" MIN.).

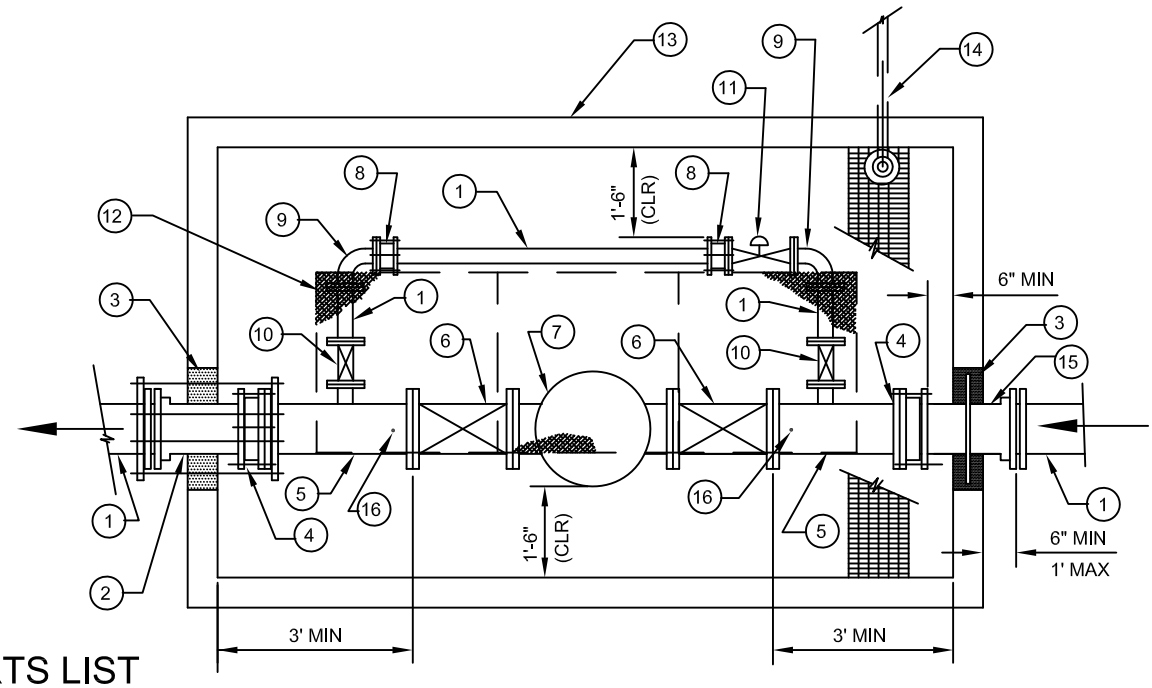
SNOHOMISH WASHINGTON

REDUCED PRESSURE DETECTOR ASSEMBLY (RPDA) 2 1/2"AND LARGER

Approved By:
S L S
City Engineer
Date: April 2010

519
Number

City of Snohomish Public Works Department



PARTS LIST

1. DUCTILE IRON PIPE
2. SPOOL (MJxPE) WITH SHACKLE BOLTS TO BE USED IN PRECAST VAULTS
3. NON-SHRINK GROUT
4. FLANGE COUPLING ADAPTOR (FLxMJ)
5. TEE (ALL FL)
6. GV CL 200 (FLxFL)
7. PRV (FLxFL)
8. FLANGE COUPLING ADAPTOR
9. 90° ELL (ALL MJ W/MEGA LUGS)
10. GV (FLxFL)
11. PRV (FLxFL)
12. UTILITY VAULT CO LID WITH TRAFFIC LOADED LOCKING STEEL COVERS OR EQUAL.
13. UTILITY VAULT CO PRECAST VAULT.
14. 2" GRAVITY SUMP DRAIN EXTEND TO DAY-LIGHT OR TO STORM DRAINAGE SYSTEM.
15. WALL SLEEVE (FLxPE) TO BE USED WITH CAST IN PLACE VAULTS.
16. 1/4" GAUGE TAPS WITH 1/4" BALL VALVES FOR ISOLATION.

NOTES

1. MINIMUM VAULT INSIDE HEIGHT SHALL BE 78", OR AS APPROVED BY THE ENGINEER.
2. MINIMUM CLEARANCE BETWEEN PRV VALVES AND FLOOR SHALL BE 12".
3. PROVIDE LIQUID FILLED 2 1/2" GAUGES AMETEK SERIES 550L OR EQUAL.
4. ALL EQUIPMENT MUST BE RATED FOR SOURCE PRESSURE.
5. PIPING AND VALVES SHALL BE SUPPORTED BY POURED-IN-PLACE CONCRETE OR STEEL STANDS. NUMBER OF AND PLACEMENT OF STANDS TO BE DETERMINED BY CITY ENGINEER ACCORDING TO VALVE SIZE.
6. BRAND, MINIMUM CLEARANCES, TYPE OF PRV AND ACCESSORIES TO BE DETERMINED BY CITY OF SNOHOMISH PUBLIC WORKS DIVISION.

SNOHOMISH
WASHINGTON

TYPICAL PRV INSTALLATION

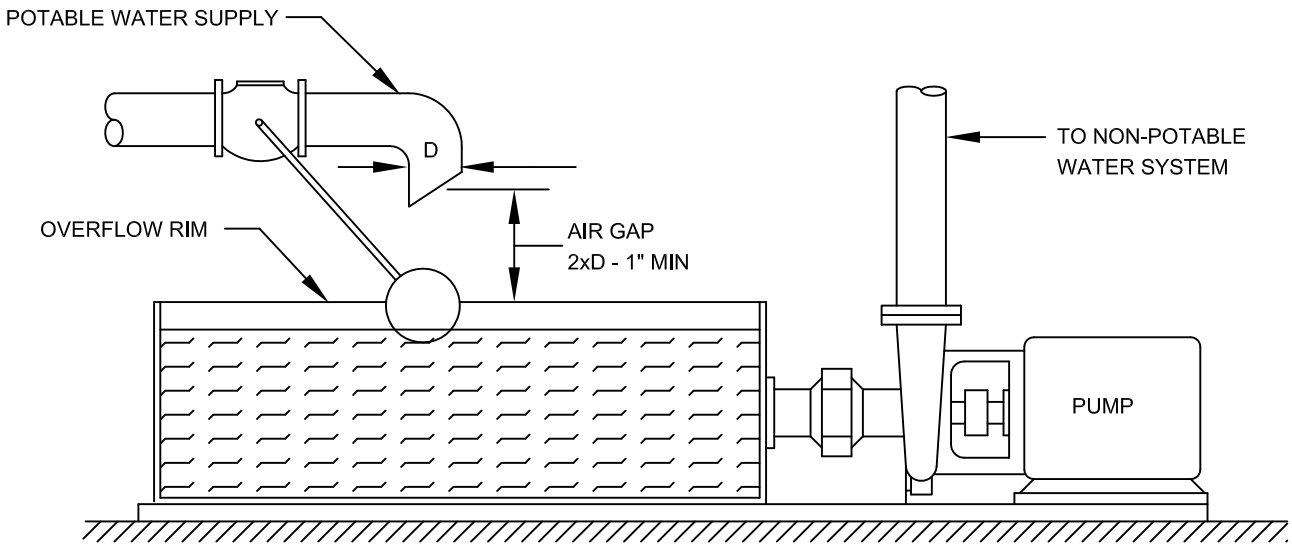
Approved By:
S L S
City Engineer
Date: April 2010

520
Number

City of Snohomish Public Works Department

APPROVED AIR GAP SEPARATION

AN APPROVED AIR GAP IS A PHYSICAL SEPARATION BETWEEN THE FREE FLOWING DISCHARGE END OF A POTABLE WATER SUPPLY PIPELINE AND THE OVERFLOW RIM OF AN OPEN OR NON-PRESSURE RECEIVING VESSEL. THESE VERTICAL, PHYSICAL SEPARATIONS MUST BE AT LEAST TWICE THE DIAMETER OF THE INLET PIPE BUT NEVER LESS THAN ONE INCH. IF SPLASHING IS A PROBLEM, TUBULAR SCREENS MAY BE ATTACHED OR THE SUPPLY LINE OUTLET MAY BE CUT AT A 45 DEGREE ANGLE. IF SUPPLY LINE IS CUT AT A 45 DEGREE ANGLE THE AIR GAP DISTANCE IS MEASURED FROM THE CENTER OF THE ANGLE. HOSES ARE NOT ALLOWED. BYPASSES ARE NOT ALLOWED. THE INSPECTION OF AIR GAPS SHALL BE INCLUDED IN THE YEARLY TESTING PROGRAM FOR BACKFLOW DEVICES.



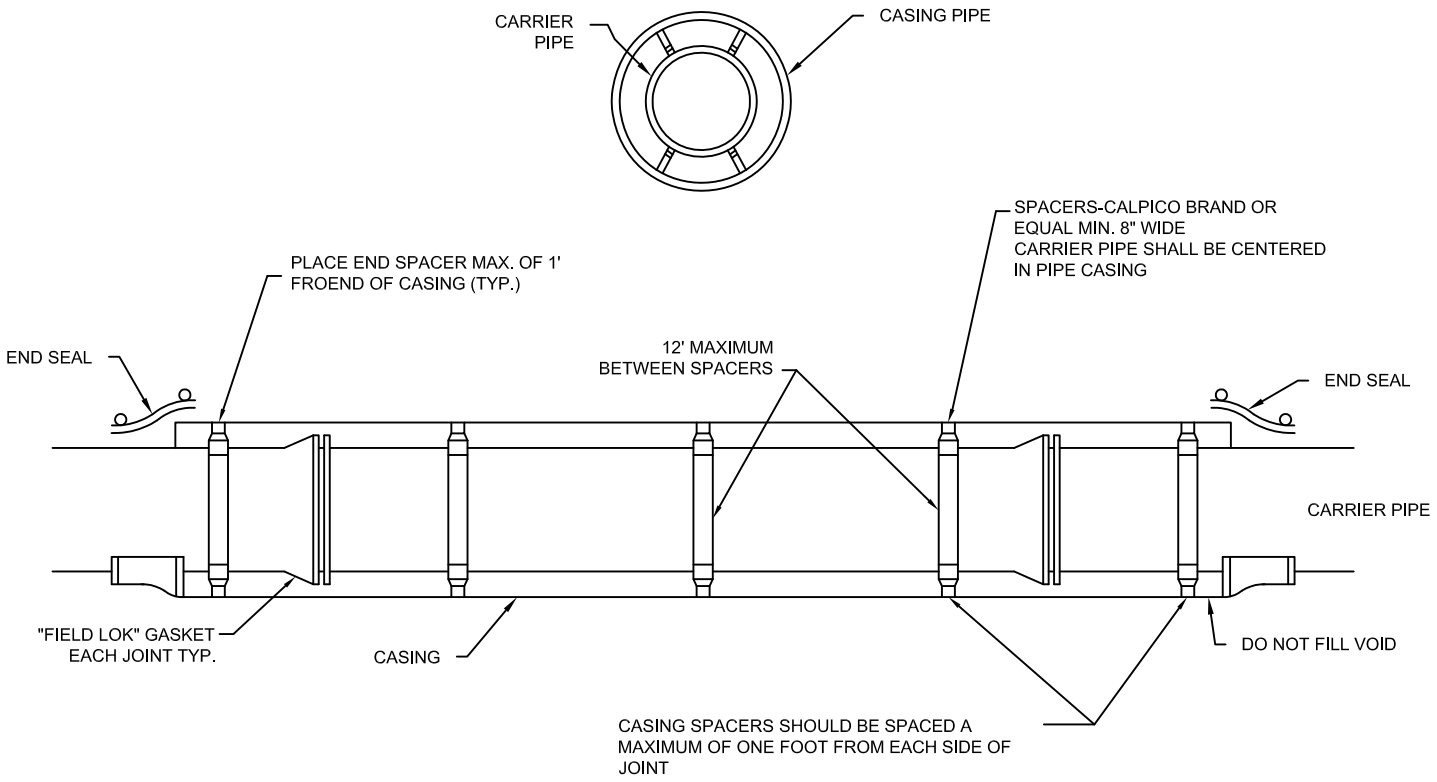
SNOHOMISH
WASHINGTON

AIR GAP FOR MAKEUP TANK

Approved By:
S L S
City Engineer
Date: April 2010

521
Number

City of Snohomish Public Works Department



CARRIER PIPE DIAMETER	4"	6"	8"	10"	12"
CASING DIAMETER (MJ/MEGALUG JOINT CARRIER PIPE)	14"	16"	18"	20"	22" *
STEEL CASING THICKNESS	0.25"	0.25"	0.25"	0.25"	0.25
SPACER BAND WIDTH	8"	8"	8"	8"	8"

* USE 24" DUCTILE IRON PIPE

NOTES:

1. ANY VARIATION TO THIS DETAIL SHALL BE REVIEWED AND APPROVED BY CITY ENGINEER.
2. CASING SHALL BE SEALED AT BOTH ENDS.
3. PIPE CASINGS SHALL EXTEND 6 FEET BEYOND THE EDGE OF PAVEMENT. IF THE RIGHT-OF-WAY IS NOT OWNED BY CITY OFSNOHOMISH, PIPE CASING SHALL EXTEND 6 FEET BEYOND RIGHT-OF-WAY.

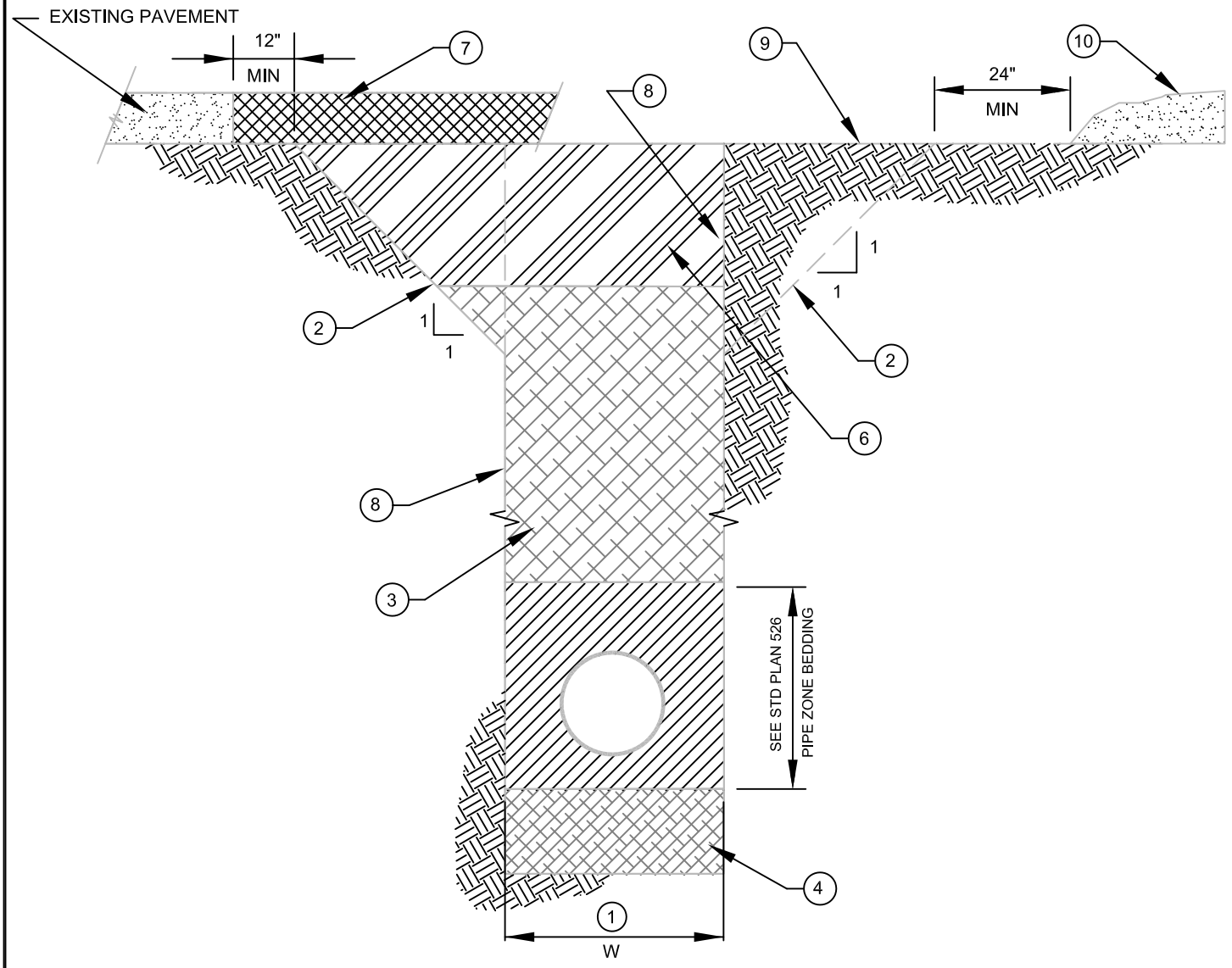
SNOHOMISH WASHINGTON

STEEL CASING FOR WATER MAIN

Approved By: S L S
City Engineer
Date: April 2010

522
Number

City of Snohomish Public Works Department



NOTES

1. W = MAXIMUM WIDTH OF TRENCH FOR PIPES 15" OR LESS IN DIA, W-40 FOR PIPES 18" OR GREATER, W = 1-1/2 X I.D. + 18".
2. ALTERNATE LAID-BACK TRENCH TO MEET O.S.H.A. REQUIREMENTS (NO SLOPES STEEPER THAN 1:1 EXCEPT FOR ROCKE).
3. NATIVE MATERIAL IF ALLOWED IN ADVANCE BY THE CITY ENGINEER, OR IMPORTED GRAVEL BORROW AS DIRECTED SHALL COMPACT TO 90% MAXIMUM DENSITY EXCEPT FROM SUBGRADE DOWN 4' WHICH SHALL BE CSTC.
4. CLASS A FOUNDATION GRAVEL, IF REQUIRED BY THE CITY ENGINEER TO REPLACE UNSUITABLE MATERIAL.
5. FOR ADDITIONAL COMPACTION INFORMATIONS SEE STANDARD PLAN 527
6. THE TOP 4' OF BACKFIL SHALL BE CSTC. THE CONTRACTOR HAS THE OPTION TO FILL THE ENTIRE EXCAVATION TO SUBGRADE WITH CSTC.
7. SEE CITY OF SNOHOMISH STANDARD PLAN 316 FOR PAVEMENT PATCH DETAILS.
8. VERTICAL TRENCH WALLS WITH SHORING SHALL CONFORM TO O.S.H.A. REGULATIONS.
9. SUBGRADE OR GROUND SURFACE IN NON-PAVED AREAS.
10. EXCAVATED NATIVE MATERIAL OR APPROVED STOCKPILED BACKFILL MATERILAL

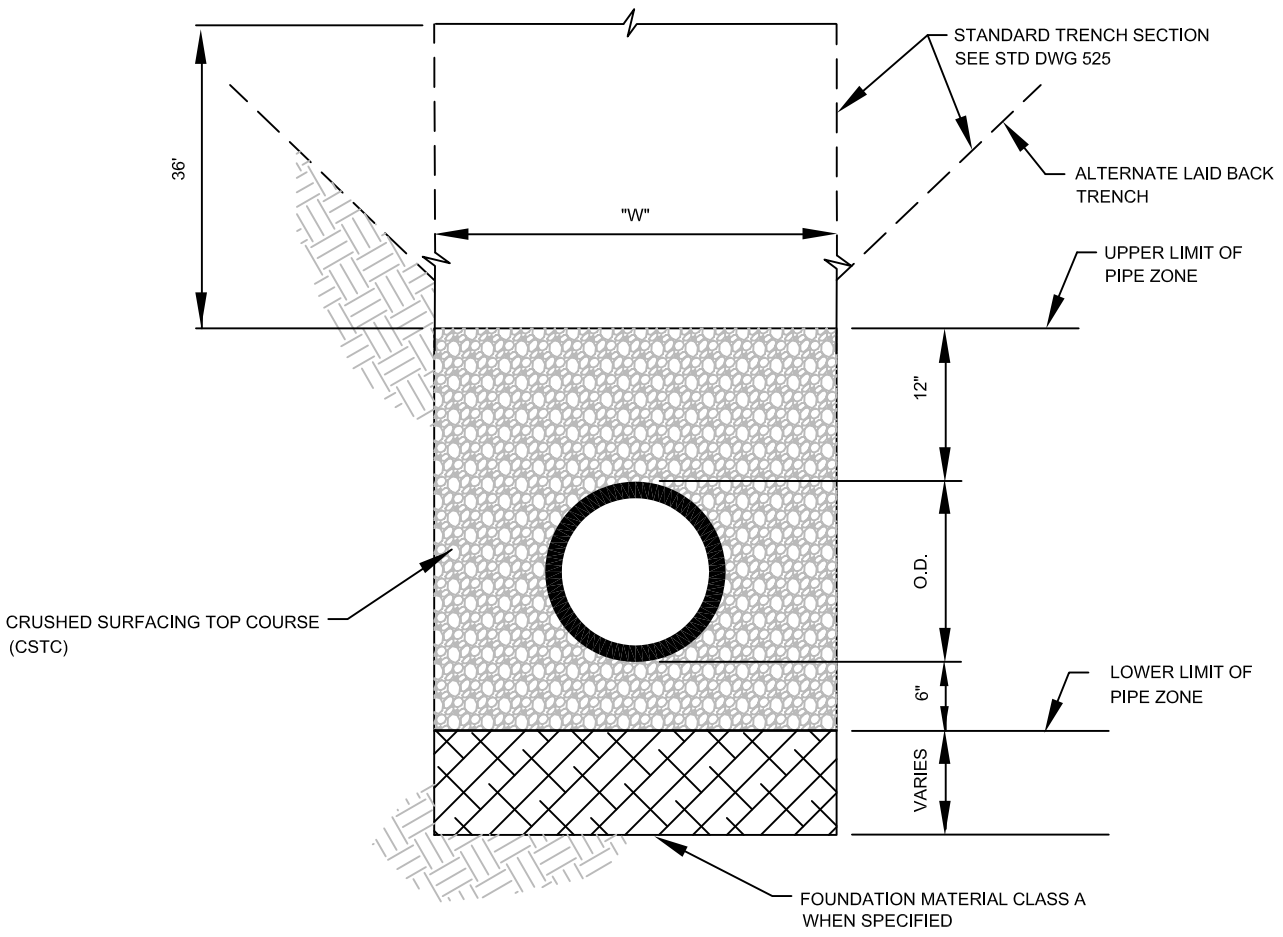
SNOHOMISH
WASHINGTON

TYPICAL WATER TRENCH SECTION

Approved By:
S L S
City Engineer
Date: April 2010

523
Number

City of Snohomish Public Works Department



LIMITS OF TRENCH:

W = MAXIMUM WIDTH OF TRENCH. FOR PIPES 15" OR LESS IN DIAMETER W = 40". FOR PIPES 18" OR GREATER W = 1 1/2 x I.D. + 18".

BEDDING AND FOUNDATION MATERIALS:

1. BEDDING MATERIAL SHALL BE CRUSHED SURFACING TOP COURSE CONFORMING TO SECTION 9-03.08(3) OF THE STANDARD SPECIFICATIONS FOR ROAD, BRIDGE AND MUNICIPAL CONSTRUCTION WSDOT/APWA
2. FOUNDATION MATERIAL, IF REQUIRED, SHALL BE FOUNDATION MATERIAL CLASS A CONFORMING TO SECTION 9-03.12 OF THE STANDARD SPECIFICATION FOR ROAD , BRIDGE AND MUNICIAPL CONSTRUCTION WSDOT/APWA.

PROCEDURE FOR COMPACTION:

1. PROVIDE UNIFORM SUPPORT UNDER BARREL.
2. COMPACT BEDDING MATERIAL TO 90% MAXIMUM DENSITY EXCEPT DIRECTLY OVER PIPE. HAND TAMP ONLY.
3. HAND TAMP UNDER HAUNCHES.
4. FOR ADDITIONAL COMPACTION INFORMATION SEE STANDARD PLAN NO. 527.

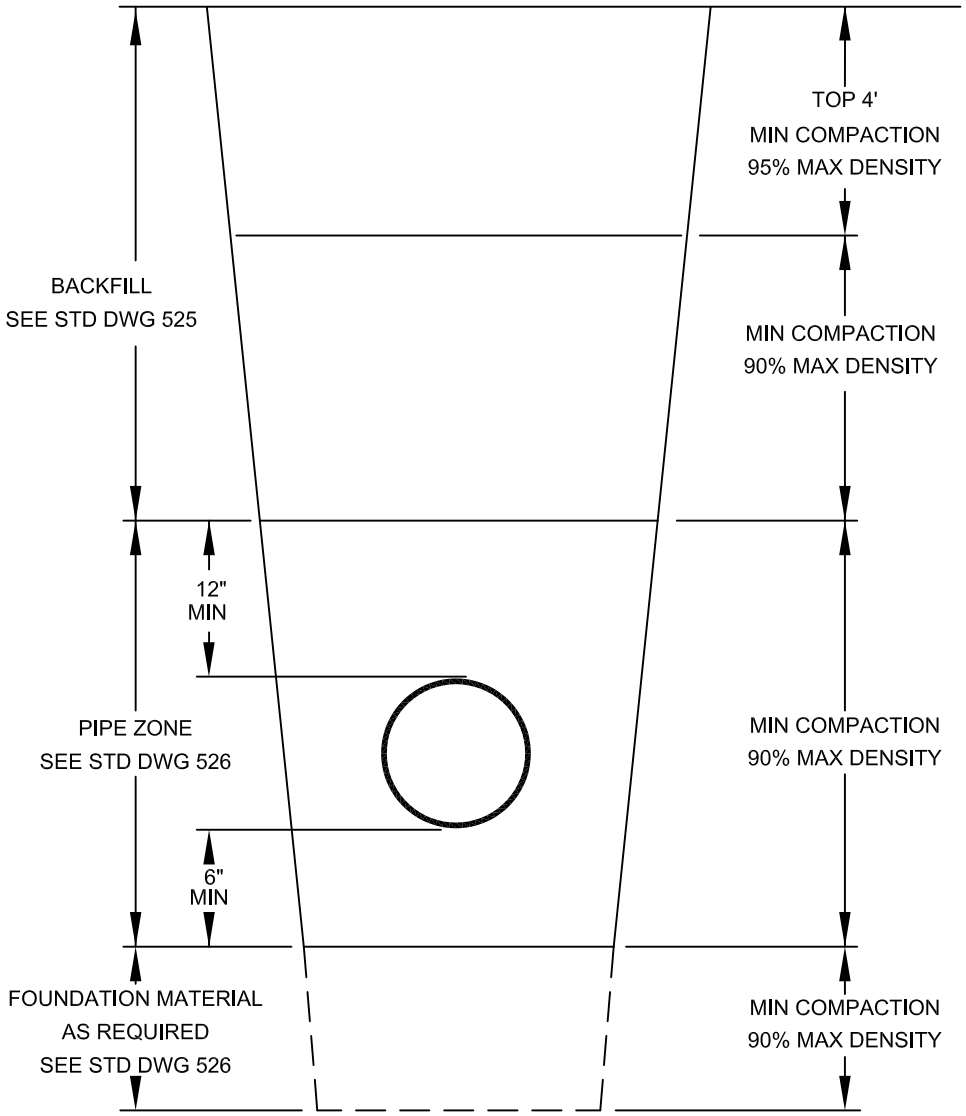
SNOHOMISH
WASHINGTON

BEDDING FOR WATER PIPE IN TRENCHES

Approved By:
S L S
City Engineer
Date: April 2010

524
Number

City of Snohomish Public Works Department



SECTION

NTS

NOTES:

1. ALL BACKFLL MATERIAL SHALL BE PLACED IN LIFTS NOT TO EXCEED 12 INCHES BEFORE COMPACTION UNLESS AUTHORIZED BY THE CITY ENGINEER DUE TO THE CHARACTER OF THE MATERIAL AND COMPACTING EQUIPMENT.
2. MECHANICAL COMPACTION OF BACKFILL MATERIAL SHALL NOT BEGIN UNTIL THE DEPTH OF COMPACTED MATERIAL IS 2 FEET ABOVE THE TOP OF PIPE.
3. EACH LIFT SHALL BE MECHANICALLY COMPACTED TO THE REQUIRED DENSITY PRIOR TO PLACING SUCCEEDING LIFTS OF BACKFILL MATERIAL
4. COMPACTON TESTS SHALL BE AS REQUIRED BY THE CITY ENGINEER, BUT IN NO CASE LESS THAN 2 TESTS EVERY 200 FEET OF TRENCH LENGTH (ONE AT SUBGRADE AND ONE AT 50% OF TRENCH DEPTH).
5. IN PLACE DENSITY WILL BE DETERMINED BY ONE OR MORE OF THE FOLLOWING METHODS
   ASTM D1556 - TEST FOR DENSITY OF SOIL IN PLACE BY THE SAND CONE METHOD.
   ASTM D2167 - (RUBBER. BALLON METHOD)
   ASTM D2922 - (NUCLEAR METHOD)
6. LABORATORY DENSITY WILL BE DETERMINED BY ASTM D698, MOISTURE DENSITY RELATIONS OF SOILS AND SOIL AGGREGATE MIXTURES.

SNOHOMISH
WASHINGTON

TYPICAL WATER TRENCH COMPACTION

Approved By:
S L S
City Engineer
Date: April 2010

525
Number

City of Snohomish Public Works Department

ENGINEERING STANDARDS

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ENGINEERING STANDARDS

SECTION 6

SANITARY SEWER

6-1 DESIGN

6-1.01 GENERAL

These sewer engineering standards (Standards) set forth minimum standards for the planning, design, and construction of sanitary sewer facilities in the City of Snohomish.

Although these Standards are intended to apply to physical development within the City, the standards may not apply for all situations. Compliance with these Standards does not relieve the party responsible for the project of the responsibility to apply conservative and sound professional judgment. These standards are minimum standards and are intended to assist, but not substitute for competent work by design professionals. The City Engineer may require more stringent requirements than would normally be required under these Standards due to special conditions and/or environmental constraints.

6-1.02 SEWER MAIN EXTENSION

Sewer main extensions shall be required when a parcel does not front an existing sewer main. It is the policy of the City of Snohomish that when practical all sewer main extensions shall be extended to a point 10 feet beyond the farthest property corner, or as directed by the City Engineer. The sewer shall be sized for the ultimate development of the tributary area and in accordance with the City's Sewer Comprehensive Plan. The City Engineer may reserves the right to require the installation of a larger size main if it is determined a larger size main is needed to meet future system demand requirements.

Sewer main replacement and upgrade shall be required when the existing sewer main is not adequate for the proposed use. Such criteria used to determine adequacy include but are not limited to age, pipe diameter, material type and overall condition of the existing sewer main. If the proposed development requires a lift station and/or force main upgrade, construction shall be at the expense of the party responsible for the development. Improvements shall be extended from the project to a point where the system is deemed reliable by the City Engineer.

Condominiums, townhomes, duplexes, triplexes and fourplexes with four living units or less shall have separate sewer service lines from each unit to the sewer main. All dwellings that exceed four units shall be reviewed on a case by case basis as to the number of sewer service lines that will be required.

Connections to existing trunk mains that are 12 inches or larger are prohibited unless approved in writing by the City Engineer. Connections to the Cemetery Creek sewer trunk main are prohibited without written approval from the Public Works Director.

If a project directly benefits other property owners, a reimbursement agreement with the City in accordance with SMC Chapter 15.17 may be appropriate.

6-1.03 MANHOLE

Manhole Location

Sewer manholes shall be located such that the center of the frame and cover shall be in the center of the traveled lane or as otherwise directed by the City Engineer.

If sewer main construction requires construction within an easement, the minimum easement width shall be 20 feet. Sewer mains shall be placed in the center of the easement and parallel to the easement lines unless otherwise directed by the City Engineer.

Manholes placed outside of the right-of-way shall maintain permanent unobstructed maintenance vehicle access at all times.

Manholes shall be installed at the end of each sewer main; at all changes in grade, size or alignment; and at all sewer main intersections. The distance between manholes on a sewer main run shall not be greater than 300 feet.

Manhole Diameter

The manhole diameter shall depend on size, location and the number of connections for pipes. The minimum diameter for manholes shall be 48-inches for a maximum pipe size of 12-inches; 54-inches for a maximum pipe size of 24-inches; 72-inches for a maximum pipe size of 36-inches and 96-inches for a maximum pipe size of 48-inches.

The minimum distance between cutout holes is 8-inches for a 48-inch or 54-inch manhole and 12-inches for a 72-inch or 96-inch manhole measured on the inside of the manhole. The cutout hole size shall be equal to the outer pipe diameter plus manhole wall thickness. A minimum access diameter of 24-inches shall be provided. Please reference Standard Plans 604, 606, 607, and 608.

Manhole Depth

The minimum sewer manhole depth shall be 7-feet. When manhole depths exceed 25-feet, the manhole base slabs shall be designed by a professional structural engineer licensed in the State of Washington.

Manhole Inverts

The drop in the invert elevation across a manhole shall typically range from 0.1 to 0.2 feet. The maximum allowable drop in the invert elevation of a manhole shall be 1.0 feet.

When a side sewer connects directly to a manhole, the invert of the side sewer shall be equal to or above the sewer main crown but shall not exceed 18-inches above the invert of the sewer main.

Manhole Covers

Manholes in unpaved areas and easements shall have hinged bolt-locking covers per Standard Plan 613. All manholes in paved areas and sidewalks shall also have standard bolt locking covers.

Cleanouts

Cleanouts in the public sewer system are not an acceptable substitute for manholes. Cleanouts shall be installed on 6-inch private side sewers. Cleanout locations and spacing for private side sewers are governed under Section 6-1.13 of this Section.

6-1.04 SEWER PIPE

Sewer Pipe Sizing

No public gravity sewer main conveying raw wastewater shall be less than 8-inches in diameter. The pipe diameter and slope shall be selected to obtain the greatest practical velocities to minimize settling problems. Oversize sewers will not be approved to justify flatter slopes.

New sewer mains shall be designed so that under complete collection system build out, peak flow including inflow/infiltration (I/I) shall not exceed 50% capacity of the main. Existing sewer mains shall not exceed a peak flow of 75% capacity to preserve capacity requirements of the existing sewer main. Existing sewer mains that will exceed 75% capacity when new flows are introduced shall be replaced and upsized by the party responsible for the development to preserve capacity requirements of the sewer main.

Storm drainage connections to the City sewer system are prohibited outside of the Combined Sewer Overflow area. This area is designated on the City's wastewater system map and may be viewed on the City's website.

Pipe Material

Sewer pipe shall be SDR 35 PVC conforming to ASTM D3034 where invert depths are from 5-feet to 14-feet. When invert depths are shallower than 5-feet or deeper than 14-feet, AWWA C900 PVC pipe shall be used. If the depth exceeds 14-feet on any portion

of the pipe segment, the entire segment between manholes shall be C900 including associated side sewers.

Sewer Depth

In general, gravity sewer mains shall be sufficiently deep to receive wastewater from the basements of buildings located in the area to be served by the sewer mains. Gravity sewer mains shall also be constructed sufficiently deep in consideration of future collection system development in the area to serve new construction. The minimum depth of gravity sewer is 5 feet.

Slope

Sewers shall be installed with uniform slope between manholes. All sewers shall be designed and constructed to give mean velocities of not less than 2.0 feet per second when flowing full, based on Manning's formula. The following are the minimum slopes, however slopes greater than these are desirable.

Table 6-1
Minimum Pipe Slope by Pipe Diameter

Sewer Pipe Diameter (inches)	Minimum Slope (% or feet per 100 feet)
4	2.00
6	1.00
8	0.40
10	0.28
12	0.22
15	0.15
18	0.12
21	0.10
24	0.08
27	0.07
30	0.06
36	0.05

Maximum sewer main slope shall not induce velocities greater than 10 feet per second under daily peak flows.

Pipe anchor blocks shall be installed where the sewer main slope exceeds 20%. Each pipe length shall have one anchor block spaced at 20-foot on center.

Hill holders shall be required on sewer mains when unpaved slopes exceed 20% grade. Minimum hill holder spacing shall be 20-feet on center.

Alignment

Sewer mains shall be installed with straight alignment between manholes. Curvilinear sewer mains are not permitted in the City of Snohomish.

Change in Pipe Size

Where a smaller sewer pipe joins a larger one, the invert of the larger sewer pipe at the manhole shall be lowered sufficiently to maintain the same energy gradient. An approximate method for securing these results is to place the crowns of both pipes at the same elevation.

6-1.05 STREAM CROSSINGS

Placement and construction of sewer main stream crossings shall be done in accordance with all applicable State and local laws and regulations, including but not limited to obtaining necessary permits from the Washington State Department of Fish and Wildlife and Department of Ecology (DOE).

Sewer systems shall be designed to eliminate or minimize the number of stream crossings. When a sewer main must cross a stream, the crossing shall be designed to cross the stream as nearly perpendicular to the stream flow as possible and shall be free from change in grade.

Sewer systems located along streams shall be located outside of the stream bed and sufficiently away from the stream to provide for future possible stream widening and to prevent pollution by siltation during construction. Sewer structures shall not interfere with the free discharge of flood flows of the stream.

The top of all sewers entering or crossing streams shall be at sufficient depth below the natural bottom of the stream bed to protect the sewer line. In general, the following cover requirements shall be met:

1. A minimum of 1-foot of cover above the top of the casing pipe if the sewer pipe is located in rock.

2. A minimum of 5-feet of cover above the top of the casing pipe in all other material.

6-1.06 HORIZONTAL SEPARATION FROM OTHER UTILITIES

Sewer mains shall be installed at least 10-feet horizontally from any existing or proposed water main. The distance shall be measured edge to edge. In cases where it is not practical to maintain a 10-foot separation, the City Engineer may allow deviation on a case-by-case basis using DOE criteria. Side sewer and water service lines shall have

minimum horizontal clearances of 10-feet unless otherwise approved by the City Engineer. The following table lists minimum required horizontal clearances from sewers:

Table 6-2

Horizontal Clearances from Sewer

Utility	Clearance from Sewer (in feet)
Cable	5
Gas	5
Power	5
Storm Drain	5
Telephone, Fiber Optic	5
Water	10

6-1.07 VERTICAL SEPARATION FROM OTHER UTILITIES

Sewers crossing water mains shall be installed to provide a minimum vertical distance of 18-inches between the outside of the sewer and the outside of the water main. The City prefers that the sewer main be installed below the water main. Where a sewer crosses a water main, one full length of water main (18-feet minimum) shall be used with the pipe centered over the sewer for maximum joint separation. When the above conditions cannot be met, the City Engineer may approve a variance, but shall require that the sewer be constructed of ductile iron or C-900 pipe, be encased and be pressure tested prior to activation. Washington State Department of Ecology requirements shall also apply.

Sewers crossing water mains shall be installed to provide a minimum vertical distance of 18-inches between the outside of the sewer and the outside of the water main. The City prefers that the sewer main be installed below the water main. Where a sewer crosses a water main, one full length of sewer main (18-feet minimum) shall be used with the pipe centered over the water main for maximum joint separation. When the above conditions cannot be met, the City Engineer may approve a deviation in accordance with applicable DOE requirements and shall also require that the sewer main constructed of ductile iron or C-900 pipe, be encased and be pressure tested prior to activation.

Table 6-3

Vertical Clearances from Sewer

Utility	Clearance from Sewer (in feet)
Cable	1
Gas	1
Power	1

Storm Drain	1
Telephone, Fiber Optic	1
Water	1.5

6-1.08 STEEL CASING

Sewer pipe shall be encased in a steel or ductile iron casing when crossing under improvements where the ability to remove and replace pipe without disturbance to the improvement is needed. Casings are required when:

1. Crossing under rockeries over 5-feet high;
2. Crossing under retaining wall footings over 5-feet wide;
3. Crossing under reinforced earth retaining walls;
4. Crossing under streams or wetlands; and
5. Crossing under railways and highways.

Casings shall extend a minimum of 5 feet past each edge of the structure, or a distance equal to the depth of pipe, whichever is greater. The carrier pipe shall be supported by casing spacers per Standard Plan 618. The minimum vertical clearance between the bottom of the wall (or footing) and top of the pipe (or casing) shall be 2 feet. The pipe trench at the casing shall be backfilled with gravel backfill material when the vertical clearance is less than 3 feet.

Ductile iron pipe shall be encased in a steel casing when crossing under a railroad or highway where open cut is not allowed. Casings shall extend a minimum of six feet (6') beyond the edges of the right-of-way. The casing pipe and carrier pipe shall be installed in accordance with the applicable Federal, State and local regulations. In the case of railroad crossings, the project shall also comply with regulations established by the railroad company. Casing spacers shall be placed under the carrier pipe to ensure approximate centering within the casing pipe and to prevent damage during installation. Voids between all steel casings and native soil shall be pressure grouted. See Standard Plan 618.

6-1.09 SEWER CONNECTIONS

A side sewer shall be connected to the sewer main with a 6 inch factory tee connection or at a manhole. Where an existing side sewer stub is not available, a ROMAC tapping saddle is required for an existing sewer main. See Standard Plan 603.

All new sewer mains shall connect to existing sewer mains at manholes. If an existing manhole is not available, a new saddle manhole on existing sewer main is required per Standard Plan 605.

Existing manhole diameter must be adequate to accommodate the new sewer mains. If the existing manhole is not adequate, the existing manhole shall be upgraded at the expense of the party responsible for the project. If the existing manhole access is less than 24 inches in diameter, and/or concentric cone (manhole over 7 feet deep), the manhole shall be upgraded to include new 24 inch ring and cover and/or eccentric cone. At the connection to the existing sewer system, new sewer connections shall be physically plugged until all tests have been completed and the City Engineer approves the removal of the plugs.

6-1.10 FAT, OIL, AND GREASE SEPARATION

Oil/Water Separator

An oil/water separator is required whenever an industrial or commercial business generates or has the potential to generate fats, oils, or greases exceeding 100 milligrams per liter which will be discharged to the sewer system. An oil/water separation device shall be installed on the private property where the potential industrial or commercial source of the fat, oils, or greases is located by the property owner. Water discharged from any oil/water separator to the sewer system shall not contain more than 100 milligrams per liter of fats, oils or greases.

The oil/water separator shall be covered with removable sections. Access and inspection covers, weighing not more than 30 lbs., with suitable hand holds, shall be installed directly above the inspection “tee” and oil/grit collection compartments.

Only wastewater from floor drains and covered parking areas shall drain to the separator. The location and design shall eliminate the possibility of stormwater reaching the separator.

The separator shall be located within 20 feet from the driveway for access by maintenance vehicles.

A sampling tee shall be located on the outlet with a minimum 18 inch drop below the invert. Access to the separator shall be available for inspection and compliance determination sampling at all times.

When pre-treatment is no longer required, the inlet and outlet pipes shall be permanently plugged, the separation chambers pumped out, and the vault removed, or filled with compacted crushed rock or controlled density fill.

Grease Trap and Interceptor

The size and design of grease traps and interceptors and hydro-mechanical grease interceptors (HGI’s) shall conform to the Uniform Plumbing Code and shall be approved by the City Building Official. Traps and interceptors shall be located on private property outside the building within 20 feet of driveway for access by maintenance vehicles. An

HGI may be located inside the building, and shall remain privately owned and maintained at the owner's or occupant's expense. A maintenance program must be submitted and approved that includes maintenance, testing requirements and reporting intervals. These facilities shall be available for the inspection by the Building Official at all times.

When pre-treatment is no longer required, the inlet and outlet pipes shall be permanently plugged, the separation chambers pumped out, and the vault removed, or filled with compacted crushed rock or controlled density fill.

6-1.11 EASEMENTS

All public sewer mains, manholes, air valves, lift stations, and other appurtenances not in public right-of-way shall be within public utility easements designated on submitted plans to provide the City with permanent access to these facilities, as well as easements for future sewer extension, as required. Unless otherwise approved by the City Engineer, the minimum width of the easement for sewer mains and appurtenances shall be 20 feet.

Easement areas shall be kept free from obstructions at all times. No structures, driveways or landscaping shall be constructed within the utility easement. Maintenance vehicle access shall be constructed with a suitable driving surface approved by the City Engineer.

Utility easements shall be fully executed and recorded with the Snohomish County Auditor prior to project acceptance by the City. The form of easement documents shall be subject to the approval of the City Attorney. Easement drawings and legal descriptions shall be included as exhibits. If off-site easements are required on properties not owned by the party responsible for the project, the party shall acquire the necessary easements at their own expense before construction plans will be approved by the City Engineer.

6-1.12 SIDE SEWERS

A side sewer stub shall extend from the main line to 10 feet past the edge of the property line. A side sewer stub shall also extend an additional 5 feet beyond any easements including the standard 10 feet utility easement required on lots fronting public right-of-way. Pipe of 6-inches in diameter shall be used within the public right-of-way or easement unless expected flow requires a larger size of line. See Standard Plan 601.

All residential side sewer lines on private property from the end of the 6 inch diameter stub to the building may be a minimum of 4 inches in diameter with a single connection per lot. Condominiums, town homes, duplexes, triplexes and fourplexes with four living units or less shall have separate sewer service lines from each unit to the main.

For a multi-family development four-plex and larger, a side sewer for each separate building is required and must be at least 6-inches in diameter. For side sewers serving more than ten units or serving more than one building, side sewers shall be a minimum of 8-inches in diameter and must be connected to a manhole.

Maximum distances between side sewer clean-outs shall be 100 feet. All side sewer cleanouts on commercial and multi-family developments shall include at grade access with covers per the City Standard Plan 602.

A 6-inch minimum pipe shall be used for all commercial side sewers unless otherwise directed by the City Engineer.

6-1.13 SEPTIC TANKS

Septic systems are generally not allowed. The City Engineer may approve a septic tank system if he/she determines that public sewer service is not available or it is not "practical" to provide public sewer service and the party has obtained the necessary permits from the Snohomish Health District.

6-1.14 PRIVATE GRINDER PUMP

Use of a grinder pump requires approval by the City Engineer and will be evaluated on a case-by-case basis. The party responsible for the project shall demonstrate to the satisfaction of the City Engineer that there is no other feasible means of sewer service available prior to grinder pump approval. The City Engineer may require the applicant to deepen the existing gravity sewer at their expense to eliminate the need for a grinder pump.

The design engineer shall specify pumps with proper flow rate and dynamic head and provide pump curves from the manufacturer to the City Engineer for review and approval. The minimum diameter of the force main shall be 2 inches (Schedule 80 PVC or approved equal). Interior grinder pump systems shall meet the requirements of the current Uniform Plumbing Code. Exterior grinder pumps systems are subject to the approval of the City Engineer.

6-1.15 LIFT (PUMP) STATION

All sewers shall be gravity when possible. Use of lift stations requires approval by the City Engineer and will be evaluated on a case-by-case basis. The City may require the applicant to deepen the existing gravity sewer at their expense to eliminate the need for pumping. The applicant shall demonstrate that there is no other feasible means of sewer service available.

Gravity flow sewer lines shall be used when possible. Use of lift stations requires approval by the City Engineer and will be evaluated on a case-by-case basis. The City Engineer may require the party responsible for the project to deepen the existing gravity sewer at their expense to eliminate the need for pumping. The party responsible shall demonstrate to the satisfaction of the City Engineer that there is no other feasible means of sewer service available prior to lift station approval.

Lift station design parameters are in Appendix 6A of these Engineering Standards.

6-1.16 MONITORING MANHOLE

Monitoring manholes are required for all industrial/commercial applications and other applications as determined by the City Engineer. The monitoring manhole shall be in an accessible location for inspection by City staff at any time and under all weather conditions. The depths of monitoring manholes shall be 4 feet minimum and 8 feet maximum. If the depth is less than 7 feet, a flat top manhole shall be used. Monitoring manholes shall be 48 inch Type I (or larger) with locking rings and covers. The rim elevations of the monitoring manholes shall be set to finished grade or as directed by the City Engineer.

6-2 MATERIAL

All materials shall be new and undamaged. The same manufacturer of each item shall be used throughout the work. All materials not specifically referenced shall comply with applicable sections of ANSI, ASTM, AWWA, the current WSDOT/APWA Standard Specifications and these Standards, and approved by the City Engineer.

When specific manufacturers or models are specified in these Standards, no substitutions will be allowed without prior approval by the City Engineer. If required by the City Engineer, the party responsible for the project shall furnish certification from the manufacturer of the materials being supplied that the inspection and all of the specified tests have been made and the results thereof comply with the requirements of the reference standards.

The party responsible for the project shall provide construction material submittals for approval to the City Engineer.

6-2.01 MANHOLES

Manholes shall be constructed of pre-cast sections with confined O-ring rubber gasket joints, and with either a pre-cast base or a cast-in-place base in accordance with the City Standard Plans 604, 605, 606, 607 and 608. Any request to deviate from these details is subject to the review and approval of the City Engineer.

Manholes shall be constructed in accordance with AASHTO M-199 (ASTM C 478) unless otherwise shown on plans and approved by the City Engineer. All pre-cast concrete and reinforced cast-in-place concrete shall be Class 4000. Non-reinforced concrete in channel and shelf shall be Class 3000. Concrete blocks may only be used for final adjustment of the casting to final street grade. Pre-cast bases shall be furnished with cutouts or knockouts. Knockouts shall have a minimum wall thickness of 2 inches.

Grouting

All structures shall be grouted with non-shrink grout inside, outside, in-between sections and risers and under castings. Jet-Set shall not be used. All structures that are not

watertight shall be re-grouted by and at the expense of the party responsible for the work until such structures are watertight.

All base reinforcing steel shall have a minimum yield strength of 60,000 psi and be placed in the upper half of the base with one inch minimum clearance.

6-2.02 MANHOLE RING AND COVER

Ductile iron rings and cast iron rings and covers shall conform to the Standard Plan 612 and Section 9-05.15 of the current WSDOT/APWA Standard Specifications.

Manhole rings shall be gray iron conforming to the requirements of AASHTO M 105 Grade 30B. Manhole covers shall be ductile iron conforming to ASTM A536, GR 80-55-06, East Jordan Iron Works or an equivalent approved by the City Engineer. Rings and covers shall be tested for accuracy of fit and shall be locked down with a 5 ⅝ inch stainless steel socket head cap screws. All castings shall have a bituminous coating.

6-2.03 GRAVITY SEWER PIPE & FITTINGS

Sanitary sewer pipe shall be PVC or ductile iron meeting the following requirements unless otherwise directed by the City Engineer:

1) Polyvinyl Chloride (PVC) sanitary sewer pipe and fittings shall conform to the requirements of ASTM D-3034 SDR-35 with joints and rubber gaskets conforming to ASTM D 3212. All pipes shall be clearly marked with the data of manufacture. All pipes shall be provided with a reference mark for proper spigot insertion. Joint gaskets shall be fabricated from a compound of which the basic polymer shall be a synthetic rubber consisting of styrene, butadiene, polyisoprene or any combination thereof and shall meet the requirements of ASTM D-3212.

2) Ductile iron sewer pipe shall conform to ANSI A-21.51 or AWWA C-151 and shall be epoxy coated, push-on joint (Tyton joints only) or mechanical joint. Cement-lined ductile iron pipe shall not be used for sanitary sewer. The ductile iron pipe shall be Class 52, unless otherwise approved by the City Engineer.

3) AWWA C900 PVC pipe shall be pressure class 150 (SDR 18) unless otherwise approved by the City Engineer. Pipe joints shall be manufactured using an integral bell with an elastomeric gasket push-on type joint. Elastomeric gaskets shall conform to ASTM F477. All fittings shall be PVC, compatible with C900 with respect to joint dimensions and physical properties.

6-2.04 SIDE SEWER

Side sewer service lines shall be PVC, ASTM D-3034 SDR-35, with flexible gasket joints. Depths greater than 14 feet shall be AWWA C900 or CL 52 epoxy coated DIP.

6-2.05 SIDE SEWER AND FORCE MAIN TRACER TAPE AND LOCATE WIRE

Utility pipe tracer tape shall be detectable below ground surface and color coded. Tracer tape shall be detectable type, up to 6 inches in width, and buried 24 inches to 48 inches below finished grades. The color of the tape for sanitary sewer shall be green with black printing reading "CAUTION SANITARY SEWER BURIED BELOW". Tracer tape shall be “Lineguard Type II Detectable”, or an equivalent approved by the City Engineer. Utility locate wire (10 Ga.) shall be installed on all sewer force mains. All end of spool connections shall be welded and watertight.

6-2.06 PLUGS

Plugs shall be able to withstand all test pressures without leakage. All plugs shall be approved by the Public Works Inspector.

6-2.07 BACKWATER CHECK VALVE

Backwater check valves installed on 4 inch through 8 inch diameter side sewer lines shall be rubber flapper swing type check valves. Flapper shall be constructed from steel reinforced rubber with 45 durometer standard rubber hardness. Valve seat shall be at a 45 degree angle to direction of flow. Flow area through valve shall equal full pipe area. Valve body shall be coated cast iron, East Jordan Ironworks or an equivalent approved by the City Engineer, with flanged ends and bolted over to allow removal of flapper without removing valve from line.

The backwater valve shall be housed in a 48 inch diameter pre-cast concrete valve chamber with concentric 48 inch by 24 inch concentric reducing cone, or concrete meter box, depending on depth. The 24 inch frame and cover shall be marked “SEWER”.

6-2.08 STEEL CASING

Steel casing shall be black steel pipe conforming to ASTM A53. Casing thickness shall be 0.250 inch for casing 24 inches or less in diameter and 0.375 inch for casings over 24 inches in diameter.

6-2.09 CASING SPACER

Casing spacers and end seals shall be sized for pipe installation and shall be manufactured by Advance Products & Systems, Cascade Waterworks, Pipeline Seal and Insulators Co., or an equivalent approved by the City Engineer. See Standard Plan 618.

6-2.10 CONTROLLED DENSITY FILL

Controlled Density Fill (CDF) shall conform to the requirements of Section 2-09.3(1) E of the current WSDOT/APWA Standard Specifications.

6-2.11 CONCRETE

Concrete used for pads, thrust blocking, encasement, or slope anchor shall be mixed from materials acceptable to the City Engineer and shall have a 30 day compressive strength of not less than 3,000 psi. The mix shall contain five (5) sacks of cement per cubic yard and shall be of such consistency that the slump is between 1 inch and 5 inches.

6-2.12 BEDDING MATERIAL

Bedding material shall be ⅜ inch minus manufactured washed pea gravel. Pipe bedding shall be ⅜ inch minus pea gravel meeting the requirements of Section 9-03.12 of the WSDOT/APWA Standard. Bedding will be to the pipe zone shown on Standard Plan 616.

6-3 CONSTRUCTION

6-3.01 GENERAL REQUIREMENTS

All work shall be constructed as shown in the plans and in accordance with the current WSDOT/APWA Standards and Specifications, and these Standards. Materials shall be installed in compliance with the manufacturer's instructions and specifications, except where a higher quality of workmanship is required by the plans and these Standards. All work shall be in accordance with any applicable Federal, State, and local laws and regulations. The party responsible for the project shall arrange for necessary inspections by these agencies and shall submit evidence of their approval if requested by the City Engineer.

Construction shall not start prior to approval of the construction plans by the City Engineer.

6-3.02 MATERIAL SUBMITTALS

Five (5) sets of material submittals shall be submitted to the City Engineer for approval after the plans are approved for construction. The City Engineer shall either approve or disapprove and state the reasons for disapproval. A new corrected set of material submittals shall be submitted to the City Engineer for approval. Construction shall not proceed until the City Engineer has approved the materials.

The City Engineer's review of material submittals covers only general conformity to the plans and these Standards. The party responsible for constructing the project is responsible for quantity determinations. No quantities shall be verified by the City. Review and approval of material submittals by the City Engineer does not relieve the party responsible for construction of the project from the obligation to furnish required items in accordance with the plans and these Standards.

6-3.03 PRE-CONSTRUCTION CONFERENCE

The Contractor shall contact the Public Works Inspector (360-282-3193) to schedule a pre-construction conference after plans are approved and prior to staking and commencing construction. The pre-construction conference shall include the party responsible for the project, the project engineer, and representatives from the permit agencies, other utility companies, and City staff. An on-site tailgate meeting between the party responsible for the project and the Public Works Inspector shall be arranged by the party at least 48 hours prior to commencing construction.

6-3.04 CONSTRUCTION SCHEDULE

The party responsible for the project shall provide the City Engineer with the sewer construction schedule a minimum of five (5) business days prior to start of sewer system construction to arrange staking inspection and to give permitting agencies and customers two (2) business days notice. No construction is allowed until the construction plans have been approved.

6-3.05 EASEMENT

Prior to the start of the sewer system n construction, the party responsible for the project shall acquire an record all necessary easements for the construction, installation and maintenance of any sewer mains or facilities proposed in the project that will not be located in existing City right-of-way or City easements.

6-3.06 PERMITS

The party responsible for a sewer system extension project shall obtain all necessary permits from the City and other appropriate State and Local agencies and entities at their expense.

6-3.07 TRAFFIC CONTROL PLAN

The party responsible for the project shall submit a traffic control plan prepared by the party responsible for the project or the project engineer to the City Engineer for approval. No work shall commence until a traffic control plan has been approved by the City Engineer.

6-3.08 HANDLING OF PIPE

All types of pipe shall be handled in a manner that prevents damage to the pipe, pipe lining or coating. Pipe and fittings shall be loaded and unloaded using forks or cable choker in a manner that avoids shock or damage. Under no circumstances shall the pipe be dropped, skidded, or rolled against other pipe. Damaged pipe will be rejected, and the party responsible for the work shall immediately place all damaged pipe apart from the undamaged. All damaged pipe shall be removed from the project site within 24 hours.

Pipe shall be stacked in such a manner as to prevent damage to the pipe, to prevent dirt and debris from entering the pipe, and to prevent any movement of the pipe. The bottom tiers of the stack shall be kept off the ground on timbers, rails or other similar supports. Pipe on succeeding tiers shall be alternated by bell and plain end. Timbers of 4 inch × 4 inch shall be placed between tiers and chocks shall be placed at each end to prevent movement. Each size of pipe shall be stacked separately.

Threaded pipe ends shall be protected by couplings or other means until the pipe is installed. Dirt or other foreign material shall be prevented from entering the pipe or pipe joints during handling and installation. When pipe installation is not in progress, the open ends of the pipe shall be closed by a watertight plug or by other means approved by the City Engineer.

6-3.09 STAKING

Staking shall be performed by or under the direct supervision of a land surveyor licensed in the State of Washington utilized/hired by the party responsible for the work. Two (2) business days notice shall be provided to the City Engineer to inspect construction staking prior to construction.

The minimum staking of sewer lines shall be as follows unless otherwise directed by the City Engineer:

1) Staking location of sewer mains and side sewers every 50 feet with cut or fill to invert of pipe.

2) Staking location of all manholes for alignment and grade with cut or fill to rim and pipe inverts.

3) Staking front lot corners prior to installation for side sewer tees.

6-3.10 DEVIATION FROM PLANS

No deviations from the approved plans and these Standards shall be allowed without the City Engineer's written approval. If major changes are required, the project engineer shall revise, sign and submit the plans for the City Engineer's approval prior to restart of construction.

6-3.11 INSPECTION AND TESTING

The City Engineer shall have access to the project site for the purpose of inspections and testing at all times. Proper facilities shall be provided for such access, inspection, and testing.

If any work is covered without approval or consent of the City Engineer, it must be uncovered for inspection if required by the City Engineer.

Before a pressure test is to be observed by the City Engineer, the party responsible for the project shall ensure that whatever preliminary tests are necessary to ensure that the material and/or equipment are in accordance with the plans and these Standards are done and successfully completed.

Written and verbal notices of deficiency shall be given to the party responsible for the work. The party responsible for the work shall correct such deficiencies before final inspection by the City Engineer.

The sewer system must successfully pass the pressure test before final project inspection by the City Engineer.

6-3.12 WATER QUALITY

The party responsible for the work is required to implement water pollution control BMPs and maintain these until the project is accepted by the City Engineer. The party responsible for the work shall familiarize themselves with the requirements of the DOE, the current adopted stormwater manual for western Washington, and other regulatory agencies having jurisdiction over such matters.

The oil and chemical storage site for the project shall be approved by the City Engineer and the area shall be diked. There shall be no disposal of waste oil or oil products on the project site. A waste oil disposal tank shall be provided if deemed necessary by the City Engineer.

The party responsible for the work shall submit a Notice of Intent (NOI) to the DOE if applicable. A copy of the NOI form is available at the front counter of City Hall located at 116 Union Avenue or can be obtained at the DOE website.

6-3.13 CONSTRUCTION ON EXISTING EASEMENTS

All work within public utility easements shall be performed in accordance with terms and conditions of the respective easement. Each easement area shall be restored to equal to or better than the condition of the easement area that existed prior to the work. Work shall not be performed within any public utility easement area unless such work is specifically authorized by the City Engineer. The party responsible for the project shall provide advance written notification to and shall coordinate the authorized work with the persons and/or entities owning property that is adjacent to the easement area.

6-3.14 PRE-CONSTRUCTION PHOTOS

Prior to commencement of work, photographs shall be provided to the City that clearly show the conditions of the project site immediately before the anticipated start of the work. Photographs will be obtained as follows:

1) 50 foot interval in easements up station and down station.

2) Any other locations as directed by the City Engineer.

The photographs shall be 4 inch x 6 inch, color prints, contained in albums, catalogued, and cross-referenced. Digital copies of all photographs shall also be provided to the City Engineer.

6-3.15 UNDERGROUND UTILITIES

The plans show the approximate locations of various existing utilities known to the project engineer such as gas lines, water mains, storm drainage, power lines, telephone lines, TV cables, fiber optics, and other obstructions based on information obtained from various sources. The party responsible for the project shall be responsible to check for interferences and obstructions by inquiry from the different utilities and by underground exploration before commencing excavation.

The party responsible for the project shall request field locates and notify the owners of underground utilities about the scheduled commencement of excavation through the one-call system (1-800-424-5555).

Notice shall be made to owners of underground utilities not less than two (2) business days or more than ten (10) business days prior to scheduled date of commencement of excavation. Test pits, for the purpose of locating underground utilities or structures in advance of the construction, shall be excavated and backfilled. Test pits shall be backfilled immediately after their purpose has been satisfied and the surface restored and maintained in a manner satisfactory to the City Engineer or the applicable agency.

Excavation around and under active utilities shall be performed with special care to ensure that utility service is not interrupted. Where it is necessary to cut, move or reconnect any service lines, arrangements shall be made with the respective utility owners.

The party responsible for the project shall indemnify, defend and hold harmless the City from any claim for damage of utilities and/or disruption to services resulting from project activities/operations. The form of the indemnification agreement shall be subject to the approval of the City Attorney.

6-3.16 TRENCH EXCAVATION

Trench excavation and backfill operations within State right-of-way: All excavation and backfill within the State right-of-way shall adhere to the current WSDOT/APWA Standard Specifications.

Trench excavation and backfill operations within Snohomish County right-of-way: Excavation within the Snohomish County right-of-way shall conform first to the Snohomish County Road Standards, and secondly to standards set forth by the current WSDOT/APWA Standard Specifications.

Trench excavation and backfill operations within City right-of-way: Excavation within City right-of-way shall conform to these Standards.

Clearing and grubbing limits may be established by the City or governing agency for certain areas. Debris resulting from the clearing and grubbing shall be appropriately disposed of in accordance with applicable State and local laws and regulations.

Trenches shall be excavated to the line and grade shown in the plans or as designated by the City Engineer. Higher strength pipe or special bedding may be required because of excess trench width.

Unsuitable material below the depth of the bedding shall be removed to the extent approved by the City Engineer and replaced with materials approved by the City Engineer.

The length of trench excavation in advance of pipe laying shall be kept to a minimum and shall not exceed more than 150 feet without prior written approval of the City Engineer.

When trenching operations take place in the City right-of-way, the pavement and all other improvements shall be restored as required by the right-of-way permit.

When excavation of rock is encountered, all rock shall be removed to provide a clearance below, on each side of all pipe, and fittings of at least 6 inches for pipe sizes 24 inches or smaller and 9 inches for pipe sizes 30 inches or larger. Material removed shall be replaced with appropriate backfill material, which shall be compacted to 95% standard proctor. See Standard Plan 617.

6-3.17 SHEETING AND SHORING

Sheeting and shoring shall be provided and installed as necessary to protect workers, the work and existing utilities and other properties in compliance with OSHA and WISHA requirements. Removal of the sheeting and shoring shall be accomplished in such a manner that there will be no damage to the work or to other properties.

6-3.18 TRENCH DEWATERING

Sufficient pumping equipment shall be provided and maintained on the project site to keep the trench free from standing water. Surface runoff shall not be allowed to flow to the trench. The trench water or other deleterious materials shall not be allowed to enter the pipe at any time. If, at anytime, water is found to be entering the new sewer pipe, the pipe shall be plugged and all work shall cease until the trench water is completely pumped out or otherwise controlled, to the satisfaction of the City Engineer. Any method used must be in accordance with the specifications and requirements of the City and current DOE stormwater manual for western Washington and any other applicable State regulations.

6-3.19 MANHOLE

Manhole Foundation

Unless otherwise directed by the City, manhole bases (pre-cast base sections) shall be placed on a minimum thickness of 6 inches of crushed surfacing base course meeting the requirements of Section 9-03.9(3) of the current WSDOT/APWA Standard Specifications. The crushed surfacing base course must be compacted to 95% of standard density.

Manhole Sections

Manhole sections shall be placed and aligned so as to provide vertical sides and vertical alignment of the ladder steps. The completed manholes shall be rigid, true to dimension and watertight. Rough or uneven surfaces shall not be permitted inside or outside. All manhole sections shall be newly manufactured and free of breaks or cracks.

Joints between pre-cast manhole elements shall be rubber gasketed in a manner similar to pipe joints conforming to ASTM C-443 and they shall be grouted inside and outside. The grout used between joints in the pre-cast sections and for laying manhole adjusting bricks shall be composed of two-part cement to one-part of plaster sand. All joints shall be thoroughly wetted and completely filled with non-shrink grout, smoothed both inside and outside. Grout shall be ½ inch minimum thick and 3 inch minimum on each side of joints. The exterior joints shall receive Riser Wrap® or an equivalent approved by the City Engineer that overlaps the manufacturer's water proofing by a minimum of 1 inch when required by the City Engineer. Shop drawings of the joint design shall be submitted to the City Engineer for approval prior to manufacture.

Completed joints shall show no visible leakage and shall conform to the dimensional requirements of ASTM 478. They must be inspected before backfill.

Grout such as "JetSet®" or similar products shall not be used in the City of Snohomish.

Lift Holes and Steel Loops

All manhole lift holes shall be completely filled with expanding grout and smoothed both inside and outside to ensure water tightness. All steel loops must be removed, flush with the manhole structure. The stubs shall be covered with grout and smoothed. Rough or uneven surfaces shall not be permitted.

Manhole Channels

All manholes shall be channeled unless otherwise approved by the City Engineer. Channels shall match existing sewer grades. Channels shall converge with smooth transitions rounded into well finished junctions. Channel sides shall be carried up

vertically to the crown elevation of the various pipes. Concrete shelves between channels shall be smoothly finished, warped evenly and sloped to drain.

All standard manholes shall have a minimum drop of 0.10 feet to a maximum drop of 2.0 feet between the invert in and the invert out. Shallow manholes shall have a minimum drop of 0.10 feet to a maximum drop of 0.50 feet between the invert in and the invert out.

Pipe Connections at Manholes

All pipes except PVC pipe entering or leaving the manhole shall be provided with flexible joints within ½ of a pipe diameter or 12 inches, whichever is greater, from the outside face of the manhole structure. The flexible joint shall be placed on firmly compacted bedding, particularly within the area of the manhole excavation which normally is deeper than that of the sewer trench. Special care shall be taken to see that the openings through which pipes enter the manhole are completely and firmly rammed full of non-shrink grout to ensure water tightness.

PVC pipe connected to manholes shall be provided with a manhole adapter complete with gasket and approved by the City Engineer. No PVC pipe joint shall be placed within 10 feet of the outside face of the manhole.

All stubbed out pipes placed through manhole walls for future connections shall be suitably plugged and blocked, with bell end left intact in a manner acceptable to the City Engineer.

Kor-N-Seal® factory installed boots are allowed.

Drop Manholes

Drop manholes shall be constructed with an inside drop connection per City Standard Plan 611 for all manholes.

Ladders

Manhole ladders shall be installed in accordance with the approved manufacturer's recommended procedures and City Standard Plans 608 and 609 on sides of manholes opposite the pipe and channels if possible.

Connection to Existing Manholes

When connecting to an existing manhole, the existing manhole diameter must be adequate to accommodate the new sewer mains. If not, the existing manhole shall be upgraded or repaired as directed by the City Engineer at the expense of the party responsible for the project. If the existing manhole access is less than 24 inches in diameter, and/or concentric cone (manhole over 7 feet deep), the manhole shall be upgraded to include a new 24 inch ring and cover and/or eccentric cone. If connection to

an existing manhole places a channel directly under access opening, the ladder shall be moved and the cone section rotated to place the access over concrete shelf.

Invert elevations shall be verified by the party responsible for the work prior to construction. The crown elevation of laterals shall be the same as the crown elevation of the incoming pipe unless specified. The existing base shall be reshaped to provide a channel equivalent to that specified for a new manhole.

Excavation shall be done completely around the manhole to prevent unbalanced loading. The manhole shall be kept in operation at all times and the necessary precautions shall be taken to prevent debris or other material from entering the sewer, including a tight pipeline bypass through the existing channel if required by the City Engineer.

Connection of new sewer pipe to an existing manhole shall be accomplished by using core drilled holes to match the size of pipe. All openings shall be pipe O.D. plus structure wall thickness. The transition of connecting channels shall be constructed so as not to interrupt existing flow patterns.

Upstream pipes, except PVC pipe, penetrating the walls of manholes shall be placed with the bell facing out such that the bell is placed snug against the outside wall of the structure as the angle of penetration allows. Pipe, except PVC pipe, leaving or entering the manhole shall be provided with a flexible joint within ½ of a pipe diameter, or 12 inches, whichever is greater. After pipes have been placed to their final position, they shall be grouted tight with non-shrink grout. PVC pipe shall be connected to an existing manhole per these Standards.

The party responsible for the work shall assure the workers' exposure to asbestos material be at or below the limit prescribed in WAC 296-62-07705.

Asbestos cement pipe shall be cut with a reed wheel cutter with controlled flowing water. Contaminated clothing shall be transported in sealed, impermeable bags and labeled in accordance with WAC 296-62-07705. Asbestos cement pipe shall be left and buried in the trench.

Pipe Plugging at Connections

At the connection to the existing sewer system, the party responsible for the work shall physically plug all new sewer connections until all tests have been completed and the City Engineer approves the removal of the plugs.

Saddle Manhole

A saddle manhole shall be constructed per City Standard Plan 605 and these Standards. The existing pipe shall not be cut until approval is received from the City Engineer.

Manhole Ring and Cover

Manhole rings and covers shall be installed per Standard Plan 612. All castings shall be coated with bituminous coating prior to delivery at the job site. Manholes in paved areas shall match the finished grade of the pavement.

Grade Adjustment

For manholes located in the City right-of-way, not less than 4 inches and not more than 26 inches shall be provided between the top of the cone (or slab for flat top manholes) and the bottom of the manhole frame. Grade adjustments shall be done within 24 hours after paving. Paving, repaving, and patching shall be completed within 72 hours. Manhole rim elevations in unpaved areas (planters and grassed areas) shall be at grade or as directed by the City Engineer. See Standard Plan 614. Locking covers shall be used for all manholes. Manholes shall not be located in areas subject to inflow. If a manhole must be located in an area subject to inflow in the opinion of the City Engineer, the manhole shall be equipped with a PRECO® sewer guard watertight manhole insert or an equivalent approved by the City Engineer.

6-3.20 SEWER MAIN

General

No broken or defective sewer pipe and related materials shall be used. The maximum permissible trench width between the foundation level and to 12 inches above the pipe shall be 40 inches for pipe 15 inches or smaller or 1½ times the inside diameter plus 18 inches for pipes 18 inches or larger. See Standard Plan 615. If the maximum trench width is exceeded without authorization from the City Engineer, the Contractor will be required to provide pipe of higher pressure class or to provide a higher class of bedding, at the discretion of the City Engineer.

During excavation and installation of the sewer lines and placement of trench backfill, excavations shall be kept free of water. Surface run-off shall be controlled so as to prevent entry and collection of water in excavations. The static water level shall be drawn down a minimum of 1 foot below the bottom of the excavation so as to maintain the undisturbed state of the foundation soils and along the placement of any fill or backfill to the required density. The dewatering system shall be installed and operated so that the groundwater level outside the excavation area is not reduced to the extent which would damage or endanger adjacent structures or property.

Pipe Bedding

Pipe bedding shall be ⅜ inch minus manufactured washed pea gravel. Bedding will be to the pipe zone shown on Standard Plan 616. The pipe zone is identified as 6 inches below the bottom of the bell to 12 inches above the top of pipe for pipe sizes 24 inches or smaller and 9 inches below the bottom of the pipe to 12 inches above the top of pipe for 30 inches or larger.

Bedding shall be installed and spread smoothly so that the pipe is uniformly supported. Subsequent lifts are not to exceed 6 inches in thickness and shall be installed to the crown of the pipe. All lifts shall be individually compacted to 90% of the maximum density as determined by ASTM D-698. A 12 inch lift of material shall be placed and compacted over the crown of the pipe prior to backfilling the trench.

Laying Sewer Pipe

All sewer main installations shall have line and grade set by survey, prior to construction. Staking shall show each manhole cut to all inverts. All sewer mains shall be straight between manholes at a minimum depth of 7 feet measured from the invert, unless otherwise approved in writing by the City Engineer or shown on the approved plans.

Any method may be used that accurately transfers the control points provided by the Surveyor in laying the pipe to the designated alignment and grade (such as "line and batter board" and "laser beam" etc.).

When using the "line and batter board" method, the line and grade shall be transferred into the ditch where they shall be carried by means of a taut grade line supported on firmly set batter boards at intervals of not more than 30 feet. Not less than three batter boards shall be in use at one time. In the event that the batter boards do not line up, work shall immediately stop and the situation remedied before work proceeds.

When using a "laser beam" to set pipe alignment and grade, the position of laser beam from surface hubs provided by the surveyor shall be constantly checked to ensure the laser beam is still on alignment and grade. In the event the laser beam is found out of position, work shall immediately stop and the necessary corrections shall be made to the laser beam equipment and pipe installed before work proceeds.

There shall be a minimum horizontal clearance between sewer and water main of 10 feet, unless a design alternative has been specifically approved by the City Engineer. Sanitary sewers shall be installed lower than water mains whenever possible. Where sanitary sewers and water mains cross, there shall be a minimum vertical separation of 18 inches between water mains and sanitary sewer mains unless an alternative design has been specifically approved by the City Engineer. Water mains shall be above sewer mains. Where clearances cannot be met, party responsible for the work shall use a pipe casing or other approved methods shall be used to protect the domestic water supply from potential cross contamination.

Sanitary sewers laid 14 feet and deeper must be epoxy coated ductile iron pipe of Class 52 or C-900 PVC.

Trenches shall be excavated to a depth and grade required. Pipe bedding shall be placed to provide a uniform and continuous bearing and support for the pipe on solid undisturbed or compacted ground.

Sewer lines shall be laid upgrade from the starting point of connection on the existing sewer or from a designated starting point, as approved by the City Engineer. Sewer pipe shall be installed with the bell end forward or upgrade. After placing a length of pipe in the trench, the spigot shall be centered in the bell and the pipe forced home and brought to correct line and grade. During joining, the pipe shall be partially supported to minimize unequal lateral pressure and to maintain concentricity. Pipe handling after the gasket has been affixed shall be carefully controlled to avoid disturbing and dislocating the gasket. Any disturbed or dislocated gaskets shall be removed, cleaned, replaced and lubricated before joining the sections.

The maximum amount of open trench on streets and roadway shoulders shall not exceed 150 feet at any one time unless otherwise approved by the City Engineer. The project site shall be a safe environment at all times. At the end of each day all open trenches must either be backfilled or covered with steel plates and barricaded with attached flashing yellow lights to prevent vehicles, people and animals from falling into the trench.

Experienced and qualified personnel shall be on-site to oversee the construction process during all shoring operations. Where conditions exist which require shoring, trenches shall be adequately shored to protect existing property, utilities, pavement etc. and to provide safe working conditions inside and above the trench. Shoring shall be designed and installed in accordance with applicable local, State and Federal laws and regulations. A combination of shoring and over break, tunneling, boring, sliding trench shields, or other methods may be used to accomplish the work, provided the method(s) meet all applicable local, State and Federal laws and regulations.

Compaction tests shall be required for all backfilled trenches in paved public roadways and in roadway shoulders. A minimum of one test location shall be chosen by the City Engineer for every 200 lineal feet of sewer main installed. The City Engineer has the discretion to require additional tests in locations specified by the City Engineer. All testing shall be at the expense of the party responsible for the project.

All excavated trench material deemed by the City Engineer to be unsuitable for trench backfill shall be removed from the project site.

No construction materials, soil, debris etc. shall be stockpiled in the City right-of-way unless specific permission is granted in writing by the City Engineer.

Under no circumstances shall pipe materials be dropped or dumped into the trench. Broken or otherwise defective pipe shall be removed from the job site and replaced.

Every precaution shall be taken to ensure foreign material does not enter the pipe. When pipe laying is not in progress, the open ends of the pipe shall be closed by a water tight plug or other means approved by the City Engineer. If water is in the trench when work resumes, the seal on the pipe shall remain in place until the trench is completely pumped dry. No pipe shall be laid in water, or when in the opinion of the City Engineer, trench conditions are unsuitable.

No willows, poplars, cottonwoods, birches, soft maple, gum or any other tree or shrub whose roots are likely to obstruct public sewers are allowed within 30 feet of any public sewer. Any of these trees found to be located within 30 feet of a proposed sewer main shall be removed by the party responsible for the work.

Connection to Existing Pipe

When connecting to the end of an existing pipe known to have a bell at the end of the pipe, a new pipe of the same material as the existing pipe shall be used. The plans can specify connection by inserting a spigot of the new pipe into the existing bell end, with a "donut" gasket.

When connecting to the end of an existing pipe known to have a plain end, or needing to be cut, plans shall specify the use of a coupling to connect new and existing lines.

Couplings and O-ring adapters utilized for joining pipes of dissimilar materials or different nominal sizes shall be flexible elastomeric PVC as manufactured by Fernco®, Inc. or an equivalent approved by the City Engineer. Couplings shall be supplied with 316 stainless steel band clamps, fasteners and shear rings as applicable to the sizes and types that are being connected together.

Plugs and Connections

All fittings shall be capped or plugged with a plug of an approved material and gasketed with the same gasket material as the pipe unit, or the pipe shall be fitted with an approved mechanical stopper, or the pipe shall have an integrally cast knock-out plug. The plug shall be able to withstand all test pressures without leaking.

Jointing

Where it is necessary to break out or connect to an existing sewer during construction, only new pipe having the same inside diameter will be used in reconnecting the sewer. Where joints must be made between pipes with a mismatched wall thickness, a flexible gasket coupling, adapter or coupling-adapter shall be used to make a watertight joint. Couplings shall be those manufactured by "Romac", "Smith Blair", or an equivalent approved by the City Engineer for reinforced pipes and "Fernco®" or an equivalent approved by the City Engineer for non-reinforced pipes.

Jacking, Auguring and Tunneling

Jacking, auguring and tunneling shall be done in accordance with current WSDOT/APWA Standard Specifications.

Slope

All sanitary sewers shall be designed and constructed to give mean velocities of not less than 2.0 feet per second when flowing full. The slopes shall meet the minimum required in these Standards.

Sewer Abandonment

Existing sewer lines to be abandoned shall be removed or filled completely with sand, concrete or controlled density fill. At the manhole connection where the existing sewer main is to be abandoned, the manhole shall be rechanneled with 3,000 psi cement concrete.

Cleaning, Testing and Television Inspection of Sanitary Sewer Pipe

All sanitary sewer pipes shall be cleaned and tested after backfilling. Testing shall be by either exfiltration or low pressure air method unless the ground water table is such that the City Engineer requires an infiltration test.

The party responsible for the project shall clean and flush all sewer lines with clean water using approved jet vactoring equipment prior to testing.

All testing shall be under the direction and in the presence of the City Engineer. The party responsible for the work shall notify the City Engineer at least 2 business days prior to the start of any testing. Cleaning and testing of sewer lines shall be completed within 15 business days after backfilling of sewer lines and structures. Any additional delay will require the written consent of the City Engineer. The party responsible for the work shall furnish all labor, materials, tools, and equipment necessary to make clean and test the sewer lines. Any damage resulting from testing shall be repaired by the party responsible for the work to the satisfaction of the City Engineer.

All tees and stubs shall be plugged with flexible jointed caps, or acceptable alternate, securely fastened to withstand the internal test pressure. These plugs or caps shall be readily removable and their removal shall provide a socket suitable for making a flexible jointed lateral connection or extension.

Testing of sewer mains shall include a television inspection by the party responsible for the project. Television inspection shall be done after the air test has passed and before the roadway is paved. Immediately prior to a television inspection, enough water shall run down the line so it comes out the lower manhole. A copy of the DVD and written report shall be submitted to the City Engineer. Acceptance of the sewer will be made after the DVD has been reviewed and approved by the City Engineer. Any tap to an existing system needs to be televised as well. No ponding within the sanitary sewer line will be accepted. Contact the Public Works Inspector for television testing procedure form.

If larger diameter pipe is tested one joint at a time, leakage allowances shall be converted from GPH per 100 feet to GPH per joint by dividing the number of joints occurring in 100 feet. If leakage exceeds the allowable amount, corrective measures shall be taken and the line shall be re-tested to the satisfaction of the City Engineer.

A mandrel test in accordance with Section 7-17.3(2)G of the current WSDOT/APWA Standards and Specifications may be required by the City Engineer for sewers mains (not side sewer lines).

If any sewer installation fails to meet the requirements of the test method used, the source or sources of the leakage shall be determined and all defective pipes replaced. The complete pipe installation shall meet the requirements of the test method used before being considered acceptable. Replacement of defective pipe shall be in accordance with a replacement plan approved by the City Engineer.

Exfiltration Test

Prior to exfiltration leakage testing, the pipe may be filled with clear water to permit normal absorption into the pipe walls. The leakage test shall be completed within 24 hours after filling the pipe. When under test, the allowable leakage shall be in accordance with the following. Specified allowances assume pre-wetted pipe.

Leakage shall be no more than 0.28 GPH per inch diameter per 100 feet of sewer, with a hydrostatic head of 6 feet above the crown at the upper end of the test section, or above the natural groundwater table at the time of test, whichever is higher. The length of pipe tested shall be limited so that the pressure at the lower end of the section tested does not exceed 16 feet of head above the invert, and in no case shall the length of pipe being tested be greater than 700 lineal feet or the distance between manholes, whichever is shorter.

Where the test head is other than 6 feet, the maximum leakage shall not exceed 0.28 GPH per inch of diameter, per 100 feet of pipe length times the square root of the test head. The leakage can be determined from the following equation:

Maximum leakage (in gallons per hour): $$0.28 * \frac{\sqrt{H}}{\sqrt{6}} * Dx * \frac{L}{100}$$

Where: D = diameter (in.)
L = length of pipe (ft.)
H = test head (ft.).

When the test is to be made one joint at a time, the leakage per joint shall not exceed the computed allowable leakage per length of pipe.

Television Inspection

All sanitary sewers shall be inspected by the use of a Closed Circuit Television (CCTV) camera and footage provided to the City on DVD. All deficiencies noted by the CCTV camera inspection shall be corrected to the satisfaction of the City Engineer prior to final acceptance of the project. No VHS tapes will be accepted.

The following information shall be electronically generated and displayed on the CCTV footage at the beginning of each sewer main inspection:

1) All video and inspection reporting shall be done in DVD format.
2) All joints shall have a 360 degree view to inspect for faulty joints.
3) An inspection of each lateral connection, documentation of the location of each lateral connection and the total length shall be shown on the inspection report.
4) Inspection reports shall include the project name, city, address, date, pipe size, type of pipe, direction, surveyed footage, and current weather conditions.
5) CCTV data shall be Granite XP version 2 or approved equal.
6) A 1-inch diameter target ball clearly visible shall be used.
7) Before the camera is set in the pipe, a 5 gallon bucket of water contrasting color dye shall be poured into a manhole.
8) All DVDs shall be labeled with the Contractor's name, location of the video, the test date and manhole numbers on a professional video inspection label.
9) Re-inspection of pipes is required within 10-12 months of acceptance date.

6-3.21 SANITARY SIDE SEWERS

Fittings and Clean-outs for Side Sewers

All fittings shall be factory produced and shall be designed for installation on the pipe to be used. Fittings shall be of the same quality and material as the pipe used, except when installing a PVC insert on existing pipe.

Side sewers shall be connected to the tee provided in the public sewer where such is available, utilizing approved fittings or adapters. Where no tee is provided or available, connection shall be made by the use of a Romac tapping saddle. See Standard Plan 603.

All side sewers shall have a 6 inch clean-out at the property line per Standard Plan 602. The riser portion of the clean-out shall be PVC unless otherwise approved by the City Engineer. For longer side sewer installations, extra clean-outs will be required at spacing not to exceed 100 feet.

Marking of Side Sewers

Tracer tape shall be installed over side sewer pipes and side sewer stubs. The tracer tape shall be placed 24 inches to 48 inches below the finished grade and it shall extend its full length. The location of all side sewers shall be marked with a 10 gauge wire and 2 inch x

4 inch wood marker at the termination of the stub. The marker shall be connected to the pipe at the invert and wrapped around marker post. Above the ground surface, it shall be painted white with black letters of 3 inches in height “SEWER xx INVERT DEPTH xx”. Offset markers may be used when the side sewer location is within an existing driveway or other obstacle.

Testing of Side Sewers

When a new side sewer line is installed, the entire length of new pipe installed shall be tested. All side sewers shall be tested after backfill. Side sewer lines that are reconstructed or repaired to a length of 10 feet or more shall be tested for water-tightness. Testing of newly reconstructed sections of side sewer lines consisting of a single length of pipe is not required. Testing shall be performed in the presence of the City Engineer in accordance with these Standards and Section 7-17 Sanitary Sewers of the 2010 WSDOT/APWA Standard Specifications.

In cases where a new tap is made on the main, the first joint of pipe off the main shall be installed with a test tee, so that an inflatable rubber ball can be inserted for sealing off the side sewer installation for testing. In cases where the side sewer stub is existing to the property line, the test ball may be inserted through the clean-out wye to test the new portion of the side sewer installation.

Side Sewer As-Built Plans

The as-built drawings for side sewer lines shall show the following:

1) Location of the side sewer line, its connection with the building(s) and all dimensions.
2) Show station as distance of side sewer line tee from the center of the next downstream manhole.
3) The depth and point of connection of the side sewer line to the sanitary sewer main.
4) Any additional information which might be deemed necessary by the City Engineer.

Side Sewer Demolition

Side sewer demolition shall be performed prior to removal of building foundation. The side sewer for each building shall be excavated and removed from the house connection to the property line or the main as required by the City Engineer. The end of the side sewer shall be capped to remain in place. Side sewer demolition shall be performed in the presence of the City Engineer.

6-3.22 CLEANOUT

All clean-outs in paved areas, City right-of-way and easements shall be extended to grade and a 30” x 30” x 12” concrete pad shall be installed per Standard Plan 602. All clean-outs in private unpaved areas shall be installed per Standard Plan 602.

6-3.23 GREASE TRAP AND INTERCEPTOR

Grease traps and interceptors shall be installed and sized according to the criteria in the current Uniform Plumbing Code. Grease trap and interceptors shall be located on private property, and they shall remain privately owned and maintained by property owner or occupant. These facilities shall be available for the inspection by the City's Building Official at all times.

6-3.24 PRIVATE GRINDER PUMP

Private grinder pumps shall be installed in accordance with the manufacture's procedures and per approved plans by the City Engineer. The force main shall be pressure tested at 150% of the total dynamic head. All inspections must be completed prior to backfilling.

6-3.25 LIFT (PUMP) STATIONS

Lift stations shall be constructed in accordance with approved plans and specifications prepared by a engineer licensed in the State of Washington. Please reference Appendix A6 for lift (pump) station standards.

6-3.26 SPECIFICATIONS NOT COVERED BY THESE STANDARDS

In the event a construction or installation specification relating to sanitary sewers is not covered by these Standards, the City Engineer may require compliance with other applicable manuals or standards.

END OF SECTION

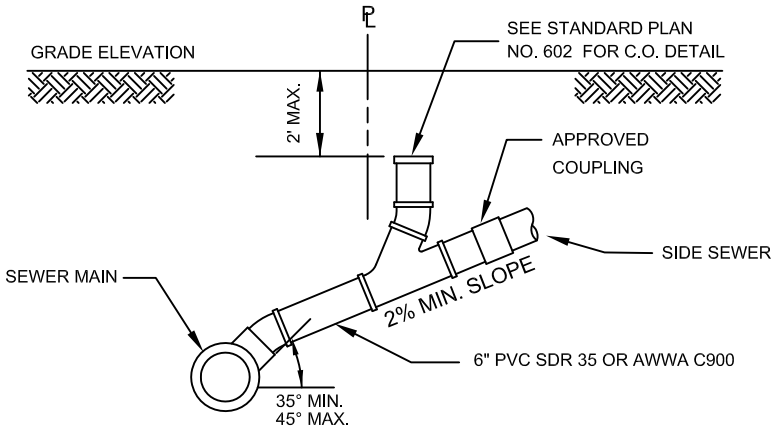
CITY OF SNOHOMISH

ENGINEERING DESIGN AND CONSTRUCTION STANDARDS

SANITARY SEWER

DRAWING INDEX

STANDARD PLAN NO.	PLAN DESCRIPTION	REVISION DATE
601	Typical Side Sewer Connections	April 2010
602	Sewer Clean-Out	April 2010
603	Typical Side Sewer Tap To Existing Sewer Mains	April 2010
604	Precast Shallow Manhole with Base April 2010	April 2010
605	Saddle Manhole 48” to 54”	April 2010
606	Type 1 Manhole 48" And 54"	April 2010
607	Type 2 Manhole 72" And 96"	April 2010
608	Type 3 Manhole 48” Through 96"	April 2010
609	Polypropylene Ladder	April 2010
610	Polypropylene Plastic Step	April 2010
611	Inside Drop Manhole Connection	April 2010
612	Manhole Ring and Cover	April 2010
613	Hinged Manhole Cover	April 2010
614	Manhole Pad and Adjustment	April 2010
615	Typical Sewer Trench Section	April 2010
616	Bedding For Sewer Pipe In Trenches	April 2010
617	Typical Sewer Trench Compaction	April 2010
618	Casing Detail	April 2010

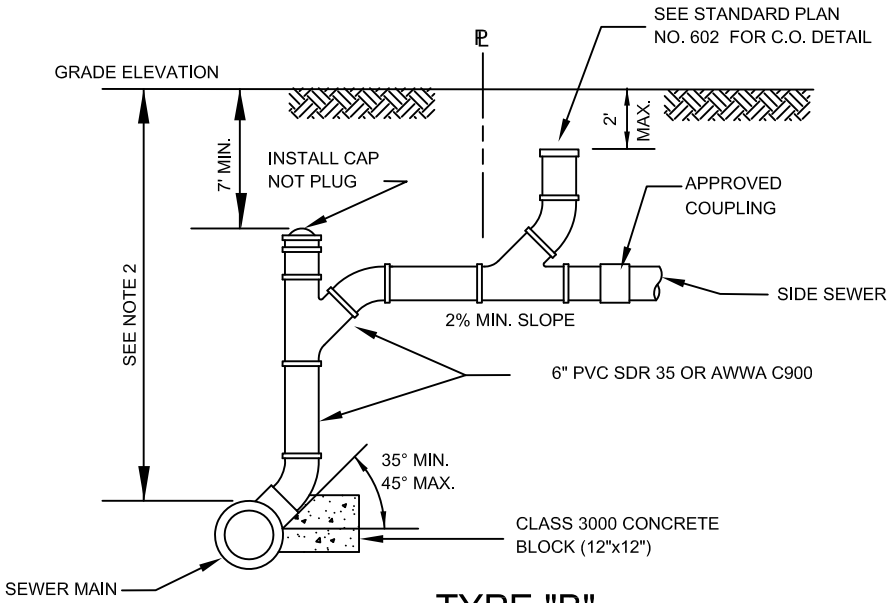


TYPE "A"

NTS

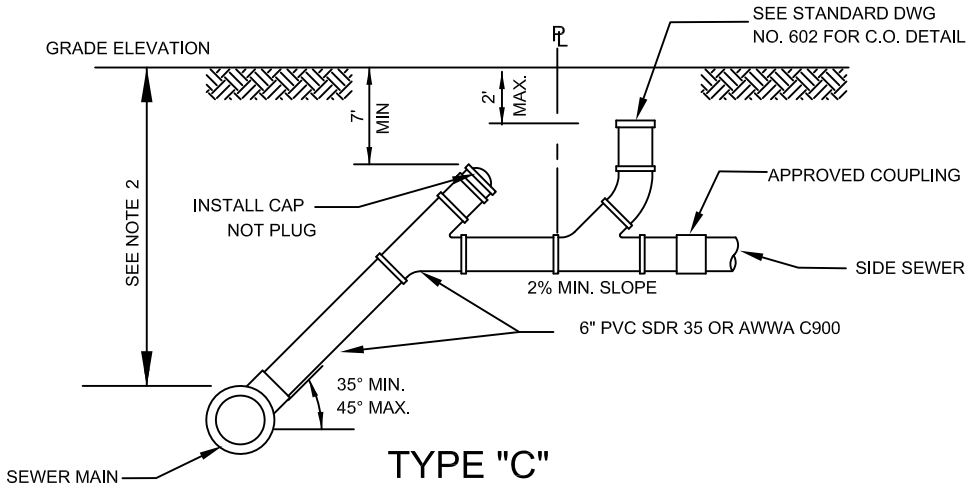
NOTES

1. SIDE SEWER CONNECTIONS TO NEW MAINS SHALL BE FACTORY TEES.
2. TYPE B&C SHALL BE USED ONLY WHEN MAIN DEPTH EXCEEDS 15 FEET OR AS APPROVED BY THE CITY ENGINEER.
3. APPROVED COUPLING SHALL BE FERNCO OR APPROVED EQUAL.
4. SIDE SEWER SHALL BE EXTENDED 10' BEYOND THE PROPERTY LINE TO PREVENT DAMAGE TO THE CLEAN OUT AND TO MINIMIZE CONFLICTS WITH OTHER UTILITIES WHEN SERVICE IS CONNECTED
5. SIDE SEWERS SHALL BE MARKED PER SECTION 6.3.20 OF THESE STANDARDS.



TYPE "B"

NTS



TYPE "C"

NTS

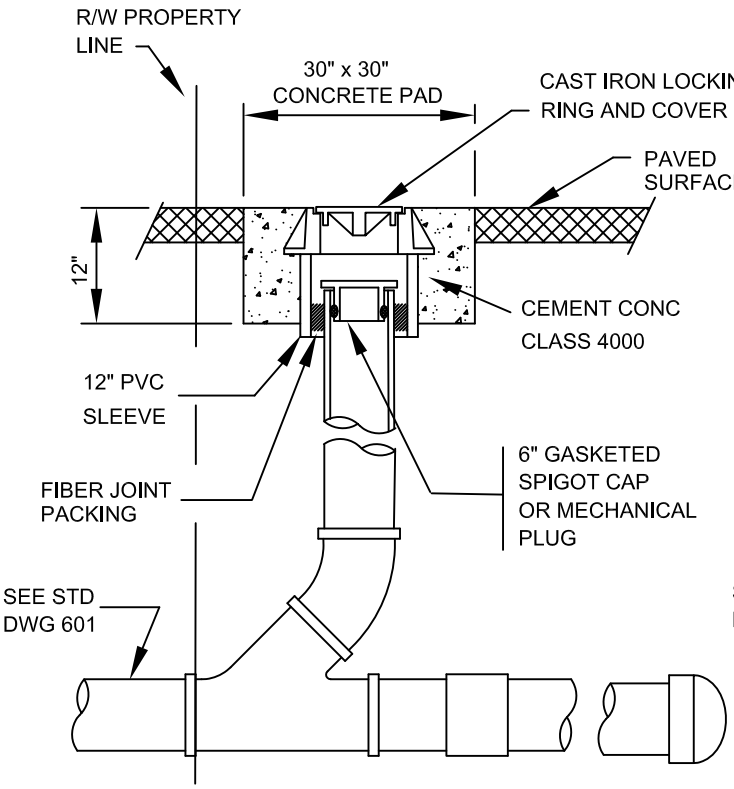
SNOHOMISH
WASHINGTON

TYPICAL SIDE SEWER CONNECTIONS

Approved By:
S L S
City Engineer
Date: April 2010

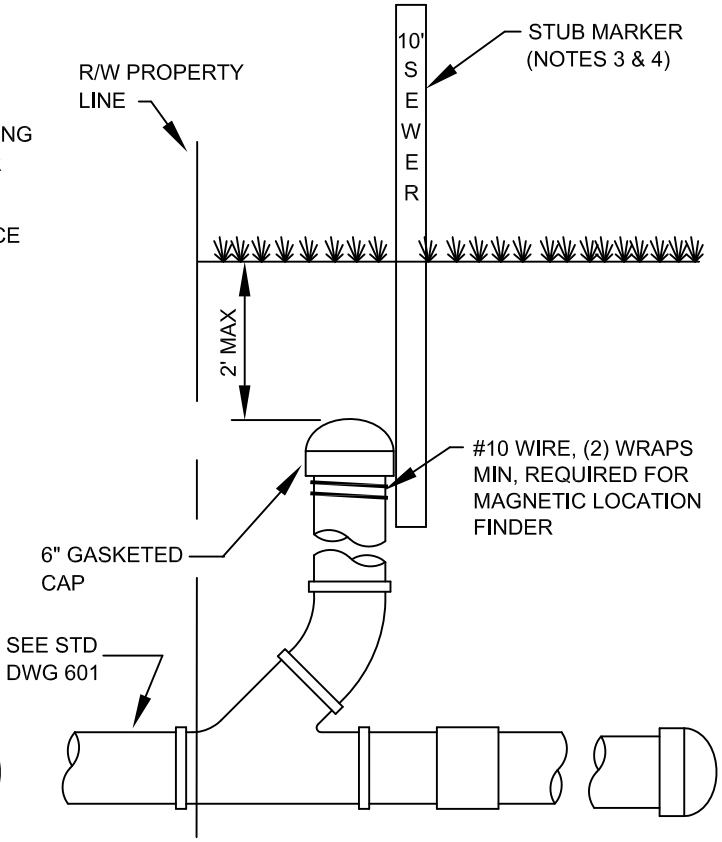
601
Number

City of Snohomish Public Works Department



PAVED AREAS, CITY RIGHT OF WAYS AND EASEMENTS

NTS

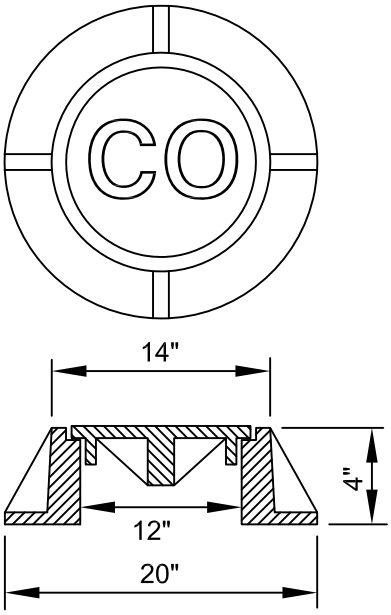


UNPAVED PRIVATE AREAS

NTS

NOTES

1. CLEAN-OUT PIPE AND FITTINGS SHALL BE PVC.
2. FOR NEW PLATS THE VERTICAL RISER PORTION OF THE CLEAN-OUT WILL BE CONSTRUCTED AT TIME OF CONNECTION TO BUILDING TO MINIMIZE DAMAGE. THE 6" WYE AND 6" PVC PIPE W/GASKETED CAPS WILL BE INSTALLED PRIOR TO BUILDING CONNECTION.
3. A 2"x4" STUB MARKER SHALL EXTEND DOWN TO A MINUMUM OF 24" BELOW GROUND. A MIN OF 36" SHALL EXTEND ABOVE GROUND.
4. THE STUB MARKER SHALL BE PAINTED WITH WHITE TRAFFIC PAINT AND THE WORD "SEWER" AND THE DEPTH IN FEET FROM GROUND TO SEWER STUB INVERT SHALL BE PAINTED ON THE MARKER WITH 3" HIGH BLACK PAINTED LETTERS.



12" CAST IRON LOCKING RING AND COVER

NTS

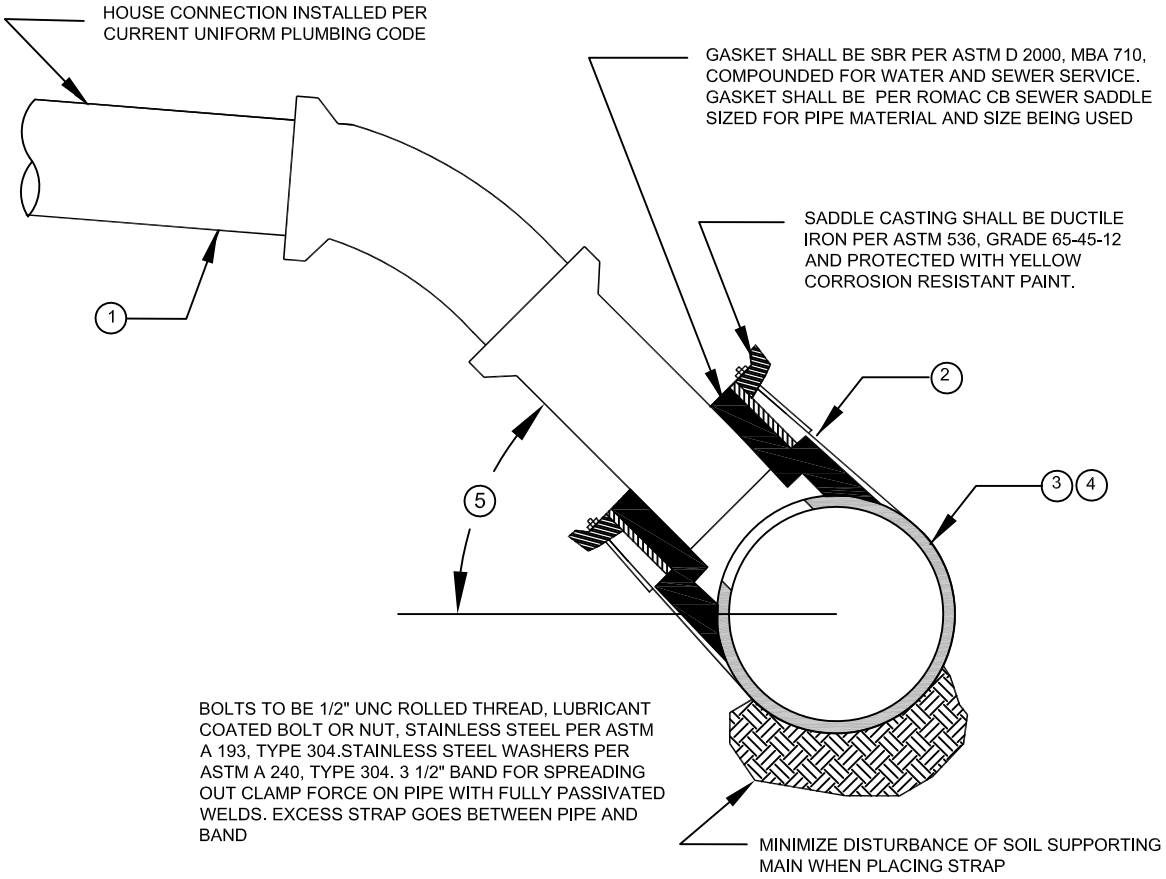
SNOHOMISH WASHINGTON

SEWER CLEAN OUT

Approved By: S L S, City Engineer
Date: April 2010

602 Number

City of Snohomish Public Works Department



NOTES

1. PVC SIDE SEWER. FOR REMAINDER OF PVC SERVICE SEE STD PLAN 601.
2. ROMAC "CB" SEWER SADDLE OR APPROVED EQUAL.
3. EXISTING SANITARY SEWER MAIN.
4. CORE DRILL EXISTING MAINLINE PIPE PER MANUFACTURER'S SPECIFICATIONS.
5. 35° MIN, 45° MAX

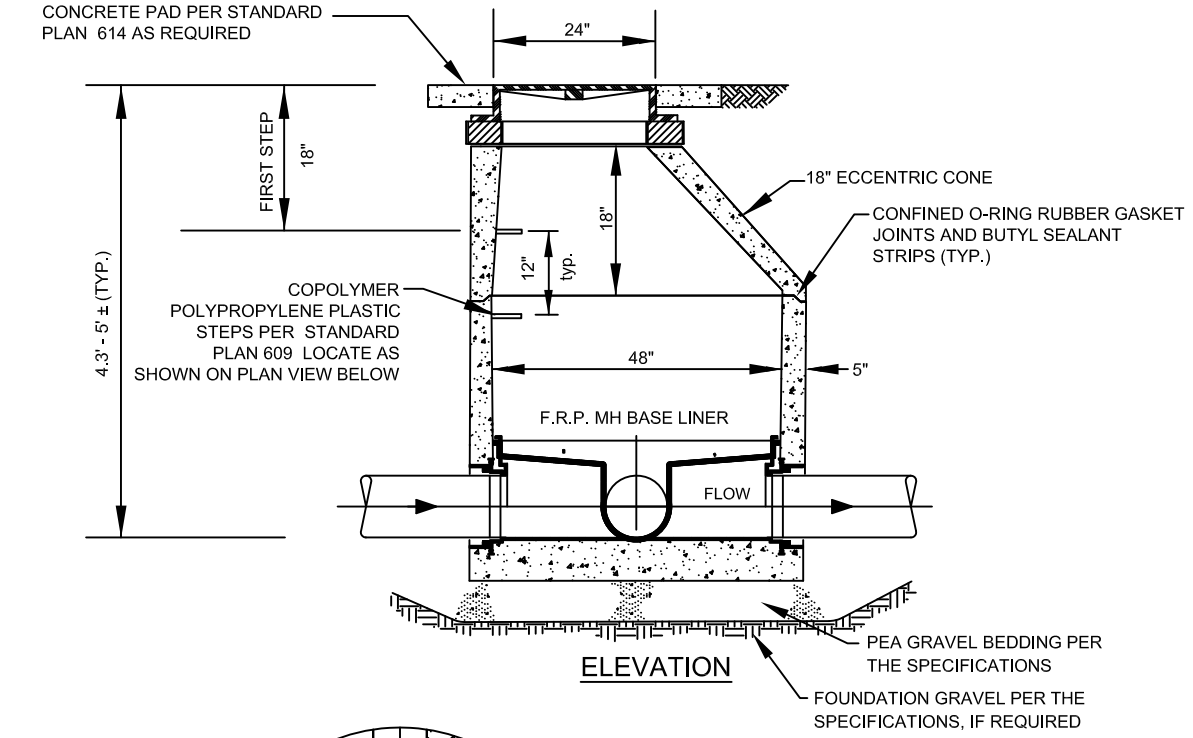
SNOHOMISH
WASHINGTON

TYPICAL SIDE SEWER TAP TO EXISTING SEWER MAINS

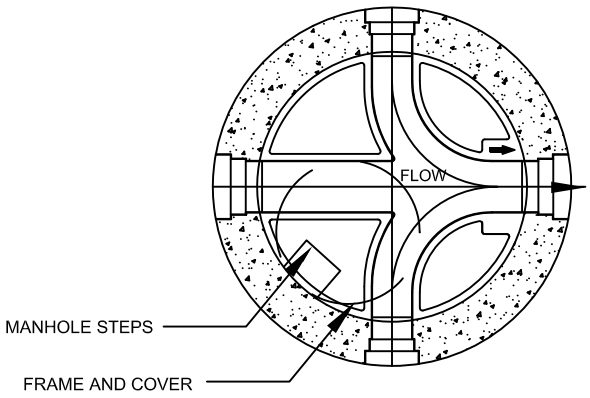
Approved By:
S L S
City Engineer
Date: April 2010

603
Number

City of Snohomish Public Works Department



ELEVATION



PLAN

PRECAST SHALLOW MANHOLE W/ BASE LINER

NOT TO SCALE

Notes:

1. USE 18" ECCENTRIC CONE AND 2' BASE FOR SHALLOW MANHOLE APPLICATION.
2. MINIMUM DROP IN THE INVERT ELEVATION THROUGH THE MANHOLE SHALL BE 0.1'. MAXIMUM DROP IN THE INVERT ELEVATION THROUGH THE MANHOLE SHALL BE 0.5'.
3. MARKER POSTS MAY BE REQUIRED IN EASEMENTS.
4. THE MANHOLE BASE LINER SHALL BE A FIBERGLASS REINFORCED PLASTIC (F.R.P.) BASE LINER SYSTEM.
5. GROUT ADJUSTING RINGS INSIDE, OUTSIDE AND BETWEEN. GROUT UNDER CASTING.

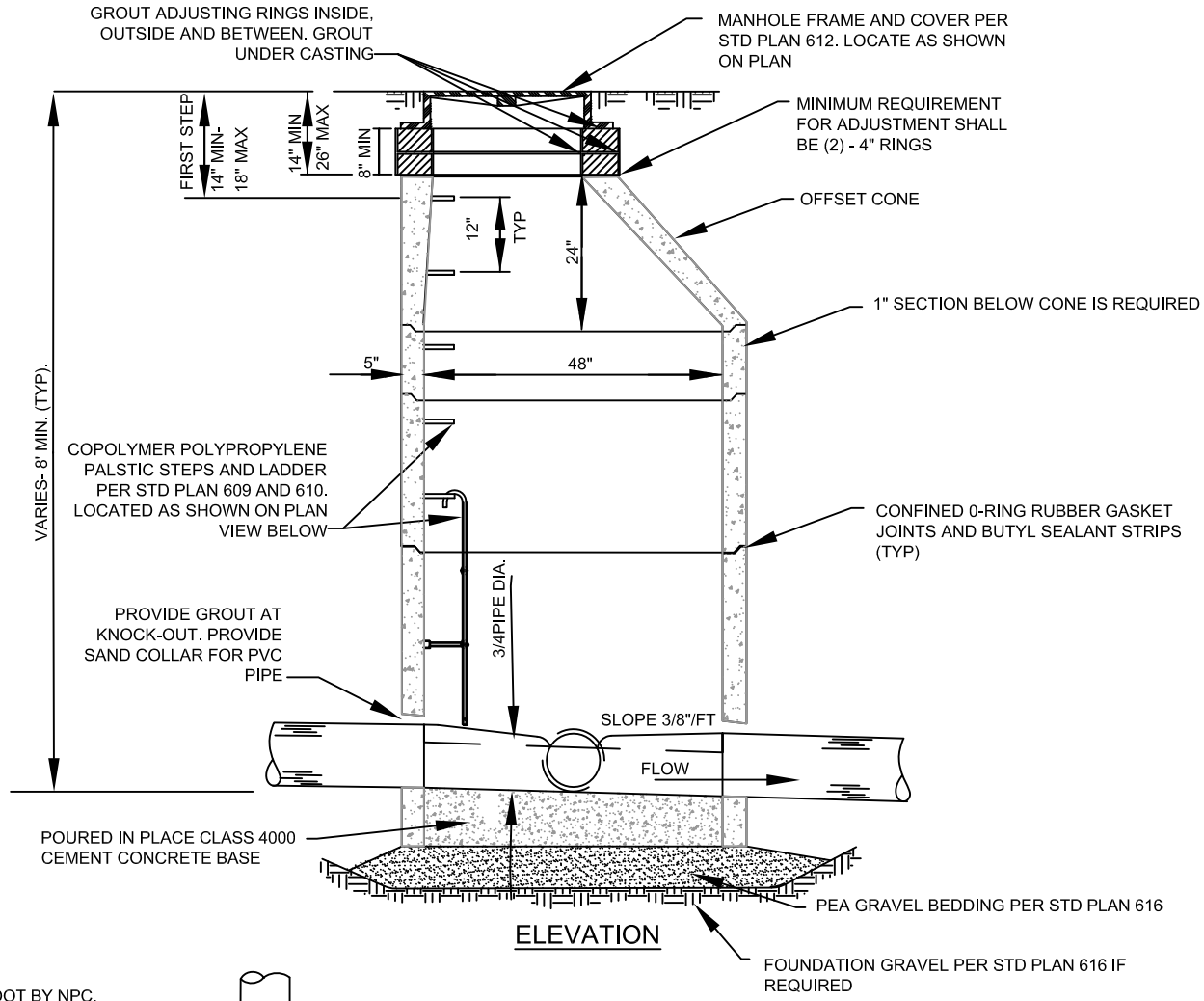
SNOHOMISH WASHINGTON

PRECAST SHALLOW MANHOLE WITH BASE LINER

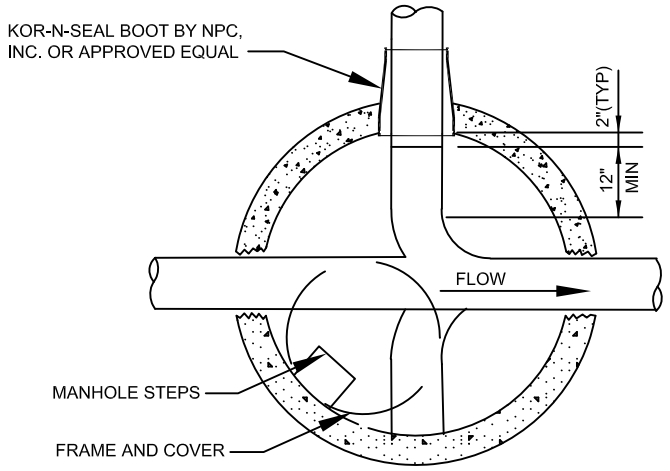
Approved By: S L S
City Engineer
Date: April 2010

604
Number

City of Snohomish Public Works Department



ELEVATION



PLAN

NOTES:

1. MINIMUM DROP IN THE INVERT ELEVATION THROUGH THE MANHOLE SHALL BE 0.1". MAXIMIM DROP IN THE INVERT ELEVATION THROUGH THE MANHOLE SHALL BE 0.5"
2. MARKER POSTS MAY BE REQUIRED IN EASEMENTS.

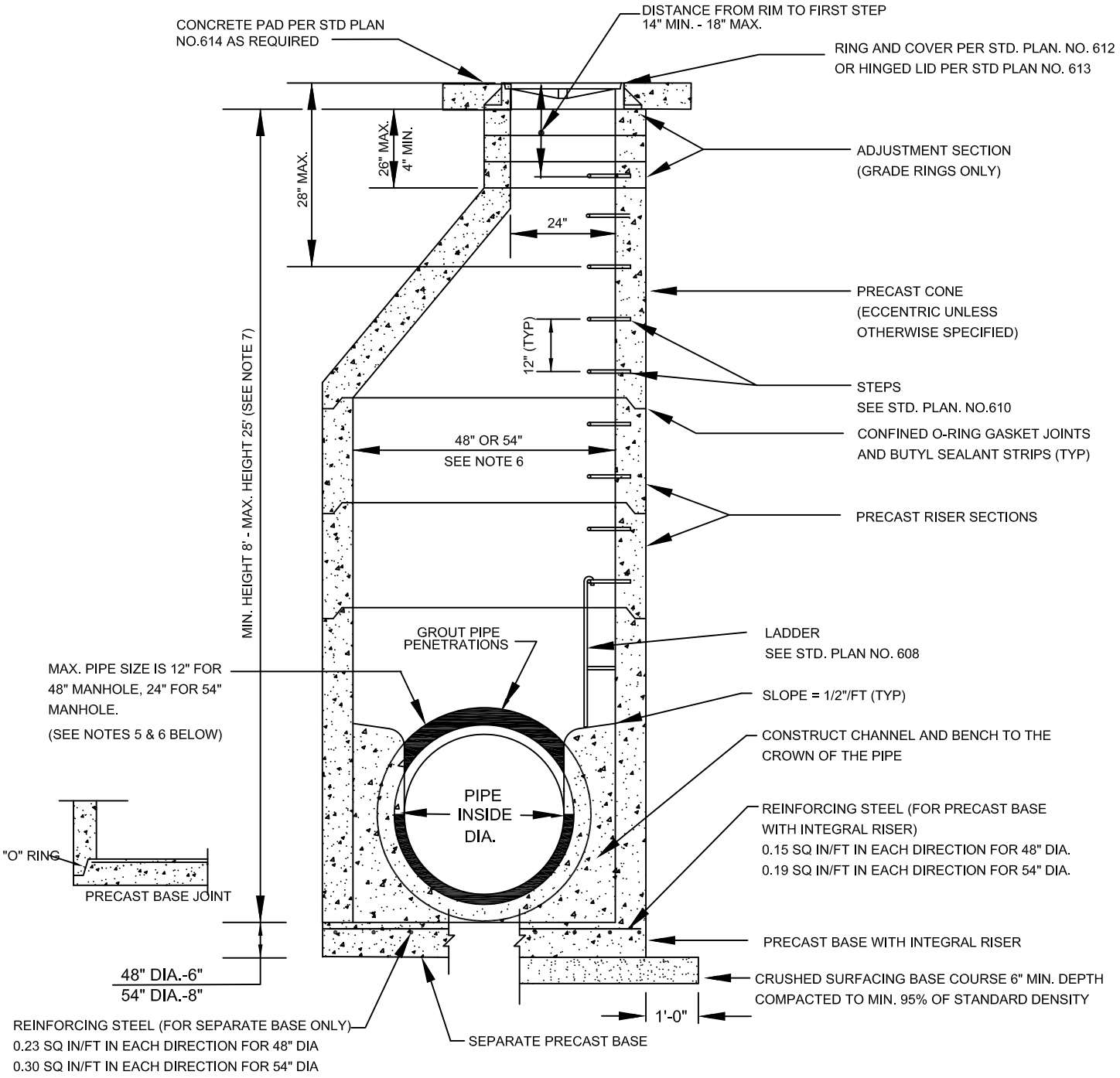
SNOHOMISH WASHINGTON

SADDLE MANHOLE - 48" to 54"

Approved By: S L S
City Engineer
Date: April 2010

605
Number

City of Snohomish Public Works Department



NOTES

1. MANHOLES TO BE CONSTRUCTED IN ACCORDANCE WITH AASHTO M-199 (ASTM C 478) UNLESS OTHERWISE SHOWN ON PLANS OR NOTED IN STANDARD SPECIFICATIONS.
2. NON-REINFORCED CONCRETE IN CHANNEL AND SHELF SHALL BE CLASS 3000. ALL PRECAST CONCRETE SHALL BE CLASS 4000.
3. PRECAST BASES SHALL BE FURNISHED WITH CUTOUTS OR KNOCKOUTS. KNOCKOUTS SHALL HAVE A WALL THICKNESS OF 2" MINIMUM AND BE CORE DRILLED ONLY. NO HAMMERING IS ALLOWED; KOR-N-SEAL FACTORY INSTALLED BOOTS ARE ALLOWED.
4. ALL BASE REINFORCING STEEL SHALL HAVE A MINIMUM YIELD STRENGTH OF 60,000 PSI AND BE PLACED IN THE UPPER HALF OF THE BASE WITH 1" MINIMUM CLEARANCE.
5. KNOCKOUT OR CUTOUT HOLE SIZE IS EQUAL TO PIPE OUTER DIAMETER PLUS MANHOLE WALL THICKNESS. MAXIMUM PIPE SIZE IS 12" FOR 48" MANHOLE, 24" FOR 54" MANHOLE. MINIMUM DISTANCE BETWEEN HOLES IS 8" (MEASURED ON THE INSIDE OF THE MANHOLE).
6. MANHOLE SIZE DEPENDS ON SIZE, LOCATION AND NUMBER OF HOLES FOR PIPES. MANHOLE DESIGN AND SIZE SHALL BE APPROVED AND WARRANTED BY THE MANHOLE SUPPLIER.
7. FOR HEIGHTS OVER 25' MANHOLE BASE SLAB DESIGN SHALL BE DESIGNED BY A LICENSED STRUCTURAL ENGINEER.
8. GROUT ADJUSTING RINGS INSIDE, OUTSIDE AND BETWEEN. GROUT UNDER CASTING.

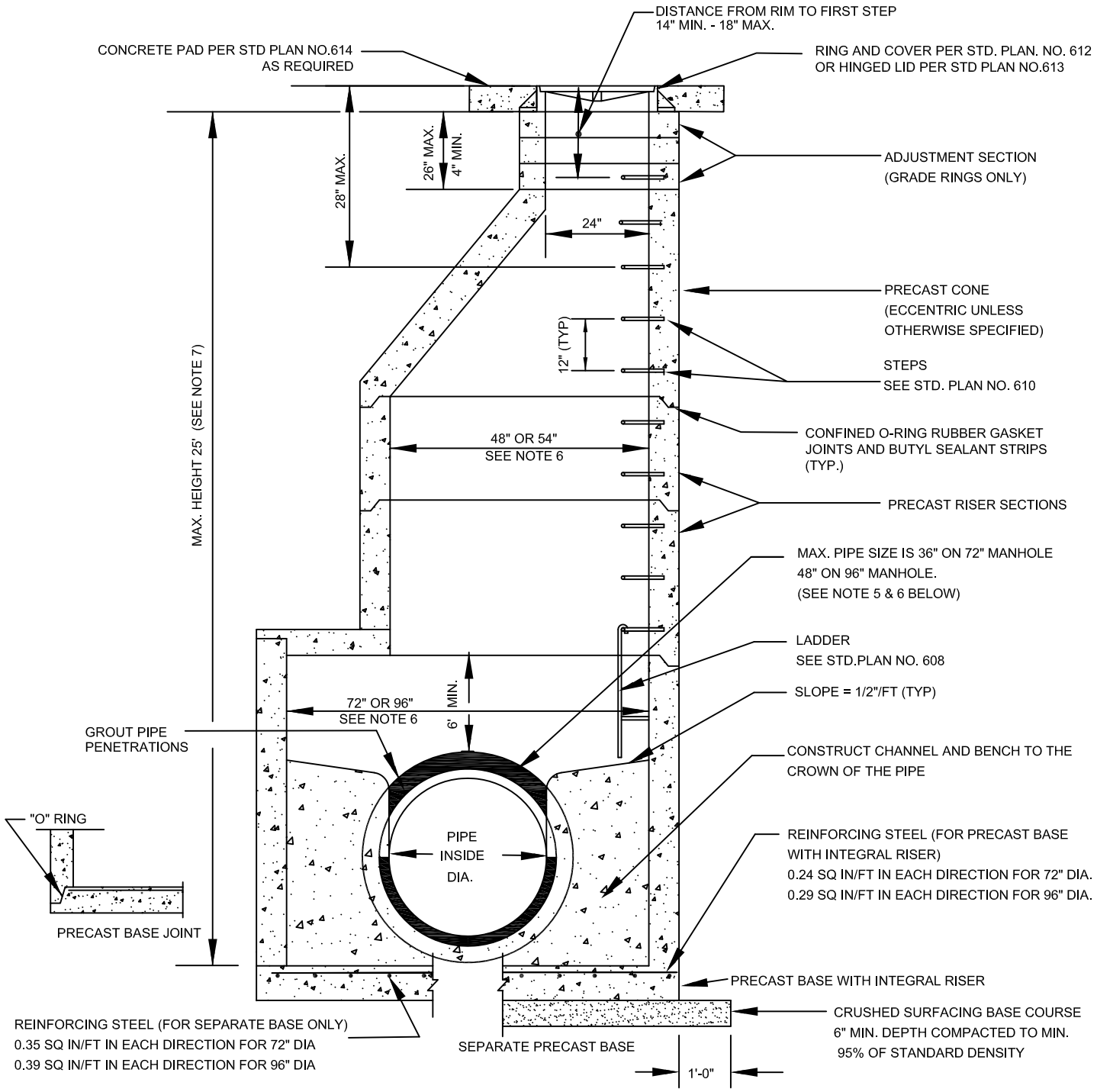
SNOHOMISH WASHINGTON

TYPE 1 MANHOLE (48" AND 54")

Approved By: S L S
City Engineer
Date: April 2010

606
Number

City of Snohomish Public Works Department



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4. ALL BASE REINFORCING STEEL SHALL HAVE A MINIMUM YIELD STRENGTH OF 60,000 PSI AND BE PLACED IN THE UPPER HALF OF THE BASE WITH 1" MINIMUM CLEARANCE.
5. KNOCKOUT OR CUTOUT HOLE SIZE IS EQUAL TO PIPE OUTER DIAMETER PLUS MANHOLE WALL THICKNESS. MAXIMUM HOLE SIZE IS 36" FOR 72" MANHOLE, 48" FOR 96" MANHOLE. MINIMUM DISTANCE BETWEEN HOLES IS 12"(MEASURED ON THE INSIDE OF THE MANHOLE).
6. MANHOLE SIZE DEPENDS ON SIZE, LOCATION AND NUMBER OF HOLES FOR PIPES. MANHOLE DESIGN AND SIZE SHALL BE APPROVED AND WARRANTED BY THE MANHOLE SUPPLIER.
7. FOR HEIGHTS OVER 25' MANHOLE BASE SLAB DESIGN SHALL BE DESIGNED BY A LICENSED STRUCTURAL ENGINEER.
8. GROUT ADJUSTING RINGS INSIDE, OUTSIDE AND BETWEEN. GROUT UNDER CASTING.

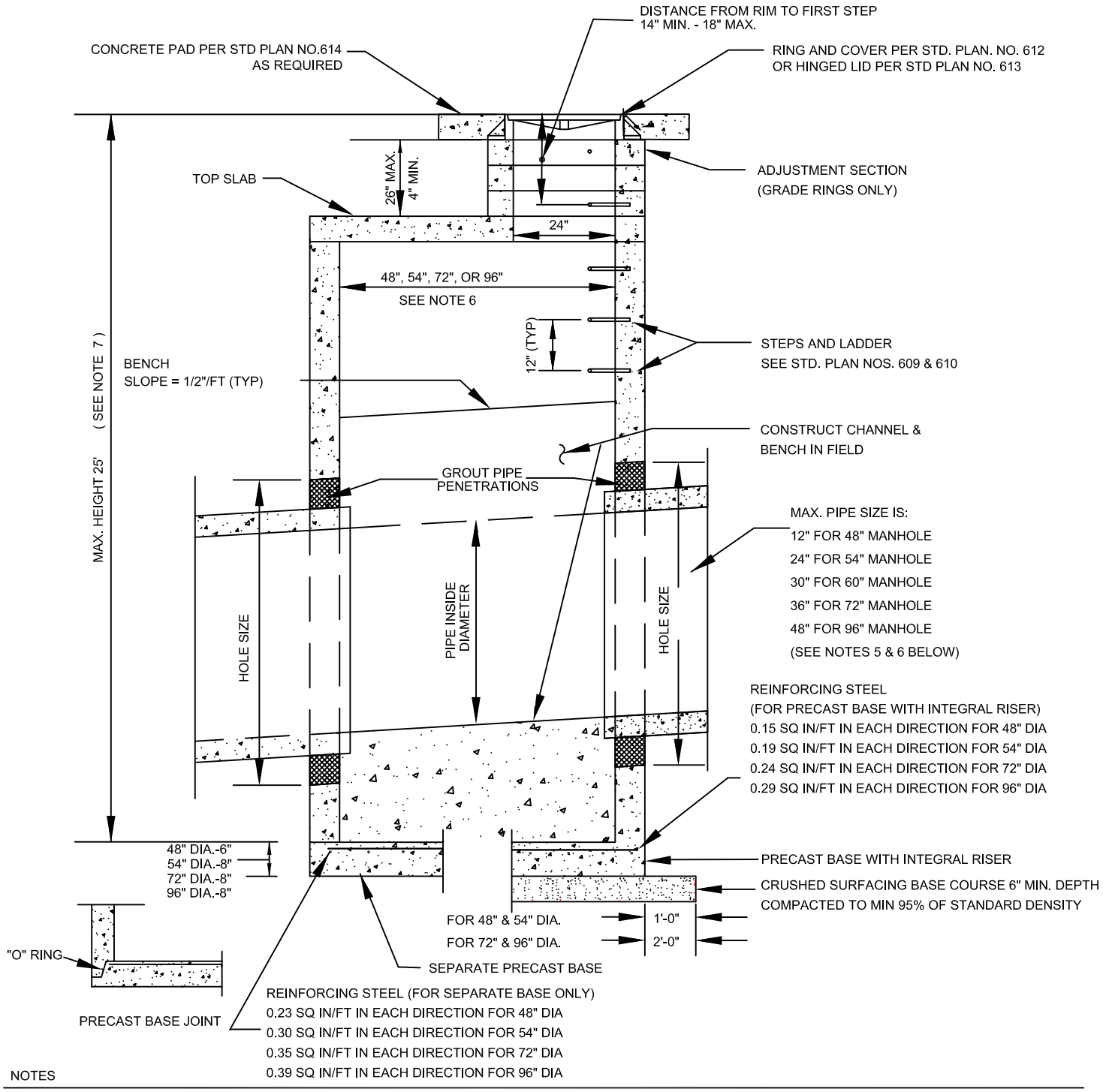
SNOHOMISH
WASHINGTON

TYPE 2 MANHOLE (72" AND 96")

Approved By:
S L S
City Engineer
Date: April 2010

607
Number

City of Snohomish Public Works Department



NOTES

1. MANHOLES TO BE CONSTRUCTED IN ACCORDANCE WITH AASHTO M-199 (ASTM C 478) UNLESS OTHERWISE SHOWN ON PLANS OR NOTED IN STANDARD SPECIFICATIONS.
2. NON-REINFORCED CONCRETE IN CHANNEL AND SHELF SHALL BE CLASS 3000. ALL PRECAST CONCRETE SHALL BE CLASS 4000.
3. PRECAST BASES SHALL BE FURNISHED WITH CUTOUTS OR KNOCKOUTS. KNOCKOUTS SHALL HAVE A WALL THICKNESS OF 2" MINIMUM AND BE CORE DRILLED ONLY. NO HAMMERING IS ALLOWED; KOR-N-SEAL FACTORY INSTALLED BOOTS ARE ALLOWED.
4. ALL BASE REINFORCING STEEL SHALL HAVE A MINIMUM YIELD STRENGTH OF 60,000 PSI AND BE PLACED IN THE UPPER HALF OF THE BASE WITH 1" MINIMUM CLEARANCE.
5. KNOCKOUT OR CUTOUT HOLE SIZE IS EQUAL TO PIPE OUTER DIAMETER PLUS MANHOLE WALL THICKNESS. MAXIMUM PIPE SIZE IS 12" FOR 48" MANHOLE, 24" FOR 54" MANHOLE, 36" FOR 72" MANHOLE AND 48" FOR 96" MANHOLE. MINIMUM DISTANCE BETWEEN HOLES IS 8" (MEASURED ON THE INSIDE OF THE MANHOLE).
6. MANHOLE SIZE DEPENDS ON SIZE, LOCATION AND NUMBER OF HOLES FOR PIPES. MANHOLE DESIGN AND SIZE SHALL BE APPROVED AND WARRANTED BY THE MANHOLE SUPPLIER.
7. FOR HEIGHTS OVER 25' MANHOLE BASE SLAB DESIGN SHALL BE DESIGNED BY A LICENSED STRUCTURAL ENGINEER.
8. GROUT ADJUSTING RINGS INSIDE, OUTSIDE AND BETWEEN. GROUT UNDER CASTING.

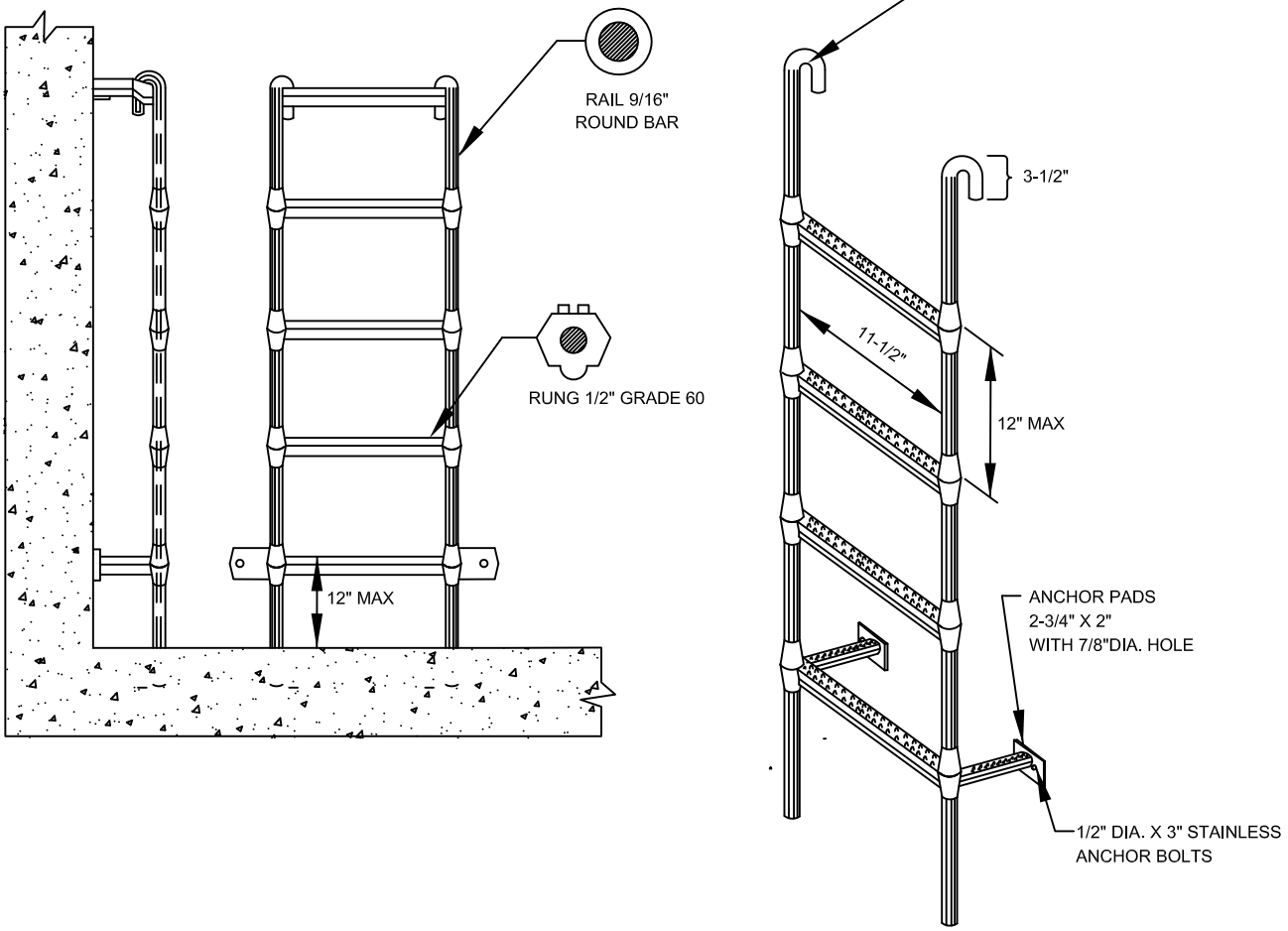
SNOHOMISH
WASHINGTON

TYPE 3 MANHOLE
(48" THROUGH 96")

Approved By:
S L S
City Engineer
Date: April 2010

608
Number

City of Snohomish Public Works Department



NOTES:

1. STEPS SHALL BE REINFORCED COPOLYMER PROPOLENE PLASTIC CONFORMING TO:

   (A) ASTM C 478 AND AASHTO M-199, HORIZONTAL LOAD SHALL BE 1500 LBS.

   (B) ASTM A615 GRADE 60 (DEFORMED REINFORCING STEEL BAR).

   (C) POLYPROPYLENE CONFORMS TO D-4101.

2. MANHOLE STEPS SHALL HAVE MOLDED SAFETY HAND GRIP. RED REFLECTORS ARE REQUIRED.

3. ALL FABRICATION DIMENSIONS INDICATED ARE MINIMUM.

4. THE ENTIRE POLYPROPYLENE PLASTIC MATERIAL SURROUNDING THE REINFORCING STEEL BAR SHALL BE CAST MONOLITHICALLY. MINIMUM COVER SHALL BE $\frac{3}{16}$ INCH.

5. STEPS SHALL BE AT A MINIMUM OF 12 INCHES.

6. STEPS SHALL BE INSTALLED IN ACCORDANCE WITH THE APPROVED MANUFATURER'S RECOMMENDED PROCEDURE.

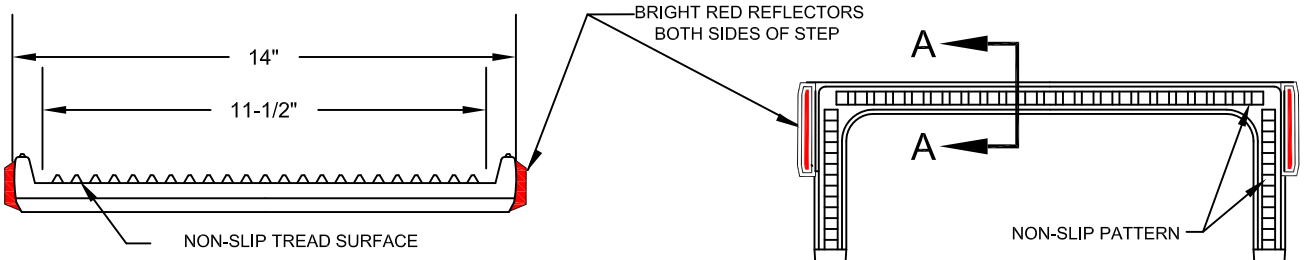
SNOHOMISH
WASHINGTON

POLYPROPYLENE LADDER

Approved By:
S L S
City Engineer
Date: April 2010

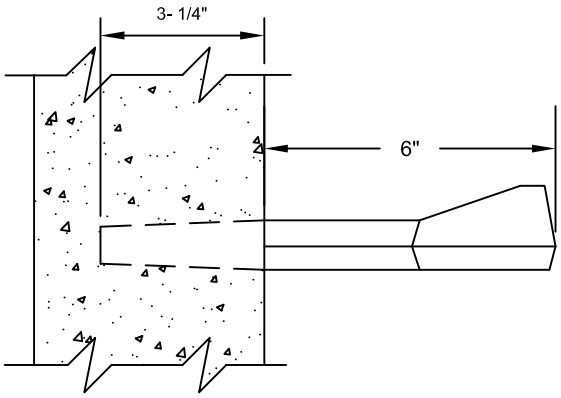
609
Number

City of Snohomish Public Works Department

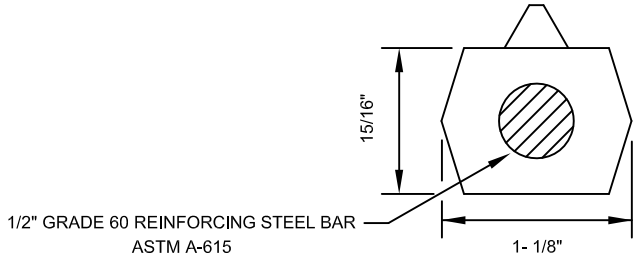


ELEVATION

PLAN



SIDE ELEVATION



SECTION A-A

NOTES:

1. STEPS SHALL BE REINFORCED COPOLYMER PROPOLENE PLASTIC CONFORMING TO:

   (A) ASTM C 478 AND AASHTO M-199, HORIZONTAL LOAD SHALL BE 1500 LBS.

   (B) ASTM A-615 GRADE 60 (DEFORMED REINFORCING STEEL BAR).

   (C) POLYPROPYLENE CONFORMS TO D-4101.

2. MANHOLE STEPS SHALL HAVE MOLDED SAFET YHAND GRIP. RED REFLECTORS ARE REQUIRED.
3. ALL FABRICATION DIMENSIONS INDICATED ARE MINIMUM.
4. THE ENTIRE POLYPROPYLENE PLASTIC MATERIAL SURROUNDING THE REINFORCING STEEL BAR SHALL BE CAST MONOLITHICALLY. MINIMUM COVER SHALL BE $\frac{3}{16}$ INCH.
5. STEPS SHALL BE SPACED AT A MAXIMUM OF 12 INCHES.
6. STEPS SHALL BE INSTALLED IN ACCORDANCE WITH THE APPROVED MANUFATURER'S RECOMMENDED PROCEDURE.

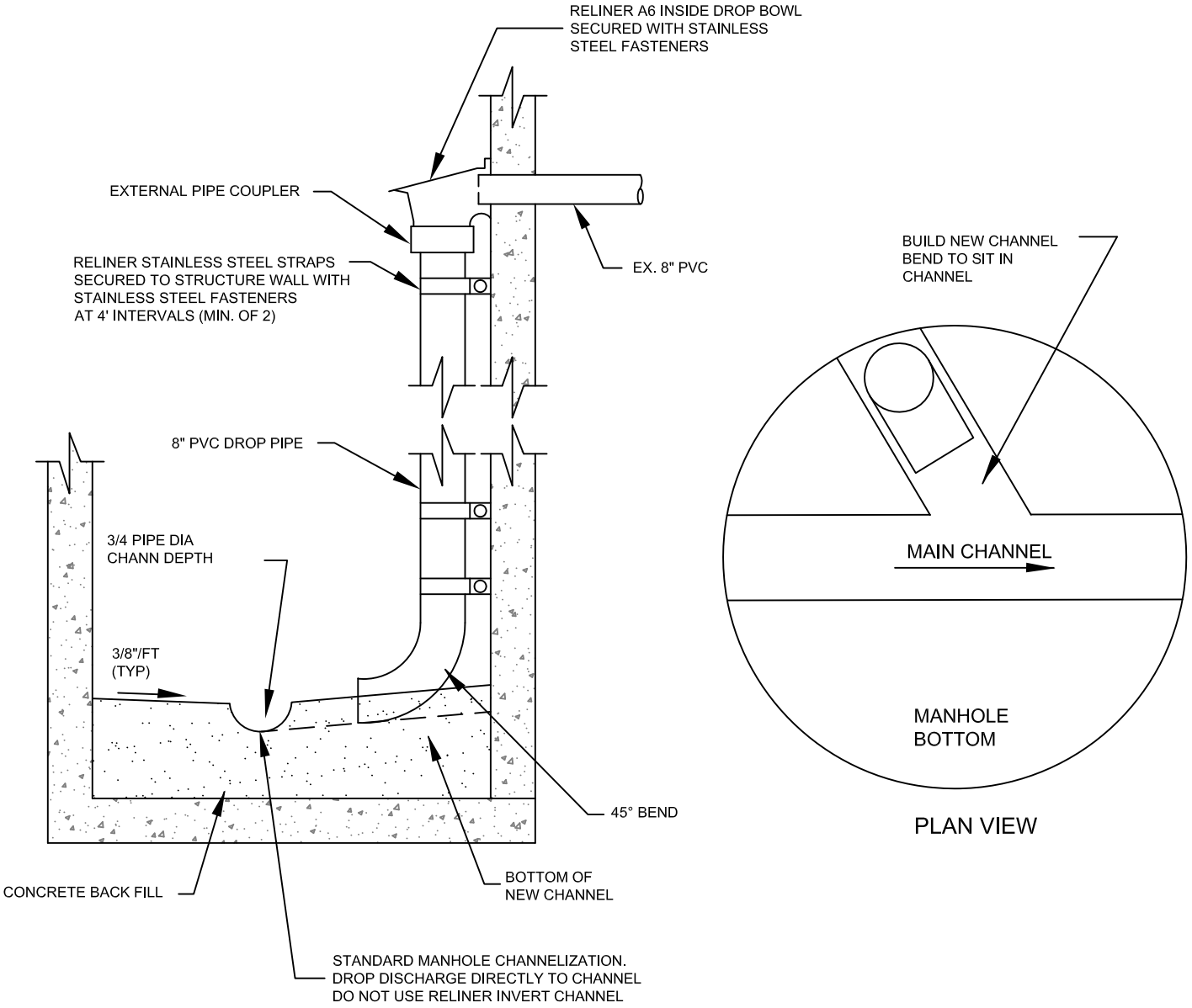
SNOHOMISH
WASHINGTON

POLYPROPYLENE PLASTIC STEP

Approved By:
S L S
City Engineer
Date: April 2010

610
Number

City of Snohomish Public Works Department



NTS

NOTE:
DROP INLET SHALL BE RELINER-DURAN W/A6 BOWL OR EQUAL AS APPROVED BY CITY.
INSTALL PER MANUFACTURER'S SPECIFICATIONS.

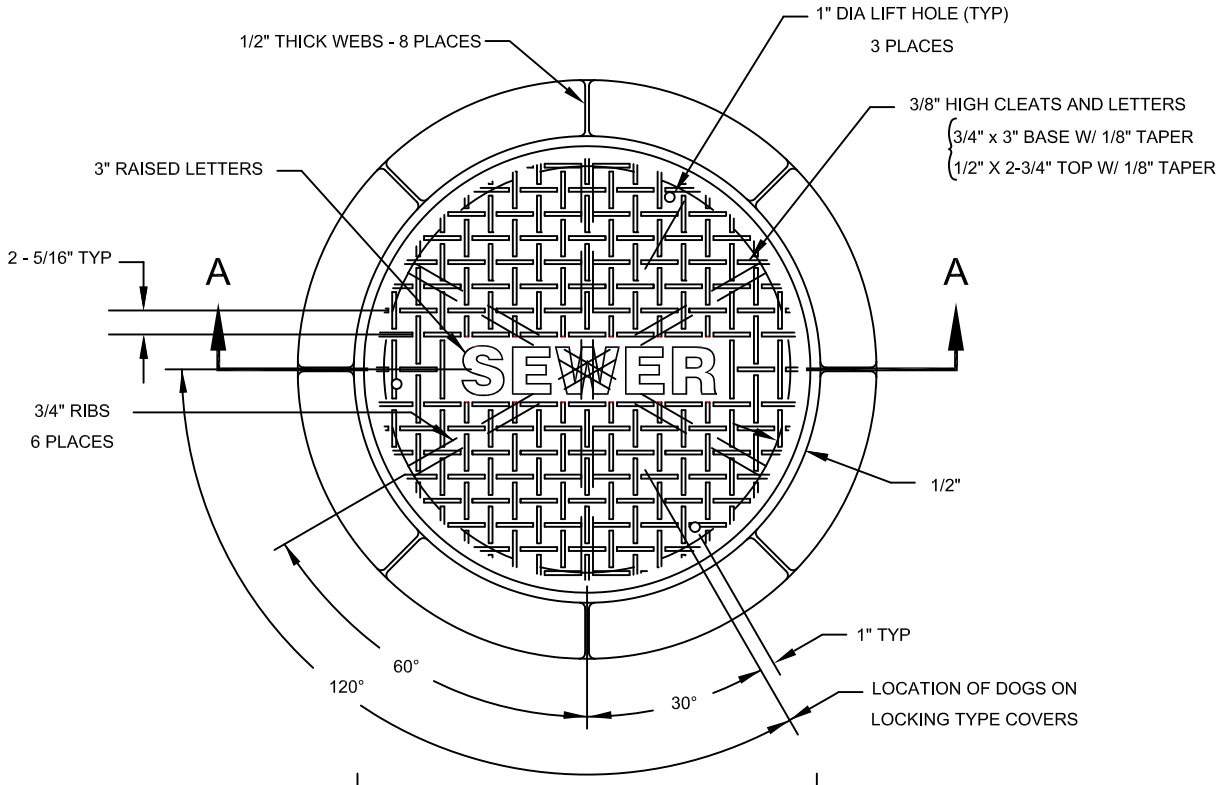
SNOHOMISH
WASHINGTON

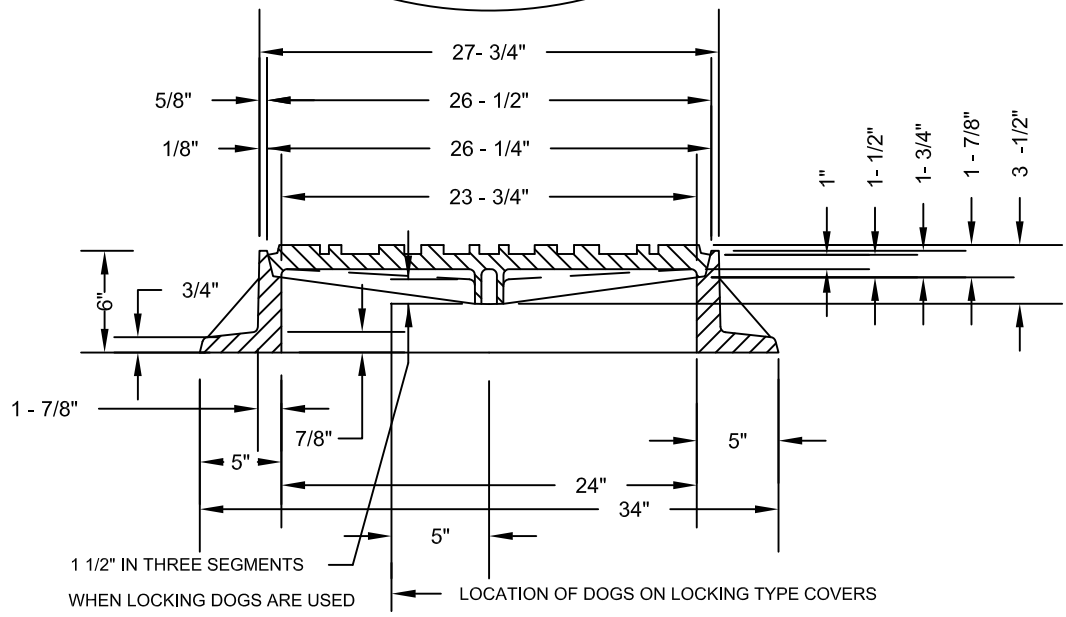
INSIDE DROP MANHOLE CONNECTION

Approved By:
S L S
City Engineer
Date: April 2010

611
Number

City of Snohomish Public Works Department





SECTION A-A

NOTES:

1. MANHOLE RINGS SHALL BE GRAY IRON CONFORMING TO THE REQUIREMENTS OF AASHTO M 105, GRADE 30B.
2. MANHOLE COVER TO BE DUCTILE IRON CONFORMING TO ASTM A536, GR 80-55-06 - EAST JORDAN IRON WORKS
3. LOCKING COVER SHALL BE USED IN ALL LOCATIONS. THE LID SHALL BE LOCKED DOWN WITH 3-5/8" S.S. SOCKET HEAD CAP SCREWS.
4. RING AND COVER SHALL BE TESTED FOR ACCURACY OF FIT AND SHALL BE MARKED IN SETS FOR DELIVERY.
5. ALL CASTINGS TO HAVE A BITUMINOUS COATING.

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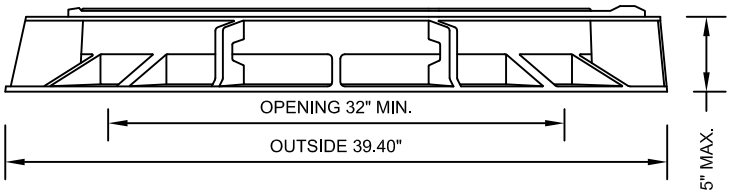
WASHINGTON

MANHOLE RING AND COVER

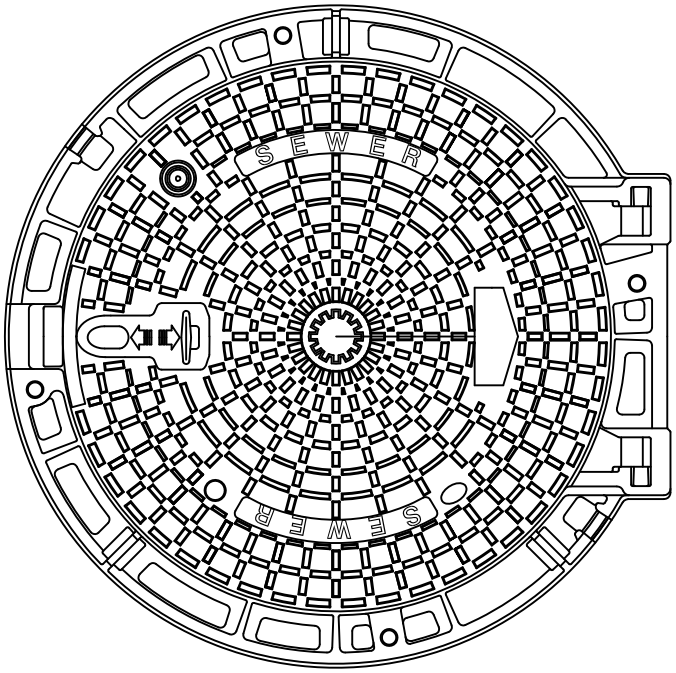
Approved By:
S L S
City Engineer
Date: April 2010

612
Number

City of Snohomish Public Works Department



ELEVATION



PLAN

NOTES:

1. MANHOLE COVER AND FRAME SHALL BE PARMEX OR APPROVED EQUAL. COVER AND FRAME SHALL BE MANUFACTURED FROM DUCTILE IRON.
2. COVER SHALL BE STAMPED SEWER.
3. COVERS SHALL BE DULY HINGED AND INCORPORATE A 90° BLOCKING SYSTEM TO PREVENT ACCIDENTAL CLOSURE. COVER SHALL BE ONE MAN OPERABLE USING STANDARD TOOLS AND SHALL BE CAPABLE OF WITHSTANDING A TEST LOAD OF 80,000 LBS.
4. FRAMES SHALL BE CIRCULAR, INCORPORATE A SEATING RING AND A FITTED PLUG IN EACH HINGE HOUSING AND BE AVAILABLE IN A 32" CLEAR OPENING. THE FRAME DEPTH SHALL NOT EXCEED 5 ", AND THE FLANGE SHALL INCORPORATE BEDDING SLOTS, BOLT HOLES AND LIFTING EYES.
5. ALL COMPONENTS SHALL BE BLACK COATED.
6. FRAME WEIGHT: 107 LBS
   COVER WIEGHT: 162 LBS.
   TOTAL WEIGHT: 269 LBS.

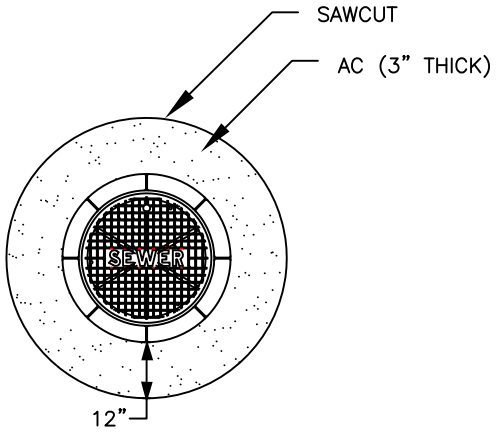
SNOHOMISH
WASHINGTON

HINGED MANHOLE COVER

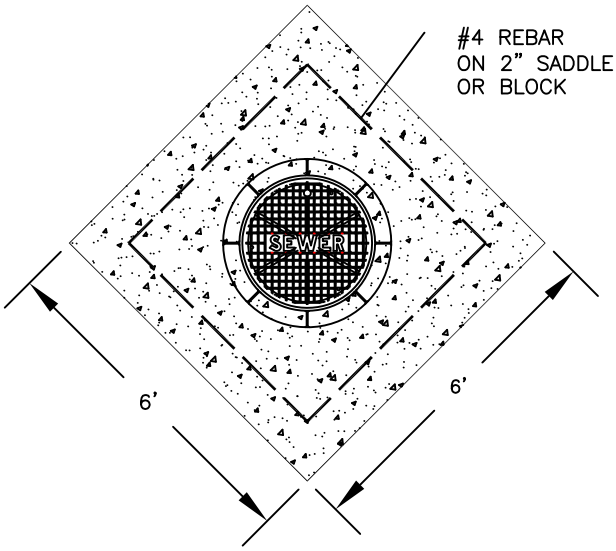
Approved By:
S L S
City Engineer
Date: April 2010

613
Number

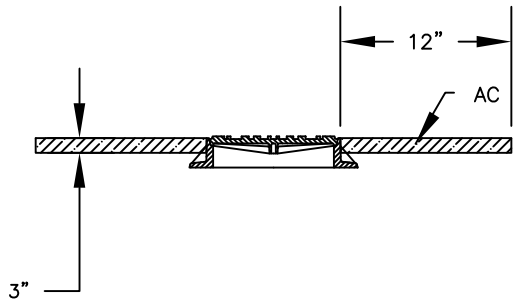
City of Snohomish Public Works Department

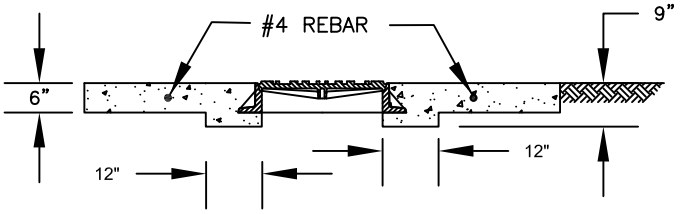


PAVED TRAFFIC AREAS



OUTSIDE PAVED TRAFFIC AREAS





NTS

NOTES:

1. CONCRETE SHALL CONFORM TO CURRENT STANDARDS AND SHALL BE 5.5 SACK MIX. (MINIMUM 3000-PSI 28-DAY STRENGTH)
2. WHERE DEPTH OF NECK EXCEEDS 26 INCHES, ADJUST MANHOLE TO GRADE BY INSERTING A NEW MANHOLE BARREL SECTION BETWEEN THE CONE AND EXISTING BARREL.
3. ADJUSTMENT RINGS/BLOCK, TOP OF CONE SECTION, AND BOTTOM OF IRON RING, SHALL BE WET STACKED IN 3/4" GROUT, PLASTER SMOOTH INSIDE AND OUT.
4. STEPS OR RUNGS SHALL BE ADDED AS NEEDED PER STND PLAN 608 AND 609.
5. PRECAST ADJUSTMENT RINGS SHALL BE CAST WITH GROOVE TO ALLOW FIELD INSTALLATION OF SAFETY STEP.
6. CONCRETE PERIMETER SEAL SHALL EXTEND TO 12 INCH MINIMUM OR 2 INCHES BELOW THE BOTTOM OF THE ADJUSTMENT RINGS OR BLOCKS.
7. SAWCUT ONLY. NO OVERCUTTING LIMITS OF PATCH.
8. IN NON-TRAFFIC AREAS (LANDSCAPED) THE CONCRETE PAD IS 4" THICK WITHOUT REINFORCEMENT.

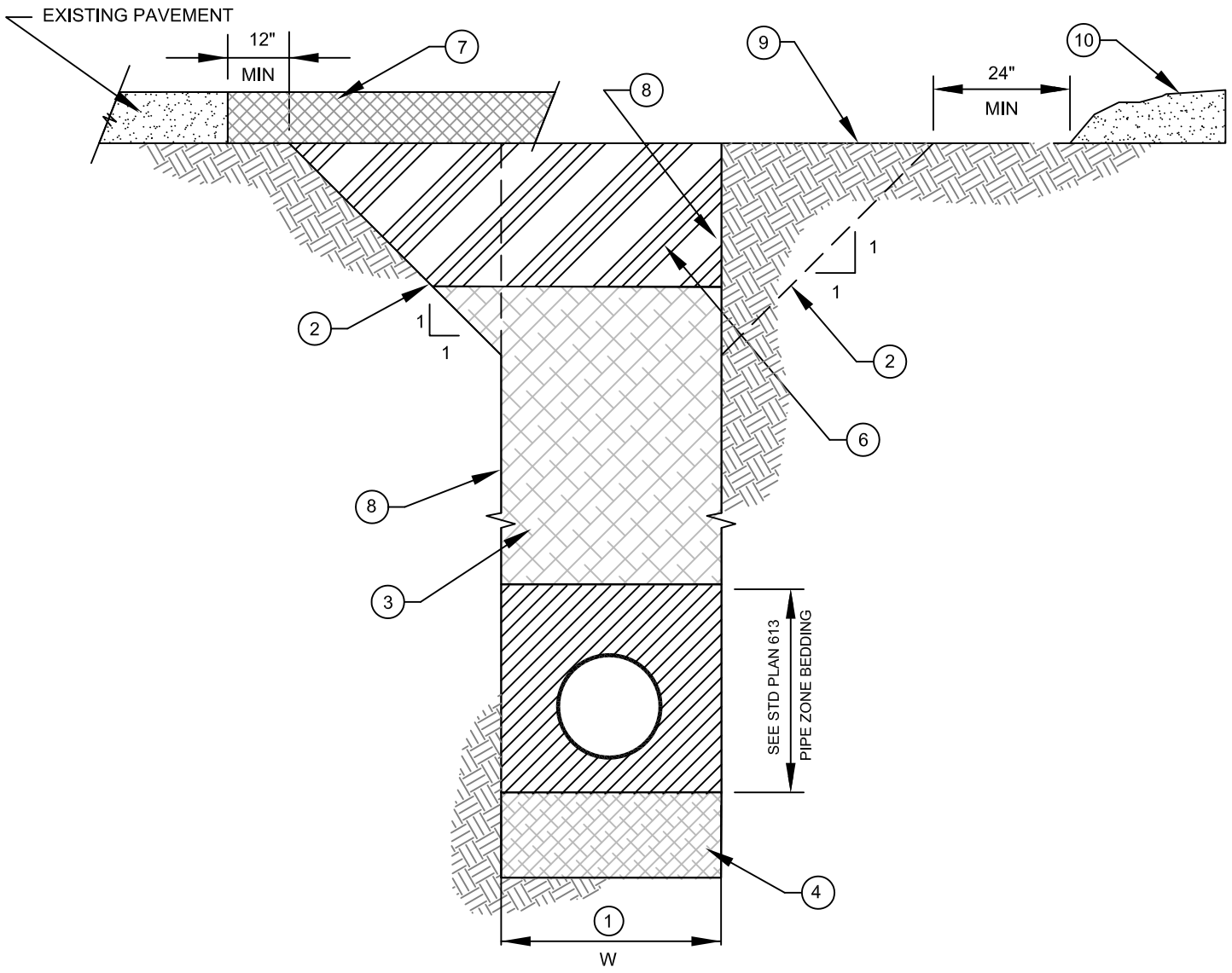
SNOHOMISH WASHINGTON

MANHOLE PAD AND ADJUSTMENT

Approved By:
S L S
City Engineer
Date: April 2010

614
Number

City of Snohomish Public Works Department



NOTES

1. W = MAXIMUM WIDTH OF TRENCH. FOR PIPES 15" OR LESS IN DIA, W = 40", FOR PIPES 18" OR GREATER, W = 1-1/2 X I.D. + 18".
2. ALTERNATE LAID-BACK TRENCH TO MEET O.S.H.A. REQUIREMENTS (NO SLOPES STEEPER THAN 1:1 EXCEPT FOR ROCK).
3. NATIVE MATERIAL IF ALLOWED IN ADVANCE BY THE CITY ENGINEER, OR IMPORTED GRAVEL BORROW AS DIRECTED SHALL COMPACT TO 90 % MAXIMUM DENSITY EXCEPT FROM SUBGRADE DOWN 4' WHICH SHALL BE CSTC.
4. CLASS A FOUNDATION GRAVEL, IF REQUIRED BY THE CITY ENGINEER TO REPLACE UNSUITABLE MATERIAL.
5. FOR ADDITIONAL COMPACTION INFORMATION SEE STANDARD PLAN 617.
6. THE TOP 4' OF BACKFILL SHALL BE CSTC. THE CONTRACTOR HAS THE OPTION TO FILL THE ENTIRE EXCAVATION TO SUBGRADE WITH CSTC. MINIMUM COMPATION 95% MAXIMUM DENSITY.
7. SEE CITY OF SNOHOMISH STANDARD PLAN 316 FOR PAVEMENT PATCH DETAILS.
8. VERTICAL TRENCH WALLS WITH SHORING SHALL CONFORM TO O.S.H.A. REGULATIONS.
9. SUBGRADE OR GROUND SURFACE IN NON-PAVED AREAS.
10. EXCAVATED NATIVE MATERIAL OR APPROVED STOCKPILED BACKFILL MATERIAL.

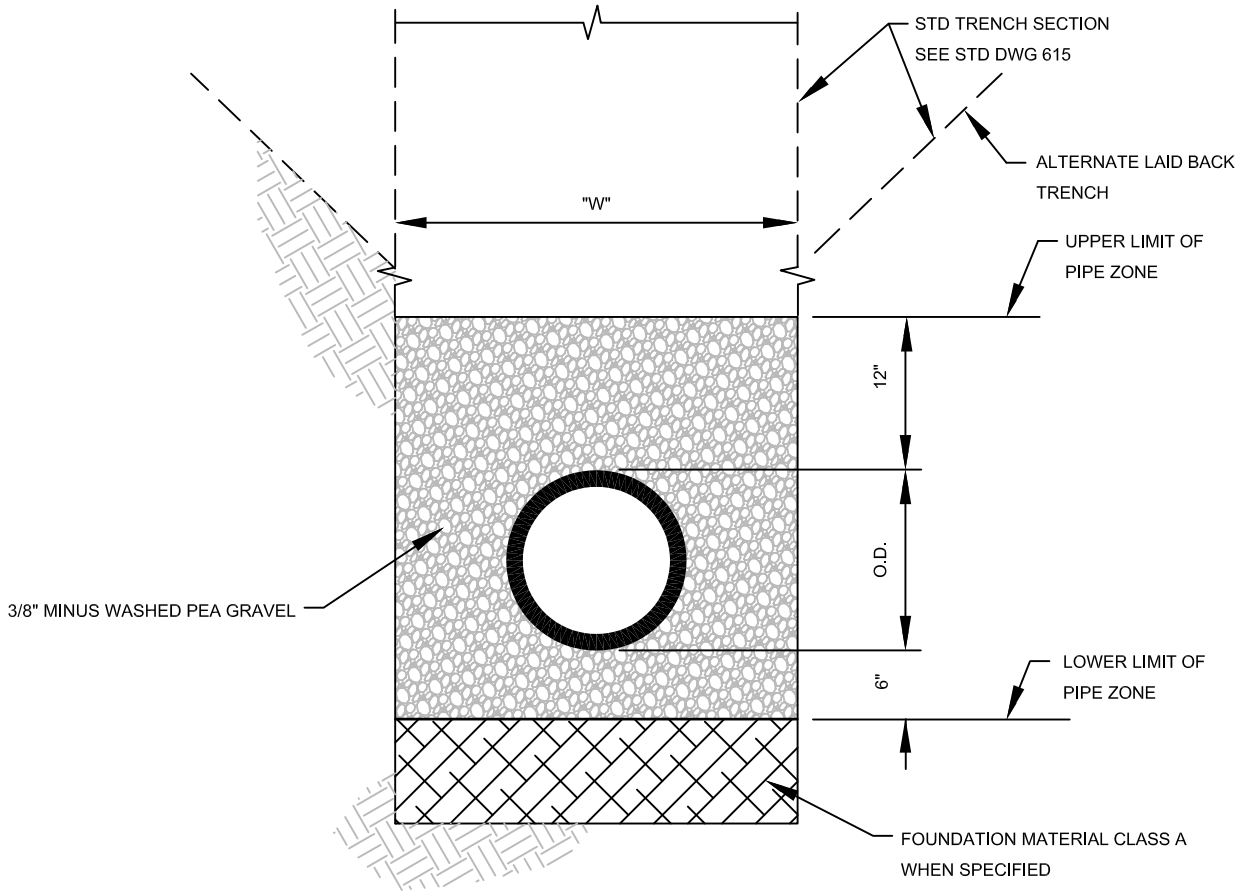
SNOHOMISH
WASHINGTON

TYPICAL SEWER TRENCH SECTION

Approved By:
S L S
City Engineer
Date: April 2010

615
Number

City of Snohomish Public Works Department



LIMITS OF TRENCH:

1. W = MAXIMUM WIDTH OF TRENCH. FOR PIPES 15" OR LESS IN DIA W = 40". FOR PIPES 18" OR GREATER, W = 1-1/2 X I.D. + 18"

BEDDING AND FOUNDATION MATERIALS:

1. BEDDING MATERIAL SHALL BE PEA GRAVEL CONCFORMING TO SECTION 9-03.12 (3) OF THE STANDARD SPECIFICATIONS FOR ROAD, BRIDGE AND MUNICIPAL CONSTRUCTION WSDOT/APWA.
2. FOUNDATION MATERIAL, IF REQUIRED, SHALL BE FOUNDATION MATERIAL CLASS A CONFORMING TO SECTION 9-03.12 OF THE STANDARD SPECIFICATIONS FOR ROAD, BRIDGE AND MUNICIPAL CONSTRUCTION WSDOT/APWA.

PROCEDURE FOR COMPACTION:

1. PROVIDE UNIFORM SUPPORT UNDER PIPE BARREL.
2. COMPACT BEDDING MATERIAL TO 90% MAXIMUM DENSITY EXCEPT DIRECTLY OVER PIPE. HAND TAMP ONLY.
3. HAND TAMP UNDER HAUNCHES.
4. FOR ADDITIONAL COMPACTION INFORMATION SEE STANDARD PLAN 617.

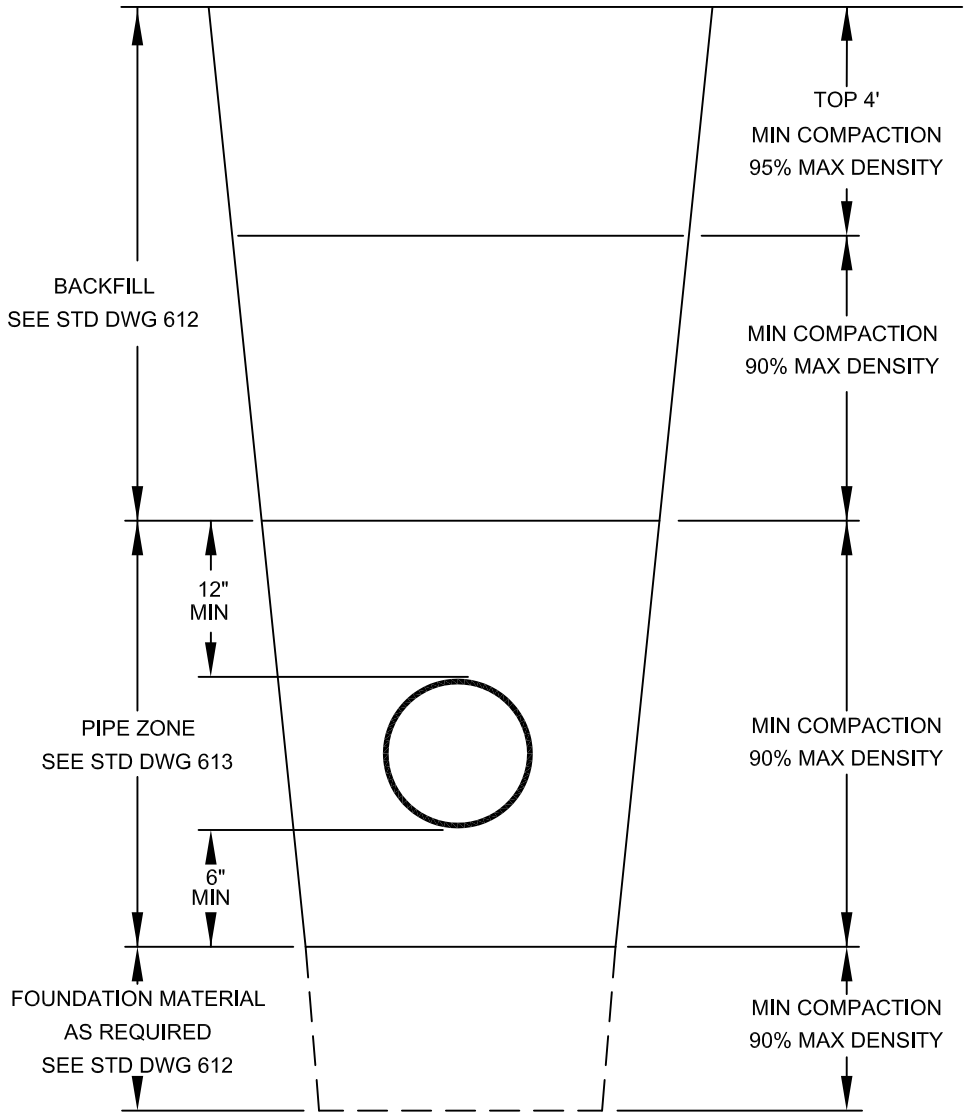
SNOHOMISH
WASHINGTON

BEDDING FOR SEWER PIPE IN TRENCHES

Approved By:
S L S
City Engineer
Date: April 2010

616
Number

City of Snohomish Public Works Department



SECTION

NTS

NOTES:

1. ALL BACKFLL MATERIAL SHALL BE PLACED IN LIFTS NOT TO EXCEED 12 INCHES BEFORE COMPACTION UNLESS AUTHORIZED BY THE CITY ENGINEER DUE TO THE CHARACTER OF THE MATERIAL AND COMPACTING EQUIPMENT.
2. MECHANICAL COMPACTION OF BACKFILL MATERIAL SHALL NOT BEGIN UNTIL THE DEPTH OF COMPACTED MATERIAL IS 2 FEET ABOVE THE TOP OF PIPE.
3. EACH LIFT SHALL BE MECHANICALLY COMPACTED TO THE REQUIRED DENSITY PRIOR TO PLACING SUCCEEDING LIFTS OF BACKFILL MATERIAL
4. COMPACTON TESTS SHALL BE AS REQUIRED BY THE CITY ENGINEER, BUT IN NO CASE LESS THAN 2 TESTS EVERY 200 FEET OF TRENCH LENGTH (ONE AT SUBGRADE AND ONE AT 50% OF TRENCH DEPTH).
5. IN PLACE DENSITY WILL BE DETERMINED BY ONE OR MORE OF THE FOLLOWING METHODS
   ASTM D1556 – TEST FOR DENSITY OF SOIL IN PLACE BY THE SAND CONE METHOD.
   ASTM D2167 – (RUBBER. BALLON METHOD)
   ASTM D2922 – (NUCLEAR METHOD)
6. LABORATORY DENSITY WILL BE DETERMINED BY ASTM D698, MOISTURE DENSITY RELATIONS OF SOILS AND SOIL AGGREGATE MIXTURES.

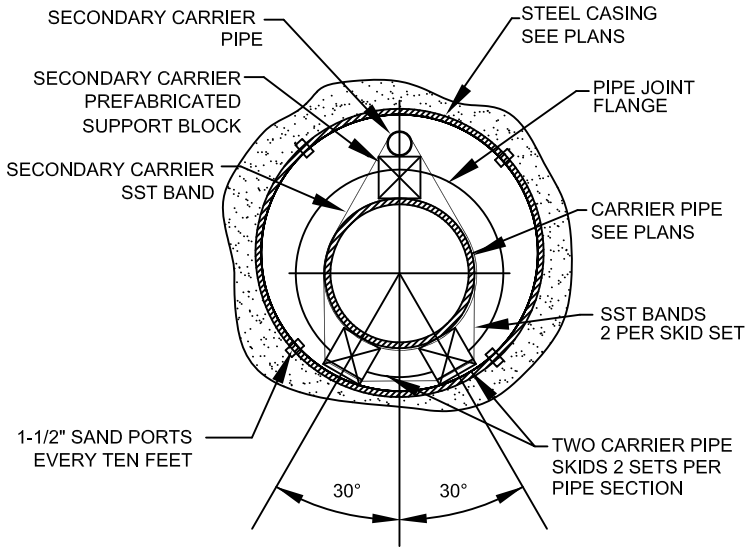
SNOHOMISH
WASHINGTON

TYPICAL SEWER TRENCH COMPACTION

Approved By:
S L S
City Engineer
Date: April 2010

617
Number

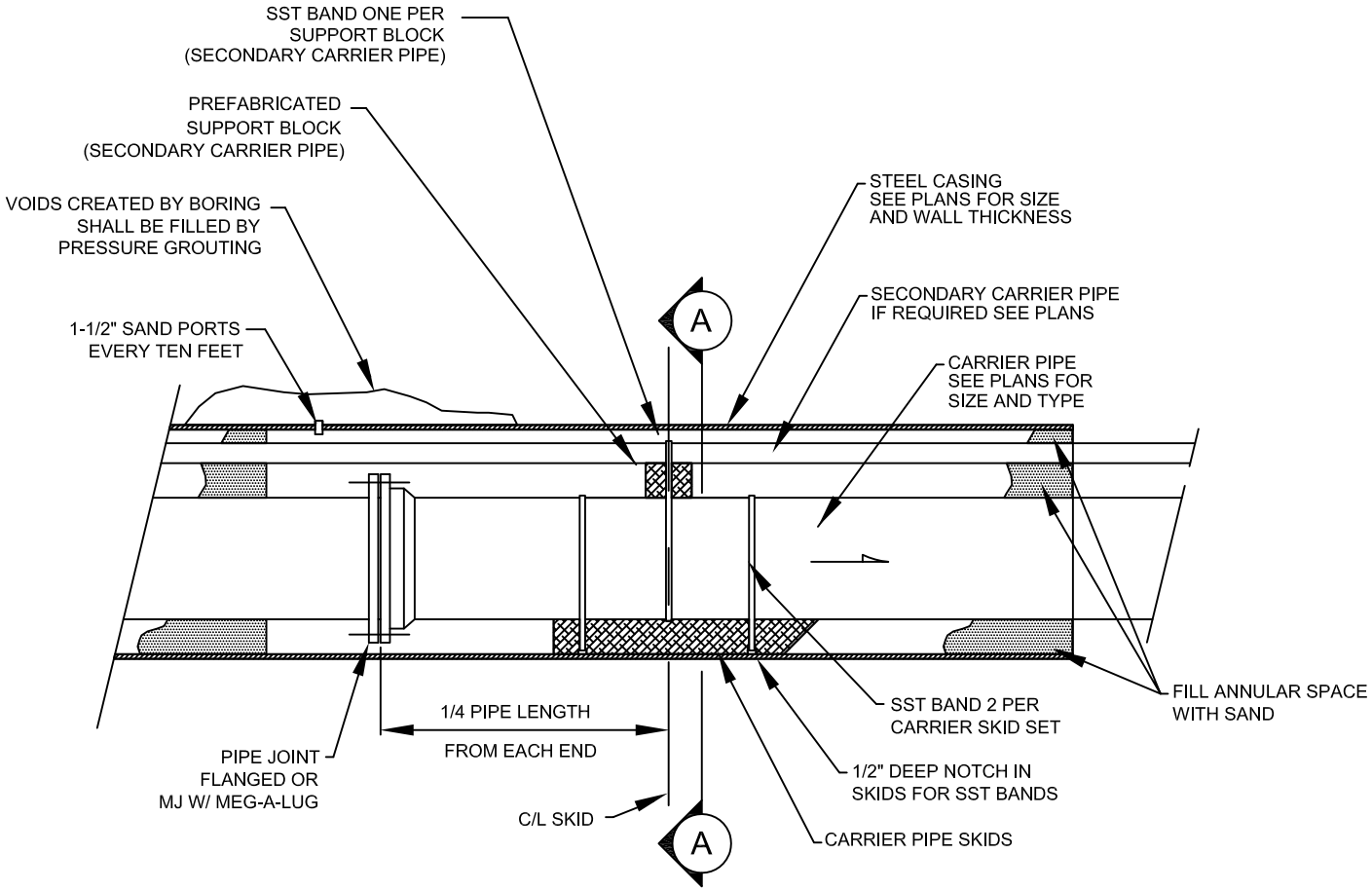
City of Snohomish Public Works Department



A TYPICAL SECTION
NTS

CONSTRUCTION NOTES:

1. ALL JOINTS OF CARRIER PIPE WITHIN CASING SHALL BE FLANGED (FL) OR MECHANICAL JOINT (MJ) FITTINGS WITH MEG-A-LUG
2. RESTRAINTS.
   CARRIER PIPE WILL BE PRESSURE TESTED BY CONTRACTOR AND TV'D BY CITY CREWS BEFORE FILLING ANULAR SPACE OF CASING PIPE WITH SAND.
3. CARRIER SKIDS SHALL BE SECURELY ATTACHED TO CARRIER PIPE W/STAINLESS STEEL BANDS (MIN 2 BANDS PER SKID SET).
4. CARRIER SKIDS SHALL BE ROUNDED OR BEVELED ON LEADING EDGE, AND SHALL BE NOTCHED TO RECEIVE SST BANDS.
5. CARRIER SKIDS SHALL BE APPROVED PREFABRICATED SKIDS. A 1" MIN CLEARANCE SHALL BE MAINTAINED BETWEEN JOINT FLANGE AND CASING.
6. SECONDARY CARRIER PIPES SHALL BE SECURED TO THE TOP OF THE MAIN CARRIER PIPE AS SHOWN.
7. SECONDARY CARRIER PIPE WILL BE TESTED BEFORE CASING ANNULAR SPACE IS FILLED.



SNOHOMISH WASHINGTON

CASING DETAIL

Approved By: S L S
City Engineer
Date: April 2010

618
Number

City of Snohomish Public Works Department

Appendix M

Fuzzy Filter Pilot Report

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Pilot Testing of:

Synthetic Compressible Media Filter

City of Snohomish Public Works
Snohomish, WA
October 10, 2018

SCHREIBER®
Pure Ingenuity

Synthetic Compressible Media Filter
Pilot Test Results

October 10, 2018

Location: Snohomish, WA
Application: Lagoon Effluent

Pilot Filter: Unit FX13
Filter Size/Area= 18"x18" / 2.25 SF Filtration area

Summary:

The pilot test was conducted from August 20, 2018 through September 14, 2018 at the Snohomish Waste Water Treatment Plant. The pilot unit employed was a compressible synthetic media ("Fuzzy") filter in an up-flow configuration.

The 18" square pilot unit employs a filtration media of quasi-spherical fiber balls. The media bed is approximately 30" deep (uncompressed) and has a solids storage capacity of approximately 1.2 lbs TSS/CF of uncompressed media. Two horizontal perforated steel plates are used to positively retain the media from above and below. The plate perforations are 1/2" diameter, which precludes media passage. The lower plate is fixed in position while the upper plate is vertically adjustable by means of a mechanical actuator. The upper plate serves to compress the media to a desired level between 0% and 40% of the original bed depth. This adjustment changes the pore size of the media and the filtration characteristics. Hydraulic loading rates for the filter typically range from 20 gpm/sf to 30 gpm/sf; units have been operated as high as 40 gpm/sf.

The filter washes by violently fluidizing the bed by air injection and scrubbing the fiber media balls against each other to break loose stored solids. On washing, the upper plate rises to allow vertical space for media expansion and agitation. Since the specific gravity of the media is roughly 1.05, very little energy is required to fluidize and scrub the media. An upflow wash water stream at 10 gpm/sf is used in conjunction with air scouring at 15 scfm/sf; filter influent is used for wash water as opposed to stored filter effluent. After washing, the media is recompressed and begins a purge or filter–to-waste stage, similar to granular filters.

The flexibility of the filter is due to the variable compression of the media. Initial operation consists of varying different parameters (compression, flow rate, terminal head loss, and any chemical addition) to determine the optimum settings for a particular application.

Advantages of this technology are:

Simple design with great operational flexibility

- Adjustable Pore Size thru changes in compression to 4 microns
- Very High Solids Storage - 85% void volume available
- High Flow Rate/Unit Area (30 to 45 gpm/sq ft) - 750 gpm handled by a single 5' x 5' filter @ 30 gpm/ft^2
- Small Footprint – 20% of sand filter area; reduced foundation, piping & valve expense
- No Media Loss - no under drains and media to replace
- Excellent Media Longevity – 10 years+ life
- No Clean Water Required For media washing, influent is used – Sand filters often require filtered water and a separate storage tank and pumps for backwashing
- Low wash water consumption - normally 1% to 2%
- Complete Automation with PLC & Touch Screen Interface

Individual Filter sizes are available from 70 to 2000 gpm – Filters can be combined to achieve any flow requirement

Wash Water Discharge

The filter test unit is mounted in an enclosed trailer complete with instrumentation and automatic controls. Flow for the pilot filter's influent is controlled the by a manual hand valve on the influent line for filtering and washing located in the piping between the feed pump and the Fuzzy Filter. The hydraulic loading rate was tested at different flux rates (gpm/ft^2) which were between 26 and 34 gpm/ft^2. Typical flux rates are 30 gpm/ft^2. Due to the sensitivity of the 4" hand valve on the influent pump line the hydraulic loading varied slightly for each filter run. The Influent water to the filter was fed from the main influent box at the existing sand filters.

Influent Location

Filter's Effluent Discharge

Wash Water Discharge

This procedure is intended to adapt the filter to the characteristic particle size distribution of the specific influent. At 20% to 40% compression, removals of the majority of particles of 4-micron size are typical. For 10 micron and larger particle size, removals equal or exceed those for granular filters. Smaller particle sizes are removed at correspondingly smaller percentages. At higher compressions, smaller size particles would be removed due to the smaller pore size in the media.

Dirty filter media nearing time for wash cycle

Media in wash cycle—air scouring/agitation to release solids entrapped in the media

Recompressed clean media after a wash cycle

Test Operation Discussion

The filter test site was located in Snohomish, WA. Testing was performed on the effluent of the 4th lagoon. This is the same water that the existing sand filters receive. The effluent of both the sand filters and the Fuzzy Filter pilot was sent to the chlorination/dechlorination contact tanks.

The objective of the Fuzzy Filter pilot test was to determine if the filter can remove suspended solids/ turbidity (algae) from the plants effluent, so it could be used instead of the sand filters. The Pilot System used the plant's existing set-up for its chemical feed system. We first ran tests using the plant's existing polymer SP-218-GH and then we tested with JC-1679, a blend supplied by Jenfitch. The SP-218-GH is a cationic acrylamide copolymer which is normally fed at 2 ml/min as a neat solution into a poly-blend chemical feed system and then is injected into the main feed pipe to the sand filters. The JC-1679 is an Aluminum Chlorohydrate blend (ACH) that was fed at 2, 4 or 6 ml/min as a neat solution into the poly-blend chemical feed system and then is injected into the main feed pipe to the sand filters. The influent pump feeding the Pilot Fuzzy Filter was located in the influent box of the sand filters. Influent and effluent turbidities were automatically recorded at 5 minute intervals along with the flow and pressure readings. The pilot testing objective was to compare the Fuzzy Filter to the performance of the existing sand filters.

Performance Summary

The filter media cleaned acceptably when the SP-218-GH was employed, but the filter media cleaned better when no polymer or the JC-1679 was used. This observation was made by both Schreiber and plant personal during the air scouring and backwashing procedure of the media. The Schreiber online turbidity meters measures readings every 3 seconds which are signal averaged over 30 seconds to provide each output value. The online turbidity meters were calibrated several times using 20NTU formazin and verified several times using the Hach ICE-PIC 20NTU. TSS samples were grab samples of both the influent to the filter and the effluent exiting the filter. These samples were taken and analyzed for filtration runs performance during the plant operator working hours.

Flow and inlet pressure (head loss indicator) were also continuously monitored and recorded. Filter operation was continuous for each full test period. Manual wash was initiated from the touch screen for some runs to facilitate resetting variables for the next test or showing plant personal how the filter operated and washed. The majority of media wash operations were timer initiated with or without chemical feed The typical control scheme places the filter in wash by pressure or run time (that were set at 3 to 5.5 hours), which ever occurs first. These values are adjustable.

There were more than 65 test runs performed, and data was recorded for the first 65 runs performed by the Fuzzy Filter Pilot Test System. The test used Schreiber standard 20D media.

Conclusions

The Snohomish waste water plant chemical feed system was used. The chemical feed system was a poly-blend chemical system. The injection port was the plant's normal injection point to the effluent of the 4th lagoon before entering the existing sand filters. There were eighteen(18) TSS samples taken during these runs. The SP-218-GH was added at a rate of 2ml/min of 100% concentration to the flow of about 800,000 GPD. There was a total of twenty-nine (29) runs with SP-218-GH where data was collected.

Test runs with SP-218-GH:

- The influent turbidity values varied from 5.43 NTU to 7.61 NTU with an overall average of 6.26 NTU.
- The effluent turbidity values varied from 3.22 NTU to 5.69 NTU with an overall average of 4.00 NTU.
- The test run average % removal on turbidity varied from 24.4% to 54.1% with an overall average of 36.15%. The low % removal was due to the green color of the water.

The following TSS test results were only done while the Schreiber Testing Manager was on site.

- The influent TSS varied from 12.33 mg/L to 17.09 mg/L with an overall average of 14.74 mg/L.
- The effluent TSS varied from 4.92 mg/L to 8.88 mg/L with an overall average of 6.33 mg/L.
- The test run average % removal on TSS varied from 48.1% to 62.5% with an overall average of 57.24%.

Conclusions (continue)

The Snohomish waste water plant chemical feed system was used. The chemical feed system was a poly-blend chemical system. The injection port was the plant's normal injection point to the effluent of the 4th lagoon before entering the existing sand filters. There were sixteen(16) TSS samples taken during these runs. The JC-1679 was added at a different rates of 2, 4, and 6 ml/min of 100% concentration to the flow of about 800,000 GPD. There was a total of twelve(12) runs with JC-1679 where data was collected.

Test Runs with JC-1679

- The influent turbidity varied from 5.18 NTU to 6.08 NTU with the overall average of 5.48 NTU.
- The effluent turbidity varied from 3.26 NTU to 3.90 NTU with the overall average of 3.63 NTU.
- The test run average % removal on turbidity varied from 28.4 % to 37.0% with an overall average 33.82%

The following TSS test results were only done while the Schreiber Testing Manager was on site.

- The influent TSS varied from 15.00 mg/L to 11.00 mg/L with an overall average of 12.13 mg/L.
- The effluent TSS varied from 3.83 mg/L to 6.25 mg/L with an overall average of 5.65 mg/L.
- The test run average % removal on TSS varied from 44.8% to 67.8% with an overall average of 53.01%.

There were no runs without chemicals while the Schreiber Testing Manager was on site, so no TSS tests were done.

Test Runs without chemicals:

- The influent turbidity varied from 5.03 NTU to 7.08 NTU with the overall average of 5.65 NTU.
- The effluent turbidity varied from 3.90 NTU to 5.07 NTU with the overall average of 3.28 NTU.
- The test run average % removal on turbidity varied from 12.7 % to 33.8% with an overall average 23.02%

Conclusions (continue)

During the filter test while the SP-218-GH was being fed into the system, it was noticed that the solids would build up more on one side (glass side) causing a small amount of solids to be released up the glass. The wash timer was reduced to stop the release or short circuiting of solids. Also, media darkened further into the runs using SP-218-GH. Wash stage timers were increased to extend the wash cycle. Sunday August 26th a media bleach procedure was performed. JC-1679 permitted an increase of the wash timer up to 5.5 hours for longer run times.

New Chemical Feed—JC-1679 Dosage Schedule

Monday, August 27th—2 ml/min

Tuesday, August 28th—4 ml/min

Wednesday August 29th—6 ml/min

The filter was turned off on Friday September 14th, before the Schreiber Rep arrived at the plant for break down. The Rep powered the unit back up, but was unable to download the data. The data from mid morning September 8th until the morning September 14th is therefore not available for this report.

The Plant personal also perform some TSS sampling and testing on both the sand filter and Fuzzy Filter. The results are included on the average sheet in this report.

The following pages show testing data tables of filtration parameters for August 20, 2018 through September 8, 2018 . Average sheet showing:

1. Turbidity
2. Flow rate
3. Hydraulic loading
4. Gallons filtered
5. Wash water volume
6. % wash water
7. Factor initiating a wash
8. Media % compression
9. Comment section

APPENDICES

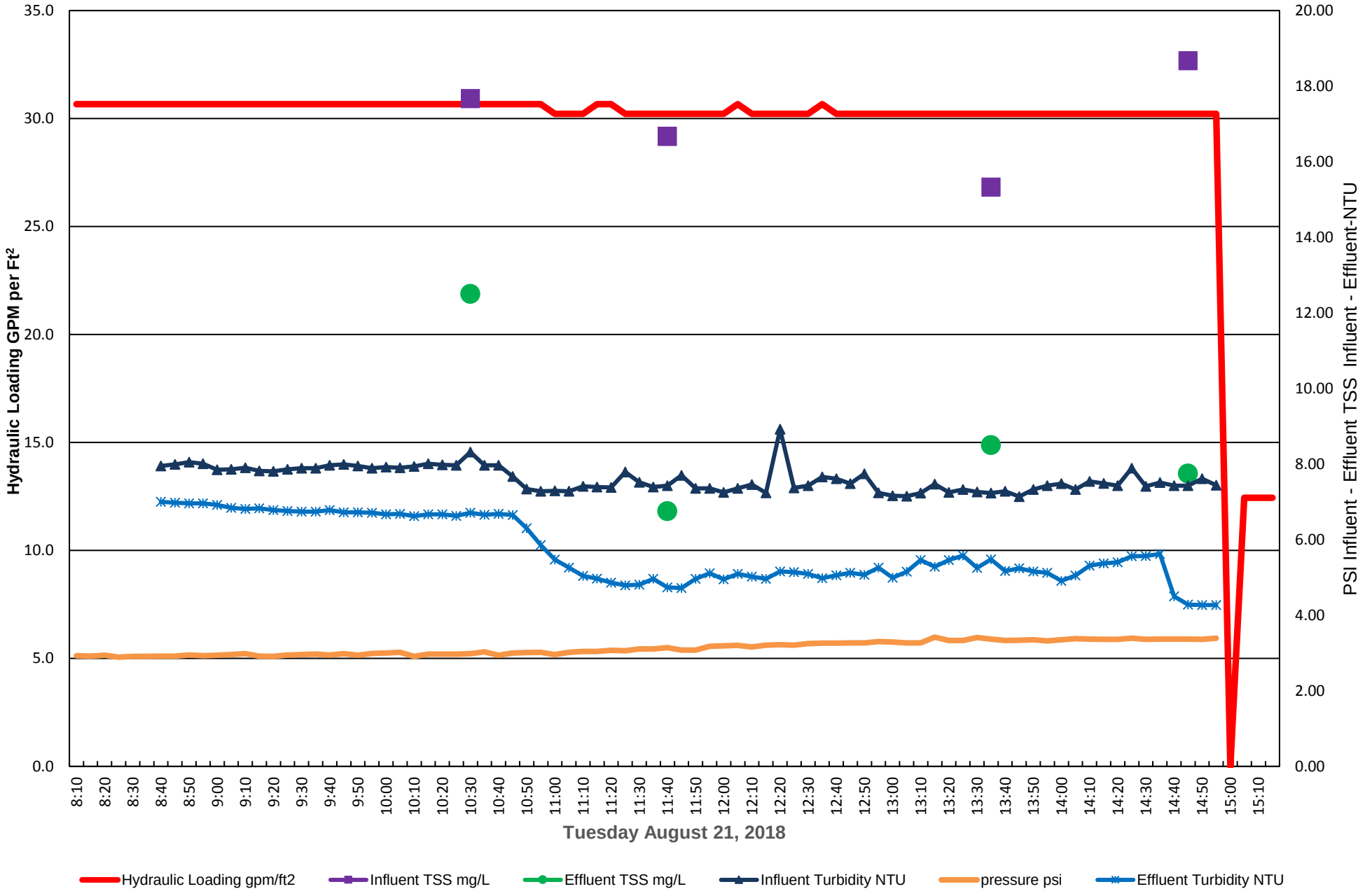
1. Test Runs (Turbidity, Flow, and Pressure)
   A. Raw Data
2. Average Summary Sheet and Plant Data
3. Pilot Filter Drawing

APPENDIX 1

Influent and Effluent Turbidity, Flow, and Pressure

SCHREIBER FuzzyFilter

Snohomish, WA Filter Test
20% Compression



8/21/2018
Tuesday
20% Compression
Run #1

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
8:10							2.93	69	30.7	Working on Both TU Meters	20	5.0	345	
8:15							2.92	69	30.7				345	
8:20							2.94	69	30.7				345	
8:25							2.89	69	30.7				345	
8:30							2.91	69	30.7				345	
8:35							2.91	69	30.7				345	
8:40	7.95	7.00	11.9%				2.92	69	30.7				345	
8:45	7.99	6.98	12.6%				2.92	69	30.7				345	
8:50	8.05	6.96	13.5%				2.95	69	30.7				345	
8:55	8.01	6.96	13.1%				2.93	69	30.7				345	
9:00	7.85	6.92	11.8%				2.94	69	30.7				345	
9:05	7.86	6.84	13.0%				2.96	69	30.7				345	
9:10	7.90	6.81	13.8%				2.98	69	30.7				345	
9:15	7.82	6.83	12.7%				2.92	69	30.7				345	
9:20	7.81	6.78	13.2%				2.91	69	30.7				345	
9:25	7.86	6.76	14.0%				2.95	69	30.7				345	
9:30	7.89	6.74	14.6%				2.96	69	30.7				345	
9:35	7.89	6.74	14.6%				2.97	69	30.7				345	
9:40	7.97	6.78	14.9%				2.95	69	30.7				345	
9:45	7.99	6.72	15.9%				2.98	69	30.7				345	
9:50	7.95	6.72	15.5%				2.94	69	30.7				345	
9:55	7.89	6.71	15.0%				2.99	69	30.7				345	
10:00	7.92	6.67	15.8%				3.00	69	30.7				345	
10:05	7.90	6.68	15.4%				3.02	69	30.7				345	
10:10	7.94	6.62	16.6%				2.91	69	30.7				345	
10:15	8.01	6.67	16.7%				2.97	69	30.7				345	
10:20	7.98	6.67	16.4%				2.97	69	30.7				345	
10:25	7.97	6.63	16.8%				2.97	69	30.7				345	
10:30	8.32	6.71	19.4%	17.67	12.50	29.3%	2.98	69	30.7	TSS Samples			345	
10:35	7.97	6.66	16.4%				3.03	69	30.7				345	
10:40	7.97	6.68	16.2%				2.94	69	30.7				345	
10:45	7.67	6.65	13.3%				3.00	69	30.7	Plant starting to feed SP-218-GH at 2ml/min to a flow of 800,000gallons/day flow			345	
10:50	7.34	6.30	14.2%				3.01	69	30.7				345	
10:55	7.28	5.86	19.5%				3.02	69	30.7				345	
11:00	7.29	5.47	25.0%				2.96	68	30.2				340	
11:05	7.28	5.26	27.7%				3.02	68	30.2				340	
11:10	7.41	5.04	32.0%				3.04	68	30.2				340	
11:15	7.39	4.97	32.7%				3.04	69	30.7				345	
11:20	7.38	4.86	34.1%				3.07	69	30.7				345	
11:25	7.79	4.79	38.5%				3.06	68	30.2				340	
11:30	7.52	4.81	36.0%				3.11	68	30.2				340	
11:35	7.39	4.96	32.9%				3.11	68	30.2				340	
11:40	7.43	4.74	36.2%	16.67	6.75	59.5%	3.14	68	30.2	TSS Samples			340	
11:45	7.70	4.72	38.7%				3.08	68	30.2				340	
11:50	7.36	4.96	32.6%				3.08	68	30.2				340	
11:55	7.36	5.11	30.6%				3.18	68	30.2				340	
12:00	7.25	4.95	31.7%				3.19	68	30.2				340	
12:05	7.36	5.09	30.8%				3.20	69	30.7				345	
12:10	7.46	5.02	32.7%				3.16	68	30.2				340	
12:15	7.24	4.96	31.5%				3.21	68	30.2				340	

Run #1	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading		Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate		Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
12:20	8.92	5.16	42.2%				3.22	68	30.2				340		
12:25	7.37	5.14	30.3%				3.21	68	30.2				340		
12:30	7.43	5.09	31.5%				3.25	68	30.2				340		
12:35	7.66	4.98	35.0%				3.26	69	30.7				345		
12:40	7.61	5.06	33.5%				3.26	68	30.2				340		
12:45	7.48	5.12	31.6%				3.27	68	30.2				340		
12:50	7.74	5.07	34.5%				3.27	68	30.2				340		
12:55	7.24	5.26	27.3%				3.30	68	30.2				340		
13:00	7.16	4.99	30.3%				3.29	68	30.2				340		
13:05	7.15	5.15	28.0%				3.27	68	30.2				340		
13:10	7.23	5.46	24.5%				3.27	68	30.2				340		
13:15	7.47	5.28	29.3%				3.42	68	30.2				340		
13:20	7.25	5.46	24.7%				3.33	68	30.2				340		
13:25	7.33	5.58	23.9%				3.33	68	30.2				340		
13:30	7.26	5.25	27.7%				3.41	68	30.2				340		
13:35	7.23	5.48	24.2%	15.33	8.50	44.6%	3.37	68	30.2	TSS Samples			340		
13:40	7.28	5.17	29.0%				3.33	68	30.2				340		
13:45	7.14	5.24	26.6%				3.34	68	30.2				340		
13:50	7.33	5.16	29.6%				3.35	68	30.2				340		
13:55	7.43	5.12	31.1%				3.32	68	30.2				340		
14:00	7.48	4.91	34.4%				3.35	68	30.2				340		
14:05	7.33	5.05	31.1%				3.38	68	30.2				340		
14:10	7.54	5.31	29.6%				3.37	68	30.2				340		
14:15	7.49	5.37	28.3%				3.36	68	30.2				340		
14:20	7.43	5.40	27.3%				3.36	68	30.2				340		
14:25	7.89	5.56	29.5%				3.39	68	30.2				340		
14:30	7.41	5.57	24.8%				3.36	68	30.2				340		
14:35	7.51	5.62	25.2%				3.37	68	30.2				340		
14:40	7.43	4.50	39.4%				3.37	68	30.2				340		
14:45	7.43	4.28	42.4%	18.67	7.75	58.5%	3.37	68	30.2	TSS Samples			340		
14:50	7.61	4.27	43.9%				3.36	68	30.2				340		
14:55	7.44	4.27	42.6%				3.39	68	30.2				340		
15:00								0		Manual Wash				0	
15:05								28	12.4					140	**Percent**
15:10								28	12.4					140	**of wash**
15:15								28	12.4					140	**%**
Averages	7.61	5.69	25.3%	17.09	8.88	48.1%	3.13	66.26	29.79				28070	420	1.47%

8/21/2018
Tuesday
30% Compression
Run #2

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
15:20	7.10	4.20	40.8%				3.08	69	30.7	2ml/min SP-218-GH	30	5.0	345	
15:25	8.60	6.25	27.3%				3.02	67	29.8				335	
15:30	7.21	5.76	20.1%				3.06	69	30.7				345	
15:35	7.18	5.54	22.8%				3.10	68	30.2				340	
15:40	7.29	5.30	27.3%				3.02	68	30.2				340	
15:45	7.27	5.19	28.6%				3.09	69	30.7				345	
15:50	7.25	5.06	30.2%				3.08	68	30.2				340	
15:55	7.37	4.96	32.7%				3.15	68	30.2				340	
16:00	7.37	4.92	33.2%				3.14	68	30.2				340	
16:05	7.41	4.92	33.6%				3.18	68	30.2				340	
16:10	7.41	4.88	34.1%				3.20	68	30.2				340	
16:15	7.45	4.85	34.9%				3.23	68	30.2				340	
16:20	7.49	4.77	36.3%				3.26	68	30.2				340	
16:25	7.34	4.74	35.4%				3.21	68	30.2				340	
16:30	7.33	4.70	35.9%				3.29	68	30.2				340	
16:35	7.46	4.70	37.0%				3.34	68	30.2				340	
16:40	7.33	4.67	36.3%				3.45	68	30.2				340	
16:45	7.52	4.65	38.2%				3.35	68	30.2				340	
16:50	7.43	4.63	37.7%				3.37	68	30.2				340	
16:55	7.47	4.61	38.3%				3.33	68	30.2				340	
17:00	7.39	4.60	37.8%				3.41	68	30.2				340	
17:05	7.43	4.59	38.2%				3.41	68	30.2				340	
17:10	7.49	4.58	38.9%				3.43	68	30.2				340	
17:15	7.33	4.55	37.9%				3.43	68	30.2				340	
17:20	7.82	4.54	41.9%				3.47	68	30.2				340	
17:25	7.51	4.54	39.5%				3.53	68	30.2				340	
17:30	7.61	4.53	40.5%				3.52	68	30.2				340	
17:35	7.49	4.53	39.5%				3.51	68	30.2				340	
17:40	7.41	4.50	39.3%				3.54	68	30.2				340	
17:45	7.61	4.49	41.0%				3.58	68	30.2				340	
17:50	7.54	4.49	40.5%				3.55	68	30.2				340	
17:55	7.47	4.49	39.9%				3.53	68	30.2				340	
18:00	7.51	4.48	40.3%				3.55	68	30.2				340	
18:05	7.52	4.47	40.6%				3.58	68	30.2				340	
18:10	7.62	4.46	41.5%				3.56	68	30.2				340	
18:15	7.60	4.47	41.2%				3.52	68	30.2				340	
18:20	7.59	4.47	41.1%				3.55	68	30.2				340	
18:25	7.63	4.48	41.3%				3.58	67	29.8				335	
18:30	7.44	4.50	39.5%				3.56	68	30.2				340	
18:35	7.69	4.50	41.5%				3.60	68	30.2				340	
18:40	7.71	4.52	41.4%				3.55	68	30.2				340	
18:45	7.71	4.53	41.2%				3.55	67	29.8				335	
18:50	7.49	4.53	39.5%				3.55	68	30.2				340	
18:55	7.69	4.53	41.1%				3.55	68	30.2				340	
19:00	7.64	4.54	40.6%				3.55	68	30.2				340	
19:05	7.54	4.52	40.1%				3.54	67	29.8				335	
19:10	7.47	4.52	39.5%				3.56	68	30.2				340	
19:15	7.68	4.55	40.8%				3.54	67	29.8				335	
19:20	7.46	4.55	39.0%				3.49	67	29.8				335	
19:25	7.92	4.50	43.2%				3.52	67	29.8				335	
19:30	7.66	4.50	41.3%				3.55	68	30.2				340	
19:35	7.74	4.52	41.6%				3.57	68	30.2				340	

Run #2	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading			Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate		Percent	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments	Compression %		gallons/day	gallons/day	
19:40	7.62	4.52	40.7%				3.53	68	30.2				340		
19:45	7.43	4.54	38.9%				3.52	67	29.8				335		
19:50	7.59	4.55	40.1%				3.50	68	30.2				340		
19:55	7.61	4.60	39.6%				3.52	68	30.2				340		
20:00	7.57	4.60	39.2%				3.51	67	29.8				335		
20:05	7.54	4.63	38.6%				3.51	67	29.8				335		
20:10	7.59	4.64	38.9%				3.53	67	29.8				335		
20:15	7.40	4.71	36.4%				3.52	67	29.8				335		
20:20	7.41	4.74	36.0%				3.50	67	29.8				335		
20:25	7.51	4.74	36.9%				3.52	67	29.8				335		
20:30	7.45	4.83	35.2%				3.52	68	30.2				340		
20:35	7.41	4.84	34.7%				3.51	67	29.8				335		
20:40	7.37	4.90	33.5%				3.54	67	29.8				335		
20:45	7.41	4.99	32.7%				3.49	67	29.8				335		
20:50	7.58	5.12	32.5%				3.51	67	29.8				335		
20:55	7.40	5.28	28.6%				3.55	67	29.8				335		
21:00	7.51	5.39	28.2%				3.58	67	29.8				335		
21:05	7.33	5.57	24.0%				3.51	67	29.8				335		
21:10	7.38	5.76	22.0%				3.57	67	29.8				335		
21:15	7.48	6.09	18.6%				3.59	68	30.2				340		
21:20	7.75	6.35	18.1%				3.55	67	29.8				335		
21:25	7.71	6.64	13.9%				3.51	67	29.8				335		
21:30	7.29	6.78	7.0%				3.56	67	29.8				335		
21:35	7.60	7.10	6.6%				3.57	67	29.8				335		
21:40	7.51	7.15	4.8%				3.58	67	29.8				335		
21:45	7.60	7.23	4.9%				3.58	68	30.2				340		
21:50	7.40	7.04	4.9%				3.55	67	29.8				335		
21:55	7.56	7.05	6.7%				3.57	67	29.8				335		
22:00	7.49	7.10	5.2%				3.56	67	29.8				335		
22:05	7.67	7.12	7.2%				3.58	67	29.8				335		
22:10	7.74	7.00	9.6%				3.57	67	29.8				335		
22:15	7.69	7.03	8.6%				3.57	67	29.8				335		
22:20	7.87	7.20	8.5%				3.53	67	29.8				335		
22:25	7.71	7.12	7.7%				3.57	67	29.8				335		
22:30	7.81	7.12	8.8%				3.59	68	30.2				340		
22:35	7.48	7.13	4.7%				3.58	68	30.2				340		
22:40	7.66	7.09	7.4%				3.52	67	29.8				335		
22:45	7.69	7.11	7.5%				3.58	67	29.8				335		
22:50								0		Wash Timer				0	
22:55								29	12.9					145	**Percent**
23:00								29	12.9					145	**of wash**
23:05								29	12.9					145	**%**
Averages	7.53	5.21	30.7%					65.67	29.50				30430	435	1.41%

8-21-18 -8-22-18
Tuesday - Wednesday
30% Compression
Run #3

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
23:10	7.86	7.78	1.0%				3.04	64	28.4	2ml/min SP-218-GH	30	5.0	320	
23:15	8.22	8.76	-6.6%				3.02	68	30.2				340	
23:20	7.18	5.44	24.2%				3.06	68	30.2				340	
23:25	7.00	5.11	27.0%				3.05	68	30.2				340	
23:30	6.91	4.95	28.4%				3.03	69	30.7				345	
23:35	6.93	4.84	30.2%				3.05	69	30.7				345	
23:40	6.90	4.75	31.2%				3.08	69	30.7				345	
23:45	7.03	4.71	33.0%				3.12	69	30.7				345	
23:50	6.88	4.64	32.6%				3.13	68	30.2				340	
23:55	6.93	4.58	33.9%				3.05	68	30.2				340	
0:00	7.19	4.52	37.1%				3.17	68	30.2	8/22/2018			340	
0:05	7.03	4.50	36.0%				3.18	68	30.2				340	
0:10	6.94	4.45	35.9%				3.21	68	30.2				340	
0:15	6.95	4.40	36.7%				3.16	68	30.2				340	
0:20	6.89	4.37	36.6%				3.25	68	30.2				340	
0:25	7.08	4.35	38.6%				3.24	68	30.2				340	
0:30	7.42	4.37	41.1%				3.28	68	30.2				340	
0:35	7.29	4.34	40.5%				3.30	68	30.2				340	
0:40	7.12	4.28	39.9%				3.33	68	30.2				340	
0:45	7.25	4.29	40.8%				3.31	68	30.2				340	
0:50	7.01	4.24	39.5%				3.35	68	30.2				340	
0:55	7.01	4.20	40.1%				3.41	68	30.2				340	
1:00	7.09	4.18	41.0%				3.39	68	30.2				340	
1:05	6.83	4.16	39.1%				3.42	68	30.2				340	
1:10	7.07	4.14	41.4%				3.41	68	30.2				340	
1:15	6.98	4.12	41.0%				3.44	68	30.2				340	
1:20	7.01	4.11	41.4%				3.42	68	30.2				340	
1:25	6.81	4.09	39.9%				3.38	68	30.2				340	
1:30	6.95	4.07	41.4%				3.45	68	30.2				340	
1:35	7.17	4.07	43.2%				3.40	68	30.2				340	
1:40	6.90	4.06	41.2%				3.38	68	30.2				340	
1:45	6.96	4.06	41.7%				3.50	68	30.2				340	
1:50	6.97	4.06	41.8%				3.43	68	30.2				340	
1:55	6.84	4.06	40.6%				3.47	68	30.2				340	
2:00	7.05	4.06	42.4%				3.51	68	30.2				340	
2:05	6.93	4.06	41.4%				3.46	68	30.2				340	
2:10	6.92	4.06	41.3%				3.47	68	30.2				340	
2:15	7.18	4.08	43.2%				3.45	68	30.2				340	
2:20	7.25	4.08	43.7%				3.46	68	30.2				340	
2:25	6.96	4.08	41.4%				3.47	68	30.2				340	
2:30	6.79	4.08	39.9%				3.43	68	30.2				340	
2:35	6.66	4.10	38.4%				3.48	68	30.2				340	
2:40	7.34	4.09	44.3%				3.48	68	30.2				340	
2:45	7.00	4.10	41.4%				3.46	68	30.2				340	
2:50	7.01	4.11	41.4%				3.48	68	30.2				340	
2:55	7.00	4.11	41.3%				3.47	68	30.2				340	
3:00	6.95	4.13	40.6%				3.44	68	30.2				340	
3:05	7.00	4.12	41.1%				3.46	68	30.2				340	
3:10	7.23	4.14	42.7%				3.47	68	30.2				340	
3:15	6.96	4.17	40.1%				3.47	68	30.2				340	
3:20	7.28	4.21	42.2%				3.48	68	30.2				340	
3:25	6.83	4.27	37.5%				3.48	68	30.2				340	

Run #3	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading			Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate			Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments		%		gallons/day	gallons/day	
3:30	6.81	4.28	37.2%				3.43	68	30.2					340		
3:35	7.18	4.30	40.1%				3.48	68	30.2					340		
3:40	7.03	4.34	38.3%				3.47	68	30.2					340		
3:45	7.13	4.34	39.1%				3.46	68	30.2					340		
3:50	7.09	4.34	38.8%				3.47	68	30.2					340		
3:55	7.01	4.40	37.2%				3.46	68	30.2					340		
4:00	7.62	4.58	39.9%				3.46	68	30.2					340		
4:05	7.13	4.85	32.0%				3.46	68	30.2					340		
4:10	6.94	5.06	27.1%				3.48	68	30.2					340		
4:15	7.23	4.93	31.8%				3.47	68	30.2					340		
4:20	6.93	5.24	24.4%				3.46	68	30.2					340		
4:25	6.83	5.48	19.8%				3.47	68	30.2					340		
4:30	6.90	5.67	17.8%				3.53	68	30.2					340		
4:35	7.17	5.88	18.0%				3.47	68	30.2					340		
4:40	7.27	6.09	16.2%				3.51	68	30.2					340		
4:45	6.89	6.18	10.3%				3.51	68	30.2					340		
4:50	6.94	6.32	8.9%				3.48	68	30.2					340		
4:55	6.83	6.38	6.6%				3.48	68	30.2					340		
5:00	7.33	6.52	11.1%				3.48	68	30.2					340		
5:05	7.62	6.56	13.9%				3.49	68	30.2					340		
5:10	7.00	6.17	11.9%				3.46	68	30.2					340		
5:15	6.81	6.27	7.9%				3.39	68	30.2					340		
5:20	7.04	6.18	12.2%				3.48	68	30.2					340		
5:25	7.11	6.10	14.2%				3.48	68	30.2					340		
5:30	7.18	6.08	15.3%				3.49	68	30.2					340		
5:35	7.15	6.10	14.7%				3.40	68	30.2					340		
5:40	6.83	6.02	11.9%				3.49	68	30.2					340		
5:45	6.95	6.05	12.9%				3.45	68	30.2					340		
5:50	7.22	6.22	13.9%				3.44	68	30.2					340		
5:55	7.52	6.67	11.3%				3.45	68	30.2					340		
6:00	6.91	6.16	10.9%				3.51	68	30.2					340		
6:05	7.19	6.18	14.0%				3.47	68	30.2					340		
6:10	7.05	6.40	9.2%				3.52	68	30.2					340		
6:15	7.24	6.32	12.7%				3.50	68	30.2					340		
6:20	6.97	6.28	9.9%				3.50	67	29.8					335		
6:25	8.25	6.28	23.9%				3.46	68	30.2					340		
6:30	7.59	6.45	15.0%				3.44	68	30.2					340		
6:35	7.11	6.45	9.3%				3.46	68	30.2					340		
6:40	6.93	6.25	9.8%				3.46	68	30.2					340		
6:45								29	12.9	Wash Timer					145	
6:50								29	12.9						145	**Percent**
6:55								29	12.9						145	**of wash**
7:00								28	12.4						140	**%**
Averages	7.09	5.01	29.4%					66.34	29.48					30935	575	1.82%

8/22/2018
Wednesday
30% Compression
Run #4

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft²	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
7:05							3.07	69	30.7	2ml/min SP-218-GH	30	5.0	345	
7:10							3.04	69	30.7				345	
7:15							3.04	69	30.7				345	
7:20	6.65	4.63	30.4%				3.04	69	30.7				345	
7:25	6.39	4.50	29.6%				3.02	69	30.7				345	
7:30	6.43	4.45	30.8%				3.08	69	30.7				345	
7:35	6.49	4.40	32.2%				3.12	68	30.2				340	
7:40	6.42	4.31	32.9%				3.13	69	30.7				345	
7:45	6.85	4.26	37.8%				3.14	68	30.2				340	
7:50	6.42	4.22	34.3%				3.15	68	30.2				340	
7:55	6.41	4.20	34.5%				3.21	68	30.2				340	
8:00	6.73	4.18	37.9%				3.24	68	30.2				340	
8:05	6.90	4.25	38.4%				3.24	68	30.2				340	
8:10	6.68	4.14	38.0%				3.27	68	30.2				340	
8:15	6.60	4.12	37.6%				3.28	68	30.2				340	
8:20	6.58	4.12	37.4%	15.67	6.75	56.9%	3.24	68	30.2	TSS Samples			340	
8:25	6.47	4.12	36.3%				3.30	68	30.2				340	
8:30	6.43	4.10	36.2%				3.36	68	30.2				340	
8:35	6.53	4.02	38.4%				3.36	68	30.2				340	
8:40	6.55	4.00	38.9%				3.37	68	30.2				340	
8:45	6.45	3.97	38.4%				3.42	68	30.2				340	
8:50	6.68	3.98	40.4%				3.37	68	30.2				340	
8:55	6.80	3.97	41.6%				3.43	68	30.2				340	
9:00	6.85	3.95	42.3%				3.44	68	30.2				340	
9:05	6.71	3.93	41.4%				3.44	68	30.2				340	
9:10	6.57	3.92	40.3%				3.44	68	30.2				340	
9:15	6.69	3.99	40.4%				3.51	68	30.2				340	
9:20	6.45	3.98	38.3%				3.49	68	30.2				340	
9:25	6.59	3.96	39.9%				3.48	68	30.2				340	
9:30	6.79	3.96	41.7%				3.48	68	30.2				340	
9:35	6.75	3.97	41.2%				3.47	68	30.2				340	
9:40	6.81	3.94	42.1%				3.47	68	30.2				340	
9:45	7.13	3.97	44.3%				3.49	68	30.2				340	
9:50	6.71	3.94	41.3%				3.53	68	30.2				340	
9:55	6.85	3.94	42.5%				3.47	68	30.2				340	
10:00	6.58	3.97	39.7%	15.00	5.00	66.7%	3.46	68	30.2	TSS Samples			340	
10:05	6.76	4.07	39.8%				3.46	68	30.2				340	
10:10	6.81	4.07	40.2%				3.48	68	30.2				340	
10:15	6.75	4.14	38.7%				3.48	68	30.2				340	
10:20	6.86	4.16	39.4%				3.43	68	30.2				340	
10:25	6.82	4.10	39.9%				3.50	68	30.2				340	
10:30	6.97	4.06	41.8%				3.42	68	30.2				340	
10:35	7.39	4.08	44.8%				3.47	68	30.2				340	
10:40	6.89	4.07	40.9%				3.47	68	30.2				340	
10:45	6.73	4.24	37.0%				3.49	68	30.2				340	
10:50	7.11	4.18	41.2%				3.49	68	30.2				340	
10:55	7.11	4.18	41.2%				3.48	68	30.2				340	
11:00	6.92	4.20	39.3%				3.47	68	30.2				340	
11:05	7.31	4.22	42.3%				3.43	68	30.2				340	
11:10	6.90	4.24	38.6%				3.45	68	30.2				340	
11:15	7.06	4.20	40.5%				3.47	68	30.2				340	
11:20	6.81	4.24	37.7%				3.44	68	30.2				340	

Run #4	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading		Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate		Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
11:25	6.87	4.32	37.1%				3.44	68	30.2				340		
11:30	6.90	4.38	36.5%	15.67	6.50	58.5%	3.46	68	30.2	TSS Samples			340		
11:35	7.13	4.42	38.0%				3.38	68	30.2				340		
11:40	6.85	4.50	34.3%				3.45	68	30.2				340		
11:45	7.04	4.56	35.2%				3.44	68	30.2				340		
11:50	7.00	4.69	33.0%				3.45	68	30.2				340		
11:55	7.31	4.89	33.1%				3.46	68	30.2				340		
12:00	7.18	5.00	30.4%				3.44	68	30.2				340		
12:05	7.36	5.09	30.8%				3.45	68	30.2				340		
12:10	7.10	5.32	25.1%				3.46	68	30.2				340		
12:15	7.62	5.53	27.4%				3.47	68	30.2				340		
12:20	7.08	5.81	17.9%				3.40	68	30.2				340		
12:25	6.98	6.10	12.6%				3.47	68	30.2				340		
12:30	7.71	6.29	18.4%				3.44	68	30.2				340		
12:35	7.17	6.29	12.3%				3.47	68	30.2				340		
12:40	6.89	6.47	6.1%				3.37	68	30.2				340		
12:45	6.93	6.56	5.3%				3.45	68	30.2				340		
12:50								0		Mannual Wash				0	
12:55								32	14.2					160	
13:00								32	14.2					160	**Percent**
13:05								32	14.2					160	**of wash**
13:10								38	16.9					190	**%**
Averages	6.84	4.42	35.3%	15.45	6.08	60.6%		65.31	29.42				23495	670	2.77%

8/22/2018
Wednesday
40% Compression
Run #5

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
13:15							3.16	69	30.7	2ml/min SP-218-GH	40	5.1	345	
13:20	7.94	5.04	36.5%				3.19	68	30.2				340	
13:25	7.28	4.68	35.7%				3.14	69	30.7				345	
13:30	7.15	4.50	37.1%				3.17	68	30.2				340	
13:35	7.01	4.37	37.7%				3.29	68	30.2				340	
13:40	7.02	4.32	38.5%				3.25	69	30.7				345	
13:45	6.97	4.27	38.7%				3.30	68	30.2				340	
13:50	6.96	4.23	39.2%				3.42	68	30.2				340	
13:55	7.06	4.21	40.4%				3.41	68	30.2				340	
14:00	7.01	4.20	40.1%				3.44	68	30.2				340	
14:05	7.04	4.18	40.6%				3.42	68	30.2				340	
14:10	6.98	4.17	40.3%				3.54	68	30.2				340	
14:15	7.03	4.33	38.4%	15.00	6.50	56.7%	3.58	68	30.2	TSS Samples			340	
14:20	6.93	4.32	37.7%				3.60	68	30.2				340	
14:25	7.01	4.32	38.4%				3.77	68	30.2				340	
14:30	7.01	4.30	38.7%				3.68	68	30.2				340	
14:35	6.92	4.34	37.3%				3.73	68	30.2				340	
14:40	6.99	4.33	38.1%				3.64	68	30.2				340	
14:45	6.96	4.28	38.5%				3.74	68	30.2				340	
14:50	7.11	4.18	41.2%				3.79	68	30.2				340	
14:55	6.85	4.14	39.6%				3.87	68	30.2				340	
15:00	7.09	4.14	41.6%				3.80	68	30.2				340	
15:05	7.08	4.16	41.2%				3.83	68	30.2				340	
15:10	6.93	4.14	40.3%				3.91	67	29.8				335	
15:15	6.95	4.14	40.4%	16.33	6.75	58.7%	3.86	68	30.2	TSS Samples			340	
15:20	6.97	4.16	40.3%				3.92	68	30.2				340	
15:25	6.95	4.16	40.1%				3.86	68	30.2				340	
15:30	7.03	4.17	40.7%				3.91	68	30.2				340	
15:35	6.94	4.14	40.3%				3.96	67	29.8				335	
15:40	6.91	4.17	39.7%				3.80	68	30.2				340	
15:45	7.38	4.22	42.8%				3.90	68	30.2				340	
15:50	6.89	4.60	33.2%				3.92	67	29.8				335	
15:55	6.83	4.52	33.8%				3.87	67	29.8				335	
16:00	6.94	4.40	36.6%				3.87	68	30.2				340	
16:05	6.79	4.22	37.8%				3.92	67	29.8				335	
16:10	6.83	4.19	38.7%				3.85	68	30.2				340	
16:15	7.08	4.24	40.1%				3.83	68	30.2				340	
16:20	6.91	4.24	38.6%				3.85	67	29.8				335	
16:25	7.00	4.25	39.3%				3.84	67	29.8				335	
16:30	7.16	4.30	39.9%				3.76	68	30.2				340	
16:35	7.19	4.52	37.1%				3.90	67	29.8				335	
16:40	7.28	4.48	38.5%				3.84	68	30.2				340	
16:45	7.13	4.40	38.3%				3.94	67	29.8				335	
16:50	7.28	4.39	39.7%				3.81	67	29.8				335	
16:55	7.08	4.44	37.3%				3.82	68	30.2				340	
17:00	7.31	4.48	38.7%				3.79	68	30.2				340	
17:05	7.28	4.51	38.0%				3.89	68	30.2				340	
17:10	7.31	4.52	38.2%				3.86	68	30.2				340	
17:15	7.31	4.56	37.6%				3.86	68	30.2				340	
17:20	7.50	4.58	38.9%				3.86	67	29.8				335	
17:25	7.23	4.58	36.7%				3.89	67	29.8				335	
17:30	7.41	4.63	37.5%				3.88	67	29.8				335	

Run #5	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading		Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate		Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
17:35	7.21	4.64	35.6%				3.88	67	29.8				335		
17:40	7.25	4.63	36.1%				3.90	67	29.8				335		
17:45	7.42	4.65	37.3%				3.88	67	29.8				335		
17:50	7.23	4.79	33.7%				3.88	67	29.8				335		
17:55	7.29	4.91	32.6%				3.88	67	29.8				335		
18:00	7.45	5.03	32.5%				3.90	67	29.8				335		
18:05	7.52	5.11	32.0%				3.90	67	29.8				335		
18:10	7.58	5.23	31.0%				3.84	67	29.8				335		
18:15	7.46	5.49	26.4%				3.88	67	29.8				335		
18:20	7.70	5.78	24.9%				3.88	67	29.8				335		
18:25	7.46	6.11	18.1%				3.87	67	29.8				335		
18:30	7.34	6.45	12.1%				3.88	67	29.8				335		
18:35	7.26	6.79	6.5%				3.84	67	29.8				335		
18:40	7.29	6.94	4.8%				3.83	67	29.8				335		
18:45								30	13.3	Wash Timer				150	
18:50								31	13.8					155	**Percent**
18:55								31	13.8					155	**of wash**
19:00								31	13.8					155	**%**
Averages	7.15	4.58	35.9%	15.67	6.63	57.7%		65.53	29.12				22320	615	2.68%

8/22/2018
Wednesday
40% Compression
Run #6

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
19:05	8.47	6.22	26.6%				3.14	68	30.2	2ml/min SP-218-GH	40	5.1	340	
19:10	7.00	5.30	24.3%				3.19	68	30.2				340	
19:15	6.76	4.93	27.1%				3.19	68	30.2				340	
19:20	6.75	4.83	28.4%				3.22	68	30.2				340	
19:25	6.79	4.74	30.2%				3.31	68	30.2				340	
19:30	6.78	4.64	31.6%				3.28	68	30.2				340	
19:35	6.75	4.58	32.1%				3.43	68	30.2				340	
19:40	6.75	4.54	32.7%				3.28	68	30.2				340	
19:45	6.71	4.50	32.9%				3.40	68	30.2				340	
19:50	6.91	4.48	35.2%				3.53	68	30.2				340	
19:55	6.67	4.52	32.2%				3.54	68	30.2				340	
20:00	6.68	4.42	33.8%				3.51	68	30.2				340	
20:05	6.84	4.40	35.7%				3.55	68	30.2				340	
20:10	6.75	4.38	35.1%				3.57	68	30.2				340	
20:15	6.93	4.34	37.4%				3.60	68	30.2				340	
20:20	6.79	4.32	36.4%				3.57	68	30.2				340	
20:25	6.76	4.24	37.3%				3.68	68	30.2				340	
20:30	6.83	4.24	37.9%				3.72	67	29.8				335	
20:35	6.90	4.24	38.6%				3.79	68	30.2				340	
20:40	6.85	4.22	38.4%				3.73	67	29.8				335	
20:45	6.70	4.20	37.3%				3.75	67	29.8				335	
20:50	6.83	4.18	38.8%				3.74	68	30.2				340	
20:55	6.89	4.18	39.3%				3.75	67	29.8				335	
21:00	6.90	4.17	39.6%				3.82	67	29.8				335	
21:05	6.96	4.17	40.1%				3.78	67	29.8				335	
21:10	6.97	4.19	39.9%				3.73	67	29.8				335	
21:15	6.91	4.20	39.2%				3.77	67	29.8				335	
21:20	7.05	4.20	40.4%				3.83	68	30.2				340	
21:25	7.00	4.20	40.0%				3.80	68	30.2				340	
21:30	6.90	4.20	39.1%				3.89	67	29.8				335	
21:35	7.09	4.21	40.6%				3.82	67	29.8				335	
21:40	6.98	4.22	39.5%				3.79	67	29.8				335	
21:45	6.89	4.23	38.6%				3.90	67	29.8				335	
21:50	6.96	4.22	39.4%				3.80	67	29.8				335	
21:55	7.00	4.22	39.7%				3.79	67	29.8				335	
22:00	6.90	4.20	39.1%				3.82	67	29.8				335	
22:05	7.01	4.21	39.9%				3.81	67	29.8				335	
22:10	7.15	4.22	41.0%				3.79	67	29.8				335	
22:15	6.82	4.22	38.1%				3.75	68	30.2				340	
22:20	6.75	4.20	37.8%				3.84	67	29.8				335	
22:25	6.73	4.22	37.3%				3.76	67	29.8				335	
22:30	6.89	4.24	38.5%				3.77	67	29.8				335	
22:35	6.79	4.26	37.3%				3.76	68	30.2				340	
22:40	6.76	4.27	36.8%				3.81	68	30.2				340	
22:45	6.65	4.27	35.8%				3.77	68	30.2				340	
22:50	6.79	4.30	36.7%				3.76	68	30.2				340	
22:55	6.71	4.33	35.5%				3.78	67	29.8				335	
23:00	6.77	4.41	34.9%				3.78	67	29.8				335	
23:05	6.68	4.48	32.9%				3.77	68	30.2				340	
23:10	6.82	4.50	34.0%				3.80	67	29.8				335	
23:15	6.71	4.63	31.0%				3.81	68	30.2				340	
23:20	6.77	4.78	29.4%				3.84	67	29.8				335	

Run #6	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate			Wash Trip Point	Filter run	wash	
Time	**NTU**	**NTU**	**%**	**mg/l**	**mg/l**	**%**	**Pressure psi**	**Flow gpm**	**gpm/ft^2**	**Comments**	**Percent Compression %**		**gallons/day**	**gallons/day**	
23:25	6.77	4.86	28.2%				3.81	67	29.8				335		
23:30	6.85	4.99	27.2%				3.84	67	29.8				335		
23:35	6.98	5.27	24.5%				3.84	68	30.2				340		
23:40	6.91	5.53	20.0%				3.89	67	29.8				335		
23:45	7.00	5.75	17.9%				3.87	67	29.8				335		
23:50	6.67	5.88	11.8%				3.81	67	29.8				335		
23:55	6.66	6.07	8.9%				3.83	67	29.8				335		
0:00	6.97	6.24	10.5%				3.82	68	30.2	8/23/2018			340		
0:05	6.83	6.50	4.8%				3.84	67	29.8				335		
0:10	7.03	6.60	6.1%				3.84	67	29.8				335		
0:15	6.84	6.72	1.8%				3.82	67	29.8				335		
0:20	6.61	6.51	1.5%				3.82	68	30.2				340		
0:25	7.13	7.01	1.7%				3.85	68	30.2				340		
0:30	7.25	7.15	1.4%				3.91	67	29.8				335		
0:35								0		Wash Timer				0	
0:40								30	13.3					150	**Percent**
0:45								30	13.3					150	**of wash**
0:50								30	13.3					150	**%**
Averages	6.88	4.76	30.8%					64.91	29.27				22270	450	1.98%

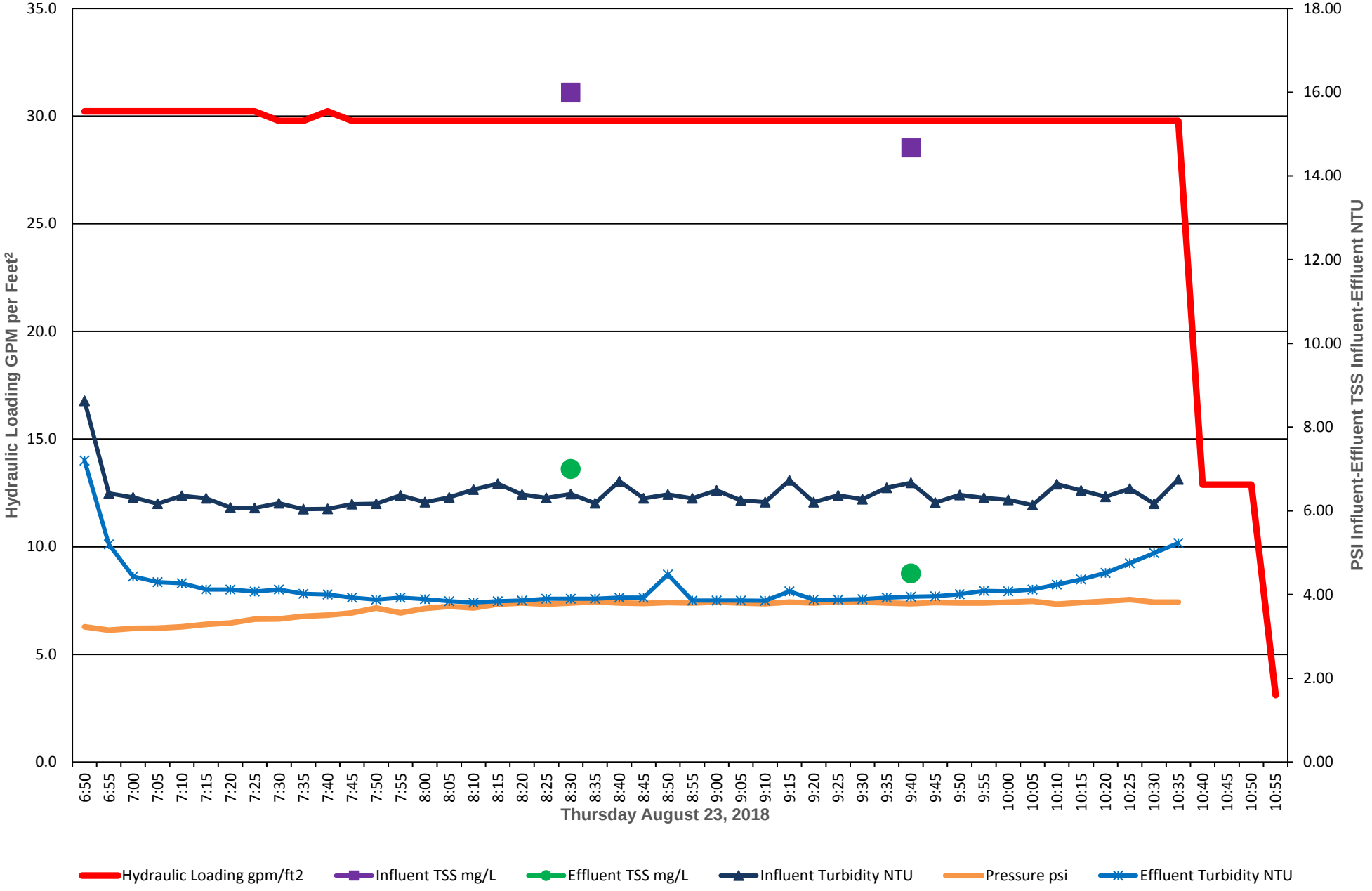
8/23/2018
Thursday
40% Compression
Run #7

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
0:55							3.17	68	30.2	2ml/min SP-218-GH	40	5.1	340	
1:00							3.14	68	30.2				340	
1:05	6.35	4.75	25.2%				3.10	69	30.7				345	
1:10	6.20	4.50	27.4%				3.20	68	30.2				340	
1:15	6.30	4.44	29.5%				3.20	68	30.2				340	
1:20	6.22	4.38	29.6%				3.22	68	30.2				340	
1:25	6.20	4.29	30.8%				3.37	68	30.2				340	
1:30	6.17	4.20	31.9%				3.41	68	30.2				340	
1:35	6.11	4.16	31.9%				3.42	68	30.2				340	
1:40	6.27	4.11	34.4%				3.43	68	30.2				340	
1:45	6.19	4.08	34.1%				3.48	68	30.2				340	
1:50	6.22	4.05	34.9%				3.49	68	30.2				340	
1:55	6.37	4.03	36.7%				3.54	68	30.2				340	
2:00	6.27	4.01	36.0%				3.62	68	30.2				340	
2:05	6.11	3.96	35.2%				3.65	68	30.2				340	
2:10	6.25	3.96	36.6%				3.67	67	29.8				335	
2:15	6.35	3.95	37.8%				3.71	67	29.8				335	
2:20	6.18	3.94	36.2%				3.80	67	29.8				335	
2:25	6.19	3.91	36.8%				3.76	67	29.8				335	
2:30	6.19	3.89	37.2%				3.71	67	29.8				335	
2:35	6.53	3.90	40.3%				3.82	68	30.2				340	
2:40	6.23	3.98	36.1%				3.81	68	30.2				340	
2:45	6.51	3.95	39.3%				3.82	68	30.2				340	
2:50	6.46	3.90	39.6%				3.81	67	29.8				335	
2:55	6.29	3.88	38.3%				3.93	67	29.8				335	
3:00	6.15	3.86	37.2%				3.86	67	29.8				335	
3:05	6.25	3.86	38.2%				3.91	67	29.8				335	
3:10	6.13	3.86	37.0%				3.88	67	29.8				335	
3:15	6.28	3.88	38.2%				3.85	67	29.8				335	
3:20	6.19	3.92	36.7%				3.83	68	30.2				340	
3:25	6.18	3.90	36.9%				3.80	67	29.8				335	
3:30	6.47	3.90	39.7%				3.68	67	29.8				335	
3:35	6.40	3.93	38.6%				3.83	67	29.8				335	
3:40	6.57	3.96	39.7%				3.84	67	29.8				335	
3:45	6.41	4.00	37.6%				3.83	67	29.8				335	
3:50	6.39	4.00	37.4%				3.79	67	29.8				335	
3:55	6.39	4.04	36.8%				3.72	67	29.8				335	
4:00	6.38	4.14	35.1%				3.83	67	29.8				335	
4:05	6.51	4.17	35.9%				3.78	67	29.8				335	
4:10	6.81	4.42	35.1%				3.79	68	30.2				340	
4:15	6.35	4.54	28.5%				3.76	67	29.8				335	
4:20	6.28	4.73	24.7%				3.68	67	29.8				335	
4:25	6.47	4.94	23.6%				3.77	67	29.8				335	
4:30	6.29	5.12	18.6%				3.79	67	29.8				335	
4:35	6.42	5.30	17.4%				3.80	67	29.8				335	
4:40	6.67	5.55	16.8%				3.80	67	29.8				335	
4:45	6.63	5.74	13.4%				3.72	67	29.8				335	
4:50	6.66	6.06	9.0%				3.80	67	29.8				335	
4:55	6.55	6.20	5.3%				3.83	67	29.8				335	
5:00	6.43	6.27	2.5%				3.85	67	29.8				335	
5:05	6.51	6.36	2.3%				3.76	67	29.8				335	
5:10	6.46	6.34	1.9%				3.79	67	29.8				335	

Run #7	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	**NTU**	**NTU**	**%**	**mg/l**	**mg/l**	**%**	**Pressure psi**	**Flow gpm**	**gpm/ft^2**	**Comments**	**%**		**gallons/day**	**gallons/day**	
5:15	6.51	6.32	2.9%				3.78	67	29.8				335		
5:20	7.14	6.33	11.3%				3.81	67	29.8				335		
5:25	6.34	6.08	4.1%				3.82	67	29.8				335		
5:30	6.71	6.14	8.5%				3.84	67	29.8				335		
5:35	6.58	6.24	5.2%				3.79	67	29.8				335		
5:40	6.28	6.02	4.1%				3.80	67	29.8				335		
5:45	6.17	6.06	1.8%				3.81	67	29.8				335		
5:50	6.24	6.01	3.7%				3.86	67	29.8				335		
5:55	6.21	6.08	2.1%				3.78	67	29.8				335		
6:00	6.14	6.05	1.5%				3.73	67	29.8				335		
6:05	6.17	6.06	1.8%				3.81	67	29.8				335		
6:10	6.15	6.00	2.4%				3.80	67	29.8				335		
6:15	6.68	6.09	8.8%				3.79	67	29.8				335		
6:20	6.61	6.00	9.2%				3.78	67	29.8				335		
6:25	6.43	6.02	6.4%				3.79	67	29.8				335		
6:30								31	13.8	Wash Timer				155	
6:35								31	13.8					155	**Percent**
6:40								31	13.8					155	**of wash**
6:45								31	13.8					155	**%**
Averages	6.37	4.81	24.4%					65.27	29.01				22550	620	2.68%

SCHREIBER FuzzyFilter

Snohomish, WA Filter Test
40% Compression



8/23/2018
Thursday
40% Compression
Run #8

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
6:50	8.63	7.20	16.6%				3.23	68	30.2	2ml/min SP-218-GH	40	5.1	340		
6:55	6.42	5.20	19.0%				3.15	68	30.2				340		
7:00	6.32	4.43	29.9%				3.19	68	30.2				340		
7:05	6.17	4.30	30.3%				3.20	68	30.2				340		
7:10	6.36	4.27	32.9%				3.23	68	30.2				340		
7:15	6.30	4.12	34.6%				3.29	68	30.2				340		
7:20	6.08	4.12	32.2%				3.32	68	30.2				340		
7:25	6.07	4.07	32.9%				3.41	68	30.2				340		
7:30	6.18	4.12	33.3%				3.42	67	29.8				335		
7:35	6.04	4.02	33.4%				3.48	67	29.8				335		
7:40	6.05	4.00	33.9%				3.51	68	30.2				340		
7:45	6.16	3.93	36.2%				3.56	67	29.8				335		
7:50	6.17	3.88	37.1%				3.68	67	29.8				335		
7:55	6.37	3.93	38.3%				3.56	67	29.8				335		
8:00	6.21	3.89	37.4%				3.67	67	29.8				335		
8:05	6.32	3.84	39.2%				3.72	67	29.8				335		
8:10	6.51	3.81	41.5%				3.68	67	29.8				335		
8:15	6.65	3.84	42.3%				3.77	67	29.8				335		
8:20	6.39	3.86	39.6%				3.80	67	29.8				335		
8:25	6.31	3.90	38.2%				3.77	67	29.8				335		
8:30	6.40	3.90	39.1%	16.00	7.00	56.3%	3.80	67	29.8	TSS Samples			335		
8:35	6.18	3.90	36.9%				3.83	67	29.8				335		
8:40	6.71	3.93	41.4%				3.80	67	29.8				335		
8:45	6.30	3.93	37.6%				3.79	67	29.8				335		
8:50	6.39	4.48	29.9%				3.81	67	29.8				335		
8:55	6.30	3.86	38.7%				3.80	67	29.8				335		
9:00	6.49	3.86	40.5%				3.82	67	29.8				335		
9:05	6.25	3.86	38.2%				3.80	67	29.8				335		
9:10	6.21	3.85	38.0%				3.78	67	29.8				335		
9:15	6.73	4.08	39.4%				3.82	67	29.8				335		
9:20	6.21	3.88	37.5%				3.80	67	29.8				335		
9:25	6.37	3.88	39.1%				3.83	67	29.8				335		
9:30	6.28	3.89	38.1%				3.82	67	29.8				335		
9:35	6.55	3.93	40.0%				3.80	67	29.8				335		
9:40	6.67	3.95	40.8%	14.67	4.50	69.3%	3.78	67	29.8	TSS Samples			335		
9:45	6.20	3.96	36.1%				3.81	67	29.8				335		
9:50	6.38	4.01	37.1%				3.80	67	29.8				335		
9:55	6.31	4.09	35.2%				3.80	67	29.8				335		
10:00	6.26	4.08	34.8%				3.82	67	29.8				335		
10:05	6.14	4.12	32.9%				3.84	67	29.8				335		
10:10	6.64	4.24	36.1%				3.77	67	29.8				335		
10:15	6.49	4.36	32.8%				3.81	67	29.8				335		
10:20	6.34	4.52	28.7%				3.84	67	29.8				335		
10:25	6.53	4.75	27.3%				3.88	67	29.8				335		
10:30	6.17	4.99	19.1%				3.82	67	29.8				335		
10:35	6.75	5.23	22.5%				3.82	67	29.8				335		
10:40								29	12.9	Manual Wash				145	
10:45								29	12.9					145	**Percent**
10:50								29	12.9					145	**of wash**
10:55								7	3.1	Added Media				35	**%**
Averages	6.39	4.18	34.6%	15.34	5.75	62.5%		63.70	28.31				15455	470	2.95%

8/23/2018
Thursday
40% Compression
Run #9

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
11:20							1.66	77	34.2	Added Media	40	5.2	385	
11:25							3.35	68	30.2	2ml/min SP-218-GH			340	
11:30	8.37	4.53	45.9%				3.33	68	30.2				340	
11:35	6.46	4.10	36.5%				3.34	68	30.2				340	
11:40	6.41	3.99	37.8%				3.39	68	30.2				340	
11:45	6.47	3.92	39.4%				3.52	68	30.2				340	
11:50	6.47	3.87	40.2%				3.69	68	30.2				340	
11:55	6.47	3.82	41.0%				3.70	68	30.2				340	
12:00	6.54	3.79	42.0%				3.73	68	30.2				340	
12:05	6.40	3.76	41.3%				3.85	68	30.2				340	
12:10	6.57	3.76	42.8%				3.98	68	30.2				340	
12:15	6.25	3.75	40.0%				4.03	67	29.8				335	
12:20	6.24	3.73	40.2%				4.08	67	29.8				335	
12:25	6.35	3.71	41.6%				4.12	67	29.8				335	
12:30	6.42	3.70	42.4%	14.00	6.75	51.8%	4.17	65	28.9	TSS Samples			325	
12:35	6.42	3.70	42.4%				4.29	67	29.8				335	
12:40	6.47	3.71	42.7%				4.48	67	29.8				335	
12:45	6.53	3.70	43.3%				4.40	67	29.8				335	
12:50	6.35	3.70	41.7%				4.42	66	29.3				330	
12:55	6.47	3.70	42.8%				4.30	57	25.3				285	
13:00	6.07	3.65	39.9%				4.39	56	24.9				280	
13:05	5.97	3.65	38.9%				4.35	56	24.9				280	
13:10	6.10	3.66	40.0%				4.35	57	25.3				285	
13:15	6.28	3.68	41.4%				4.38	56	24.9				280	
13:20	6.37	3.71	41.8%				4.35	56	24.9				280	
13:25	6.51	3.71	43.0%				4.32	58	25.8				290	
13:30	6.69	3.75	43.9%	13.00	6.75	48.1%	4.36	59	26.2	TSS Samples			295	
13:35	6.66	3.75	43.7%				4.36	59	26.2				295	
13:40	6.23	3.74	40.0%				4.35	58	25.8				290	
13:45	6.28	3.73	40.6%				4.34	58	25.8				290	
13:50	6.42	3.75	41.6%				4.37	59	26.2				295	
13:55	6.42	3.78	41.1%				4.38	60	26.7				300	
14:00	6.48	3.78	41.7%				4.40	60	26.7				300	
14:05	6.37	3.79	40.5%				4.54	61	27.1				305	
14:10	6.48	3.80	41.4%				4.37	61	27.1				305	
14:15	6.48	3.81	41.2%				4.42	61	27.1				305	
14:20	6.45	3.81	40.9%				4.46	61	27.1				305	
14:25	6.37	3.81	40.2%				4.48	61	27.1				305	
14:30	6.53	3.81	41.7%				4.46	61	27.1				305	
14:35	6.30	3.91	37.9%				4.48	61	27.1				305	
14:40	6.58	3.84	41.6%	15.00	6.25	58.3%	4.51	61	27.1	TSS Samples			305	
14:45	6.45	3.84	40.5%				4.53	61	27.1				305	
14:50	6.42	3.84	40.2%				4.44	61	27.1				305	
14:55	6.50	3.85	40.8%				4.43	61	27.1				305	
15:00	6.40	3.88	39.4%				4.66	60	26.7				300	
15:05	6.53	3.89	40.4%				4.59	61	27.1				305	
15:10	6.39	3.92	38.7%				4.63	60	26.7				300	
15:15	6.73	3.88	42.3%				4.47	62	27.6				310	
15:20	6.73	3.92	41.8%				4.74	62	27.6				310	
15:25	6.57	3.95	39.9%				4.62	62	27.6				310	
15:30	6.39	3.94	38.3%				4.78	62	27.6				310	
15:35	6.59	3.96	39.9%				4.63	62	27.6				310	

Run #9	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate			Percent Compression	Wash Trip Point	Filter run	wash	
Time	**NTU**	**NTU**	**%**	**mg/l**	**mg/l**	**%**	**psi**	**gpm**	**gpm/ft^2**	**Comments**		**%**		**gallons/day**	**gallons/day**	
15:40	6.43	4.01	37.6%				4.68	62	27.6					310		
15:45	6.51	4.07	37.5%				4.76	62	27.6					310		
15:50	6.45	4.09	36.6%				4.72	62	27.6					310		
15:55	6.41	4.13	35.6%				4.74	62	27.6					310		
16:00	6.42	4.20	34.6%				4.77	62	27.6					310		
16:05	6.45	4.35	32.6%				4.84	62	27.6					310		
16:10	6.37	4.52	29.0%				4.85	62	27.6					310		
16:15	6.30	4.66	26.0%				4.94	62	27.6					310		
16:20	6.47	4.87	24.7%				4.89	62	27.6					310		
16:25	6.55	5.15	21.4%				4.97	62	27.6					310		
16:30	6.56	5.42	17.4%				5.01	62	27.6					310		
16:35	6.56	5.72	12.8%				5.09	62	27.6					310		
16:40	6.33	5.93	6.3%				5.08	62	27.6					310		
16:45	6.30	6.15	2.4%				5.13	61	27.1					305		
16:50								0		Wash Timer					0	
16:55								29	12.9						145	
17:00								29	12.9						145	**Percent**
17:05								29	12.9						145	**of wash**
17:10								29	12.9						145	**%**
Averages	6.46	4.03	37.6%	14.00	6.58	53.0%		59.77	26.95					20640	580	2.73%

8/23/2018
Thursday
40% Compression
Run #10

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
17:15	7.76	6.10	21.4%				3.60	64	28.4	2ml/min SP-218-GH	40	5.2	320	
17:20	6.25	4.79	23.4%				3.59	64	28.4				320	
17:25	5.83	4.29	26.4%				3.46	64	28.4				320	
17:30	5.85	4.14	29.2%				3.56	64	28.4				320	
17:35	5.95	4.08	31.4%				3.62	64	28.4				320	
17:40	6.03	4.06	32.7%				3.79	65	28.9				325	
17:45	5.92	4.02	32.1%				3.77	64	28.4				320	
17:50	5.92	4.00	32.4%				4.00	64	28.4				320	
17:55	5.95	3.97	33.3%				4.03	64	28.4				320	
18:00	5.79	3.96	31.6%				4.06	64	28.4				320	
18:05	6.07	3.94	35.1%				4.20	64	28.4				320	
18:10	6.03	3.93	34.8%				4.26	64	28.4				320	
18:15	5.92	3.93	33.6%				4.24	64	28.4				320	
18:20	5.88	3.91	33.5%				4.33	64	28.4				320	
18:25	6.16	3.90	36.7%				4.37	64	28.4				320	
18:30	6.09	3.89	36.1%				4.40	64	28.4				320	
18:35	6.01	3.87	35.6%				4.53	64	28.4				320	
18:40	5.94	3.86	35.0%				4.56	64	28.4				320	
18:45	5.91	3.85	34.9%				4.59	63	28.0				315	
18:50	6.16	3.86	37.3%				4.66	64	28.4				320	
18:55	5.97	3.83	35.8%				4.67	64	28.4				320	
19:00	6.12	3.88	36.6%				4.66	63	28.0				315	
19:05	6.16	3.89	36.9%				4.57	63	28.0				315	
19:10	6.15	3.90	36.6%				4.43	63	28.0				315	
19:15	6.22	3.89	37.5%				4.64	63	28.0				315	
19:20	6.25	3.89	37.8%				4.66	63	28.0				315	
19:25	6.05	3.91	35.4%				4.53	63	28.0				315	
19:30	6.15	3.91	36.4%				4.55	63	28.0				315	
19:35	6.10	3.91	35.9%				4.63	63	28.0				315	
19:40	6.18	3.91	36.7%				4.60	63	28.0				315	
19:45	6.37	3.94	38.1%				4.60	63	28.0				315	
19:50	6.56	3.94	39.9%				4.63	63	28.0				315	
19:55	6.28	3.94	37.3%				4.52	63	28.0				315	
20:00	6.55	3.91	40.3%				4.63	64	28.4				320	
20:05	6.42	3.93	38.8%				4.53	63	28.0				315	
20:10	6.49	3.94	39.3%				4.51	63	28.0				315	
20:15	6.32	3.96	37.3%				4.70	63	28.0				315	
20:20	6.33	3.96	37.4%				4.66	63	28.0				315	
20:25	6.32	3.94	37.7%				4.64	63	28.0				315	
20:30	6.19	3.95	36.2%				4.74	63	28.0				315	
20:35	6.13	3.96	35.4%				4.63	63	28.0				315	
20:40	6.22	3.96	36.3%				4.64	63	28.0				315	
20:45	6.37	4.00	37.2%				4.72	63	28.0				315	
20:50	6.22	4.06	34.7%				4.70	63	28.0				315	
20:55	6.13	4.06	33.8%				4.64	63	28.0				315	
21:00	6.32	4.14	34.5%				4.64	63	28.0				315	
21:05	6.39	4.25	33.5%				4.71	63	28.0				315	
21:10	6.48	4.42	31.8%				4.71	63	28.0				315	
21:15	6.29	4.50	28.5%				4.76	63	28.0				315	
21:20	6.45	4.76	26.2%				4.76	63	28.0				315	
21:25	6.23	5.03	19.3%				4.74	63	28.0				315	
21:30	6.52	5.36	17.8%				4.76	63	28.0				315	

Run #10	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate			Percent Compression	Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments		%		gallons/day	gallons/day	
21:35	6.30	5.72	9.2%				4.79	63	28.0					315		
21:40	6.36	6.13	3.6%				4.91	63	28.0					315		
21:45	6.43	6.02	6.4%				4.72	63	28.0					315		
21:50	6.27	6.08	3.0%				4.88	63	28.0					315		
21:55	6.18	6.03	2.4%				4.80	63	28.0					315		
22:00	6.34	6.06	4.4%				4.80	63	28.0					315		
22:05	6.34	6.02	5.0%				4.78	63	28.0					315		
22:10	6.32	6.09	3.6%				4.82	63	28.0					315		
22:15	6.51	6.08	6.6%				4.76	63	28.0					315		
22:20	6.33	6.02	4.9%				4.83	63	28.0					315		
22:25	6.40	6.07	5.2%				4.81	63	28.0					315		
22:30	6.36	6.01	5.5%				4.85	63	28.0					315		
22:35	6.36	6.09	4.2%				4.80	64	28.4					320		
22:40	6.21	6.04	2.7%				4.78	63	28.0					315		
22:45								0		Wash Timer					0	
22:50								32	14.2						160	
22:55								32	14.2						160	**Percent**
23:00								32	14.2						160	**of wash**
23:05								32	14.2						160	**%**
Averages	6.23	4.51	27.6%					60.69	27.36					20905	640	2.97%

8/23/2018 - 8-24-18
Thursday - Friday
40% Compression
Run #11

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
23:10							3.46	65	28.9	2ml/min SP-218-GH	40	5.2	325	
23:15							3.42	65	28.9				325	
23:20	5.81	4.19	27.9%				3.51	65	28.9				325	
23:25	5.74	4.06	29.3%				3.46	65	28.9				325	
23:30	5.77	4.00	30.7%				3.70	65	28.9				325	
23:35	5.77	3.94	31.7%				3.67	65	28.9				325	
23:40	5.84	4.02	31.2%				3.88	65	28.9				325	
23:45	5.85	3.99	31.8%				3.91	65	28.9				325	
23:50	5.85	3.91	33.2%				3.96	65	28.9				325	
23:55	5.90	3.88	34.2%				4.14	64	28.4				320	
0:00	5.87	3.84	34.6%				4.12	64	28.4	8/24/2018			320	
0:05	5.81	3.82	34.3%				4.19	64	28.4				320	
0:10	5.89	3.80	35.5%				4.32	64	28.4				320	
0:15	6.06	3.77	37.8%				4.40	64	28.4				320	
0:20	5.92	3.79	36.0%				4.35	64	28.4				320	
0:25	5.89	3.76	36.2%				4.52	64	28.4				320	
0:30	5.99	3.75	37.4%				4.47	64	28.4				320	
0:35	5.95	3.74	37.1%				4.47	64	28.4				320	
0:40	6.13	3.76	38.7%				4.50	64	28.4				320	
0:45	6.13	3.76	38.7%				4.52	64	28.4				320	
0:50	6.09	3.76	38.3%				4.51	64	28.4				320	
0:55	5.93	3.76	36.6%				4.52	64	28.4				320	
1:00	5.92	3.78	36.1%				4.47	64	28.4				320	
1:05	6.17	3.80	38.4%				4.46	64	28.4				320	
1:10	5.92	3.82	35.5%				4.58	64	28.4				320	
1:15	6.05	3.82	36.9%				4.47	63	28.0				315	
1:20	6.21	3.84	38.2%				4.48	64	28.4				320	
1:25	6.02	3.84	36.2%				4.65	64	28.4				320	
1:30	5.96	3.84	35.6%				4.60	64	28.4				320	
1:35	6.22	3.84	38.3%				4.47	64	28.4				320	
1:40	6.02	3.86	35.9%				4.49	64	28.4				320	
1:45	6.25	3.86	38.2%				4.57	64	28.4				320	
1:50	5.98	3.87	35.3%				4.51	63	28.0				315	
1:55	6.23	3.88	37.7%				4.68	64	28.4				320	
2:00	6.09	3.90	36.0%				4.59	64	28.4				320	
2:05	5.99	3.96	33.9%				4.62	63	28.0				315	
2:10	6.18	3.97	35.8%				4.64	64	28.4				320	
2:15	6.23	3.97	36.3%				4.56	64	28.4				320	
2:20	6.30	4.02	36.2%				4.64	63	28.0				315	
2:25	5.99	4.08	31.9%				4.72	63	28.0				315	
2:30	6.15	4.11	33.2%				4.77	64	28.4				320	
2:35	6.19	4.13	33.3%				4.60	64	28.4				320	
2:40	6.15	4.20	31.7%				4.72	63	28.0				315	
2:45	5.91	4.29	27.4%				4.66	64	28.4				320	
2:50	6.55	4.47	31.8%				4.75	63	28.0				315	
2:55	6.70	4.83	27.9%				4.65	63	28.0				315	
3:00	6.23	5.16	17.2%				4.69	63	28.0				315	
3:05	6.50	5.55	14.6%				4.72	63	28.0				315	
3:10	6.27	5.96	4.9%				4.70	63	28.0				315	
3:15	6.12	5.02	18.0%				4.68	63	28.0				315	
3:20	6.11	5.50	10.0%				4.67	63	28.0				315	
3:25	6.29	5.06	19.6%				4.67	63	28.0				315	

Run #11	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	**NTU**	**NTU**	**%**	**mg/l**	**mg/l**	**%**	**Pressure psi**	**Flow gpm**	**gpm/ft^2**	**Comments**	**%**		**gallons/day**	**gallons/day**	
3:30	6.22	6.01	3.4%				4.76	63	28.0				315		
3:35	6.13	5.90	3.8%				4.80	63	28.0				315		
3:40	6.18	6.04	2.3%				4.76	63	28.0				315		
3:45	6.13	5.11	16.6%				4.67	63	28.0				315		
3:50	6.28	5.19	17.4%				4.78	63	28.0				315		
3:55	6.97	5.32	23.7%				4.71	63	28.0				315		
4:00	6.09	5.52	9.4%				4.73	63	28.0				315		
4:05	6.42	5.47	14.8%				4.67	63	28.0				315		
4:10	6.03	5.34	11.4%				4.72	63	28.0				315		
4:15	6.26	5.00	20.1%				4.72	63	28.0				315		
4:20	6.09	5.32	12.6%				4.67	63	28.0				315		
4:25	5.97	5.42	9.2%				4.68	63	28.0				315		
4:30	5.97	5.44	8.9%				4.71	63	28.0				315		
4:35	5.95	5.61	5.7%				4.67	63	28.0				315		
4:40	5.89	5.55	5.8%				4.78	63	28.0				315		
4:45								28	12.4	Wash Timer				140	
4:50								28	12.4					140	**Percent**
4:55								28	12.4					140	**of wash**
5:00								28	12.4					140	**%**
Averages	6.09	4.43	27.3%					61.69	27.42				21340	560	2.56%

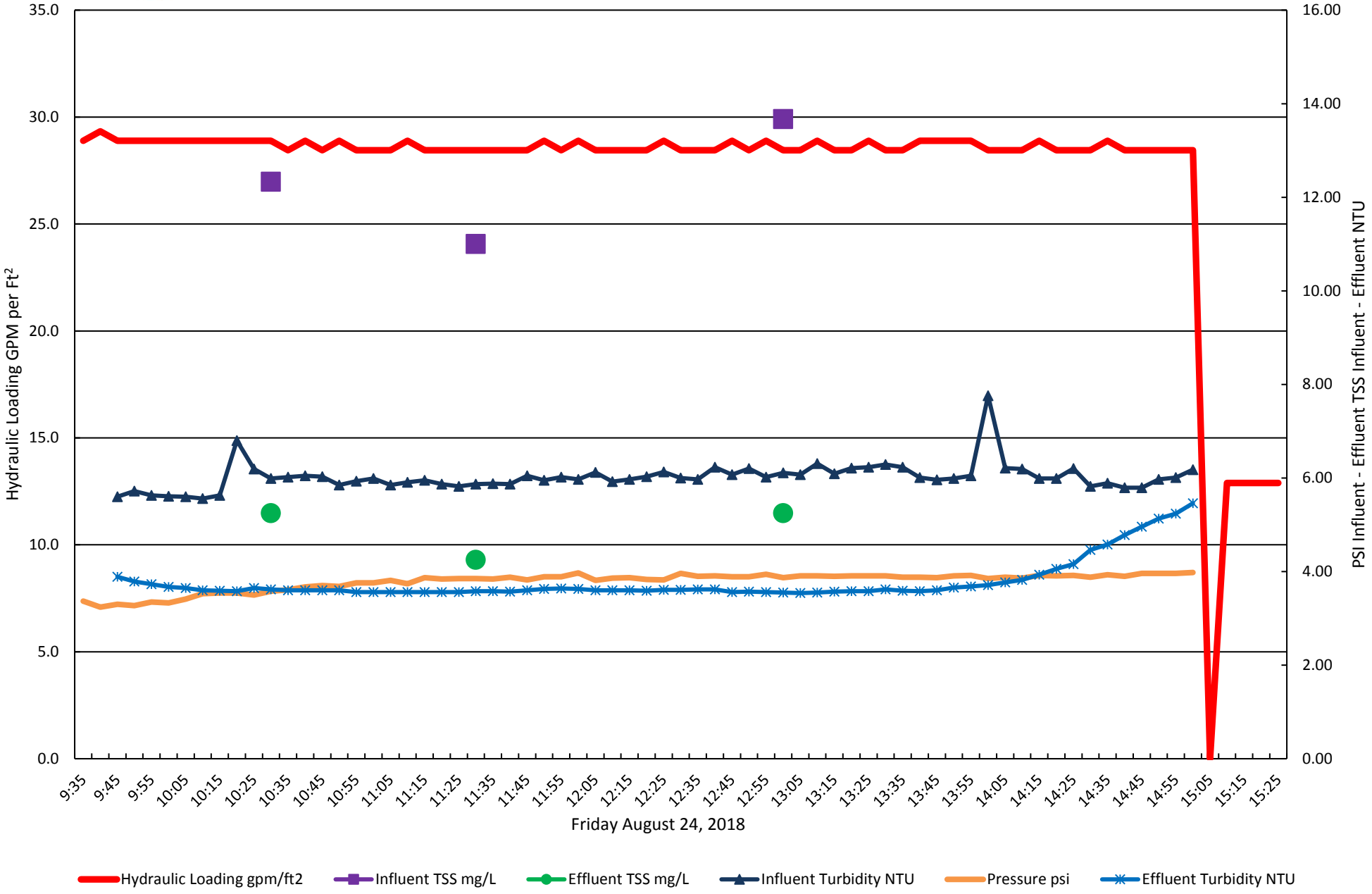
8/24/2018
Friday
40% Compression
Run #12

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
5:05							3.48	65	28.9	2ml/min SP-218-GH	40	5.2	325		
5:10							3.46	65	28.9				325		
5:15	5.47	4.00	26.9%				3.68	65	28.9				325		
5:20	5.53	3.88	29.8%				3.63	65	28.9				325		
5:25	5.46	3.71	32.1%				3.60	65	28.9				325		
5:30	5.53	3.68	33.5%				3.64	65	28.9				325		
5:35	5.50	3.65	33.6%				3.84	65	28.9				325		
5:40	5.47	3.64	33.5%				3.77	64	28.4				320		
5:45	5.41	3.62	33.1%				3.91	64	28.4				320		
5:50	5.41	3.60	33.5%				4.00	64	28.4				320		
5:55	5.59	3.58	36.0%				4.15	64	28.4				320		
6:00	5.83	3.58	38.6%				4.14	64	28.4				320		
6:05	5.60	3.56	36.4%				4.34	64	28.4				320		
6:10	5.64	3.56	36.9%				4.40	64	28.4				320		
6:15	5.62	3.55	36.8%				4.32	64	28.4				320		
6:20	5.65	3.53	37.5%				4.39	64	28.4				320		
6:25	5.70	3.52	38.2%				4.45	64	28.4				320		
6:30	5.72	3.52	38.5%				4.39	64	28.4				320		
6:35	5.72	3.55	37.9%				4.52	64	28.4				320		
6:40	5.75	3.53	38.6%				4.52	64	28.4				320		
6:45	5.59	3.53	36.9%				4.52	64	28.4				320		
6:50	5.65	3.54	37.3%				4.58	64	28.4				320		
6:55	5.63	3.56	36.8%				4.49	64	28.4				320		
7:00	5.74	3.55	38.2%				4.49	64	28.4				320		
7:05	6.58	3.56	45.9%				4.54	63	28.0				315		
7:10	5.79	3.55	38.7%				4.49	64	28.4				320		
7:15	5.79	3.56	38.5%				4.45	63	28.0				315		
7:20	5.99	3.56	40.6%				4.63	63	28.0				315		
7:25	5.89	3.58	39.2%				4.58	64	28.4				320		
7:30	5.77	3.60	37.6%				4.41	63	28.0				315		
7:35	5.79	3.60	37.8%				4.57	64	28.4				320		
7:40	5.92	3.60	39.2%				4.46	64	28.4				320		
7:45	5.95	3.62	39.2%				4.67	64	28.4				320		
7:50	5.99	3.67	38.7%				4.52	64	28.4				320		
7:55	6.22	3.66	41.2%				4.54	64	28.4				320		
8:00	5.78	3.74	35.3%				4.50	63	28.0				315		
8:05	5.75	3.74	35.0%				4.70	63	28.0				315		
8:10	5.75	3.80	33.9%				4.60	64	28.4				320		
8:15	5.90	3.90	33.9%				4.62	63	28.0				315		
8:20	5.77	3.97	31.2%				4.74	63	28.0				315		
8:25	7.01	4.09	41.7%				4.55	63	28.0				315		
8:30	5.91	4.18	29.3%	13.33	5.50	58.7%	4.56	63	28.0	TSS Samples			315		
8:35	5.96	4.29	28.0%				4.72	64	28.4				320		
8:40	5.75	4.53	21.2%				4.60	63	28.0				315		
8:45	5.85	4.76	18.6%				4.69	63	28.0				315		
8:50	6.01	4.92	18.1%				4.64	63	28.0				315		
8:55	5.77	5.04	12.7%				4.61	64	28.4				320		
9:00	5.68	5.20	8.5%				4.56	63	28.0				315		
9:05	5.76	5.36	6.9%				4.61	63	28.0				315		
9:10								0		Manuqal Wash				0	
9:15								29	12.9					145	
9:20								28	12.4					140	**Percent**

Run #12	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading			Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate			Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments		%		gallons/day	gallons/day	
9:25								28	12.4						140	**of wash**
9:30								28	12.4						140	**%**
Averages	5.78	3.84	33.5%	13.33	5.50	58.7%		60.02	27.18					15640	565	3.49%

SCHREIBER FuzzyFilter

Snohomish, WA Filter Test
30% Compression



8/24/2018
Friday
30% Compression
Run #13

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
9:35							3.37	65	28.9	2ml/min SP-218-GH	30	5.2	325	
9:40							3.24	66	29.3				330	
9:45	5.60	3.89	30.5%				3.30	65	28.9				325	
9:50	5.72	3.79	33.7%				3.27	65	28.9				325	
9:55	5.63	3.73	33.7%				3.35	65	28.9				325	
10:00	5.61	3.67	34.6%				3.33	65	28.9				325	
10:05	5.60	3.65	34.8%				3.41	65	28.9				325	
10:10	5.56	3.60	35.3%				3.53	65	28.9				325	
10:15	5.63	3.59	36.2%				3.54	65	28.9				325	
10:20	6.80	3.58	47.4%				3.54	65	28.9				325	
10:25	6.19	3.65	41.0%				3.50	65	28.9				325	
10:30	5.99	3.62	39.6%	12.33	5.25	57.4%	3.58	65	28.9	TSS Samples			325	
10:35	6.02	3.60	40.2%				3.61	64	28.4				320	
10:40	6.05	3.60	40.5%				3.67	65	28.9				325	
10:45	6.03	3.60	40.3%				3.70	64	28.4				320	
10:50	5.85	3.60	38.5%				3.68	65	28.9				325	
10:55	5.93	3.56	40.0%				3.76	64	28.4				320	
11:00	5.99	3.56	40.6%				3.76	64	28.4				320	
11:05	5.85	3.56	39.1%				3.81	64	28.4				320	
11:10	5.91	3.56	39.8%				3.74	65	28.9				325	
11:15	5.95	3.56	40.2%				3.87	64	28.4				320	
11:20	5.87	3.56	39.4%				3.84	64	28.4				320	
11:25	5.82	3.56	38.8%				3.85	64	28.4				320	
11:30	5.87	3.58	39.0%	11.00	4.25	61.4%	3.85	64	28.4	TSS Samples			320	
11:35	5.88	3.58	39.1%				3.84	64	28.4				320	
11:40	5.87	3.57	39.2%				3.88	64	28.4				320	
11:45	6.05	3.60	40.5%				3.82	64	28.4				320	
11:50	5.95	3.63	39.0%				3.89	65	28.9				325	
11:55	6.02	3.64	39.5%				3.89	64	28.4				320	
12:00	5.97	3.63	39.2%				3.97	65	28.9				325	
12:05	6.12	3.60	41.2%				3.81	64	28.4				320	
12:10	5.92	3.60	39.2%				3.86	64	28.4				320	
12:15	5.97	3.60	39.7%				3.87	64	28.4				320	
12:20	6.03	3.59	40.5%				3.83	64	28.4				320	
12:25	6.13	3.61	41.1%				3.82	65	28.9				325	
12:30	6.00	3.61	39.8%				3.96	64	28.4				320	
12:35	5.97	3.62	39.4%				3.90	64	28.4				320	
12:40	6.23	3.62	41.9%				3.91	64	28.4				320	
12:45	6.07	3.56	41.4%				3.89	65	28.9				325	
12:50	6.20	3.57	42.4%				3.89	64	28.4				320	
12:55	6.02	3.56	40.9%				3.94	65	28.9				325	
13:00	6.11	3.55	41.9%	13.67	5.25	61.6%	3.87	64	28.4	TSS Samples			320	
13:05	6.07	3.54	41.7%				3.91	64	28.4				320	
13:10	6.31	3.55	43.7%				3.91	65	28.9				325	
13:15	6.09	3.57	41.4%				3.90	64	28.4				320	
13:20	6.21	3.58	42.4%				3.91	64	28.4				320	
13:25	6.23	3.58	42.5%				3.91	65	28.9				325	
13:30	6.29	3.62	42.4%				3.91	64	28.4				320	
13:35	6.23	3.59	42.4%				3.88	64	28.4				320	
13:40	6.01	3.58	40.4%				3.88	65	28.9				325	
13:45	5.96	3.60	39.6%				3.87	65	28.9				325	
13:50	5.99	3.66	38.9%				3.91	65	28.9				325	

Run #13	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading			Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate			Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments		%		gallons/day	gallons/day	
13:55	6.05	3.68	39.2%				3.92	65	28.9					325		
14:00	7.76	3.71	52.2%				3.85	64	28.4					320		
14:05	6.21	3.77	39.3%				3.88	64	28.4					320		
14:10	6.19	3.82	38.3%				3.86	64	28.4					320		
14:15	5.99	3.93	34.4%				3.92	65	28.9					325		
14:20	5.99	4.06	32.2%				3.91	64	28.4					320		
14:25	6.20	4.16	32.9%				3.92	64	28.4					320		
14:30	5.82	4.46	23.4%				3.88	64	28.4					320		
14:35	5.89	4.58	22.2%				3.93	65	28.9					325		
14:40	5.79	4.78	17.4%				3.90	64	28.4					320		
14:45	5.79	4.96	14.3%				3.96	64	28.4					320		
14:50	5.97	5.13	14.1%				3.96	64	28.4					320		
14:55	6.01	5.24	12.8%				3.96	64	28.4					320		
15:00	6.18	5.46	11.7%				3.98	64	28.4					320		
15:05								0		Wash Timer					0	
15:10								29	12.9						145	
15:15								29	12.9						145	**Percent**
15:20								29	12.9						145	**of wash**
15:25								29	12.9						145	**%**
Averages	6.02	3.78	37.2%	12.33	4.92	60.1%		61.54	27.74					21265	580	2.66%

8/24/2018
Friday
30% Compression
Run #14

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
15:30							3.26	65	28.9	2ml/min SP-218-GH	30	5.2	325	
15:35							3.30	65	28.9				325	
15:40	5.57	4.01	28.0%				3.32	65	28.9				325	
15:45	5.59	3.91	30.1%				3.31	66	29.3				330	
15:50	5.49	3.86	29.7%				3.37	65	28.9				325	
15:55	5.60	3.81	32.0%				3.39	65	28.9				325	
16:00	5.82	3.78	35.1%				3.50	65	28.9				325	
16:05	5.63	3.74	33.6%				3.58	65	28.9				325	
16:10	5.59	3.74	33.1%				3.54	65	28.9				325	
16:15	5.79	3.73	35.6%				3.56	65	28.9				325	
16:20	5.80	3.71	36.0%				3.61	65	28.9				325	
16:25	5.99	3.71	38.1%				3.59	65	28.9				325	
16:30	5.81	3.71	36.1%				3.72	65	28.9				325	
16:35	5.89	3.70	37.2%				3.68	65	28.9				325	
16:40	5.89	3.70	37.2%				3.71	65	28.9				325	
16:45	5.80	3.69	36.4%				3.82	65	28.9				325	
16:50	5.96	3.72	37.6%				3.82	65	28.9				325	
16:55	5.87	3.66	37.6%				3.82	65	28.9				325	
17:00	5.91	3.64	38.4%				3.97	65	28.9				325	
17:05	5.84	3.62	38.0%				3.88	65	28.9				325	
17:10	5.94	3.62	39.1%				3.92	65	28.9				325	
17:15	5.69	3.61	36.6%				3.86	65	28.9				325	
17:20	5.79	3.60	37.8%				4.00	65	28.9				325	
17:25	6.07	3.61	40.5%				3.94	64	28.4				320	
17:30	5.99	3.61	39.7%				3.98	64	28.4				320	
17:35	5.97	3.60	39.7%				3.88	64	28.4				320	
17:40	5.96	3.62	39.3%				3.98	64	28.4				320	
17:45	6.11	3.70	39.4%				4.05	64	28.4				320	
17:50	5.71	3.63	36.4%				4.00	65	28.9				325	
17:55	5.84	3.61	38.2%				3.94	64	28.4				320	
18:00	5.89	3.61	38.7%				3.89	65	28.9				325	
18:05	5.99	3.61	39.7%				3.94	65	28.9				325	
18:10	5.94	3.62	39.1%				3.93	65	28.9				325	
18:15	5.89	3.63	38.4%				3.93	64	28.4				320	
18:20	5.99	3.66	38.9%				3.92	64	28.4				320	
18:25	5.87	3.66	37.6%				3.92	64	28.4				320	
18:30	6.28	3.66	41.7%				3.98	65	28.9				325	
18:35	6.20	3.70	40.3%				3.95	64	28.4				320	
18:40	5.97	3.71	37.9%				3.99	64	28.4				320	
18:45	5.97	3.75	37.2%				3.89	65	28.9				325	
18:50	6.12	3.79	38.1%				3.83	65	28.9				325	
18:55	5.87	3.81	35.1%				3.91	64	28.4				320	
19:00	6.18	3.85	37.7%				3.94	65	28.9				325	
19:05	6.23	4.06	34.8%				3.93	64	28.4				320	
19:10	6.32	4.09	35.3%				3.96	64	28.4				320	
19:15	6.46	4.16	35.6%				3.97	64	28.4				320	
19:20	6.15	4.14	32.7%				3.92	64	28.4				320	
19:25	6.22	4.12	33.8%				3.93	64	28.4				320	
19:30	5.91	4.17	29.4%				3.97	65	28.9				325	
19:35	6.17	4.34	29.7%				3.94	64	28.4				320	
19:40	6.13	4.42	27.9%				3.95	64	28.4				320	
19:45	6.19	4.58	26.0%				3.93	64	28.4				320	

Run #14	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading			Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate			Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments		%		gallons/day	gallons/day	
19:50	6.19	4.69	24.2%				3.97	64	28.4					320		
19:55	6.00	4.79	20.2%				3.96	64	28.4					320		
20:00	5.92	4.97	16.0%				3.96	64	28.4					320		
20:05	5.94	5.00	15.8%				3.96	65	28.9					325		
20:10	6.13	5.14	16.2%				3.92	64	28.4					320		
20:15	6.23	5.22	16.2%				3.94	64	28.4					320		
20:20	6.14	5.34	13.0%				3.94	65	28.9					325		
20:25	5.92	5.04	14.9%				3.98	64	28.4					320		
20:30	5.81	5.06	12.9%				3.86	64	28.4					320		
20:35	5.89	5.03	14.6%				3.94	65	28.9					325		
20:40	6.31	5.17	18.1%				3.90	65	28.9					325		
20:45	6.08	5.04	17.1%				3.95	65	28.9					325		
20:50	5.92	5.08	14.2%				3.93	64	28.4					320		
20:55	6.12	5.09	16.8%				3.87	64	28.4					320		
21:00								29	12.9	Wash Timer					145	
21:05								28	12.4						140	**Percent**
21:10								28	12.4						140	**of wash**
21:15								28	12.4						140	**%**
Averages	5.96	4.07	31.7%					62.50	27.78					21310	565	2.58%

8-24-18 -8-25-18
Friday - Saturday
30% Compression
Run #15

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
21:20							3.30	65	28.9	2ml/min SP-218-GH	30	5.2	325	
21:25							3.27	66	29.3				330	
21:30	5.67	4.24	25.2%				3.36	65	28.9				325	
21:35	5.51	3.97	27.9%				3.28	65	28.9				325	
21:40	5.64	3.89	31.0%				3.34	65	28.9				325	
21:45	5.59	3.84	31.3%				3.51	65	28.9				325	
21:50	5.58	3.79	32.1%				3.45	65	28.9				325	
21:55	5.76	3.76	34.7%				3.51	65	28.9				325	
22:00	5.70	3.74	34.4%				3.52	65	28.9				325	
22:05	5.92	3.73	37.0%				3.56	65	28.9				325	
22:10	5.82	3.70	36.4%				3.59	65	28.9				325	
22:15	5.87	3.66	37.6%				3.65	65	28.9				325	
22:20	5.66	3.65	35.5%				3.67	65	28.9				325	
22:25	6.26	3.65	41.7%				3.69	65	28.9				325	
22:30	5.78	3.63	37.2%				3.73	65	28.9				325	
22:35	5.73	3.61	37.0%				3.72	65	28.9				325	
22:40	5.84	3.61	38.2%				3.72	65	28.9				325	
22:45	5.94	3.61	39.2%				3.80	64	28.4				320	
22:50	6.30	3.60	42.9%				3.73	65	28.9				325	
22:55	6.08	3.58	41.1%				3.84	65	28.9				325	
23:00	6.10	3.68	39.7%				3.85	65	28.9				325	
23:05	5.93	3.72	37.3%				3.81	65	28.9				325	
23:10	6.03	3.68	39.0%				3.84	64	28.4				320	
23:15	5.58	3.67	34.2%				3.85	64	28.4				320	
23:20	6.57	3.68	44.0%				3.88	65	28.9				325	
23:25	6.01	3.68	38.8%				3.87	65	28.9				325	
23:30	6.14	3.67	40.2%				3.83	64	28.4				320	
23:35	5.77	3.69	36.0%				3.86	65	28.9				325	
23:40	5.83	3.70	36.5%				3.84	65	28.9				325	
23:45	5.61	3.68	34.4%				3.86	64	28.4				320	
23:50	5.63	3.70	34.3%				3.87	64	28.4				320	
23:55	5.54	3.70	33.2%				3.88	64	28.4				320	
0:00	6.02	3.73	38.0%				3.92	64	28.4	8/25/2018			320	
0:05	5.77	3.74	35.2%				3.87	65	28.9				325	
0:10	5.79	3.74	35.4%				3.89	65	28.9				325	
0:15	5.60	3.74	33.2%				3.90	64	28.4				320	
0:20	5.92	3.78	36.1%				3.90	65	28.9				325	
0:25	5.65	3.78	33.1%				3.90	65	28.9				325	
0:30	5.59	3.76	32.7%				3.93	64	28.4				320	
0:35	5.72	3.77	34.1%				3.91	64	28.4				320	
0:40	5.94	3.79	36.2%				3.87	65	28.9				325	
0:45	5.95	3.86	35.1%				3.96	65	28.9				325	
0:50	5.87	3.92	33.2%				3.89	65	28.9				325	
0:55	6.06	3.94	35.0%				3.83	64	28.4				320	
1:00	6.41	3.96	38.2%				3.90	65	28.9				325	
1:05	5.99	4.02	32.9%				3.97	64	28.4				320	
1:10	5.84	4.12	29.5%				3.93	64	28.4				320	
1:15	6.14	4.17	32.1%				3.88	64	28.4				320	
1:20	6.07	4.22	30.5%				3.88	64	28.4				320	
1:25	6.11	4.48	26.7%				3.95	64	28.4				320	
1:30	6.71	4.58	31.7%				3.89	65	28.9				325	
1:35	5.86	4.80	18.1%				3.98	64	28.4				320	

Run #15	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate			Wash Trip Point	Filter run	wash	
Time	**NTU**	**NTU**	**%**	**mg/l**	**mg/l**	**%**	**Pressure psi**	**Flow gpm**	**gpm/ft^2**	**Comments**	**Percent Compression %**		**gallons/day**	**gallons/day**	
1:40	5.99	4.96	17.2%				3.85	65	28.9				325		
1:45	5.91	5.13	13.2%				3.90	64	28.4				320		
1:50	6.01	5.20	13.5%				3.88	65	28.9				325		
1:55	5.83	5.25	9.9%				3.90	64	28.4				320		
2:00	5.63	5.37	4.6%				3.98	64	28.4				320		
2:05	6.02	5.50	8.6%				3.88	64	28.4				320		
2:10	5.95	5.65	5.0%				3.81	64	28.4				320		
2:15	5.77	5.08	12.0%				3.90	64	28.4				320		
2:20	6.05	5.08	16.0%				3.83	64	28.4				320		
2:25	5.80	5.07	12.6%				3.95	64	28.4				320		
2:30	5.57	5.06	9.2%				3.86	64	28.4				320		
2:35	5.85	5.09	13.0%				3.88	64	28.4				320		
2:40	5.77	5.05	12.5%				3.88	64	28.4				320		
2:45	5.38	5.09	5.4%				3.91	64	28.4				320		
2:50	5.54	5.01	9.6%				3.89	64	28.4				320		
2:55								29	12.9	Wash Timer				145	
3:00								28	12.4					140	**Percent**
3:05								29	12.9					145	**of wash**
3:10								28	12.4					140	**%**
Averages	5.87	4.14	29.5%					62.52	27.79				21625	570	2.57%

8/25/2018
Saturday
30% Compression
Run #16

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
3:15							3.29	64	28.4	2ml/min SP-218-GH	30	5.2	320	
3:20							3.29	66	29.3				330	
3:25	5.63	4.07	27.7%				3.31	65	28.9				325	
3:30	5.46	3.75	31.3%				3.31	65	28.9				325	
3:35	5.47	3.64	33.5%				3.33	65	28.9				325	
3:40	5.42	3.58	33.9%				3.38	65	28.9				325	
3:45	5.86	3.55	39.4%				3.45	65	28.9				325	
3:50	5.60	3.52	37.1%				3.49	65	28.9				325	
3:55	5.73	3.50	38.9%				3.54	65	28.9				325	
4:00	5.36	3.50	34.7%				3.64	65	28.9				325	
4:05	5.32	3.47	34.8%				3.62	65	28.9				325	
4:10	5.54	3.47	37.4%				3.67	65	28.9				325	
4:15	5.54	3.45	37.7%				3.71	65	28.9				325	
4:20	5.69	3.44	39.5%				3.78	65	28.9				325	
4:25	5.94	3.42	42.4%				3.80	65	28.9				325	
4:30	5.75	3.42	40.5%				3.84	65	28.9				325	
4:35	5.71	3.40	40.5%				3.82	65	28.9				325	
4:40	5.87	3.40	42.1%				3.90	64	28.4				320	
4:45	5.75	3.40	40.9%				3.93	65	28.9				325	
4:50	5.90	3.39	42.5%				3.94	65	28.9				325	
4:55	6.05	3.56	41.2%				3.92	64	28.4				320	
5:00	5.77	3.52	39.0%				3.95	65	28.9				325	
5:05	5.78	3.50	39.4%				3.90	64	28.4				320	
5:10	5.95	3.51	41.0%				3.88	64	28.4				320	
5:15	5.67	3.54	37.6%				3.91	64	28.4				320	
5:20	5.84	3.55	39.2%				3.89	64	28.4				320	
5:25	5.77	3.58	38.0%				3.97	64	28.4				320	
5:30	5.70	3.57	37.4%				3.88	65	28.9				325	
5:35	5.72	3.55	37.9%				3.91	65	28.9				325	
5:40	5.91	3.52	40.4%				3.87	65	28.9				325	
5:45	5.50	3.55	35.5%				3.88	64	28.4				320	
5:50	5.67	3.57	37.0%				3.86	65	28.9				325	
5:55	5.65	3.57	36.8%				3.85	64	28.4				320	
6:00	5.87	3.58	39.0%				3.83	62	27.6				310	
6:05	5.65	3.58	36.6%				3.85	62	27.6				310	
6:10	5.84	3.64	37.7%				3.86	62	27.6				310	
6:15	5.85	3.64	37.8%				3.87	63	28.0				315	
6:20	5.79	3.70	36.1%				3.84	62	27.6				310	
6:25	5.65	3.73	34.0%				3.79	62	27.6				310	
6:30	5.96	3.72	37.6%				3.89	62	27.6				310	
6:35	5.90	3.75	36.4%				3.90	62	27.6				310	
6:40	5.97	3.84	35.7%				3.84	62	27.6				310	
6:45	5.57	3.88	30.3%				3.88	62	27.6				310	
6:50	5.57	3.85	30.9%				3.81	62	27.6				310	
6:55	6.65	3.91	41.2%				3.87	62	27.6				310	
7:00	5.61	4.09	27.1%				3.85	62	27.6				310	
7:05	5.55	4.25	23.4%				3.95	62	27.6				310	
7:10	5.90	4.36	26.1%				3.86	62	27.6				310	
7:15	5.64	4.53	19.7%				3.81	62	27.6				310	
7:20	5.76	4.84	16.0%				3.80	62	27.6				310	
7:25	5.66	4.97	12.2%				3.79	62	27.6				310	
7:30	5.57	5.11	8.3%				3.85	62	27.6				310	

Run #16	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	**NTU**	**NTU**	**%**	**mg/l**	**mg/l**	**%**	**Pressure psi**	**Flow gpm**	**gpm/ft^2**	**Comments**	**%**		**gallons/day**	**gallons/day**	
7:35	5.79	5.15	11.1%				3.84	63	28.0				315		
7:40	5.54	5.30	4.3%				3.79	62	27.6				310		
7:45	5.49	5.44	0.9%				3.83	62	27.6				310		
7:50	5.81	5.49	5.5%				3.76	63	28.0				315		
7:55	5.91	5.64	4.6%				3.77	62	27.6				310		
8:00	5.95	5.65	5.0%				3.82	63	28.0				315		
8:05	5.53	5.08	8.1%				3.85	62	27.6				310		
8:10	5.79	5.09	12.1%				3.84	62	27.6				310		
8:15	5.62	5.00	11.0%				3.85	62	27.6				310		
8:20	5.85	5.02	14.2%				3.86	62	27.6				310		
8:25	5.50	5.00	9.1%				3.79	62	27.6				310		
8:30	5.78	5.04	12.8%				3.83	62	27.6				310		
8:35	5.67	5.00	11.8%				3.80	62	27.6				310		
8:40	5.99	5.04	15.9%				3.80	62	27.6				310		
8:45	5.72	5.06	11.5%				3.80	62	27.6				310		
8:50								33	14.7	Wash Timer				165	
8:55								33	14.7					165	**Percent**
9:00								32	14.2					160	**of wash**
9:05								32	14.2					160	**%**
Averages	5.73	4.07	29.0%					61.66	27.41				21240	650	2.97%

8/25/2018
Saturday
20% Compression
Run #17

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
9:10							3.13	63	28.0	2ml/min SP-218-GH	20	5.2	315	
9:15							3.09	64	28.4				320	
9:20	5.27	4.19	20.5%				3.14	63	28.0				315	
9:25	5.22	3.88	25.7%				3.18	63	28.0				315	
9:30	5.20	3.78	27.3%				3.13	63	28.0				315	
9:35	5.21	3.82	26.7%				3.17	63	28.0				315	
9:40	5.26	3.80	27.8%				3.15	63	28.0				315	
9:45	5.32	3.75	29.5%				3.21	63	28.0				315	
9:50	5.27	3.73	29.2%				3.20	63	28.0				315	
9:55	5.42	3.71	31.5%				3.25	63	28.0				315	
10:00	5.37	3.70	31.1%				3.13	63	28.0				315	
10:05	5.38	3.68	31.6%				3.25	63	28.0				315	
10:10	5.31	3.66	31.1%				3.29	63	28.0				315	
10:15	5.36	3.63	32.3%				3.32	63	28.0				315	
10:20	5.35	3.60	32.7%				3.27	63	28.0				315	
10:25	5.39	3.58	33.6%				3.28	63	28.0				315	
10:30	5.52	3.54	35.9%				3.38	63	28.0				315	
10:35	5.57	3.50	37.2%				3.33	63	28.0				315	
10:40	5.53	3.50	36.7%				3.41	63	28.0				315	
10:45	5.67	3.48	38.6%				3.44	63	28.0				315	
10:50	5.60	3.46	38.2%				3.38	63	28.0				315	
10:55	5.45	3.45	36.7%				3.44	63	28.0				315	
11:00	5.49	3.45	37.2%				3.43	63	28.0				315	
11:05	5.41	3.44	36.4%				3.44	63	28.0				315	
11:10	5.55	3.44	38.0%				3.48	63	28.0				315	
11:15	5.59	3.44	38.5%				3.50	63	28.0				315	
11:20	5.50	3.43	37.6%				3.60	63	28.0				315	
11:25	5.53	3.43	38.0%				3.54	63	28.0				315	
11:30	5.45	3.42	37.2%				3.61	63	28.0				315	
11:35	5.41	3.42	36.8%				3.52	63	28.0				315	
11:40	5.61	3.40	39.4%				3.55	63	28.0				315	
11:45	5.52	3.40	38.4%				3.52	63	28.0				315	
11:50	5.72	3.40	40.6%				3.51	63	28.0				315	
11:55	5.77	3.40	41.1%				3.61	63	28.0				315	
12:00	5.82	3.42	41.2%				3.52	63	28.0				315	
12:05	5.74	3.45	39.9%				3.57	63	28.0				315	
12:10	5.57	3.45	38.1%				3.59	63	28.0				315	
12:15	5.54	3.43	38.1%				3.58	63	28.0				315	
12:20	5.70	3.42	40.0%				3.59	62	27.6				310	
12:25	5.63	3.42	39.3%				3.59	63	28.0				315	
12:30	5.52	3.42	38.0%				3.66	63	28.0				315	
12:35	5.47	3.48	36.4%				3.58	63	28.0				315	
12:40	5.65	3.48	38.4%				3.59	63	28.0				315	
12:45	5.75	3.50	39.1%				3.58	63	28.0				315	
12:50	5.48	3.44	37.2%				3.65	63	28.0				315	
12:55	5.72	3.43	40.0%				3.58	63	28.0				315	
13:00	5.67	3.43	39.5%				3.59	63	28.0				315	
13:05	5.87	3.43	41.6%				3.58	63	28.0				315	
13:10	5.66	3.43	39.4%				3.63	63	28.0				315	
13:15	5.71	3.44	39.8%				3.52	63	28.0				315	
13:20	5.81	3.45	40.6%				3.61	63	28.0				315	
13:25	5.83	3.46	40.7%				3.62	63	28.0				315	

Run #17	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate			Wash Trip Point	Filter run	wash	
Time	**NTU**	**NTU**	**%**	**mg/l**	**mg/l**	**%**	**psi**	**Flow gpm**	**gpm/ft^2**	**Comments**	**Percent Compression %**		**gallons/day**	**gallons/day**	
13:30	5.85	3.46	40.9%				3.65	63	28.0				315		
13:35	5.86	3.50	40.3%				3.58	63	28.0				315		
13:40	5.66	3.47	38.7%				3.59	62	27.6				310		
13:45	5.69	3.47	39.0%				3.68	63	28.0				315		
13:50	5.53	3.47	37.3%				3.59	63	28.0				315		
13:55	5.79	3.48	39.9%				3.59	63	28.0				315		
14:00	5.57	3.48	37.5%				3.67	63	28.0				315		
14:05	5.47	3.48	36.4%				3.64	62	27.6				310		
14:10	5.50	3.49	36.5%				3.63	62	27.6				310		
14:15	5.60	3.50	37.5%				3.62	63	28.0				315		
14:20	5.68	3.51	38.2%				3.56	63	28.0				315		
14:25	5.85	3.56	39.1%				3.62	63	28.0				315		
14:30	5.70	3.56	37.5%				3.62	63	28.0				315		
14:35	5.74	3.56	38.0%				3.63	63	28.0				315		
14:40	5.52	3.56	35.5%				3.64	63	28.0				315		
14:45								28	12.4	Wash Timer				140	
14:50								28	12.4					140	**Percent**
14:55								28	12.4					140	**of wash**
15:00								28	12.4					140	**%**
Averages	5.56	3.52	36.6%					60.99	27.10				21090	560	2.59%

8/25/2018
Saturday
20% Compression
Run #18

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
15:05							3.15	63	28.0	2ml/min SP-218-GH	20	5.2	315	
15:10							3.20	63	28.0				315	
15:15	5.32	3.96	25.6%				3.16	64	28.4				320	
15:20	5.30	3.88	26.8%				3.15	63	28.0				315	
15:25	5.39	3.82	29.1%				3.21	63	28.0				315	
15:30	5.34	3.75	29.8%				3.28	63	28.0				315	
15:35	5.42	3.72	31.4%				3.21	63	28.0				315	
15:40	5.37	3.70	31.1%				3.36	63	28.0				315	
15:45	5.43	3.66	32.6%				3.23	63	28.0				315	
15:50	5.42	3.62	33.2%				3.16	63	28.0				315	
15:55	5.49	3.61	34.2%				3.26	63	28.0				315	
16:00	5.40	3.61	33.1%				3.22	63	28.0				315	
16:05	5.33	3.58	32.8%				3.32	63	28.0				315	
16:10	5.39	3.55	34.1%				3.33	63	28.0				315	
16:15	5.43	3.54	34.8%				3.33	63	28.0				315	
16:20	5.43	3.54	34.8%				3.36	63	28.0				315	
16:25	5.45	3.52	35.4%				3.34	63	28.0				315	
16:30	5.42	3.50	35.4%				3.39	63	28.0				315	
16:35	5.37	3.50	34.8%				3.34	63	28.0				315	
16:40	5.39	3.50	35.1%				3.40	63	28.0				315	
16:45	5.18	3.48	32.8%				3.43	63	28.0				315	
16:50	5.31	3.46	34.8%				3.42	63	28.0				315	
16:55	5.19	3.46	33.3%				3.44	63	28.0				315	
17:00	5.32	3.43	35.5%				3.41	63	28.0				315	
17:05	5.31	3.43	35.4%				3.46	63	28.0				315	
17:10	5.29	3.43	35.2%				3.49	63	28.0				315	
17:15	5.61	3.43	38.9%				3.53	63	28.0				315	
17:20	5.33	3.40	36.2%				3.48	63	28.0				315	
17:25	5.55	3.40	38.7%				3.54	63	28.0				315	
17:30	5.46	3.40	37.7%				3.46	63	28.0				315	
17:35	5.57	3.40	39.0%				3.58	63	28.0				315	
17:40	5.64	3.40	39.7%				3.55	63	28.0				315	
17:45	5.55	3.42	38.4%				3.55	63	28.0				315	
17:50	5.49	3.37	38.6%				3.56	63	28.0				315	
17:55	5.48	3.37	38.5%				3.60	63	28.0				315	
18:00	5.45	3.35	38.5%				3.60	63	28.0				315	
18:05	5.50	3.35	39.1%				3.60	63	28.0				315	
18:10	5.50	3.35	39.1%				3.61	63	28.0				315	
18:15	5.67	3.34	41.1%				3.59	63	28.0				315	
18:20	5.45	3.34	38.7%				3.62	63	28.0				315	
18:25	5.77	3.34	42.1%				3.60	62	27.6				310	
18:30	5.55	3.34	39.8%				3.63	63	28.0				315	
18:35	5.64	3.34	40.8%				3.63	63	28.0				315	
18:40	5.69	3.35	41.1%				3.59	63	28.0				315	
18:45	5.50	3.36	38.9%				3.62	63	28.0				315	
18:50	5.70	3.36	41.1%				3.63	63	28.0				315	
18:55	5.67	3.37	40.6%				3.67	63	28.0				315	
19:00	5.53	3.38	38.9%				3.65	63	28.0				315	
19:05	5.54	3.38	39.0%				3.58	63	28.0				315	
19:10	5.45	3.40	37.6%				3.63	63	28.0				315	
19:15	5.73	3.40	40.7%				3.65	63	28.0				315	
19:20	5.56	3.42	38.5%				3.63	63	28.0				315	

Run #18	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate			Percent Compression	Wash Trip Point	Filter run	wash	
Time	**NTU**	**NTU**	**%**	**mg/l**	**mg/l**	**%**	**Pressure psi**	**Flow gpm**	**gpm/ft^2**	**Comments**		**%**		**gallons/day**	**gallons/day**	
19:25	5.49	3.42	37.7%				3.68	62	27.6					310		
19:30	5.62	3.43	39.0%				3.60	62	27.6					310		
19:35	5.35	3.42	36.1%				3.65	62	27.6					310		
19:40	5.21	3.43	34.2%				3.60	62	27.6					310		
19:45	5.43	3.44	36.6%				3.66	62	27.6					310		
19:50	5.31	3.45	35.0%				3.76	62	27.6					310		
19:55	5.25	3.53	32.8%				3.62	63	28.0					315		
20:00	5.24	3.54	32.4%				3.69	62	27.6					310		
20:05	5.39	3.56	34.0%				3.66	62	27.6					310		
20:10	5.32	3.56	33.1%				3.65	63	28.0					315		
20:15	5.21	3.56	31.7%				3.67	62	27.6					310		
20:20	5.15	3.56	30.9%				3.58	62	27.6					310		
20:25	5.01	3.56	28.9%				3.67	62	27.6					310		
20:30	5.01	3.55	29.1%				3.67	63	28.0					315		
20:35								0		Wash Timer					0	
20:40								29	12.9						145	
20:45								29	12.9						145	**Percent**
20:50								29	12.9						145	**of wash**
20:55								29	12.9						145	**%**
Averages	5.43	3.48	35.8%					60.04	27.07					20735	580	2.72%

8-25-18 - 8-26-18
Saturday - Sunday
20% Compression
Run #19

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
21:00							3.12	63	28.0	2ml/min SP-218-GH	20	5.2	315	
21:05							3.12	63	28.0				315	
21:10	5.38	3.90	27.5%				3.21	63	28.0				315	
21:15	5.59	3.86	30.9%				3.10	64	28.4				320	
21:20	5.85	3.84	34.4%				3.26	63	28.0				315	
21:25	5.85	3.77	35.6%				3.17	63	28.0				315	
21:30	5.57	3.70	33.6%				3.19	64	28.4				320	
21:35	5.73	3.63	36.6%				3.25	63	28.0				315	
21:40	5.57	3.58	35.7%				3.18	63	28.0				315	
21:45	5.90	3.56	39.7%				3.28	63	28.0				315	
21:50	5.38	3.54	34.2%				3.29	63	28.0				315	
21:55	5.55	3.52	36.6%				3.35	63	28.0				315	
22:00	5.97	3.52	41.0%				3.34	63	28.0				315	
22:05	5.46	3.48	36.3%				3.29	63	28.0				315	
22:10	5.43	3.46	36.3%				3.36	63	28.0				315	
22:15	5.44	3.45	36.6%				3.39	63	28.0				315	
22:20	5.41	3.42	36.8%				3.43	63	28.0				315	
22:25	5.69	3.42	39.9%				3.49	63	28.0				315	
22:30	5.52	3.40	38.4%				3.46	63	28.0				315	
22:35	5.41	3.41	37.0%				3.46	63	28.0				315	
22:40	5.57	3.38	39.3%				3.45	63	28.0				315	
22:45	5.31	3.38	36.3%				3.49	63	28.0				315	
22:50	5.53	3.37	39.1%				3.47	63	28.0				315	
22:55	5.67	3.36	40.7%				3.53	63	28.0				315	
23:00	5.57	3.36	39.7%				3.49	63	28.0				315	
23:05	5.41	3.41	37.0%				3.50	63	28.0				315	
23:10	5.51	3.40	38.3%				3.56	63	28.0				315	
23:15	5.31	3.40	36.0%				3.57	63	28.0				315	
23:20	5.23	3.40	35.0%				3.59	63	28.0				315	
23:25	5.58	3.40	39.1%				3.58	63	28.0				315	
23:30	5.53	3.38	38.9%				3.58	63	28.0				315	
23:35	5.67	3.40	40.0%				3.57	63	28.0				315	
23:40	5.85	3.34	42.9%				3.61	63	28.0				315	
23:45	5.59	3.33	40.4%				3.60	62	27.6				310	
23:50	5.59	3.32	40.6%				3.49	63	28.0				315	
23:55	5.60	3.32	40.7%				3.59	63	28.0				315	
0:00	5.83	3.32	43.1%				3.62	62	27.6	8/26/2018			310	
0:05	5.72	3.32	42.0%				3.55	63	28.0				315	
0:10	5.61	3.32	40.8%				3.55	62	27.6				310	
0:15	5.65	3.32	41.2%				3.56	63	28.0				315	
0:20	5.35	3.33	37.8%				3.55	63	28.0				315	
0:25	5.39	3.34	38.0%				3.56	63	28.0				315	
0:30	5.59	3.35	40.1%				3.54	63	28.0				315	
0:35	5.37	3.38	37.1%				3.54	63	28.0				315	
0:40	5.49	3.40	38.1%				3.47	63	28.0				315	
0:45	5.65	3.40	39.8%				3.56	63	28.0				315	
0:50	5.42	3.40	37.3%				3.55	63	28.0				315	
0:55	5.42	3.40	37.3%				3.54	63	28.0				315	
1:00	5.32	3.45	35.2%				3.54	63	28.0				315	
1:05	5.19	3.45	33.5%				3.58	63	28.0				315	
1:10	5.62	3.53	37.2%				3.53	63	28.0				315	
1:15	5.77	3.55	38.5%				3.54	63	28.0				315	

Run #19	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	**NTU**	**NTU**	**%**	**mg/l**	**mg/l**	**%**	**psi**	**Flow gpm**	**gpm/ft^2**	**Comments**	**%**		**gallons/day**	**gallons/day**	
1:20	5.32	3.55	33.3%				3.55	63	28.0				315		
1:25	5.43	3.53	35.0%				3.48	63	28.0				315		
1:30	5.12	3.58	30.1%				3.53	63	28.0				315		
1:35	5.67	3.63	36.0%				3.51	63	28.0				315		
1:40	5.33	3.75	29.6%				3.57	63	28.0				315		
1:45	5.26	3.84	27.0%				3.54	63	28.0				315		
1:50	5.11	3.90	23.7%				3.52	63	28.0				315		
1:55	5.28	4.02	23.9%				3.50	63	28.0				315		
2:00	5.29	4.11	22.3%				3.52	63	28.0				315		
2:05	5.08	4.22	16.9%				3.53	62	27.6				310		
2:10	5.26	4.31	18.1%				3.49	62	27.6				310		
2:15	5.12	4.41	13.9%				3.54	63	28.0				315		
2:20	5.26	4.51	14.3%				3.56	63	28.0				315		
2:25	5.20	4.56	12.3%				3.53	63	28.0				315		
2:30								0		Wash Timer				0	
2:35								30	13.3					150	
2:40								29	12.9					145	**Percent of wash %**
2:45								29	12.9					145	
2:50								29	12.9					145	
Averages	5.49	3.57	34.9%					60.17	27.12				20775	585	2.74%

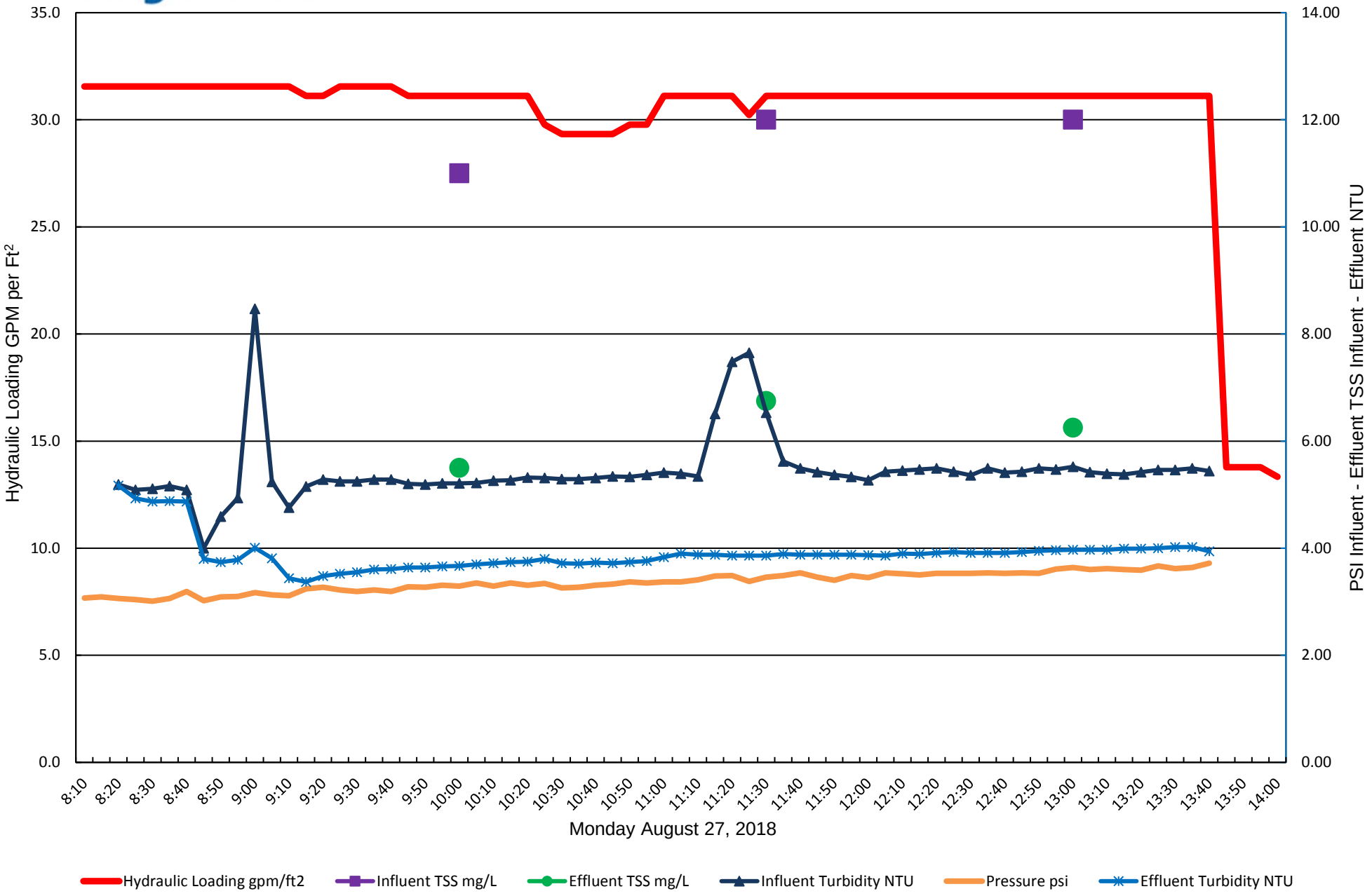
8/26/2018
Sunday
20% Compression
Run #20

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
2:55							3.10	63	28.0	2ml/min SP-218-GH	20	5.1	315	
3:00							3.13	64	28.4				320	
3:05	5.21	3.74	28.2%				3.14	64	28.4				320	
3:10	5.23	3.65	30.2%				3.13	63	28.0				315	
3:15	5.39	3.60	33.2%				3.23	64	28.4				320	
3:20	5.02	3.56	29.1%				3.21	64	28.4				320	
3:25	5.33	3.51	34.1%				3.21	63	28.0				315	
3:30	5.33	3.50	34.3%				3.23	63	28.0				315	
3:35	5.26	3.45	34.4%				3.23	64	28.4				320	
3:40	5.33	3.44	35.5%				3.28	64	28.4				320	
3:45	5.27	3.42	35.1%				3.29	64	28.4				320	
3:50	5.82	3.39	41.8%				3.33	63	28.0				315	
3:55	5.52	3.38	38.8%				3.27	63	28.0				315	
4:00	5.25	3.36	36.0%				3.31	63	28.0				315	
4:05	5.26	3.35	36.3%				3.38	63	28.0				315	
4:10	5.59	3.34	40.3%				3.30	63	28.0				315	
4:15	5.45	3.32	39.1%				3.36	63	28.0				315	
4:20	5.65	3.32	41.2%				3.45	63	28.0				315	
4:25	5.69	3.32	41.7%				3.47	63	28.0				315	
4:30	5.50	3.31	39.8%				3.44	63	28.0				315	
4:35	5.47	3.30	39.7%				3.43	63	28.0				315	
4:40	5.50	3.29	40.2%				3.46	63	28.0				315	
4:45	5.47	3.33	39.1%				3.51	63	28.0				315	
4:50	5.39	3.31	38.6%				3.44	63	28.0				315	
4:55	5.56	3.30	40.6%				3.51	63	28.0				315	
5:00	5.50	3.30	40.0%				3.48	63	28.0				315	
5:05	5.65	3.30	41.6%				3.51	63	28.0				315	
5:10	5.69	3.30	42.0%				3.53	63	28.0				315	
5:15	5.52	3.30	40.2%				3.58	63	28.0				315	
5:20	5.63	3.30	41.4%				3.58	63	28.0				315	
5:25	5.70	3.31	41.9%				3.56	63	28.0				315	
5:30	5.59	3.30	41.0%				3.58	63	28.0				315	
5:35	5.41	3.30	39.0%				3.61	63	28.0				315	
5:40	5.44	3.29	39.5%				3.59	63	28.0				315	
5:45	5.80	3.32	42.8%				3.66	63	28.0				315	
5:50	5.81	3.32	42.9%				3.52	63	28.0				315	
5:55	5.84	3.32	43.2%				3.60	63	28.0				315	
6:00	5.61	3.30	41.2%				3.54	63	28.0				315	
6:05	5.83	3.25	44.3%				3.59	63	28.0				315	
6:10	6.04	3.25	46.2%				3.61	63	28.0				315	
6:15	5.60	3.25	42.0%				3.60	63	28.0				315	
6:20	5.96	3.25	45.5%				3.59	63	28.0				315	
6:25	5.83	3.27	43.9%				3.60	63	28.0				315	
6:30	5.77	3.28	43.2%				3.56	63	28.0				315	
6:35	5.56	3.28	41.0%				3.58	63	28.0				315	
6:40	5.43	3.30	39.2%				3.56	63	28.0				315	
6:45	5.44	3.32	39.0%				3.60	63	28.0				315	
6:50	5.59	3.32	40.6%				3.63	63	28.0				315	
6:55	5.58	3.32	40.5%				3.62	63	28.0				315	
7:00	5.70	3.34	41.4%				3.59	63	28.0				315	
7:05	5.60	3.35	40.2%				3.62	63	28.0				315	
7:10	5.69	3.35	41.1%				3.55	63	28.0				315	

Run #20	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading			Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate		Percent	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments	Compression %		gallons/day	gallons/day	
7:15	5.63	3.38	40.0%				3.54	63	28.0				315		
7:20	5.74	3.42	40.4%				3.57	63	28.0				315		
7:25	5.95	3.42	42.5%				3.49	63	28.0				315		
7:30	5.83	3.42	41.3%				3.49	63	28.0				315		
7:35	5.79	3.44	40.6%				3.59	63	28.0				315		
7:40	6.37	3.48	45.4%				3.62	63	28.0				315		
7:45	5.92	3.50	40.9%				3.52	63	28.0				315		
7:50	5.45	3.53	35.2%				3.53	63	28.0				315		
7:55	6.63	3.59	45.9%				3.56	63	28.0				315		
8:00	5.70	3.62	36.5%				3.57	63	28.0				315		
8:05	6.12	3.72	39.2%				3.57	63	28.0				315		
8:10	6.07	3.83	36.9%				3.36	63	28.0				315		
8:15	6.10	3.88	36.4%				3.53	63	28.0				315		
8:20	6.07	3.90	35.7%				3.53	63	28.0				315		
8:25								29	12.9	Wash Timer				145	
8:30								29	12.9					145	**Percent**
8:35								29	12.9					145	**of wash**
8:40								29	12.9					145	**%**
Averages	5.64	3.40	39.6%					61.16	27.18				20825	580	2.71%

SCHREIBER FuzzyFilter

Snohomish, WA Filter Test
20% Compression



8/27/2018
Monday
20% Compression
Run #21

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
8:10							3.07	71	31.6	Worked out the old polymer	20	5.0	355	
8:15							3.09	71	31.6				355	
8:20	5.19	5.17	0.4%				3.06	71	31.6				355	
8:25	5.09	4.93	3.1%				3.04	71	31.6				355	
8:30	5.11	4.87	4.7%				3.01	71	31.6				355	
8:35	5.16	4.88	5.4%				3.06	71	31.6				355	
8:40	5.09	4.87	4.3%				3.19	71	31.6				355	
8:45	4.00	3.80	5.0%				3.02	71	31.6				355	
8:50	4.59	3.74	18.5%				3.09	71	31.6				355	
8:55	4.94	3.78	23.5%				3.10	71	31.6	Started feeding 1679 at 2ml/min			355	
9:00	8.47	4.01	52.7%				3.17	71	31.6	To total flow			355	
9:05	5.24	3.81	27.3%				3.13	71	31.6				355	
9:10	4.76	3.44	27.7%				3.11	71	31.6				355	
9:15	5.15	3.37	34.6%				3.24	70	31.1				350	
9:20	5.28	3.48	34.1%				3.27	70	31.1				350	
9:25	5.25	3.52	33.0%				3.22	71	31.6				355	
9:30	5.25	3.55	32.4%				3.19	71	31.6				355	
9:35	5.28	3.60	31.8%				3.22	71	31.6				355	
9:40	5.28	3.61	31.6%				3.19	71	31.6				355	
9:45	5.20	3.64	30.0%				3.28	70	31.1				350	
9:50	5.19	3.64	29.9%				3.27	70	31.1				350	
9:55	5.21	3.66	29.8%				3.31	70	31.1				350	
10:00	5.21	3.67	29.6%	11.00	5.50	50.0%	3.29	70	31.1	TSS Samples			350	
10:05	5.22	3.70	29.1%				3.35	70	31.1				350	
10:10	5.26	3.72	29.3%				3.29	70	31.1				350	
10:15	5.27	3.74	29.0%				3.35	70	31.1				350	
10:20	5.32	3.75	29.5%				3.31	70	31.1				350	
10:25	5.31	3.80	28.4%				3.34	67	29.8				335	
10:30	5.29	3.72	29.7%				3.26	66	29.3				330	
10:35	5.29	3.71	29.9%				3.27	66	29.3				330	
10:40	5.31	3.73	29.8%				3.31	66	29.3				330	
10:45	5.34	3.72	30.3%				3.33	66	29.3				330	
10:50	5.33	3.74	29.8%				3.37	67	29.8				335	
10:55	5.37	3.76	30.0%				3.35	67	29.8				335	
11:00	5.41	3.83	29.2%				3.37	70	31.1				350	
11:05	5.39	3.90	27.6%				3.37	70	31.1				350	
11:10	5.34	3.88	27.3%				3.41	70	31.1				350	
11:15	6.51	3.88	40.4%				3.48	70	31.1				350	
11:20	7.48	3.86	48.4%				3.49	70	31.1				350	
11:25	7.65	3.86	49.5%				3.38	68	30.2				340	
11:30	6.53	3.86	40.9%	12.00	6.75	43.8%	3.46	70	31.1	TSS Samples			350	
11:35	5.62	3.89	30.8%				3.49	70	31.1				350	
11:40	5.49	3.88	29.3%				3.54	70	31.1				350	
11:45	5.42	3.88	28.4%				3.46	70	31.1				350	
11:50	5.37	3.88	27.7%				3.40	70	31.1				350	
11:55	5.33	3.88	27.2%				3.49	70	31.1				350	
12:00	5.27	3.87	26.6%				3.45	70	31.1				350	
12:05	5.43	3.86	28.9%				3.54	70	31.1				350	
12:10	5.45	3.90	28.4%				3.52	70	31.1				350	
12:15	5.47	3.89	28.9%				3.50	70	31.1				350	
12:20	5.49	3.91	28.8%				3.53	70	31.1				350	
12:25	5.43	3.93	27.6%				3.53	70	31.1				350	

Run #21	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal	Pressure	Flow	Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	**NTU**	**NTU**	**%**	**mg/l**	**mg/l**	**%**	**psi**	**gpm**	**gpm/ft^2**	**Comments**	**%**		**gallons/day**	**gallons/day**	
12:30	5.36	3.91	27.1%				3.53	70	31.1				350		
12:35	5.49	3.91	28.8%				3.54	70	31.1				350		
12:40	5.41	3.91	27.7%				3.53	70	31.1				350		
12:45	5.43	3.93	27.6%				3.54	70	31.1				350		
12:50	5.49	3.95	28.1%				3.53	70	31.1				350		
12:55	5.47	3.96	27.6%				3.61	70	31.1				350		
13:00	5.52	3.97	28.1%	12.00	6.25	47.9%	3.64	70	31.1	TSS Samples			350		
13:05	5.42	3.97	26.8%				3.60	70	31.1				350		
13:10	5.39	3.97	26.3%				3.62	70	31.1				350		
13:15	5.38	3.99	25.8%				3.60	70	31.1				350		
13:20	5.42	3.99	26.4%				3.59	70	31.1				350		
13:25	5.46	4.00	26.7%				3.67	70	31.1				350		
13:30	5.46	4.02	26.4%				3.62	70	31.1				350		
13:35	5.49	4.02	26.8%				3.64	70	31.1				350		
13:40	5.44	3.94	27.6%				3.72	70	31.1				350		
13:45								31	13.8	Wash Timer				155	
13:50								31	13.8					155	**Percent**
13:55								31	13.8					155	**of wash**
14:00								30	13.3					150	**%**
Averages	5.45	3.90	28.4%	11.67	6.17	47.1%		67.65	30.07				23400	615	2.56%

8/27/2018
Monday
20% Compression
Run #22

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
14:05	8.09	4.02	50.3%				3.12	70	31.1	2ml/min JC-1679	20	5.0	350	
14:10	7.36	4.16	43.5%				3.08	70	31.1				350	
14:15	5.37	4.78	11.0%				3.17	70	31.1				350	
14:20	5.32	4.70	11.7%				3.10	71	31.6				355	
14:25	5.31	4.64	12.6%				3.00	71	31.6				355	
14:30	5.39	4.58	15.0%				3.09	71	31.6				355	
14:35	5.39	4.54	15.8%				3.09	70	31.1				350	
14:40	5.33	4.46	16.3%				3.09	71	31.6				355	
14:45	5.16	3.73	27.7%				3.13	70	31.1				350	
14:50	5.22	3.78	27.6%				3.09	70	31.1				350	
14:55	5.27	3.81	27.7%				3.12	70	31.1				350	
15:00	5.22	3.81	27.0%				3.15	70	31.1				350	
15:05	5.22	3.78	27.6%				3.17	70	31.1				350	
15:10	5.25	3.76	28.4%	11.33	6.25	44.8%	3.22	70	31.1	TSS Samples			350	
15:15	5.29	3.73	29.5%				3.23	70	31.1				350	
15:20	5.35	3.72	30.5%				3.40	70	31.1				350	
15:25	5.27	3.70	29.8%				3.13	71	31.6				355	
15:30	5.33	3.68	31.0%				3.18	70	31.1				350	
15:35	5.35	3.67	31.4%				3.19	70	31.1				350	
15:40	5.37	3.66	31.8%				3.21	70	31.1				350	
15:45	5.35	3.66	31.6%				3.14	70	31.1				350	
15:50	5.34	3.63	32.0%				3.23	71	31.6				355	
15:55	5.40	3.61	33.1%				3.25	70	31.1				350	
16:00	5.41	3.70	31.6%				3.32	70	31.1				350	
16:05	5.49	3.76	31.5%				3.36	70	31.1				350	
16:10	5.49	3.72	32.2%				3.33	70	31.1				350	
16:15	5.49	3.70	32.6%				3.30	70	31.1				350	
16:20	5.46	3.68	32.6%				3.24	70	31.1				350	
16:25	5.50	3.67	33.3%				3.41	70	31.1				350	
16:30	5.51	3.66	33.6%				3.37	70	31.1				350	
16:35	5.53	3.64	34.2%				3.43	70	31.1				350	
16:40	5.48	3.61	34.1%				3.36	70	31.1				350	
16:45	5.47	3.60	34.2%				3.25	70	31.1				350	
16:50	5.48	3.56	35.0%				3.27	70	31.1				350	
16:55	5.53	3.54	36.0%				3.36	70	31.1				350	
17:00	5.52	3.60	34.8%				3.38	70	31.1				350	
17:05	5.54	3.69	33.4%				3.30	70	31.1				350	
17:10	5.52	3.70	33.0%				3.41	70	31.1				350	
17:15	5.52	3.71	32.8%				3.36	70	31.1				350	
17:20	5.48	3.69	32.7%				3.43	70	31.1				350	
17:25	5.48	3.68	32.8%				3.54	70	31.1				350	
17:30	5.46	3.68	32.6%				3.41	70	31.1				350	
17:35	5.47	3.65	33.3%				3.45	70	31.1				350	
17:40	5.42	3.62	33.2%				3.43	70	31.1				350	
17:45	5.39	3.58	33.6%				3.44	70	31.1				350	
17:50	5.41	3.56	34.2%				3.43	70	31.1				350	
17:55	5.42	3.54	34.7%				3.49	70	31.1				350	
18:00	5.44	3.54	34.9%				3.46	70	31.1				350	
18:05	5.41	3.53	34.8%				3.54	70	31.1				350	
18:10	5.39	3.53	34.5%				3.47	70	31.1				350	
18:15	5.39	3.54	34.3%				3.46	70	31.1				350	
18:20	5.41	3.68	32.0%				3.49	70	31.1				350	

Run #22	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
18:25	5.36	3.70	31.0%				3.59	69	30.7				345		
18:30	5.39	3.69	31.5%				3.49	70	31.1				350		
18:35	5.37	3.68	31.5%				3.49	70	31.1				350		
18:40	5.39	3.68	31.7%				3.66	70	31.1				350		
18:45	5.43	3.69	32.0%				3.58	70	31.1				350		
18:50	5.41	3.67	32.2%				3.53	70	31.1				350		
18:55	5.40	3.67	32.0%				3.64	70	31.1				350		
19:00	5.37	3.67	31.7%				3.54	69	30.7				345		
19:05	5.35	3.75	29.9%				3.56	70	31.1				350		
19:10	5.29	3.74	29.3%				3.71	70	31.1				350		
19:15	5.29	3.74	29.3%				3.61	70	31.1				350		
19:20	5.31	3.73	29.8%				3.63	69	30.7				345		
19:25	5.33	3.74	29.8%				3.62	69	30.7				345		
19:30	5.35	3.74	30.1%				3.66	69	30.7				345		
19:35	5.33	3.74	29.8%				3.70	69	30.7				345		
19:40	5.27	3.74	29.0%				3.67	69	30.7				345		
19:45	5.29	3.72	29.7%				3.60	70	31.1				350		
19:50	5.31	3.74	29.6%				3.71	69	30.7				345		
19:55	5.27	3.74	29.0%				3.75	69	30.7				345		
20:00	5.29	3.73	29.5%				3.75	69	30.7				345		
20:05	5.32	3.74	29.7%				3.73	69	30.7				345		
20:10								30	13.3	Wash Timer				150	
20:15								30	13.3					150	**Percent**
20:20								30	13.3					150	**of wash**
20:25								30	13.3					150	**%**
Averages	5.45	3.77	30.8%	11.33	6.25	44.8%		67.86	30.16				25525	600	2.30%

8/27/2018 - 8-28-18
Monday - Tuesday
20% Compression
Run #23

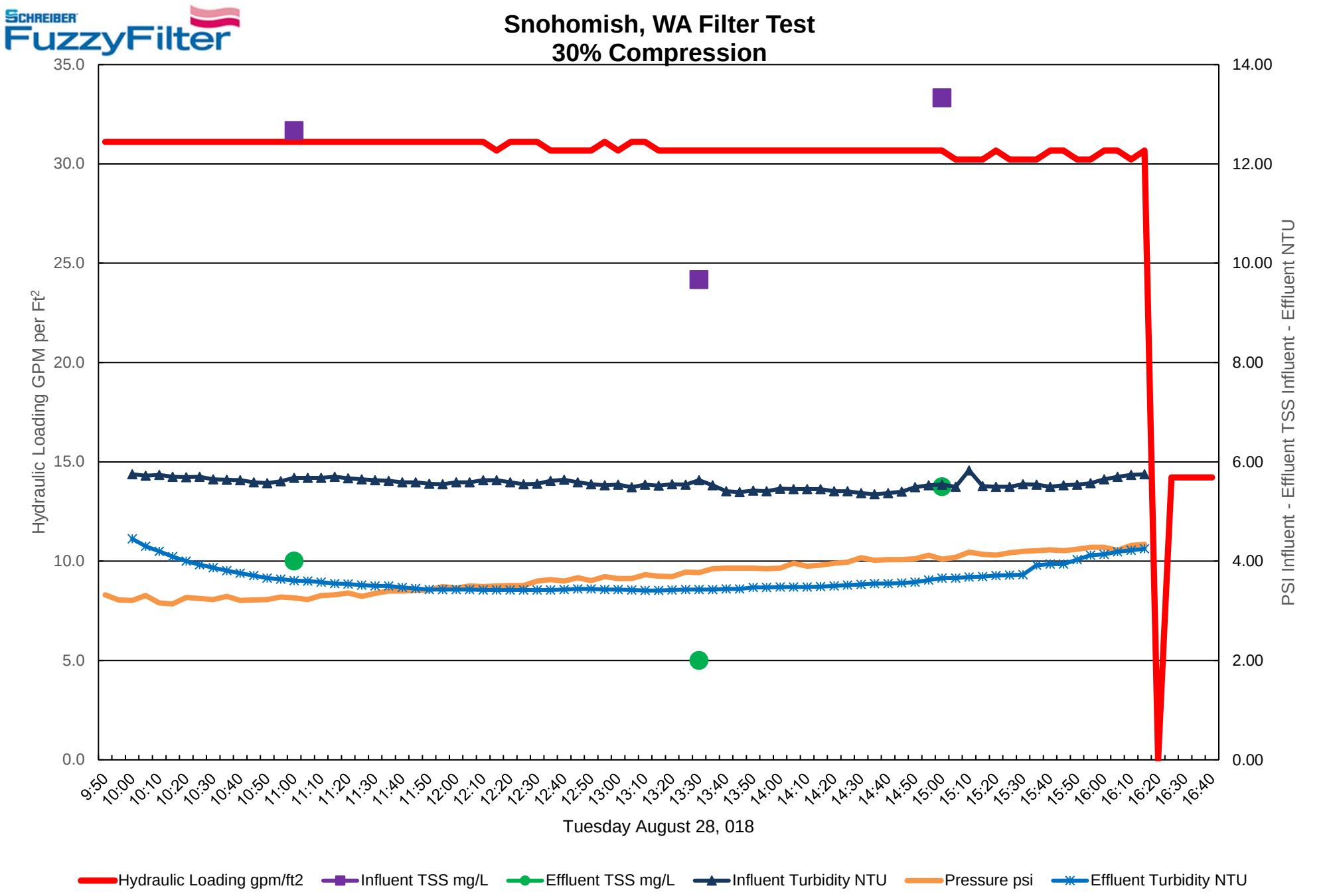
Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
20:30							3.10	70	31.1	2ml/min JC-1679	20	5.0	350	
20:35							3.12	70	31.1				350	
20:40							3.03	70	31.1				350	
20:45	5.15	4.49	12.8%				3.06	70	31.1				350	
20:50	5.15	4.36	15.3%				3.06	70	31.1				350	
20:55	5.21	4.28	17.9%				3.07	71	31.6				355	
21:00	5.22	4.18	19.9%				3.03	70	31.1				350	
21:05	5.23	4.14	20.8%				3.10	70	31.1				350	
21:10	5.31	4.06	23.5%				3.13	70	31.1				350	
21:15	5.41	4.01	25.9%				3.11	70	31.1				350	
21:20	5.97	4.00	33.0%				3.12	70	31.1				350	
21:25	6.55	4.05	38.2%				3.13	70	31.1				350	
21:30	7.25	4.12	43.2%				3.13	70	31.1				350	
21:35	7.52	4.13	45.1%				3.16	70	31.1				350	
21:40	7.37	4.09	44.5%				3.13	70	31.1				350	
21:45	7.23	4.02	44.4%				3.17	70	31.1				350	
21:50	7.29	3.98	45.4%				3.18	70	31.1				350	
21:55	7.57	3.96	47.7%				3.15	70	31.1				350	
22:00	7.66	4.01	47.7%				3.26	70	31.1				350	
22:05	7.61	3.96	48.0%				3.13	70	31.1				350	
22:10	7.52	3.93	47.7%				3.22	70	31.1				350	
22:15	7.41	3.89	47.5%				3.21	70	31.1				350	
22:20	7.21	3.88	46.2%				3.23	70	31.1				350	
22:25	6.91	3.84	44.4%				3.28	70	31.1				350	
22:30	6.79	3.80	44.0%				3.30	70	31.1				350	
22:35	6.65	3.76	43.5%				3.22	70	31.1				350	
22:40	6.57	3.75	42.9%				3.29	70	31.1				350	
22:45	6.53	3.73	42.9%				3.30	70	31.1				350	
22:50	6.48	3.76	42.0%				3.39	70	31.1				350	
22:55	6.40	3.70	42.2%				3.31	69	30.7				345	
23:00	6.31	3.68	41.7%				3.32	70	31.1				350	
23:05	6.17	3.65	40.8%				3.25	69	30.7				345	
23:10	6.14	3.63	40.9%				3.33	69	30.7				345	
23:15	6.12	3.63	40.7%				3.41	70	31.1				350	
23:20	6.10	3.62	40.7%				3.31	70	31.1				350	
23:25	6.06	3.62	40.3%				3.36	70	31.1				350	
23:30	6.07	3.61	40.5%				3.41	70	31.1				350	
23:35	6.04	3.64	39.7%				3.39	70	31.1				350	
23:40	6.03	3.63	39.8%				3.55	69	30.7				345	
23:45	6.01	3.63	39.6%				3.46	70	31.1				350	
23:50	5.92	3.62	38.9%				3.47	69	30.7				345	
23:55	5.92	3.61	39.0%				3.50	69	30.7				345	
0:00	5.93	3.62	39.0%				3.55	69	30.7	8/28/2018			345	
0:05	5.87	3.62	38.3%				3.53	69	30.7				345	
0:10	5.81	3.62	37.7%				3.50	69	30.7				345	
0:15	5.82	3.62	37.8%				3.49	69	30.7				345	
0:20	5.74	3.62	36.9%				3.59	69	30.7				345	
0:25	5.70	3.62	36.5%				3.55	69	30.7				345	
0:30	5.69	3.64	36.0%				3.58	69	30.7				345	
0:35	5.71	3.64	36.3%				3.65	69	30.7				345	
0:40	5.72	3.65	36.2%				3.71	69	30.7				345	
0:45	5.72	3.66	36.0%				3.62	69	30.7				345	

Run #23	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
0:50	5.65	3.68	34.9%				3.72	69	30.7				345		
0:55	5.64	3.68	34.8%				3.63	69	30.7				345		
1:00	5.62	3.70	34.2%				3.62	69	30.7				345		
1:05	5.58	3.70	33.7%				3.67	69	30.7				345		
1:10	5.63	3.71	34.1%				3.72	69	30.7				345		
1:15	5.69	3.72	34.6%				3.68	69	30.7				345		
1:20	5.67	3.76	33.7%				3.75	69	30.7				345		
1:25	5.70	3.76	34.0%				3.70	69	30.7				345		
1:30	5.66	3.78	33.2%				3.63	69	30.7				345		
1:35	5.65	3.79	32.9%				3.73	69	30.7				345		
1:40	5.64	3.81	32.4%				3.75	69	30.7				345		
1:45	5.61	3.82	31.9%				3.69	69	30.7				345		
1:50	5.60	3.83	31.6%				3.76	69	30.7				345		
1:55	5.58	3.83	31.4%				3.73	69	30.7				345		
2:00	5.62	3.85	31.5%				3.83	69	30.7				345		
2:05	5.62	3.86	31.3%				3.84	69	30.7				345		
2:10	5.61	3.93	29.9%				3.82	69	30.7				345		
2:15	5.61	3.99	28.9%				3.82	69	30.7				345		
2:20	5.56	4.01	27.9%				3.88	69	30.7				345		
2:25	5.57	4.02	27.8%				3.93	69	30.7				345		
2:30	5.55	4.07	26.7%				3.82	69	30.7				345		
2:35								28	12.4	Wash Timer				140	
2:40								28	12.4					140	**Percent**
2:45								28	12.4					140	**of wash**
2:50								28	12.4					140	**%**
Averages	6.08	3.83	37.0%					67.35	29.93				25370	560	2.16%

8/28/2018
Tuesday
20% Compression
Run #24

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
2:55							3.09	70	31.1	2ml/min JC-1679	20	5.0	350	
3:00							3.07	71	31.6				355	
3:05							3.01	70	31.1				350	
3:10	5.46	4.42	19.0%				3.06	70	31.1				350	
3:15	5.44	4.30	21.0%				3.08	71	31.6				355	
3:20	5.41	4.20	22.4%				3.10	70	31.1				350	
3:25	5.42	4.10	24.4%				3.10	70	31.1				350	
3:30	5.41	4.02	25.7%				3.16	70	31.1				350	
3:35	5.39	3.95	26.7%				3.11	70	31.1				350	
3:40	5.40	3.89	28.0%				3.08	70	31.1				350	
3:45	5.41	3.84	29.0%				3.09	70	31.1				350	
3:50	5.41	3.78	30.1%				3.10	70	31.1				350	
3:55	5.45	3.76	31.0%				3.15	70	31.1				350	
4:00	5.45	3.72	31.7%				3.17	70	31.1				350	
4:05	5.43	3.67	32.4%				3.15	70	31.1				350	
4:10	5.42	3.63	33.0%				3.13	70	31.1				350	
4:15	5.41	3.66	32.3%				3.21	70	31.1				350	
4:20	5.42	3.64	32.8%				3.10	70	31.1				350	
4:25	5.42	3.61	33.4%				3.17	70	31.1				350	
4:30	5.37	3.58	33.3%				3.21	70	31.1				350	
4:35	5.40	3.56	34.1%				3.20	70	31.1				350	
4:40	5.39	3.54	34.3%				3.23	70	31.1				350	
4:45	5.40	3.53	34.6%				3.21	70	31.1				350	
4:50	5.42	3.52	35.1%				3.27	70	31.1				350	
4:55	5.41	3.50	35.3%				3.24	70	31.1				350	
5:00	5.45	3.52	35.4%				3.22	70	31.1				350	
5:05	5.46	3.52	35.5%				3.39	70	31.1				350	
5:10	5.43	3.50	35.5%				3.32	70	31.1				350	
5:15	5.45	3.50	35.8%				3.29	70	31.1				350	
5:20	5.46	3.50	35.9%				3.27	70	31.1				350	
5:25	5.43	3.50	35.5%				3.33	70	31.1				350	
5:30	5.45	3.50	35.8%				3.41	70	31.1				350	
5:35	5.46	3.50	35.9%				3.38	70	31.1				350	
5:40	5.45	3.50	35.8%				3.35	70	31.1				350	
5:45	5.46	3.50	35.9%				3.46	70	31.1				350	
5:50	5.54	3.46	37.5%				3.42	70	31.1				350	
5:55	5.58	3.43	38.5%				3.52	70	31.1				350	
6:00	5.58	3.42	38.7%				3.46	70	31.1				350	
6:05	5.52	3.43	37.9%				3.40	70	31.1				350	
6:10	5.53	3.43	38.0%				3.52	70	31.1				350	
6:15	5.46	3.43	37.2%				3.46	70	31.1				350	
6:20	5.46	3.40	37.7%				3.49	70	31.1				350	
6:25	5.46	3.40	37.7%				3.50	70	31.1				350	
6:30	5.44	3.40	37.5%				3.49	70	31.1				350	
6:35	5.41	3.40	37.2%				3.61	70	31.1				350	
6:40	5.35	3.40	36.4%				3.53	70	31.1				350	
6:45	5.36	3.41	36.4%				3.60	70	31.1				350	
6:50	5.38	3.42	36.4%				3.59	69	30.7				345	
6:55	5.37	3.42	36.3%				3.57	69	30.7				345	
7:00	5.35	3.42	36.1%				3.58	69	30.7				345	
7:05	5.32	3.42	35.7%				3.63	69	30.7				345	
7:10	5.34	3.43	35.8%				3.65	69	30.7				345	

Run #24	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
7:15	5.33	3.43	35.6%				3.62	70	31.1				350		
7:20	5.29	3.44	35.0%				3.60	69	30.7				345		
7:25	5.29	3.45	34.8%				3.64	70	31.1				350		
7:30	5.34	3.46	35.2%				3.64	70	31.1				350		
7:35	5.33	3.47	34.9%				3.66	69	30.7				345		
7:40	5.31	3.47	34.7%				3.68	69	30.7				345		
7:45	5.29	3.47	34.4%				3.71	69	30.7				345		
7:50	5.31	3.47	34.7%				3.73	70	31.1				350		
7:55	5.47	3.51	35.8%				3.69	69	30.7				345		
8:00	5.43	3.53	35.0%				3.67	69	30.7				345		
8:05	5.39	3.53	34.5%				3.76	69	30.7				345		
8:10	5.38	3.53	34.4%				3.73	69	30.7				345		
8:15	5.41	3.55	34.4%				3.75	69	30.7				345		
8:20	5.39	3.58	33.6%				3.73	69	30.7				345		
8:25	5.43	3.58	34.1%				3.70	70	31.1				350		
8:30	5.39	3.59	33.4%	11.33	6.00	47.0%	3.78	69	30.7	TSS Samples			345		
8:35	5.35	3.60	32.7%				3.82	69	30.7				345		
8:40	5.33	3.62	32.1%				3.85	69	30.7				345		
8:45	5.31	3.63	31.6%				3.85	69	30.7				345		
8:50	5.30	3.65	31.1%				3.92	69	30.7				345		
8:55	5.30	3.66	30.9%				3.96	69	30.7				345		
9:00	5.34	3.67	31.3%				3.84	69	30.7				345		
9:05	5.31	3.68	30.7%				3.88	69	30.7				345		
9:10	5.33	3.69	30.8%				3.87	69	30.7				345		
9:15	5.34	3.71	30.5%				3.92	69	30.7				345		
9:20	5.39	3.73	30.8%				3.91	69	30.7				345		
9:25								0		Wash Timer				0	
9:30								30	13.3					150	
9:35								30	13.3					150	**Percent**
9:40								29	12.9					145	**of wash**
9:45								29	12.9					145	**%**
Averages	5.40	3.59	33.5%	11.33	6.00	47.0%		66.92	30.10				27180	590	2.12%


Snohomish, WA Filter Test
30% Compression
Hydraulic Loading GPM per Ft2
PSI Influent - Effluent TSS Influent - Effluent NTU
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
9:50
10:00
10:10
10:20
10:30
10:40
10:50
11:00
11:10
11:20
11:30
11:40
11:50
12:00
12:10
12:20
12:30
12:40
12:50
13:00
13:10
13:20
13:30
13:40
13:50
14:00
14:10
14:20
14:30
14:40
14:50
15:00
15:10
15:20
15:30
15:40
15:50
16:00
16:10
16:20
16:30
16:40
Tuesday August 28, 018
Hydraulic Loading gpm/ft2
Influent TSS mg/L
Effluent TSS mg/L
Influent Turbidity NTU
Pressure psi
Effluent Turbidity NTU

8/28/2018
Tuesday
30% Compression
Run #25

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
9:50							3.32	70	31.1	Feeding 1679 at 4ml/min	30	5.0	350	
9:55							3.22	70	31.1				350	
10:00	5.75	4.45	22.6%				3.21	70	31.1				350	
10:05	5.72	4.30	24.8%				3.31	70	31.1				350	
10:10	5.74	4.20	26.8%				3.16	70	31.1				350	
10:15	5.70	4.09	28.2%				3.14	70	31.1				350	
10:20	5.69	4.00	29.7%				3.27	70	31.1				350	
10:25	5.70	3.93	31.1%				3.25	70	31.1				350	
10:30	5.65	3.87	31.5%				3.23	70	31.1				350	
10:35	5.64	3.81	32.4%				3.29	70	31.1				350	
10:40	5.63	3.76	33.2%				3.21	70	31.1				350	
10:45	5.59	3.71	33.6%				3.22	70	31.1				350	
10:50	5.57	3.66	34.3%				3.23	70	31.1				350	
10:55	5.61	3.64	35.1%				3.28	70	31.1				350	
11:00	5.68	3.61	36.4%	12.67	4.00	68.4%	3.26	70	31.1	TSS Samples			350	
11:05	5.68	3.60	36.6%				3.23	70	31.1				350	
11:10	5.68	3.58	37.0%				3.31	70	31.1				350	
11:15	5.70	3.55	37.7%				3.32	70	31.1				350	
11:20	5.67	3.54	37.6%				3.36	70	31.1				350	
11:25	5.65	3.52	37.7%				3.29	70	31.1				350	
11:30	5.63	3.50	37.8%				3.35	70	31.1				350	
11:35	5.62	3.50	37.7%				3.40	70	31.1				350	
11:40	5.59	3.47	37.9%				3.40	70	31.1				350	
11:45	5.59	3.45	38.3%				3.41	70	31.1				350	
11:50	5.56	3.43	38.3%				3.42	70	31.1				350	
11:55	5.55	3.43	38.2%				3.49	70	31.1				350	
12:00	5.59	3.43	38.6%				3.46	70	31.1				350	
12:05	5.59	3.43	38.6%				3.50	70	31.1				350	
12:10	5.63	3.42	39.3%				3.49	70	31.1				350	
12:15	5.63	3.42	39.3%				3.50	69	30.7				345	
12:20	5.59	3.42	38.8%				3.51	70	31.1				350	
12:25	5.55	3.42	38.4%				3.51	70	31.1				350	
12:30	5.56	3.42	38.5%				3.60	70	31.1				350	
12:35	5.62	3.42	39.1%				3.63	69	30.7				345	
12:40	5.64	3.43	39.2%				3.60	69	30.7				345	
12:45	5.59	3.44	38.5%				3.67	69	30.7				345	
12:50	5.55	3.44	38.0%				3.61	69	30.7				345	
12:55	5.53	3.43	38.0%				3.69	70	31.1				350	
13:00	5.54	3.43	38.1%				3.65	69	30.7				345	
13:05	5.49	3.42	37.7%				3.65	70	31.1				350	
13:10	5.54	3.41	38.4%				3.73	70	31.1				350	
13:15	5.52	3.41	38.2%				3.70	69	30.7				345	
13:20	5.55	3.42	38.4%				3.69	69	30.7				345	
13:25	5.54	3.43	38.1%				3.78	69	30.7				345	
13:30	5.63	3.43	39.1%	9.67	2.00	79.3%	3.77	69	30.7	TSS Samples			345	
13:35	5.53	3.43	38.0%				3.85	69	30.7				345	
13:40	5.41	3.44	36.4%				3.86	69	30.7				345	
13:45	5.39	3.44	36.2%				3.86	69	30.7				345	
13:50	5.42	3.47	36.0%				3.86	69	30.7				345	
13:55	5.41	3.47	35.9%				3.85	69	30.7				345	
14:00	5.46	3.48	36.3%				3.86	69	30.7				345	
14:05	5.45	3.48	36.1%				3.96	69	30.7				345	

Run #25	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading			Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate			Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments		%		gallons/day	gallons/day	
14:10	5.45	3.48	36.1%				3.90	69	30.7					345		
14:15	5.45	3.49	36.0%				3.92	69	30.7					345		
14:20	5.41	3.50	35.3%				3.96	69	30.7					345		
14:25	5.41	3.52	34.9%				3.98	69	30.7					345		
14:30	5.37	3.53	34.3%				4.07	69	30.7					345		
14:35	5.35	3.55	33.6%				4.02	69	30.7					345		
14:40	5.37	3.55	33.9%				4.03	69	30.7					345		
14:45	5.40	3.56	34.1%				4.03	69	30.7					345		
14:50	5.49	3.58	34.8%				4.05	69	30.7					345		
14:55	5.53	3.62	34.5%				4.12	69	30.7					345		
15:00	5.54	3.66	33.9%	13.33	5.50	58.7%	4.04	69	30.7	TSS Samples				345		
15:05	5.50	3.66	33.5%				4.08	68	30.2					340		
15:10	5.83	3.68	36.9%				4.18	68	30.2					340		
15:15	5.51	3.69	33.0%				4.14	68	30.2					340		
15:20	5.50	3.71	32.5%				4.12	69	30.7					345		
15:25	5.50	3.72	32.4%				4.17	68	30.2					340		
15:30	5.55	3.73	32.8%				4.20	68	30.2					340		
15:35	5.54	3.92	29.2%				4.21	68	30.2					340		
15:40	5.50	3.94	28.4%				4.23	69	30.7					345		
15:45	5.53	3.94	28.8%				4.21	69	30.7					345		
15:50	5.54	4.03	27.3%				4.24	68	30.2					340		
15:55	5.57	4.12	26.0%				4.28	68	30.2					340		
16:00	5.65	4.14	26.7%				4.28	69	30.7					345		
16:05	5.70	4.19	26.5%				4.22	69	30.7					345		
16:10	5.74	4.22	26.5%				4.32	68	30.2					340		
16:15	5.75	4.25	26.1%				4.34	69	30.7					345		
16:20								0		Wash Timer					0	
16:25								32	14.2						160	
16:30								32	14.2						160	**Percent**
16:35								32	14.2						160	**of wash**
16:40								32	14.2						160	**%**
Averages	5.57	3.64	34.6%	11.89	3.83	67.8%		66.70	30.01					27040	640	2.31%

8/28/2018
Tuesday
30% Compression
Run #26

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
16:45							3.18	70	31.1	4ml/min JC-1679	30	5.2	350	
16:50							3.10	70	31.1				350	
16:55	5.76	4.47	22.4%				3.30	70	31.1				350	
17:00	5.75	4.32	24.9%				3.20	70	31.1				350	
17:05	5.70	4.22	26.0%				3.28	70	31.1				350	
17:10	5.67	4.12	27.3%				3.25	70	31.1				350	
17:15	5.60	4.03	28.0%				3.21	70	31.1				350	
17:20	5.59	3.98	28.8%				3.33	70	31.1				350	
17:25	5.57	3.92	29.6%				3.35	70	31.1				350	
17:30	5.55	3.85	30.6%				3.26	70	31.1				350	
17:35	5.60	3.85	31.3%				3.27	70	31.1				350	
17:40	5.61	3.82	31.9%				3.34	70	31.1				350	
17:45	5.64	3.76	33.3%				3.28	70	31.1				350	
17:50	5.62	3.72	33.8%				3.26	70	31.1				350	
17:55	5.59	3.67	34.3%				3.29	70	31.1				350	
18:00	5.57	3.61	35.2%				3.40	70	31.1				350	
18:05	5.61	3.56	36.5%				3.39	70	31.1				350	
18:10	5.66	3.56	37.1%				3.32	70	31.1				350	
18:15	5.67	3.54	37.6%				3.41	70	31.1				350	
18:20	5.69	3.52	38.1%				3.41	70	31.1				350	
18:25	5.67	3.50	38.3%				3.41	70	31.1				350	
18:30	5.59	3.49	37.6%				3.44	70	31.1				350	
18:35	5.55	3.52	36.6%				3.48	70	31.1				350	
18:40	5.55	3.45	37.8%				3.59	70	31.1				350	
18:45	5.55	3.45	37.8%				3.53	70	31.1				350	
18:50	5.59	3.45	38.3%				3.45	70	31.1				350	
18:55	5.59	3.44	38.5%				3.56	69	30.7				345	
19:00	5.55	3.43	38.2%				3.53	70	31.1				350	
19:05	5.54	3.42	38.3%				3.49	70	31.1				350	
19:10	5.55	3.42	38.4%				3.45	70	31.1				350	
19:15	5.59	3.42	38.8%				3.50	70	31.1				350	
19:20	5.55	3.42	38.4%				3.51	69	30.7				345	
19:25	5.60	3.42	38.9%				3.61	70	31.1				350	
19:30	5.57	3.43	38.4%				3.53	69	30.7				345	
19:35	5.55	3.43	38.2%				3.59	69	30.7				345	
19:40	5.52	3.43	37.9%				3.65	69	30.7				345	
19:45	5.47	3.44	37.1%				3.60	69	30.7				345	
19:50	5.51	3.42	37.9%				3.63	69	30.7				345	
19:55	5.52	3.42	38.0%				3.62	69	30.7				345	
20:00	5.50	3.42	37.8%				3.72	69	30.7				345	
20:05	5.50	3.42	37.8%				3.69	69	30.7				345	
20:10	5.49	3.42	37.7%				3.77	69	30.7				345	
20:15	5.50	3.42	37.8%				3.67	69	30.7				345	
20:20	5.47	3.43	37.3%				3.82	69	30.7				345	
20:25	5.55	3.44	38.0%				3.78	69	30.7				345	
20:30	5.53	3.46	37.4%				3.77	69	30.7				345	
20:35	5.47	3.48	36.4%				3.77	69	30.7				345	
20:40	5.47	3.49	36.2%				3.79	69	30.7				345	
20:45	5.49	3.51	36.1%				3.84	69	30.7				345	
20:50	5.50	3.54	35.6%				3.92	69	30.7				345	
20:55	5.47	3.53	35.5%				3.89	69	30.7				345	
21:00	5.44	3.55	34.7%				3.93	69	30.7				345	

Run #26	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate			Percent Compression	Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments		%		gallons/day	gallons/day	
21:05	5.45	3.57	34.5%				3.87	69	30.7					345		
21:10	5.49	3.58	34.8%				3.95	69	30.7					345		
21:15	5.55	3.61	35.0%				3.99	69	30.7					345		
21:20	5.52	3.63	34.2%				4.10	69	30.7					345		
21:25	5.50	3.64	33.8%				4.04	68	30.2					340		
21:30	5.47	3.66	33.1%				3.96	69	30.7					345		
21:35	5.46	3.68	32.6%				3.99	69	30.7					345		
21:40	5.45	3.70	32.1%				4.11	69	30.7					345		
21:45	5.43	3.73	31.3%				4.07	69	30.7					345		
21:50	5.41	3.75	30.7%				4.14	68	30.2					340		
21:55	5.39	3.85	28.6%				4.23	69	30.7					345		
22:00	5.41	3.89	28.1%				4.12	68	30.2					340		
22:05	5.41	3.90	27.9%				4.09	68	30.2					340		
22:10	5.38	3.93	27.0%				4.16	69	30.7					345		
22:15	5.37	3.95	26.4%				4.24	68	30.2					340		
22:20	5.41	3.99	26.2%				4.27	68	30.2					340		
22:25	5.42	4.02	25.8%				4.16	68	30.2					340		
22:30	5.41	3.98	26.4%				4.23	68	30.2					340		
22:35	5.39	3.99	26.0%				4.19	68	30.2					340		
22:40	5.40	4.02	25.6%				4.25	68	30.2					340		
22:45	5.45	4.02	26.2%				4.31	68	30.2					340		
22:50	5.47	4.02	26.5%				4.30	68	30.2					340		
22:55	5.49	3.99	27.3%				4.27	68	30.2					340		
23:00	5.47	4.01	26.7%				4.29	68	30.2					340		
23:05	5.41	4.02	25.7%				4.34	68	30.2					340		
23:10	5.53	4.06	26.6%				4.36	68	30.2					340		
23:15								0		Wash Timer					0	
23:20								32	14.2						160	
23:25								31	13.8						155	**Percent**
23:30								32	14.2						160	**of wash**
23:35								31	13.8						155	**%**
Averages	5.53	3.69	33.3%					66.54	29.93					26985	630	2.28%

8/29/2018
Wednesday
30% Compression
Run #27

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
23:40							3.16	70	31.1	4ml/min JC-1679	30	5.2	350	
23:45							3.16	70	31.1				350	
23:50	5.47	4.14	24.3%				3.12	70	31.1				350	
23:55	5.50	4.08	25.8%				3.21	70	31.1				350	
0:00	5.55	3.99	28.1%				3.16	70	31.1	8/29/2018			350	
0:05	5.53	3.91	29.3%				3.21	70	31.1				350	
0:10	5.51	3.83	30.5%				3.19	70	31.1				350	
0:15	5.52	3.78	31.5%				3.24	70	31.1				350	
0:20	5.46	3.72	31.9%				3.23	70	31.1				350	
0:25	5.46	3.68	32.6%				3.23	70	31.1				350	
0:30	5.41	3.61	33.3%				3.30	70	31.1				350	
0:35	5.44	3.59	34.0%				3.27	70	31.1				350	
0:40	5.49	3.55	35.3%				3.24	70	31.1				350	
0:45	5.46	3.50	35.9%				3.40	70	31.1				350	
0:50	5.45	3.50	35.8%				3.25	70	31.1				350	
0:55	5.50	3.48	36.7%				3.39	70	31.1				350	
1:00	5.49	3.45	37.2%				3.42	70	31.1				350	
1:05	5.44	3.43	36.9%				3.33	70	31.1				350	
1:10	5.41	3.45	36.2%				3.39	69	30.7				345	
1:15	5.43	3.44	36.6%				3.44	70	31.1				350	
1:20	5.44	3.42	37.1%				3.46	69	30.7				345	
1:25	5.45	3.42	37.2%				3.48	69	30.7				345	
1:30	5.45	3.40	37.6%				3.49	69	30.7				345	
1:35	5.46	3.40	37.7%				3.53	69	30.7				345	
1:40	5.50	3.40	38.2%				3.54	69	30.7				345	
1:45	5.52	3.40	38.4%				3.59	69	30.7				345	
1:50	5.51	3.38	38.7%				3.58	69	30.7				345	
1:55	5.52	3.38	38.8%				3.66	69	30.7				345	
2:00	5.50	3.38	38.5%				3.57	69	30.7				345	
2:05	5.52	3.38	38.8%				3.69	69	30.7				345	
2:10	5.49	3.37	38.6%				3.65	69	30.7				345	
2:15	5.49	3.37	38.6%				3.79	69	30.7				345	
2:20	5.52	3.37	38.9%				3.69	69	30.7				345	
2:25	5.53	3.38	38.9%				3.83	69	30.7				345	
2:30	5.50	3.38	38.5%				3.86	69	30.7				345	
2:35	5.50	3.38	38.5%				3.79	69	30.7				345	
2:40	5.49	3.38	38.4%				3.83	69	30.7				345	
2:45	5.55	3.41	38.6%				3.86	69	30.7				345	
2:50	5.50	3.41	38.0%				3.91	69	30.7				345	
2:55	5.49	3.42	37.7%				3.89	68	30.2				340	
3:00	5.50	3.42	37.8%				3.93	68	30.2				340	
3:05	5.55	3.45	37.8%				3.94	69	30.7				345	
3:10	5.57	3.46	37.9%				3.99	68	30.2				340	
3:15	5.55	3.48	37.3%				3.99	69	30.7				345	
3:20	5.51	3.52	36.1%				3.97	69	30.7				345	
3:25	5.52	3.52	36.2%				4.04	69	30.7				345	
3:30	5.52	3.53	36.1%				4.05	69	30.7				345	
3:35	5.59	3.55	36.5%				4.06	68	30.2				340	
3:40	5.57	3.58	35.7%				4.00	68	30.2				340	
3:45	5.62	3.58	36.3%				4.09	69	30.7				345	
3:50	5.63	3.60	36.1%				4.13	68	30.2				340	
3:55	5.61	3.61	35.7%				4.11	68	30.2				340	

Run #27	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading			Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate			Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments		%		gallons/day	gallons/day	
4:00	5.65	3.63	35.8%				4.13	68	30.2					340		
4:05	5.65	3.65	35.4%				4.19	68	30.2					340		
4:10	5.64	3.65	35.3%				4.17	68	30.2					340		
4:15	5.60	3.67	34.5%				4.24	68	30.2					340		
4:20	5.59	3.67	34.3%				4.16	68	30.2					340		
4:25	5.55	3.68	33.7%				4.24	68	30.2					340		
4:30	5.54	3.69	33.4%				4.20	68	30.2					340		
4:35	5.50	3.70	32.7%				4.24	68	30.2					340		
4:40	5.52	3.72	32.6%				4.26	68	30.2					340		
4:45	5.51	3.71	32.7%				4.25	68	30.2					340		
4:50	5.47	3.72	32.0%				4.28	68	30.2					340		
4:55	5.52	3.73	32.4%				4.38	68	30.2					340		
5:00	5.53	3.74	32.4%				4.31	68	30.2					340		
5:05	5.49	3.74	31.9%				4.33	68	30.2					340		
5:10	5.55	3.75	32.4%				4.34	68	30.2					340		
5:15	5.57	3.78	32.1%				4.36	68	30.2					340		
5:20	5.49	3.79	31.0%				4.38	68	30.2					340		
5:25	5.47	3.81	30.3%				4.38	68	30.2					340		
5:30	5.52	3.84	30.4%				4.32	68	30.2					340		
5:35	5.57	3.92	29.6%				4.41	68	30.2					340		
5:40	5.61	3.96	29.4%				4.39	68	30.2					340		
5:45	5.69	4.00	29.7%				4.38	68	30.2					340		
5:50	5.70	4.04	29.1%				4.43	68	30.2					340		
5:55	5.73	4.08	28.8%				4.43	68	30.2					340		
6:00	5.71	4.11	28.0%				4.42	68	30.2					340		
6:05	5.66	4.08	27.9%				4.43	68	30.2					340		
6:10								32	14.2	Wash Timer					160	
6:15								32	14.2						160	**Percent**
6:20								31	13.8						155	**of wash**
6:25								32	14.2						160	**%**
Averages	5.53	3.62	34.5%					67.01	29.78					26840	635	2.31%

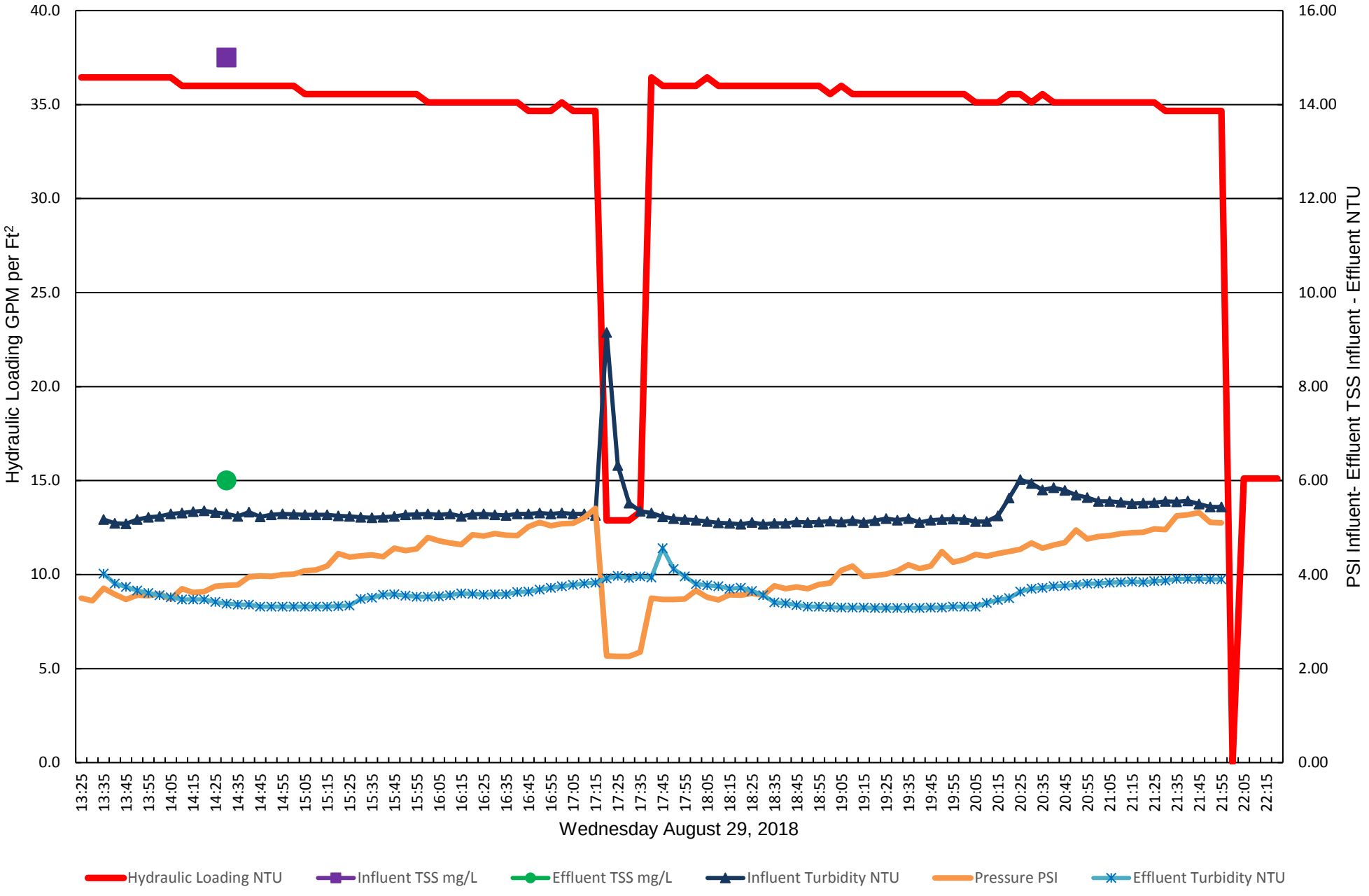
8/29/2018
Wednesday
30% Comprerssion
Run #28

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
6:30							3.21	70	31.1	4ml/min JC-1679	30	5.2	350	
6:35							3.11	70	31.1				350	
6:40	5.48	4.20	23.4%				3.12	70	31.1				350	
6:45	5.53	4.06	26.6%				3.16	70	31.1				350	
6:50	5.47	3.96	27.6%				3.21	70	31.1				350	
6:55	5.41	3.87	28.5%				3.19	70	31.1				350	
7:00	5.42	3.79	30.1%				3.25	70	31.1				350	
7:05	5.42	3.75	30.8%				3.23	70	31.1				350	
7:10	5.43	3.68	32.2%				3.26	70	31.1				350	
7:15	5.42	3.63	33.0%				3.21	70	31.1				350	
7:20	5.42	3.58	33.9%				3.21	70	31.1				350	
7:25	5.41	3.54	34.6%				3.29	70	31.1				350	
7:30	5.39	3.48	35.4%				3.26	69	30.7				345	
7:35	5.40	3.46	35.9%				3.27	70	31.1				350	
7:40	5.47	3.44	37.1%				3.30	70	31.1				350	
7:45	5.46	3.42	37.4%				3.18	70	31.1				350	
7:50	5.41	3.40	37.2%				3.29	70	31.1				350	
7:55	5.45	3.41	37.4%				3.35	69	30.7				345	
8:00	5.44	3.36	38.2%				3.42	70	31.1				350	
8:05	5.48	3.34	39.1%				3.41	69	30.7				345	
8:10	6.07	3.34	45.0%				3.46	69	30.7				345	
8:15	5.57	3.35	39.9%				3.41	69	30.7				345	
8:20	5.46	3.36	38.5%				3.53	70	31.1				350	
8:25	5.47	3.31	39.5%				3.52	69	30.7				345	
8:30	5.47	3.34	38.9%	13.00	4.25	67.3%	3.46	68	30.2	TSS Samples			340	
8:35	5.47	3.31	39.5%				3.40	70	31.1				350	
8:40	5.47	3.33	39.1%				3.47	69	30.7				345	
8:45	5.44	3.30	39.3%				3.51	69	30.7				345	
8:50	5.46	3.29	39.7%				3.55	69	30.7				345	
8:55	5.43	3.28	39.6%				3.57	69	30.7				345	
9:00	5.43	3.28	39.6%				3.56	69	30.7				345	
9:05	5.41	3.28	39.4%				3.54	69	30.7				345	
9:10	5.38	3.28	39.0%				3.60	69	30.7				345	
9:15	5.34	3.28	38.6%				3.66	69	30.7				345	
9:20	5.42	3.28	39.5%				3.60	69	30.7				345	
9:25	5.35	3.29	38.5%				3.68	69	30.7				345	
9:30	5.34	3.30	38.2%				3.70	69	30.7				345	
9:35	5.36	3.30	38.4%				3.70	69	30.7				345	
9:40	5.39	3.29	39.0%				3.70	69	30.7				345	
9:45	5.45	3.32	39.1%				3.77	69	30.7				345	
9:50	5.43	3.33	38.7%				3.77	69	30.7				345	
9:55	5.39	3.34	38.0%				3.80	69	30.7				345	
10:00	5.41	3.34	38.3%	13.33	5.50	58.7%	3.76	69	30.7	TSS Samples			345	
10:05	5.37	3.34	37.8%				3.85	69	30.7				345	
10:10	5.39	3.34	38.0%				3.82	69	30.7				345	
10:15	5.42	3.37	37.8%				3.85	69	30.7				345	
10:20	5.44	3.38	37.9%				3.93	69	30.7				345	
10:25	5.41	3.40	37.2%				3.98	68	30.2				340	
10:30	5.43	3.40	37.4%				3.91	68	30.2				340	
10:35	5.47	3.41	37.7%				3.94	69	30.7				345	
10:40	5.44	3.45	36.6%				3.94	69	30.7				345	
10:45	5.47	3.45	36.9%				3.94	69	30.7				345	

Run #28	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading		Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate		Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
10:50	5.53	3.48	37.1%				4.14	80	35.6	Increased Flow and JC-1679 Feed rate to 6ml/min			400		
10:55	5.70	4.63	18.8%				4.06	81	36.0				405		
11:00	5.62	4.09	27.2%				4.34	80	35.6				400		
11:05	5.55	3.88	30.1%				4.18	81	36.0				405		
11:10	5.55	3.81	31.4%				4.22	80	35.6				400		
11:15	5.61	3.78	32.6%				4.35	80	35.6				400		
11:20	5.56	3.76	32.4%				4.40	80	35.6				400		
11:25	5.50	3.75	31.8%				4.32	80	35.6				400		
11:30	5.49	3.75	31.7%	11.00	7.50	31.8%	4.35	80	35.6	TSS Samples			400		
11:35	5.45	3.77	30.8%				4.37	80	35.6				400		
11:40	5.44	3.80	30.1%				4.52	80	35.6				400		
11:45	5.49	3.82	30.4%				4.38	80	35.6				400		
11:50	5.47	3.86	29.4%				4.46	80	35.6				400		
11:55	5.46	3.92	28.2%				4.66	80	35.6				400		
12:00	5.46	3.93	28.0%				4.49	80	35.6				400		
12:05	5.41	3.96	26.8%				4.50	79	35.1				395		
12:10	5.43	4.02	26.0%				4.47	80	35.6				400		
12:15	5.47	4.00	26.9%				4.59	79	35.1				395		
12:20	5.49	4.01	27.0%				4.51	79	35.1				395		
12:25	5.47	4.02	26.5%				4.61	80	35.6				400		
12:30	5.53	4.04	26.9%				4.84	79	35.1				395		
12:35	5.63	4.01	28.8%				4.63	79	35.1				395		
12:40	5.55	4.01	27.7%				4.82	79	35.1				395		
12:45	5.62	4.04	28.1%				4.69	79	35.1				395		
12:50	5.57	4.04	27.5%				4.79	79	35.1				395		
12:55	5.59	4.06	27.4%				4.75	79	35.1				395		
13:00	5.62	4.10	27.0%	12.67	6.50	48.7%	4.79	79	35.1	TSS Samples			395		
13:05								32	14.2	Wash Timer				160	
13:10								33	14.7				165	165	**Percent**
13:15								32	14.2				160	160	**of wash**
13:20								31	13.8				155	155	**%**
Averages	5.47	3.62	33.9%	12.50	5.94	52.5%		70.89	31.51				29260	640	2.14%

SCHREIBER FuzzyFilter

Snohomish, WA Filter Test
40% Compression



8/29/2018
Wednesday
40% Compression

Run #29 Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
13:25							3.50	82	36.4	6ml/min JC-1679	40	5.5	410	
13:30							3.44	82	36.4				410	
13:35	5.17	4.02	22.2%				3.71	82	36.4				410	
13:40	5.09	3.81	25.1%				3.58	82	36.4				410	
13:45	5.08	3.74	26.4%				3.47	82	36.4				410	
13:50	5.17	3.66	29.2%				3.56	82	36.4				410	
13:55	5.22	3.61	30.8%				3.55	82	36.4				410	
14:00	5.24	3.56	32.1%				3.59	82	36.4				410	
14:05	5.29	3.52	33.5%				3.48	82	36.4				410	
14:10	5.31	3.47	34.7%				3.70	81	36.0				405	
14:15	5.34	3.47	35.0%				3.62	81	36.0				405	
14:20	5.36	3.47	35.3%				3.64	81	36.0				405	
14:25	5.32	3.42	35.7%				3.75	81	36.0				405	
14:30	5.29	3.38	36.1%	15.00	6.00	60.0%	3.77	81	36.0				405	
14:35	5.24	3.36	35.9%				3.78	81	36.0				405	
14:40	5.33	3.36	37.0%				3.95	81	36.0				405	
14:45	5.23	3.32	36.5%				3.97	81	36.0				405	
14:50	5.27	3.32	37.0%				3.96	81	36.0				405	
14:55	5.29	3.32	37.2%				4.00	81	36.0				405	
15:00	5.28	3.32	37.1%				4.01	81	36.0				405	
15:05	5.27	3.32	37.0%				4.08	80	35.6				400	
15:10	5.27	3.32	37.0%				4.10	80	35.6				400	
15:15	5.27	3.32	37.0%				4.18	80	35.6				400	
15:20	5.25	3.33	36.6%				4.45	80	35.6				400	
15:25	5.24	3.34	36.3%				4.37	80	35.6				400	
15:30	5.22	3.48	33.3%				4.40	80	35.6				400	
15:35	5.21	3.51	32.6%				4.42	80	35.6				400	
15:40	5.22	3.57	31.6%				4.38	80	35.6				400	
15:45	5.24	3.58	31.7%				4.56	80	35.6				400	
15:50	5.27	3.55	32.6%				4.51	80	35.6				400	
15:55	5.28	3.53	33.1%				4.55	80	35.6				400	
16:00	5.29	3.53	33.3%				4.79	79	35.1				395	
16:05	5.27	3.54	32.8%				4.72	79	35.1				395	
16:10	5.29	3.56	32.7%				4.67	79	35.1				395	
16:15	5.24	3.60	31.3%				4.64	79	35.1				395	
16:20	5.28	3.59	32.0%				4.85	79	35.1				395	
16:25	5.29	3.57	32.5%				4.82	79	35.1				395	
16:30	5.27	3.58	32.1%				4.87	79	35.1				395	
16:35	5.26	3.58	31.9%				4.84	79	35.1				395	
16:40	5.29	3.63	31.4%				4.83	79	35.1				395	
16:45	5.29	3.64	31.2%				5.02	78	34.7				390	
16:50	5.31	3.68	30.7%				5.11	78	34.7				390	
16:55	5.29	3.72	29.7%				5.04	78	34.7				390	
17:00	5.31	3.75	29.4%				5.08	79	35.1				395	
17:05	5.29	3.78	28.5%				5.09	78	34.7				390	
17:10	5.29	3.81	28.0%				5.21	78	34.7				390	
17:15	5.26	3.83	27.2%				5.41	78	34.7				390	
17:20	9.15	3.92	57.2%				2.27	29	12.9				145	
17:25	6.32	3.97	37.2%				2.26	29	12.9				145	
17:30	5.52	3.93	28.8%				2.26	29	12.9				145	
17:35	5.35	3.96	26.0%				2.35	30	13.3				150	
17:40	5.31	3.94	25.8%				3.50	82	36.4				410	

Run #29	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading			Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate			Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments		%		gallons/day	gallons/day	
17:45	5.23	4.56	12.8%				3.47	81	36.0					405		
17:50	5.19	4.12	20.6%				3.47	81	36.0					405		
17:55	5.17	3.96	23.4%				3.48	81	36.0					405		
18:00	5.16	3.80	26.4%				3.66	81	36.0					405		
18:05	5.13	3.77	26.5%				3.52	82	36.4					410		
18:10	5.10	3.75	26.5%				3.46	81	36.0					405		
18:15	5.09	3.70	27.3%				3.57	81	36.0					405		
18:20	5.07	3.72	26.6%				3.56	81	36.0					405		
18:25	5.11	3.65	28.6%				3.60	81	36.0					405		
18:30	5.07	3.56	29.8%				3.56	81	36.0					405		
18:35	5.09	3.41	33.0%				3.76	81	36.0					405		
18:40	5.09	3.39	33.4%				3.70	81	36.0					405		
18:45	5.12	3.35	34.6%				3.74	81	36.0					405		
18:50	5.11	3.32	35.0%				3.70	81	36.0					405		
18:55	5.12	3.32	35.2%				3.79	81	36.0					405		
19:00	5.14	3.31	35.6%				3.82	80	35.6					400		
19:05	5.12	3.30	35.5%				4.09	81	36.0					405		
19:10	5.15	3.30	35.9%				4.18	80	35.6					400		
19:15	5.11	3.30	35.4%				3.96	80	35.6					400		
19:20	5.15	3.29	36.1%				3.98	80	35.6					400		
19:25	5.19	3.29	36.6%				4.01	80	35.6					400		
19:30	5.16	3.29	36.2%				4.08	80	35.6					400		
19:35	5.19	3.29	36.6%				4.21	80	35.6					400		
19:40	5.11	3.29	35.6%				4.13	80	35.6					400		
19:45	5.16	3.30	36.0%				4.18	80	35.6					400		
19:50	5.17	3.30	36.2%				4.49	80	35.6					400		
19:55	5.18	3.32	35.9%				4.26	80	35.6					400		
20:00	5.17	3.32	35.8%				4.32	80	35.6					400		
20:05	5.13	3.32	35.3%				4.43	79	35.1					395		
20:10	5.13	3.40	33.7%				4.39	79	35.1					395		
20:15	5.25	3.46	34.1%				4.45	79	35.1					395		
20:20	5.63	3.50	37.8%				4.49	80	35.6					400		
20:25	6.02	3.64	39.5%				4.54	80	35.6					400		
20:30	5.94	3.70	37.7%				4.67	79	35.1					395		
20:35	5.80	3.72	35.9%				4.56	80	35.6					400		
20:40	5.85	3.75	35.9%				4.63	79	35.1					395		
20:45	5.79	3.76	35.1%				4.68	79	35.1					395		
20:50	5.69	3.78	33.6%				4.95	79	35.1					395		
20:55	5.64	3.81	32.4%				4.76	79	35.1					395		
21:00	5.56	3.81	31.5%				4.81	79	35.1					395		
21:05	5.56	3.83	31.1%				4.83	79	35.1					395		
21:10	5.54	3.84	30.7%				4.87	79	35.1					395		
21:15	5.51	3.85	30.1%				4.89	79	35.1					395		
21:20	5.52	3.84	30.4%				4.90	79	35.1					395		
21:25	5.53	3.86	30.2%				4.97	79	35.1					395		
21:30	5.56	3.87	30.4%				4.96	78	34.7					390		
21:35	5.55	3.91	29.5%				5.25	78	34.7					390		
21:40	5.57	3.91	29.8%				5.27	78	34.7					390		
21:45	5.50	3.91	28.9%				5.32	78	34.7					390		
21:50	5.44	3.90	28.3%				5.11	78	34.7					390		
21:55	5.44	3.90	28.3%				5.10	78	34.7					390		
22:00								0		Wash Timer					0	
22:05								34	15.1						170	
22:10								34	15.1						170	**Percent**
22:15								34	15.1						170	**of wash**
22:20								34	15.1						170	**%**
Averages	5.35	3.60	32.7%	15.00	6.00	60.0%		75.68	33.95					40185	680	1.66%

8-29-18 - 8-30-18
Wednesday - Thursday
40% Compression
Run #30

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
22:25							3.74	81	36.0	6ml/min JC-1679	40	5.1	405		
22:30							3.47	81	36.0				405		
22:35	5.45	3.86	29.2%				3.43	82	36.4				410		
22:40	5.44	3.76	30.9%				3.47	82	36.4				410		
22:45	5.46	3.68	32.6%				3.47	81	36.0				405		
22:50	5.44	3.62	33.5%				3.49	81	36.0				405		
22:55	5.36	3.54	34.0%				3.79	81	36.0				405		
23:00	5.37	3.47	35.4%				3.62	81	36.0				405		
23:05	5.37	3.43	36.1%				3.49	81	36.0				405		
23:10	5.34	3.39	36.5%				3.86	81	36.0				405		
23:15	5.34	3.37	36.9%				3.62	81	36.0				405		
23:20	5.34	3.35	37.3%				3.67	81	36.0				405		
23:25	5.36	3.32	38.1%				3.67	81	36.0				405		
23:30	5.37	3.32	38.2%				3.68	81	36.0				405		
23:35	5.41	3.30	39.0%				3.78	81	36.0				405		
23:40	5.45	3.30	39.4%				3.89	81	36.0				405		
23:45	5.44	3.30	39.3%				3.90	81	36.0				405		
23:50	5.49	3.29	40.1%				3.85	81	36.0				405		
23:55	5.47	3.29	39.9%				3.96	80	35.6				400		
0:00	5.47	3.30	39.7%				4.00	80	35.6	8/30/2018			400		
0:05	5.49	3.30	39.9%				4.06	80	35.6				400		
0:10	5.48	3.30	39.8%				4.07	80	35.6				400		
0:15	5.45	3.30	39.4%				4.17	80	35.6				400		
0:20	5.45	3.30	39.4%				4.44	80	35.6				400		
0:25	5.43	3.30	39.2%				4.26	80	35.6				400		
0:30	5.39	3.31	38.6%				4.30	80	35.6				400		
0:35	5.43	3.33	38.7%				4.24	80	35.6				400		
0:40	5.39	3.33	38.2%				4.33	80	35.6				400		
0:45	5.36	3.34	37.7%				4.40	80	35.6				400		
0:50	5.33	3.34	37.3%				4.48	80	35.6				400		
0:55	5.32	3.36	36.8%				4.51	79	35.1				395		
1:00	5.33	3.38	36.6%				4.51	79	35.1				395		
1:05	5.33	3.38	36.6%				4.60	79	35.1				395		
1:10	5.31	3.40	36.0%				4.67	79	35.1				395		
1:15	5.35	3.41	36.3%				4.69	79	35.1				395		
1:20	5.42	3.45	36.3%				4.70	79	35.1				395		
1:25	5.45	3.48	36.1%				4.94	79	35.1				395		
1:30	5.42	3.49	35.6%				4.80	79	35.1				395		
1:35	5.43	3.52	35.2%				4.88	79	35.1				395		
1:40	5.42	3.60	33.6%				4.90	79	35.1				395		
1:45	5.44	3.60	33.8%				4.98	78	34.7				390		
1:50	5.42	3.60	33.6%				5.15	79	35.1				395		
1:55	5.42	3.61	33.4%				5.21	78	34.7				390		
2:00	5.39	3.63	32.7%				4.96	79	35.1				395		
2:05	5.37	3.66	31.8%				5.07	79	35.1				395		
2:10	5.39	3.69	31.5%				5.08	78	34.7				390		
2:15	5.41	3.70	31.6%				5.13	79	35.1				395		
2:20	5.45	3.74	31.4%				5.14	79	35.1				395		
2:25								0		Wash Pressure				0	
2:30								32	14.2					160	
2:35								32	14.2					160	**Percent**
2:40								32	14.2					160	**of wash**

Run #30	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading			Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate			Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments		%		gallons/day	gallons/day	
2:45								32	14.2						160	%
Averages	5.41	3.45	36.2%					74.85	33.91					19195	640	3.23%

8/30/2018
Thursday
40% Compression
Run #31

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
2:50	5.55	3.94	29.0%				3.54	81	36.0	6ml/min JC-1679	40	5.1	405	
2:55	5.45	3.98	27.0%				3.42	82	36.4				410	
3:00	5.42	3.79	30.1%				3.44	82	36.4				410	
3:05	5.35	3.68	31.2%				3.49	82	36.4				410	
3:10	5.35	3.60	32.7%				3.46	81	36.0				405	
3:15	5.36	3.52	34.3%				3.48	82	36.4				410	
3:20	5.34	3.46	35.2%				3.54	82	36.4				410	
3:25	5.36	3.40	36.6%				3.53	81	36.0				405	
3:30	5.31	3.37	36.5%				3.58	81	36.0				405	
3:35	5.34	3.32	37.8%				3.69	82	36.4				410	
3:40	5.28	3.29	37.7%				3.65	81	36.0				405	
3:45	5.27	3.28	37.8%				3.72	81	36.0				405	
3:50	5.27	3.25	38.3%				3.82	81	36.0				405	
3:55	5.22	3.24	37.9%				3.79	81	36.0				405	
4:00	5.26	3.24	38.4%				4.07	81	36.0				405	
4:05	5.25	3.23	38.5%				4.18	81	36.0				405	
4:10	5.22	3.22	38.3%				4.18	80	35.6				400	
4:15	5.27	3.22	38.9%				4.04	80	35.6				400	
4:20	5.32	3.22	39.5%				4.08	80	35.6				400	
4:25	5.33	3.22	39.6%				4.12	80	35.6				400	
4:30	5.34	3.22	39.7%				4.31	80	35.6				400	
4:35	5.32	3.24	39.1%				4.16	80	35.6				400	
4:40	5.33	3.24	39.2%				4.47	80	35.6				400	
4:45	5.33	3.25	39.0%				4.35	80	35.6				400	
4:50	5.35	3.26	39.1%				4.40	80	35.6				400	
4:55	5.35	3.26	39.1%				4.45	80	35.6				400	
5:00	5.39	3.27	39.3%				4.40	80	35.6				400	
5:05	5.39	3.29	39.0%				4.52	80	35.6				400	
5:10	5.40	3.30	38.9%				4.55	80	35.6				400	
5:15	5.46	3.34	38.8%				4.66	80	35.6				400	
5:20	5.37	3.36	37.4%				4.75	79	35.1				395	
5:25	5.37	3.37	37.2%				4.65	79	35.1				395	
5:30	5.31	3.38	36.3%				4.78	79	35.1				395	
5:35	5.34	3.38	36.7%				4.78	80	35.6				400	
5:40	5.33	3.40	36.2%				4.89	79	35.1				395	
5:45	5.27	3.43	34.9%				4.73	79	35.1				395	
5:50	5.27	3.45	34.5%				5.03	79	35.1				395	
5:55	5.29	3.47	34.4%				4.89	79	35.1				395	
6:00	5.30	3.50	34.0%				4.80	79	35.1				395	
6:05	5.37	3.51	34.6%				5.08	79	35.1				395	
6:10	5.36	3.53	34.1%				5.05	79	35.1				395	
6:15	5.39	3.58	33.6%				5.12	78	34.7				390	
6:20	5.36	3.59	33.0%				5.04	79	35.1				395	
6:25	5.37	3.61	32.8%				4.99	79	35.1				395	
6:30	5.40	3.62	33.0%				5.25	79	35.1				395	
6:35	5.39	3.70	31.4%				5.03	79	35.1				395	
6:40	5.39	3.70	31.4%				4.95	79	35.1				395	
6:45	5.37	3.71	30.9%				4.98	79	35.1				395	
6:50	5.27	3.71	29.6%				5.04	79	35.1				395	
6:55	5.24	3.71	29.2%				5.07	79	35.1				395	
7:00	5.24	3.71	29.2%				5.22	79	35.1				395	
7:05	5.22	3.71	28.9%				5.28	79	35.1				395	

Run #31	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
7:10	5.21	3.74	28.2%				5.01	79	35.1				395		
7:15	5.23	3.75	28.3%				5.06	79	35.1				395		
7:20	5.21	3.74	28.2%				5.21	79	35.1				395		
7:25	5.23	3.76	28.1%				5.02	79	35.1				395		
7:30	5.33	3.81	28.5%				5.07	78	34.7				390		
7:35	5.31	3.83	27.9%				5.07	79	35.1				395		
7:40	5.34	3.88	27.3%				5.18	78	34.7				390		
7:45	5.37	3.90	27.4%				4.60	62	27.6	Reduce flow and lower the JC-1679 to 4ml/min			310		
7:50	5.82	3.44	40.9%				4.81	68	30.2				340		
7:55	5.46	3.52	35.5%				4.92	68	30.2				340		
8:00	5.39	3.63	32.7%				4.85	68	30.2				340		
8:05	5.41	3.68	32.0%				4.81	68	30.2				340		
8:10	5.36	3.72	30.6%	11.00	6.00	45.5%	4.91	67	29.8	TSS Samples			335		
8:15	5.33	3.73	30.0%				4.88	67	29.8				335		
8:20	5.27	3.76	28.7%				4.87	68	30.2				340		
8:25	5.21	3.79	27.3%				5.02	67	29.8				335		
8:30	5.22	3.79	27.4%				4.97	67	29.8				335		
8:35	5.27	3.81	27.7%				4.91	67	29.8				335		
8:40	5.25	3.86	26.5%				4.82	67	29.8	Visitors			335		
8:45								29	12.9	Manual Wash				145	
8:50								29	12.9					145	**Percent**
8:55								29	12.9					145	**of wash**
9:00								30	13.3					150	**%**
Averages	5.33	3.53	33.9%	11.00	6.00	45.5%		75.09	33.37				27575	585	2.08%

8/30/2018
Thursday
30% Compression
Run #32

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
9:05							3.18	70	31.1	4ml/min JC-1679	30	5.1	350	
9:10							3.13	70	31.1				350	
9:15							3.08	70	31.1				350	
9:20	5.12	3.92	23.4%				3.22	70	31.1				350	
9:25	5.13	3.82	25.5%				3.09	70	31.1				350	
9:30	5.16	3.76	27.1%				3.18	70	31.1				350	
9:35	5.21	3.68	29.4%				3.16	70	31.1				350	
9:40	5.19	3.61	30.4%				3.08	70	31.1				350	
9:45	5.19	3.56	31.4%				3.25	70	31.1				350	
9:50	5.21	3.52	32.4%				3.11	70	31.1				350	
9:55	5.24	3.48	33.6%				3.18	70	31.1				350	
10:00	5.19	3.42	34.1%				3.22	70	31.1				350	
10:05	5.22	3.40	34.9%				3.25	70	31.1				350	
10:10	5.27	3.37	36.1%	12.33	6.00	51.3%	3.19	70	31.1	TSS Samples			350	
10:15	5.23	3.34	36.1%				3.31	70	31.1				350	
10:20	5.24	3.32	36.6%				3.28	70	31.1				350	
10:25	5.25	3.29	37.3%				3.26	70	31.1				350	
10:30	5.26	3.28	37.6%				3.32	70	31.1				350	
10:35	5.21	3.26	37.4%				3.29	70	31.1				350	
10:40	5.22	3.24	37.9%				3.37	70	31.1				350	
10:45	5.18	3.23	37.6%				3.32	70	31.1				350	
10:50	5.19	3.20	38.3%				3.32	70	31.1				350	
10:55	5.21	3.20	38.6%				3.31	70	31.1				350	
11:00	5.21	3.19	38.8%				3.31	70	31.1				350	
11:05	5.24	3.20	38.9%				3.41	70	31.1				350	
11:10	5.26	3.18	39.5%				3.36	70	31.1				350	
11:15	5.26	3.18	39.5%				3.36	69	30.7				345	
11:20	5.22	3.19	38.9%				3.40	70	31.1				350	
11:25	5.22	3.18	39.1%				3.49	70	31.1				350	
11:30	5.21	3.18	39.0%				3.50	70	31.1				350	
11:35	5.20	3.16	39.2%				3.43	70	31.1				350	
11:40	5.16	3.16	38.8%	12.33	4.00	67.6%	3.51	69	30.7	TSS Samples			345	
11:45	5.19	3.16	39.1%				3.53	70	31.1				350	
11:50	5.19	3.16	39.1%				3.53	70	31.1				350	
11:55	5.18	3.16	39.0%				3.57	70	31.1				350	
12:00	5.22	2.24	57.1%				3.64	69	30.7				345	
12:05	5.06	2.63	48.0%				3.67	69	30.7				345	
12:10	5.11	2.86	44.0%				3.68	69	30.7				345	
12:15	5.16	2.93	43.2%				3.72	69	30.7				345	
12:20	5.15	2.98	42.1%				3.74	69	30.7				345	
12:25	5.19	3.01	42.0%				3.67	69	30.7				345	
12:30	5.17	3.04	41.2%				3.74	70	31.1				350	
12:35	5.14	3.14	38.9%				3.78	69	30.7				345	
12:40	5.11	3.12	38.9%				3.77	69	30.7				345	
12:45	5.11	3.12	38.9%				3.79	69	30.7				345	
12:50	5.14	3.14	38.9%				3.85	69	30.7				345	
12:55	5.15	3.14	39.0%				3.84	69	30.7				345	
13:00	5.19	3.16	39.1%				3.79	69	30.7				345	
13:05	5.21	3.19	38.8%				3.89	69	30.7				345	
13:10	5.21	3.22	38.2%				3.89	69	30.7				345	
13:15	5.14	3.24	37.0%				3.89	69	30.7				345	
13:20	5.14	3.24	37.0%				3.88	69	30.7				345	

Run #32	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
13:25	5.11	3.28	35.8%				3.92	69	30.7				345		
13:30	5.11	3.26	36.2%				4.00	69	30.7				345		
13:35	5.13	3.26	36.5%				3.97	69	30.7				345		
13:40	5.17	3.29	36.4%				3.99	69	30.7				345		
13:45	5.16	3.29	36.2%				3.95	69	30.7				345		
13:50	5.18	3.32	35.9%				4.01	69	30.7				345		
13:55	5.17	3.34	35.4%				4.02	69	30.7				345		
14:00	5.22	3.36	35.6%				4.02	69	30.7				345		
14:05	5.25	3.39	35.4%				3.94	69	30.7				345		
14:10	5.18	3.41	34.2%				4.06	69	30.7				345		
14:15	5.14	3.42	33.5%				4.09	69	30.7				345		
14:20	5.13	3.42	33.3%				4.18	69	30.7				345		
14:25	5.12	3.44	32.8%				4.07	69	30.7				345		
14:30	5.12	3.45	32.6%				4.12	69	30.7				345		
14:35	5.15	3.45	33.0%				4.15	69	30.7				345		
14:40								30	13.3	Wash Timer				150	
14:45								29	12.9					145	**Percent**
14:50								29	12.9					145	**of wash**
14:55								29	12.9					145	**%**
Averages	5.18	3.26	37.0%	12.33	5.00	59.4%		67.24	29.88				23285	585	2.45%

8/30/2018
Thursday
30% Compression
Run #33

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
15:00							3.18	71	31.6		30	5.2	355	
15:05							3.12	70	31.1				350	
15:10	5.16	3.98	22.9%				3.09	70	31.1				350	
15:15	5.11	3.85	24.7%				3.11	70	31.1				350	
15:20	5.06	3.75	25.9%				3.17	70	31.1				350	
15:25	5.36	3.94	26.5%				3.19	70	31.1				350	
15:30	5.17	4.04	21.9%				3.18	71	31.6				355	
15:35	5.19	4.10	21.0%				3.14	70	31.1	?? 1679 Feed??			350	
										not turn off until 8-31-18 3:40pm				
15:40	5.17	4.14	19.9%				3.29	70	31.1	per plant personal			350	
15:45	5.15	4.16	19.2%				3.17	70	31.1				350	
15:50	5.13	4.16	18.9%				3.16	70	31.1				350	
15:55	5.13	4.24	17.3%				3.20	70	31.1				350	
16:00	5.12	4.20	18.0%				3.21	70	31.1				350	
16:05	5.12	4.17	18.6%				3.14	70	31.1				350	
16:10	5.11	4.17	18.4%				3.21	70	31.1				350	
16:15	5.07	4.26	16.0%				3.22	70	31.1				350	
16:20	5.09	4.22	17.1%				3.22	70	31.1				350	
16:25	5.05	4.22	16.4%				3.31	70	31.1				350	
16:30	5.05	4.22	16.4%				3.16	70	31.1				350	
16:35	5.07	4.23	16.6%				3.25	70	31.1				350	
16:40	5.12	4.23	17.4%				3.32	70	31.1				350	
16:45	5.12	4.22	17.6%				3.32	70	31.1				350	
16:50	5.18	4.20	18.9%				3.26	70	31.1				350	
16:55	5.23	4.22	19.3%				3.22	70	31.1				350	
17:00	5.22	4.20	19.5%				3.20	70	31.1				350	
17:05	5.23	4.20	19.7%				3.34	70	31.1				350	
17:10	5.25	4.24	19.2%				3.32	69	30.7				345	
17:15	5.22	4.22	19.2%				3.32	70	31.1				350	
17:20	5.26	4.26	19.0%				3.27	69	30.7				345	
17:25	5.26	4.28	18.6%				3.28	70	31.1				350	
17:30	5.23	4.27	18.4%				3.33	70	31.1				350	
17:35	5.21	4.29	17.7%				3.29	70	31.1				350	
17:40	5.24	4.28	18.3%				3.36	70	31.1				350	
17:45	5.26	4.33	17.7%				3.42	69	30.7				345	
17:50	5.25	4.30	18.1%				3.37	69	30.7				345	
17:55	5.24	4.26	18.7%				3.33	70	31.1				350	
18:00	5.23	4.25	18.7%				3.42	70	31.1				350	
18:05	5.24	4.25	18.9%				3.42	70	31.1				350	
18:10	5.25	4.27	18.7%				3.33	70	31.1				350	
18:15	5.24	4.23	19.3%				3.46	69	30.7				345	
18:20	5.21	4.19	19.6%				3.41	69	30.7				345	
18:25	5.19	4.19	19.3%				3.53	69	30.7				345	
18:30	5.17	4.25	17.8%				3.49	69	30.7				345	
18:35	5.17	4.28	17.2%				3.51	69	30.7				345	
18:40	5.16	4.29	16.9%				3.49	69	30.7				345	
18:45	5.11	4.26	16.6%				3.45	69	30.7				345	
18:50	5.11	4.25	16.8%				3.47	69	30.7				345	
18:55	5.13	4.24	17.3%				3.46	69	30.7				345	
19:00	5.10	4.22	17.3%				3.49	69	30.7				345	
19:05	5.12	4.22	17.6%				3.50	69	30.7				345	
19:10	5.09	4.19	17.7%				3.50	69	30.7				345	

Run #33	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
19:15	5.15	4.19	18.6%				3.65	69	30.7				345		
19:20	5.20	4.18	19.6%				3.59	69	30.7				345		
19:25	5.16	4.21	18.4%				3.57	69	30.7				345		
19:30	5.11	4.17	18.4%				3.60	69	30.7				345		
19:35	5.14	4.17	18.9%				3.61	69	30.7				345		
19:40	5.16	4.16	19.4%				3.63	69	30.7				345		
19:45	5.20	4.17	19.8%				3.56	69	30.7				345		
19:50	5.19	4.16	19.8%				3.66	69	30.7				345		
19:55	5.09	4.16	18.3%				3.66	69	30.7				345		
20:00	5.04	4.12	18.3%				3.64	69	30.7				345		
20:05	5.04	4.12	18.3%				3.67	69	30.7				345		
20:10	5.06	4.12	18.6%				3.70	69	30.7				345		
20:15	5.07	4.13	18.5%				3.65	69	30.7				345		
20:20	5.04	4.12	18.3%				3.73	69	30.7				345		
20:25	5.08	4.13	18.7%				3.74	69	30.7				345		
20:30	5.09	4.13	18.9%				3.74	69	30.7				345		
20:35								31	13.8	Wash Timer				155	
20:40								31	13.8					155	**Percent**
20:45								31	13.8					155	**of wash**
20:50								31	13.8					155	**%**
Averages	5.16	4.19	18.8%					67.38	29.95				23300	620	2.59%

8/3/2018
Thursday
30% compression
Run #34

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
20:55							3.17	70	31.1		30	5.2	350	
21:00							3.08	70	31.1				350	
21:05	5.11	4.29	16.0%				3.23	70	31.1				350	
21:10	5.08	4.29	15.6%				3.11	70	31.1				350	
21:15	5.09	4.29	15.7%				3.15	70	31.1				350	
21:20	5.07	4.29	15.4%				3.14	70	31.1				350	
21:25	5.09	4.29	15.7%				3.25	70	31.1				350	
21:30	5.12	4.26	16.8%				3.15	70	31.1				350	
21:35	5.09	4.23	16.9%				3.19	70	31.1				350	
21:40	5.09	4.22	17.1%				3.17	69	30.7				345	
21:45	5.11	4.20	17.8%				3.27	70	31.1				350	
21:50	5.01	4.19	16.4%				3.19	69	30.7				345	
21:55	5.05	4.14	18.0%				3.31	70	31.1				350	
22:00	5.07	4.12	18.7%				3.31	70	31.1				350	
22:05	5.05	4.13	18.2%				3.19	70	31.1				350	
22:10	5.10	4.11	19.4%				3.26	70	31.1				350	
22:15	5.10	4.12	19.2%				3.29	69	30.7				345	
22:20	5.09	4.08	19.8%				3.26	69	30.7				345	
22:25	5.09	4.08	19.8%				3.20	70	31.1				350	
22:30	5.09	4.10	19.4%				3.26	69	30.7				345	
22:35	5.11	4.15	18.8%				3.27	70	31.1				350	
22:40	5.09	4.09	19.6%				3.29	69	30.7				345	
22:45	5.11	4.06	20.5%				3.22	69	30.7				345	
22:50	5.13	4.07	20.7%				3.28	69	30.7				345	
22:55	5.14	4.08	20.6%				3.32	69	30.7				345	
23:00	5.11	4.06	20.5%				3.45	69	30.7				345	
23:05	5.07	4.05	20.1%				3.35	69	30.7				345	
23:10	5.02	4.03	19.7%				3.34	68	30.2				340	
23:15	5.06	4.09	19.2%				3.37	69	30.7				345	
23:20	5.01	4.06	19.0%				3.32	69	30.7				345	
23:25	5.03	4.05	19.5%				3.42	68	30.2				340	
23:30	5.06	4.00	20.9%				3.37	69	30.7				345	
23:35	5.02	3.97	20.9%				3.41	68	30.2				340	
23:40	5.01	3.96	21.0%				3.41	68	30.2				340	
23:45	5.03	3.96	21.3%				3.44	69	30.7				345	
23:50	5.06	3.94	22.1%				3.44	68	30.2				340	
23:55	5.07	3.97	21.7%				3.44	68	30.2				340	
0:00	5.04	3.96	21.4%				3.49	68	30.2	8/31/2018			340	
0:05	5.06	3.95	21.9%				3.50	68	30.2				340	
0:10	5.06	3.95	21.9%				3.44	68	30.2				340	
0:15	5.06	3.94	22.1%				3.46	69	30.7				345	
0:20	5.01	3.94	21.4%				3.50	68	30.2				340	
0:25	4.97	3.95	20.5%				3.52	68	30.2				340	
0:30	5.02	3.92	21.9%				3.43	68	30.2				340	
0:35	5.00	3.94	21.2%				3.54	68	30.2				340	
0:40	5.02	3.93	21.7%				3.63	68	30.2				340	
0:45	4.99	3.93	21.2%				3.56	68	30.2				340	
0:50	5.02	3.93	21.7%				3.64	68	30.2				340	
0:55	4.99	3.92	21.4%				3.66	68	30.2				340	
1:00	4.98	3.91	21.5%				3.63	68	30.2				340	
1:05	4.99	3.91	21.6%				3.62	68	30.2				340	
1:10	5.02	3.91	22.1%				3.70	68	30.2				340	

Run #34	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate			Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	Percent Compression %		gallons/day	gallons/day	
1:15	4.99	3.91	21.6%				3.77	68	30.2				340		
1:20	5.01	3.93	21.6%				3.67	68	30.2				340		
1:25	5.01	3.93	21.6%				3.65	68	30.2				340		
1:30	5.01	3.93	21.6%				3.74	68	30.2				340		
1:35	4.99	3.91	21.6%				3.79	68	30.2				340		
1:40	5.02	3.92	21.9%				3.82	68	30.2				340		
1:45	5.02	3.92	21.9%				3.84	68	30.2				340		
1:50	5.02	3.94	21.5%				3.75	68	30.2				340		
1:55	4.98	3.95	20.7%				3.77	68	30.2				340		
2:00	4.99	3.94	21.0%				3.84	68	30.2				340		
2:05	4.98	3.93	21.1%				3.74	68	30.2				340		
2:10	5.02	3.93	21.7%				3.77	68	30.2				340		
2:15	5.06	3.95	21.9%				3.83	68	30.2				340		
2:20	5.21	3.96	24.0%				3.85	68	30.2				340		
2:25	5.08	3.96	22.0%				3.85	68	30.2				340		
2:30								31	13.8	Wash Timer				155	
2:35								31	13.8					155	**Percent**
2:40								31	13.8					155	**of wash**
2:45								31	13.8					155	**%**
Averages	5.05	4.03	20.2%					66.59	29.60				23020	620	2.62%

8/31/2018
Friday
30% Compression
Run #35

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
2:50							3.17	69	30.7		30	5.2	345	
2:55							3.22	69	30.7				345	
3:00	5.19	4.24	18.3%				3.10	69	30.7				345	
3:05	5.15	4.23	17.9%				3.13	69	30.7				345	
3:10	5.12	4.24	17.2%				3.18	69	30.7				345	
3:15	5.12	4.22	17.6%				3.17	69	30.7				345	
3:20	5.17	4.22	18.4%				3.07	69	30.7				345	
3:25	5.16	4.20	18.6%				3.23	69	30.7				345	
3:30	5.21	4.20	19.4%				3.15	69	30.7				345	
3:35	5.20	4.25	18.3%				3.17	69	30.7				345	
3:40	5.22	4.22	19.2%				3.23	69	30.7				345	
3:45	5.22	4.19	19.7%				3.20	69	30.7				345	
3:50	5.19	4.22	18.7%				3.16	69	30.7				345	
3:55	5.21	4.17	20.0%				3.27	69	30.7				345	
4:00	5.20	4.14	20.4%				3.20	69	30.7				345	
4:05	5.21	4.09	21.5%				3.26	68	30.2				340	
4:10	5.21	4.06	22.1%				3.25	69	30.7				345	
4:15	5.21	4.03	22.6%				3.33	69	30.7				345	
4:20	5.16	4.02	22.1%				3.27	69	30.7				345	
4:25	5.16	4.04	21.7%				3.24	69	30.7				345	
4:30	5.17	4.01	22.4%				3.29	69	30.7				345	
4:35	5.19	3.99	23.1%				3.29	68	30.2				340	
4:40	5.16	3.99	22.7%				3.37	69	30.7				345	
4:45	5.11	3.99	21.9%				3.27	69	30.7				345	
4:50	5.09	3.96	22.2%				3.35	69	30.7				345	
4:55	5.04	3.96	21.4%				3.34	69	30.7				345	
5:00	5.06	3.94	22.1%				3.44	68	30.2				340	
5:05	5.10	3.94	22.7%				3.37	69	30.7				345	
5:10	5.14	3.94	23.3%				3.35	68	30.2				340	
5:15	5.11	3.94	22.9%				3.31	68	30.2				340	
5:20	5.12	3.94	23.0%				3.41	68	30.2				340	
5:25	5.11	3.94	22.9%				3.40	68	30.2				340	
5:30	5.10	3.93	22.9%				3.48	68	30.2				340	
5:35	5.15	3.94	23.5%				3.44	68	30.2				340	
5:40	5.25	3.97	24.4%				3.52	68	30.2				340	
5:45	5.29	3.99	24.6%				3.45	68	30.2				340	
5:50	5.40	4.00	25.9%				3.48	68	30.2				340	
5:55	5.33	4.01	24.8%				3.45	68	30.2				340	
6:00	5.28	4.01	24.1%				3.58	68	30.2				340	
6:05	5.19	3.99	23.1%				3.49	68	30.2				340	
6:10	5.22	3.99	23.6%				3.62	68	30.2				340	
6:15	5.29	3.99	24.6%				3.54	68	30.2				340	
6:20	5.31	4.02	24.3%				3.61	68	30.2				340	
6:25	5.34	4.03	24.5%				3.58	68	30.2				340	
6:30	5.31	4.04	23.9%				3.61	68	30.2				340	
6:35	5.35	4.03	24.7%				3.73	68	30.2				340	
6:40	5.35	4.04	24.5%				3.70	68	30.2				340	
6:45	5.29	4.04	23.6%				3.67	68	30.2				340	
6:50	5.29	4.04	23.6%				3.61	68	30.2				340	
6:55	5.28	4.03	23.7%				3.70	68	30.2				340	
7:00	5.24	4.03	23.1%				3.65	68	30.2				340	
7:05	5.24	4.02	23.3%				3.67	68	30.2				340	

Run #35	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	**NTU**	**NTU**	**%**	**mg/l**	**mg/l**	**%**	**Pressure psi**	**Flow gpm**	**gpm/ft^2**	**Comments**	**%**		**gallons/day**	**gallons/day**	
7:10	5.17	4.03	22.1%				3.64	68	30.2				340		
7:15	5.17	4.02	22.2%				3.77	68	30.2				340		
7:20	5.12	4.01	21.7%				3.80	68	30.2				340		
7:25	5.14	4.01	22.0%				3.82	68	30.2				340		
7:30	5.09	4.01	21.2%				3.86	67	29.8				335		
7:35	5.09	4.01	21.2%				3.72	68	30.2				340		
7:40	5.07	3.99	21.3%				3.81	68	30.2				340		
7:45	5.13	4.02	21.6%				3.79	67	29.8				335		
7:50	5.14	4.01	22.0%				3.88	68	30.2				340		
7:55	5.11	4.04	20.9%				3.86	68	30.2				340		
8:00	5.09	4.04	20.6%				3.93	68	30.2				340		
8:05	5.12	4.04	21.1%				3.88	68	30.2				340		
8:10	5.14	4.12	19.8%				3.82	68	30.2				340		
8:15	5.21	4.12	20.9%				3.90	68	30.2				340		
8:20								0		Wash Timer				0	
8:25								32	14.2					160	
8:30								32	14.2					160	**Percent**
8:35								32	14.2					160	**of wash**
8:40								32	14.2					160	**%**
Averages	5.19	4.05	21.9%					65.34	29.45				22555	640	2.76%

8/13/2018
Friday
30% Compression
Run #36

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
8:45	5.27	4.28	18.8%				3.14	69	30.7		30	5.2	345	
8:50	5.15	4.31	16.3%				3.14	69	30.7				345	
8:55	5.13	4.18	18.5%				3.12	69	30.7				345	
9:00	5.14	4.16	19.1%				3.16	69	30.7				345	
9:05	5.14	4.17	18.9%				3.16	69	30.7				345	
9:10	5.11	4.16	18.6%				3.13	69	30.7				345	
9:15	5.12	4.16	18.8%				3.16	69	30.7				345	
9:20	5.11	4.16	18.6%				3.26	69	30.7				345	
9:25	5.01	4.10	18.2%				3.14	69	30.7				345	
9:30	5.04	4.11	18.5%				3.17	69	30.7				345	
9:35	5.06	4.08	19.4%				3.09	69	30.7				345	
9:40	4.99	4.05	18.8%				3.17	69	30.7				345	
9:45	4.93	4.04	18.1%				3.28	69	30.7				345	
9:50	4.97	4.03	18.9%				3.14	69	30.7				345	
9:55	5.11	4.02	21.3%				3.21	69	30.7				345	
10:00	5.12	4.03	21.3%				3.23	69	30.7				345	
10:05	5.04	4.04	19.8%				3.23	68	30.2				340	
10:10	5.01	4.02	19.8%				3.17	68	30.2				340	
10:15	5.01	3.99	20.4%				3.22	69	30.7				345	
10:20	4.99	3.97	20.4%				3.24	69	30.7				345	
10:25	5.03	3.96	21.3%				3.24	69	30.7				345	
10:30	5.06	3.99	21.1%				3.23	68	30.2				340	
10:35	5.06	3.98	21.3%				3.21	69	30.7				345	
10:40	5.12	3.98	22.3%				3.33	68	30.2				340	
10:45	5.16	3.98	22.9%				3.24	69	30.7				345	
10:50	5.14	4.03	21.6%				3.29	69	30.7				345	
10:55	5.14	4.02	21.8%				3.31	69	30.7				345	
11:00	5.16	4.02	22.1%				3.32	69	30.7				345	
11:05	5.18	4.02	22.4%				3.33	68	30.2				340	
11:10	5.21	4.04	22.5%				3.33	69	30.7				345	
11:15	5.21	4.02	22.8%				3.27	69	30.7				345	
11:20	5.18	4.01	22.6%				3.32	68	30.2				340	
11:25	5.15	4.04	21.6%				3.33	68	30.2				340	
11:30	5.17	4.06	21.5%				3.41	68	30.2				340	
11:35	5.19	4.06	21.8%				3.38	68	30.2				340	
11:40	5.21	4.04	22.5%				3.39	69	30.7				345	
11:45	5.16	4.03	21.9%				3.41	68	30.2				340	
11:50	5.15	4.01	22.1%				3.41	68	30.2				340	
11:55	5.17	3.99	22.8%				3.30	68	30.2				340	
12:00	5.15	4.02	21.9%				3.49	68	30.2				340	
12:05	5.18	4.02	22.4%				3.37	68	30.2				340	
12:10	5.13	4.00	22.0%				3.48	68	30.2				340	
12:15	5.14	3.98	22.6%				3.63	68	30.2				340	
12:20	5.06	3.98	21.3%				3.48	68	30.2				340	
12:25	5.05	3.96	21.6%				3.51	68	30.2				340	
12:30	5.05	3.96	21.6%				3.54	68	30.2				340	
12:35	5.02	3.96	21.1%				3.63	68	30.2				340	
12:40	5.01	3.96	21.0%				3.58	68	30.2				340	
12:45	5.06	3.97	21.5%				3.53	68	30.2				340	
12:50	4.94	3.95	20.0%				3.50	68	30.2				340	
12:55	4.89	3.92	19.8%				3.52	68	30.2				340	
13:00	4.89	3.92	19.8%				3.56	68	30.2				340	

Run #36	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
13:05	4.89	3.91	20.0%				3.55	68	30.2				340		
13:10	4.89	3.91	20.0%				3.58	68	30.2				340		
13:15	4.94	3.94	20.2%				3.60	68	30.2				340		
13:20	4.96	3.94	20.6%				3.67	68	30.2				340		
13:25	5.02	3.94	21.5%				3.66	68	30.2				340		
13:30	4.99	3.97	20.4%				3.65	68	30.2				340		
13:35	4.96	3.95	20.4%				3.70	68	30.2				340		
13:40	4.96	3.96	20.2%				3.65	68	30.2				340		
13:45	4.94	3.96	19.8%				3.65	68	30.2				340		
13:50	4.93	3.94	20.1%				3.73	68	30.2				340		
13:55	4.92	3.94	19.9%				3.64	68	30.2				340		
14:00	4.94	3.94	20.2%				3.78	68	30.2				340		
14:05	5.02	3.96	21.1%				3.74	68	30.2				340		
14:10	5.02	3.97	20.9%				3.78	68	30.2				340		
14:15								0		Wash Timer				0	
14:20								29	12.9				145	145	
14:25								29	12.9				145	145	**Percent**
14:30								29	12.9				145	145	**of wash**
14:35								29	12.9				145	145	**%**
Averages	5.06	4.02	20.7%					65.23	29.40				23155	580	2.44%

8/31/2018
Friday
30% Compression
Run #37

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
14:40							3.23	69	30.7		30	5.2	345	
14:45							3.24	69	30.7				345	
14:50	5.12	4.20	18.0%				3.19	69	30.7				345	
14:55	5.13	4.21	17.9%				3.13	69	30.7				345	
15:00	5.19	4.25	18.1%				3.21	68	30.2				340	
15:05	5.24	4.26	18.7%				3.23	68	30.2				340	
15:10	5.19	4.24	18.3%				3.25	68	30.2				340	
15:15	5.17	4.25	17.8%				3.21	68	30.2				340	
15:20	5.21	4.24	18.6%				3.19	68	30.2				340	
15:25	5.26	4.22	19.8%				3.17	69	30.7				345	
15:30	5.26	4.20	20.2%				3.13	69	30.7				345	
15:35	5.25	4.19	20.2%				3.20	69	30.7				345	
15:40	5.19	4.18	19.5%				3.25	68	30.2	1679 Turned off			340	
15:45	5.17	4.16	19.5%				3.20	68	30.2	No Chemical until 9-4-48 at 8:00 am			340	
15:50	5.16	4.15	19.6%				3.21	69	30.7				345	
15:55	5.17	4.14	19.9%				3.27	69	30.7				345	
16:00	5.17	4.12	20.3%				3.20	69	30.7				345	
16:05	5.20	4.11	21.0%				3.20	69	30.7				345	
16:10	5.24	4.19	20.0%				3.23	69	30.7				345	
16:15	5.21	4.16	20.2%				3.29	68	30.2				340	
16:20	5.16	4.14	19.8%				3.24	69	30.7				345	
16:25	5.06	4.14	18.2%				3.23	68	30.2				340	
16:30	5.03	4.10	18.5%				3.19	68	30.2				340	
16:35	4.98	4.01	19.5%				3.15	68	30.2				340	
16:40	4.97	3.99	19.7%				3.26	68	30.2				340	
16:45	4.94	3.96	19.8%				3.29	68	30.2				340	
16:50	4.97	3.96	20.3%				3.31	68	30.2				340	
16:55	5.00	3.99	20.2%				3.29	69	30.7				345	
17:00	5.02	3.98	20.7%				3.40	68	30.2				340	
17:05	5.09	3.99	21.6%				3.31	69	30.7				345	
17:10	5.14	3.99	22.4%				3.35	68	30.2				340	
17:15	5.19	4.00	22.9%				3.37	69	30.7				345	
17:20	5.08	4.00	21.3%				3.35	68	30.2				340	
17:25	5.07	3.98	21.5%				3.40	68	30.2				340	
17:30	5.09	3.98	21.8%				3.34	68	30.2				340	
17:35	5.07	3.96	21.9%				3.31	68	30.2				340	
17:40	5.06	3.98	21.3%				3.38	68	30.2				340	
17:45	5.09	3.98	21.8%				3.38	68	30.2				340	
17:50	5.09	3.98	21.8%				3.46	68	30.2				340	
17:55	5.11	3.98	22.1%				3.33	68	30.2				340	
18:00	5.13	3.99	22.2%				3.40	68	30.2				340	
18:05	5.11	4.00	21.7%				3.45	68	30.2				340	
18:10	5.11	3.99	21.9%				3.44	68	30.2				340	
18:15	5.07	3.98	21.5%				3.37	68	30.2				340	
18:20	5.04	3.98	21.0%				3.39	68	30.2				340	
18:25	5.02	3.96	21.1%				3.50	68	30.2				340	
18:30	5.01	3.96	21.0%				3.47	68	30.2				340	
18:35	5.04	3.97	21.2%				3.49	68	30.2				340	
18:40	5.06	3.97	21.5%				3.50	68	30.2				340	
18:45	5.07	3.97	21.7%				3.48	68	30.2				340	
18:50	5.02	3.97	20.9%				3.47	68	30.2				340	

Run #37	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
18:55	5.04	3.96	21.4%				3.54	68	30.2				340		
19:00	5.04	3.96	21.4%				3.52	68	30.2				340		
19:05	5.08	3.99	21.5%				3.65	68	30.2				340		
19:10	5.05	3.98	21.2%				3.49	68	30.2				340		
19:15	5.08	3.98	21.7%				3.62	68	30.2				340		
19:20	5.19	4.00	22.9%				3.63	68	30.2				340		
19:25	5.09	3.99	21.6%				3.57	68	30.2				340		
19:30	5.08	3.97	21.9%				3.61	68	30.2				340		
19:35	5.10	3.98	22.0%				3.65	68	30.2				340		
19:40	5.13	4.04	21.2%				3.61	68	30.2				340		
19:45	5.06	4.06	19.8%				3.64	68	30.2				340		
19:50	5.07	4.06	19.9%				3.63	68	30.2				340		
19:55	5.07	4.08	19.5%				3.81	68	30.2				340		
20:00	5.04	4.08	19.0%				3.70	68	30.2				340		
20:05	5.13	4.10	20.1%				3.71	68	30.2				340		
20:10								28	12.4	Wash Timer				140	
20:15								28	12.4					140	Percent
20:20								28	12.4					140	of wash
20:25								28	12.4					140	%
Averages	5.10	4.06	20.6%					65.94	29.31				22520	560	2.43%

8/31/2018
Friday
30% Compression
Run #38

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
20:30							3.16	68	30.2		30	5.2	340	
20:35							3.09	69	30.7				345	
20:40	5.04	4.11	18.5%				3.13	69	30.7				345	
20:45	5.10	4.10	19.6%				3.06	68	30.2				340	
20:50	5.25	4.14	21.1%				3.15	69	30.7				345	
20:55	5.28	4.14	21.6%				3.22	68	30.2				340	
21:00	5.31	4.14	22.0%				3.22	69	30.7				345	
21:05	5.23	4.15	20.7%				3.19	69	30.7				345	
21:10	5.16	4.11	20.3%				3.11	68	30.2				340	
21:15	5.16	4.19	18.8%				3.26	69	30.7				345	
21:20	5.14	4.14	19.5%				3.19	68	30.2				340	
21:25	5.12	4.14	19.1%				3.23	69	30.7				345	
21:30	5.07	4.10	19.1%				3.11	69	30.7				345	
21:35	5.12	4.13	19.3%				3.14	68	30.2				340	
21:40	5.11	4.09	20.0%				3.20	68	30.2				340	
21:45	5.09	4.12	19.1%				3.19	68	30.2				340	
21:50	5.09	4.09	19.6%				3.33	68	30.2				340	
21:55	5.02	4.06	19.1%				3.22	68	30.2				340	
22:00	5.06	4.04	20.2%				3.24	68	30.2				340	
22:05	5.05	4.06	19.6%				3.25	68	30.2				340	
22:10	5.04	4.04	19.8%				3.27	68	30.2				340	
22:15	5.01	4.02	19.8%				3.33	68	30.2				340	
22:20	5.02	4.02	19.9%				3.20	68	30.2				340	
22:25	5.06	3.99	21.1%				3.30	68	30.2				340	
22:30	5.06	3.98	21.3%				3.28	68	30.2				340	
22:35	4.99	4.02	19.4%				3.32	68	30.2				340	
22:40	4.98	3.99	19.9%				3.30	68	30.2				340	
22:45	5.04	3.98	21.0%				3.27	68	30.2				340	
22:50	5.04	3.97	21.2%				3.38	68	30.2				340	
22:55	5.07	3.97	21.7%				3.36	68	30.2				340	
23:00	5.00	3.97	20.6%				3.43	68	30.2				340	
23:05	4.98	3.99	19.9%				3.40	68	30.2				340	
23:10	5.01	3.97	20.8%				3.46	68	30.2				340	
23:15	4.98	3.96	20.5%				3.41	68	30.2				340	
23:20	4.98	3.89	21.9%				3.42	68	30.2				340	
23:25	4.94	3.86	21.9%				3.39	68	30.2				340	
23:30	5.04	3.85	23.6%				3.48	68	30.2				340	
23:35	4.95	3.85	22.2%				3.48	68	30.2				340	
23:40	5.01	3.86	23.0%				3.52	68	30.2				340	
23:45	4.98	3.85	22.7%				3.50	68	30.2				340	
23:50	4.91	3.85	21.6%				3.56	68	30.2				340	
23:55	4.94	3.86	21.9%				3.55	68	30.2				340	
0:00	4.97	3.86	22.3%				3.51	68	30.2	9/1/2018			340	
0:05	4.97	3.86	22.3%				3.55	68	30.2				340	
0:10	4.98	3.85	22.7%				3.56	68	30.2				340	
0:15	5.01	3.86	23.0%				3.69	68	30.2				340	
0:20	4.99	3.86	22.6%				3.60	68	30.2				340	
0:25	4.97	3.85	22.5%				3.71	68	30.2				340	
0:30	4.95	3.85	22.2%				3.62	68	30.2				340	
0:35	4.93	3.85	21.9%				3.61	68	30.2				340	
0:40	4.95	3.85	22.2%				3.64	68	30.2				340	
0:45	4.93	3.86	21.7%				3.68	68	30.2				340	

Run #38	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
0:50	4.96	3.85	22.4%				3.72	67	29.8				335		
0:55	4.97	3.84	22.7%				3.72	68	30.2				340		
1:00	5.04	3.84	23.8%				3.70	68	30.2				340		
1:05	4.99	3.88	22.2%				3.71	68	30.2				340		
1:10	4.99	3.87	22.4%				3.78	68	30.2				340		
1:15	5.00	3.88	22.4%				3.70	68	30.2				340		
1:20	5.02	3.86	23.1%				3.81	68	30.2				340		
1:25	4.95	3.86	22.0%				3.81	68	30.2				340		
1:30	5.01	3.87	22.8%				3.78	68	30.2				340		
1:35	4.98	3.88	22.1%				3.79	68	30.2				340		
1:40	5.02	3.88	22.7%				3.86	68	30.2				340		
1:45	5.02	3.88	22.7%				3.78	68	30.2				340		
1:50	5.02	3.89	22.5%				3.88	68	30.2				340		
1:55	5.01	3.91	22.0%				3.85	68	30.2				340		
2:00	5.04	3.90	22.6%				3.87	67	29.8				335		
2:05								30	13.3	Wash Timer				150	
2:10								30	13.3					150	**Percent**
2:15								30	13.3					150	**of wash**
2:20								30	13.3					150	**%**
Averages	5.03	3.96	21.3%					65.94	29.31				22810	600	2.56%

9/1/2018
Saturday
30% Compression
Run #39

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
2:25							3.16	68	30.2		30	5.2	340	
2:30							3.09	69	30.7				345	
2:35	5.04	4.11	18.5%				3.13	69	30.7				345	
2:40	5.10	4.10	19.6%				3.06	68	30.2				340	
2:45	5.25	4.14	21.1%				3.15	69	30.7				345	
2:50	5.28	4.14	21.6%				3.22	68	30.2				340	
2:55	5.31	4.14	22.0%				3.22	69	30.7				345	
3:00	5.23	4.15	20.7%				3.19	69	30.7				345	
3:05	5.16	4.11	20.3%				3.11	68	30.2				340	
3:10	5.16	4.19	18.8%				3.26	69	30.7				345	
3:15	5.14	4.14	19.5%				3.19	68	30.2				340	
3:20	5.12	4.14	19.1%				3.23	69	30.7				345	
3:25	5.07	4.10	19.1%				3.11	69	30.7				345	
3:30	5.12	4.13	19.3%				3.14	68	30.2				340	
3:35	5.11	4.09	20.0%				3.20	68	30.2				340	
3:40	5.09	4.12	19.1%				3.19	68	30.2				340	
3:45	5.09	4.09	19.6%				3.33	68	30.2				340	
3:50	5.02	4.06	19.1%				3.22	68	30.2				340	
3:55	5.06	4.04	20.2%				3.24	68	30.2				340	
4:00	5.05	4.06	19.6%				3.25	68	30.2				340	
4:05	5.04	4.04	19.8%				3.27	68	30.2				340	
4:10	5.01	4.02	19.8%				3.33	68	30.2				340	
4:15	5.02	4.02	19.9%				3.20	68	30.2				340	
4:20	5.06	3.99	21.1%				3.30	68	30.2				340	
4:25	5.06	3.98	21.3%				3.28	68	30.2				340	
4:30	4.99	4.02	19.4%				3.32	68	30.2				340	
4:35	4.98	3.99	19.9%				3.30	68	30.2				340	
4:40	5.04	3.98	21.0%				3.27	68	30.2				340	
4:45	5.04	3.97	21.2%				3.38	68	30.2				340	
4:50	5.07	3.97	21.7%				3.36	68	30.2				340	
4:55	5.00	3.97	20.6%				3.43	68	30.2				340	
5:00	4.98	3.99	19.9%				3.40	68	30.2				340	
5:05	5.01	3.97	20.8%				3.46	68	30.2				340	
5:10	4.98	3.96	20.5%				3.41	68	30.2				340	
5:15	4.98	3.89	21.9%				3.42	68	30.2				340	
5:20	4.94	3.86	21.9%				3.39	68	30.2				340	
5:25	5.04	3.85	23.6%				3.48	68	30.2				340	
5:30	4.95	3.85	22.2%				3.48	68	30.2				340	
5:35	5.01	3.86	23.0%				3.52	68	30.2				340	
5:40	4.98	3.85	22.7%				3.50	68	30.2				340	
5:45	4.91	3.85	21.6%				3.56	68	30.2				340	
5:50	4.94	3.86	21.9%				3.55	68	30.2				340	
5:55	4.97	3.86	22.3%				3.51	68	30.2				340	
6:00	4.97	3.86	22.3%				3.55	68	30.2				340	
6:05	4.98	3.85	22.7%				3.56	68	30.2				340	
6:10	5.01	3.86	23.0%				3.69	68	30.2				340	
6:15	4.99	3.86	22.6%				3.60	68	30.2				340	
6:20	4.97	3.85	22.5%				3.71	68	30.2				340	
6:25	4.95	3.85	22.2%				3.62	68	30.2				340	
6:30	4.93	3.85	21.9%				3.61	68	30.2				340	
6:35	4.95	3.85	22.2%				3.64	68	30.2				340	
6:40	4.93	3.86	21.7%				3.68	68	30.2				340	

Run #39	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate			Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	Percent Compression %		gallons/day	gallons/day	
6:45	4.96	3.85	22.4%				3.72	67	29.8				335		
6:50	4.97	3.84	22.7%				3.72	68	30.2				340		
6:55	5.04	3.84	23.8%				3.70	68	30.2				340		
7:00	4.99	3.88	22.2%				3.71	68	30.2				340		
7:05	4.99	3.87	22.4%				3.78	68	30.2				340		
7:10	5.00	3.88	22.4%				3.70	68	30.2				340		
7:15	5.02	3.86	23.1%				3.81	68	30.2				340		
7:20	4.95	3.86	22.0%				3.81	68	30.2				340		
7:25	5.01	3.87	22.8%				3.78	68	30.2				340		
7:30	4.98	3.88	22.1%				3.79	68	30.2				340		
7:35	5.02	3.88	22.7%				3.86	68	30.2				340		
7:40	5.02	3.88	22.7%				3.78	68	30.2				340		
7:45	5.02	3.89	22.5%				3.88	68	30.2				340		
7:50	5.01	3.91	22.0%				3.85	68	30.2				340		
7:55	5.04	3.90	22.6%				3.87	67	29.8	Wash Timer			335		
8:00								30	13.3					150	
8:05								30	13.3					150	**Percent**
8:10								30	13.3					150	**of wash**
8:15								30	13.3					150	**%**
Averages	5.03	3.96	21.3%					65.94	29.31				22810	600	2.56%

9/1/2018
Saturday
30% Compression
Run #40

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
8:20							3.16	68	30.2		30	5.2	340	
8:25							3.12	69	30.7				345	
8:30	5.06	4.01	20.8%				3.13	68	30.2				340	
8:35	5.02	4.00	20.3%				3.17	69	30.7				345	
8:40	5.01	4.04	19.4%				3.22	68	30.2				340	
8:45	5.01	4.04	19.4%				3.22	68	30.2				340	
8:50	5.06	4.04	20.2%				3.17	68	30.2				340	
8:55	4.99	4.03	19.2%				3.15	68	30.2				340	
9:00	5.00	4.02	19.6%				3.09	69	30.7				345	
9:05	4.99	3.99	20.0%				3.21	68	30.2				340	
9:10	5.07	4.00	21.1%				3.15	69	30.7				345	
9:15	5.01	4.02	19.8%				3.20	68	30.2				340	
9:20	5.01	4.00	20.2%				3.11	68	30.2				340	
9:25	5.04	3.98	21.0%				3.18	68	30.2				340	
9:30	5.01	3.91	22.0%				3.19	68	30.2				340	
9:35	4.98	3.89	21.9%				3.24	68	30.2				340	
9:40	5.02	3.89	22.5%				3.32	68	30.2				340	
9:45	5.01	3.88	22.6%				3.20	68	30.2				340	
9:50	5.01	3.87	22.8%				3.25	68	30.2				340	
9:55	4.99	3.87	22.4%				3.24	68	30.2				340	
10:00	5.03	3.87	23.1%				3.26	69	30.7				345	
10:05	5.01	3.86	23.0%				3.30	69	30.7				345	
10:10	5.01	3.84	23.4%				3.26	68	30.2				340	
10:15	5.03	3.84	23.7%				3.18	68	30.2				340	
10:20	5.06	3.84	24.1%				3.41	68	30.2				340	
10:25	5.07	3.84	24.3%				3.29	68	30.2				340	
10:30	5.11	3.84	24.9%				3.31	68	30.2				340	
10:35	5.04	3.84	23.8%				3.31	68	30.2				340	
10:40	5.01	3.84	23.4%				3.34	68	30.2				340	
10:45	4.99	3.84	23.0%				3.32	68	30.2				340	
10:50	5.02	3.83	23.7%				3.39	68	30.2				340	
10:55	5.04	3.83	24.0%				3.38	68	30.2				340	
11:00	5.06	3.83	24.3%				3.34	68	30.2				340	
11:05	5.12	3.83	25.2%				3.47	68	30.2				340	
11:10	5.04	3.84	23.8%				3.34	68	30.2				340	
11:15	5.10	3.84	24.7%				3.40	68	30.2				340	
11:20	5.08	3.84	24.4%				3.48	68	30.2				340	
11:25	5.07	3.84	24.3%				3.55	68	30.2				340	
11:30	5.07	3.82	24.7%				3.45	68	30.2				340	
11:35	5.05	3.83	24.2%				3.54	68	30.2				340	
11:40	5.07	3.83	24.5%				3.42	68	30.2				340	
11:45	5.09	3.90	23.4%				3.52	68	30.2				340	
11:50	5.09	3.92	23.0%				3.63	68	30.2				340	
11:55	5.13	3.93	23.4%				3.63	68	30.2				340	
12:00	5.15	3.93	23.7%				3.63	68	30.2				340	
12:05	5.22	3.89	25.5%				3.60	68	30.2				340	
12:10	5.23	3.89	25.6%				3.62	68	30.2				340	
12:15	5.21	3.88	25.5%				3.67	68	30.2				340	
12:20	5.16	3.88	24.8%				3.59	68	30.2				340	
12:25	5.16	3.88	24.8%				3.76	68	30.2				340	
12:30	5.18	3.88	25.1%				3.73	68	30.2				340	
12:35	5.17	3.88	25.0%				3.67	68	30.2				340	

Run #40	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
12:40	5.15	3.88	24.7%				3.72	68	30.2				340		
12:45	5.12	3.89	24.0%				3.80	68	30.2				340		
12:50	5.11	3.88	24.1%				3.74	68	30.2				340		
12:55	5.17	3.89	24.8%				3.75	67	29.8				335		
13:00	5.24	3.88	26.0%				3.84	67	29.8				335		
13:05	5.11	3.88	24.1%				3.68	67	29.8				335		
13:10	5.08	3.91	23.0%				3.86	68	30.2				340		
13:15	5.07	3.90	23.1%				3.77	68	30.2				340		
13:20	5.07	3.90	23.1%				3.88	67	29.8				335		
13:25	5.07	3.89	23.3%				3.86	67	29.8				335		
13:30	5.02	3.91	22.1%				3.81	68	30.2				340		
13:35	5.05	3.90	22.8%				3.87	67	29.8				335		
13:40	5.09	3.89	23.6%				3.86	68	30.2				340		
13:45	5.09	3.92	23.0%				3.89	67	29.8				335		
13:50	5.08	3.92	22.8%				3.91	67	29.8				335		
13:55								32	14.2	Wash Timer				160	
14:00								32	14.2					160	**Percent**
14:05								32	14.2					160	**of wash**
14:10								32	14.2					160	**%**
Averages	5.07	3.90	23.1%					65.94	29.31				22770	640	2.73%

9/1/2018
Saturday
30% compression
Run #41

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
14:15							3.17	68	30.2		30	5.2	340	
14:20							3.07	68	30.2				340	
14:25	5.00	3.95	21.0%				3.20	69	30.7				345	
14:30	4.93	3.94	20.1%				3.14	69	30.7				345	
14:35	4.92	3.94	19.9%				3.13	69	30.7				345	
14:40	4.93	3.96	19.7%				3.20	69	30.7				345	
14:45	4.93	3.96	19.7%				3.15	69	30.7				345	
14:50	4.93	3.95	19.9%				3.15	68	30.2				340	
14:55	4.89	3.93	19.6%				3.25	69	30.7				345	
15:00	4.91	3.92	20.2%				3.11	68	30.2				340	
15:05	4.99	3.91	21.6%				3.12	68	30.2				340	
15:10	4.99	3.93	21.2%				3.10	68	30.2				340	
15:15	5.01	3.92	21.8%				3.25	68	30.2				340	
15:20	5.01	3.92	21.8%				3.21	69	30.7				345	
15:25	5.01	3.91	22.0%				3.22	68	30.2				340	
15:30	5.01	3.90	22.2%				3.25	68	30.2				340	
15:35	5.01	3.90	22.2%				3.35	68	30.2				340	
15:40	4.94	3.88	21.5%				3.23	68	30.2				340	
15:45	4.89	3.86	21.1%				3.16	68	30.2				340	
15:50	4.93	3.86	21.7%				3.23	68	30.2				340	
15:55	4.97	3.84	22.7%				3.24	68	30.2				340	
16:00	4.96	3.84	22.6%				3.30	68	30.2				340	
16:05	4.93	3.85	21.9%				3.17	68	30.2				340	
16:10	4.92	3.85	21.7%				3.31	68	30.2				340	
16:15	4.93	3.83	22.3%				3.31	68	30.2				340	
16:20	4.96	3.86	22.2%				3.31	68	30.2				340	
16:25	4.94	3.86	21.9%				3.31	68	30.2				340	
16:30	4.94	3.86	21.9%				3.42	68	30.2				340	
16:35	4.99	3.84	23.0%				3.36	68	30.2				340	
16:40	5.01	3.88	22.6%				3.32	68	30.2				340	
16:45	5.02	3.88	22.7%				3.34	68	30.2				340	
16:50	4.99	3.87	22.4%				3.29	68	30.2				340	
16:55	4.91	3.86	21.4%				3.33	68	30.2				340	
17:00	4.88	3.85	21.1%				3.43	68	30.2				340	
17:05	4.86	3.88	20.2%				3.43	68	30.2				340	
17:10	4.94	3.86	21.9%				3.43	68	30.2				340	
17:15	4.95	3.84	22.4%				3.49	68	30.2				340	
17:20	5.00	3.86	22.8%				3.45	68	30.2				340	
17:25	5.07	3.96	21.9%				3.37	68	30.2				340	
17:30	5.09	3.99	21.6%				3.55	68	30.2				340	
17:35	5.09	3.99	21.6%				3.50	68	30.2				340	
17:40	5.09	3.96	22.2%				3.50	68	30.2				340	
17:45	5.12	3.94	23.0%				3.54	68	30.2				340	
17:50	5.12	3.95	22.9%				3.51	68	30.2				340	
17:55	5.02	3.92	21.9%				3.59	68	30.2				340	
18:00	4.99	3.91	21.6%				3.56	68	30.2				340	
18:05	5.07	3.91	22.9%				3.54	68	30.2				340	
18:10	5.09	3.91	23.2%				3.61	68	30.2				340	
18:15	5.13	3.93	23.4%				3.55	68	30.2				340	
18:20	5.12	3.97	22.5%				3.61	68	30.2				340	
18:25	5.19	3.96	23.7%				3.59	68	30.2				340	
18:30	5.16	3.98	22.9%				3.70	68	30.2				340	

Run #41	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
18:35	5.17	3.97	23.2%				3.62	68	30.2				340		
18:40	5.19	3.99	23.1%				3.63	68	30.2				340		
18:45	5.22	3.99	23.6%				3.65	68	30.2				340		
18:50	5.15	3.98	22.7%				3.73	68	30.2				340		
18:55	5.17	3.99	22.8%				3.68	68	30.2				340		
19:00	5.20	3.99	23.3%				3.71	68	30.2				340		
19:05	5.26	4.02	23.6%				3.78	68	30.2				340		
19:10	5.21	4.02	22.8%				3.59	68	30.2				340		
19:15	5.22	4.01	23.2%				3.72	68	30.2				340		
19:20	5.21	4.02	22.8%				3.79	68	30.2				340		
19:25	5.20	4.02	22.7%				3.71	68	30.2				340		
19:30	5.32	3.99	25.0%				3.73	66	29.3				330		
19:35	5.21	3.92	24.8%				3.74	66	29.3				330		
19:40	5.21	3.94	24.4%				3.73	66	29.3				330		
19:45	5.15	3.94	23.5%				3.70	66	29.3				330		
19:50								31	13.8	Wash Timer				155	
19:55								31	13.8					155	**Percent**
20:00								30	13.3					150	**of wash**
20:05								31	13.8					155	**%**
Averages	5.04	3.92	22.2%					65.89	29.28				22775	615	2.63%

9/1/2018
Saturday
30% Compression
Run #42

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
20:10							3.17	67	29.8		30	5.2	335	
20:15							3.13	67	29.8				335	
20:20	5.22	4.14	20.7%				3.13	67	29.8				335	
20:25	5.21	4.13	20.7%				3.10	67	29.8				335	
20:30	5.18	4.10	20.8%				3.12	68	30.2				340	
20:35	5.11	4.10	19.8%				3.12	67	29.8				335	
20:40	5.12	4.07	20.5%				3.17	67	29.8				335	
20:45	5.16	4.05	21.5%				3.15	67	29.8				335	
20:50	5.09	4.05	20.4%				3.17	67	29.8				335	
20:55	5.07	4.04	20.3%				3.20	67	29.8				335	
21:00	5.08	4.03	20.7%				3.20	67	29.8				335	
21:05	5.08	4.02	20.9%				3.21	67	29.8				335	
21:10	5.06	4.00	20.9%				3.20	67	29.8				335	
21:15	5.07	3.99	21.3%				3.26	67	29.8				335	
21:20	5.01	3.98	20.6%				3.24	67	29.8				335	
21:25	5.07	3.96	21.9%				3.21	67	29.8				335	
21:30	5.05	3.98	21.2%				3.20	67	29.8				335	
21:35	5.07	3.96	21.9%				3.22	67	29.8				335	
21:40	5.12	3.97	22.5%				3.20	67	29.8				335	
21:45	5.07	3.97	21.7%				3.21	67	29.8				335	
21:50	5.05	3.95	21.8%				3.24	67	29.8				335	
21:55	5.04	3.98	21.0%				3.23	67	29.8				335	
22:00	5.12	3.95	22.9%				3.26	67	29.8				335	
22:05	5.12	3.97	22.5%				3.26	67	29.8				335	
22:10	5.15	3.98	22.7%				3.22	67	29.8				335	
22:15	5.11	3.99	21.9%				3.25	67	29.8				335	
22:20	5.12	4.07	20.5%				3.28	67	29.8				335	
22:25	5.11	4.10	19.8%				3.40	67	29.8				335	
22:30	5.17	4.06	21.5%				3.30	67	29.8				335	
22:35	5.27	4.05	23.1%				3.32	67	29.8				335	
22:40	5.19	4.06	21.8%				3.39	67	29.8				335	
22:45	5.16	4.15	19.6%				3.34	67	29.8				335	
22:50	5.12	4.16	18.8%				3.30	67	29.8				335	
22:55	5.19	4.11	20.8%				3.34	67	29.8				335	
23:00	5.11	4.12	19.4%				3.35	67	29.8				335	
23:05	5.21	4.10	21.3%				3.33	67	29.8				335	
23:10	5.19	4.07	21.6%				3.35	67	29.8				335	
23:15	5.19	4.06	21.8%				3.37	67	29.8				335	
23:20	5.20	4.06	21.9%				3.37	67	29.8				335	
23:25	5.18	4.02	22.4%				3.33	66	29.3				330	
23:30	5.15	4.03	21.7%				3.43	66	29.3				330	
23:35	5.19	4.13	20.4%				3.42	66	29.3				330	
23:40	5.19	4.14	20.2%				3.47	66	29.3				330	
23:45	5.19	4.14	20.2%				3.44	67	29.8				335	
23:50	5.17	4.18	19.1%				3.37	66	29.3				330	
23:55	5.19	4.17	19.7%				3.44	66	29.3				330	
0:00	5.32	4.17	21.6%				3.49	66	29.3	9/2/2018			330	
0:05	5.21	4.19	19.6%				3.49	66	29.3				330	
0:10	5.25	4.18	20.4%				3.49	66	29.3				330	
0:15	5.20	4.16	20.0%				3.51	66	29.3				330	
0:20	5.21	4.15	20.3%				3.45	66	29.3				330	
0:25	5.22	4.16	20.3%				3.49	66	29.3				330	

Run #42	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
0:30	5.22	4.16	20.3%				3.53	66	29.3				330		
0:35	5.17	4.17	19.3%				3.52	67	29.8				335		
0:40	5.16	4.17	19.2%				3.52	66	29.3				330		
0:45	5.22	4.15	20.5%				3.51	67	29.8				335		
0:50	5.15	4.12	20.0%				3.54	66	29.3				330		
0:55	5.13	4.11	19.9%				3.54	67	29.8				335		
1:00	5.12	4.10	19.9%				3.57	66	29.3				330		
1:05	5.15	4.11	20.2%				3.59	67	29.8				335		
1:10	5.21	4.08	21.7%				3.61	66	29.3				330		
1:15	5.16	4.09	20.7%				3.57	66	29.3				330		
1:20	5.12	4.04	21.1%				3.57	66	29.3				330		
1:25	5.19	4.02	22.5%				3.65	66	29.3				330		
1:30	5.13	4.00	22.0%				3.66	66	29.3				330		
1:35	5.11	4.02	21.3%				3.68	66	29.3				330		
1:40								0		Wash Timer				0	
1:45								27	12.0					135	
1:50								27	12.0					135	**Percent**
1:55								27	12.0					135	**of wash**
2:00								28	12.4					140	**%**
Averages	5.15	4.07	20.9%					63.52	28.63				22005	545	2.42%

9/2/2018
Sunday
30% compression

Run #43 Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
2:05							3.14	67	29.8		30	5.2	335	
2:10							3.07	67	29.8				335	
2:15	5.00	4.04	19.2%				3.15	67	29.8				335	
2:20	5.11	4.06	20.5%				3.07	67	29.8				335	
2:25	5.01	4.06	19.0%				3.17	67	29.8				335	
2:30	5.04	4.07	19.2%				3.19	67	29.8				335	
2:35	5.04	4.06	19.4%				3.02	67	29.8				335	
2:40	5.17	4.04	21.9%				3.09	67	29.8				335	
2:45	5.23	4.06	22.4%				3.15	67	29.8				335	
2:50	5.27	4.04	23.3%				3.20	67	29.8				335	
2:55	5.40	4.04	25.2%				3.19	67	29.8				335	
3:00	5.41	4.05	25.1%				3.20	67	29.8				335	
3:05	5.27	4.05	23.1%				3.20	67	29.8				335	
3:10	5.22	4.04	22.6%				3.26	67	29.8				335	
3:15	5.16	3.99	22.7%				3.21	67	29.8				335	
3:20	5.15	3.97	22.9%				3.17	66	29.3				330	
3:25	5.17	3.96	23.4%				3.21	67	29.8				335	
3:30	5.21	3.94	24.4%				3.22	67	29.8				335	
3:35	5.16	3.93	23.8%				3.20	67	29.8				335	
3:40	5.17	3.94	23.8%				3.33	67	29.8				335	
3:45	5.11	3.94	22.9%				3.30	67	29.8				335	
3:50	5.11	3.94	22.9%				3.24	67	29.8				335	
3:55	5.17	3.94	23.8%				3.24	66	29.3				330	
4:00	5.17	3.92	24.2%				3.21	67	29.8				335	
4:05	5.20	3.92	24.6%				3.28	66	29.3				330	
4:10	5.26	3.91	25.7%				3.31	67	29.8				335	
4:15	5.18	3.99	23.0%				3.34	67	29.8				335	
4:20	5.12	3.94	23.0%				3.29	66	29.3				330	
4:25	5.34	3.96	25.8%				3.29	66	29.3				330	
4:30	5.44	4.04	25.7%				3.39	66	29.3				330	
4:35	5.72	4.02	29.7%				3.32	66	29.3				330	
4:40	5.84	4.08	30.1%				3.31	66	29.3				330	
4:45	5.83	4.10	29.7%				3.37	66	29.3				330	
4:50	5.74	4.06	29.3%				3.37	66	29.3				330	
4:55	5.96	4.09	31.4%				3.45	66	29.3				330	
5:00	5.88	4.11	30.1%				3.42	65	28.9				325	
5:05	5.70	4.08	28.4%				3.46	66	29.3				330	
5:10	5.53	4.02	27.3%				3.46	66	29.3				330	
5:15	5.48	3.99	27.2%				3.54	66	29.3				330	
5:20	5.44	3.97	27.0%				3.57	66	29.3				330	
5:25	5.43	3.98	26.7%				3.51	66	29.3				330	
5:30	5.27	3.94	25.2%				3.62	66	29.3				330	
5:35	5.27	3.96	24.9%				3.56	66	29.3				330	
5:40	5.22	3.96	24.1%				3.54	66	29.3				330	
5:45	5.20	3.94	24.2%				3.60	66	29.3				330	
5:50	5.22	3.96	24.1%				3.60	65	28.9				325	
5:55	5.24	3.96	24.4%				3.64	66	29.3				330	
6:00	5.27	3.94	25.2%				3.72	65	28.9				325	
6:05	5.32	3.95	25.8%				3.66	65	28.9				325	
6:10	5.39	3.92	27.3%				3.62	65	28.9				325	
6:15	5.50	3.93	28.5%				3.75	65	28.9				325	
6:20	5.65	3.94	30.3%				3.73	65	28.9				325	

Run #43	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading			Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate			Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments		%		gallons/day	gallons/day	
6:25	5.47	3.94	28.0%				3.71	65	28.9					325		
6:30	5.36	3.92	26.9%				3.82	65	28.9					325		
6:35	5.35	3.88	27.5%				3.76	65	28.9					325		
6:40	5.52	3.90	29.3%				3.82	65	28.9					325		
6:45	5.93	3.94	33.6%				3.77	65	28.9					325		
6:50	6.85	4.13	39.7%				3.89	65	28.9					325		
6:55	6.93	4.26	38.5%				3.88	65	28.9					325		
7:00	6.35	4.23	33.4%				3.88	65	28.9					325		
7:05	5.90	4.09	30.7%				3.85	65	28.9					325		
7:10	5.57	4.06	27.1%				3.92	64	28.4					320		
7:15	5.45	3.94	27.7%				3.99	64	28.4					320		
7:20	5.36	3.96	26.1%				3.96	64	28.4					320		
7:25	5.39	3.96	26.5%				4.01	64	28.4					320		
7:30	5.50	3.97	27.8%				3.94	64	28.4					320		
7:35								0		Wash Timer					0	
7:40								27	12.0						135	
7:45								27	12.0						135	**Percent**
7:50								27	12.0						135	**of wash**
7:55								27	12.0						135	**%**
Averages	5.43	4.00	26.3%					62.85	28.33					21770	540	2.42%

9/2/2018
Sunday
30% Compression
Run #44

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
8:00							3.20	66	29.3		30	5.2	330	
8:05							3.07	66	29.3				330	
8:10	5.95	4.14	30.4%				3.17	66	29.3				330	
8:15	5.69	4.10	27.9%				3.35	66	29.3				330	
8:20	5.63	4.04	28.2%				3.23	66	29.3				330	
8:25	5.49	4.00	27.1%				3.18	66	29.3				330	
8:30	5.41	3.98	26.4%				3.20	66	29.3				330	
8:35	5.37	3.99	25.7%				3.18	66	29.3				330	
8:40	5.35	3.96	26.0%				3.21	66	29.3				330	
8:45	5.29	4.00	24.4%				3.22	66	29.3				330	
8:50	5.31	3.96	25.4%				3.19	66	29.3				330	
8:55	5.29	3.94	25.5%				3.24	66	29.3				330	
9:00	5.29	3.94	25.5%				3.26	66	29.3				330	
9:05	5.39	3.94	26.9%				3.23	66	29.3				330	
9:10	5.43	3.94	27.4%				3.23	66	29.3				330	
9:15	5.49	3.94	28.2%				3.32	66	29.3				330	
9:20	5.35	3.92	26.7%				3.31	66	29.3				330	
9:25	5.39	3.91	27.5%				3.30	66	29.3				330	
9:30	5.32	3.90	26.7%				3.37	66	29.3				330	
9:35	5.32	3.90	26.7%				3.33	66	29.3				330	
9:40	5.27	3.89	26.2%				3.38	66	29.3				330	
9:45	5.29	3.89	26.5%				3.38	66	29.3				330	
9:50	5.31	3.89	26.7%				3.49	66	29.3				330	
9:55	5.30	3.90	26.4%				3.45	66	29.3				330	
10:00	5.25	3.89	25.9%				3.39	66	29.3				330	
10:05	5.26	3.83	27.2%				3.39	66	29.3				330	
10:10	5.26	3.82	27.4%				3.38	66	29.3				330	
10:15	5.33	3.83	28.1%				3.52	66	29.3				330	
10:20	5.29	3.85	27.2%				3.49	66	29.3				330	
10:25	5.28	3.83	27.5%				3.53	66	29.3				330	
10:30	5.32	3.83	28.0%				3.57	66	29.3				330	
10:35	5.27	3.83	27.3%				3.45	66	29.3				330	
10:40	5.54	3.84	30.7%				3.55	66	29.3				330	
10:45	5.47	3.84	29.8%				3.57	66	29.3				330	
10:50	5.39	3.84	28.8%				3.54	66	29.3				330	
10:55	5.34	3.83	28.3%				3.56	66	29.3				330	
11:00	5.34	3.84	28.1%				3.62	66	29.3				330	
11:05	5.31	3.85	27.5%				3.65	66	29.3				330	
11:10	5.32	3.87	27.3%				3.64	66	29.3				330	
11:15	5.40	3.86	28.5%				3.66	65	28.9				325	
11:20	5.56	3.88	30.2%				3.68	66	29.3				330	
11:25	5.53	3.91	29.3%				3.74	66	29.3				330	
11:30	5.53	3.91	29.3%				3.70	65	28.9				325	
11:35	5.67	3.94	30.5%				3.69	65	28.9				325	
11:40	5.67	3.95	30.3%				3.75	65	28.9				325	
11:45	5.70	3.96	30.5%				3.77	65	28.9				325	
11:50	5.77	3.97	31.2%				3.78	66	29.3				330	
11:55	5.82	3.99	31.4%				3.84	65	28.9				325	
12:00	5.69	4.00	29.7%				3.81	65	28.9				325	
12:05	5.58	4.00	28.3%				3.77	65	28.9				325	
12:10	5.52	4.00	27.5%				3.91	65	28.9				325	
12:15	5.52	3.99	27.7%				3.76	65	28.9				325	

Run #44	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading			Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate			Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments		%		gallons/day	gallons/day	
12:20	5.46	3.98	27.1%				3.85	65	28.9					325		
12:25	5.47	3.97	27.4%				3.87	65	28.9					325		
12:30	5.49	4.00	27.1%				3.98	65	28.9					325		
12:35	5.47	3.97	27.4%				3.89	65	28.9					325		
12:40	5.47	3.98	27.2%				3.89	65	28.9					325		
12:45	5.46	3.99	26.9%				3.94	64	28.4					320		
12:50	5.45	4.00	26.6%				3.94	64	28.4					320		
12:55	5.46	4.02	26.4%				3.92	65	28.9					325		
13:00	5.41	4.04	25.3%				3.94	64	28.4					320		
13:05	5.38	4.03	25.1%				3.94	64	28.4					320		
13:10	5.39	4.03	25.2%				3.95	64	28.4					320		
13:15	5.41	4.06	25.0%				3.92	64	28.4					320		
13:20	5.43	4.05	25.4%				3.92	65	28.9					325		
13:25	5.43	4.07	25.0%				3.93	65	28.9					325		
13:30								0		Wash Timer					0	
13:35								28	12.4						140	
13:40								28	12.4						140	**Percent**
13:45								28	12.4						140	**of wash**
13:50								27	12.0						135	**%**
Averages	5.44	3.94	27.5%					62.49	28.17					21630	555	2.50%

9/2/2018
Sunday
30% Compression
Run #45

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
13:55							3.10	67	29.8		30	5.2	335	
14:00							3.16	67	29.8				335	
14:05	5.57	4.28	23.2%				3.11	67	29.8				335	
14:10	5.59	4.30	23.1%				3.16	67	29.8				335	
14:15	5.57	4.32	22.4%				3.15	67	29.8				335	
14:20	5.55	4.30	22.5%				3.17	67	29.8				335	
14:25	5.56	4.32	22.3%				3.15	67	29.8				335	
14:30	5.55	4.35	21.6%				3.19	67	29.8				335	
14:35	5.61	4.30	23.4%				3.19	67	29.8				335	
14:40	5.59	4.42	20.9%				3.19	67	29.8				335	
14:45	5.59	4.40	21.3%				3.18	67	29.8				335	
14:50	5.61	4.34	22.6%				3.29	67	29.8				335	
14:55	5.65	4.30	23.9%				3.24	67	29.8				335	
15:00	5.54	4.24	23.5%				3.21	67	29.8				335	
15:05	5.53	4.24	23.3%				3.21	67	29.8				335	
15:10	5.59	4.24	24.2%				3.25	67	29.8				335	
15:15	5.60	4.24	24.3%				3.26	67	29.8				335	
15:20	5.57	4.36	21.7%				3.22	67	29.8				335	
15:25	5.65	4.38	22.5%				3.23	67	29.8				335	
15:30	5.61	4.50	19.8%				3.24	66	29.3				330	
15:35	5.55	4.46	19.6%				3.30	67	29.8				335	
15:40	5.50	4.42	19.6%				3.31	67	29.8				335	
15:45	5.49	4.37	20.4%				3.33	66	29.3				330	
15:50	5.47	4.33	20.8%				3.26	67	29.8				335	
15:55	5.49	4.30	21.7%				3.32	67	29.8				335	
16:00	5.49	4.26	22.4%				3.34	66	29.3				330	
16:05	5.47	4.18	23.6%				3.30	66	29.3				330	
16:10	5.47	4.28	21.8%				3.34	67	29.8				335	
16:15	5.47	4.34	20.7%				3.32	67	29.8				335	
16:20	5.40	4.36	19.3%				3.35	66	29.3				330	
16:25	5.44	4.42	18.8%				3.42	66	29.3				330	
16:30	5.42	4.44	18.1%				3.37	66	29.3				330	
16:35	5.45	4.51	17.2%				3.41	66	29.3				330	
16:40	5.47	4.45	18.6%				3.35	66	29.3				330	
16:45	5.52	4.40	20.3%				3.41	66	29.3				330	
16:50	5.52	4.37	20.8%				3.42	66	29.3				330	
16:55	5.43	4.43	18.4%				3.38	66	29.3				330	
17:00	5.49	4.47	18.6%				3.46	66	29.3				330	
17:05	5.50	4.48	18.5%				3.47	66	29.3				330	
17:10	5.51	4.48	18.7%				3.45	66	29.3				330	
17:15	5.55	4.51	18.7%				3.50	66	29.3				330	
17:20	5.55	4.43	20.2%				3.52	66	29.3				330	
17:25	5.57	4.43	20.5%				3.48	66	29.3				330	
17:30	5.62	4.28	23.8%				3.51	66	29.3				330	
17:35	5.72	4.29	25.0%				3.51	66	29.3				330	
17:40	5.59	4.29	23.3%				3.54	66	29.3				330	
17:45	5.62	4.51	19.8%				3.49	66	29.3				330	
17:50	5.59	4.54	18.8%				3.57	66	29.3				330	
17:55	5.57	4.54	18.5%				3.59	66	29.3				330	
18:00	5.64	4.50	20.2%				3.58	66	29.3				330	
18:05	5.78	4.44	23.2%				3.57	66	29.3				330	
18:10	5.74	4.35	24.2%				3.63	66	29.3				330	

Run #45	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading		Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate		Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
18:15	5.77	4.31	25.3%				3.62	66	29.3				330		
18:20	5.65	4.42	21.8%				3.62	66	29.3				330		
18:25	5.65	4.45	21.2%				3.62	66	29.3				330		
18:30	5.59	4.48	19.9%				3.61	66	29.3				330		
18:35	5.59	4.45	20.4%				3.64	66	29.3				330		
18:40	5.59	4.34	22.4%				3.63	66	29.3				330		
18:45	5.57	4.32	22.4%				3.71	66	29.3				330		
18:50	5.75	4.30	25.2%				3.71	66	29.3				330		
18:55	5.74	4.34	24.4%				3.69	66	29.3				330		
19:00	5.67	4.33	23.6%				3.65	66	29.3				330		
19:05	5.62	4.32	23.1%				3.76	66	29.3				330		
19:10	5.55	4.31	22.3%				3.75	66	29.3				330		
19:15	5.55	4.28	22.9%				3.77	66	29.3				330		
19:20	5.56	4.48	19.4%				3.76	66	29.3				330		
19:25								31	13.8					155	
19:30								31	13.8					155	**Percent**
19:35								31	13.8					155	**of wash**
19:40								31	13.8					155	**%**
Averages													21905	620	2.75%

9/2/2018
Sunday
30% Compression
Run #46

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
19:45							3.16	67	29.8		30	5.2	335	
19:50							3.13	67	29.8				335	
19:55	5.66	4.30	24.0%				3.07	66	29.3				330	
20:00	5.58	4.32	22.6%				3.12	67	29.8				335	
20:05	5.49	4.26	22.4%				3.19	67	29.8				335	
20:10	5.51	4.27	22.5%				3.16	67	29.8				335	
20:15	5.46	4.26	22.0%				3.25	67	29.8				335	
20:20	5.41	4.27	21.1%				3.17	67	29.8				335	
20:25	5.38	4.25	21.0%				3.17	67	29.8				335	
20:30	5.37	4.24	21.0%				3.30	66	29.3				330	
20:35	5.39	4.20	22.1%				3.16	67	29.8				335	
20:40	5.33	4.21	21.0%				3.19	67	29.8				335	
20:45	5.31	4.21	20.7%				3.21	67	29.8				335	
20:50	5.41	4.20	22.4%				3.23	66	29.3				330	
20:55	5.37	4.18	22.2%				3.22	67	29.8				335	
21:00	5.34	4.18	21.7%				3.24	67	29.8				335	
21:05	5.42	4.17	23.1%				3.22	66	29.3				330	
21:10	5.47	4.16	23.9%				3.33	66	29.3				330	
21:15	5.43	4.17	23.2%				3.19	67	29.8				335	
21:20	5.39	4.16	22.8%				3.23	67	29.8				335	
21:25	5.38	4.14	23.0%				3.29	67	29.8				335	
21:30	5.39	4.23	21.5%				3.29	66	29.3				330	
21:35	5.40	4.17	22.8%				3.29	66	29.3				330	
21:40	5.32	4.18	21.4%				3.31	66	29.3				330	
21:45	5.41	4.18	22.7%				3.28	67	29.8				335	
21:50	5.44	4.19	23.0%				3.51	66	29.3				330	
21:55	5.44	4.17	23.3%				3.30	66	29.3				330	
22:00	5.34	4.14	22.5%				3.35	66	29.3				330	
22:05	5.30	4.16	21.5%				3.34	66	29.3				330	
22:10	5.35	4.14	22.6%				3.41	66	29.3				330	
22:15	5.27	4.14	21.4%				3.37	66	29.3				330	
22:20	5.33	4.13	22.5%				3.39	66	29.3				330	
22:25	5.34	4.14	22.5%				3.43	66	29.3				330	
22:30	5.34	4.08	23.6%				3.43	66	29.3				330	
22:35	5.36	4.06	24.3%				3.43	66	29.3				330	
22:40	5.31	4.08	23.2%				3.45	66	29.3				330	
22:45	5.44	4.14	23.9%				3.43	66	29.3				330	
22:50	5.49	4.19	23.7%				3.47	66	29.3				330	
22:55	5.35	4.18	21.9%				3.46	66	29.3				330	
23:00	5.36	4.16	22.4%				3.49	66	29.3				330	
23:05	5.33	4.16	22.0%				3.44	66	29.3				330	
23:10	5.30	4.12	22.3%				3.49	66	29.3				330	
23:15	5.27	4.15	21.3%				3.58	66	29.3				330	
23:20	5.29	4.18	21.0%				3.54	66	29.3				330	
23:25	5.25	4.16	20.8%				3.53	66	29.3				330	
23:30	5.29	4.12	22.1%				3.51	66	29.3				330	
23:35	5.27	4.09	22.4%				3.55	66	29.3				330	
23:40	5.25	4.07	22.5%				3.58	66	29.3				330	
23:45	5.23	4.07	22.2%				3.58	66	29.3				330	
23:50	5.22	4.07	22.0%				3.64	66	29.3				330	
23:55	5.25	4.05	22.9%				3.70	66	29.3				330	
0:00	5.37	4.06	24.4%				3.71	66	29.3	9/3/2018			330	

Run #46	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading			Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate			Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments		%		gallons/day	gallons/day	
0:05	5.33	4.07	23.6%				3.66	66	29.3					330		
0:10	5.26	4.08	22.4%				3.63	66	29.3					330		
0:15	5.29	4.08	22.9%				3.69	66	29.3					330		
0:20	5.35	4.07	23.9%				3.73	66	29.3					330		
0:25	4.93	4.08	17.2%				3.69	66	29.3					330		
0:30	4.86	4.08	16.0%				3.76	66	29.3					330		
0:35	4.79	4.08	14.8%				3.80	66	29.3					330		
0:40	4.72	4.09	13.3%				3.73	66	29.3					330		
0:45	4.89	4.08	16.6%				3.77	66	29.3					330		
0:50	4.91	4.08	16.9%				3.79	65	28.9					325		
0:55	4.99	4.09	18.0%				3.82	65	28.9					325		
1:00	5.07	4.09	19.3%				3.79	66	29.3					330		
1:05	4.99	4.09	18.0%				3.80	65	28.9					325		
1:10	4.93	4.09	17.0%				3.84	66	29.3					330		
1:15	4.94	4.10	17.0%				3.89	66	29.3					330		
1:20								32	14.2	Wash Timer					160	
1:25								32	14.2						160	**Percent**
1:30								31	13.8						155	**of wash**
1:35								31	13.8						155	**%**
Averages	5.29	4.15	21.6%					64.25	28.56					22180	630	2.76%

9/3/2018
Monday
30% Compression
Run #47

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
1:40							3.17	67	29.8		30	5.2	335	
1:45							3.10	67	29.8				335	
1:50	5.12	4.16	18.8%				3.20	67	29.8				335	
1:55	5.14	4.22	17.9%				3.26	67	29.8				335	
2:00	5.12	4.21	17.8%				3.17	67	29.8				335	
2:05	5.21	4.24	18.6%				3.19	67	29.8				335	
2:10	5.15	4.22	18.1%				3.17	67	29.8				335	
2:15	5.29	4.20	20.6%				3.29	67	29.8				335	
2:20	5.39	4.20	22.1%				3.12	67	29.8				335	
2:25	5.24	4.18	20.2%				3.23	67	29.8				335	
2:30	5.34	4.19	21.5%				3.23	67	29.8				335	
2:35	5.24	4.16	20.6%				3.22	67	29.8				335	
2:40	5.33	4.16	22.0%				3.23	67	29.8				335	
2:45	5.27	4.15	21.3%				3.30	66	29.3				330	
2:50	5.28	4.16	21.2%				3.23	67	29.8				335	
2:55	5.29	4.16	21.4%				3.23	67	29.8				335	
3:00	5.21	4.12	20.9%				3.30	67	29.8				335	
3:05	5.14	4.12	19.8%				3.30	67	29.8				335	
3:10	5.16	4.12	20.2%				3.27	67	29.8				335	
3:15	5.29	4.12	22.1%				3.30	67	29.8				335	
3:20	5.16	4.10	20.5%				3.29	67	29.8				335	
3:25	5.11	4.10	19.8%				3.32	66	29.3				330	
3:30	5.12	4.18	18.4%				3.36	66	29.3				330	
3:35	5.07	4.16	17.9%				3.45	66	29.3				330	
3:40	5.11	4.14	19.0%				3.32	66	29.3				330	
3:45	5.12	4.14	19.1%				3.36	66	29.3				330	
3:50	5.26	4.16	20.9%				3.40	66	29.3				330	
3:55	5.37	4.22	21.4%				3.44	67	29.8				335	
4:00	5.27	4.20	20.3%				3.52	66	29.3				330	
4:05	5.22	4.18	19.9%				3.46	66	29.3				330	
4:10	5.16	4.14	19.8%				3.46	66	29.3				330	
4:15	5.09	4.16	18.3%				3.60	66	29.3				330	
4:20	5.12	4.15	18.9%				3.47	66	29.3				330	
4:25	5.14	4.16	19.1%				3.52	66	29.3				330	
4:30	5.11	4.16	18.6%				3.51	66	29.3				330	
4:35	5.08	4.14	18.5%				3.53	66	29.3				330	
4:40	5.11	4.12	19.4%				3.58	66	29.3				330	
4:45	5.09	4.12	19.1%				3.65	66	29.3				330	
4:50	5.01	4.12	17.8%				3.63	66	29.3				330	
4:55	5.09	4.09	19.6%				3.61	66	29.3				330	
5:00	5.01	4.08	18.6%				3.66	66	29.3				330	
5:05	5.02	4.09	18.5%				3.60	66	29.3				330	
5:10	5.02	4.10	18.3%				3.72	66	29.3				330	
5:15	5.01	4.11	18.0%				3.70	66	29.3				330	
5:20	5.11	4.10	19.8%				3.69	66	29.3				330	
5:25	5.06	4.11	18.8%				3.81	66	29.3				330	
5:30	5.04	4.12	18.3%				3.75	66	29.3				330	
5:35	5.23	4.16	20.5%				3.80	66	29.3				330	
5:40	6.53	4.26	34.8%				3.78	66	29.3				330	
5:45	8.60	4.91	42.9%				3.94	66	29.3				330	
5:50	8.09	5.12	36.7%				3.96	65	28.9				325	
5:55	7.29	4.97	31.8%				3.94	65	28.9				325	

Run #47	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	**NTU**	**NTU**	**%**	**mg/l**	**mg/l**	**%**	**Pressure psi**	**Flow gpm**	**gpm/ft^2**	**Comments**	**%**		**gallons/day**	**gallons/day**	
6:00	6.51	4.79	26.4%				3.89	65	28.9				325		
6:05	6.15	4.65	24.4%				3.98	65	28.9				325		
6:10	5.89	4.52	23.3%				3.93	65	28.9				325		
6:15	5.69	4.47	21.4%				3.97	67	29.8				335		
6:20	5.47	4.56	16.6%				4.02	67	29.8				335		
6:25	5.36	4.47	16.6%				3.99	67	29.8				335		
6:30	5.23	4.44	15.1%				4.03	67	29.8				335		
6:35	5.16	4.43	14.1%				4.01	67	29.8				335		
6:40	5.14	4.41	14.2%				4.02	67	29.8				335		
6:45	5.12	4.42	13.7%				4.04	67	29.8				335		
6:50	5.07	4.45	12.2%				4.00	67	29.8				335		
6:55	5.09	4.46	12.4%				4.03	67	29.8				335		
7:00	5.12	4.50	12.1%				4.00	67	29.8				335		
7:05	5.14	4.55	11.5%				4.07	67	29.8				335		
7:10								0		Wash Timer				0	
7:15								31	13.8					155	
7:20								31	13.8					155	**Percent**
7:25								31	13.8					155	**of wash**
7:30								31	13.8					155	**%**
Averages	5.38	4.27	20.6%					63.48	28.62				21915	620	2.75%

9/3/2018
Monday
30% compression
Run #48

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
7:35							3.12	69	30.7		30	5.2	345	
7:40							3.11	69	30.7				345	
7:45	5.17	4.20	18.8%				3.19	69	30.7				345	
7:50	5.09	4.24	16.7%				3.11	69	30.7				345	
7:55	5.02	4.28	14.7%				3.19	69	30.7				345	
8:00	5.02	4.29	14.5%				3.20	69	30.7				345	
8:05	5.01	4.28	14.6%				3.19	69	30.7				345	
8:10	5.00	4.28	14.4%				3.27	69	30.7				345	
8:15	4.94	4.24	14.2%				3.28	69	30.7				345	
8:20	4.97	4.23	14.9%				3.18	68	30.2				340	
8:25	4.96	4.22	14.9%				3.20	69	30.7				345	
8:30	4.93	4.23	14.2%				3.25	69	30.7				345	
8:35	4.99	4.24	15.0%				3.25	69	30.7				345	
8:40	5.01	4.24	15.4%				3.30	68	30.2				340	
8:45	5.02	4.22	15.9%				3.30	69	30.7				345	
8:50	5.15	4.22	18.1%				3.28	69	30.7				345	
8:55	5.11	4.24	17.0%				3.28	69	30.7				345	
9:00	5.11	4.25	16.8%				3.26	68	30.2				340	
9:05	6.95	4.31	38.0%				3.38	68	30.2				340	
9:10	5.21	4.40	15.5%				3.42	69	30.7				345	
9:15	4.98	4.22	15.3%				3.32	68	30.2				340	
9:20	4.94	4.14	16.2%				3.38	68	30.2				340	
9:25	4.95	4.12	16.8%				3.30	68	30.2				340	
9:30	4.99	4.10	17.8%				3.43	68	30.2				340	
9:35	4.98	4.10	17.7%				3.46	68	30.2				340	
9:40	4.89	4.08	16.6%				3.42	68	30.2				340	
9:45	4.88	4.09	16.2%				3.43	68	30.2				340	
9:50	4.95	4.11	17.0%				3.47	68	30.2				340	
9:55	5.01	4.12	17.8%				3.48	68	30.2				340	
10:00	5.11	4.15	18.8%				3.47	68	30.2				340	
10:05	5.07	4.16	17.9%				3.46	68	30.2				340	
10:10	5.14	4.17	18.9%				3.43	68	30.2				340	
10:15	5.24	4.18	20.2%				3.41	68	30.2				340	
10:20	5.14	4.21	18.1%				3.41	68	30.2				340	
10:25	5.16	4.25	17.6%				3.58	68	30.2				340	
10:30	5.22	4.22	19.2%				3.53	68	30.2				340	
10:35	5.16	4.25	17.6%				3.51	68	30.2				340	
10:40	5.19	4.26	17.9%				3.67	68	30.2				340	
10:45	5.19	4.27	17.7%				3.71	67	29.8				335	
10:50	5.12	4.28	16.4%				3.65	68	30.2				340	
10:55	5.15	4.26	17.3%				3.63	68	30.2				340	
11:00	5.15	4.28	16.9%				3.63	68	30.2				340	
11:05	5.13	4.30	16.2%				3.69	68	30.2				340	
11:10	5.15	4.30	16.5%				3.63	68	30.2				340	
11:15	5.14	4.30	16.3%				3.70	68	30.2				340	
11:20	5.16	4.31	16.5%				3.69	68	30.2				340	
11:25	5.15	4.32	16.1%				3.69	68	30.2				340	
11:30	5.21	4.32	17.1%				3.71	68	30.2				340	
11:35	5.19	4.35	16.2%				3.73	68	30.2				340	
11:40	5.11	4.34	15.1%				3.79	67	29.8				335	
11:45	5.15	4.33	15.9%				3.80	68	30.2				340	
11:50	5.07	4.33	14.6%				3.76	67	29.8				335	

Run #48	Influent	Effluent	Percent	Influent	Effluent	Percent			Loading			Percent	Wash Trip	Filter		
	Turbidity	Turbidity	Removal	TSS	TSS	Removal	Pressure	Flow	Rate			Compression	Point	run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	psi	gpm	gpm/ft^2	Comments		%		gallons/day	gallons/day	
11:55	5.04	4.32	14.3%				3.78	67	29.8					335		
12:00	5.07	4.33	14.6%				3.82	67	29.8					335		
12:05	5.02	4.32	13.9%				3.82	68	30.2					340		
12:10	5.03	4.32	14.1%				3.83	67	29.8					335		
12:15	5.14	4.33	15.8%				3.84	67	29.8					335		
12:20	5.09	4.36	14.3%				3.90	67	29.8					335		
12:25	5.07	4.35	14.2%				3.91	67	29.8					335		
12:30	5.06	4.35	14.0%				3.85	67	29.8					335		
12:35	5.09	4.35	14.5%				3.88	67	29.8					335		
12:40	5.17	4.48	13.3%				3.89	67	29.8					335		
12:45	5.10	4.46	12.5%				3.85	67	29.8					335		
12:50	5.14	4.52	12.1%				3.89	67	29.8					335		
12:55	5.10	4.49	12.0%				3.90	67	29.8					335		
13:00	5.06	4.56	9.9%				3.92	67	29.8					335		
13:05								0		Wash Timer					0	
13:10								28	12.4						140	
13:15								28	12.4						140	**Percent**
13:20								27	12.0						135	**of wash**
13:25								28	12.4						140	**%**
Averages	5.11	4.27	16.4%					64.77	29.20					22440	555	2.41%

9/3/2018
Monday
30% Compression
Run #49

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
13:30							3.11	68	30.2		30	5.2	340	
13:35							3.17	68	30.2				340	
13:40	5.04	4.41	12.5%				3.21	68	30.2				340	
13:45	5.09	4.43	13.0%				3.30	68	30.2				340	
13:50	5.11	4.42	13.5%				3.21	68	30.2				340	
13:55	5.02	4.40	12.4%				3.22	68	30.2				340	
14:00	4.98	4.37	12.2%				3.20	68	30.2				340	
14:05	5.06	4.38	13.4%				3.23	68	30.2				340	
14:10	5.17	4.40	14.9%				3.17	68	30.2				340	
14:15	5.20	4.42	15.0%				3.24	68	30.2				340	
14:20	5.14	4.43	13.8%				3.22	68	30.2				340	
14:25	5.12	4.47	12.7%				3.23	68	30.2				340	
14:30	5.14	4.42	14.0%				3.26	67	29.8				335	
14:35	5.13	4.42	13.8%				3.32	68	30.2				340	
14:40	5.17	4.44	14.1%				3.30	68	30.2				340	
14:45	5.17	4.43	14.3%				3.23	68	30.2				340	
14:50	5.25	4.43	15.6%				3.28	67	29.8				335	
14:55	5.27	4.44	15.7%				3.31	68	30.2				340	
15:00	5.29	4.47	15.5%				3.30	68	30.2				340	
15:05	5.31	4.45	16.2%				3.35	68	30.2				340	
15:10	5.29	4.49	15.1%				3.33	67	29.8				335	
15:15	5.28	4.50	14.8%				3.34	68	30.2				340	
15:20	5.32	4.47	16.0%				3.34	67	29.8				335	
15:25	5.41	4.48	17.2%				3.38	67	29.8				335	
15:30	5.42	4.52	16.6%				3.37	67	29.8				335	
15:35	5.51	4.50	18.3%				3.44	67	29.8				335	
15:40	5.43	4.72	13.1%				3.35	67	29.8				335	
15:45	5.35	4.66	12.9%				3.46	67	29.8				335	
15:50	5.31	4.63	12.8%				3.48	67	29.8				335	
15:55	5.35	4.68	12.5%				3.44	67	29.8				335	
16:00	5.32	4.70	11.7%				3.48	67	29.8				335	
16:05	5.40	4.70	13.0%				3.40	67	29.8				335	
16:10	5.42	4.65	14.2%				3.50	67	29.8				335	
16:15	5.37	4.62	14.0%				3.55	67	29.8				335	
16:20	5.29	4.52	14.6%				3.53	67	29.8				335	
16:25	5.31	4.52	14.9%				3.46	67	29.8				335	
16:30	5.27	4.51	14.4%				3.50	67	29.8				335	
16:35	5.37	4.50	16.2%				3.56	67	29.8				335	
16:40	5.29	4.67	11.7%				3.56	67	29.8				335	
16:45	5.22	4.63	11.3%				3.58	67	29.8				335	
16:50	5.26	4.64	11.8%				3.57	67	29.8				335	
16:55	5.27	4.65	11.8%				3.59	67	29.8				335	
17:00	5.27	4.68	11.2%				3.62	67	29.8				335	
17:05	5.22	4.68	10.3%				3.56	67	29.8				335	
17:10	5.21	4.68	10.2%				3.63	67	29.8				335	
17:15	5.23	4.75	9.2%				3.63	67	29.8				335	
17:20	5.22	4.66	10.7%				3.67	67	29.8				335	
17:25	5.29	4.65	12.1%				3.80	67	29.8				335	
17:30	5.21	4.62	11.3%				3.71	67	29.8				335	
17:35	5.24	4.60	12.2%				3.71	67	29.8				335	
17:40	5.25	4.56	13.1%				3.74	67	29.8				335	
17:45	5.21	4.56	12.5%				3.69	67	29.8				335	

Run #49	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	%		gallons/day	gallons/day	
17:50	5.18	4.60	11.2%				3.64	67	29.8				335		
17:55	5.19	4.70	9.4%				3.79	67	29.8				335		
18:00	5.19	4.72	9.1%				3.76	67	29.8				335		
18:05	5.19	4.71	9.2%				3.68	66	29.3				330		
18:10	5.24	4.70	10.3%				3.73	67	29.8				335		
18:15	5.17	4.67	9.7%				3.74	67	29.8				335		
18:20	5.21	4.67	10.4%				3.83	67	29.8				335		
18:25	5.33	4.62	13.3%				3.88	67	29.8				335		
18:30	5.20	4.60	11.5%				3.90	66	29.3				330		
18:35	5.09	4.70	7.7%				3.84	66	29.3				330		
18:40	5.09	4.72	7.3%				3.87	67	29.8				335		
18:45	5.15	4.74	8.0%				3.85	66	29.3				330		
18:50	5.17	4.71	8.9%				3.90	66	29.3				330		
18:55	5.17	4.71	8.9%				3.97	66	29.3				330		
19:00								25	11.1	Wash Timer				125	
19:05								30	13.3					150	
19:10								30	13.3					150	**Percent**
19:15								31	13.8					155	**of wash**
19:20								30	13.3					150	**%**
Averages	5.24	4.57	12.7%					64.52	28.68				22175	730	3.19%

9/3/2018
Monday
30% Compression
Run #50

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
19:25							3.16	68	30.2		30	5.2	340	
19:30							3.09	68	30.2				340	
19:35	5.12	4.54	11.3%				3.23	68	30.2				340	
19:40	5.11	4.53	11.4%				3.21	68	30.2				340	
19:45	5.12	4.52	11.7%				3.21	68	30.2				340	
19:50	5.09	4.52	11.2%				3.20	68	30.2				340	
19:55	5.14	4.50	12.5%				3.22	68	30.2				340	
20:00	5.11	4.52	11.5%				3.24	67	29.8				335	
20:05	5.26	4.50	14.4%				3.25	68	30.2				340	
20:10	5.12	4.50	12.1%				3.24	68	30.2				340	
20:15	5.15	4.48	13.0%				3.20	68	30.2				340	
20:20	5.13	4.50	12.3%				3.28	68	30.2				340	
20:25	5.04	4.46	11.5%				3.23	68	30.2				340	
20:30	5.09	4.44	12.8%				3.36	67	29.8				335	
20:35	5.07	4.42	12.8%				3.30	68	30.2				340	
20:40	5.12	4.42	13.7%				3.35	68	30.2				340	
20:45	5.14	4.40	14.4%				3.30	68	30.2				340	
20:50	5.19	4.40	15.2%				3.31	67	29.8				335	
20:55	5.25	4.42	15.8%				3.43	67	29.8				335	
21:00	5.21	4.42	15.2%				3.36	67	29.8				335	
21:05	5.17	4.38	15.3%				3.30	68	30.2				340	
21:10	5.12	4.38	14.5%				3.38	67	29.8				335	
21:15	5.13	4.36	15.0%				3.25	67	29.8				335	
21:20	5.10	4.37	14.3%				3.37	68	30.2				340	
21:25	5.16	4.35	15.7%				3.40	67	29.8				335	
21:30	5.06	4.35	14.0%				3.34	67	29.8				335	
21:35	5.07	4.38	13.6%				3.40	67	29.8				335	
21:40	5.15	4.32	16.1%				3.37	67	29.8				335	
21:45	5.09	4.32	15.1%				3.46	67	29.8				335	
21:50	5.02	4.32	13.9%				3.42	67	29.8				335	
21:55	5.02	4.30	14.3%				3.51	67	29.8				335	
22:00	4.96	4.30	13.3%				3.47	67	29.8				335	
22:05	5.11	4.32	15.5%				3.58	67	29.8				335	
22:10	5.45	4.37	19.8%				3.48	67	29.8				335	
22:15	5.98	4.52	24.4%				3.53	67	29.8				335	
22:20	6.23	4.56	26.8%				3.62	67	29.8				335	
22:25	6.66	4.68	29.7%				3.55	67	29.8				335	
22:30	6.78	4.76	29.8%				3.62	67	29.8				335	
22:35	6.78	4.79	29.4%				3.59	67	29.8				335	
22:40	6.80	4.83	29.0%				3.66	67	29.8				335	
22:45	6.80	4.82	29.1%				3.54	67	29.8				335	
22:50	6.71	4.83	28.0%				3.73	67	29.8				335	
22:55	6.68	4.82	27.8%				3.70	67	29.8				335	
23:00	6.66	4.79	28.1%				3.66	67	29.8				335	
23:05	6.63	4.77	28.1%				3.74	67	29.8				335	
23:10	6.42	4.79	25.4%				3.73	67	29.8				335	
23:15	6.40	4.78	25.3%				3.80	67	29.8				335	
23:20	6.35	4.78	24.7%				3.77	67	29.8				335	
23:25	6.23	4.75	23.8%				3.86	67	29.8				335	
23:30	6.19	4.73	23.6%				3.82	67	29.8				335	
23:35	6.44	4.73	26.6%				3.85	66	29.3				330	
23:40	6.37	4.73	25.7%				3.97	66	29.3				330	

Run #50	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate		Percent Compression	Wash Trip Point	Filter run	wash	
Time	**NTU**	**NTU**	**%**	**mg/l**	**mg/l**	**%**	**psi**	**gpm**	**gpm/ft^2**	**Comments**	**%**		**gallons/day**	**gallons/day**	
23:45	6.43	4.76	26.0%				4.00	66	29.3				330		
23:50	6.55	4.78	27.0%				4.00	66	29.3				330		
23:55	6.62	4.84	26.9%				3.91	66	29.3				330		
0:00	6.68	4.87	27.1%				3.96	66	29.3	9/4/2018			330		
0:05	6.55	4.84	26.1%				3.99	66	29.3				330		
0:10	7.29	4.92	32.5%				4.12	66	29.3				330		
0:15	8.83	5.32	39.8%				4.03	66	29.3				330		
0:20	8.63	5.54	35.8%				4.06	66	29.3				330		
0:25	7.56	5.37	29.0%				4.20	66	29.3				330		
0:30	6.85	5.14	25.0%				4.15	66	29.3				330		
0:35	6.48	5.00	22.8%				4.13	66	29.3				330		
0:40	7.23	5.06	30.0%				4.13	66	29.3				330		
0:45	7.80	5.24	32.8%				4.01	66	29.3				330		
0:50	7.32	5.30	27.6%				4.19	66	29.3				330		
0:55								26	11.6	Wash Timer				130	
1:00								29	12.9					145	
1:05								28	12.4					140	**Percent**
1:10								28	12.4					140	**of wash**
1:15								28	12.4					140	**%**
Averages	5.97	4.65	22.1%					64.25	28.56				22115	695	3.05%

9/4/2018
Tuesday
30% Compression
Run #51

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
1:20							3.16	68	30.2		30	5.2	340	
1:25							3.17	68	30.2				340	
1:30	6.36	4.63	27.2%				3.17	68	30.2				340	
1:35	6.76	4.71	30.3%				3.21	68	30.2				340	
1:40	6.65	4.75	28.6%				3.18	68	30.2				340	
1:45	6.42	4.68	27.1%				3.18	68	30.2				340	
1:50	6.16	4.64	24.7%				3.24	68	30.2				340	
1:55	6.04	4.56	24.5%				3.22	68	30.2				340	
2:00	5.95	4.53	23.9%				3.22	68	30.2				340	
2:05	5.85	4.50	23.1%				3.37	68	30.2				340	
2:10	5.98	4.48	25.1%				3.31	68	30.2				340	
2:15	6.03	4.52	25.0%				3.25	68	30.2				340	
2:20	6.03	4.52	25.0%				3.30	68	30.2				340	
2:25	5.97	4.49	24.8%				3.34	68	30.2				340	
2:30	5.99	4.49	25.0%				3.37	68	30.2				340	
2:35	6.33	4.53	28.4%				3.37	68	30.2				340	
2:40	6.53	4.58	29.9%				3.37	68	30.2				340	
2:45	7.04	4.63	34.2%				3.42	68	30.2				340	
2:50	7.18	4.72	34.3%				3.50	68	30.2				340	
2:55	6.91	4.72	31.7%				3.45	67	29.8				335	
3:00	6.60	4.63	29.8%				3.54	67	29.8				335	
3:05	6.44	4.59	28.7%				3.57	67	29.8				335	
3:10	6.31	4.56	27.7%				3.58	68	30.2				340	
3:15	6.36	4.57	28.1%				3.61	68	30.2				340	
3:20	6.20	4.58	26.1%				3.59	67	29.8				335	
3:25	6.12	4.54	25.8%				3.70	67	29.8				335	
3:30	6.11	4.52	26.0%				3.62	67	29.8				335	
3:35	6.01	4.50	25.1%				3.68	67	29.8				335	
3:40	5.94	4.47	24.7%				3.68	67	29.8				335	
3:45	5.94	4.43	25.4%				3.74	67	29.8				335	
3:50	5.93	4.44	25.1%				3.79	67	29.8				335	
3:55	5.85	4.46	23.8%				3.78	67	29.8				335	
4:00	5.89	4.44	24.6%				3.78	67	29.8				335	
4:05	5.79	4.46	23.0%				3.80	67	29.8				335	
4:10	5.83	4.46	23.5%				3.78	67	29.8				335	
4:15	5.89	4.45	24.4%				3.84	67	29.8				335	
4:20	5.83	4.47	23.3%				3.82	67	29.8				335	
4:25	5.84	4.48	23.3%				3.86	67	29.8				335	
4:30	5.81	4.48	22.9%				3.93	67	29.8				335	
4:35	5.91	4.48	24.2%				3.90	67	29.8				335	
4:40	5.84	4.51	22.8%				3.92	67	29.8				335	
4:45	5.87	4.50	23.3%				3.93	67	29.8				335	
4:50	5.87	4.51	23.2%				3.93	67	29.8				335	
4:55	5.84	4.50	22.9%				3.98	67	29.8				335	
5:00	5.76	4.52	21.5%				3.93	67	29.8				335	
5:05	5.83	4.50	22.8%				3.99	67	29.8				335	
5:10	5.80	4.52	22.1%				4.02	67	29.8				335	
5:15	5.82	4.58	21.3%				3.98	67	29.8				335	
5:20	5.85	4.55	22.2%				4.00	67	29.8				335	
5:25	5.84	4.55	22.1%				3.98	67	29.8				335	
5:30	5.86	4.56	22.2%				4.03	67	29.8				335	
5:35	5.79	4.56	21.2%				4.01	67	29.8				335	

Run #51	Influent Turbidity	Effluent Turbidity	Percent Removal	Influent TSS	Effluent TSS	Percent Removal			Loading Rate			Wash Trip Point	Filter run	wash	
Time	NTU	NTU	%	mg/l	mg/l	%	Pressure psi	Flow gpm	gpm/ft^2	Comments	Percent Compression %		gallons/day	gallons/day	
5:40	5.83	4.55	22.0%				4.03	67	29.8				335		
5:45	5.89	4.54	22.9%				4.02	67	29.8				335		
5:50	6.25	4.60	26.4%				4.08	67	29.8				335		
5:55	6.52	4.73	27.5%				4.07	67	29.8				335		
6:00	6.37	4.74	25.6%				4.05	66	29.3				330		
6:05	6.14	4.70	23.5%				4.07	67	29.8				335		
6:10	5.90	4.70	20.3%				4.02	67	29.8				335		
6:15	5.85	4.58	21.7%				4.02	67	29.8				335		
6:20	5.87	4.58	22.0%				4.10	66	29.3				330		
6:25	5.80	4.62	20.3%				4.00	66	29.3				330		
6:30	5.77	4.65	19.4%				4.06	67	29.8				335		
6:35	5.84	4.61	21.1%				4.05	67	29.8				335		
6:40	5.77	4.61	20.1%				3.97	67	29.8				335		
6:45	5.82	4.61	20.8%				4.03	66	29.3				330		
6:50								33	14.7	Wash Timer				165	
6:55								33	14.7					165	**Percent**
7:00								33	14.7					165	**of wash**
7:05								33	14.7					165	**%**
Averages	6.07	4.56	24.8%					65.30	29.02				22195	660	2.89%

9/4/2018
Tuesday
30% Compression
Run #52

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
7:10							3.19	68	30.2		30	5.2	340	
7:15							3.17	68	30.2				340	
7:20	6.03	4.68	22.4%				3.17	68	30.2				340	
7:25	5.94	4.65	21.7%				3.18	68	30.2				340	
7:30	5.91	4.65	21.3%				3.11	68	30.2				340	
7:35	5.89	4.63	21.4%				3.16	68	30.2				340	
7:40	5.80	4.62	20.3%				3.20	68	30.2				340	
7:45	5.85	4.61	21.2%				3.13	68	30.2				340	
7:50	5.87	4.58	22.0%				3.22	68	30.2				340	
7:55	5.80	4.56	21.4%				3.23	68	30.2				340	
8:00	5.80	4.53	21.9%				3.26	68	30.2	Started Feeding SP-218-GH at 2ml/min			340	
8:05	5.77	4.53	21.5%				3.25	68	30.2				340	
8:10	5.74	4.52	21.3%				3.26	67	29.8				335	
8:15	5.70	4.25	25.4%				3.26	67	29.8				335	
8:20	5.67	4.01	29.3%				3.30	67	29.8				335	
8:25	5.67	4.00	29.5%				3.33	67	29.8				335	
8:30	5.41	3.58	33.8%				3.42	68	30.2				340	
8:35	5.60	3.22	42.5%				3.52	67	29.8				335	
8:40	5.76	3.08	46.5%				3.45	67	29.8				335	
8:45	5.70	2.96	48.1%				3.49	67	29.8				335	
8:50	5.56	2.90	47.8%				3.55	67	29.8				335	
8:55	5.70	2.88	49.5%				3.55	67	29.8				335	
9:00	5.63	2.84	49.6%				3.57	67	29.8				335	
9:05	5.81	2.84	51.1%				3.66	67	29.8				335	
9:10	5.67	2.82	50.3%				3.75	67	29.8				335	
9:15	5.57	2.81	49.6%				3.67	67	29.8				335	
9:20	5.64	2.80	50.4%				3.84	66	29.3				330	
9:25	5.73	2.78	51.5%				3.76	67	29.8				335	
9:30	5.77	2.78	51.8%				3.79	67	29.8				335	
9:35	5.42	2.78	48.7%				3.78	67	29.8				335	
9:40	5.49	2.78	49.4%				3.81	66	29.3				330	
9:45	5.44	2.78	48.9%				3.76	66	29.3				330	
9:50	5.64	2.88	48.9%				3.79	66	29.3				330	
9:55	5.55	2.83	49.0%				3.78	67	29.8				335	
10:00	5.55	2.83	49.0%				3.80	67	29.8				335	
10:05	5.29	2.87	45.7%				3.72	66	29.3				330	
10:10	5.29	2.89	45.4%				3.80	66	29.3				330	
10:15	5.31	2.89	45.6%				3.81	66	29.3				330	
10:20	5.06	2.92	42.3%				3.83	66	29.3				330	
10:25	5.29	2.93	44.6%				3.81	67	29.8				335	
10:30	5.36	2.99	44.2%				3.68	66	29.3				330	
10:35	5.57	3.00	46.1%				3.75	66	29.3				330	
10:40	5.41	3.07	43.3%				3.72	66	29.3				330	
10:45	5.75	3.16	45.0%				3.77	66	29.3				330	
10:50	5.44	3.29	39.5%				3.78	67	29.8				335	
10:55	5.04	3.38	32.9%				3.82	66	29.3				330	
11:00	5.02	3.46	31.1%				3.72	66	29.3				330	
11:05	5.04	3.60	28.6%				3.82	66	29.3				330	
11:10	4.95	3.81	23.0%				3.76	66	29.3				330	
11:15	5.10	4.04	20.8%				3.71	66	29.3				330	

Run #52 Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
11:20	4.91	4.21	14.3%				3.71	66	29.3	Solids sliding up the plex-glass window, because the polymer they are feeding gums the media.			330		
11:25	4.90	4.38	10.6%				3.70	66	29.3				330		
11:30	5.55	4.70	15.3%				3.67	66	29.3				330		
11:35	5.17	4.88	5.6%				3.84	66	29.3				330		
11:40							3.76	67	29.8				335		
11:45							3.76	67	29.8				335		
11:50							3.68	66	29.3				330		
11:55								0		Manual Wash				0	
12:00								29	12.9	They lower the wash timer to 3.5 hours				145	
12:05								29	12.9					145	**Percent**
12:10								29	12.9					145	**of wash**
12:15								28	12.4					140	**%**
Averages	5.53	3.55	35.8%					63.31	28.60				19050	575	2.93%

9/4/2018
Tuesday
30% Compression
Run #53

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
12:20							3.31	68	30.2	2 ml/min SP-218-GH	30	5.2	340		
12:25							3.29	68	30.2				340		
12:30	5.47	3.41	37.7%				3.32	68	30.2				340		
12:35	5.61	3.30	41.2%				3.32	68	30.2				340		
12:40	5.54	3.24	41.5%				3.38	68	30.2				340		
12:45	5.57	3.21	42.4%				3.35	68	30.2				340		
12:50	5.81	3.20	44.9%				3.51	68	30.2				340		
12:55	5.96	3.17	46.8%				3.56	68	30.2				340		
13:00	5.89	3.16	46.3%				3.53	68	30.2				340		
13:05	5.99	3.16	47.2%				3.50	68	30.2				340		
13:10	5.99	3.16	47.2%				3.60	67	29.8				335		
13:15	5.81	3.14	46.0%				3.61	68	30.2				340		
13:20	5.77	3.17	45.1%				3.60	67	29.8				335		
13:25	5.89	3.16	46.3%				3.64	67	29.8				335		
13:30	5.69	3.17	44.3%				3.73	67	29.8				335		
13:35	5.92	3.18	46.3%				3.75	67	29.8				335		
13:40	5.91	3.16	46.5%				3.82	67	29.8				335		
13:45	6.10	3.16	48.2%				3.77	67	29.8				335		
13:50	5.94	3.15	47.0%				3.84	67	29.8				335		
13:55	5.93	3.15	46.9%				3.82	67	29.8				335		
14:00	5.79	3.22	44.4%				3.81	67	29.8				335		
14:05	5.87	3.22	45.1%				3.89	67	29.8				335		
14:10	6.03	3.31	45.1%				3.89	67	29.8				335		
14:15	5.98	3.36	43.8%				3.87	67	29.8				335		
14:20	5.84	3.26	44.2%				3.83	67	29.8				335		
14:25	5.87	3.25	44.6%				3.93	67	29.8				335		
14:30	5.72	3.16	44.8%				3.95	66	29.3				330		
14:35	5.83	3.14	46.1%				3.95	67	29.8				335		
14:40	5.88	3.16	46.3%				4.06	67	29.8				335		
14:45	5.67	3.22	43.2%				3.98	67	29.8				335		
14:50	5.77	3.18	44.9%				3.98	67	29.8				335		
14:55	5.81	3.16	45.6%				4.00	66	29.3				330		
15:00	5.79	3.16	45.4%				3.89	67	29.8				335		
15:05	6.11	3.17	48.1%				3.94	67	29.8				335		
15:10	5.90	3.17	46.3%				4.01	66	29.3				330		
15:15	6.01	3.29	45.3%				3.98	66	29.3				330		
15:20	5.86	3.36	42.7%				3.95	67	29.8				335		
15:25	6.21	3.48	44.0%				4.03	67	29.8				335		
15:30	5.87	3.59	38.8%				3.99	67	29.8				335		
15:35	5.77	3.56	38.3%				4.03	66	29.3				330		
15:40	5.91	3.58	39.4%				4.01	67	29.8				335		
15:45	5.99	3.58	40.2%				3.97	67	29.8				335		
15:50								0		Manual Wash				0	
15:55								28	12.4	Lowered Wash Timer to 3 hours				140	
16:00								28	12.4					140	**Percent**
16:05								28	12.4					140	**of wash**
16:10								28	12.4					140	**%**
Averages	5.86	3.25	44.5%					62.38	28.33				14100	560	3.82%

9/4/2018
Tuesday
30% Compression
Run #54

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
16:15							3.22	68	30.2	2 ml/min SP-218-GH	30	5.2	340		
16:20							3.26	68	30.2				340		
16:25	5.63	3.48	38.2%				3.33	68	30.2				340		
16:30	5.50	3.44	37.5%				3.38	68	30.2				340		
16:35	5.67	3.36	40.7%				3.48	68	30.2				340		
16:40	5.74	3.33	42.0%				3.46	67	29.8				335		
16:45	5.88	3.29	44.0%				3.48	67	29.8				335		
16:50	5.77	3.28	43.2%				3.52	68	30.2				340		
16:55	5.82	3.26	44.0%				3.57	67	29.8				335		
17:00	5.75	3.38	41.2%				3.61	67	29.8				335		
17:05	5.77	3.29	43.0%				3.67	67	29.8				335		
17:10	5.61	3.25	42.1%				3.71	67	29.8				335		
17:15	5.90	3.24	45.1%				3.68	67	29.8				335		
17:20	5.99	3.23	46.1%				3.75	67	29.8				335		
17:25	5.90	3.39	42.5%				3.78	68	30.2				340		
17:30	6.23	3.37	45.9%				3.78	67	29.8				335		
17:35	5.91	3.37	43.0%				3.86	67	29.8				335		
17:40	5.89	3.35	43.1%				3.85	68	30.2				340		
17:45	5.73	3.38	41.0%				3.87	67	29.8				335		
17:50	6.03	3.40	43.6%				3.88	67	29.8				335		
17:55	5.84	3.38	42.1%				3.89	67	29.8				335		
18:00	5.71	3.36	41.2%				3.90	67	29.8				335		
18:05	5.90	3.32	43.7%				3.89	67	29.8				335		
18:10	5.89	3.24	45.0%				3.99	67	29.8				335		
18:15	5.77	3.39	41.2%				3.96	68	30.2				340		
18:20	5.80	3.36	42.1%				3.97	67	29.8				335		
18:25	5.89	3.43	41.8%				4.01	67	29.8				335		
18:30	5.67	3.42	39.7%				4.01	67	29.8				335		
18:35	6.03	3.41	43.4%				4.01	67	29.8				335		
18:40	5.68	3.37	40.7%				4.05	67	29.8				335		
18:45	5.71	3.36	41.2%				4.03	67	29.8				335		
18:50	5.52	3.32	39.9%				4.04	67	29.8				335		
18:55	5.64	3.32	41.1%				4.02	67	29.8				335		
19:00	5.87	3.32	43.4%				4.05	67	29.8				335		
19:05	6.07	3.30	45.6%				4.05	67	29.8				335		
19:10	6.00	3.30	45.0%				3.99	67	29.8				335		
19:15	5.79	3.32	42.7%				4.03	67	29.8				335		
19:20	5.83	3.48	40.3%				4.05	67	29.8				335		
19:25	5.88	3.55	39.6%				4.00	67	29.8				335		
19:30	5.56	3.56	36.0%				3.99	67	29.8				335		
19:35	5.55	3.66	34.1%				4.01	67	29.8				335		
19:40	5.65	3.74	33.8%				3.98	67	29.8	3.5 hours			335		
19:45								31	13.8	Wash Timer				155	
19:50								31	13.8					155	**Percent**
19:55								31	13.8					155	**of wash**
20:00								31	13.8					155	**%**
Averages	5.80	3.38	41.8%					64.07	28.47				14115	620	4.21%

9/4/2018
Tuesday
30% Compression
Run #55

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
20:05							3.27	69	30.7	2 ml/min SP-218-GH	30	5.2	345		
20:10							3.28	68	30.2				340		
20:15	5.52	3.52	36.2%				3.31	68	30.2				340		
20:20	5.66	3.44	39.2%				3.31	68	30.2				340		
20:25	5.58	3.38	39.4%				3.40	68	30.2				340		
20:30	5.64	3.36	40.4%				3.42	67	29.8				335		
20:35	5.37	3.32	38.2%				3.41	67	29.8				335		
20:40	5.49	3.30	39.9%				3.48	67	29.8				335		
20:45	5.79	3.28	43.4%				3.52	67	29.8				335		
20:50	5.87	3.27	44.3%				3.61	67	29.8				335		
20:55	6.01	3.26	45.8%				3.69	67	29.8				335		
21:00	5.52	3.24	41.3%				3.74	67	29.8				335		
21:05	5.55	3.21	42.2%				3.60	67	29.8				335		
21:10	5.85	3.20	45.3%				3.73	67	29.8				335		
21:15	5.57	3.20	42.5%				3.81	67	29.8				335		
21:20	5.77	3.20	44.5%				3.89	66	29.3				330		
21:25	5.74	3.20	44.3%				3.83	67	29.8				335		
21:30	5.85	3.20	45.3%				3.82	67	29.8				335		
21:35	5.95	3.20	46.2%				3.92	67	29.8				335		
21:40	5.67	3.20	43.6%				3.86	66	29.3				330		
21:45	5.87	3.20	45.5%				3.88	66	29.3				330		
21:50	5.69	3.25	42.9%				3.85	67	29.8				335		
21:55	5.74	3.27	43.0%				3.80	67	29.8				335		
22:00	5.85	3.24	44.6%				3.89	67	29.8				335		
22:05	5.79	3.20	44.7%				3.93	67	29.8				335		
22:10	5.57	3.20	42.5%				3.87	67	29.8				335		
22:15	5.57	3.20	42.5%				3.86	67	29.8				335		
22:20	5.84	3.21	45.0%				3.91	67	29.8				335		
22:25	6.37	3.23	49.3%				3.83	67	29.8				335		
22:30	6.09	3.29	46.0%				3.90	67	29.8				335		
22:35	5.99	3.28	45.2%				3.85	67	29.8				335		
22:40	6.37	3.25	49.0%				3.89	66	29.3				330		
22:45	6.45	3.28	49.1%				3.84	66	29.3				330		
22:50	6.21	3.29	47.0%				3.94	66	29.3				330		
22:55	6.08	3.31	45.6%				3.86	66	29.3				330		
23:00	6.12	3.34	45.4%				3.87	67	29.8				335		
23:05	6.26	3.30	47.3%				3.80	67	29.8				335		
23:10	6.11	3.35	45.2%				3.84	67	29.8				335		
23:15	6.48	3.46	46.6%				3.84	67	29.8				335		
23:20	6.27	3.55	43.4%				3.80	67	29.8				335		
23:25	6.03	3.72	38.3%				3.82	67	29.8				335		
23:30	6.28	3.86	38.5%				3.81	66	29.3				330		
23:35	6.25	4.07	34.9%				3.84	66	29.3				330		
23:40								29	12.9	Wash Timer				145	
23:45								29	12.9					145	**Percent**
23:50								29	12.9					145	**of wash**
23:55								29	12.9					145	**%**
Averages	5.89	3.33	43.6%					63.70	28.31				14390	580	3.87%

9/5/2018
Wednesday
30% Compression
Run #56

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
0:00							3.29	67	29.8	2 ml/min SO-218-GH	30	5.2	335		
0:05							3.28	68	30.2				340		
0:10	5.67	3.46	39.0%				3.30	67	29.8				335		
0:15	5.81	3.41	41.3%				3.42	67	29.8				335		
0:20	6.11	3.36	45.0%				3.38	67	29.8				335		
0:25	5.75	3.37	41.4%				3.39	67	29.8				335		
0:30	5.65	3.34	40.9%				3.45	67	29.8				335		
0:35	5.67	3.31	41.6%				3.54	67	29.8				335		
0:40	5.89	3.29	44.1%				3.56	67	29.8				335		
0:45	5.77	3.28	43.2%				3.62	67	29.8				335		
0:50	6.07	3.27	46.1%				3.66	67	29.8				335		
0:55	5.74	3.27	43.0%				3.62	66	29.3				330		
1:00	6.90	3.25	52.9%				3.64	67	29.8				335		
1:05	6.61	3.30	50.1%				3.73	67	29.8				335		
1:10	6.45	3.30	48.8%				3.76	67	29.8				335		
1:15	6.40	3.28	48.8%				3.76	66	29.3				330		
1:20	6.11	3.26	46.6%				3.79	66	29.3				330		
1:25	6.13	3.26	46.8%				3.82	66	29.3				330		
1:30	5.92	3.25	45.1%				3.75	66	29.3				330		
1:35	6.06	3.24	46.5%				3.85	67	29.8				335		
1:40	5.67	3.21	43.4%				3.81	66	29.3				330		
1:45	5.93	3.17	46.5%				3.86	67	29.8				335		
1:50	5.68	3.17	44.2%				3.95	66	29.3				330		
1:55	5.79	3.18	45.1%				3.85	66	29.3				330		
2:00	5.74	3.19	44.4%				3.80	66	29.3				330		
2:05	5.87	3.20	45.5%				3.83	66	29.3				330		
2:10	5.74	3.25	43.4%				3.80	67	29.8				335		
2:15	5.83	3.24	44.4%				3.87	66	29.3				330		
2:20	5.71	3.27	42.7%				3.83	66	29.3				330		
2:25	5.74	3.34	41.8%				3.78	67	29.8				335		
2:30	5.66	3.43	39.4%				3.77	66	29.3				330		
2:35	5.72	3.62	36.7%				3.80	66	29.3				330		
2:40	5.49	3.82	30.4%				3.78	66	29.3				330		
2:45	5.36	4.01	25.2%				3.81	66	29.3				330		
2:50	5.50	4.22	23.3%				3.76	66	29.3				330		
2:55	5.43	4.52	16.8%				3.78	67	29.8				335		
3:00	5.91	4.76	19.5%				3.64	66	29.3				330		
3:05	6.49	5.09	21.6%				3.74	67	29.8				335		
3:10	6.56	5.34	18.6%				3.81	66	29.3				330		
3:15	6.49	5.42	16.5%				3.74	66	29.3				330		
3:20	6.79	5.54	18.4%				3.73	66	29.3				330		
3:25	6.15	5.79	5.9%				3.73	67	29.8				335		
3:30								0		Wash Timer				0	
3:35								29	12.9					145	
3:40								28	12.4					140	**Percent**
3:45								28	12.4					140	**of wash**
3:50								28	12.4					140	**%**
Averages	5.95	3.68	38.1%					61.85	28.09				13970	565	3.89%

9/5/2018
Wednesday
30% Compression
Run #57

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
3:55							3.41	67	29.8	2ml/min SP-218-GH	30	5.2	335		
4:00							3.28	67	29.8				335		
4:05	5.94	3.38	43.1%				3.32	67	29.8				335		
4:10	5.71	3.30	42.2%				3.35	67	29.8				335		
4:15	6.01	3.24	46.1%				3.41	67	29.8				335		
4:20	6.00	3.24	46.0%				3.47	67	29.8				335		
4:25	5.83	3.22	44.8%				3.52	67	29.8				335		
4:30	6.59	3.23	51.0%				3.53	67	29.8				335		
4:35	6.19	3.23	47.8%				3.52	67	29.8				335		
4:40	6.12	3.21	47.5%				3.63	67	29.8				335		
4:45	6.34	3.22	49.2%				3.63	67	29.8				335		
4:50	6.05	3.21	46.9%				3.69	66	29.3				330		
4:55	6.15	3.20	48.0%				3.75	67	29.8				335		
5:00	6.20	3.20	48.4%				3.75	67	29.8				335		
5:05	6.54	3.20	51.1%				3.79	66	29.3				330		
5:10	7.04	3.21	54.4%				3.82	66	29.3				330		
5:15	6.82	3.21	52.9%				3.88	66	29.3				330		
5:20	6.78	3.20	52.8%				3.88	66	29.3				330		
5:25	6.15	3.19	48.1%				3.88	67	29.8				335		
5:30	6.08	3.18	47.7%				3.88	66	29.3				330		
5:35	6.55	3.17	51.6%				3.96	67	29.8				335		
5:40	6.27	3.19	49.1%				3.93	64	28.4				320		
5:45	6.70	3.19	52.4%				3.93	64	28.4				320		
5:50	6.13	3.17	48.3%				3.89	64	28.4				320		
5:55	6.29	3.19	49.3%				3.94	64	28.4				320		
6:00	6.29	3.19	49.3%				3.94	64	28.4				320		
6:05	6.56	3.21	51.1%				3.93	64	28.4				320		
6:10	6.56	3.20	51.2%				3.96	64	28.4				320		
6:15	6.24	3.21	48.6%				3.93	64	28.4				320		
6:20	6.43	3.28	49.0%				3.93	64	28.4				320		
6:25	6.39	3.29	48.5%				3.91	64	28.4				320		
6:30	6.42	3.34	48.0%				3.85	64	28.4				320		
6:35	6.63	3.37	49.2%				3.92	64	28.4				320		
6:40	6.49	3.39	47.8%				3.81	64	28.4				320		
6:45	6.35	3.47	45.4%				3.88	64	28.4				320		
6:50	6.42	3.56	44.5%				3.87	64	28.4				320		
6:55	6.26	3.63	42.0%				3.86	64	28.4				320		
7:00	6.45	3.74	42.0%				3.83	64	28.4				320		
7:05	6.27	3.88	38.1%				3.80	64	28.4				320		
7:10	6.32	4.10	35.1%				3.83	64	28.4				320		
7:15	6.43	4.42	31.3%				3.84	64	28.4				320		
7:20								0		Wash Timer				0	
7:25								30	13.3					150	
7:30								29	12.9					145	**Percent**
7:35								29	12.9					145	**of wash**
7:40								29	12.9					145	**%**
Averages	6.33	3.34	47.3%					60.83	27.63				13405	585	4.18%

9/5/2018
Wednesday
30% Compression
Run #58

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
7:45							3.25	65	28.9	2ml/min SP-218-GH	30	5.2	325		
7:50							3.29	65	28.9				325		
7:55	6.10	3.37	44.8%				3.30	65	28.9				325		
8:00	6.04	3.31	45.2%				3.28	65	28.9				325		
8:05	5.92	3.26	44.9%				3.41	65	28.9				325		
8:10	6.03	3.25	46.1%				3.47	65	28.9				325		
8:15	6.17	3.20	48.1%				3.48	65	28.9				325		
8:20	6.20	3.17	48.9%				3.46	65	28.9				325		
8:25	5.99	3.15	47.4%				3.56	64	28.4				320		
8:30	6.03	3.12	48.3%				3.58	64	28.4				320		
8:35	6.28	3.12	50.3%				3.67	64	28.4				320		
8:40	6.25	3.09	50.6%				3.67	64	28.4				320		
8:45	6.18	3.07	50.3%				3.68	65	28.9				325		
8:50	6.53	3.06	53.1%				3.67	64	28.4				320		
8:55	6.31	3.03	52.0%				3.81	64	28.4				320		
9:00	6.20	3.02	51.3%				3.74	64	28.4				320		
9:05	6.37	3.02	52.6%				3.81	64	28.4				320		
9:10	6.28	3.02	51.9%				3.82	64	28.4				320		
9:15	6.25	3.02	51.7%				3.86	64	28.4				320		
9:20	6.29	2.98	52.6%				3.83	64	28.4				320		
9:25	5.93	2.94	50.4%				3.87	64	28.4				320		
9:30	6.00	2.97	50.5%				3.84	64	28.4				320		
9:35	6.85	2.99	56.4%				3.90	64	28.4				320		
9:40	6.58	3.00	54.4%				3.77	64	28.4				320		
9:45	6.50	3.04	53.2%				3.85	64	28.4				320		
9:50	7.09	3.09	56.4%				3.85	64	28.4				320		
9:55	6.90	3.12	54.8%				3.86	64	28.4				320		
10:00	6.66	3.16	52.6%				3.82	64	28.4				320		
10:05	6.53	3.22	50.7%				3.87	64	28.4				320		
10:10	6.43	3.33	48.2%				3.87	64	28.4				320		
10:15	6.56	3.44	47.6%				3.85	64	28.4				320		
10:20	6.38	3.56	44.2%				3.85	64	28.4				320		
10:25	6.46	3.59	44.4%				3.79	64	28.4				320		
10:30	7.21	3.78	47.6%				3.82	64	28.4				320		
10:35	7.05	3.92	44.4%				3.82	64	28.4				320		
10:40	6.87	4.01	41.6%				3.79	64	28.4				320		
10:45								0		Wash Timer				0	
10:50								30	13.3					150	
10:55								30	13.3					150	**Percent**
11:00								30	13.3					150	**of wash**
11:05								30	13.3					150	**%**
Averages	6.39	3.22	49.7%					59.34	27.03				11565	600	4.93%

9/5/2018
Wednesday
30% compression
Run #59

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
11:10	7.20	4.44	38.3%				3.24	64	28.4	2 ml/min SP-218-GH	30	5.2	320		
11:15	6.00	3.50	41.7%				3.25	65	28.9				325		
11:20	6.40	3.37	47.3%				3.29	65	28.9				325		
11:25	6.25	3.34	46.6%				3.35	64	28.4				320		
11:30	6.13	3.29	46.3%				3.35	64	28.4				320		
11:35	6.55	3.26	50.2%				3.37	64	28.4				320		
11:40	6.30	3.24	48.6%				3.46	64	28.4				320		
11:45	6.31	3.23	48.8%				3.52	64	28.4				320		
11:50	6.29	3.22	48.8%				3.57	64	28.4				320		
11:55	6.57	3.22	51.0%				3.52	64	28.4				320		
12:00	6.79	3.22	52.6%				3.60	64	28.4				320		
12:05	6.65	3.20	51.9%				3.62	64	28.4				320		
12:10	6.54	3.20	51.1%				3.63	64	28.4				320		
12:15	7.27	3.20	56.0%				3.70	64	28.4				320		
12:20	7.06	3.19	54.8%				3.69	64	28.4				320		
12:25	8.52	3.19	62.6%				3.73	64	28.4				320		
12:30	8.15	3.21	60.6%				3.76	64	28.4				320		
12:35	6.70	3.20	52.2%				3.77	64	28.4				320		
12:40	6.90	3.20	53.6%				3.79	64	28.4				320		
12:45	7.41	3.21	56.7%				3.81	64	28.4				320		
12:50	7.51	3.22	57.1%				3.86	63	28.0				315		
12:55	7.38	3.20	56.6%				3.80	64	28.4				320		
13:00	7.56	3.21	57.5%				3.89	64	28.4				320		
13:05	7.19	3.32	53.8%				3.90	64	28.4				320		
13:10	7.52	3.34	55.6%				3.89	64	28.4				320		
13:15	7.39	3.30	55.3%				3.96	64	28.4				320		
13:20	7.48	3.27	56.3%				3.92	63	28.0				315		
13:25	7.34	3.23	56.0%				3.95	63	28.0				315		
13:30	7.56	3.22	57.4%				3.95	64	28.4				320		
13:35	7.52	3.26	56.6%				3.94	64	28.4				320		
13:40	7.93	3.25	59.0%				3.96	64	28.4				320		
13:45	7.92	3.26	58.8%				3.91	64	28.4				320		
13:50	7.96	3.29	58.7%				3.96	63	28.0				315		
13:55	7.68	3.30	57.0%				3.94	64	28.4				320		
14:00	7.97	3.34	58.1%				3.98	63	28.0				315		
14:05	8.27	3.37	59.3%				3.92	64	28.4				320		
14:10								31	13.8	Wash Timer				155	
14:15								32	14.2					160	**Percent**
14:20								32	14.2					160	**of wash**
14:25								32	14.2					160	
Averages	7.17	3.29	54.1%					60.70	26.98				11505	635	5.23%

9/5/2018
Wednesday
30% Compression
Run #60

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
14:30							3.25	65	28.9	2ml/min SP-218-Gh	30	5.02	325		
14:35							3.22	65	28.9				325		
14:40	6.15	3.66	40.5%				3.31	65	28.9				325		
14:45	5.97	3.55	40.5%				3.30	64	28.4				320		
14:50	5.79	3.50	39.6%				3.49	64	28.4				320		
14:55	6.01	3.47	42.3%				3.46	64	28.4				320		
15:00	6.15	3.44	44.1%				3.47	65	28.9	Turned off SP-218-GH			325		
15:05	6.05	3.42	43.5%				3.58	64	28.4				320		
15:10	6.25	3.40	45.6%				3.55	64	28.4				320		
15:15	6.25	3.40	45.6%				3.60	64	28.4				320		
15:20	6.61	3.40	48.6%				3.64	64	28.4				320		
15:25	7.18	3.70	48.5%				3.70	64	28.4				320		
15:30	7.01	4.01	42.8%				3.61	64	28.4				320		
15:35	7.37	4.16	43.6%				3.74	64	28.4				320		
15:40	7.28	4.27	41.3%				3.69	64	28.4				320		
15:45	6.73	4.34	35.5%				3.68	64	28.4				320		
15:50	6.81	4.38	35.7%				3.73	64	28.4				320		
15:55	6.77	4.47	34.0%				3.78	64	28.4				320		
16:00	6.68	4.50	32.6%				3.73	64	28.4				320		
16:05	6.81	4.55	33.2%				3.77	64	28.4				320		
16:10	6.83	4.75	30.5%				3.71	64	28.4				320		
16:15	6.62	4.76	28.1%				3.76	64	28.4				320		
16:20	6.55	4.76	27.3%				3.77	64	28.4				320		
16:25	6.83	4.75	30.5%				3.78	64	28.4				320		
16:30	6.61	4.75	28.1%				3.75	64	28.4				320		
16:35	6.75	4.80	28.9%				3.81	64	28.4				320		
16:40	6.49	4.86	25.1%				3.80	64	28.4				320		
16:45	6.44	4.90	23.9%				3.82	64	28.4				320		
16:50	6.45	4.97	22.9%				3.86	64	28.4				320		
16:55	6.83	4.99	26.9%				3.88	64	28.4				320		
17:00	6.64	4.96	25.3%				3.84	64	28.4				320		
17:05	6.49	4.97	23.4%				3.88	64	28.4				320		
17:10	6.58	4.97	24.5%				3.90	64	28.4				320		
17:15	6.50	4.96	23.7%				3.89	64	28.4				320		
17:20	6.65	4.94	25.7%				3.92	64	28.4				320		
17:25	6.65	4.93	25.9%				3.96	64	28.4				320		
17:30	6.50	4.89	24.8%				4.01	64	28.4				320		
17:35								30	13.3	Wash Timer				150	
17:40								30	13.3					150	**Percent**
17:45								30	13.3					150	**of wash**
17:50								30	13.3					150	**%**
Averages	6.58	4.36	33.8%					60.78	27.01				11860	600	4.82%

9/5/2018
Wednesday
30% Compression
Run #61

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
17:55							3.22	65	28.9	SP-218-GH OFF	30	5.2	325		
18:00							3.22	65	28.9				325		
18:05	6.38	4.96	22.3%				3.30	65	28.9				325		
18:10	6.36	4.98	21.7%				3.22	65	28.9				325		
18:15	6.55	5.02	23.4%				3.25	65	28.9				325		
18:20	6.37	5.04	20.9%				3.23	65	28.9				325		
18:25	6.32	5.06	19.9%				3.24	65	28.9				325		
18:30	6.34	5.06	20.2%				3.31	65	28.9				325		
18:35	6.43	5.06	21.3%				3.32	65	28.9				325		
18:40	6.50	5.10	21.5%				3.32	65	28.9				325		
18:45	6.41	5.06	21.1%				3.27	65	28.9				325		
18:50	6.49	5.04	22.3%				3.34	65	28.9				325		
18:55	6.41	5.24	18.3%				3.36	65	28.9				325		
19:00	6.33	5.17	18.3%				3.35	65	28.9				325		
19:05	6.36	5.15	19.0%				3.33	65	28.9				325		
19:10	6.35	5.15	18.9%				3.38	64	28.4				320		
19:15	6.46	5.05	21.8%				3.39	65	28.9				325		
19:20	6.33	5.10	19.4%				3.45	65	28.9				325		
19:25	6.39	5.09	20.3%				3.44	65	28.9				325		
19:30	6.33	5.10	19.4%				3.44	65	28.9				325		
19:35	6.50	5.11	21.4%				3.53	64	28.4				320		
19:40	6.56	5.10	22.3%				3.49	65	28.9				325		
19:45	6.57	5.10	22.4%				3.50	65	28.9				325		
19:50	6.42	5.11	20.4%				3.50	64	28.4				320		
19:55	6.43	5.09	20.8%				3.54	65	28.9				325		
20:00	6.47	5.06	21.8%				3.53	64	28.4				320		
20:05	6.39	5.05	21.0%				3.60	64	28.4				320		
20:10	6.49	5.04	22.3%				3.56	64	28.4				320		
20:15	6.43	5.00	22.2%				3.55	64	28.4				320		
20:20	6.39	5.07	20.7%				3.58	64	28.4				320		
20:25	6.53	5.07	22.4%				3.63	64	28.4				320		
20:30	6.38	5.08	20.4%				3.64	64	28.4				320		
20:35	6.60	5.10	22.7%				3.63	65	28.9				325		
20:40	6.41	5.06	21.1%				3.67	64	28.4				320		
20:45	6.37	5.03	21.0%				3.69	64	28.4				320		
20:50	6.33	5.04	20.4%				3.66	64	28.4				320		
20:55								35	15.6	Wash Timer				175	
21:00								29	12.9					145	
21:05								30	13.3					150	**Percent**
21:10								29	12.9					145	**of wash**
21:15								29	12.9					145	**%**
Averages	6.42	5.07	21.0%					60.46	26.87				11635	760	6.13%

9/5/2018
Wednesday
30% Compression
Run #62

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
21:20							3.20	64	28.4	SP-218-GH OFF	30	5.2	320		
21:25							3.16	64	28.4				320		
21:30	6.35	4.90	22.8%				3.31	64	28.4				320		
21:35	6.38	4.92	22.9%				3.16	64	28.4				320		
21:40	6.27	4.91	21.7%				3.21	64	28.4				320		
21:45	6.41	4.92	23.2%				3.24	64	28.4				320		
21:50	6.32	4.92	22.2%				3.22	64	28.4				320		
21:55	6.38	4.90	23.2%				3.26	63	28.0				315		
22:00	6.35	4.89	23.0%				3.24	64	28.4				320		
22:05	6.31	4.88	22.7%				3.29	63	28.0				315		
22:10	6.35	4.88	23.1%				3.27	64	28.4				320		
22:15	6.28	4.83	23.1%				3.30	63	28.0				315		
22:20	6.28	4.85	22.8%				3.31	63	28.0				315		
22:25	6.41	4.94	22.9%				3.27	63	28.0				315		
22:30	6.34	4.90	22.7%				3.36	63	28.0				315		
22:35	6.39	4.93	22.8%				3.36	64	28.4				320		
22:40	6.30	4.91	22.1%				3.29	63	28.0				315		
22:45	6.32	4.91	22.3%				3.38	63	28.0				315		
22:50	6.35	4.89	23.0%				3.40	63	28.0				315		
22:55	6.28	4.94	21.3%				3.37	63	28.0				315		
23:00	6.34	4.90	22.7%				3.37	63	28.0				315		
23:05	6.35	4.88	23.1%				3.46	63	28.0				315		
23:10	6.34	4.86	23.3%				3.48	63	28.0				315		
23:15	6.28	4.86	22.6%				3.52	63	28.0				315		
23:20	6.35	4.88	23.1%				3.44	63	28.0				315		
23:25	6.25	4.84	22.6%				3.44	63	28.0				315		
23:30	6.27	4.84	22.8%				3.53	63	28.0				315		
23:35	6.27	4.84	22.8%				3.60	63	28.0				315		
23:40	6.24	4.83	22.6%				3.58	63	28.0				315		
23:45	6.27	4.84	22.8%				3.57	63	28.0				315		
23:50	6.23	4.83	22.5%				3.65	63	28.0				315		
23:55	6.25	4.84	22.6%				3.66	63	28.0				315		
0:00	6.29	4.88	22.4%				3.66	63	28.0				315		
0:05	6.28	4.88	22.3%				3.68	63	28.0				315		
0:10	6.25	4.89	21.8%				3.75	62	27.6				310		
0:15	6.23	4.88	21.7%				3.77	63	28.0				315		
0:20	6.33	4.89	22.7%				3.66	63	28.0				315		
0:25								30	13.3	Wash Timer				150	
0:30								29	12.9					145	**Percent**
0:35								29	12.9					145	**of wash**
0:40								29	12.9					145	**%**
Averages	6.31	4.88	22.6%					59.93	26.63				11700	585	4.76%

9/6/2018
Thursday
30% Compression
Run #63

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
0:45	6.66	5.02	24.6%				3.17	64	28.4	SP-218-GH OFF	30	5.2	320		
0:50	6.63	4.97	25.0%				3.17	64	28.4				320		
0:55	6.50	4.97	23.5%				3.28	64	28.4				320		
1:00	6.44	4.98	22.7%				3.23	64	28.4				320		
1:05	6.39	4.96	22.4%				3.20	64	28.4				320		
1:10	6.47	4.96	23.3%				3.16	64	28.4				320		
1:15	6.96	5.02	27.9%				3.24	64	28.4				320		
1:20	7.31	5.14	29.7%				3.21	64	28.4				320		
1:25	7.33	5.13	30.0%				3.27	64	28.4				320		
1:30	7.43	5.14	30.8%				3.34	64	28.4				320		
1:35	7.37	5.08	31.1%				3.22	64	28.4				320		
1:40	6.96	5.02	27.9%				3.39	64	28.4				320		
1:45	6.88	4.93	28.3%				3.37	63	28.0				315		
1:50	6.67	4.91	26.4%				3.44	63	28.0				315		
1:55	6.65	4.88	26.6%				3.42	64	28.4				320		
2:00	6.70	4.87	27.3%				3.38	63	28.0				315		
2:05	6.73	4.90	27.2%				3.45	64	28.4				320		
2:10	6.62	4.88	26.3%				3.47	63	28.0				315		
2:15	6.61	4.88	26.2%				3.45	63	28.0				315		
2:20	6.53	4.84	25.9%				3.45	63	28.0				315		
2:25	6.52	4.84	25.8%				3.56	63	28.0				315		
2:30	6.49	4.83	25.6%				3.59	63	28.0				315		
2:35	6.57	4.86	26.0%				3.60	63	28.0				315		
2:40	6.66	4.86	27.0%				3.62	63	28.0				315		
2:45	6.78	4.88	28.0%				3.61	63	28.0				315		
2:50	7.06	4.92	30.3%				3.66	63	28.0				315		
2:55	7.06	4.94	30.0%				3.79	63	28.0				315		
3:00	7.42	5.03	32.2%				3.75	63	28.0				315		
3:05	7.57	5.07	33.0%				3.77	63	28.0				315		
3:10	7.26	5.08	30.0%				3.77	62	27.6				310		
3:15	7.03	5.01	28.7%				3.83	62	27.6				310		
3:20	7.32	5.10	30.3%				3.84	62	27.6				310		
3:25	7.39	5.14	30.4%				3.83	62	27.6				310		
3:30	7.23	5.14	28.9%				3.82	62	27.6				310		
3:35	6.96	5.06	27.3%				3.89	62	27.6				310		
3:40	7.06	5.09	27.9%				3.91	62	27.6				310		
3:45	7.39	5.14	30.4%					0		Wash Timer				0	
3:50								31	13.8				155	155	
3:55								31	13.8				155	155	**Percent**
4:00								30	13.3				150	150	**of wash**
4:05								31	13.8				155	155	**%**
Average	6.91	4.99	27.8%					58.49	26.64				11990	615	4.88%

9/6/2018
Thursday
30% Compression
Run #64

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
4:10							3.18	63	28.0	SP-218-GH OFF	30	5.2	315		
4:15							3.16	64	28.4				320		
4:20	7.62	5.20	31.8%				3.20	63	28.0				315		
4:25	7.28	5.15	29.3%				3.15	63	28.0				315		
4:30	7.17	5.14	28.3%				3.19	63	28.0				315		
4:35	7.09	5.12	27.8%				3.17	63	28.0				315		
4:40	7.00	5.09	27.3%				3.24	63	28.0				315		
4:45	7.06	5.08	28.0%				3.26	63	28.0				315		
4:50	7.09	5.05	28.8%				3.29	63	28.0				315		
4:55	7.60	5.12	32.6%				3.23	63	28.0				315		
5:00	7.68	5.22	32.0%				3.35	63	28.0				315		
5:05	7.62	5.14	32.5%				3.35	63	28.0				315		
5:10	7.28	5.05	30.6%				3.36	63	28.0				315		
5:15	7.22	5.02	30.5%				3.38	63	28.0				315		
5:20	7.09	4.98	29.8%				3.43	63	28.0				315		
5:25	7.09	4.97	29.9%				3.42	63	28.0				315		
5:30	7.15	5.00	30.1%				3.45	63	28.0				315		
5:35	7.31	5.06	30.8%				3.46	63	28.0				315		
5:40	7.11	5.02	29.4%				3.50	63	28.0				315		
5:45	7.04	4.99	29.1%				3.50	62	27.6				310		
5:50	7.00	4.98	28.9%				3.55	62	27.6				310		
5:55	6.95	4.98	28.3%				3.57	63	28.0				315		
6:00	6.94	4.97	28.4%				3.61	63	28.0				315		
6:05	6.93	4.91	29.1%				3.61	62	27.6				310		
6:10	6.99	4.97	28.9%				3.65	62	27.6				310		
6:15	6.98	4.96	28.9%				3.65	63	28.0				315		
6:20	6.88	4.97	27.8%				3.69	62	27.6				310		
6:25	6.90	4.96	28.1%				3.69	62	27.6				310		
6:30	6.93	4.98	28.1%				3.73	62	27.6				310		
6:35	6.90	4.96	28.1%				3.72	62	27.6				310		
6:40	6.78	4.96	26.8%				3.77	62	27.6				310		
6:45	6.91	5.01	27.5%				3.79	63	28.0				315		
6:50	6.77	4.97	26.6%				3.80	62	27.6				310		
6:55	6.77	4.94	27.0%				3.80	63	28.0				315		
7:00	6.77	4.96	26.7%				3.83	62	27.6				310		
7:05	6.69	4.96	25.9%				3.88	62	27.6				310		
7:10								0						0	
7:15								30	13.3					150	
7:20								30	13.3					150	**Percent**
7:25								30	13.3					150	**of wash**
7:30								30	13.3					150	**%**
Averages	7.08	5.02	29.0%					57.98	26.41				11285	600	5.05%

9/6/2018
Thursday
30% Compression
Run #65

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
7:35							3.14	64	28.4	SP-218-GH back on at 2ml/min	30	5.2	320		
7:40							3.18	63	28.0				315		
7:45	6.26	3.74	40.3%				3.16	64	28.4				320		
7:50	6.60	3.65	44.7%				3.24	64	28.4				320		
7:55	6.72	3.58	46.7%				3.28	63	28.0				315		
8:00	6.55	3.53	46.1%				3.23	63	28.0				315		
8:05	6.37	3.50	45.1%				3.39	63	28.0				315		
8:10	6.41	3.47	45.9%				3.41	63	28.0				315		
8:15	6.66	3.48	47.7%				3.47	63	28.0				315		
8:20	6.62	3.46	47.7%				3.47	63	28.0				315		
8:25	6.71	3.47	48.3%				3.55	63	28.0				315		
8:30	6.56	3.43	47.7%				3.55	63	28.0				315		
8:35	6.37	3.42	46.3%				3.61	63	28.0				315		
8:40	6.65	3.42	48.6%				3.66	63	28.0				315		
8:45	6.41	3.41	46.8%				3.64	63	28.0				315		
8:50	6.61	3.40	48.6%				3.68	63	28.0				315		
8:55	6.38	3.38	47.0%				3.63	63	28.0				315		
9:00	6.39	3.39	46.9%				3.73	63	28.0				315		
9:05	6.41	3.38	47.3%				3.71	62	27.6				310		
9:10	6.26	3.37	46.2%				3.71	62	27.6				310		
9:15	6.38	3.37	47.2%				3.77	63	28.0				315		
9:20	7.13	3.38	52.6%				3.80	63	28.0				315		
9:25	6.61	3.38	48.9%				3.85	63	28.0				315		
9:30	6.25	3.37	46.1%				3.82	62	27.6				310		
9:35	6.16	3.37	45.3%				3.83	62	27.6				310		
9:40	6.43	3.36	47.7%				3.86	62	27.6				310		
9:45	6.40	3.37	47.3%				3.84	62	27.6				310		
9:50	6.73	3.40	49.5%				3.86	62	27.6				310		
9:55	6.31	3.40	46.1%				3.86	62	27.6				310		
10:00	6.46	3.42	47.1%				3.85	62	27.6				310		
10:05	6.45	3.42	47.0%				3.87	62	27.6				310		
10:10	6.27	3.42	45.5%				3.91	62	27.6				310		
10:15	6.26	3.40	45.7%				3.90	63	28.0				315		
10:20	6.33	3.40	46.3%				3.90	62	27.6				310		
10:25	6.07	3.42	43.7%				3.86	63	28.0				315		
10:30								0						0	
10:35								30	13.3					150	
10:40								30	13.3					150	**Percent**
10:45								30	13.3					150	**of wash**
10:50								30	13.3					150	**%**
Averages	6.46	3.44	46.8%					57.90	26.39				10980	600	5.18%

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Hydraulic Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
													0	
													0	
													0	

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
													0	
													0	
													0	

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
													0	
													0	
													0	

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
													0	
													0	
													0	

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day
													0	
													0	
													0	

Time	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Pressure psi	Flow gpm	Loading Rate gpm/ft^2	Comments	Percent Compression %	Wash Trip Point	Filter run gallons/day	wash gallons/day	
													0		
													0		
													0		
													0		
													0		
													0		
													0		
													0		
													0		
													0		
													0		
													0		
													0		**Percent**
													0		**of wash**
													0		**%**
Averages													0	0	#DIV/0!

APPENDIX 2

Average Summary Sheet and Plant Data

Run Averages

SP-218-GH

Date	Run	Run Time mins.	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Flow gpm	Hydraulic Loading gpm/ft2	Total Filter Gallons	Total Wash Water Gallons	Percent Wash Water %	Type Of Wash	Compression %	Comments
8/21/2018	1	405	7.61	5.69	25.3	17.09	8.88	48.1	66.26	29.79	28070	420	1.47	M	20	Started Run No polymer. Added polymer SP-218-GH at 2ml/min to 800,000 GPD flow. This is the lowest amount that can be feed.
8/21/2018	2	450	7.53	5.21	30.7				65.67	29.50	30430	435	1.41	T	30	2ml/min SP-218-GH
8/22/2018	3	450	7.09	5.01	29.4				66.34	29.48	30935	575	1.82	T	30	2ml/min SP-218-GH
8/22/2018	4	340	6.84	4.42	35.3	15.45	6.08	60.6	65.31	29.42	23495	670	2.77	M	30	2ml/min SP-218-GH
8/22/2018	5	330	7.15	4.58	35.9	15.67	6.63	57.7	65.53	29.12	22320	615	2.68	T	40	2ml/min SP-218-GH. Had to shorten the Wash Timer and make the wash cycle longer.
8/22/2018	6	330	6.88	4.76	30.8				64.91	29.27	22270	450	1.98	T	40	2ml/min SP-218-GH
8/23/2018	7	330	6.37	4.81	24.4				65.27	29.01	22550	620	2.65	T	40	2ml/min SP-218-GH
8/23/2018	8	225	6.39	4.18	34.6	15.34	5.75	62.5	63.70	28.31	15455	470	2.95	M	40	2ml/min SP-218-GH and added media
8/23/2018	9	330	6.46	4.03	37.6	14.00	6.58	53.0	59.77	26.95	20640	580	2.73	T	40	2ml/min SP-218-GH
8/23/2018	10	330	6.23	4.51	27.6				60.69	27.36	20905	640	2.97	T	40	2ml/min SP-218-GH
8/24/2018	11	330	6.09	4.43	27.3				61.69	27.42	21340	560	2.56	T	40	2ml/min SP-218-GH
8/24/2018	12	240	5.78	3.84	33.5	13.33	5.50	58.7	60.02	27.18	15640	565	3.49	M	40	2ml/min SP-218-GH
8/24/2018	13	330	6.02	3.78	37.2	12.33	4.92	60.1	61.54	27.74	21265	580	2.66	T	30	2ml/min SP-218-GH
8/24/2018	14	330	5.96	4.07	31.7				62.50	27.78	21310	565	2.58	T	30	2ml/min SP-218-GH
8/24/2018	15	330	5.87	4.14	29.5				62.52	27.79	21625	570	2.57	T	30	2ml/min SP-218-GH
8/25/2018	16	330	5.73	4.07	29.0				61.66	27.41	21240	650	2.97	T	30	2ml/min SP-218-GH
8/25/2018	17	330	5.56	3.52	36.6				60.99	27.10	21090	560	2.59	T	20	2ml/min SP-218-GH
8/25/2018	18	330	5.43	3.48	35.8				60.04	27.07	20735	580	2.72	T	20	2ml/min SP-218-GH
8/25/2018	19	330	5.49	3.57	34.9				60.17	27.12	20775	585	2.74	T	20	2ml/min SP-218-GH
8/26/2018	20	330	5.64	3.40	39.6				61.16	27.18	20825	580	2.71	T	20	2ml/min SP-218-GH. Turn off PS-218 and Chlorine washed the filter.
9/4/2018	52	285	5.53	3.55	35.8				63.31	28.60	19050	575	2.93	M	30	Started Back Feeding SP-218-GH 2ml/min. They saw solids sliding up the glass wall of the filter due to the polymer they are feeding. Lowered wash Timer.
9/4/2018	53	210	5.86	3.25	44.5				62.38	28.33	14100	560	3.82	M	30	SP-218-GH 2ml/min
9/4/2018	54	210	5.80	3.38	41.8				64.07	28.47	14115	620	4.21	T	30	SP-218-GH 2ml/min
9/4/2018	55	210	5.89	3.33	43.6				63.70	28.31	14390	580	3.87	T	30	SP-218-GH 2ml/min
9/5/2018	56	210	5.95	3.68	38.1				61.85	28.09	13971	565	3.89	T	30	SP-218-GH 2ml/min
9/5/2018	57	210	6.33	3.34	47.3				60.83	27.63	13405	585	4.18	T	30	SP-218-GH 2ml/min
9/5/2018	58	180	6.39	3.22	49.7				59.34	27.03	11565	600	4.93	T	30	SP-218-GH 2ml/min Also lower Wash Timer
9/5/2018	59	180	7.17	3.29	54.1				60.70	26.98	11505	635	5.23	T	30	SP-218-GH 2ml/min
9/6/2018	65	180	6.46	3.44	46.8				57.90	26.39	10980	600	5.18	T	30	SP-218-GH 2ml/min
Average		296.7	6.26	4.00	36.15	14.74	6.33	57.24	62.41	27.99	19517.1	572.1	3.08			

JC- 1679

Date	Run	Run Time mins.	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Flow gpm	Hydraulic Loading gpm/ft2	Total Filter Gallons	Total Wash Water Gallons	Percent Wash Water %	Type Of Wash	Compression %	Comments
8/27/2018	21	330	5.45	3.90	28.4	11.67	6.17	47.1	67.65	30.07	23400	615	2.56	T	20	Started Feeding JC-1679 at 2ml/min to 800,000 GPD Flow.
8/27/2018	22	330	5.45	3.77	30.8	11.33	6.25	44.8	67.86	30.16	25525	600	2.30	T	20	2ml/min JC-1679
8/27/2018	23	330	6.08	3.83	37.0				67.35	29.93	25370	560	2.16	T	20	2ml/min JC-1680
8/28/2018	24	390	5.40	3.59	33.5	11.33	6.00	47.0	66.92	30.10	27180	590	2.12	T	20	2ml/min JF-1679 and increased Wash Timer
8/28/2018	25	390	5.57	3.64	34.6	11.89	3.83	67.8	66.70	30.01	27040	640	2.31	T	30	4ml/min JC-1679
8/28/2018	26	390	5.53	3.69	33.3				66.54	29.93	26985	630	2.28	T	30	4ml/min JC-1680
8/29/2018	27	390	5.53	3.62	34.5				67.01	29.78	26840	635	2.31	T	30	4ml/min JC-1681
8/29/2018	28	390	5.47	3.62	33.9	12.50	5.94	52.5	70.89	31.51	29260	640	2.14	T	30	Started Run at 4ml/min JC-1679 Changed to 6ml/min 4.4 hours into the run.
8/29/2018	29	510	5.35	3.60	32.7	15.00	6.00	60.0	75.68	33.95	40185	680	1.66	T	40	6ml/min JC-1679 Increased Wash Timer time and Flow to filter.
8/29/2018	30	240	5.41	3.45	36.2				74.85	33.91	19195	640	3.23	P	40	6ml/min JC-1679
8/30/2018	31	350	5.33	3.53	33.9	11.00	6.00	45.5	75.09	33.37	27575	585	2.08	M	40	4ml/min JC-1679, lower the flow to the filter and Visitors
8/30/2018	32	330	5.18	3.26	37.0	12.33	5.00	59.4	67.24	29.88	23285	585	2.45	T	30	4ml/min JC-1679
Average		364.2	5.48	3.63	33.82	12.13	5.65	53.01	69.48	31.05	26820.0	616.7	2.30			

Date	Run	Run Time mins.	Influent Turbidity NTU	Effluent Turbidity NTU	Percent Removal %	Influent TSS mg/l	Effluent TSS mg/l	Percent Removal %	Flow gpm	Hydraulic Loading gpm/ft2	Total Filter Gallons	Total Wash Water Gallons	Percent Wash Water %	Type Of Wash	Compression %	Comments
Possibly no JC-1679 being added																
8/30/2018	33	330	5.16	4.19	18.8				67.38	29.95	23300	620	2.59	T	30	??4ml/min JC-1679 The drop off of Turbidity removal?
8/30/2018	34	330	5.05	4.03	20.2				66.59	29.60	23020	620	2.62	T	30	??4ml/min JC-1679 The drop off of Turbidity removal?
8/31/2018	35	330	5.19	4.05	21.9				65.34	29.45	22555	640	2.76	T	30	??4ml/min JC-1679 The drop off of Turbidity removal?
8/31/2017	36	330	5.06	4.02	20.7				65.23	29.40	23155	580	2.44	T	30	??4ml/min JC-1679 The drop off of Turbidity removal?
Average		330	5.12	4.07	20.40				66.14	29.60	23007.5	615.0	2.60			
NO SP-218-GH or JC-1679 being Added																
8/31/2018	37	330	5.10	4.06	20.6				65.94	29.31	22520	560	2.43	T	30	JC-1679 turned off 1 hour into run.
8/31/2018	38	330	5.03	3.96	21.3				65.94	29.31	22810	600	2.56	T	30	No Chemicals
9/1/2018	39	330	5.03	3.96	21.3				65.94	29.31	22810	600	2.56	T	30	
9/1/2018	40	330	5.07	3.90	23.1				65.94	29.31	22770	640	2.73	T	30	
9/1/2018	41	330	5.04	3.92	22.2				65.89	29.28	22775	615	2.63	T	30	
9/1/2018	42	330	5.15	4.07	20.9				63.52	28.63	22005	545	2.42	T	30	
9/2/2017	43	330	5.43	4.00	26.3				62.85	28.33	21770	540	2.42	T	30	
9/2/2018	44	330	5.44	3.94	27.5				62.49	28.17	21530	555	2.50	T	30	
9/2/2018	45	330	5.27	3.97	24.8				63.31	28.14	21715	760	3.38	T	30	
9/2/2018	46	330	5.29	4.15	21.6				64.25	28.56	22180	630	2.76	T	30	
9/3/2018	47	330	5.39	4.27	20.6				63.48	28.62	21915	620	2.75	T	30	
9/3/2018	48	330	5.11	4.27	16.4				64.77	29.20	22440	555	2.41	T	30	
9/3/2018	49	330	5.24	4.57	12.7				64.52	28.68	22175	730	3.19	T	30	
9/3/2018	50	330	5.97	4.65	22.1				64.25	28.56	22115	695	3.05	T	30	
9/4/2018	51	330	6.07	4.56	24.8				65.30	29.02	22195	660	2.89	T	30	
9/5/2018	60	180	6.58	4.36	33.8				60.78	27.01	11860	600	4.82	T	30	Turn off SP-218-GH
9/5/2018	61	180	6.42	5.07	21.0				60.46	26.87	11635	760	6.13	T	30	
9/5/2018	62	180	6.31	4.88	22.6				59.93	26.63	11700	585	4.76	T	30	
9/6/2018	63	180	6.91	4.99	27.8				58.49	26.64	11990	615	4.88	T	30	
9/6/2018	64	180	7.08	5.02	29.0				57.98	26.41	11285	600	5.05	T	30	
Average		292.5	5.65	4.33	23.02				63.30	28.30	19609.75	623.3	3.32			

All results are in mg/L

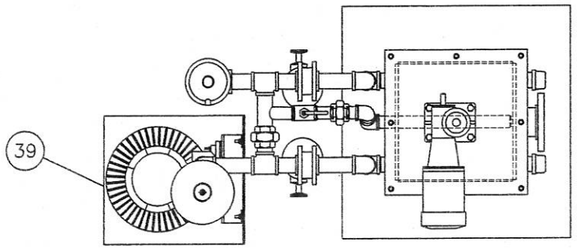
Date	Sand Filter		Fuzzy Filter	
	Influer	Effluent	Influent	Effluent
8/22/2018	12	4	13	4
8/23/2018	9	4	11	3
8/26/2018				
8/27/2018	7	3	8	1
8/28/2018	6	2	6	1
8/29/2018	8	4	7	4
9/4/2018	11	5	12	2
9/5/2018	11	4	12	3
9/6/2018	14	5	10	3
9/7/2018	13	5	12	2
9/10/2018	10	3	9	2
9/11/2018	7	2	10	1
9/12/2018	11	3	10	2

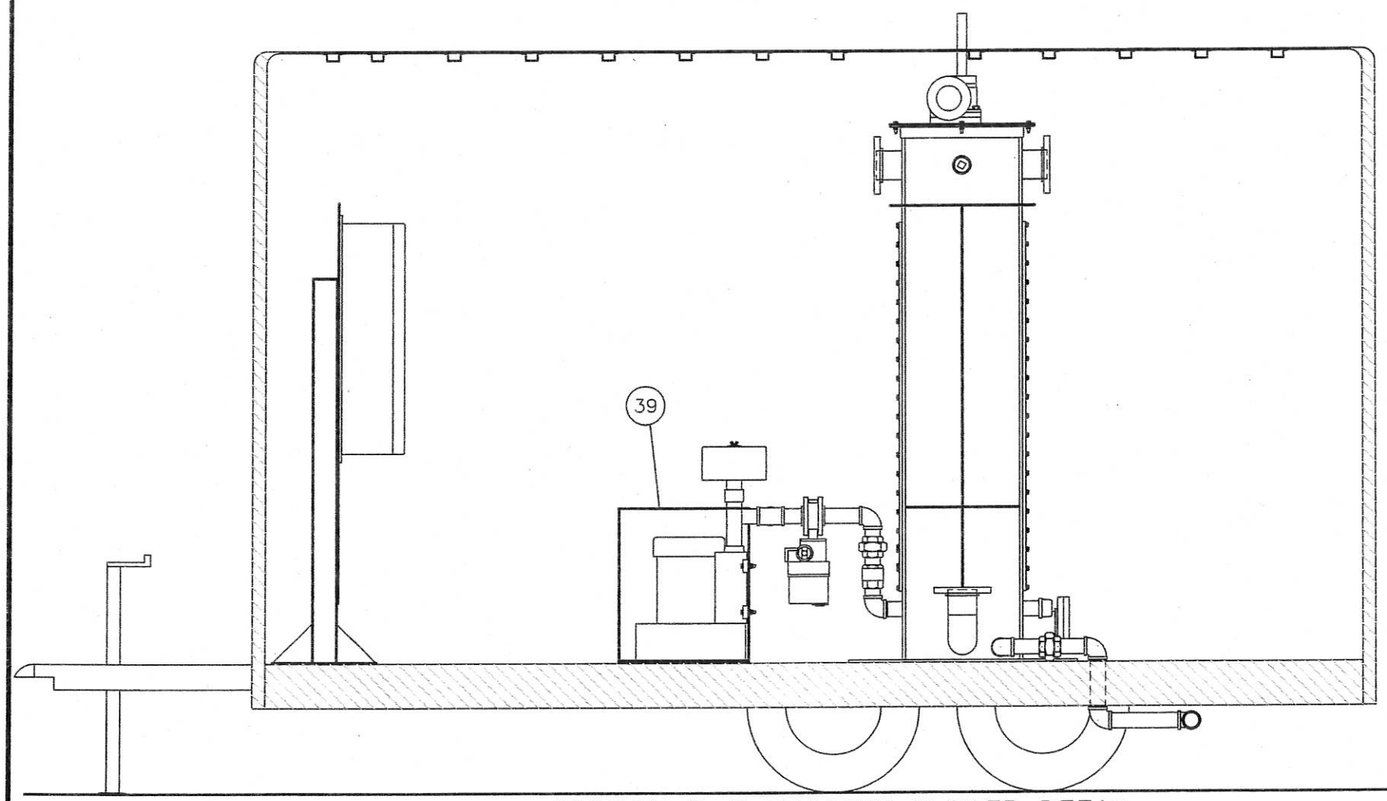
This are the samples that the Snohomish plant people took and Tested.

APPENDIX 3

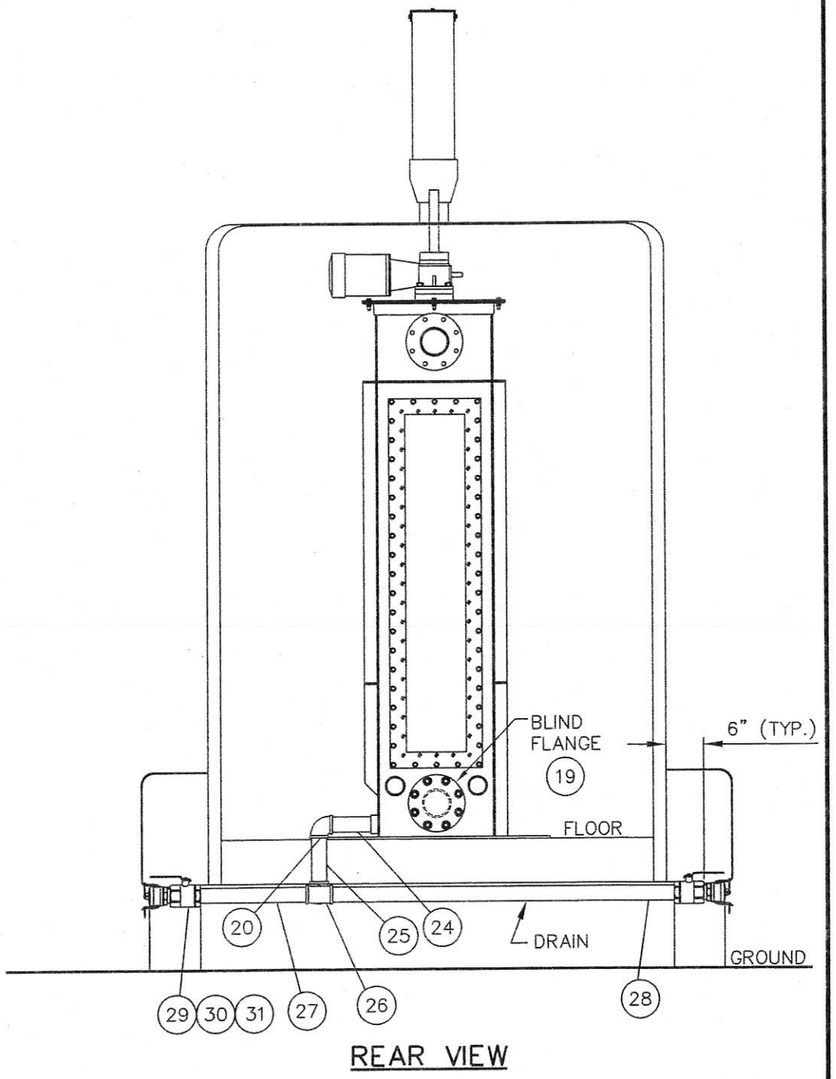
Pilot Filter Drawing

NOTICE: THIS DRAWING IS THE PROPERTY OF SCHREIBER CORP., INC. AND IS LOANED SUBJECT TO THE CONDITION THAT IT SHALL NOT BE REPRODUCED, COPIED, LOANED OR OTHERWISE DISPOSED OF DIRECTLY OR INDIRECTLY. IT SHALL BE USED AS A MEANS OF REFERENCE TO WORK FURNISHED BY THIS COMPANY ONLY AND IS NOT TO BE SUBMITTED TO OUTSIDE PARTIES FOR EXAMINATION WITHOUT OUR CONSENT.





SECTION B-B SHOWING BLOWER DETAIL



REAR VIEW

REV.	BY	DATE	DESCRIPTION
2	djp	8/07/03	REDESIGNED INFLUENT PIPE AND MODIFIED DRAIN
1	CBW	08/14/02	ADDED DETAIL "D"

SIGN	DATE
DRAWN: CBW	06/10/02
CHECK:	
APP'D:	
SCALE: 1"=1'	WEIGHT:

JOB NUMBER	CLIENT
2156	WEF SHOW UNIT

TITLE: TRAILER MOUNTED FUZZY FILTER

EQUIP: MODEL 18"x18"

SCHREIBER CORPORATION, INC.
TRUSSVILLE, ALABAMA

DWG. SIZE	SHEET 4 OF 4	CAD NO. ODJ24D
B	DWG. NO. 50-4900-00024	REVISION 2