COMPREHENSIVE

SEWER SYSTEM PLAN

June 2019
Revised April 2020

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SEWER SYSTEM PLAN

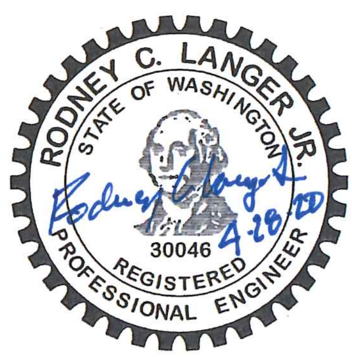
BIRCH BAY WATER AND SEWER DISTRICT

WHATCOM COUNTY, WASHINGTON

June 2019
Revised April 2020

CHS ENGINEERS, LLC

This report was prepared under the supervision of a Registered Professional Engineer.



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BIRCH BAY

WATER AND SEWER DISTRICT

Birch Bay, Washington

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Birch Bay Sewer and Water District

COMPREHENSIVE SEWER SYSTEM PLAN

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SUMMARY AND RECOMMENDATIONS

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SUMMARY AND RECOMMENDATIONS

BACKGROUND

The subject of this report is the public wastewater collection system and wastewater treatment plant (WWTP) owned, operated and maintained by Birch Bay Water and Sewer District. The District is a municipal corporation in the form of a special purpose district, subject to RCW 57. The District provides public water and sewer service to an unincorporated portion of northwestern Whatcom County. Much of this area has been designated as the Birch Bay Urban Growth Area (UGA) in the Whatcom County *Comprehensive Plan*.

Birch Bay Water and Sewer District is located immediately south of the City of Blaine (see Figure 1.1). The District is bordered on the west by Birch Bay and Semiahmoo Bay and on the north by the Blaine city limits and Drayton Harbor. The eastern boundary line is irregular ranging from a minimum of 1,000 feet to over one and one half miles east of Birch Bay. The southern boundary is also irregular from Alderson Road south to Grandview Road. The District presently includes approximately 6,670 acres. The sewer service area and District boundary are shown on Figure 1.2.

The District was formed as Whatcom County Water District No. 8 in 1968. The first wastewater engineering report was completed in 1970 and the WWTP went into service in December 1976. The District office and WWTP are located at 7096 Point Whitehorn Road, Birch Bay, Washington 98320-96745, telephone 360-371-7100, facsimile 360-371-2806. The WWTP discharges to the Strait of Georgia under NPDES Permit WA 002955-6 (see Appendix D) and per the terms of an aquatic lands lease (see Appendix E).

This plan has been prepared in the context of the following land use planning documents.

- September 2004 (revised May 2009) *Whatcom County Birch Bay Community Plan* (BBCP)
- *Whatcom County Population and Employment Projection and Urban Growth Area Allocations Phase I Technical Report* (revised November 1, 2013, BERK)
- November 2015 *Whatcom County 2016 Comprehensive Plan and Development Regulations Update and Urban Growth Areas Review Environmental Impact Statement*
- *Whatcom County Comprehensive Plan* (2016)

The following is a summary of the *Comprehensive Sewer System Plan*, presenting the principal findings and recommendations of this report.

SEWER SERVICE AREA

The sewer service area designated in this plan is all the area within the Birch Bay UGA as designated by Whatcom County, and plats and parcels outside of the present UGA already served by the District sewer system. The sewer service area does not include all area within the current District boundary. The service area includes the following areas outside of the District boundary: a parcel in Blaine south of the Semiahmoo development, the Loomis Trail development and golf course area, an area northeast of the intersection of Blaine Road and Birch Bay – Lynden Road and an area southeast of the intersection of Alderson Road and Blaine Road.

The District also provides domestic wastewater service for the BP Cherry Point Refinery by agreement. The refinery is located outside the District boundary and direct sewer service area.

POPULATION

The annual population growth rate within the Birch Bay UGA increased an average of 1.56% per year from 2010 to 2013. The projected annual growth rate from 2013 through 2036 in the Birch Bay UGA is 2.3353%. In the sewer service area outside the UGA, the population growth rate varies by select study area. In addition, there are several miscellaneous rural parcels currently being served. These parcels are assumed to have no growth over the planning period. Collectively the effective annual growth rate outside the UGA is forecast to be 0.41%. The population in the District sewer service area is forecast to reach 9,386 in 2020 and 13,643 in 2038.

The baseline for the forecast is 2013, based on the County's detailed analysis. The forecast population for the end of 2018 is 8,982. Based on actual growth in sewer connections for 2013-2018, the estimated sewer service area population is about 8,350, 630 persons or about 7.0% below the forecast. This can be interpreted that growth is lagging the forecast by about three and one-half years.

DESIGN CRITERIA

The design criteria used in this comprehensive plan is based on *Criteria for Sewage Works Design* established by the State of Washington Department of Ecology, District historical design criteria, actual usage records and other accepted standards for wastewater system design and construction.

SEWER SYSTEM

Birch Bay Water and Sewer District has been providing sewer service for the Birch Bay area since the fall of 1976 when the District's 0.5 million gallon per day (mgd) wastewater treatment plant went into operation. The plant capacity was expanded to 1.0 mgd in 1986 and again to 1.28 mgd in 1999. With the completion of aeration system improvements in 2016, the permitted capacity increased to 1.44 mgd. For the period 2009 through 2017, the maximum daily flow was 3.351 mgd (December 2010) and the maximum month average daily flow was 1.276 mgd (January 2018).

The collection system is composed of nearly 63 miles of gravity and pressure sewers including 11 pump stations. The pump stations along Birch Bay Drive are the backbone of the collection system.

CAPITAL IMPROVEMENT PLAN

This *Comprehensive Sewer System Plan* identifies projects that will be necessary to extend sewer service throughout the Birch Bay Urban Growth Area. Several of the projects are recommended for completion over the next ten years, as summarized in Table S.1.

RECOMMENDATIONS

On the basis of the information presented in this report, it is recommended that Birch Bay Water and Sewer District:

1. Conduct a public hearing to receive input on the plan; specifically, the Capital Improvement Plan. Review and update the Capital Improvement Plan annually.

2. Adopt the *Comprehensive Sewer System Plan* for improvements as set forth herein.

3. Submit copies of this report to appropriate regulatory agencies for approval.

4. Review and update the sewer general facilities charge based on the adopted Capital Improvement Plan.

5. Complete the recommended wastewater treatment plant and collection system improvements.

6. Continue evaluation of the sewer collection system for excessive inflow and infiltration and implement measures to eliminate such flows.

7. Periodically review the Plan and update it to conform to actual growth patterns and population levels and to remain consistent with land use designations in the sewer service area.

TABLE S.1

10-YEAR CAPITAL IMPROVEMENT PLAN (2019-2028)

ID	Capital Improvement	Project Description	Estimated Project Cost (2019 $)	Recommended Year of Completion
		I. WWTP		
T-1	Toxicity Testing	Acute and Chronic Toxicity Testing	$ 12,000	2024
T-2	Evaluation	Outfall Evaluation and Effluent Mixing Study	$ 25,000	2024
T-3	NPDES	NPDES Permit Renewal	$ 1,000	2024
T-4	WWTP Eng. Report	Engineering Report Update, including evaluation in support of Projects T-7, T-8 and T-9	$ 60,000	2020
T-5	UV Upgrade	Replace UV Modules with high output modules (4 total) and replace appurtenances	$ 373,000	2019
T-6	Headworks Odor Control Upgrades	Implement upgrades, per findings of Project T-4, for odor and moisture containment and additional blower with scrubber (see note 1)	$ 170,000	2020
T-7	Biosolids Management Upgrade	Implement first phase of upgrades, per findings of Project T-4, for improved solids thickening, digester cover and replacement south basin diffusers (see note 1)	$ 200,000	2019 & 2020
T-8	WWTP Site Work	Misc. site improvements to address paving and drainage deficiencies (Consider coordinating timing to follow access road water main work)	$ 84,000	2021
T-9	WWTP Upgrades	Aeration/Clarification Upgrades based on 2012 WWTP Engineering Report, as refined by Project T-4 (see note 1)	$ 6,800,000	2022-2024
		Subtotal for WWTP Projects	**$ 7,725,000**	

TABLE S.1

10-YEAR CAPITAL IMPROVEMENT PLAN (2019-2028)

ID	Capital Improvement	Project Description	Estimated Project Cost (2019 $)	Recommended Year of Completion
		II. PUMPING		
P-1	PS #8 Structure	Replace top slab of generator vault.	$ 268,000	2019
P-2	PS #3 Pump Upgrade Phase 1	Evaluate/replace ex. pumps to restore to nameplate capacity/increase capacity to 3,800 gpm, for full use of 16" force main, including replacement generator (approx. 10 years of growth capacity, anticipated to be accommodated in ex. structures and piping)	$ 520,000	2020-2021
P-3	PS #4 and FM Upgrade Phase 1	Upgrade PS #4 to 3,400 gpm (approximately 10 years of growth capacity), including 16" replacement force main crossing of Terrel Creek and replacement generator (Configuration to be confirmed with Project O-1 but additional wet well and generator vault not anticipated)	$ 556,000	2021-2022
P-4	PS BR Upgrade	Upgrade PS BR to 1,000 gpm capacity (upgrades will require larger generator and motor controls centers, but upgrade is anticipated to be accommodated in ex. structures and piping)	$ 312,000	2022-2023
P-5	PS #5 and FM Upgrade	Upgrade PS #5 to 2,600 gpm, including replacement 12" force main split to two interceptors. Include manual transfer switch and receptacle for portable standby power connection.	$ 337,000	2024
P-6	PS #6 and FM Upgrade	Upgrade PS #6 to 2,200 gpm, including replacement generator and replacement 12" force main split to two interceptors.	$ 480,000	2026
P-7	PS #7 and FM Upgrade	Upgrade PS #7 to 1,600 gpm, including replacement 10" force main and split to two interceptors.	$ 303,000	2028
		Subtotal for Pumping Projects	**$ 2,776,000**	

TABLE S.1

10-YEAR CAPITAL IMPROVEMENT PLAN (2019-2028)

ID	Capital Improvement	Project Description	Estimated Project Cost (2019 $)	Recommended Year of Completion
		III. COLLECTION		
C-1	Collection System Evaluation & Repair - I/I	Continue evaluation of sanitary sewer system to identify sources of extraneous wastewater flow and continue program for remediation	$ 315,000	2020-2038
C-2a	24" - PS #3 to #4 - Ph I	Install 2,201 ft. of 24" parallel gravity sewer interceptor from MH743-040 to 743-034 (consider 36" diameter to support feasibility of gravity bypass of PS #4 from Alderson Rd. interceptor, to be confirmed by Project O-1)	$ 1,363,000	2020-2021
C-3a	18" - PS #4 to #5 - Ph I	Install 1,242 ft. of 18" parallel gravity sewer interceptor from MH 743-117A to 743-095. Modify piping at PS #5 to allow gravity overflow from wet well to new interceptor	$ 703,000	2023
C-4a	15" - PS #5 to #6 - Ph I	Install 1,931 ft. of 15" parallel gravity sewer interceptor from MH 743-126 to 743-121	$ 966,000	2025
C-5a	15" - PS #6 to #7 - Ph I	Install 1,551 ft. of 15" parallel gravity sewer interceptor from MH 742-050A to 742-037. Modify piping at PS #7 to allow gravity overflow from wet well to new interceptor	$ 776,000	2027
C-6	12" - PS #8 to MH 742-105	Install 1,742 ft. of 12" parallel gravity sewer between PS #8 and MH 742-105. Alternatively extend PS #8 force main to MH 742-105 and upgrade pumps to maintain adequate capacity	$ 819,000	2028
C-7	12" - Alderson Road (Parallel)	Install 2,401 ft. of 12" parallel gravity sewer between MH 743-080 to MH 743-042 (or as adjusted to integrate with the findings of Project O-1). May omit 492 ft. along existing steep segments, if diameter restriction is acceptable.	$ 1,128,000	2028

TABLE S.1

10-YEAR CAPITAL IMPROVEMENT PLAN (2019-2028)

ID	Capital Improvement	Project Description	Estimated Project Cost (2019 $)	Recommended Year of Completion
C-9	15" Alderson Rd.	Construct 850 ft. of 15" gravity sewer from MH 743-080 east along Alderson Road to replace the temporary gravity tightline receiving discharge from the Blaine Road PS force main and provide local service.	$ 425,000	2020
C-10	12" Alderson Rd.	Construct 850 ft. of 12" gravity sewer from Project C-9 continuing east along Alderson Road, to replace the temporary gravity tightline receiving discharge from the Blaine Road PS force main and to provide local service.	$ 400,000	2024
		Subtotal for Collection Projects	**$ 6,895,000**	
		IV. OTHER		
O-1	PS #4 Pre-design Update	PS #4 and Alderson Rd. Gravity Sewer Bypass Pre-design Evaluation Update	$ 25,000	2020
O-2	SCADA	SCADA System Upgrades	$ 45,000	2019
O-3	Phones	Phone System Upgrades	$ 15,000	2019
O-4	Vehicles	Replacement Vehicles	$ 298,000	2019, 2022, 2023, 2024, 2027, 2028
O-5	CSP	Comprehensive Sewer Plan Update	$ 150,000	2027
O-6	Reclaim Water ER	Water Reclamation Engineering Report	$ 60,000	TBD, before 2029
O-7	Record	Digital Records Project	$ 9,000	2019
O-8	Facility	Facility Upgrade/Building Upgrades	$ 29,000	2019
O-9	Financial Management Policy	Update Financial Management Policy	$ 7,500	2019
		Subtotal for Other Projects	**$ 638,500**	
		Grand Total	**$ 18,034,500**	

1 Estimated project cost is a preliminary estimate for budget purposes. Project cost is subject to refinement of project objectives, scope and more detailed cost estimate.

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CHAPTER 1 – INTRODUCTION

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CHAPTER 1

INTRODUCTION

This document sets forth the *Comprehensive Sewer System Plan* for Birch Bay Water and Sewer District (BBWSD), Whatcom County, Washington. The Plan has been prepared in conformance with the requirements of the Washington Administrative Code (WAC) 173-240-050. The Plan was approved by the Board of Commissioners of Birch Bay Water and Sewer District and then submitted to the Department of Ecology (DOE) and Whatcom County for review and approval. The District, as lead agency, reviewed a SEPA Checklist for the adoption of this plan and issued a Declaration of Non-Significance (see Appendix C). The plan was prepared to continue the District's compliance with the adopted water quality management plan (i.e., the District's wastewater treatment plant NPDES permit) as discussed in Chapter 6, and Appendix D.

The subject of this report is the municipal wastewater treatment plant (WWTP) and wastewater collection and conveyance system owned and operated by Birch Bay Water and Sewer District. The District is a municipal corporation in the form of a special purpose district, subject to RCW 57. The District provides public water and sewer service to an unincorporated portion of northwestern Whatcom County. This area has been designated as the Birch Bay Urban Growth Area (UGA) in the Whatcom County Comprehensive Plan. The District office and WWTP are located at 7096 Point Whitehorn Road, Birch Bay, Washington, 98320-9675, telephone 360-371-7100, facisimile 360-371-2806.

1.1 AUTHORIZATION

Recognizing the need for the continuing development of the District's sewer facilities and recent evaluation of the wastewater treatment plant and portions of the collection system, the District's Commissioners authorized CHS Engineers, LLC to proceed with the studies required to prepare an update of the 2009 *Comprehensive Sewer System Plan*.

1.2 PURPOSE AND SCOPE

The purpose of this report is to establish an updated *Comprehensive Sewer System Plan* that presents an orderly development of sewer facilities within Birch Bay Water and Sewer District. The studies leading to the preparation of this report included:

A. A review of existing planning data, recent technical studies and material pertaining to the study area.

B. Using Whatcom County planning data, a projection of anticipated population in the District service area was made to forecast wastewater generation through year 2038.
C. An evaluation of existing sewer facilities to determine their current and future adequacy, and a review of existing design criteria.

Significant changes in the system and service area since completion of the 2009 Plan include:

- reduction in size of Birch Bay and Blaine UGAs, within previously designated sewer service area
- relocation of Pump Station #1
- replacement of Pump Station #3 14" force main with 14", 16" and 18" force mains
- replacement of WWTP Headworks facility
- completion of WWTP aeration upgrades

Using the information obtained from these preliminary steps, a general plan and map showing future facilities for the sewer system was developed. These facilities were analyzed to insure appropriate sizing and preliminary cost estimates were made for the proposed facilities.

1.3 BOUNDARY AND SERVICE AREA

Birch Bay Water and Sewer District is located immediately south of the City of Blaine in the extreme northwestern corner of Whatcom County (see Figure 1.1). The District is bordered on the west by Birch Bay and Semiahmoo Bay and on the north by the Blaine city limits and Drayton Harbor. The eastern boundary line is irregular ranging from a minimum of 1,000 feet to over one and one half miles east of Birch Bay. The southern boundary is also irregular from Alderson Road south to Grandview Road (see Figure 1.2). The District presently includes approximately 6,670 acres.

The revised future service area was outlined based upon consideration of the following elements (see Figure 1.2). Refer to Section 3.4 for further discussion of the delineation of the sewer service area.
A. Existing District boundaries and extent of existing sewer system.
B. Parcels served by existing sewer system.
C. Birch Bay UGA
The area included in the Blaine and Birch Bay UGAs has been reduced significantly since the 2009 Plan was completed. In particular, most of the area between Lincoln Road and the Blaine city limits has been removed from either UGA.

Property owners with property outside of the District boundary may petition the District to annex additional land to the District boundary. The following is a list of criteria which the District Commissioners use to review requests for annexation to the District.

1. Is area contiguous with present District boundary?
2. Is the location within the District's future sewer service area?
3. Is the area of reasonable size and shape?
4. Does the District have sufficient treatment plant capacity to serve the area?
5. Does the District have the ability to service the area with present interceptor lines and pump stations (i.e., location relative to urban growth area)?
6. Is the annexation request consistent with the comprehensive land use plan or any other applicable supporting document or policy of neighboring jurisdictions?
7. Consideration is given to the District's financial position.
8. Public health and environment impacts

If the Commissioners determine that the above criteria have been substantially met, or that problems associated with the proposed annexation can be resolved, the Board will normally authorize preparation of an annexation petition for circulation, signature and eventual submittal to the Board. Annexations are subject to review by the Whatcom County Boundary Review Board. The complete process is addressed in District Code Section 1.12.

The District provides sewer service, by agreement, to an area outside the sewer service area described in Section 3.4. The subject area is the BP Cherry Point Refinery, southeast of the District's wastewater treatment plant (see Figure 1.2).

As of the end of December 2018, the District provided direct sanitary sewer service to 7,061 equivalent living units (ELUs) through 4,727 connections.[1] Most ELUs are residential and the balance are multiple residential units or other uses represented as ELUs. ELUs are defined at time of connection based on District Code. Various methods of equivalency determination have been used in the past. Presently the Code requires evaluation of plumbing fixture units to determine ELUs for non-residential uses.

1.4 HISTORY

In 1960, Birch Bay residents began to consider the possibility of creating a public utility to provide service for anticipated community growth. The majority of the Birch Bay area was being served by two private water companies: Birch Bay

[1] This count includes 155 non-residential ELUs in the direct sewer service area but does not include an additional 394 ELUs served by agreement for the BP Cherry Point Refinery.

Water Company and Birch Point Water Association. The residents of Birch Bay were concerned about the ability of these water companies to meet future water needs.

The registered voters in the Birch Bay area petitioned Whatcom County to place a proposition to form a water district on the ballot and in September of 1960, Whatcom County Water District No. 5 was voted into existence. With the creation of this District, area residents had the vehicle through which the necessary financing could be obtained to meet the demands for future water supply and transmission for the developing community.

Challenges were encountered almost immediately after the formation of the district, when it tried to acquire the larger of the two existing private water systems by condemnation. The condemnation proceedings created an undesirable atmosphere that led to the eventual dissolution of Water District No. 5 by the voters in September of 1964. In the years following, various modifications and extensions were made to the private system in an attempt to meet increased demands for water.

The increase in residents and resort activity continued in the Birch Bay area and, in 1966, a problem of even greater intensity than the water shortage became apparent. Birch Bay was becoming polluted by malfunctioning and overloaded septic systems. A sewerage system was needed to abate the pollution of the shallow bay on which the economic livelihood of the community depended. The urgent nature of the pollution problem was impressed upon the residents by the Washington State Health Department, which precipitated efforts to correct the worsening condition through formation of a public utility.

In 1967, the State Legislature enacted legislation authorizing water districts to perform the dual function of operating and maintaining both a water system and a sewerage system, thereby dispensing with the necessity of forming two utility districts. Whatcom County Water District No. 8 was subsequently formed by a majority vote of the people on February 6, 1968, to address the community's water and sewage problem.

The Water District Commissioners, with strong community backing, proceeded promptly with the steps to achieve its primary goal of providing a sewage collection and treatment system for the Birch Bay area.

Federal funds for a preliminary engineering study for water and sewer systems were acquired and negotiations for acquisition were begun with the owner of the Birch Bay Water Company. On September 16, 1969, a minimum scope comprehensive plan and its attendant proposed bond authorization was put before the people and passed by a substantial margin. On January 1 of the following year, the Birch Bay Water Company was sold to Water District No. 8, at which time the water district commenced operation of the water system.

Water District No. 8 applied for and received a non-interest bearing loan from the Housing and Urban Development division of the federal government in the early part of 1969. The loan was for the financing of an engineering report on a comprehensive water and sewerage plan. Hill, Ingman, Chase & Co. was directed to proceed with the report on May 26, 1969. The report was completed and accepted by the Water District in May of 1970.

Construction of the sewage collection system and treatment plant began in April, 1975. Major portions of the collection system were completed by autumn of 1976 and on December 7, 1976, the wastewater treatment plant went into operation.

Twenty-two annexations have occurred since the formation of the District, with the first in 1971 and the most recent in September 2011. Water District No. 6 merged with Birch Bay Water District No. 8 on December 15, 1987.

Whatcom County Water District No. 8 changed its name to Birch Bay Water District No. 8 on October 20, 1983. The name was changed again on January 1, 1988 to Birch Bay Water and Sewer District.

The District entered into a sewer service agreement with ARCO (now BP) on January 25, 2001, to provide domestic wastewater service for the Cherry Point Refinery. The service agreement was amended in November 2008 to expand the area to be served by the agreement.

The District has formed seven sewer utility local improvement districts (ULIDs) to finance and construct portions of the existing sewer system. The location of the ULIDs are shown on Figure 1.3. The District also completed a project to provide sewer service to the Birch Bay View development on Selder Rd. The project was funded by the District with cost recovery through collection of local facilities charges.

1.5 RELATED AGENCIES

Several organizations, agencies and governmental bodies are involved with the aspects of planning, financing, regulating and operating waste treatment works and collection systems for the planning area. Various rules, procedures and requirements are applicable to the process of providing sewage service and all must be considered. Presented below is a list and brief description of the entities associated with providing wastewater services for the planning area; the list is not intended to be all-inclusive.

- Environmental Protection Agency (EPA) - the lead federal agency responsible for financing the planning and construction of waste treatment systems; reviews plans and evaluates environmental impacts of projects with federal funding.

- U.S. Army Corps of Engineers - responsible for navigable waters; issues permits for construction in tidelands and wetlands.
- U.S. Department of Housing and Urban Development (HUD) - responsible for funding community development projects in special need areas; administers the National Flood Insurance Program and delineates flood hazard zones for insurance purposes
- Washington State Department of Ecology (DOE) - the lead State agency responsible for environmental matters; determines water quality criteria and effluent limitations; administers the National Pollutant Discharge Elimination System (NPDES); oversees some programs for funding of publicly owned waste treatment systems; reviews and regulates engineering designs; reports and plans for construction of new waste treatment plants or expansions of existing plants; reviews plans for all projects.
- Washington State Department of Fish and Wildlife - responsible for wildlife throughout the State, including marine and freshwater fisheries.
- Washington State Department of Natural Resources – responsible for aquatic lands including the lease agreement for the District's marine outfall.
- Northwest Air Pollution Control Authority regulates emissions or discharges that impact air quality.
- Whatcom County - the governing agency for the unincorporated area of the District; responsible for planning and zoning; administers permits along shoreline within the authority of the Shorelines Management Program; issues local permits regulating road construction, building, etc. The County reviews and has approval authority for water and sewer special purpose district comprehensive plans per RCW 57.16.010 (7).
- Birch Bay Water and Sewer District (BBWSD) - owns and operates the wastewater collection and trunk system in the District; provides operations and maintenance services to sewer customers in the service area.

1.6 FACILITIES AND SERVICES

Facilities and service available in the Birch Bay area are listed in Table 1.1. The table indicates the appropriate entity providing or administering the service or facility.

TABLE 1.1

FACILITIES AND SERVICES

Facility/Service	Provider
Schools	Ferndale School District 502 and Blaine School District 503
Fire Protection	Whatcom County Fire Protection District No's. 13 and 7
Police Protection	Whatcom County Sheriff's Department, Bellingham, Washington
Water Supply	Birch Bay Water and Sewer District
Public Transportation	Whatcom County and Washington State Department of Transportation
Telephone	Verizon
Sewage Disposal	Birch Bay Water and Sewer District
Solid Waste	Blaine-Bay Refuse Inc.
Recreation	13 City, State or Federal Parks and 3 significant areas of Public Tidelands
Health	Surgical and in-patient services are available at St. Joseph's North Campus and St. Joseph's South Campus hospitals in Bellingham, Washington
Power	Puget Sound Energy
Gas	Cascade Natural Gas
Stormwater Management	Whatcom County Public Works

Existing wastewater treatment facilities within twenty miles of the District's WWTP include:

- Cherry Point Refinery (industrial) – 5.5 miles southeast
- City of Blaine (municipal) – 10 miles north of the entrance to Drayton Harbor
- City of Ferndale (municipal) – 10.5 miles southeast
- City of Lynden (municipal) – 14.7 miles east
- City of Bellingham (municipal) – 17.3 miles southeast
- City of Everson (municipal) – 19.3 miles east
- Lummi Gooseberry Point (municipal) – 6.5 miles southeast
- Lummi Kwina Road (municipal) – 10.2 miles southeast

K:\19-BBWSD\2015\GIS-complan data\CAD\FIG-1-1-VICINITY MAP.dwg 8.5x11 04-06-17

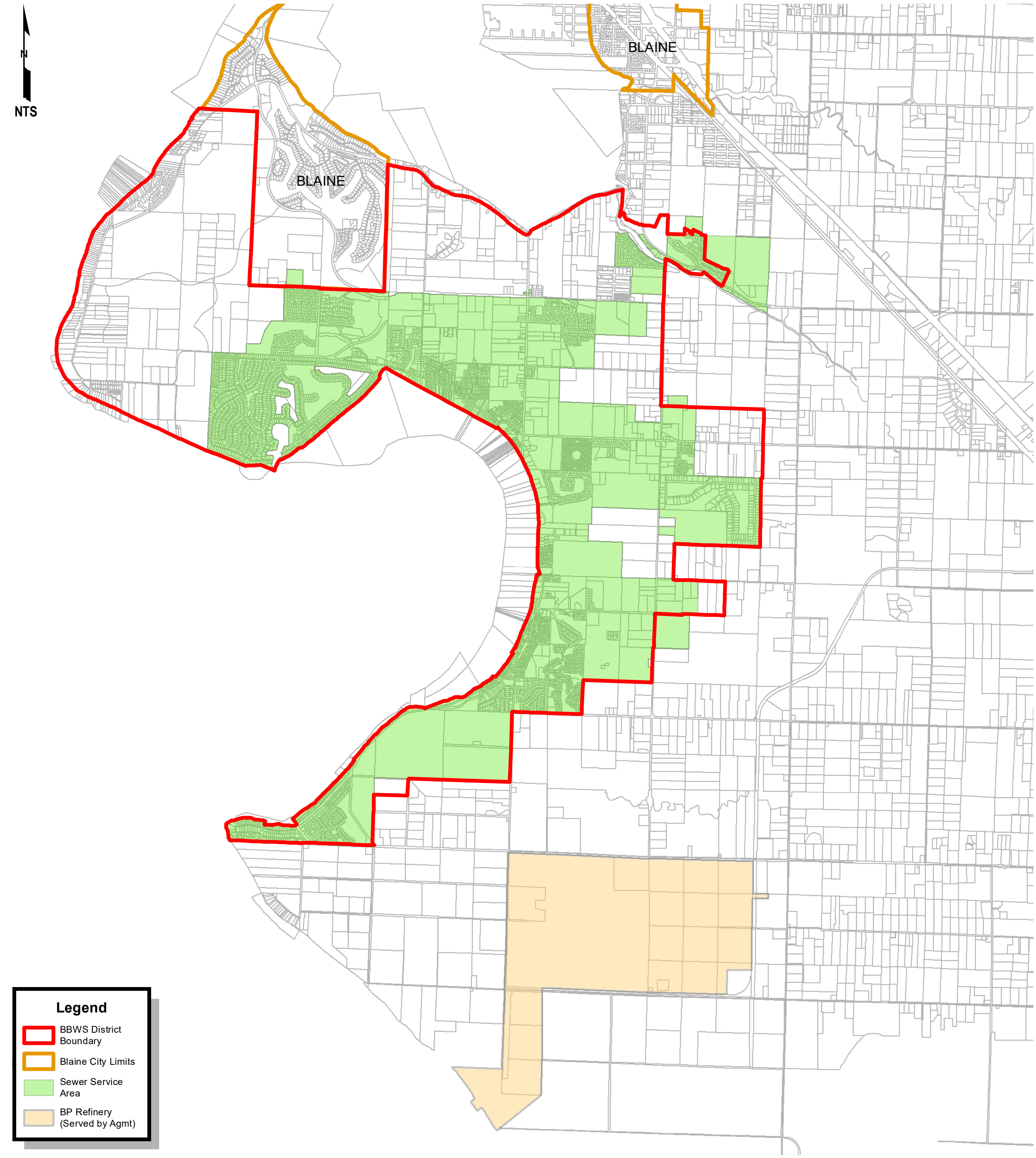
CHS ENGINEERS

Birch Bay Water and Sewer District

2019 Comprehensive Sewer System Plan

FIGURE 1.1

LOCATION MAP

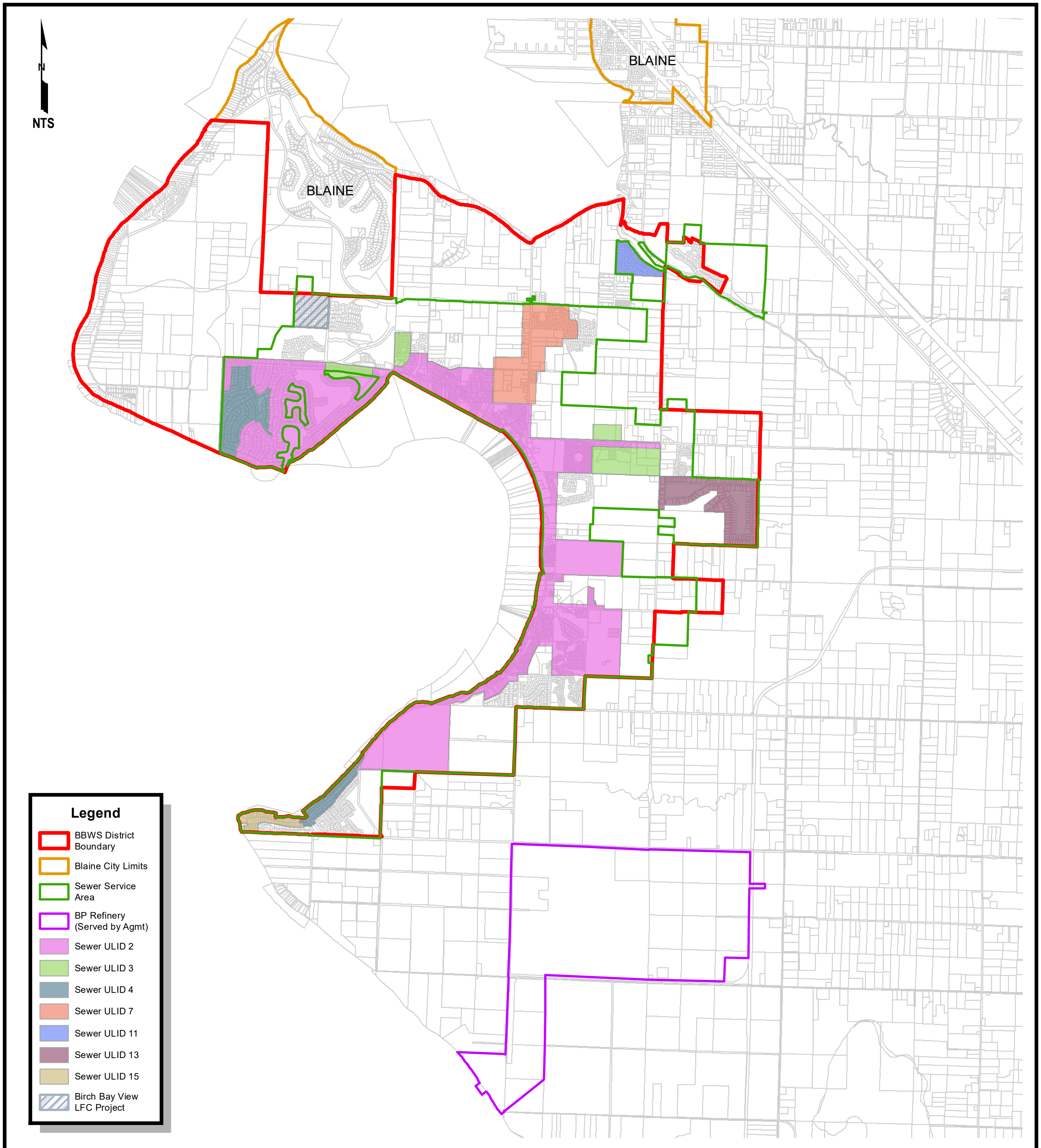


CHS ENGINEERS

Birch Bay Water & Sewer District

2019 Comprehensive Sewer System Plan

FIGURE 1.2
SEWER SERVICE AREA



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Birch Bay Water & Sewer District

2019 Comprehensive Sewer System Plan

FIGURE 1.3
SEWER ULIDs

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CHAPTER 2

PHYSICAL AND ECONOMIC CONSIDERATIONS

Physical features such as topography, location, climate and economic factors play an important role in the planning of community utility systems. Collectively, these factors have a considerable impact on the processes involved in determining the location, size and extent of facilities to be planned and the ability to the community to accept the financial burden of the improvements.

2.1 LOCATION

Birch Bay Water and Sewer District (BBWSD) encompasses the unincorporated resort, recreational and permanent residential community of Birch Bay. It is located in Whatcom County in the northwest portion of Washington State (see Figure 1.1). The Birch Bay area is approximately four miles south of the Canadian Border and immediately south of the City of Blaine. The bay is easily reached from Interstate 5 which passes about four miles to the east. The District boundary is shown on Figure 1.2.

2.2 TOPOGRAPHY AND BODIES OF WATER

The majority of BBWSD is comprised of lowlands that rise 40 to 50 feet above sea level. The highest points in the immediate Birch Bay area are Birch Point and Point Whitehorn. They have relatively steep sided bluffs that define the shoreline of Birch Bay to the north and south, respectively. At these locations, the ground elevation is 150 to 200 feet above sea level (topography is indicated on Figure 5.1).

Birch Bay is a large, circular body of water making a prominent recess in the shoreline of the Strait of Georgia. The bay is characterized by long, flat beaches and extremely shallow water that reaches a maximum depth of approximately 40 feet near its center. This shallow water is quickly warmed by the sun, resulting in an attractive aquatic recreational areas.

Large bodies of water in the vicinity of the District include Birch Bay, Drayton Harbor, the Strait of Georgia, Dakota Creek, California Creek, Terrell Creek and Lake Terrell. Each of these is indicated on Figure 2.1. There are many additional unnamed streams and ponds along or adjacent to Terrell, California and Dakota Creeks, as indicated on Figure 2.1.

2.3 CLIMATE

The Birch Bay area enjoys a mild maritime climate. Occasionally, the area experiences some extremely cold weather for a short period resulting from northeasterly winds blowing off the Canadian Plains.

Climatological data maintained by the District's treatment plant staff is presented in Table 2.1. The monthly temperature ranges from an average low of 43°F to an average high of 60° F.

The monthly precipitation in Birch Bay ranges from an average minimum of 1.15 inches in July to an average maximum of 5.67 inches in November. Three quarters of the 35-inch yearly total precipitation falls as rain from October through April.

TABLE 2.1

CLIMATOLOGICAL DATA

	Mean Temperature		Mean Precipitation
Month	**High (Deg. F)**	**Low (Deg. F)**	**Inches**
January	46	33	4.53
February	50	35	2.99
March	55	38	3.05
April	60	42	2.57
May	66	47	2.31
June	71	51	1.78
July	75	54	1.15
August	75	54	1.38
September	69	49	1.82
October	60	43	3.55
November	50	37	5.67
December	45	33	4.28
Annual Average	60	43	35.09
Climatological data as of September 2008, for the District Wastewater Treatment Plant located at 7096 Point Whitehorn Road, Birch Bay, Washington.			

2.4 INDUSTRY

The natural recreation resources of the Birch Bay area have been attracting people for many years. The construction of sewers in the area has brought about an increase in the recreation industry, which is expected to continue for some time. Years ago, many small resorts were built near the shoreline of Birch Bay. These resorts were mainly comprised of single unit cottages that were usually rented only during the summer months. In recent years, the cottages have been giving way to multi-unit condominiums that have more potential for year-round use.

Trailer parks and recreational facilities attract more people to the Birch Bay area. Birch Bay State Park, located along the southern shore of the Bay, offers camping and trailer facilities along with fishing, swimming, claming and hiking activities. Its presence, draws people to the area who contribute to the economy.

At the present time, most of the recreational activities are between the months of April and September. Assuming a continued stable economy the recreation industry should remain healthy in the Birch Bay area.

The region two miles to the southeast of Birch Bay is designated for industrial development. Two large oil refineries and an aluminum manufacturer are presently located in this region and additional industrial development is expected. The area to the south of Grandview Road and to the east of Koehn Road is zoned industrial. Except as described below,these industrial facilities have separate wastewater treatment and discharge facilities and are located outside the District's sewer service area. There is no industrial development with the District's sewer service area (also see Section 4.6 of this plan).

The BP Cherry Point Refinery has separate domestic and industrial wastewater collection and treatment systems. The effluent from the BP industrial treatment system is discharged through a BP outfall to the Strait of Georgia off Cherry Point. The District receives all domestic wastewater generated at the Refinery, for treatment at its plant on Point Whitehorn Road, and discharge through its outfall off Cherry Point, all under the District NPDES permit.

2.5 TRANSPORTATION

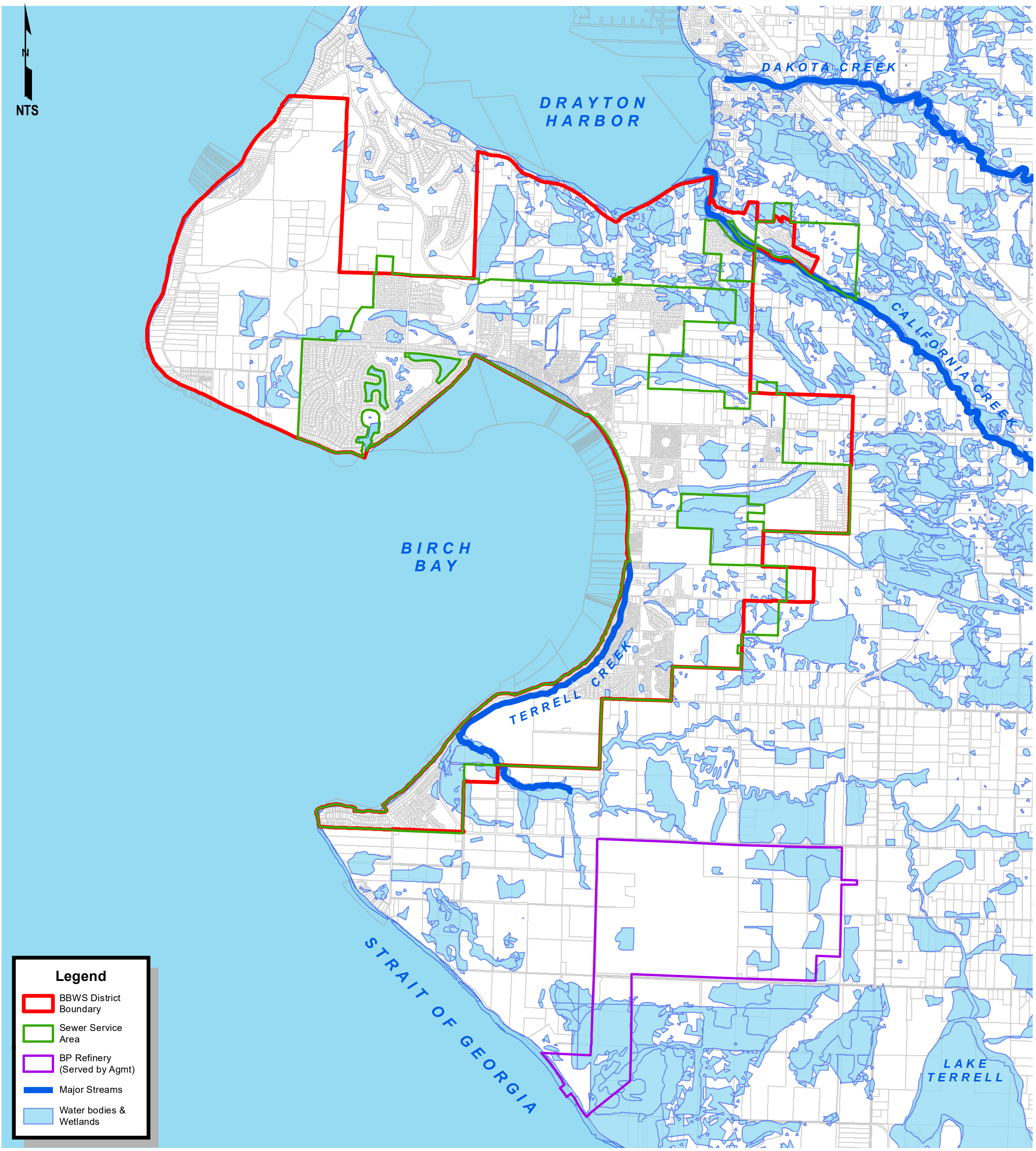
Interstate 5, in Washington State, and Provincial Route 99, in British Columbia, provide a freeway linking the Birch Bay area with population centers north and south, making it easily accessible to many people. Grandview and Blaine Roads were incorporated in the State Highway system, designated SR 548. Locally, the area is served by a network of surfaced county roads.

Rail transportation is available to the east where the Great Northern track parallels Interstate 5. A spur track serves the industrial area near Cherry Point.

2.6 WATER SUPPLY

Birch Bay Water and Sewer District provides water service for the District's water service area. The District purchases all of its water from the City of Blaine. The source of the water is the Blaine well field several miles northeast of the District sewer service area. The District has three storage reservoirs: a 2.5 million gallon reservoir 830 feet south of the intersection of Kickerville and Bay Roads; a 500,000 gallon reservoir one-half mile west of the intersection of Bayvue and Treevue Roads; and a 126,000 gallon reservoir at the north end of Birch Point. Figure 2.2 depicting the District's existing water system.

The Washington Department of Ecology (DOE) online well location database was consulted to identify wells in the sewer service area. Whatcom County has mapped published wellhead protection areas in their GIS data. Wells and wellhead protection areas are depicted on Figure 2.3.



Birch Bay Water & Sewer District

2019 Comprehensive Sewer System Plan

FIGURE 2.1
WATER BODIES

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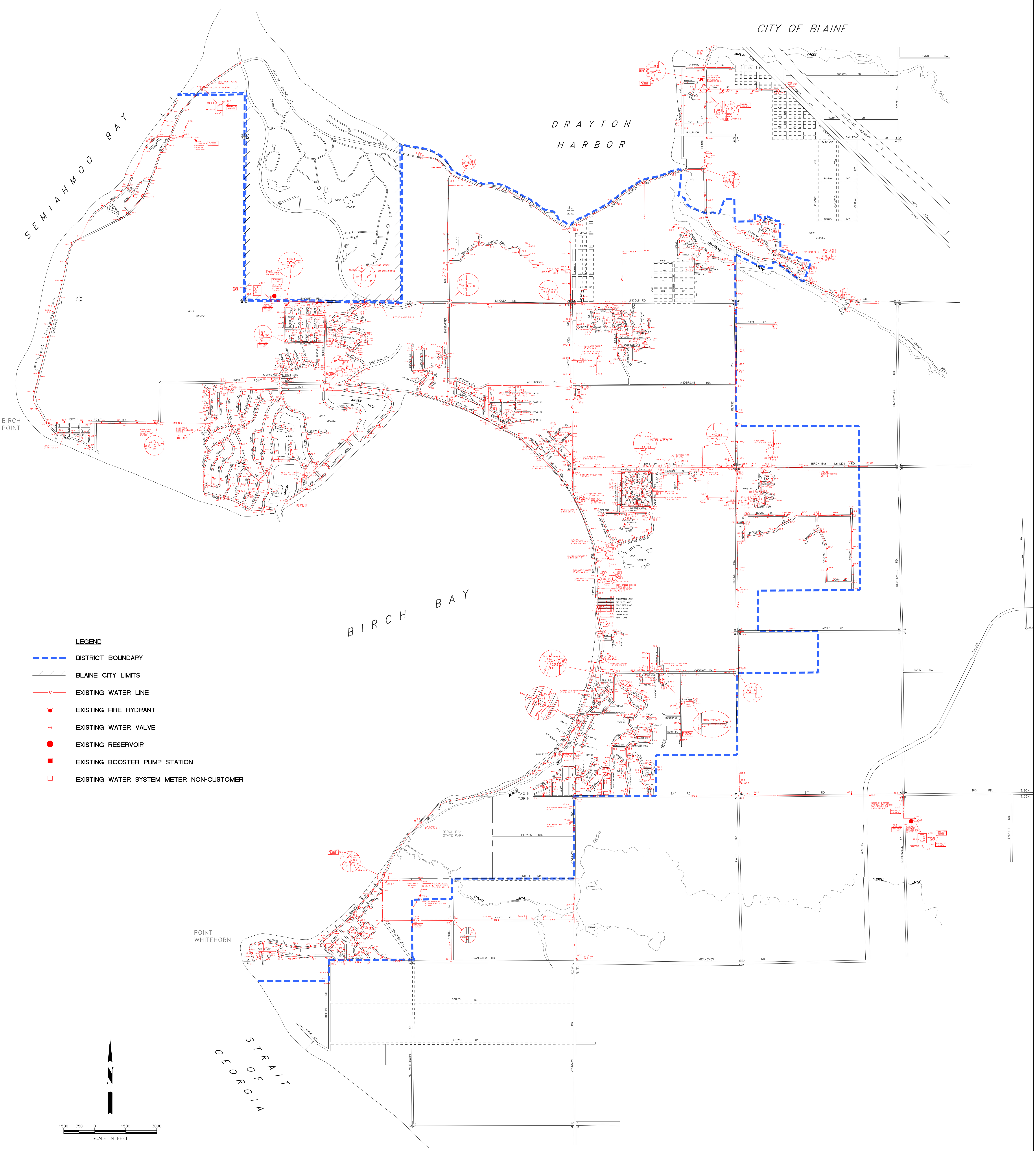

FIGURE 2.2

DISTRICT WATER SYSTEM

Birch Bay Water & Sewer District

2019 Comprehensive Sewer System Plan

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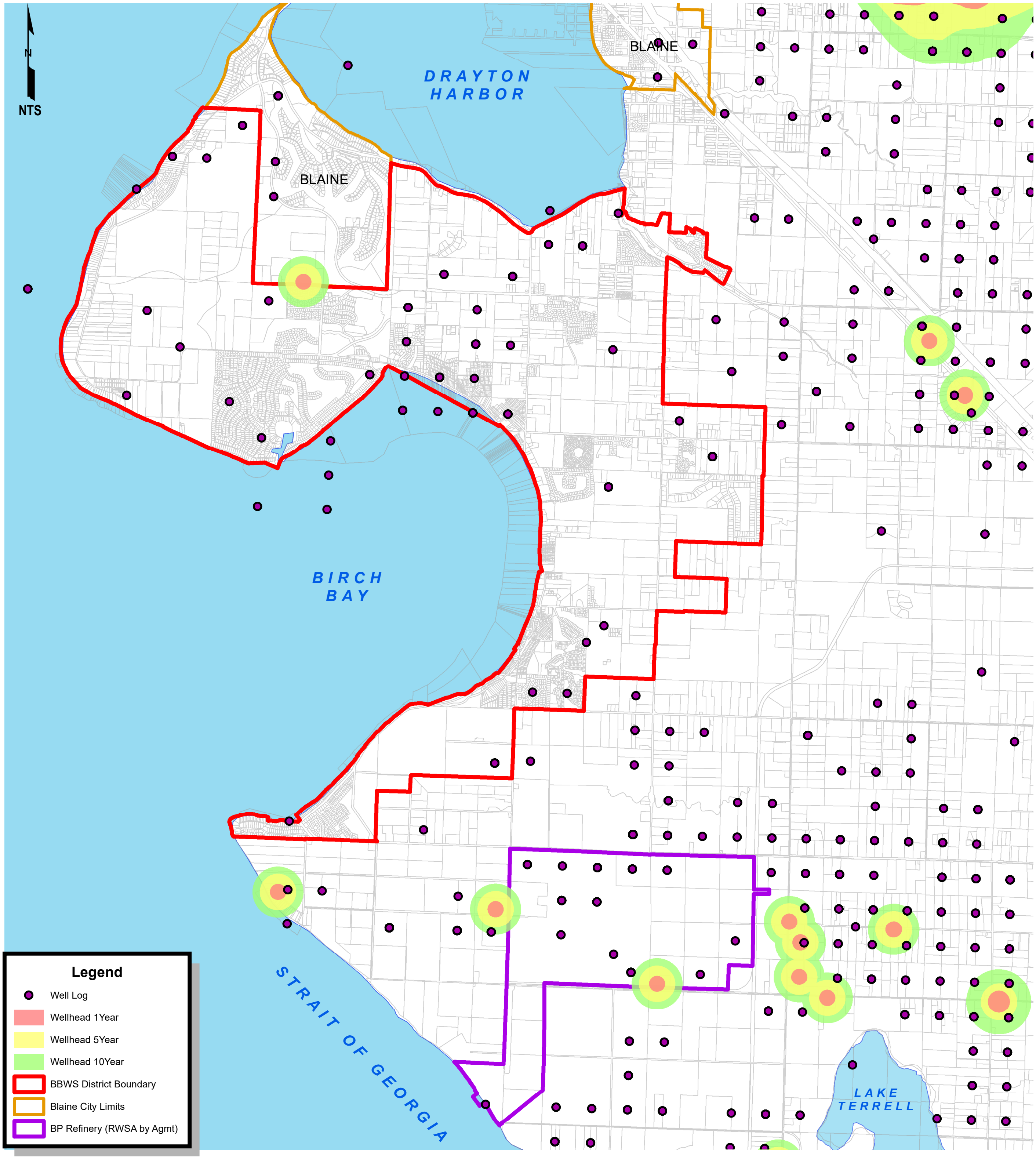

Note:
Well log locations are per DOE database and are defined to the resolition of section quarter quarter. Multiple wells may be associated with each mark.

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Birch Bay Water & Sewer District
2019 Comprehensive Sewer System Plan

FIGURE 2.3
WELLS

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CHAPTER 3

POPULATION AND LAND USE

3.1 INTRODUCTION

In order to project wastewater facility needs over a given period of time, it is necessary to establish a reasonable estimate of the probable demand on these facilities. This can be accomplished in most service areas by a study of the population trends and land use which impact the capacity and placement of sewer system facilities.

This chapter presents historical population data with population forecasts and population estimates based on land use designations. The historical data and forecasts are presented for general information only. The general basis of this *Comprehensive Sewer System Plan* is the provision of sewer service for the potential population at ultimate development of the District service area under the County's zoning designations.

There are several planning documents that impact sewer system planning by the District.

- *Whatcom County Comprehensive Plan*, Whatcom County, updated through May 2016. This study provides planning information for future development in unincorporated Whatcom County.[1]
- *Urban Growth Area Review – Birch Bay UGA Proposal*, Whatcom County. June 24, 2015. This report discusses the growth and development of the Birch Bay UGA as part of the *Whatcom County Comprehensive Plan* update. In the 2016 *Comprehensive Plan* update the County ultimately determined the UGA Reserve Area should remain as is.
- *Whatcom County 2016 Comprehensive Plan and Development Regulations and Urban Growth Areas Review – Final EIS,* Whatcom County, November 2015. This EIS was prepared for the *Comprehensive Plan* update and addresses growth in the County including the Birch Bay Urban Growth Area (UGA).
- *Birch Bay Community Plan (BBCP),* Whatcom County, September 2004 (revised May 2009). This study was prepared by the County and a broad-based community group to more specifically plan for growth and development of the Birch Bay unincorporated UGA.
- *Comprehensive Sewer System Plan*, Birch Bay Water and Sewer District, CHS Engineers, LLC, May 2009. The sewer plan addresses UGAs as

[1] *The 2016 version of the County Comprehensive Plan was current at the time this section of the CSP was prepared and is the basis for the information presented herein. The County Comprehensive Plan was subsequently updated in 2017 and again in 2018.*

delineated at that time and addresses changes in the District since the 2000 sewer plan.

- *Comprehensive Water System Plan,* Birch Bay Water and Sewer District, CHS Engineers, LLC, March 2009, as amended September 2010. This plan addresses water system needs in the District's water service area. This plan is concurrently being updated.

3.2 FACTORS AFFECTING POPULATION GROWTH

Population changes occur naturally or by migration. Natural changes are those resulting from births or deaths; migratory changes are those resulting from the movement of people into or out of an area. The variety of factors which influence such changes are innumerable.

Perhaps the factors which have the most impact on natural population changes are economic conditions, social attitudes and medical technology. With improved economic conditions and medical care, people tend to live longer and, until recently, have had more children. In the past few years, however, with a change in social attitudes, the natural birth rate has been decreasing even though economic conditions generally have been good.

The dominant influences acting to produce migratory change are economic conditions, transportation and climate. The desire for better economic opportunity or a more suitable climate results in moves of relatively long distances, such as from one state to another or from one region to another. The adequacy of transportation facilities is a factor in the rather localized migration of people seeking desirable locations in which to reside.

Migratory change in the Birch Bay area is probably more affected by economic conditions than in most communities. Because the area is a second home or vacation spot for most of the seasonal population, economics play an important role in their ability to come to Birch Bay. In making the forecasts for the area, a continued, stable economic picture for the Pacific Northwest is assumed.

Birch Bay is situated between two large metropolitan areas, Vancouver, B.C., a little over half an hour north, and the Seattle area, about two hours south by automobile. With the continued growth of these two areas, there will be an increasing demand for recreational facilities. Birch Bay, with its unique recreational resources, should continue to serve a good share of this demand.

At this time, Birch Bay's recreational industry is mainly a summer industry. It would seem reasonable to assume that as the area continues to develop, there will be more activities to attract people on a year-round basis. This would establish a stronger economic base and promote new development.

The industrial area southeast of Birch Bay should contribute to a stable year-round economic base for the area. Assuming that a portion of the people involved with the industries establish permanent homes in the area, other businesses such as food stores and miscellaneous shops will also be needed to support these new residents.

Residential development of the Birch Point area is expected to continue over the next twenty years, also increasing the permanent population. In view of the changes taking place in the area, Birch Bay should continue to grow as a recreational and residential area as well as broaden its economic base.

3.3 DISTRICT POPULATION

3.3.1 Current Population

From 2000-2010, Whatcom County's population grew by 20.6% with an annual growth rate of 1.9%. The population of Whatcom County in 2010 was 201,140. Whatcom County significantly updated their Comprehensive Plan in 2016. As part of that process, the County commissioned a population and employment projection: Whatcom County Population and Employment Projection and Urban Growth Area Allocations, Phase I Technical Report (Revised November 1, 2013) by BERK (hereafter referred to as the BERK Report). The BERK Report analyzed the Washington Office of Financial Management (OFM) May 2012 population forecasts, which projected Washington State and Whatcom County population out to 2040. The purpose of the evaluation was to provide information relating to Whatcom County population distribution between UGAs and Rural areas. The BERK Report studied OFM's low, medium and high projections versus the actual growth from 1970 to 2010. Actual average annual growth between 2000 to 2010 was 1.9% for the County. They further projected alternative low (1.1%) and high (1.5%) future growth rates, while holding the medium projection the same as OFM at 1.3% for Whatcom County.

The BERK Report presents the historical average annual growth rate of the Birch Bay UGA as 6.4% for the period 1990-2010. Between 1990-2000 the average annual growth rate was 6.9% and between 2000-2010 the average annual growth rate was 5.9%. These estimates were based on the 2013 UGA boundary, not the previous, larger UGA boundary. From 1990-2010, the Birch Bay UGA had the second highest share of growth (7.2%) in the city and unincorporated UGAs in the County, outside of Bellingham. From 2000-2010, the Birch Bay UGA had the highest share (9.4%) of growth (not including Bellingham).

The District measures its growth in terms of water and sewer system connections and equivalent living unit (ELU) totals for each system. The average annual increase in sewer ELUs from the end of 2004 through 2017 was 1.29%. The

District experienced significant development activity in the early part of this century, but that rate of new lot preparation has dropped sharply.

Because Birch Bay is both a permanent residential and resort community, estimating and forecasting population is difficult and subject to significant fluctuations depending, to a large degree, upon economic and climatic conditions.

The geographic area for the population figures presented in this *Comprehensive Sewer System Plan* is the District's sewer service area as indicated on Figure 1.2. The estimated 2013 population and housing units for the service area is presented in Table 3.1. The population for the Birch Bay UGA is from the BERK Report as adjusted and reported in the March 2015 Draft EIS and as adopted in the Whatcom County Comprehensive Plan. The 2013 population and housing units for the subareas was determined from GIS data from Whatcom County. The 2013 average household population density in the Birch Bay UGA was approximately 1.43 and the average for the sewer service area was 1.47. The District provided direct sewer service to approximately 6,625 residential ELUs as of the end of December 2013. Assuming all parcels in the sewer service area were served by public sewer at that time, the population per residential ELU was approximately 1.21.

TABLE 3.1

POPULATION ESTIMATE

Area	**1990**	**2000**	**2010**	**2013**	**2013 Housing Units**
Birch Bay UGA[1]	2,141	4,163	7,391	7,540	5,257
UGA Reserve Area	(2)	(2)	(2)	6	2
Loomis Trail	(2)	(2)	(2)	175	71
Sunday Harbor/ Maple Leaf	(2)	(2)	(2)	258	118
Plaza Park	(2)	(2)	(2)	60	30
Misc Served Parcels	(2)	(2)	(2)	10	5
Total - Sewer Service Area	**2,141**	**4,163**	**7,391**	**8,049**	**5,483**

1 - Birch Bay UGA as defined as of 2013. Population figure for Birch Bay UGA per the BERK Report, 11/13, as adjusted by County Draft EIS, 3/15, and adopted in the Whatcom County Comprehensive Plan.

2 - For 1990, 2000 and 2010, population in the remaining subareas was not determined.

3.3.2 Projected Population

The BERK Report presents the population allocation by growth area for the low, medium and high projections for the period from 2013 to 2036. The county-wide low projection of 1.1% average annual growth results in an average annual growth rate of 2.3% for the Birch Bay UGA. Outside of the unincorporated UGAs, average annual growth is projected as 0.8%. For the county-wide medium projection of 1.3% average annual growth, the report suggests a 2.7% average annual growth for the Birch Bay UGA and 1.0% for the areas outside the UGAs. For the county-wide high projection of 1.5% average annual growth, the report suggests Birch Bay UGA with an average annual growth rate of 3.2% and 1.2% for areas outside the UGAs.

Population projections for the District presented in Table 3.2 are based on the County Draft EIS (March 2015) and the Whatcom County Comprehensive Plan 2013 population in the Birch Bay UGA of 7,540. Whatcom County previously adopted Resolution 2014-013 with a population increase for the Birch Bay UGA of 5,500 for the period 2013 to 2036. However, the County Council subsequently reduced the Birch Bay UGA growth population allocation from 5,500 to 5,282 (by motion, May 10, 2016 meeting). Using these numbers, an average annual growth of 2.3353% was applied over the period for the Birch Bay UGA to achieve the County UGA target population.

As shown on Figure 3.1, there are three areas outside of the UGA being served by the District: Loomis Trail, Sunday Harbor/Maple Leaf and Plaza Park. The population growth in these areas was determined by looking at full buildout at the end of the planning period and by applying the same population per household unit as determined from the 2013 GIS data from Whatcom County. This indicates Loomis Trail with a growth rate of 0.84%, due to a limited number of available building lots. Sunday Harbor/Maple Leaf is projected at 0.075% growth, again because of limited available building lots. Plaza Park has a projected growth rate of 0.56% because of a limited number of available lots. In addition there are a few miscellaneous served parcels with no growth anticipated.

The population and wastewater flow forecasts extend through year 2038. An urban area growth rate of 1% has been applied for the Birch Bay UGA for years 2037 and 2038. This factor presumes some potential infill or rezoning in the present UGA. The 2013-2036 rural-area growth rates are extended through year 2038.

The population projections in Table 3.2 have been provided to show anticipated population growth within the Birch Bay sewer service area thru the year 2038. It is anticipated that most of the growth in sewer demand will occur within the Birch Bay UGA (see Figure 3.1).

TABLE 3.2

POPULATION PROJECTION

Area	2018	2019	2020	2021	2022	2023	2024
Birch Bay UGA[1]	8,462	8,660	8,862	9,069	9,281	9,498	9,720
UGA Reserve Area[2]	6	6	6	6	6	6	6
Loomis Trail	182	184	186	187	189	190	192
Sunday Harbor/ Maple Leaf	259	259	259	260	260	260	260
Plaza Park	62	62	62	63	63	63	64
Misc. Served Parcels	10	10	10	10	10	10	10
Total - Sewer Service Area	**8,982**	**9,181**	**9,386**	**9,595**	**9,809**	**10,028**	**10,251**
Area	**2025**	**2026**	**2027**	**2028**	**2033**	***2036***	**2038**
Birch Bay UGA[1]	9,947	10,179	10,417	10,660	11,964	*12,822*	13,080
UGA Reserve Area[2]	6	6	6	6	6	*6*	6
Loomis Trail	193	195	197	198	207	*212*	216
Sunday Harbor/ Maple Leaf	260	261	261	261	262	*262*	263
Plaza Park	64	65	65	65	67	*68*	69
Misc. Served Parcels	10	10	10	10	10	*10*	10
Total - Sewer Service Area	**10,481**	**10,715**	**10,955**	**11,200**	**12,516**	***13,381***	**13,643**

1 – Birch Bay UGA as defined as of 2016 and unchanged as of 2019. Population figure for Birch Bay UGA in 2013 is per the County Draft EIS, 3/15 as adopted in the County Comprehensive Plan. The County growth allocation for Birch Bay UGA is 5,282 (projected growth from 2013-2036 per County revision in May, 2016. The revised growth allocation was confirmed by the County in its 2016 Comprehensive Plan. The 2016 County Comprehensive Plan was updated in 2017 and again in 2018.

2 - UGA Reserve Area to remain outside UGA and is considered Rural land for this forecast, except for one parcel with existing sewer service.

The total estimated population in the sewer service area was approximately 8,049 persons as of 2013. The District added approximately 252 residential sewer ELUs for the period from 2013-2018. At the average residential density of 1.21 persons per ELU, the estimated population at the end of 2018 is about 8,350. The forecast population for the end of 2018 is 8,982. Based on actual growth in sewer connections for 2013-2018, the estimated sewer service area population is 630 persons or about 7.0% below the forecast. This can be interpreted that growth is lagging behind the forecast by about three and one-third years.

3.4 LAND USE, SERVICE AREA AND ZONING

Land Use

Whatcom County has jurisdiction over land use and zoning in unincorporated areas such as Birch Bay. The 1990 Washington State Growth Management Act (GMA) and subsequent revisions thereto have changed the way land use planning is completed in the counties with higher population in the State. The GMA requires Whatcom County to prepare and adopt a comprehensive plan that provides for and manages the growth projected for the next twenty years in a manner that is consistent with the goals of the Act.

Under the GMA, the County Comprehensive Plan must contain UGA designations, within which urban growth shall be encouraged and outside of which growth can occur only if it is not urban in nature. These areas include existing incorporated areas such as Blaine along with their growth areas. A UGA may also be designated in an unincorporated area not contiguous to an existing city if the area is already characterized by urban growth. The Birch Bay area fits this latter description and, under the *Whatcom County Comprehensive Plan*, a large portion of the District is designated as a UGA (see Figure 3.1).

Whatcom County adopted the *Birch Bay Community Plan* (BBCP) as an amendment to the County *Comprehensive Plan* on September 28, 2004. The BBCP evaluates the present conditions, projects future growth and establishes planning criteria for the Birch Bay area, including evaluation of land use, utilities, housing, transportation, public education, governance, etc. The BBCP is still in effect, however, the population forecasts and UGA boundaries have been superseded by the *Whatcom County Comprehensive Plan.*

Sewer Service Area

The explicit responsibility and authority under RCW 57.16.010 for the District to develop this Plan is to investigate and plan "..for the prevention, control, and reduction of water pollution and for the treatment and disposal of sewage and industrial and other liquid wastes now produced or which may reasonably be expected to be produced within the district…" The District also provides water service within and beyond its boundary and this service is the source for all or nearly all the liquid waste produced in the District. Therefore it is the District's purpose and intention to provide and require public sewer service to development within its boundary. District Code 8.04.080 requires any person planning to build a structure for human occupation within the District to secure from the District a letter of sewer availability, or sewer non-availability before applying for a building permit or onsite sewage disposal system permit. Thus the District requires connection to sewer where water service is provided, to the extent allowed by the GMA. The GMA and designation of UGAs impacts where the District can provide sewer service, as such is defined by the State as an "urban service". Therefore, a sewer service area is defined as discussed below, to establish a specific sewer service area, within and beyond the District boundary.

Provision of service is accomplished by connection to the existing system, District-extension of the system through the ULID process (see Chapter 7) or the developer extension process (see Chapters 7 and 8). At this stage in the District's life, the primary means of providing service is through the developer extension process.

The sewer service area for this Plan (see Figure 1.2) includes all the area within the Birch Bay UGA, plus the following areas:

- Plat of Sunday Harbor, Loomis Trail Golf Course and adjacent plats along California Creek, and presently served adjacent parcels. Parcel with golf course extends out of the sewer service area; however, because it is primarily golf course, no sewer service is anticipated. This area is served by an existing sewer collection system and portions are within the District boundary.
- Plaza Park Mobile Home Park (on Birch Bay – Lynden Road, east of Blaine Road). This area is served by an existing sewer collection system and is within the District boundary.
- Specific parcels presently served by the sewer system by connection to a nearby sewer collection system:
    - two parcels north of the intersection of Lincoln Road and Harborview Road
    - two parcels along Blaine Road west of intersection with Loomis Trail Road
    - one parcel with one residential structure immediately south of Maple Leaf Village
    - one parcel northwest of intersection of Birch Bay – Lynden Road and Blaine Road

- one parcel north of the Plat of Birch Bay View (interim service agreement – parcel is in Blaine city limits)
- one parcel in the UGA Reserve Area.

The service area extends outside the District boundary as follows:

- at the southeast corner of the intersection of Blaine Road and Alderson Road
- a parcel northeast of the intersection of Birch Bay-Lynden Road and Blaine Road
- a portion of the Loomis Trail development and immediately adjacent areas as described above
- a parcel in the City of Blaine limits.

The sewer service area includes approximately 4,110 acres.

The Washington Administrative Code (WAC) 365-196-425 addresses rural areas of a county comprehensive plan and which government services are consistent with designation as rural areas. Subsection 4(b) states that

> “Rural services do not include stormwater or sanitary sewer. Urban governmental services that pass through rural areas when connecting urban areas do not constitute an extension of urban services into a rural area provided those public services are not provided in the rural area. Sanitary sewer service may be provided only if it:
>
> > (i) is necessary to protect basic public health and safety and the environment;
> > (ii) is financially supportable at rural densities; and
> > (iii) does not permit urban development.”

Whatcom County recently amended County Code Section 20.82.030 to include language that is consistent with the WAC recognition that sewer lines may pass through areas not included in a UGA, provided service is not provided to properties in those rural areas. Whatcom County Code 20.82.030 also prohibits extension of sewer lines outside of UGAs to provide service, unless such extensions (new sewer lines with an inside diameter of six inches or more or length of 150’ or more) are shown to be necessary to protect basic public health and safety and the environment, and when such services are financially supportable at rural densities and do not permit urban development.

DOE, at WAC 173-240-020(13), defines a sewer line extension as follows:

> "Sewer line extension" means any pipe added or connected to an existing sewerage system, together with any pump stations: Provided, That the term does not include gravity side sewers that connect individual building

or dwelling units to the sewer system when these side sewers are less than one hundred fifty feet in length and not over six inches in diameter.

There are two areas for which sewer line extension through a rural area is anticipated, due to the location of existing sewer facilities, topography and the Birch Bay UGA. Existing sewer facilities are present in the Plat of Sunday Harbor and immediately south, outside the UGA. Land in the UGA immediately south of the extension of Lincoln Road, west of Blaine Road, is in the California Creek drainage basin. Sewer service for this portion of the UGA will require a gravity sewer line extension from the existing system south through the rural area to the UGA. The Birch Bay UGA Reserve area includes an existing gravity sewer line draining to a sewer pump station. These facilities are located in a regional low area due to the nature of gravity sewer systems. Land in the UGA south of Arnie Road requires a gravity sewer line extension from the existing pump station in the UGA Reserve area.

The District will not extend sewer lines to properties in rural designated areas, except under conditions as allowed by the GMA, the WAC as referenced above, and Whatcom County Code 20.82.030. Such service, if allowed as noted immediately above, may be to single parcels outside the presently defined sewer service area.

Significant areas within the District boundary are not included in the sewer service area (see Figure 3.1).

Within the sewer service area there are several large areas that are not expected to require significant sewer service:

- Parks – The County Birch Bay Conservancy Area is a 60 acre undeveloped park situated between the District WWTP and Point Whitehorn Road. Development in this area is not anticipated. The County Bay Horizon Park is located at the site of the former Blaine Air Force Station. The site includes several existing buildings with sewer service but development of the remaining open space is not anticipated. The 70-acre County Sunset Farm Equestrian Center is located on west side of Blaine Road south of Birch Bay – Lynden Road. The County Halverson Park is situated south of Anderson Road, east of Cedar Avenue. It is an undeveloped park. Birch Bay State Park is located at the south end of Birch Bay. Facilities in the park are served by a State-owned collection system and pump station. The station discharges to the District collection system upstream of Pump Station No. 1.
- Golf courses (Birch Bay Village, Sea Links and Loomis Trail) - Service to golf course areas is not considered in the study area and calculations as they will contribute no significant wastewater flows or flow due to inflow and infiltration.

- A 78-acre parcel south of Helweg Road and west of Jackson Road is owned by BP and is subject to a conservation easement. The easement prohibits development on the property.

Zoning

Figure 3.2 indicates the present zoning around Birch Bay. Table 3.3 indicates the zoning classifications, units per acre and equivalent population per acre. Population and housing unit counts from census data and Whatcom County studies discussed above were reviewed to estimate the population per housing unit ratio. Data and planning criteria for other developed areas around Puget Sound were also reviewed. The criteria of 1.32 persons per unit for single-family developments was selected for use in this Plan (see further discussion in Chapter 5 (Section 5.5). This criteria was used to estimate the equivalent population per acre for urban, residential land use. This criterion is based on the estimated future population and housing unit forecast for the service area.

The maximum allowable densities determined by the current zoning designation of each area are used for forecasting sewer flows. The zoning designations are defined by Whatcom County. The significant changes from existing zoning within the sewer service area are the following:

- Land in the vicinity of Sunday Harbor and Loomis Trail developments has been changed from UR4 to R10A.
- A couple of served parcels outside the UGA have changed from GC or UR4 to R5A.

Equivalent population for non-residential areas has been estimated in the following manner. The projected ultimate number of employees and corresponding residential equivalent population densities are shown in Table 3.4. The ultimate number of employees is estimated with this formula:

> Ultimate employees per acre = (building factor) x (employees per square foot of building) x (occupancy rate) x (43,560 square feet/acre).

The “building factor” represents the amount of building floor area over the building lot area and increases significantly for multi-story structures. This factor was estimated for the various commercial and industrial developments in the Birch Bay area. The ultimate number of employees per acre is then multiplied by 20/70 to obtain an equivalent residential population density for use in sewer system analysis and planning. This factor (20/70) is the ratio of commercial to residential per-capita sewage flows. Per-capita flows are discussed in Chapter 4.

TABLE 3.3

ZONING CLASSIFICATIONS

Abbreviation	Description	Units Per Acre	Equivalent Population Per Acre
UR4	Urban Residential	4	7.0
URM6	Urban Residential - Medium	6	10.5
URM24	Urban Residential – Multi-family	24	42.0
RC	Resort Commercial	N/A	5*
NC	Neighborhood Commercial	N/A	5*
GC	General Commercial	N/A	5*

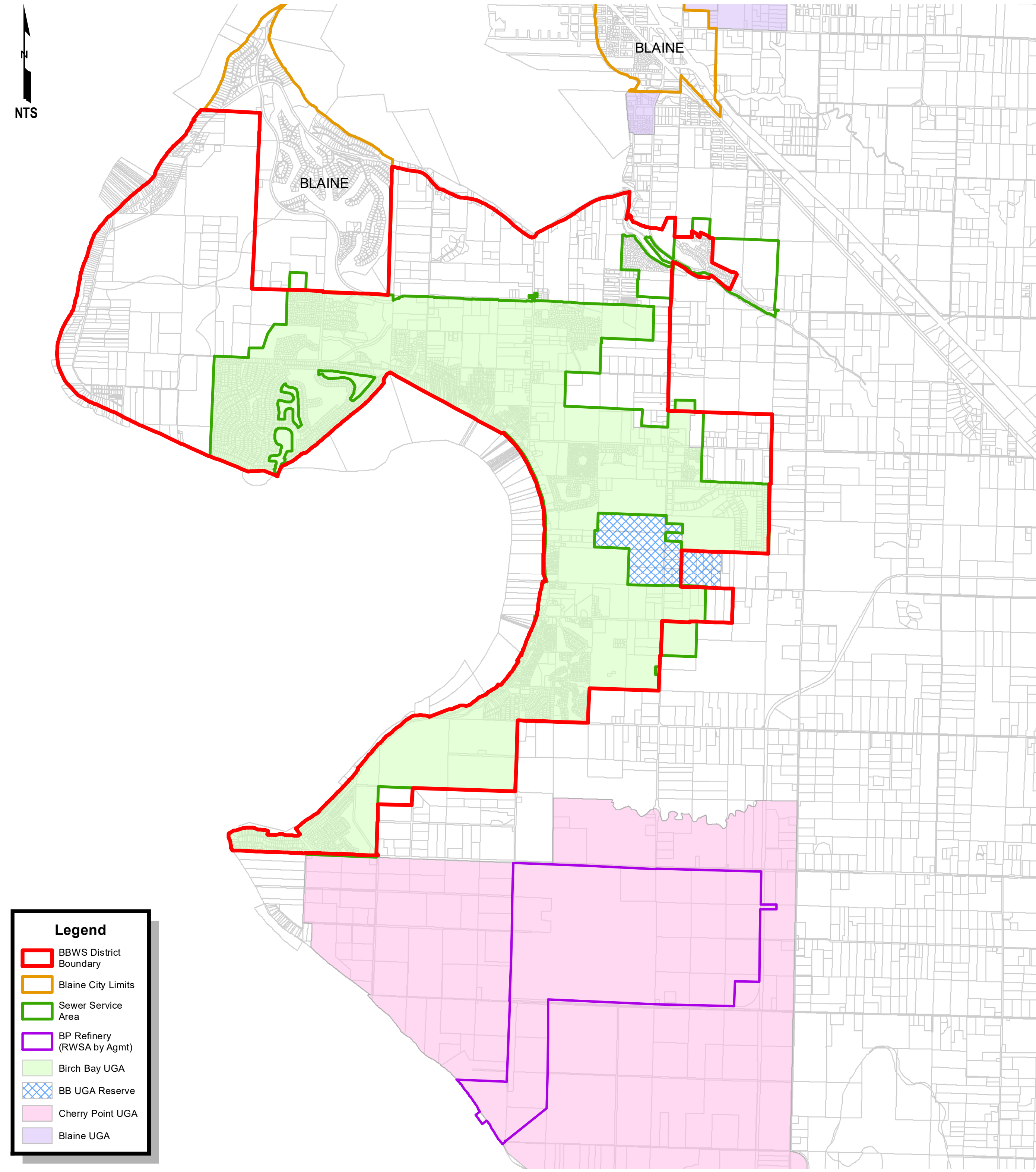
*See Table 3.4 for equivalent population determination.

TABLE 3.4

EQUIVALENT POPULATION DENSITIES

Zone	Building Factor	Employees Per Sq. Ft.	Occupancy Rate	Ultimate Employees Per Acre	Equivalent Population Per Acre
GC, NC, RC	0.8	1/2,000	0.95	16.6	5

The assumptions which support the data in Table 3.4 may not accurately reflect the potential flows from certain developments such as hotels, restaurants, or retail centers with public restrooms. As development and re-development occurs throughout the District, especially along Birch Bay Drive, population and wastewater flows should be carefully monitored. Specific projects within the subject zones should be reviewed with respect to wastewater flows and the most accurate data available should be used for sizing the collection system components necessary to serve the development.



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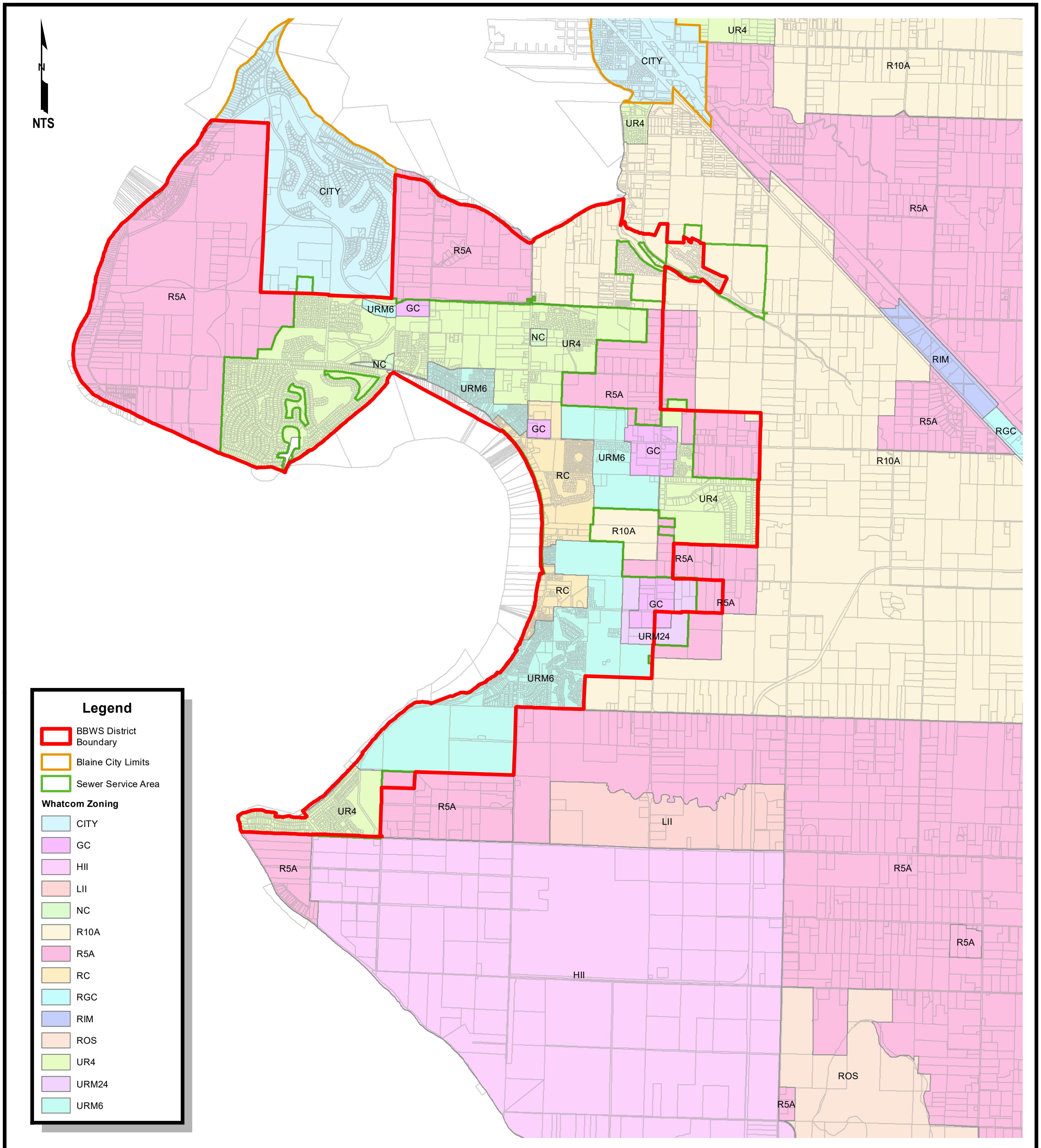
2019 Comprehensive Sewer System Plan

FIGURE 3.1
URBAN GROWTH AREAS



CHS ENGINEERS

Birch Bay Water & Sewer District

2019 Comprehensive Sewer System Plan

FIGURE 3.2
ZONING DISTRICTS

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CHAPTER 4

DESIGN CRITERIA

4.1 INTRODUCTION

Chapters 4, 5, 6 and 7 support and develop a comprehensive plan of improvements necessary to continue to provide adequate sewer service to the residents of Birch Bay and the surrounding service area. In this chapter, design criteria are established to determine the adequacy of the existing system and the requirements of future facilities. The design criteria is based on *Criteria for Sewage Works Design* published by the State of Washington Department of Ecology (DOE), actual usage records and other accepted standards for sewer system design and construction.

4.2 ABBREVIATIONS AND DEFINITIONS

In this section, a number of common technical terms and expressions have been abbreviated. These terms and their abbreviations are presented below.

acre(s)	ac
cubic feet per second	cfs
gallon(s)	gal
gallons per acre per day	gpad
gallons per capita per day	gpcd
gallons per day	gpd
gallons per minute	gpm
million gallons per day	mgd
parts per million	ppm

4.3 REFERENCE DATUM

Since hydraulic capacities of sewerage facilities are based on pipeline slopes, it is important that a common datum be used for design purposes. The planning and construction of existing and future facilities for BBWSD is based on the USC & GS NGVD 1929 - 1947 Adjustment Vertical Datum.

4.4 PERIOD OF DESIGN

In planning sewerage facilities it is necessary to evaluate both present and future service needs, and to design a system compatible with variable demands over a given length of time. This time span is known as the period of design or planning period. A 20-year period will be used in developing a system capable of handling future sewage demands.

Economy in design and construction cost is achieved by the construction of trunk and interceptor sewers with sufficient capacity to meet the present and future (ultimate) capacities of the drainage area. This is especially true in congested areas where duplication and paralleling of sewerage facilities at some future date would be extremely difficult and costly. Pump stations and treatment plants are best suited for staged construction. Basic pumping structures are designed to meet ultimate needs, but use equipment compatible with shorter term demands. Some basic treatment process components follow a similar construction and equipment schedule, but in many cases basic structures are expanded or duplicated as the need arises. The design of pipeline facilities that will not operate at full capacity for many years needs to address the impact of relatively small flows in a high capacity line. For example, gravity sewers must have adequate velocity at initial and long-term flow rates in order to avoid sedimentation within the pipeline. The diameter and length of force main must be carefully chosen to avoid sedimentation and septicity problems

4.5 DESIGN LOADING FOR SEWERAGE FACILITIES

The flow in a sanitary sewer system is composed of commercial and industrial wastes, groundwater infiltration and surface water inflows in addition to sanitary wastes. All portions of the sewer system must be capable of carrying the peak rate and volume from these sources.

Projecting sewer flows requires knowledge or estimates of the following variables: area (acres), development density (e.g. four units per acre), inflow and infiltration rate (I/I, in gallons per acre per day), residential density (persons per household), flow per capita (gallons per person per day) and peaking factor or diurnal wastewater flow pattern.

For the 2007 *Capacity Upgrade Planning Report* (CUPR), the planning criteria used in prior District sewer plans was analyzed to ensure that the assumptions are reasonable based on measured events at Pump Station #3 and the WWTP. Each variable in the calculation of peak hourly flow was reviewed. The only criterion recommended for adjustment was the sewage flow per capita, from 85 to 70 gpcd. This value is reviewed in Chapter 5, as part of WWTP flow data review.

Table 4.1 lists the design criteria for the various components which are included in the flow in sanitary sewers.

The basis for peaking factors historically used by the District as included in Table 4.1 appear to be studies by the Municipality of Metropolitan Seattle (a.k.a. King County Metro), summaries published by the American Society of Civil Engineers

(ASCE) and civil engineering reference books. The factors unique to heavy or light-industrial land use have been discontinued in this Plan because they are not applicable to the current land use in the sewer service area. The peaking factors for existing residential land use have been updated for this Plan, based on the analysis described as follows.

For planning purposes, peaking factors are used for developing forecasts of flows relative to average daily flows. It is generally recognized that peaking factors decrease with increasing average daily flow or contributing population. In a collection system, one would expect a higher peak factor closer to the source (i.e. measured at the outlet of a small sewer basin far from the WWTP) than at the inlet to the WWTP (i.e. all the sources arriving at different times based on time of travel through the collection system).

Several sources were consulted for review and confirmation of peaking factors, and which elements of the wastewater flow stream should be subject to the factors. The references include DOE criteria, an EPA study, a textbook by Metcalf & Eddy (M&E) and a design manual published jointly by the ASCE and Water Pollution Control Federation (collectively, ASCE). Detailed listings are included in the References section of this Plan.

DOE includes a recommended ratio of peak hour flow to average daily flow, as a function of population served. The potential range is from a high of about 4.2 for about 100 persons or less to a minimum of 2.5, corresponding to a population of about 25,000 person, or greater. It is not clear if I/I is included. The equation used by DOE is included in the notes for Table 5.6.

The EPA study summarizes a project that evaluated 39 sewer collection systems to determine the ratio of peak hourly flow to average daily flow, for sanitary flows only, measured at the WWTP. Selected systems were classified by customer type as either predominately residential or industrial/institutional or those systems with multiple pump stations. This study summary did not report how the peak factor varied upstream in the system. The maximum peak factor for residential systems was 1.70 and the average was only 1.23. These figures are well below the DOE minimum of 2.5.

The M&E discussion of peaking factors includes a graph of peak factor based on average flow rate and corresponding equivalent population, based on 70 gpcd. This represents sanitary sewage flow only, without I/I. The maximum factor is 4.0, for up to about 5,000 persons, then declines to about 3.6 for about 15,000 persons.

The ASCE design guide includes a similar graph compiled from seven sources, including the same source used by DOE.

The District has installed flow meters at the discharge of several of its system pump stations. Flow data is collected and trended via the SCADA system. The

Operations Manager can review flow trends by pump station basin to periodically monitor flow by basin.

Residential wastewater discharges to the sewer system are typically in a diurnal pattern, with low flows overnight, peak flow during the morning, lower flows mid-day and a secondary peak in the early evening. Such flows accumulate and the peak factor is reduced, due to the different travel times from the source to a given point in the system, such as the inlet to a pump station wet well. Once the gravity flow from a basin enters a wet well, it is stored for a period of time, then pumped to the next basin downstream. In the BBWSD system, this can occur once or up to seven times in series before a particular discharge reaches the WWTP. This flow path and pump system changes the typical diurnal flow pattern, to some degree.

Dry weather flow patterns were evaluated in support of this Plan, to better understand the sanitary flow variation over the course of a day in this system. Flows were evaluated at five stations in series to determine if peaking factors decreased through the system, as reported by some of the references summarized above. The data is from the discharge of the pumps. The actual hourly variation of flow into each wet well is not known, but presumed to be relatively close to the pump discharge pattern, given the very small volume of storage in each wet well, between pump start and stop levels.

The data revealed a relatively consistent diurnal curve for all the basins, repeated over several days of dry weather. The highest flow was between 1.2 and 1.4 times the average. This indicates that the storage in the wet well substantially reduces the peak factor, and appears to normalize it regardless of total accumulated flow, population or basin area.

Based on these observations, two sets of peaking factors are recommended. As presented in Table 4.1, a traditional set of values should be used for planning new sewer extensions or small basin studies. Reduced peaking factors are recommended, for purposes of system hydraulic modeling. Given the inability to reliably measure flows at the inlet of each wet well, a peak factor for sanitary discharge of 1.8 is recommended, regardless of basin size. This value is almost 30% higher than the observed discharge peak factor, and about the same amount lower than the DOE minimum of 2.5.

4.6 INDUSTRIAL WASTES

Present land-use designations and zoning within the sewer study area do not provide for industrial development. Therefore, loading criteria for industrial wastes does not appear to be warranted and the discharge to the treatment plant should continue to be characteristically domestic sewage.

4.7 GROUNDWATER INFILTRATION

The quantity of water which may infiltrate into a sewer is rather indeterminate and will generally increase with the age of the sewer. However, the design of the sewer system and careful construction techniques have a considerable impact on the amount of infiltration. The porosity of the pipe material and the type of pipe joint also influence the amount of groundwater that can enter a sewer. By the use of rubber gasket joints, the District can be assured that the pipe joint will be more effective, remain in better condition and last longer than would other types of joints. The use of longer length, impervious PVC pipe reduces the number of joints in a collection system and consequently helps to reduce infiltration. A significant portion of the system has been and will be constructed with this type of pipe. The design basis for groundwater infiltration for new sewer mains is 600 gallons per acre per day.

4.8 SURFACE WATER INFLOW

Surface water inflow consists of water that may enter the sewer system through illegal connections from roof, footing and area drains, as well as open side sewer connections left unplugged during construction. These types of sources are of concern in the design of a sanitary sewer system since the amount of flow from the source may exceed the design capacity of the sewer, thereby causing the sewer to become surcharged or overloaded. Even though this type of connection is strictly prohibited, it occurs, and an allowance of 500 gallons per acre per day has been and will be made for it in the design of these facilities. Evaluation of actual infiltration and inflow is discussed in Chapter 5.

TABLE 4.1

DESIGN CRITERIA FOR SEWAGE FLOWS

Parameter		Criteria		
Quantity of Sanitary Sewage (Average)		70 gpcd		
Population Density		See Chapter 3, Table 3.2		
Quantity of Sanitary and Industrial Waste (Average)		See Chapter 3, Table 3.2		
Peak Infiltration and Inflow (I/I)		(for new sewer only – see Chapter 5 for existing I/I analysis)		
Peak Infiltration		600 gpad		
Peak Storm Inflow		500 gpad		
		1100 gpad= 0.0017 cfs per acre		
Peaking Factors for Sanitary Waste (new extensions):				
	100 ac.	1,000 ac.	5,000 ac.	10,000 ac.
Residential	4	3	2.5	2.5
Peaking Factors for Sanitary Waste (system analysis): 1.8				

When the sewer system began in the mid 1970s, there was no storm drainage or management system of significance in the then-developed area. As development continued, particularly in the later 1980s and early 1990s, more attention was placed on stormwater management, including the requirement to include local collection and detention systems in new development. This situation has left portions of the service area with inadequate means of managing stormwater runoff. Faults in the sewer mains and manholes, and, in some cases, illegal connection to the sanitary sewer system attempting to alleviate local runoff problems, can result in increased flows to the District pump stations and WWTP. With the increased attention on development around Birch Bay following adoption of the *2004 Birch Bay Community Plan*, local residents and the County intensified efforts to improve stormwater management. Ultimately, in 2007 the Birch Bay Watershed and Aquatic Resources Management District (BBWARM) was formed. BBWARM has been and continues to develop and implelent a stormwater program for the Birch Bay Watershed.

The design criteria indicated above for inflow and infiltration (I/I) are presented for planning purposes only, recognizing that even a “tight” sewer system will inevitably allow some extraneous water into the manholes and pipes. The figures presented above assume a seasonably high level of infiltration and a peak inflow rate for a storm event. This supports reasonable design of future facilities. The emphasis should remain, however, on elimination of as much I/I from the system as feasible. I/I results in higher operation and maintenance costs and higher-capacity facilities at a higher capital cost.

District Code 8.04.350.A prohibits the discharge of storm drainage, surface water or similar extraneous sources to the sewer system by any customer. Resolution No. 691 includes several defined terms regarding I/I, including: “Excess I & I”, “Excess I & I Report”, “I & I” and “Wet Weather Period”. In the context of considering if I/I is unacceptable in a particular sewer basin or from a particular customer, a different threshold of I/I is necessary. The criteria of gallons per acre per day may not be suitable on a relatively small commercial property where flow monitoring or observations clearly reveal a storm-induced wastewater flow of several times the average daily dry weather flow. The District is entitled to enforcement for any violations of provisions in Sections 8.04.350 and 8.04.360.
If the Excess I & I is confirmed, corrective action is to take place. First, the General Manager must determine the Source(s) of Information. Second, an Investigation and Report directed by the General Manager shall review the evidence of Excess I & I. The General Manager will then provide the Report to the Board of Commissioners with a recommended course of action. Third, a Notice and Public Hearing shall be scheduled before the Board of Commissioners to review the Report. At least ten (10) days prior to the hearing, a written notice shall be provided to the owner of the premises suspected of discharging Excess I & I. Finally, upon review of the Report and following the public hearing, the Board shall make a determination on the matter. If it is determined that Excess I & I is being discharged from the subject remises, the Board may send a written notice the owner directing them to execute specified improvements to abate Excess I & I.
As discussed in Chapter 5, the flow data collected at several stations allows the District to evaluate smaller basins to detect higher incidences of I/I.

4.9 DESIGN OF SEWER SYSTEM FACILITIES

The recommendations which follow are for preliminary design of interceptors, trunk sewers, force mains, inverted siphons and pumping stations.

A discussion of the existing wastewater treatment plant is presented in Chapter 6.

<u>Collection and Pumping Facilities</u>
The ideal method of collecting sewage from a community is by gravity sewers; this is the most economical method when physical conditions permit. Sewage collection by this method is dependent upon the topography of the surrounding land. Many times the topography is not suited for sewage collection by gravity in which case pumping facilities must be constructed. Pumping facilities increase both initial and operating costs over those of normal gravity type sewers. There is a point, however, at which the construction costs and physical parameters

associated with gravity sewers become overwhelming and then pumping facilities must be considered regardless of the topography.

Many communities in the Northwest use a combination of gravity and pumped sewage facilities; Birch Bay is among them.

The natural drainage basins surrounding Birch Bay discharge into the bay itself. Sewage flows within these individual basins can be collected by gravity sewers, but at the point where these drainage basins reach Birch Bay, the sewage must be pumped from there to an adjacent basin and on to the treatment plant. Other areas outside the Birch Bay drainage basin, together with portions inside the drainage basin must also be served by pump systems because of economic, physical and environmental circumstances.

One of the problems encountered with the construction of gravity sewers in the Birch Bay area has been poor soil conditions. Construction of gravity sewers that require deep excavations, usually in excess of 15 feet, has been difficult with sloughing of the trench walls and poor foundation soil. These problems can result in deflection or misalignment of the sewer pipe. These conditions increase construction costs considerably on deep gravity sewers, making the use of lift stations and shallower gravity lines more economical in some cases. Prior to final design the economics of a deep gravity system versus a lift station and shallow gravity line should be reviewed in order to determine the appropriate design approach for a specific project.

Trunk and Interceptor Sewers

The sewers must be designed with sufficient capacity to carry the anticipated peak flows from the ultimate development of the tributary area. This flow represents the sum of the several loadings calculated separately for each section of sewer or tributary area. The loadings consist of the peak flow of sanitary sewage, groundwater infiltration, surface water inflow and any special quantities that must be considered.

The ability of a sewer to transport suspended solids contained in sewage is related to the velocity of flow in the sewer. A velocity of two feet per second (fps) is generally considered to be the minimum which will keep pipe surfaces relatively clean and free of deposited material. Table 4.2 presents the minimum allowable slope of various sizes of sewers to obtain a cleaning velocity under average flow conditions. Minimum slopes are not acceptable for all sewers. Sewers with low flow rates should have increased slopes or they may become maintenance problems due to deposition of solids.

The most commonly used equation for open channel and non-pressurized pipe flow is the Manning equation. It is an empirical formula utilizing the Manning resistance coefficient "n". The resistance coefficient is a function of pipe roughness and material. An "n" value of 0.013 is recognized as appropriate for gravity sewer system design. The appropriate minimum diameter for a given gravity line is determined from the Manning equation with the suitable combination of minimum velocity and desired capacity (peak flow).

TABLE 4.2

MINIMUM PIPE SLOPES

Pipe Size in Inches	Slope* (Feet/Foot)
8	0.004
10	0.0028
12	0.0022
15	0.0015
18-21	0.0012
24-30	0.0008

*Minimum slope for various sized sewer pipe necessary to maintain a cleansing velocity of two fps. See Chapter 8 for discussion of minimum District Standards

<u>Force Mains and Inverted Siphons</u>
The design of force mains and inverted siphons is predicated on the fact that they flow full and under pressure. As in the case of gravity sewers, the mains must be capable of carrying the peak flow from a given area. Proper cleaning velocities are obtained in a force main by selection of a pipe size that will insure this with a specified pumping capacity.

Inverted siphons may consist of two or three parallel lines of different sizes to obtain the desired velocities. Inlet and outlet structures provide for use of one line until the flow increases to the point where the capacity of the second line is needed.

Since the design flow is either pumped or divided between parallel lines, force mains and siphons are commonly of smaller size than adjacent gravity sewers. The empirical Hazen-Williams equation is commonly utilized for analyzing pressure flow conditions. A discharge coefficient "C" is used in the equation to account for the roughness and condition of the material. The typical value of "C" of small diameter pressure mains (PVC or ductile iron) is 130. The appropriate pipe diameter is determined from the Hazen-Williams equation with consideration for the desired minimum velocity of 3.5 fps.

<u>Pumping Stations</u>
Wastewater pumping stations are generally constructed underground, either as factory assembled package units or custom designed stations. The District's standard station is a custom, submersible duplex pump station with concrete wet well and valve vault. An on-site standby power generator is the standard

requirement unless other conditions warrant a different approach to reliability. Gravity overflow from the station's wet well to a downstream gravity sewer may be possible at lift stations. The solids are retained in the wet well during a power outage but the liquid flows downstream without overflowing the system. Once normal operation resumes, liquids and solids are pumped out of the wet well. Capacities of permanent pumping stations are based on the peak flow of all sewers tributary to the individual station. Stations are frequently designed to allow for staged increases in pumping capacities, with pumping units installed as required by growth and consequent flow increases.

Pumps are usually driven by electric motors, are of a non-clog design, and are of a number of units sufficient to pump the design peak flow with any one unit out of service. Mechanical failures are avoided by providing a duplication of pumping capabilities in each pump station. Problems resulting from power outages are avoided by providing on-site standby power generators with electrical power failure alarm systems. The stations are monitored at the treatment plant office via telemetry.

Sewer Materials

The primary material acceptable for sewer pipe construction is polyvinyl chloride (PVC) for gravity mains and high density polyethylene (HDPE) for force mains. Ductile iron pipe is also employed where its use is justified due to scouring velocities or other unique constraints. The pipes should be connected by flexible, rubber-gasket type joints, or other approved materials.

Construction of manholes with precast, reinforced-concrete bases, rings and cone sections with rubber gasketed joints and external seals at all joints is the standard. The use of manhole block construction is no longer allowed in an effort to reduce infiltration.

Sewer Locations

In general, the trunk and interceptor sewers will be located in existing street rights-of-way or in proposed street areas. Certain sewers will have to be located on easements following natural drainage courses.

The location of the sewer lines in relation to other utilities is also worth consideration. There may be some conflict in final sewer locations due to interference with water mains, drains and electrical conduits. In most cases, however, sewer lines would pass beneath the other utilities. This is especially true in the case of water mains, where it is desirable to have the sanitary sewer a minimum of 18 inches (from top of sewer to bottom of water) below the water main.

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CHAPTER 5

EXISTING SEWER SYSTEM

5.1 INTRODUCTION

In the development of a comprehensive sewer plan for the District, it is necessary to consider the condition and capacity of the existing collection system and treatment facilities in order to determine their ability to meet present and future needs.

5.2 HISTORY OF SEWER IMPROVEMENTS

The Birch Bay area saw its first sewer construction in 1973 in the form of a small collector system built in Birch Bay Village in anticipation of the District's treatment plant and interceptor sewer construction. In early 1975 a total of six sewer projects began construction, including interceptors and lateral sewers and the wastewater treatment plant. Construction was also started on seven sewage pump stations that year. These stations move the sewage around the bay from north to south where the wastewater treatment plant is located. Later that year three more projects went into construction: another pump station, lateral sewer and a developer’s sewer extension. Table 5.1 is a summary of these and all sewer system construction projects to date.

Since 1975 the majority of the sewer construction in the Birch Bay area has been on newly developed land under the developer extension policy, whereby the land owner (developer) constructs the sewer lines in accordance with Birch Bay Water and Sewer District’s rules and regulations. When construction is complete and the sewer lines have been tested and accepted by the District, the developer deeds the lines to the District. When this transaction is completed, the District accepts responsibility for operation and maintenance of the sewer lines. Many of the sewer laterals will continue to be built by this method. Portions of interceptors may be built by the developer extension method, but some interceptors and laterals will continue to be built by the District, using the utility local improvement district (ULID) method, where conditions warrant.

TABLE 5.1

HISTORY OF SEWER PROJECTS

Contract No.	Project
73-1	Birch Bay Village
74-1	Interceptors and Lateral Sewers
74-2	Interceptors and Lateral Sewers
74-3	Interceptors and Lateral Sewers
74-4	Wastewater Treatment Plant
74-5	Sewage Pump Stations No. 3-9
74-6	Outfall and Diffuser
75-1	Lateral Sewers
75-2	Sewage Pump Station No. 1
75-3	Holiday Park Phase I
76-1	Birchmont
76-2	Harbor View Estates
76-3	Bay Ridge Estates
76-4	Woodhaven
76-5	Bay Center Resort
77-1	Holiday Park Phase II
77-2	Richmond Park
78-1	Harbor View Interceptor
78-2	Birch Bay Village Div. #15
78-3	Bay Rim Park
82-1	Blaine AFB/McAlpine Road Extension
83-1	Sea Links Subdivision
86-1	Wastewater Treatment Plant Additions and Modifications
86-2	NACO West Mobile Home Park
90-1	Latitude 49 (Osberg)
91-1	Semiahmoo Center Extension
92-1	Sea-Links Phase II
92-2	Lincoln Green
92-3	Vitalis (Elaine Street)
92-4	Loomis Trail Pump Station, Force Main and Gravity Sewer
93-1	Loomis Trail Golf Course, Clubhouse and Maintenance Building

Contract No.	Project
93-2	California Creek Crossing
93-3	Sunday Harbor
93-4	Holeman Avenue
94-1	Lincoln Green (Daybreak Place)
94-2	Double R Ranch
94-3	Lincoln Green (Moonglow Place)
96-1	Point Whitehorn
96-2	Anchor Manor
97-1	Plaza Park Mobile Home Court
99-1	WWTP Improvements
99-2	Loomis Trail Force Main Bypass
00-1	Anchor Manor Phase 1B Sewer Main Extension
01-1	ARCO (Cherry Point Refinery) Sewer Service
01-2	Plaza Park RV Park Sewer Main Extension
01-3	Drayton Heights Phase 1 (Lincoln Green Sunset PL) Sewer Main Extension
01-4	Maple Leaf Village Sewer Main Extension
02-1	Anderson Park Phase 1 Sewer Main Extension
02-2	Drayton Heights Phase 2 Sewer Main Extension
02-3	Richmond Park Div. II, Phase II Sewer Main Extension
02-4	Bay-Crest Phase IA, IB, 2A Sewer Main Extension
03-1	Anchor Manor, Phase II
03-2	WWTP Headworks Improvements
03-3	Loomis Trail PUD - Sewer
03-4	Semiahmoo Center Lot 2 Sewer
03-5	Anchor Village Sewer
04-1	Baycrest North, Phase 1,3 - Sewer
04-2	Baycrest North, Phase 2,4 - Sewer
04-3	Anderson Park, Phase 1B Sewer
04-4	Baycrest, Phase 2B Sewer
04-5	Sandcastle Condos Sewer
04-6	Greens at Loomis Trail, Phase 2 - Sewer
04-7	Bayview Terrace - Sewer
05-1	Pump Station #3 Rehabilitation
05-2	Malibu Sewer Extension

Contract No.	Project
05-3	Birch Bay Condominiums Phase 1 Sewer Extension
05-4	Baycrest South Sewer
05-5	Birch Bay View Sewer Extension
05-6	Lincoln Green Tract B and C Sewer Extension
06-1	Horizon at Semiahmoo Phase 1 Sewer Extension
06-2	Whitehorn Way Sewer Extension
06-4	Bay Breeze Sewer Extension
07-2	Bay Road Sewer Extension
07-4	Broadway Lots Sewer Extension
08-1	PS #3 Force Main and PS #1 Replacement
08-3	Tides at Birch Bay - Phase 1 Sewer Extension
10-1	Bay Road 3 Lots Sewer Extension

5.3 EXISTING SEWER SYSTEM

General:

The existing collection system provides sewer service for the natural basin area surrounding Birch Bay, and for areas in the adjacent natural drainage basin for California Creek, which drains to Drayton Harbor. The major interceptor system follows the shoreline of Birch Bay and presently includes seven pump stations (#3-9), 33,368 lineal feet of 15 and 18 inch sewer pipe, and 18,166 lineal feet of 14 and 16 inch force main. Pump Station #1 collects wastewater from the Point Whitehorn area. Two additional stations (Loomis Trail and Blaine Road) collect wastewater from the northern and eastern extents of the service area, and discharge to the interceptor system along Alderson Road, upstream of Pump Station #4. One station provides domestic wastewater service to the Cherry Point Refinery industrial property southeast of the District. This station discharges to the collection system upstream of Pump Station #1.

The sewer system is comprised of 272,927 LF (51.7 miles) of 8 to 27 inch gravity sewers, 11 pump stations, 50,397 LF of force main and 7,670 LF of 27 inch outfall. The existing system is shown on Figure 5.1. Table 5.2 presents the existing sewer system pipe inventory. Table 5.3 identifies and describes the District's pump stations. The wastewater treatment plant (WWTP), located near the south limit of Birch Bay, east of Point Whitehorn, has a current permitted capacity of 1.44 million gallons per day (average daily flow for the maximum month). As of December 2018, there were 7,061 sewer equivalent living units (ELUs) connected to the sewer system.

For help with characterization of inflow and infiltration as discussed below, the unit of measure of inch of diameter per mile of pipe or simply inch-diameter-mile

(IDM) is introduced in this Plan update. The length and size of District sewer mains included in Table 5.2 can be expressed as 500 IDM, and that measure can be calculated for each basin. The outfall and force mains are excluded from this calculation. Two additional elements to be considered for inflow and infiltration analysis include the side sewer stubs (i.e. the typically 6 inch District-maintained stubs from the main to the edge of the right of way or easement) and the side sewer laterals (i.e. the typically 4 inch privately-maintained side sewers from the stub to the building plumbing connection). The lengths of these elements have been estimated by counting the approximate number of served properties, estimating how many of those properties are served by stubs shared by the adjacent property, then multiplying by the estimated average length of the stub and estimated average length of side sewer lateral. These estimates suggest an additional approximately 221 IDM of collection system. A third element is the private sewer systems serving developments such as large condominium complexes and recreational vehicle parks. Limited records are available for these developments but estimates have been made for the size and length of mains, stubs and laterals within the 15 largest private systems connected to the District collection system. These estimates suggest an additional approximately 103 IDM of collection system. The total collection system includes approximately 824 IDM of pipe and this total has been allocated to each collection system basin, as further discussed below for inflow and infiltration analysis.

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TABLE 5.2

EXISTING SEWER SYSTEM INVENTORY

		District Collection and Conveyance piping - not including side sewers or private laterals													INCH-DIA.-MILE (gravity only)			
BASIN	**# MH's**	**5"**	**6"**	**8"**	**10"**	**12"**	**14"**	**15"**	**16"**	**18"**	**21"**	**24"**	**27"**	**Total**	**District Mains**	**Side Sewers**	**Private Systems**	**Total**
1-Outfall													***7,670***	7,670				
1	95	0	0	20,606	654	172	0	0	0	0	0	0	0	21,432	32.9	16.8	4.8	**54**
PS 1 FM				1,326										1,326				
3	155	0	0	24,998	0	1,745	0	1,499		2,855	0	0	0	31,097	55.8	33.9	35.7	**126**
PS 3 FM							9,037		9,029	9,027				27,093				
4	98	0	0	13,395	373	2,465	0	1,603		1,281	35	0	0	19,152	35.7	15.5	37.1	**88**
PS 4 FM					190									190				
5	102	0	0	16,313	2,961	0	38	2,624		0	0	0	258	22,194	39.2	21.0	13.0	**73**
PS 5 FM				43										43				
6	139	0	150	23,528	1,331	3,666	0	6,519		40	0	0	0	35,234	65.3	24.1	12.0	**101**
PS 6 FM				373										373				
7	140	0	0	23,075	1,531	1,647	0	4,623		1,975	0	0	0	32,851	61.5	35.3	0.0	**97**
PS 7 FM				32										32				
8	133	0	0	25,345	500	0	62	1,124		5,736	982	23	0	33,772	66.3	18.7	0.0	**85**
PS 8 FM				600										600				
9	147	0	300	30,605	1,159	2,571	0	3,489		0	0	0	0	38,124	64.7	32.9	0.0	**98**
PS 9 FM			57											57				
Blaine Rd.	98	0	0	13,804	0	2,431	80	6,180		8	0	0	0	22,503	44.2	11.6	0.0	**56**
PS BR FM				4,911										4,911				
Loomis Trail	77	0	0	9,665	885	1,680	0	126		4,202	0	11	0	16,568	34.9	11.3	0.0	**46**
PS LT FM					7,275									7,275				
PS CP FM		6489		20										6,509				
PS St. Pk FM		*1,988*												1,988				
Total	**1,184**	**8,477**	**507**	**208,639**	**16,859**	**16,377**	**9,217**	**27,787**	**9,029**	**25,124**	**1,017**	**34**	**7,928**	**330,994**	**500.4**	**221.1**	**102.7**	**824**

Gravity	**51.7**	**miles**	**272,927**	**feet**
Force Main	**9.5**	**miles**	**50,397**	**feet**
Outfall	**1.5**	**miles**	**7,670**	**feet**
Total	**62.7**	**miles**	**330,994**	**feet**

Estimated number of side sewer stubs: 2953

TABLE 5.3

EXISTING PUMP STATIONS

No.	Type	Capacity and TDH (1)	Location	Test (2)	Capacity Shortfall (6 – gpm)	Shortfall (%)
1	Flygt CP3152-454, VFDs (tied to WWTP generator)	775 GPM @ 63' TDH	WWTP Driveway	585	(190)	-25%
3	Flygt CP3231 with generator and VFDs	2,300 GPM @ 105' TDH	Cotterill Blvd and Maple Street	1904	(396)	-17%
4	Flygt CP3172 with generator and VFDs	2,400 gpm@ 25' TDH	Alderson Road and Birch Bay Dr.	2030	(370)	-15%
5	Flygt CP3200	1,480 GPM @ 46' TDH	Birch Bay Dr and Shore Acres Resort	1500	20	N/A
6	Flygt NP3153, with generator and VFDs	1,225 GPM @ 25' TDH and 1,120 gpm @ 46' TDH (6)	Harborview Rd and Birch Bay Drive	1200 and 1340	(25) and 220	-2% and 20%
7	Flygt NP3127.095-422	1,150 GPM @ 14' TDH	Birch Bay Drive and Beach Way Drive	1158	8	N/A
8	Flygt CP3127 with generator	730 GPM @ 26.5' TDH	Salish Rd. and Nootka Loop (Birch Bay Village)	625	(106)	-14%
9	Flygt NP3102.090-1128	450 GPM @ 20' TDH	Sehome Road and Sehome Court (Birch Bay Village)	465	15	N/A
LT (3)	Flygt CP3170 with generator	450 GPM @ 100' TDH	Loomis Trail Road and Blaine Road	562	112	25%
BR (4)	Flygt CP 3152 with generator	450 GPM @ 88' TDH	Blaine Road, north of Arnie Road	403	(47)	N/A

TABLE 5.3 (cont.)

EXISTING PUMP STATIONS

No.	Type	Capacity and TDH (1)	Location	Test (2)	Capacity Shortfall (6 – gpm)	Shortfall (%)
CPR (5)	Flygt CP 3140 with MTS	153 GPM @ 110' TDH	Grandview Road east of Jackson Road	165	(12)	-56%

1 – Each station has two pumps; the capacity shown is the nameplate capacity for one pump with a second, identical pump for standby.

2 – Flow capacity (gpm) determined by discharge flow meter, average for both pumps, on 11/1/16, except OPR tested 6/4/19. Loomis Trail and Blaine Road are not equipped with discharge flow meters so test results are calculated per District draw down tests conducted 11/1/16. CPR force main was cleaned by use of foam pits on 6/4/19. Performance results following cleaning.

3 - Loomis Trail Pump Station.

4 - Blaine Road Wastewater Pump Station.

5 - Cherry Point Refinery Domestic Wastewater Pump Station

6 – Capacity shortfall is the difference between design capacity and average test flow rate.

7 – Pump 2 was purchased as a replacement with the understanding that the impeller was for a higher head application. It operates within the electrical load rating of the station equipment and provides additional capacity for the near term.

Prior Analysis:

The District's wastewater collection system was analyzed in detail in 1995 and the analysis was updated in 1999 as part of the work to complete the 2000 *Comprehensive Sewer Plan*. The analysis utilized the computer program HYDRA to track estimated flow from sanitary contributions and inflow/infiltration, based on the criteria in Chapter 4, through the collection system to the WWTP. The analysis assumed full development of the sewer service area. The HYDRA program then compared the projected flow in each sewer segment included in the model to the gravity flow, full-pipe capacity of that sewer segment. The analysis included all pipe segments serving an area larger than the capacity of an eight-inch sewer main installed at minimum slope (i.e. the smallest basin analyzed could be served by the minimum capacity eight-inch sewer). The analysis assumed that each pump station discharged continuously at its tested pump capacity to the downstream gravity sewer.

The analysis completed in 1999 revealed that, if future flows matched that anticipated using the District's planning criteria, 57 segments of existing main would be over capacity. However, two significant changes have occurred since the analysis was completed in 1999: change in sewer flow per capita and changes in sewer service area.

In 1999 and previously, the sewer flow per capita criteria was 85 gallons per day. The District reduced this to 70 gallons per capita per day with the 2009 CSP update. This value is reviewed in Section 5.5 below.

The other significant change is adjustment of the sewer service area. Several hundred acres of land at Birch Point and between Lincoln Road and Drayton Harbor have been removed from the UGA and sewer service area, thus reducing the contribution to the interceptor system along Birch Bay Drive from Pump Station #8 to Pump Station #3. An irregularly shaped area on either side of Blaine Road, north of Arnie Road, has also been removed from the UGA, but this area is designated as UGA Reserve (see Figure 3.1).

Those and other service area changes were accounted for in the District's 2007 *Capacity Upgrade Planning Report* (CUPR) which included specific recommendations for upgrades along Birch Bay Drive from Pump Station #8 to the WWTP. Analysis completed for consideration of increased domestic wastewater capacity for the BP Cherry Point Refinery supported review of the impact of the UGA change at Point Whitehorn. Review of local system needs in the vicinity of Blaine and Alderson Roads supported determination of collection system sizing along Blaine Road north to the Blaine Road Wastewater Pump Station, and west along Alderson Road to Birch Bay Drive.

The CUPR, as refined by subsequent additional review from Pump Station #3 to the WWTP, recommended the following upgrades:

- force main from Pump Station #4 to WWTP. (This has been partially implemented with the PS #3 force main replacement project which completed installation of 14, 16 and 18-inch force mains from PS #3 to the WWTP. The 18-inch pipe is presently not in service.),
- upgrade Pump Station #4 for discharge directly to WWTP and to receive flow from gravity trunks upstream along Birch Bay Drive and Alderson Road,
- install parallel gravity sewer from Pump Station #8 to #6 and from #6 to #4. These gravity segments will bypass Pump Stations #7 and #5, but will receive over flow from the latter stations,
- upgrade Pump Stations #8 and #6.

The District has completed a pre-design report to address general upgrades to its wastewater pump facilities (*General Wastewater Pump Station Pre-Design Report,* September, 2008, PS Report). The report focuses on Pump Stations #1, 5, 7 and 9 as they are the oldest stations and significant capacity upgrades are not recommended at these stations as discussed in the CUPR. However, the PS Report does recommend upgrades to all stations for consistency in access, equipment controls, etc. Specific recommendations and schedule for pump station upgrades are presented in Chapter 7, and are summarized as follows:

- repair roof of electrical equipment vault at Pump Station #8,
- upgrade/replace Pump Stations #4 and 6,
- upgrade equipment at Pump Station #5 and 7.

The wastewater treatment plant discharges to the Strait of Georgia via a 24-inch gravity outfall. The WWTP is discussed in Chapter 6.

There are no controlled overflows from the wastewater collection system to the environment.

Hydraulic Capacity Analysis:
The existing gravity collection system was evaluated for hydraulic capacity by means of a computer model. The hydraulic modeling software, SewerGEMS (Bentley Systems, Inc.), has been used to analyze the major gravity lines within the collection system for current conditions, and buildout (i.e. full development of the UGA and service area per the discussion in Chapter 3). Peak hour flows developed in the model were compared to current pump station and force main capacity to evaluate existing facilities capacity, for present and buildout conditions.

The hydraulic model consists of an integrated collection of physical attributes for the collection system and assignment of forecast flows from sanitary contributions and inflow and infiltration. The physical attributes represent the manholes (nodes) and conduits (pipe) through which the flow is conveyed. Hydraulic contributions can be introduced at nodes representative of their point of collection in the system, and those contributions can be represented in a variety of methods.

Inflow and infiltration are estimated together as a peak hour flow, as described above, and input in the model as a fixed peak hour flow, at representative nodes in each basin. In the model, the peak sanitary flow and peak inflow and infiltration are additive, whereas in reality that might be the case, or a heavy rain could occur overnight, hours before the time of peak sanitary flow.

In the model, the discharge from each of the pump stations to a downstream portion of the gravity collection is included as a fixed discharge to the downstream manhole (i.e. existing force main discharge).

The model predicts the hydraulic grade line for connected pipe segments, starting at the crown of the downstream pipe outlet, typically the inlet to the downstream pump station wet well. This presumes the downstream pump station has capacity greater than or equal to the influent flow rate and is therefore not allowing the wet well level to surcharge the upstream collection system.

About half of the collection system is modeled, as represented by 625 conduits. The upper reaches of the collection system generally do not warrant analysis as the pipes serving small sub-basins typically have more than adequate hydraulic capacity, even at minimum slopes. A schematic of the modeled system is shown in Figure 5.5 along with the sub-basin overlay. Data for each sub-basin of tributary sewer collection area is summarized for each scenario in the model input summary spreadsheets included in Appendix G. Flow is introduced from each sub-basin at a generally centrally-located manhole or node in each sub-basin.

For the existing system, sanitary flow contribution is represented by approximate count of ELUs served at the end of 2017 (2013 baseline for growth forecast adjusted to end of 2017 ELU count), 1.21 persons per ELU and 70 gallons per capita per day (gpcd) of flow. Inflow and infiltration (I/I) is represented by a peak hour rate of 1,923 gallons per acre per day (gpad) and the area of existing parcels served by the system (see discussion in Section 5.5), adjusted to 2017 proportional to ELU growth from 2013 to 2017. The future system includes the existing ELUs served, forecast additional residential housing units and an allowance for additional commercial development, with flow based on 1.33 persons per ELU and 70 gpcd of flow. Future I/I is forecast to be somewhat higher, at 2,200 gpad, for an expanded area of parcels served.

Preliminary solutions for existing conditions were developed, and considered in the context of the system capacity needs for the future condition, to support development of a capital improvement plan as presented in Chapter 7.

<u>Existing System Deficiencies – Pump Stations:</u>
As indicated in Table 5.3, several of the stations are not performing at their design capacity. The test values are the average for the results for each pair of pumps. Some stations are configured with one of the two pumps fitted with a flush-mix valve or simply a hole in the volute to support mixing of the wet well contents while that pump is on. This results in flow bypass back to the wet well and reduced efficiency and performance, but with other operational benefits. Other factors that may contribute to a performance deficiency include wear of impellers, partial clogging of the force main, air collection at local high points, or actual force main friction factors and head loss differences from such losses as anticipated during design. The following stations are operating at least 15% below design capacity: #1, 3, and 4. The tested capacity for PS #1 is greater than the future forecast peak hour flow so no action is warranted at this time. PS #3 and #4 will be evaluated and upgraded early in the 20-year planning period.

The stations are designed to deliver their design flow with only one pump in service. While some stations have the electrical capability of operating two pumps at the same time, there is typically very little additional total flow, due to the performance characteristics of the pump curves for these stations. Therefore capacity benefits of running two pumps simultaneously are not considered in this analysis.

Pump Stations #5 and 7 are configured with hydraulic overflows rather than standby generators. In the event power is lost, or if the influent flow rate exceeds the capacity of one pump, or even two pumps in operation, the wet well level will increase to the elevation of a gravity overflow to the downstream gravity interceptor. Wastewater will continue downstream and solids that remain in the wet well will be pumped out once conditions return to normal operation. The result of this scenario is an increase in the hydraulic grade line in the interceptor segments upstream of these two stations. This results in a surcharge of 4.0 feet (above the crown of the inlet pipe at the entrance to the wet well) upstream of PS #5 and nearly 3.5 feet upstream of PS #7. This condition, intended only for power outages, has not been modeled in the collection system analysis. Surcharging due to pipeline capacity deficiency would be additive to the overflow conditions summarized above.

<u>Existing System Deficiencies – Collection System:</u>
The existing system was evaluated under three scenarios, for existing estimated peak hour flows:

- Base scenario (existing system, with pumps at current design capacity, with no capacity upgrades)
- Alternative 1 – assumes only the pump stations are upgraded to have adequate peak hour capacity
- Alternative 2 – pump station upgrades and parallel interceptors to provide additional capacity.

The results for each analysis were evaluated with a focus on those pipe segments modeled to be at 85% or greater capacity. Preliminary upgrades were identified for each segment that was over 110% capacity under existing condition, surcharged greater than 0.5 feet or was situated immediately upstream of a segment that met either condition. A different threshold was applied for the future condition analysis as discussed below.

For the Base scenario, the analysis reveals that three stations (PS BR, #6 and #3) and 34 pipe segments are anticipated to have inadequate peak hour capacity under the modeled conditions. The short force mains discharging from PS #5 and #4 are too small for existing estimated peak hour flows (i.e. velocity greater than eight fps). The model results indicate that several of those pipes, and a few additional pipe segments, will be surcharged under the modeled conditions. In many cases, the subject pipe has adequate capacity. This is due to downstream capacity limitations resulting in an elevated hydraulic grade line through the subject pipe reach. These situations will be addressed with downstream capacity improvements.

For Alternative 1, the only change was to increase the fixed flows for each station to represent the minimal upgrades necessary to provide adequate capacity to transfer existing peak flows to the downstream basin or facility. That change in flow conditions increases the number and severity of anticipated capacity deficiencies in stations and the collection system. Upgrades of the three stations under capacity per the Base scenario would trigger need for upgrades at two additional stations (PS #5 and #4), and even more capacity will be required at PS #3. Under this scenario the short force main discharging from PS #6 is also undersized for the existing peak hour flow.

Alternative 2 includes the upgraded pump stations (for existing development conditions) and parallel pipes, for preliminary consideration sized to convey the portion of existing flow greater than 85% of existing pipe capacity. For example, the highest capacity deficiency (relative to 85% capacity) for the interceptor segments between PS #3 and #4 is about 1,270 gpm. For the available slope between the start and end of this interceptor, a parallel 18” pipe would provide sufficient bypass capacity. Using this approach, preliminary gravity pipeline capacity solutions (for existing conditions) are as follows:

- PS #3 to #4: 2,640 feet of 18” parallel interceptor (MH 743-040 to MH 743-029)
- PS #4 to #5: 2,481 feet of 12” parallel interceptor (MH 743-117A to MH 743-085)
- PS #5 to #6: 2,629 feet of 12” parallel interceptor (MH 743-126 to MH 743-119)

The District’s CUPR (2007) concluded, for a larger sewer service area, that the optimal set of capacity upgrades included additional force mains between PS #3 and #4, parallel to existing gravity mains, and additional parallel gravity mains upstream of PS #4.

A pre-design evaluation for upgrades to PS #4 completed in 2009 concluded that a parallel gravity main from PS #3 to #4 is recommended, rather than two extended force mains, as presented in the CUPR and 2009 CSP. In addition, the PS #4 pre-design evaluation concluded that there was a potential opportunity to transfer some gravity sewer flow from PS #4 downstream to PS #3, with a gravity sewer crossing of Terrell Creek. The referenced analysis considered the feasibility of extending a 36” gravity sewer north from PS #3 to either of two potential crossing locations. The reduction in future service area results in a future recommended 24” gravity sewer, as discussed below. The smaller diameter must be installed at a steeper slope. The difference in slope and length to the northerly crossing increases the pipe elevation by nearly one foot, which is about the clearance estimated previously. This may render the gravity option to divert some flows from Alderson Road past PS #4 infeasible. Alternatively, the larger line could be installed in anticipation of capacity needs beyond the 20-year planning period.

The recommended set of improvements will be discussed further below, in the context of improvements necessary to meet anticipated future conditions.

The anticipated capacity deficiencies are summarized in Tables 5.4 and 5.6. Table 5.4 includes a summary of evaluation of the capacity of each station’s force main for the indicated flow scenarios, based on the flow analysis summary presented in Table 5.5.

TABLE 5.4

MODELED SYSTEM DEFICIENCIES - PS & FM

System Element	Existing Deficiency - Base (gpm)	Existing Deficiency - Alt. 1 (gpm)	Future Deficiency Alt. 1 (gpm)
PS CPR	OK	OK	OK
PS CPR FM	OK	OK	OK
PS LT	OK	OK	OK
PS LT FM	Replace	Replace	Replace
PS BR	183	183	528
PS BR FM	OK	OK	OK
PS #9	OK	OK	OK
PS #9 FM	OK	OK	OK
PS #8	OK	OK	OK
PS #8 FM	OK	OK	OK
PS #7	OK	OK	443
PS #7 FM	OK	OK	Replace
PS #6	270	270	938
PS #6 FM	OK	Replace	Replace
PS #5	OK	252	1,047
PS #5 FM	Replace	Replace	Replace
PS #4	OK	324	1,742
PS #4 FM	Replace	Replace	Replace
PS #3	509	833	2,332
PS #3 FM - 16"	OK	OK	Replace
PS #1	OK	OK	OK
PS #1 FM	OK	OK	OK

1 - Base assumes no pump station upgrades.

2 - Alt. 1 assumes all pump stations with existing deficient capacity are upgraded for existing peak hour flow, if necessary.

3 - Future Deficiency indicates additional capacity needed over existing to meet future peak hour flow.

4 - Force main analysis result is based on recommended velocity between 3.0 and 8.0 feet per second. "Replace" indicates need for higher capacity pumps for LT PS (force main is oversized) or replace force main with future station capacity upgrades. PS #3 will use multiple existing force mains to meet future capacity needs.

TABLE 5.5

FORCE MAIN CAPACITY EVALUATION

	Dia. (in)	Minimum Flow (gpm)	Maximum Flow (gpm)	Existing Pump Capacity (gpm)	Existing Peak Flow - Alt 1 (gpm)	2038 Peak Flow (gpm)
PS CPR	5	148	395	***153***	***153***	***153***
PS LT	10	650	1,733	450	112	148
PS BR	8	418	1,116	***450***	***633***	***978***
PS #9	6	264	705	***450***	***323***	***401***
PS #8	8	470	1,253	***730***	***624***	***700***
PS #7	8	470	1,253	***1,150***	***1,104***	1,593
PS #6	8	470	1,253	***1,225***	1,495	2,163
PS #5	8	470	1,253	1,480	1,732	2,527
PS #4	10	734	1,957	2,400	2,724	4,142
PS #3	16	1,440	3,839	***2,300***	***3,133***	4,632
PS #1	8	356	948	***775***	***476***	***534***

1 - Minimum flow is based on minimum velocity of 3.0 fps.

2 - Maximum flow is based on velocity of 8.0 fps.

3 - Diameter is nominal inside for stations #4-9. Actual ID used for HPDE force mains for other stations. Data for PS #3 is for 16" force main only.

4 - Values in bold italics are within the recommended velocity range.

5 - Existing and future flows are as estimated in hydraulic model.

TABLE 5.6

MODELED SYSTEM DEFICIENCIES (EXIST.) - COLLECTION PIPE

General	**Existing Deficiencies – Base (before pump upgrades)**			
Pipes >85% capacity	43 (11,654 LF total - 4.3% of system)			
Pipes >100% capacity	34 (9,194 LF total)			
Surcharged Pipes	14			
Location	**Length (ft)**	**Max. Surcharge (ft)**	**Replace (ft)**	**Replace (in)**
Basin BR	161	none	n/a	n/a
Basin 7	392	none	n/a	n/a
Basin 6	3,011	none	n/a	n/a
Basin 5	2,629	none	n/a	n/a
Basin 4	2,234	0.3	696	18-21
Basin 3	3,227	2.7	2,640	21-24

General	**Existing Deficiencies - Alt 1 & 2 (after pump upgrades)**				
Pipes >85% capacity	52 (13,306 LF total - 4.9% of system)				
Pipes >100% capacity	44 (11,838 LF total)				
Surcharged Pipes	28				
Location	**Length (ft)**	**Max. Surcharge (ft)**	**Replace/ Parallel (ft)**	**Replace (in)**	**Parallel (in)**
Basin BR	161	none	n/a	n/a	n/a
Basin 7	392	none	n/a	n/a	n/a
Basin 6	3,011	none	n/a	n/a	n/a
Basin 5	2,629	2.2	2,629	18	12
Basin 4	4,133	1.6	2,481	18-21	12
Basin 3	3,227	4.7	2,640	21-24	18

1 - A deficiency is identified if modeled flow is 85% or greater of existing pipe segment capacity. Upgrades are identified if flow is equal to or greater than 110% capacity, surcharge greater than 0.5 feet or immediate downstream pipe that meets either condition.

2 - Base is without pump upgrades and Alt. 1 includes pump stations upgraded to exist. peak hour flows.

3 - Alt. 1 upgrade solution proposes replacement piping and Alt. 2 proposes parallel piping.

4 - n/a indicates a local hydraulic deficiency where no action is warranted.

A summary of existing conditions gravity pipe capacity deficiencies and minimum solutions is presented in Appendix G.

Future Conditions Deficiencies

The Future scenario is based on the existing piping system but presumes the pump stations will be upgraded to forecast future peak hour capacity as indicated by the model, by year 2038. Therefore, the future model focuses on the piping conveyance upgrades ultimately needed to provide adequate capacity for the service area.

For the Future scenario, the analysis reveals that six stations (PS BR, #7, #6, #5, #4 and #3) have inadequate peak hour capacity under the modeled conditions, as summarized in Table 5.4. Replacement force mains should be incorporated in the upgrades at PS #4 through #7. Collection system piping deficiencies are summarized in Table 5.7.

For preliminary consideration, future parallel pipes were analyzed to convey the portion of future flow greater than 85% of existing pipe capacity, as described above for the existing conditions Alternate 2 analysis. Using this approach, the preliminary pipeline capacity solutions (for future conditions) are as follows:

- PS #8 to #7: 1,742 feet of 12" parallel interceptor (MH 742-117 to MH 742-105). Alternatively, the force main from PS #8 could be extended, with pump upgrades, to discharge at MH 742-105. This approach would allow installation of a smaller, shallower pressure main to divert all of the flow from PS#8 past the segment with capacity constraint.
- PS #7 to #6: 3,058 feet of 15" parallel interceptor (MH 742-050A to MH 742-001).
- PS #6 to #5: 2,629 feet of 15" parallel interceptor (MH 743-126 to MH 743-119).
- Alderson Road to PS #4: 2,401 feet of 12" parallel interceptor (MH 743-080 and MH 743-042). The project could omit work for two segments (492 feet) of existing steep main with significant capacity, if diameter restriction were acceptable in the interceptor.
- PS #5 to #4: 2,481 feet of 18" parallel interceptor (MH 743-117A to MH 743-041).
- PS #4 to #3: 2,868 feet of 24" parallel interceptor (MH 743-040 to PS #3)[1]

[1] As described above, this extension may be implemented at 36" diameter, if that is critical for allowing gravity diversion of some flow from Alderson Road interceptor to the system downstream of PS #4. An update of the 2009 PS #4 pre-design evaluation is recommended to confirm the final details in this area.

- Baycrest/Jackson Road/Highland Drive: 587 feet of 8” parallel main or 10-15” replacement main (MH 024-002 to MH 743-016, project is low priority due to local condition and limited surcharging for forecast peak hour flow).

TABLE 5.7

MODELED SYSTEM DEFICIENCIES (FUTURE) - COLLECTION PIPE

General	**Future Deficiencies (after pump upgrades)**					
Pipes >85% capacity	61 (15,668 LF total - 5.7% of system)					
Pipes >100% capacity	59 (15,136 LF total)					
Surcharged Pipes	53					
Location	**Length (ft)**	**Max. Surcharge (ft)**	**Surcharge Above Ground?**	**Replace/ Parallel (ft)**	**Replace (in)**	**Parallel (in)**
Basin BR	441	none	n/a	n/a	n/a	n/a
Basin 7	1,742	none	n/a	1,742	18	12
Basin 6	3,058	4.7	n/a	3,058	21	15
Basin 5	2,629	9.8	Yes	2,629	21	15
Basin 4	4,882	8.6	Yes	4,882	15-24	12-18
Basin 3	3,455	11.9	Yes	3,455	10-27	8-24

1 - A deficiency is identified if modeled flow is 85% or greater of existing pipe segment capacity.

2 - n/a indicates a local hydraulic deficiency where no action is warranted.

A summary of future conditions gravity pipe capacity deficiencies and minimum solutions, including sizing for replacement instead of parallel pipe, is presented in Appendix G.

5.4 OPERATION AND MAINTENANCE

The District’s Operations Manager is responsible for operation and maintenance of the wastewater treatment plant, interceptor and collection system piping and the pump stations, under the supervision of the District General Manager. The District’s Sewer Foreman is responsible for the daily operation and maintenance of the sewer system. The District employs two additional persons who are responsible for proper operation and maintenance of the facilities. They carry out specified, scheduled tests as well as respond in times of an emergency.

The general responsibilities of each employee are outlined in Table 5.8. Specific tasks for equipment operation and maintenance are detailed in the District's operation and maintenance manuals. Figure 5.2 is an organizational chart for the District.

TABLE 5.8

SCHEDULED MAINTENANCE OF COLLECTION AND TREATMENT FACILITIES

I.	**PERSONNEL**
	Operations Manager, Sewer Foreman and two Utility Operators
II.	**TREATMENT PLANT**
A.	Check all major plant equipment morning and evening (mornings only on weekends and holidays)
B.	Bi-monthly checks and maintenance on buildings, gear boxes, motors, walkways, railings, alarms, and other equipment
C.	Routine equipment oil changes and lubrication annually, except blowers bi-annually
D.	Sludge, grit and grease removal by private contractor as required
E.	Lab sampling and analysis - 4 to 5 hours daily (weekdays)
III.	**COLLECTION SYSTEM**
A.	Check all pump stations, wet wells and standby generators twice a week (Mondays and Fridays)
B.	Bi-monthly check and maintenance on pump station vaults, grounds, pumps and alarms
C.	Run all standby generators monthly under load
D.	Pull pumps annually for seal and electrical checks (Spring/Fall)
E.	Clean wet wells bi-monthly or as needed to control grease, sludge, etc.
F.	Sewer line cleaning one day per week

5.5 INFILTRATION AND INFLOW AND EQUIVALENT LIVING UNITS

For the past 28 years, the District has progressively increased their efforts to evaluate the collection system and private connections to the system, to identify and work to minimize the amount of infiltration and inflow (I/I). The evaluation efforts include smoke testing, video inspection, manhole inspection and flow monitoring.

The system has been divided into several basins, each directly tributary to one of the District's pump stations. Basin 3 is that area directly served by Pump Station #3, and so on. The initial study efforts concentrated on Basins 3, 4 and the private sewer systems of the former Air Force Base, Leisure Park, Seabreeze RV Park, Birch Bay Resort Park and Beachwood Park. Subsequent phases of the program include evaluation of Basins 8 and 9, 6 and 7, then 5 and 1. The collection systems tributary to the Loomis Trail and Blaine Road pump stations were all constructed since 1993 and are not the focus of detailed efforts yet. Typical conditions identified from prior studies include:

- manhole rims below grade
- localized pipe sags
- private property faults (leaking side sewers, open cleanouts, connected yard or roof drains, etc.)
- minor to moderate infiltration faults
- protruding side sewers

BP owns the wastewater collection system within the refinery upstream of the Cherry Point Refinery Domestic Wastewater Pump Station. Per their agreement for sewer service they evaluate their collection system on a schedule similar to that used for the District's system.

DOE, through the NPDES permit, requires consideration of the magnitude of I/I with respect to thresholds established by the EPA in a 1985 publication titled *I/I Analysis and Project Certification*. In this publication, infiltration is considered to be non-excessive if the average dry weather flow (ADWF) is equal to or less than 120 gpcd. ADWF is defined as the highest average daily flow recorded over 7-14 days during a period of seasonal high groundwater. Inflow is considered to be non-excessive if the wet weather flow (WWF) is equal to or less than 275 gpcd. WWF is defined as the highest daily flow during a storm event. If either component of I/I is found to be excessive, EPA would not certify the project as eligible for grant funding and further evaluation of the cost effectiveness of I/I removal would be necessary prior to grant funding. The primary difficulty with flow monitoring and determining I/I values on a per capita basis in the District is the seasonal fluctuation of population. There is currently no method to reliably estimate the population for a given time of year or peak summer weekends.

The District recently submitted its 2018 Inflow and Infiltration report to DOE (included in Appendix D). It concludes that infiltration is non-excessive at 104 gpcd and that inflow is non-excessive at 224 gpcd. The 2018 report summarizes the District's recent efforts to locate sources of I/I and action taken to reduce I/I. A significant resource in this process is the addition of flow meters at the pump stations so that flow can be measured and recorded at nine of the 11 stations. Evaluation of the flow data allows the District to detect where I/I may be higher and focus their inspection and repair work more efficiently. This review can be conducted annually or following significant or unusual weather and flow conditions.

Figure 5.3 indicates the peak and average daily flows at the wastewater treatment plant (WWTP) for the period 1990 thru 2017. The highest flows are in the winter, corresponding with higher groundwater table and winter storm events. Although the peak weekend summer population may be three to four times the winter population, the flows to the plant are still considerably higher in the winter. Figure 5.4 indicates the daily flow pattern compared to daily precipitation amounts for 2014. This comparison is presented to demonstrate the clear correlation between specific rain events and increases in daily flow. This again confirms the general trend of findings in the collection system evaluation: inflow is a more significant problem than infiltration.

The District has completed repairs to manholes to address inflow and additional manhole and mainline repairs have been identified which will address both inflow and infiltration. These repairs have been made at a relatively minor cost and would be warranted even without consideration for I/I reduction due to general operation and maintenance considerations. Working with private sewer system owners and individual property owners to correct faults on the private connections to the District's system has and will continue to result in inflow and infiltration reduction at relatively minor cost to the District.

The District recognizes there is substantive but non-excessive infiltration and is taking appropriate action to reduce the extraneous flows. Also, see the discussion regarding an I/I standard in Section 4.8.

Inflow and infiltration can be estimated by mathematically isolating portions of the flow stream, as measured at a point in the system, under varying flow and weather conditions. The following discussion presents an analysis that estimates various I/I rates, based on flows measured in recent years at the WWTP.

WWTP records for the three-year period from January 2012 through December 2014, and maximum day flows from January 2015 thru May 2016, have been analyzed for this I/I analysis. Table 5.9 summarizes WWTP influent flows for the three-year period. The reported monthly average influent flows ranged from 0.64 million gallons per day (mgd) to 1.16 mgd in September 2012 and December 2012, respectively. The highest daily flow measured at the WWTP was 3.351 mgd on December 12, 2010. This extremely high flow rate is considered to be an anomaly by the District due to extreme weather conditions and unusual operating conditions. The next highest flows are for three events from 2.2 to nearly 2.5 mgd, with a volume of 2.495 million gallons on January 5, 2015. The ratios of the various rates are summarized below the table.

TABLE 5.9

WWTP INFLUENT FLOWS

Flow	Flow Rate (mgd)
Average Dry Weather Flow (2)	0.719
Annual Average Flow (2A)	0.843
Maximum Month Average Daily Flow (3)	1.186
Peak Day Flow (4)	3.351
Peak Day Flow (4A)	2.495
Calculated Peak Hour Flow (5)	2.57
Highest Measured Peak Hour Flow (6)	3.25

1 - Based on DMRs reporting WWTP influent
2 - Average of July, August, September, 2014
2A - Average of 2012 - 2014
3 - Reported for January, 2011
4 - Highest value as reported for December 12, 2010 (considered an anomaly)
4A - Second highest value as reported for January 5, 2015.
5 - Calculated using DOE's *Orange Book* Figure C1-1:

$$\frac{Q \text{ peak hourly}}{Q \text{ design average}} = \frac{18 + \text{square root }(P)}{4 + \text{square root }(P)} = \frac{18 + 2.84}{4 + 2.84} = 3.05$$

Where P = 8.049 population served (in 1,000s)
(Peak factor shall be a minimum of 2.5)
Peak Hour Flow (PHF) = 0.843 mgd x 3.05 = 2.57 mgd

6 - Actual highest PHF measured - January 5, 2015
(actual peak factor is approximately 3.86)

7 - Ratios

Max. Month : Annual Average	1.41:1
Peak Day : Annual Average	2.96:1
Peak Day : Max. Month	2.10:1
Peak Hour : Max. Month	2.74:1

Winter water use can be used to estimate wastewater volumes entering the collection system because the amount of winter water use typically is equal to wastewater flow except for a minor amount of water that does not enter the sewer system. This must be adjusted for seasonal water use, if used to calculate base flow. In this report, typical commercial (including commercial and other non-residential) winter water use is subtracted from the WWTP typical dry weather flow to arrive at a base flow, including dry weather infiltration, for calculating per capita wastewater flows for residential uses only.

The average dry weather flow for July, August and September of 2014 was 0.719 mgd. During the same period, the average daily flow from the Cherry Point

Refinery Pump Station (CPR PS) was 19,131 gallons per day (gpd). This flow should be excluded as it is entirely for a commercial service area not included in the typical discussion of population and ELUs served. The winter 2013-2014 non-residential flow (based on wet weather commercial water use) of 51,942 gpd and the CPR PS flow can be deducted from the total dry weather flow to estimate the base flow. The estimated dry weather infiltration of 30,000 gpd can also be deducted (see discussion below). The base flow therefore is estimated to be approximately 0.618 mgd, or 77 gallons per capita per day (gpcd) based on an average of 8,049 persons served in 2013. This per capita flow likely includes a small portion of year-round infiltration. This value is higher than as estimated in the *CUPR* as discussed in Chapter 4, but is based on more detailed estimates of flow and population. Residential water use for this same period and same population served was about 67 gpcd. Per the research in support of the District's 2019 *Comprehensive Water System Plan* update (draft in progress concurrently with this Plan update), the average annual water use per single family residential customer for the period 2011-2014 was 105 gpd per residence. Using this factor and the average sewer service population density of 1.47 people/residence (see Chapter 3), the annual average water use is about 71 gpcd. The estimate of per capita flow based on wastewater flow data is the least accurate given the assumptions necessary to isolate the sanitary flow. Based on the other two data samples, this Plan will continue use of a per capita flow criterion of 70 gpcd.

The amount of I/I can be estimated on an annual average, dry weather, maximum month, peak day and peak hour basis by subtracting the estimated sanitary flow at the WWTP from the annual average, dry weather, maximum month, peak day and peak hour flows at the WWTP.

For this Plan, I/I is primarily expressed in units of gallons per acre per day (gpad). The area used as the basis for expression of I/I in units of gpad is not explicitly defined or readily determined. The area from which I/I is generated is a function of the geometry and condition of the sewer system, including pipe diameter and extent of side sewer stubs, side sewer laterals on private property and the potential presence of inappropriate connections to the sewer system (e.g. onsite drainage basins, or roof, footing or crawlspace drains, etc.) The contribution area is also a function of local topography and soil and groundwater conditions. The same basis must be used for calculation of current I/I rates as will be used for forecasting future I/I flows to the sewer system.

This Plan incorporates an approach that uses the area of parcels served by the sewer system as the basis for the I/I contribution area. The estimated area of future developed parcels is added for forecasting future I/I contributions associated with extension of the wastewater collection system. This area is generally readily defined, for both current and future development conditions, but the approach may underestimate the I/I contribution area, as the area closest to the sewer main, the public right of way in most cases, is not included in the I/I contribution area. However, such area is not readily measured for the future condition.

The total area of parcels served (not including Cherry Point Refinery) as of 2013 was 1,596 acres. That total has been reduced for analysis purposes to approximately 1,300 acres by excluding areas of large parcels with limited sewer facilities (e.g. Loomis Trail Golf Course, County Park at former air force base, State Park, etc.). This does not include public right of way or parcels not connected to the system (even if the vacant parcel is served by a sewer main along its frontage). It may include critical areas if such areas are part of the parcel that is connected to the sewer system. The corresponding density of ELUs per acre of directly served parcels is about 5.54.

The calculations and data for the following future service area discussion are summarized in the hydraulic model inputs summary, included in Appendix G.

A data file provided by Whatcom County, as used to prepare the 2016 *County Comprehensive Plan*, identifies the anticipated future population for all parcels in the UGA, where growth is anticipated through 2036[2]. Upon review of the growth allocation at the parcel level, it appears the data allocated population to four significant areas not likely to develop, as discussed in Chapter 3 (e.g. BP property with conservation easement, State Park area and two County park areas). CHS has reallocated that population and housing growth proportionally through the rest of the sewer service area, but has removed the subject parcels from the future service area calculations. As adjusted, Whatcom County expects residential population growth on parcels with a combined area of 1,007 acres in that planning period. By inspection, several of these parcels are already served with sewer but the County anticipates additional population on those parcels. To estimate the additional parcel service area, 203 acres were deducted from the growth parcels total area. The resulting estimated additional residential service area is 804 acres. Adjusting this value for 20% non-developed uses (future right of way, critical areas and buffers, common area, etc.) results in 643 acres of additional residential parcel service area. The adjusted County forecast of additional housing units for this area plus housing units in the sewer service area outside the UGA (total of 3,475) was used to forecast future residential ELUs (through 2036, from the 2013 baseline).

The sewer service area population forecast based on County growth projections (13,381) and increase in housing units or ELUs from 6,648 to 10,123 by 2036 results in an increase in population per ELU from 1.21 to 1.32. This density is used to estimate additional ELUs for years 2037 and 2038, from the population estimate in Table 3.2.

The County has not, to CHS's knowledge, forecast future commercial development at the parcel level. As described in Chapter 3, there are several commercial zoning districts in the sewer service area. The parcels in each zoning district were analyzed using the County population housing data and aerial photos to support an estimate of commercial area remaining available for

[2] See Chapters 1 and 3 for existing ELU and population discussion, including forecast through year 2038.

development. The gross remaining area was reduced by 35% to account for critical areas, existing and future rights of way, etc. The total estimated commercial growth area is about 160 acres. This approach potentially overestimates the area and corresponding future commercial ELUs as some development in the commercial zoning districts may be residential (e.g. mixed use development) and accounted for in the County residential population growth forecasts at the parcel level. Applying the equivalent employee population factor of five per acre and a future population density of 1.32, as discussed in Chapter 3, results in an anticipated additional 601 commercial ELUs. This is likely an overestimate but does represent the potential impact of available commercially zoned land.

Applying the anticipated total direct service ELU count (11,081 ELUs) to the total future area of parcels directly served (2,104 acres) results in an estimated future service area density of about 5.27 ELUs per acre. This compares reasonably with the current I/I contribution area sewer connection density and suggests that the 20% allowance for future common area uses is reasonable.

Table 5.6 summarizes the infiltration/inflow analysis for the three-year period 2012-2014. The area served by the CPR PS, and corresponding flow data, is not considered in this analysis. Table 5.6 reflects the total WWTP flow record, and the adjustment to exclude the area served by this one pump station. The data contained in this table is used for analysis of the existing system, and as a baseline for forecasting infiltration and inflow in the future as discussed below.

In order to estimate the base flow, flow records at the WWTP were reviewed. A lowest, hourly low flow was observed on October 10, 2012, of 0.226 mgd during the early morning. During that hour, the lowest instantaneous flow rate was 0.058 mgd. If it is estimated that about half of that lowest flow rate is due to infiltration, about 0.03 mgd can be allocated to infiltration. Therefore, with a three-year dry weather flow average of 0.70 mgd, and subtracting the estimated dry weather infiltration, the base flow is estimated to be approximately 0.67 mgd.

During dry weather conditions, estimated I/I amounts to 23 gpad. The average annual I/I contribution is 115 gpad. The peak hour value of 1,923 gpad is relatively low when I/I values for systems tributary to the King County wastewater system are considered. A value of 1,100 gpad for peak hour I/I has been widely used by sewer systems in the Pacific Northwest. Extensive evaluation by King County in the 2000s revealed that many separate sewer systems in the Seattle area, all generally constructed beginning in the 1950s or later, generated several thousand or tens of thousands of gallons per acre per day of I/I.

The value of maximum month I/I is particularly critical, as the maximum month average daily flow is one of the primary WWTP permit criteria. Maximum month I/I is shown as 377 gpad in Table 5.10, per the historical maximum month flow of 1.186 mgd.

TABLE 5.10

ESTIMATED INFILTRATION AND INFLOW

Flow	Influent Flow at WWTP (mgd)	Adjusted Flow at WWTP (mgd)[(5)]	Base Flow (mgd)[(1)]	I/I (mgd)	Service Area (acre)[(2)]	Estimated I/I (gpad)
Dry Weather Average	0.719	0.700	0.67	0.03	1,300	23
Annual Average	0.843	0.820	0.67	0.15	1,300	115
Maximum Month[(3)]	1.186	1.16	0.67	0.49	1,300	377
Peak Day	2.495	2.454	0.67	1.78	1,300	1,372
Peak Hour[(4)]	3.25	3.170	0.67	2.50	1,300	1,923

1 - Base flow calculated from October 2012 data.

2 - Estimate of I/I contribution area - represented as the sum of the area of parcels served in 2014 (i.e. not including right of way or sewer easement).

3 - Reported for January 2011.

4 - Represents the highest flow measured at the treatment plant on January 5, 2015.

5 - Data excludes flow from Cherry Point Refinery Pump Station.

The capacity of the existing system was evaluated with a population density of 1.21 persons per connection, contributing 70 gpcd, with a peak hour I/I flow of 1,923 gpad. Sanitary flow is peaked at the value of 1.8 as presented in Chapter 4.

Future flows are forecast at the same values, for increased service area and population, with the exception of population density and I/I. Population density if forecast to increase to 1.32 persons per connection. For forecast purposes, it is assumed that I/I will increase overall with increasing age of the system. A future I/I allowance of 2,200 gpad has been selected. This is nearly 15% higher than the current estimated value.

As noted above, the District has installed flow meters at nine of the system's 11 pump stations. The meters allow collection of flow data for each collection system basin. Comparison of flows relative to basin area and basin IDM estimates supports analysis of which areas of the system may contribute more I/I than others. This allows the District to target their system analysis and repair efforts more efficiently, in the ongoing process of finding and reducing I/I.

Table 5.11 presents a comparison of area (sum of the area of parcels served) and flow by basin, for both a wet weather event and dry weather conditions. Several basins exhibit a share of total flow greater than the share of total area, for both wet and dry weather events: 4, 6, and 8. The Loomis Trail basin had minimally higher flow for the dry weather event alone.

The comparison based on area of parcels served alone does not consider the potential differences in collection system characteristics such as length of various diameters of pipe, and relative share of private sewer systems. Therefore the relative flow comparison is repeated in Table 5.12, but on the basis of inch-diameter-mile of pipe estimated for each basin, rather than area. Basins 4 and 6 exhibit a share of total flow greater than the share of total IDM, for both wet and dry weather events.

These comparisons suggest the District focus future I/I evaluation and reduction efforts in Basins 4, 6 and 8, in that order. This prioritization can be periodically reviewed for specific events and following identification and mitigation of significant sources of I/I in these or other basins.

TABLE 5.11

BASIN FLOW BY AREA CONTRIBUTION

Basin	Area - parcels served (acres)	Share of Total Area (%)	Wet Weather Flow (02/18/16) (gpd)	Share of Total Wet Flow (%)	Variance - wet weather flow vs area (%)	Dry Weather Flow (05/02/16) (gpd)	Share of Total Dry Flow (%)	Variance - dry weather flow vs area (%)
9	178	13.7	152,756	7.2	-6.5	72,744	10.4	-3.3
8	93	7.1	213,431	10.1	**3.0**	68,596	9.8	**2.7**
7	145	11.2	211,213	10.0	-1.2	65,160	9.3	-1.9
6	181	13.9	369,900	17.5	**3.6**	149,200	21.3	**7.4**
5	94	7.2	141,600	6.7	-0.5	47,900	6.8	-0.4
LT	61	4.7	84,755	4.0	-0.7	33,337	4.8	**0.1**
BR	114	8.8	100,245	4.7	-4.1	22,063	3.1	-5.7
4	160	12.3	458,300	21.7	**9.4**	103,000	14.7	**2.4**
3	191	14.7	295,607	14.0	-0.8	99,872	14.2	-0.5
1	84	6.4	88,777	4.2	-2.2	39,168	5.6	-0.8

1 - Flow per basin excludes flow from contributing basins.

2 - Cherry Point Refinery pump station flow not included.

TABLE 5.12

BASIN FLOW BY INCH-DIAMETER-MILE CONTRIBUTION

Basin	Inch-Dia.-Mile (IDM)	Share of Total IDM (%)	Wet Weather Flow (02/18/16) (gpd)	Share of Total Wet Flow (%)	Variance - wet weather flow vs IDM (%)	Dry Weather Flow (05/02/16) (gpd)	Share of Total Dry Flow (%)	Variance - dry weather flow vs IDM (%)
9	97.5	11.8	152,756	7.2	-4.6	72,744	10.4	-1.5
8	84.9	10.3	213,431	10.1	-0.2	68,596	9.8	-0.5
7	96.8	11.7	211,213	10.0	-1.8	65,160	9.3	-2.4
6	101.5	12.3	369,900	17.5	**5.2**	149,200	21.3	**9.0**
5	73.2	8.9	141,600	6.7	-2.2	47,900	6.8	-2.1
LT	46.2	5.6	84,755	4.0	-1.6	33,337	4.8	-0.8
BR	55.9	6.8	100,245	4.7	-2.0	22,063	3.1	-3.6
4	88.3	10.7	458,300	21.7	**10.9**	103,000	14.7	**4.0**
3	125.5	15.2	295,607	14.0	-1.3	99,872	14.2	-1.0
1	54.5	6.6	88,777	4.2	-2.4	39,168	5.6	-1.0

1 - Flow per basin excludes flow from contributing basins.

2 - Cherry Point Refinery pump station flow not included.

3 - IDM per basin based on collection system lengths per Table 5.2 plus estimated average of 30 feet of 6" stub per account, assuming 70% of accounts share a stub, and an estimated average 35 feet of 4" private side sewer per account. IDM also includes an estimate for properties with multiple living units or structures served by a private collection system.

5.6 SEWER CONNECTION PERMITS

District code requires property that has been assessed for sewers to connect to the sewer system where service is available. Owners of property within the District boundary are required to apply for a District letter of sewer availability prior to applying to the County Health Department for approval of an onsite sewer system to serve a proposed building to be used for human occupancy. If sewer service is available via sewer mains serving or abutting the subject property, a sewer availability letter is typically issued, per District terms and conditions. If sewer service is available pending completion of a sewer system extension (i.e. a "developer extension") or other system improvements, a sewer availability letter is typically issued, per District terms and conditions, including the requirement to enter into a developer extension agreement, construct the required system extension and/or other improvements and transfer the improvements to the

District for ownership and maintenance, prior to connection. If the District concludes that sewer service is not reasonably available, the District will indicate that sewer service is presently not available. To prepare for connection to the District sewer system a side sewer permit application must be made, including payment of the established fee and connection charges.

In the event that the County Health Department determines an existing onsite sewer system is failing, they may require connection to an existing sanitary sewer if such service is available. The property owner must meet all the same requirements for connection, just as if the property were not previously developed.

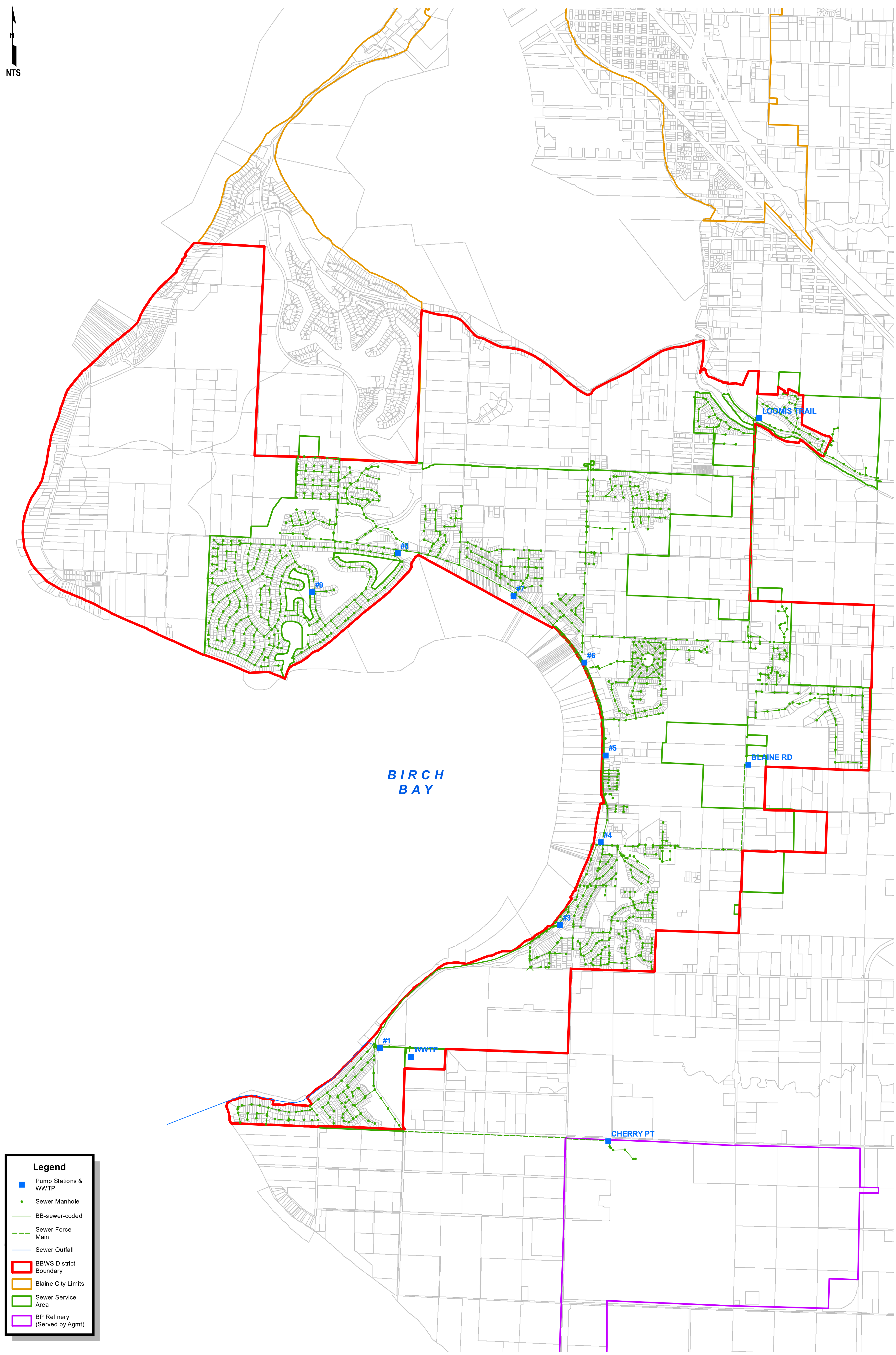
The District began the issuance of permits for connections to its sewer system in September 1976 and by December, 2018 a total of 7,301 connection permits had been issued. This is only an indication of the number of permits issued. Some side sewers have not been installed; others are modifications of side sewers installed under a previous permit. Permits are not issued for transient user connections such as RV hook-ups. The number is further reduced by sewers that are disconnected to make room for some other types of development. Table 5.13 presents a summary of permits issued.

TABLE 5.13

SUMMARY OF SEWER CONNECTION PERMITS

Year	Permits
1976-1999	5309
2000	103
2001	114
2002	156
2003	188
2004	234
2005	222
2006	322
2007	151
2008	64
2009	41
2010	53
2011	17
2012	23
2013	33
2014	37

Year	Permits
2015	49
2016	67
2017	58
2018	60
Total	7,301



Birch Bay Water & Sewer District

2019 Comprehensive Sewer System Plan

FIGURE 5.1
EXISTING SEWER FACILITIES

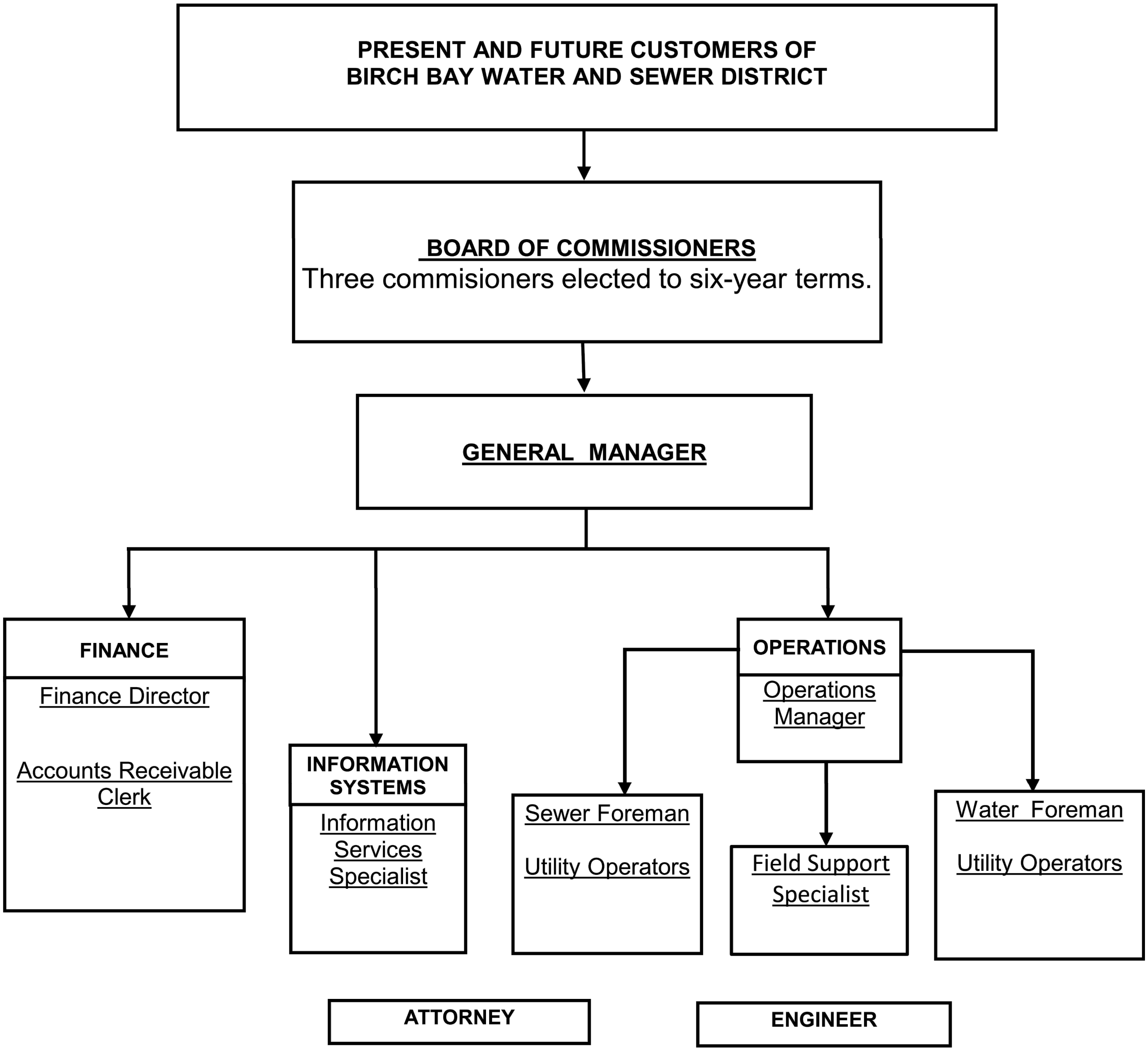


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FIGURE 5.2

ORGANIZATION CHART

USER: Scott Stcherbinine PLOTTING DATE: 12/31/2018 FILE LOCATION: K:\19-BBWSD\2015\191502 Comp Sewer Plan Update\191502 CAD Figures\191502 CSP - FIG 5-2.dwg

FIGURE 5.3
1990-2018 WWTP INFLUENT FLOW

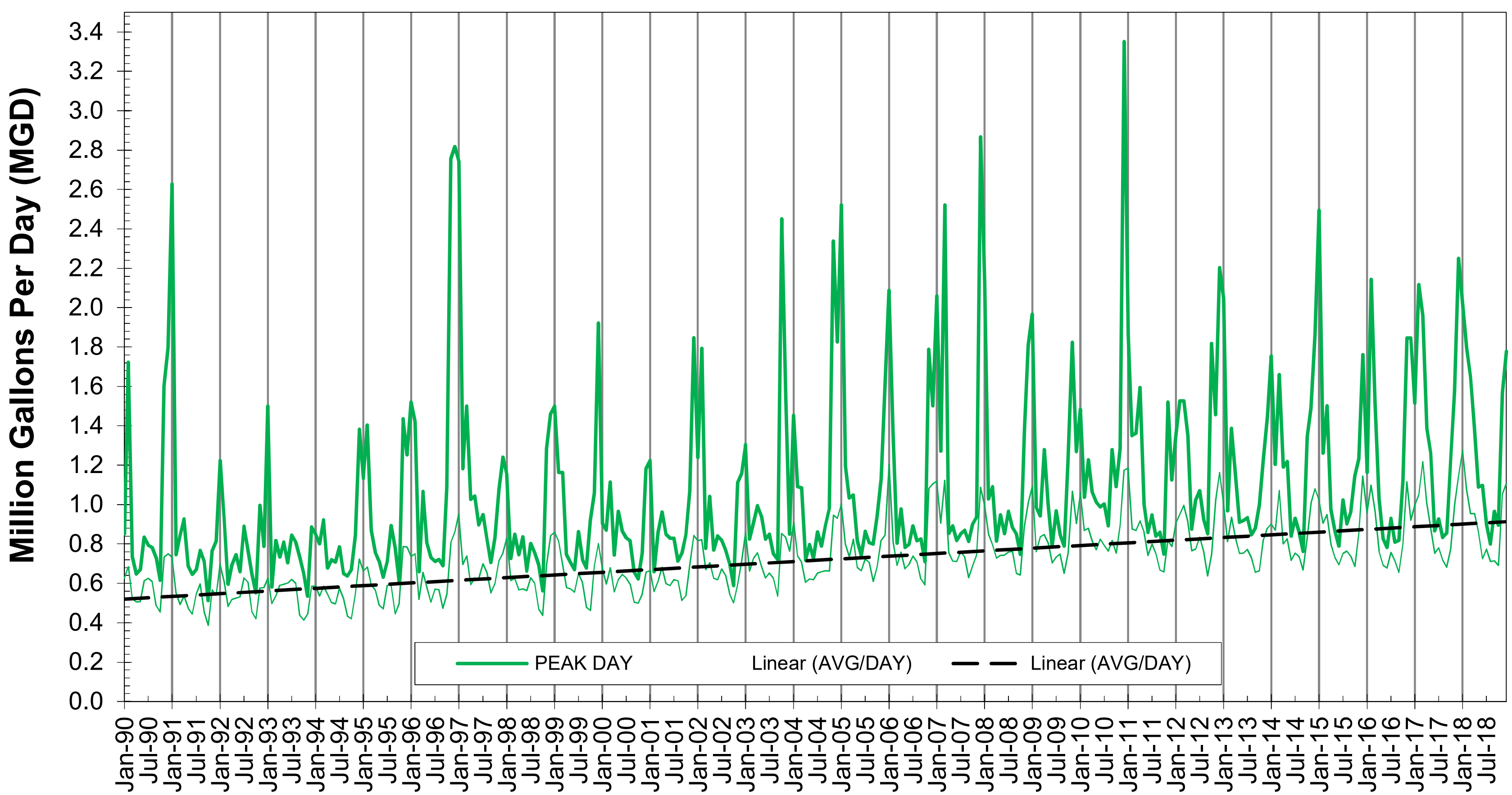
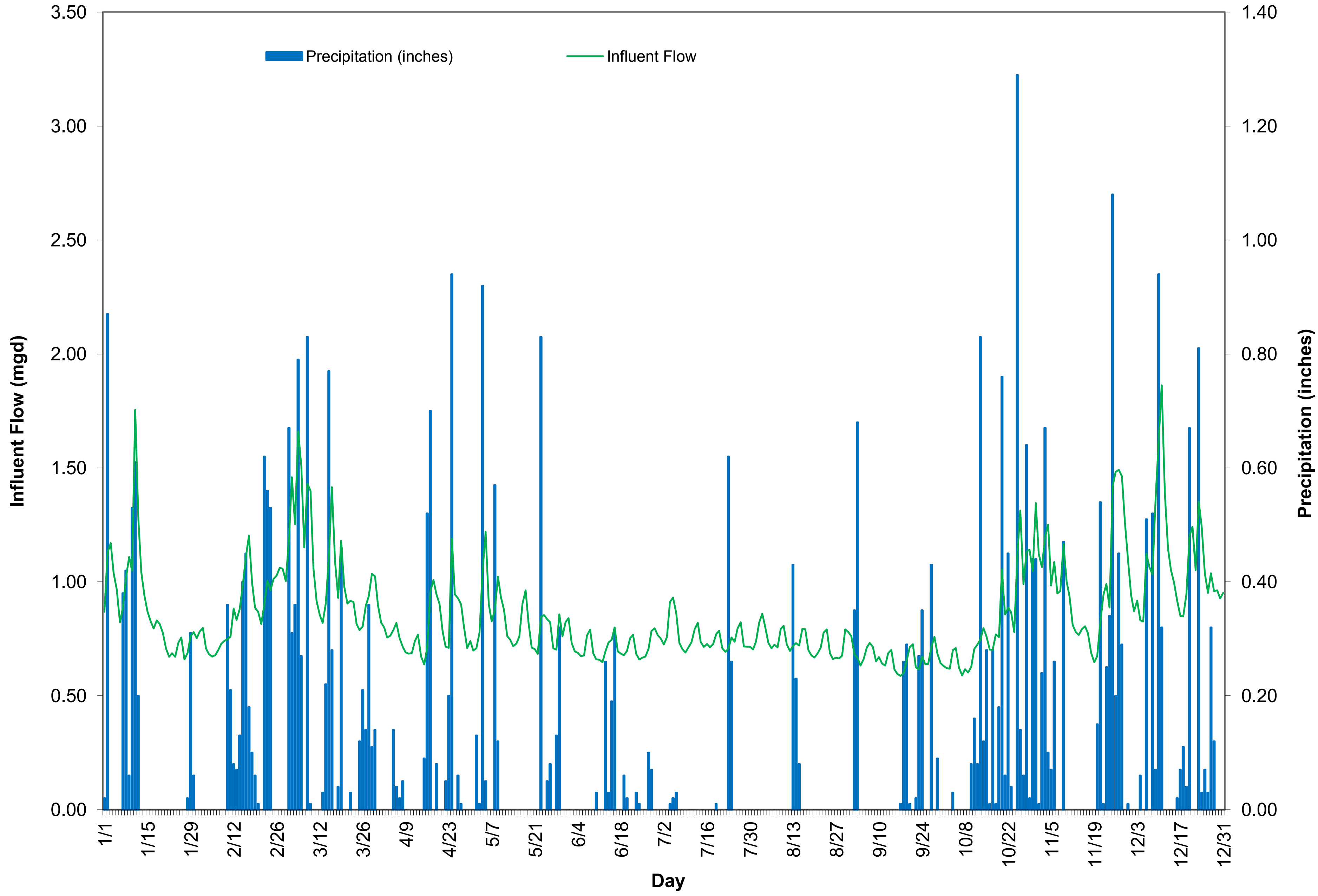


FIGURE 5.4
2014 WWTP INFLUENT FLOW VS. PRECIPITATION



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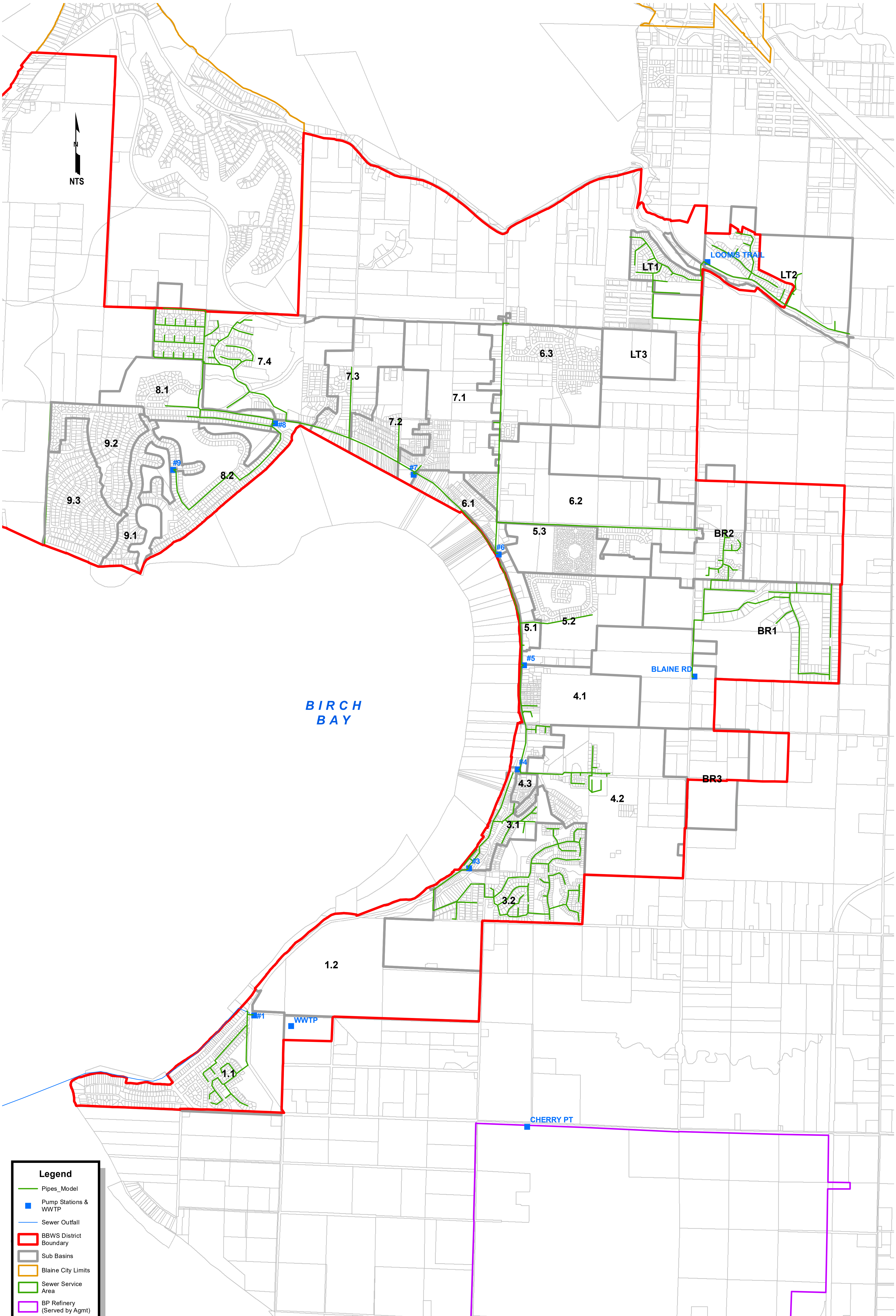

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2019 Comprehensive Sewer System Plan

FIGURE 5.5
MODELED EXISTING SEWER FACILITIES

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CHAPTER 6

WASTEWATER TREATMENT PLANT

6.1 INTRODUCTION

Background

Birch Bay Water and Sewer District owns and operates a secondary wastewater treatment plant (WWTP) with a capacity to treat a maximum month average daily flow of 1.44 million gallons per day (mgd) of domestic wastewater. The plant is located southeast of the intersection of Point Whitehorn Road and Birch Bay Drive. The plant was designed and constructed in the mid-1970s with start-up on December 7, 1976.

The WWTP operates per the requirements of National Pollutant Discharge Elimination System (NPDES) Waste Discharge Permit No. WA0029556 (NPDES Permit). The NPDES Permit was issued effective July 1, 2014 and most recently modified on November 30, 2015 by the State Department of Ecology (DOE). It expires June 30, 2019. Copies of the permit and modifications, and supporting fact sheets, are included in Appendix D. The District applied for a renewal of the permit in September 2018.

The NPDES Permit includes both influent loading limits and effluent discharge limits. The flow and loading limits were established based on the factors considered in the most recent WWTP Engineering Report (referenced below) and as recommended limits for the existing and then-proposed treatment facilities, to assure reliable treatment to meet the discharge limits. The discharge limits were set to establish the maximum allowable pollutant discharge to the Strait of Georgia. The current permit limits are summarized in Table 6.1.

Originally the plant was designed to provide for a future capacity of 1.0 mgd with pairs of primary clarifiers, aeration basins, secondary clarifiers and chlorine contact chambers. The plant was originally operated at about 0.5 mgd capacity with the eastern aeration basin serving as an aerobic digester. The primary sludge and scum collection systems operated by gravity flow alone. Waste activated sludge was pumped and digested sludge flowed by gravity to a sludge dewatering facility.

In 1977 the plant was modified by the addition of two facilities: a primary sludge pumping station using two plunger pumps; and a scum decant and sludge transfer station using two plunger pumps and one vortex pump. An 18-inch aeration basin bypass line was also constructed.

TABLE 6.1

NPDES Permit Limits Summary

Influent Loading Limits		
Maximum Month Design Flow	1.44 MGD	
BOD_5 Influent Loading for Maximum Month	2,500 lbs/day	
TSS Influent Loading for Maximum Month	2,400 lbs/day	
Effluent Limits		
	Average Monthly	**Average Weekly**
BOD_5	30 mg/L 360 lbs/day 85% Removal	45 mg/L 540 lbs/day
TSS	30 mg/L 360 lbs/day 85% Removal	45 mg/L 540 lbs/day
Total Residual Chlorine (If UV not in use)	0.35 mg/L	0.5 mg/L
	Minimum	**Maximum**
pH	6.0	9.0
	Monthly Geo. Mean	**Weekly Geo. Mean**
Fecal Coliform Bacteria	200/100 mL	400/100 mL

Significant upgrades occurred in 1986 when the east aeration basin was converted back to aeration operation and a separate aerobic digester was constructed on the east side of the plant. At that time the sludge dewatering equipment was removed and the building converted to house the blowers for the coarse bubble diffusion system for the aerated digester. The digester was also equipped with surface aerators to maintain dissolved oxygen levels during warm weather. A 20-foot diameter gravity sludge thickener was added and the scum and sludge transfer station was modified. The modifications consisted of removing the existing vortex pump and replacing it with a submersible centrifugal pump. The wet well was also modified to separate the sludge and scum flows. Scum was decanted back to the primary clarifiers by the submersible pump and the existing plunger pumps transferred sludge to the new digester. A waste mixed liquor 3-inch magnetic flow meter was also added. The operation of the primary clarifiers was modified by the construction of an additional sludge pump station using a vortex pump that was interconnected with the existing plunger pumps. The headworks was modified by removing the comminutors and installing a rotary fine screen and 10 inch magnetic flowmeter.

The onsite standby generator was upgraded in 1997.

In 1999, a third, 50-foot diameter, secondary clarifier was added along with a distribution box to distribute flow proportionally to the three secondary clarifiers. At that time, the District switched its primary means of disinfection from chlorine to ultraviolet disinfection. The return sludge pump station was also modified and converted to a variable-speed pump station. The return activated sludge lines were piped directly to the pump station and the telescoping valves were removed. The existing wide-band, non-clog submerged diffuser in the south aerobic digester was replaced with a shear-tube, medium bubble aeration diffuser. Modifications were also completed to the existing blower room to allow for better air supply to the aerobic digesters.

In 2003, an in-channel fine screen was added to the headworks to complement the existing rotary screen and the grit-removal equipment was removed in favor of manually-cleaned grit channels.

In 2013, the original headworks facility was replaced in its entirety. The replacement headworks is located within a CMU building and includes an influent box, two parallel, internally-fed horizontal rotating drum screens, influent flow meter, and grit chamber. The original headworks was removed.

In 2016, the activated sludge aeration and mixing system was converted from floating equipment to blowers and submerged fine bubble diffusers.

<u>Existing Wastewater Treatment Plant</u>

The WWTP is located on the west half of a 20-acre parcel owned by the District. The District also owns an additional 40 acres south of this parcel. The existing WWTP is located at an elevation of 25 to 35 feet above sea level and is not located within a designated 100-year floodplain.

The existing plant uses a complete mix activated sludge system to treat organic wastes and remove solids. The process flow diagram is shown in Figure 6.1. The treatment system consists of the following unit processes.

A. <u>Influent Pump Station (PS #3)</u>: This station consists of two submersible pumps in a below-grade wet well and an above grade building for controls and a standby generator. The station also serves as part of the collection system and discharges wastewater through 14- and 16-inch diameter force mains to the WWTP. An 18-inch force main is in place between PS #3 and the WWTP but is not in service nor is it connected at PS #3. The 14- and 16-

inch force mains include magnetic flow meters at the station for pump flow monitoring and as an alternate WWTP influent flow meter source

B. Headworks: This process unit consists of an influent box, two parallel, internally-fed horizontal rotating drum screens, influent flow meter, and vortex grit chamber. A Parshall Flume equipped with an ultrasonic level sensor is located between the screens and the grit chamber. The flume is used to monitor influent plant flow for recording purposes. The influent box receives flow from the 14-, 16-, and 18-inch force mains from PS #3 and the 8-inch force main from PS #1. The 8-inch force main includes a flow water immediately upstream of the influent box.

C. Primary Clarifiers: There are two 28-foot diameter circular primary clarifiers. Flow is divided to the clarifiers through a flow division (splitter) box. One clarifier can be taken out of service. The clarifiers are provided with a dual sludge pumping system consisting of a pair of above-grade plunger pumps and a single below-grade pump. The primary sludge is pumped to the sludge thickener or directly to the digesters.

D. Complete-Mix Activated Sludge Aeration Basins: The basins are concrete tanks and the secondary process is complete-mix activated sludge. The minimum acceptable detention time in the aeration basins (not including the return sludge pumping rate) is six hours to meet current effluent discharge requirements. Air is supplied from a set of three blowers with submerged fine-bubble membrane disc diffusers in each basin. A total of 504 diffusers are installed in each basin. Each 75 horsepower hybrid blower can be dedicated to a single basin or blower discharge can be combined to meet demand of certain conditions.

E. Secondary Clarifiers: A distribution box with three weirs proportions main plant flow to each of the three secondary clarifiers. There are two 33-foot diameter and one 50-foot diameter peripheral-feed circular clarifiers. The clarifiers are equipped with return activated sludge (RAS) lines that supply sludge to the main pump station. The pumps are three vertical, non-clog wastewater pumps piped and valved such that any one of them can pump either RAS back to the aeration basins or waste activated sludge (WAS) to the thickener or digester.

   The WAS meter vault, downstream from the pump station, can accept either mixed liquor from the aeration basins or WAS from the pump station. The meter vault is equipped with a bypass. The flow can be directed either to the south aerobic digester cell or the gravity sludge thickener.

F. Chlorine Contact Tank: The contact tank is divided into equal-volume east and west cells (or flow paths). The chlorine solution is applied at the effluent boxes of each secondary clarifier if the standby disinfection system is in operation.

G. Ultraviolet Disinfection Channels: Wastewater flows from the secondary clarifiers through the inlet channel of the chlorine contact tank into the UV disinfection channels. Disinfection occurs as flow travels past UV light sources, altering the microorganisms so they are unable to reproduce. The final effluent passes over a weir into a common collection box and continues on to the outlet channel of the chlorine contact tank before discharging to the effluent flow meter.

H. Effluent Flow Meter: The plant's effluent meter consists of a 9-inch Parshall flume with an ultrasonic level sensor.

I. Outfall: The effluent drops into a 24 inch gravity outfall. The outfall discharges the treated effluent into the Strait of Georgia. The outfall is 2,000 feet off Point Whitehorn and discharges at 45 feet below mean lower low water. The elevation of mean higher high water is 3.83 feet. The outfall diffuser was visually inspected in 2010 and found to be in good condition. The District recently executed a renewal of its lease with the State Department of Natural Resources for the portion of the outfall on state aquatic lands (see Appendix E).

J. Solids Processing:

   1. Gravity Sludge Thickener: A single, circular 20-foot diameter gravity sludge thickener receives WAS, primary and secondary scum, and/or primary sludge and mixed liquor to a solids concentration of 2.5 to 5.0 percent solids. A chlorine solution injection point is provided at the influent box. The thickened sludge is pumped to the aerobic digester. The thickener overflow is returned to the headworks.

   2. Aerobic Digester: The aerobic digester is divided into north and south cells of equal size. The north cell is equipped with coarse bubble aeration capability. The south cell is equipped with a shear tube, medium bubble aeration diffuser system. A floating surface aerator can be installed on either side to provide mixing and aeration in the event of diffuser or blower failure, or to provide other operational contributions. A blower building on the south side of the digester provides pressurized air from one or two of three blowers. Blowers can provide air to one or both of the aerobic digesters. The surface aeration system is used during the summer when warm weather and hot pressurized air from the blowers raise the temperature of the digester contents above optimum temperatures. Biosolids are processed in the digester and land-applied to local land under separate contracts for hauling and land application.

K. Ancillary Systems:

1. Chlorination Facilities: The chlorination facilities are housed in the chlorination building on the east side of the chlorine contact tank. The system currently has two tablet-fed chlorination systems, each capable of producing an equivalent of 50 lbs/day of chlorine. The systems are tied into common chlorine solution lines that feed into one, any, or all of four different locations: Effluent, just prior to Contact chambers, RAS lines, Gravity Thickener, and the Process Water System. Three automatic control valves are controlled manually or via PLC and SCADA, to adjust and maintain desired chlorine flow into each area.

2. Air-Gap Plant Water Pump Station: This station provides an air-gap tank and pressurization pumps between the potable water system connection and the chlorination system when process water is not available and potable water is necessary for the chlorination system. (Process Water is the primary source for the Chlorination system.)

3. Plant Process Water System: The plant process water is used for chlorination, washdown, surface sprays, and building heating & cooling water and is drawn from both the east and west cells of the chlorine contact tank. The system is pressurized by packaged pump station in the basement of the main pump station. The system contains three centrifugal pumps arranged in parallel on common intake and discharge manifolds. The pump motors are connected to VFDs. Speed and on/off operation is controlled automatically, based on system demand and pressure setpoints.

4. Sanitary Drainage System: The plant is provided with a sanitary drain system to collect domestic in-plant wastewater and various side-streams from some process units. The system drains to the main pump station sanitary system wet well. A pair of submersible centrifugal pumps discharges the collected wastewater to the headworks downstream from the rotary screen.

5. Standby Generator: The plant is equipped with a 275 kW diesel generator, 500-gallon above-grade diesel fuel storage tank, and related switchgear.

6. Operations Building: This facility provides the centralized control and monitoring systems for plant operations. Motor control centers and plant control boards are contained in one area. The building also houses the plant laboratory.

7. Maintenance Shop: A maintenance shop stocks spare parts for plant equipment and provides work space and tools for repairs.

8. Vehicle Maintenance and Storage: A six-bay metal building on the north side of the plant garages and maintains the District service vehicles. A five-bay garage for larger vehicles is provided in the southeast corner opposite the blower building.

9. District Headquarters Building: The District completed construction of the headquarters building off the north side of the Operations Building at the plant entrance in November 1992.

Existing WWTP Capacity Engineering Report

The District completed a Waste Load Assessment in January 2011, for data through 2010 (see copy in Appendix D). The Waste Load Assessment demonstrated that the plant continues to operate in compliance with the NDPES Permit and concluded that flows and loadings will not exceed 85% of WWTP capacity and permit limits for five years.

The most recent WWTP engineering report (WWTP ER, *Birch Bay Water and Sewer District Wastewater Treatment Plant Improvements,* CHS Engineers, LLC and H.R. Esvelt Engineering) was completed in April 2012. The growth forecast presented in Table 2.1 of the WWTP ER is lower than presented in this Plan, for the comparable portion of the planning period of this Plan. The timing of the “2022 Upgrades” per the WWTP ER has been adjusted in response to the updated forecast in Table 6.2 and the analysis summarized in Tables 6.3 and 6.4 below.

6.2 FLOW AND WASTE LOAD PROJECTIONS

Based upon the population projections presented in Chapter 3, the flow criteria developed in Chapters 4 and 5, and the loading criteria based on actual data for the past five years, influent flow and waste loads [biochemical oxygen demand, 5-day (BOD) and total suspended solids (TSS)] have been projected through 2038. The projections are shown in Table 6.2.

TABLE 6.2

POPULATION, ELUS, WWTP FLOWS AND LOADINGS

Year	2018	2023	2028	2033	*2036*	2038
Population(1)	8,982	10,028	11,200	12,516	*13,381*	13,643
Residential ELUs(2)	7,403	8,159	8,914	9,670	*10,123*	10,322
Non-residential ELUs(3)	670	791	912	1,033	*1,105*	1,154
Total ELUs	**8,073**	**8,949**	**9,826**	**10,702**	***11,228***	**11,475**
Sewer Service Area (ac.)(4)	1,461	1,622	1,782	1,943	*2,040*	2,104
WWTP Flows (mgd):						
Base Flow(5)	0.70	0.79	0.88	0.98	*1.04*	1.06
Dry Season Average Flow(6)	0.73	0.83	0.92	1.03	*1.09*	1.11
Average Annual Flow(6)	0.86	0.98	1.09	1.21	*1.27*	1.30
Maximum Month(6)	1.25	1.40	1.55	1.71	*1.81*	1.85
Peak Day(7)	3.32	3.79	4.27	4.79	*5.10*	5.28
Peak Hour(8)	4.14	4.72	5.31	5.93	*6.31*	6.54
WWTP Loadings (pounds per day):						
Annual Average BOD_5 loading (lbs/day)9	1,580	1,750	1,930	2,100	*2,200*	2,250
Max Month BOD_5 loading (lbs/day)10	2,080	2,310	2,530	2,760	*2,890*	2,960
Annual Average TSS loading (lbs/day)11	1,820	2,010	2,210	2,410	*2,530*	2,580
Maximum Month TSS loading (lbs/day)12	2,310	2,560	2,810	3,060	*3,220*	3,290

1 - Per Table 3.2

2 - 2036 ELU count based on 2013 residential ELU count plus 3,475 additional housing units (growth units) as forecast through 2036 by County. Population density in 2036 of 1.32 used to estimate additional ELU growth through 2038, based on corresponding population growth. Growth beyond 2036 is not associated with specific parcels or area and may be considered as contingency for higher density in some portions of service area.

3 - Non-residential ELU count for 2013, with estimated total increase by 2038 (+610), with linear growth over that period.

4 - Service area estimated for 2013 and 2038, with linear growth over that period.

5 - Base flow based on 70 gpcd times increasing population density times ELUs.

6 - Base flow plus corresponding I/I rate (Table 5.10).

7 - Base flow x Peaking Factor (1.8) + Peak Day I&I rate (Table 5.10). Peak day rate increases to 1,600 gpad linearly through 2038.

8 - Base flow x Peaking Factor (1.8) + Peak Hour I&I rate (Table 5.10). Peak hour rate increases to 2,200 linearly through 2038.

9 - 0.20 pounds per day per ELU based on actual loading for 2013-2017.

10 - 0.26 pounds per day per ELU based on actual loading for 2013-2017.

11 - 0.23 pounds per day per ELU based on actual loading for 2013-2017.

12 - 0.29 pounds per day per ELU based on actual loading for 2013-2017.

The NPDES Permit stipulates that the District must submit a plan and a schedule for continuing to maintain capacity to DOE when:

- The actual flow or waste load reaches 85 percent of any one of the influent loading criteria for three consecutive months
- The projected plant flow or loading would reach the influent loading criteria within five years.

Table 6.3 presents a summary of the current NPDES Permit influent loading limits and the anticipated timing of reaching the influent limits and planning thresholds, based on the forecast presented in Table 6.2.

TABLE 6.3

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WWTP INFLUENT LIMITS AND UPGRADE THRESHOLD - FORECAST GROWTH BASIS

Criteria	Permit Maximum	Forecast Exceedance Year[1]
Maximum Month Design Flow (mgd)	1.44	2024
BOD_5 Influent Loading for Maximum Month (lbs/day)	2,500	2027
TSS Influent Loading for Maximum Month (lbs/day)	2,400	2020

1 Forecast is based on the flowing and loading forecast presented in Table 6.2, which is based on the County growth forecast in housing units, and District flow and loading criteria as developed in Chapters 4 and 5

The maximum month average daily flow has exceeded the 85% threshold only once (January, 2018). The permitted flow is forecast to be reached by 2024. BOD_5 influent loading reached a maximum of 2,004 lbs/day in 2016 and is forecast to exceed the permit limit in 2027. TSS influent loading reached a maximum of 2,265 lbs/day in 2010, with lower values since. Since 2013, the maximum value was 2,030. TSS loading is anticipated to exceed the permit limit by 2020.

As discussed in Chapter 3, the actual growth in the sewer service area appears to be lagging the forecast as based on the County's growth factors by over three years. This suggests that WWTP improvements should be planned for completion, to assure capacity for TSS loading, by 2023, depending on actual near-term growth rates.

The actual maximum month average influent flow, BOD_5 loading and TSS loading for the period 2009-2018 was evaluated, to compare with the growth forecast referenced in Chapter 3 and used as the basis for the forecasts in Tables 6.2 and 6.3. The trendline for each of the three loading parameters was linearly extrapolated to 2038, to understand the potential timing of exceeding the respective permit limits, assuming historical trends continue. The results of the analysis are presented in Table 6.4 and Figure 6.2.

TABLE 6.4

WWTP INFLUENT LIMITS AND UPGRADE THRESHOLD - TRENDLINE BASIS

Criteria	Permit Maximum	Forecast Exceedance Year[1]
Maximum Month Design Flow (mgd)	1.44	2037
BOD_5 Influent Loading for Maximum Month (lbs/day)	2,500	2038
TSS Influent Loading for Maximum Month (lbs/day)	2,400	>2038

1 - Forecast is based on extrapolation of the linear trendline of actual data for the period 2009 - 2018.

The timing for upgrades based on ELU growth rate and design criteria indicates that improvements should be completed by 2020, or say 2023 with the previously discussed lag in actual growth relative to the County forecast. The timing for upgrades based on long-range trends in flows and loadings is much later, with flow anticipated to be controlling criterion. However, the maximum month influent flow trend since 2013 has been consistently increasing and, if extrapolated, suggests increased capacity, or rating for increased flow, will be necessary by 2021.

In accordance with RCW 90.48.495, consideration has been given to the effect of water conservation on the projected flows to the wastewater treatment plant. Since 1990, the District has been placing increased emphasis on water conservation due to increased demand and limited water supply in northwestern

Whatcom County. Water conservation efforts to date have been generally successful, although it is not possible to accurately quantify reduced water use due to conservation at this time. This is due to seasonally and annually variable population and summer weather patterns. The annual average water use per connection declined in the 1990s and the trend for peak day water use was also generally flat. Peak day usage is closely tied to warm weather and peak weekend population. The District's efforts have focused on reduction of peak day water demands (summer), which is the primary constraint on the water system, whereas the constraint on the sewer system is peak flows that are more influenced by inflow and infiltration (winter). With a gradual transition from seasonal/weekend population to year-round residential population, water use and solids loadings per connection has increased, particularly since about 2000. Water conservation will have an impact on annual average and peak summer daily flows to the WWTP. However, the design flow for the plant is maximum monthly average daily flow, which occurs in the winter months at Birch Bay (see Section 5.5). Therefore, water conservation is not expected to impact projected design flows to the WWTP.

6.3 UPGRADE AND EXPANSION OF WWTP PROCESSES

The 2012 WWTP ER recommended three sets of improvements, as follows:

- Replacement headworks (completed in 2013)
- Aeration system upgrades (completed in 2016)
- 2022 Upgrades – including removal of primary clarifiers and replacement with two anoxic basins and one additional aeration basin, and removal of one secondary clarifier to allow replacement with larger secondary clarifier, and additional UV disinfection modules.

Based on the recent flow trend evident in Figure 6.2, in context of relatively slow growth and the timing anticipated in Table 6.3, the "2022 Upgrades" should be scheduled for completion and in service by 2023.

The District should complete an updated WWTP ER prior to completing the next plant upgrade. The report update should focus on evaluation of the operation and performance of the facility and process units in recent years, to refine the recommended scope of improvements, and to confirm that the design criteria for loadings is appropriate for the existing process units, as optimized in recent years . The 2012 WWTP ER does not include an estimate of the project cost for the "2022 Upgrades". A very preliminary estimate has been prepared to support the capital improvement plan.

The next WWTP ER update will need to consider the potential or known impact of the Puget Sound Nutrient Removal Project.

The NPDES permit includes requirements for several report submittals during the permit period, including the following special scheduled assessments:

- inflow and infiltration evaluation by May 1, 2016 (complete – see Appendix D) and October 1, 2018 (complete – see Appendix D)
- operations and maintenance manual update or review confirmation letter by November 30 annually
- acute and chronic toxicity testing in December 2018 (complete) and July 2018 (Complete)
- application for permit renewal by October 1, 2018 (complete)

6.4 RECLAIMED WATER USE

The reclamation of secondary effluent, treated to meet the requirements for reclaimed water, is being considered and implemented in a number of areas in Washington as potable water resources struggle to keep pace with growth. The 1999 WWTP Engineering report included a discussion of potential uses for reclaimed water in the area.

Prior to the 2018 update of the State reclaimed water regulations, there were are four classes of reclaimed water: Class A, B, C and D. WAC 173-219 was updated in January 2018. It now defines three classes of reclaimed water: Class A+ is the highest class, followed by Class A and Class B. Class A+ treatment criteria will be established on a case-by-case basis, for direct potable reuse. Class A and Class B are designated for various potential uses including commercial industrial, institutional and irrigation uses. Release to wetlands, surface water or groundwater augmentation is allowed under certain circumstances.

The level of treatment provided by the existing plant meets the basic requirements for biological oxidation and disinfection of reclaimed water. Additional treatment process units would be necessary for coagulation and flocculation in order to meet the requirements for Class A reclaimed water. These units could include chemical addition and mixing equipment, flocculation basins, and filters with backwash and solids handling equipment or low-solids membrane filters. Enhancements of the disinfection system may be necessary to achieve the appropriate level of disinfection for each class of reclaimed water. Depending on the proposed use of the reclaimed water, a chlorine residual may be required.

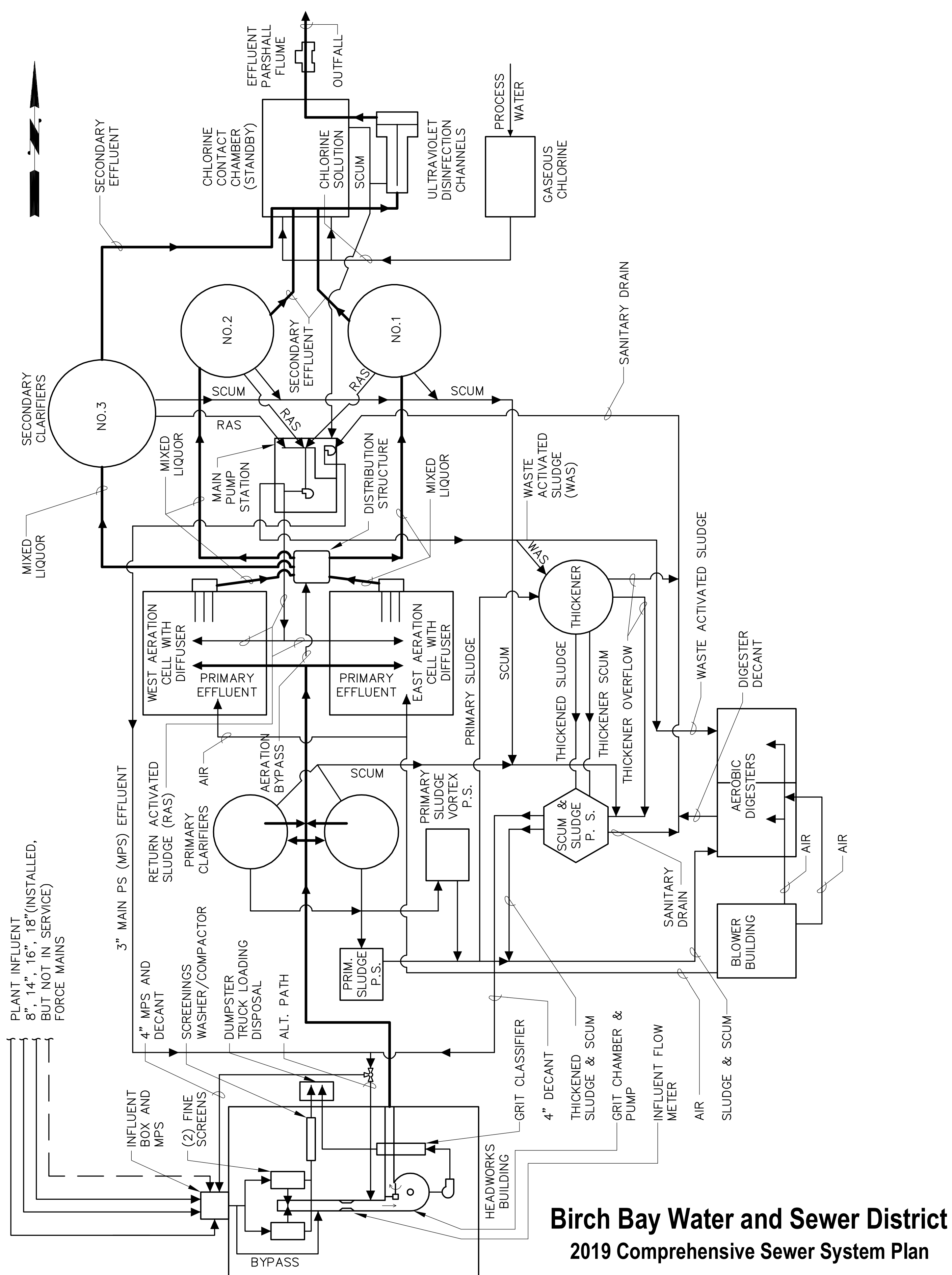
Potential existing and future candidate uses of reclaimed water, and the corresponding reclaimed water distribution system, are shown on Figure 6.3. These uses include golf course and landscape irrigation, construction (dust control and compaction), industrial process water, ship ballast, sewer flushing water and groundwater recharge. (Groundwater recharge areas are not shown, as potential areas of use have yet to be determined.) Additional lower volume uses may be identified once the primary infrastructure is in place (e.g. fire sprinkler systems and residential landscaping).

Use of the District's WWTP effluent as reclaimed water has been evaluated to varying levels of detail over the past 20 years. The two most likely candidates around Birch Bay for reclaimed water use are golf course irrigation and industrial process water. The District adopted a policy in 1991 to not supply water for customers for the purpose of irrigating new large landscape areas such as golf courses. The existing golf courses, and rumored (in the past) location for new golf courses, are in the north part of the District and it may be less expensive to serve these developments with reclaimed water from the City of Blaine. The City of Blaine supplies the Semiahmoo Golf Course with a portion of its irrigation supply need from the City's reclaimed water facility. The other likely use would be industrial process water in the Cherry Point industrial area. It may prove cost effective to develop and use a reclaimed water system from the Birch Bay WWTP rather than bring significant additional volumes of water from the Nooksack River, the current water source for the industrial area. Groundwater recharge may also prove to be feasible in the future as peak day and seasonal demands for potable water exceed the existing developed potable water sources.

Detailed planning efforts have not been completed because the initial estimates of the cost of treatment and especially the cost of the reclaimed water distribution system have exceeded the estimated benefit from the use of reclaimed water. The availability of water in Birch Bay (and to some extent in Blaine, the current source for the District's water) is not a concern for most of the year due to cooler temperatures and the seasonal population fluctuations. The demands for irrigation water coincide with the peak demands for potable water but the duration of the demand period is only a few weeks to a couple of months. The cost to develop a reclaimed water system for only a few months of use each year appears to be prohibitive. If a year-round industrial demand is identified, development of the reclaimed water system may prove to be cost-effective.

The District completed a preliminary review of potential uses and costs of reclaimed water as part of a preliminary source of water investigation (<u>Source of Supply Technical Memorandum 1</u>, Final Draft, March 2006, Kennedy/Jenks Consultants). This analysis confirmed the conclusions of earlier investigations, particularly highlighting the significant cost of conveyance infrastructure for relatively small volumes of water and short-term use each year. Additionally the potential uses of reclaimed water would offset very little current potable use in the District. The most promising uses were golf course irrigation with reclaimed water from the City of Blaine, and District reclaimed water in the industrial area. The District has initiated discussions of the latter with the Whatcom County PUD and BP, the nearest potential customers in the Cherry Point Industrial area, but the parties have not reached consensus about the eventual use or timing for reclaimed water use in the industrial area.

The aquatic lands lease (see Appendix E) also requires the District to continue to pursue opportunities for use of reclaimed water to maximize efficient use of water resources and minimize the discharge of treated effluent to the Strait of Georgia.



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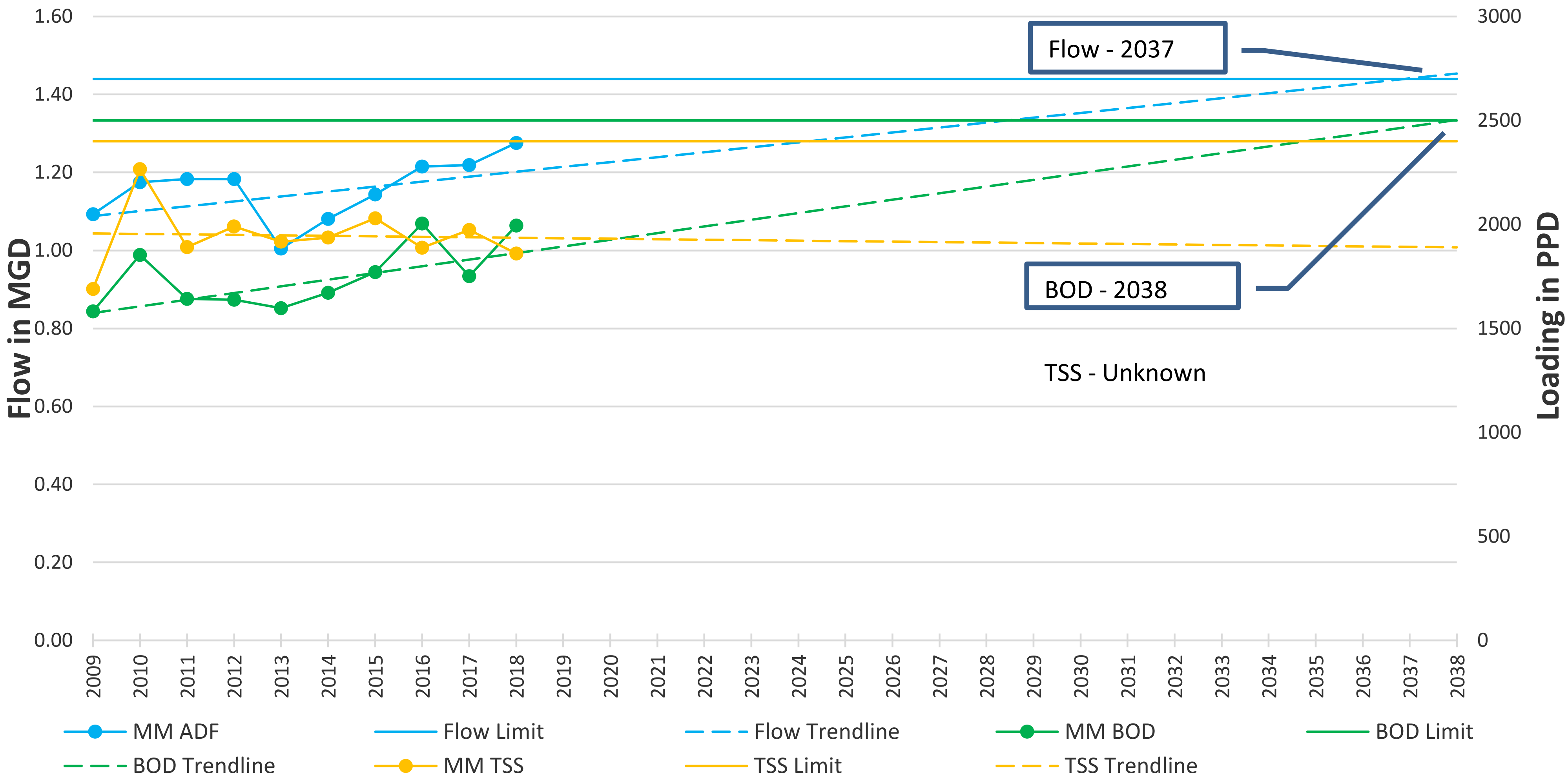
Birch Bay Water and Sewer District

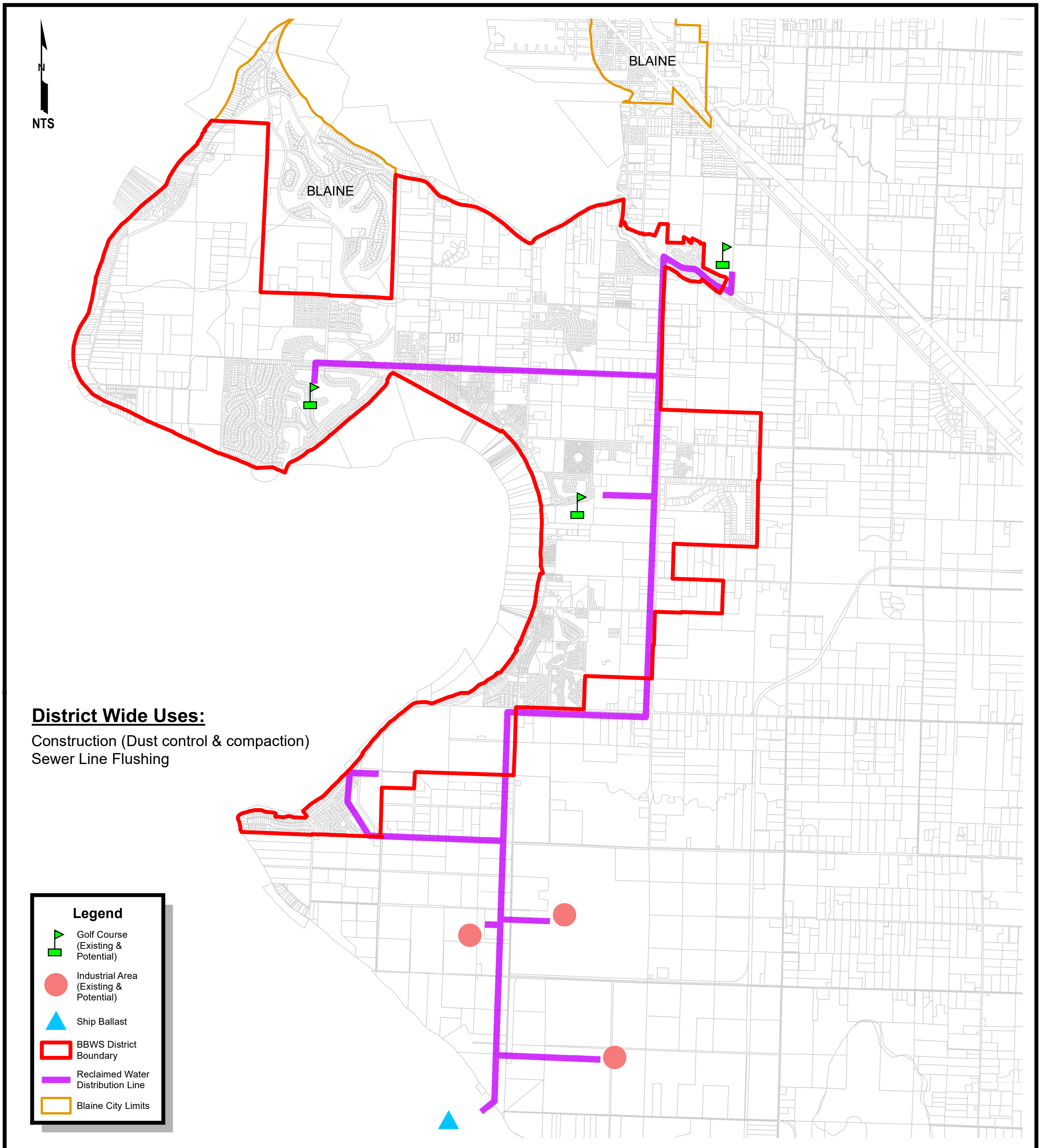
2019 Comprehensive Sewer System Plan

FIGURE 6.1

WWTP FLOW SCHEMATIC

FIGURE 6.2
WWTP FLOWS AND LOADINGS 2009 TO 2018





CHS ENGINEERS

Birch Bay Water & Sewer District

2019 Comprehensive Sewer System Plan

FIGURE 6.3
POTENTIAL RECLAIMED WATER USES

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CHAPTER 7

CAPITAL IMPROVEMENT PLAN

7.1. INTRODUCTION

The future facilities discussed in this plan have been proposed based upon the analysis presented in prior chapters and projects identified in the following prior system evaluations:

- 2012 *Engineering Report for Wastewater Treatment Plant Improvements*
- 2009 *Pump Station #4 Pre-design Evaluation*
- 2009 *Comprehensive Sewer System Plan*
- 2007 *Capacity Upgrade Planning Report*
- 2008 *General Wastewater Pump Station Pre-design Report*
- other informal studies supporting developer extension applications.

Each of these studies support evaluation and identification of the facilities necessary to extend sewer service and provide capacity for future development of the sewer service area. The capacity of future facilities is based on ultimate development of the basin in accordance with current comprehensive land use plans and the design criteria presented in previous chapters.

7.2. LATERAL SEWERS

It is expected that the majority of future lateral sewers (8-inch diameter) will be built under Birch Bay Water and Sewer District's developer extension agreement and related requirements. Lateral sewers not built under this agreement will be built by the District via the conventional utility local improvement district (ULID) method or under the District's process for local projects funded by local facilities charges. The general location of lateral sewers is indicated on Figure 7.1. Small areas may require service by means of individual residential pump stations or a small pump station due to topographic constraints, soil conditions or other factors.

7.3. INTERCEPTOR SEWERS

Interceptor sewers in the Birch Bay area are planned to carry ultimate flows from the drainage basins they serve. Sizing these lines for future flows lessens the possibility of removing and rebuilding expensive roads, as well as eliminating possible duplication of services. However, it is recommended that the sizing of proposed interceptors be reviewed prior to construction. Depending on the location or extent of development which requires the interceptor construction, it may be appropriate to construct an interceptor smaller than needed for ultimate anticipated flows. Low wastewater flows in a large interceptor can result in sedimentation and/or septic conditions leading to increased maintenance work, potential odor complaints and negative impacts on the treatment plant.

Proposed sewer interceptors are shown on Figure 7.1. Final selection of the proposed projects and recommended schedule is discussed in Section 7.10.

An extensive system of interceptors has been planned to reduce the number of permanent pump stations. However, there may be situations where a temporary pump station is warranted due to the proposed development, topography, and remoteness from existing system or other conditions. A "temporary" station would still be a station meeting the District's standard pump station requirements, including standby power equipment if appropriate, in the District’s discretion.

7.4 PUMP STATIONS AND FORCE MAINS

Pump stations and force mains have been sized for ultimate flows where located in a sub-basin tributary to the conveyance system along Birch Bay Drive. Pump stations and force mains along Birch Bay Drive have been sized for flows anticipated at full buildout of the presently defined sewer service area, at present land use and zoning density. Depending upon development in the Birch Bay area, these pumps stations and force mains can be designed and constructed under a staged program. Temporary force mains can be used in certain areas where major force mains and interceptor sewers are not needed until further development takes place in the surrounding areas. Proposed pump stations and force mains are shown on Figure 7.1. Final selection of the proposed projects and recommended schedule is discussed in Section 7.10.

Modifications to the existing pump stations will be necessary to accommodate future flows and associated force main additions/modifications. In some cases, replacement of the whole pump station may be economically appropriate due to the scope and magnitude of the necessary station modifications. Regular upgrade and rehabilitation of each station is recommended on an interval of approximately 20 years, or less for specific equipment as conditions warrant.

7.5 OUTFALL

The present effluent outfall off Point Whitehorn has a capacity of 5.9 MGD. The outfall diffuser was inspected in 2010, and was found to be in good condition.

7.6 TREATMENT AND DISPOSAL FACILITIES

Efficient wastewater treatment requires not only selection and design of a process compatible with the sewage type, but also responsible operation and maintenance of the facilities. For the domestic sewage of the Birch Bay area, the existing activated sludge process is highly compatible and proves to be relatively easy to operate and maintain.

Ongoing development within the planning area will eventually increase wastewater flows above 1.44 MGD. As actual flow or waste load reaches 85 percent of the plant design capacity, modifications necessary to accept and treat the additional wastewater should be initiated. Analysis presented in Chapter 6 indicates that the threshold for initiating planning has been reached for some permit parameters.

Recommended treatment plant modifications and improvements are discussed in Chapter 6 and in Table 7.2.

7.7 OTHER RECOMMENDATIONS

It is recommended that the District update the comprehensive sewer plan within six years, or sooner in response to County land use changes that impact sewer service area or development density therein.

The NPDES permit includes requirements for several report submittals during the permit period, including the following special scheduled assessments:

- Inflow and infiltration evaluation by May 1, 2016 (completed) and October 1, 2018 (completed)
- Operations and maintenance manual update or review confirmation letter by November 30 annually
- Acute and chronic toxicity testing by December 2018 (completed) and July 2019
- Application for permit renewal by October 1, 2018 (completed)

7.8 COST ESTIMATES

Cost estimates involve an engineering judgment based on experience, but construction costs can vary over a wide range because of the many factors which cannot be predicted such as labor availability, competitive conditions, management, environmental considerations and other intangibles affecting construction costs at the time the work is actually performed. Generally, actual costs cannot be known until bids are received, and even these may be subject to adjustment because of changed conditions. The District, in its decision making, must always keep in mind that the costs presented in this chapter are estimates.

Construction costs are estimated from prices obtained from various sources, including manufacturers and suppliers of materials and equipment and bid prices for projects in other communities in the area. In considering these estimates, it is important to realize that changes during final design quite possibly will alter the total cost to some degree, and future changes in the cost of material, labor and equipment will also have a direct impact. Prior to the initiation of the projects shown in this capital improvement program, the project costs should be reviewed and updated to reflect current conditions.

The cost estimates presented are based on 2019 prices and represent estimated total project costs. For reference the Engineering News Record Construction Cost Index for Seattle is 12,026 (February 2019). Project costs include construction cost plus a contingency of 20 percent as well as allied costs. Allied costs include consultant services, interest, taxes and District administration costs, etc. These allied costs have been estimated at 60 percent of the construction cost based on the following breakdown:

State sales tax	8.5%
Permitting, environmental, engineering design, surveying, inspection, administrative, etc.	25.0%
Legal	1.0%
Administration, interest during construction, financial fees, etc.	5.5%
Contingency	20.0%
TOTAL	60.0%

Operation and maintenance costs are not reflected in the project cost estimates. However, these costs are important and require thorough consideration during the design phase of a proposed facility or project.

7.9 FINANCING

The revenue to operate and maintain the District is collected through a monthly service charge, billed on a bi-monthly basis. Each account is charged a flat rate of $20.95 per month per equivalent living unit (ELU). All sewer accounts are charged $2.85 per hundred cubic feet (ccf) of water used, only for usage between 5 and 20 ccf used per two months. A general facilities charge is collected for each new ELU connected to system. This charge is used for capital projects of general or District-wide nature such as treatment plant and pump station construction or improvements. The current sewer general facilities charge is $4,260 per ELU, established in February, 2018 Costs associated with developer extensions are discussed in the Developer Project Manual. See Figure 7.2 for a general schematic of special purpose district finances.

There are several ways that the improvements outlined in this report can be financed aside from setting aside a portion of the monthly service charge. Rates and charges must be maintained at an adequate level to ensure a sufficiency of funds to properly maintain and operate the system and provide funds for construction of the projects identified in the *Comprehensive Sewer System Plan* through a combination of cash contributions and debt financing.

A. Developer Financing

Many of the new facilities constructed in the District will be financed by developers of presently unimproved property. All of the improvements required for service to property within new plats or commercial and industrial developments will be designed and constructed in accordance with the District's *Developer Project Manual*. In some cases, latecomer's agreements may be executed for any sewer main serving property other than the property owned by the developer that is financing the project.

B. Combination Financing by the District and Developers

It may be necessary in some cases to require the owner to construct a larger diameter line than is required by the current development in order to support the comprehensive development of the District. The District may enter into a latecomer's agreement or reimburse the developer for the extra cost of increasing the size of the line over that required to serve the property under development. Oversizing should be considered when it is necessary to construct any pipe over 10 inches in diameter in single-family residential areas to comply with the comprehensive sewer system plan. Construction of any pipe in residential, multiple family, commercial or industrial areas that is larger than the size required to serve that development is considered oversizing.

C. Revenue Bond

Interceptors, pump stations and improvements to the wastewater treatment plant that are a general benefit to a major portion of the District may be financed by revenue bonds. Improvements that will benefit primarily a single developer should be financed by the developer developing the property. The District may use whatever funds are available for the payment of the debt service on the revenue bonds. A major source of these funds is from the sewer rate revenues from the District customers. However, all funds, such as general facility fees, connection charges or latecomer charges, may be used for debt services.

Sewer system improvements that will service many different property owners in areas that are already developed may be financed through the establishment of a utility local improvement district (ULID). The financing is accomplished through the sale of revenue bonds. These bonds are retired with income from the assessments and/or other funds of the District.

D. Grant Funds/Loans

State and federal authorities have previously provided funds under the various grant programs for the construction of major improvements to or rehabilitation of sewer systems. The only known programs available at this time are the Centennial Clean Water Grant Fund, State Revolving Loan Fund, Farmers Home Administration (RDA) and Public Works Trust Fund Loan Program. The District should continuously monitor the activities of the state or federal agencies to determine the requirements of these programs or of any new grant programs that may be developed in the future.

E. Local Facilities Charge

This method of project financing is for projects initiated by the District where conditions warrant its use rather than developer extension, ULID or other methods. The Board passed a resolution authorizing the collection of local facilities charges from the properties to be served, without creating a local improvement district or utility local improvement district. For each project to be financed by this method, a project benefit area is defined for the local facilities elements of the project and the cost of the project is allocated to benefited properties on the basis of pro-rata share of ELU's. The factors to be considered in the decision whether or not to proceed with the construction project are identified in the resolution. Two public hearings are part of this process. Following the initial public hearing, the Board may decide to proceed with the project, not to proceed with the project or to postpone its decision. Under this approach, developed properties in the project area are required to connect to the system extension and pay general and local facilities charges at the time of connection, or by installment. There is also a process for deferment of connection and payment, under certain conditions. The second public hearing is held after construction of improvements for the purpose of considering public input on proposed local facilities charges.

The District develops annual capital and operation/maintenance budgets following review of prior year's expenses and growth and anticipated new customers and projects. These budgets are developed separately for the sewer and water systems, with general District administration expenses split between each utility. The utility charges (revenue projections) are also reviewed annually to determine if changes in the rates are necessary.

Table 7.1 indicates recent and projected debt service and operation/maintenance expenses. The projections in this table are very general but are based on the capital improvement plan presented below. The debt service projection is based on the current debt obligations only and additional debt service anticipated for future projects. The District adopted a Debt Policy in 2016 which earned an award of excellence by the State of Washington Municipal Treasurers Association. The Policy ensures that all debt is issued both prudently and cost effectively. As capital projects are scheduled, the District evaluates available options for financing and then determines if additional debt is necessary. The District current loans were obtained for a combination of water and sewer projects. Sewer ELUs are forecast to increase as discussed in Chapter 3.

7.10 CAPITAL IMPROVEMENT PLAN

Sewer system improvements have been identified for provision of sewer service to the presently-unserved portion of the District's sewer service area. Sewer extensions and pump stations were planned in accordance with the previous *Comprehensive Sewer Plan* and other referenced District planning efforts, and economical/topographical/political constraints. Iterative analysis of the existing and proposed sewer systems with the criteria presented in previous chapters resulted in development of the program of improvements set forth in this plan. Figure 7.1 identifies the projects necessary for service throughout the service area. Not all lateral sewers are shown on Figure 7.1. Each project (generally mains 10 inch and larger only) is briefly described in Table 7.2, including an estimate of the project cost.

Projects that are necessary for additional flow capacity as a function of forecast growth are scheduled three years beyond the year for which the existing capacity is forecast to be deficient. This is because actual growth of District connections is at least three years behind the growth based on the County forecast for the UGA, as discussed in Chapter 3. However, the schedule for the WWTP upgrade has not been deferred, because the flow and loading trend analysis presented in Chapter 6 indicates that flows are increasing at a rate greater than actual and forecast growth.

Projects which are recommended for completion in the next ten years are presented in Table 7.3. The table also lists the year of completion and the probable source of funds for each project.

TABLE 7.1

SEWER SYSTEM OPERATION/MAINTENANCE AND DEBT SERVICE EXPENSES

Description	Actual ($)		Forecast ($)				
	2017	2018	2019	2023	2028	2033	2038
Operating Revenues and Expenditures							
Total Revenues	2,515,101	2,511,826	2,289,085	3,071,730	4,586,671	6,848,763	10,226,491
Total Expenditures	1,472,752	1,640,733	1,733,008	1,950,516	2,261,182	2,871,330	3,288,840
Net Income (Loss)	1,042,349	871,093	556,077	1,121,214	2,325,489	3,977,433	6,937,651
Other Increases							
Connection Charges	179,329	198,371	186,612	630,579	835,176	954,180	1,090,140
Other Financing Sources	6,650	19,229	8,778	7,000,000	-	-	-
Total Other Increases	185,979	217,600	195,390	7,630,579	835,176	954,180	1,090,140
Other Decreases							
Debt Service	300,566	299,150	297,754	366,976	617,898	617,898	617,898
Capital Outlay	472,883	128,764	866,000	3,887,920	3,009,900	3,461,385	3,980,593
Total Other Decreases	773,449	427,914	1,163,754	4,254,896	3,627,798	4,079,283	4,598,491
All Revenues and Expenditures							
Net Income (Loss)	454,879	660,779	(412,287)	4,496,897	(467,133)	852,330	3,429,300
Estimated Annual and Monthly Debt Service and Operating Cost per ELU ($) [1]							
ELUs	7,403	7,455	8,073	8,949	9,826	10,702	11,475
Cost/ELU (annual)	240	260	252	259	293	326	340
Cost/ELU (monthly)	20	22	21	22	24	27	28

1 - This summary only includes debt service and operating expenditures. The District establishes monthly sewer service charges based on a more detailed analysis, including consideration of the impact of growth in connections and corresponding connection charge and operating revenue, capital projects including those not funded by loans and administrative costs.

The results of the hydraulic analysis for existing and future conditions as discussed in Chapter 5 were reviewed in detail in support of developing an orderly sequence of projects to provide hydraulic capacity in the collection and pumping system for forecast peak hour flows. I/I is the most significant factor in peak hour flow rates. I/I impacts in the District's system are characterized by short-term significant flow increases as a result of inflow during and immediately following high-precipitation events, or during moderate to rapid snow-melt conditions. In order to maintain appropriate biological conditions in the WWTP extended aeration process, the District can increase operational volumes in the process units in anticipation of or in response to unusual flow rates, and/or utilize the "equalizing storage" in the collection and pumping system. By letting wet wells and the interceptor along Birch Bay Drive surcharge to a limited degree, the operators can reduce the peak flow to the treatment plant, without risk of overflow or substandard treatment.

The criteria used for they hydraulic analysis are purposely conservative. The existing conditions are modeled results and the conditions presently indicated by the model have not been observed in the system except in extreme, low-frequency flow conditions.

The District realizes the ideal condition for the collection and pumping system is full capacity for pumps and pipes to not allow surcharging. However, given the significant cost of interceptor, pump station, WWTP and outfall improvements for low-frequency peak hour flow events, the capital improvement plan (CIP) has been developed to allow a reasonable level of surcharging before improvements are scheduled. Once completed, the improvements are planned to provide full capacity for forecast flows for the service area.

Given the nature of the collection and pumping system (gravity segments downstream of pump stations), the most significant surcharging occurs at the upstream end of each gravity segment. Typically, sewer capacity would be increased from downstream to upstream, but in this system either the whole gravity segment needs to be replaced, or supported by a parallel line, or the work should provide additional capacity from upstream to downstream, to manage the hydraulic grade line (HGL) at appropriate elevations. Following that work, the upstream pump station can be upgraded, and the process repeated over time further upstream along the interceptor system.

To develop the CIP, a threshold elevation was identified at which the HGL is forecast to surcharge to five feet below the ground surface at each gravity segment's upstream manhole. The timing for surcharging to that elevation was interpolated between results for the existing and future conditions' hydraulic analysis as discussed in Chapter 5. Then the timing was advanced by three years, as explained above. This approach was used for scheduling improvements to gravity piping and pump stations/force mains along Birch Bay Drive, from PS #3 to PS #7. This approach indicates that all work in this portion of the system should be completed by year 2027. The work between PS #3 and #4 should be addressed in the next two years and the next two segments should be completed by 2025.

The approach of increasing capacity from upstream heading downstream, between pump stations, can be phased such that a sufficient length of parallel replacement piping could be installed, to reduce the future condition HGL to or below the threshold indicated above, without initially completing the full length of ultimately recommended upgrades for each segment between pump stations. This approach would allow the District more time to focus its financial resources throughout the PS #3 to #7 portion of the system, initially addressing only the upstream portion of each segment and the pump stations. Once the initial round of improvements were complete, the downstream gravity pipe phase for each segment can be scheduled for completion later in the planning period.

The potential exception to this phased approach is the portion from PS #3 to PS#4. The 2009 *Pump Station #4 Pre-design Evaluation* identified a potentially feasible opportunity to route some flows from the Alderson Road interceptor past PS #4 through a new gravity bypass and interceptor directly to PS #3. This would reduce the future capacity need for PS #4 and reduce long-range operating costs. A very limited comparison suggests the phased interceptor approach is likely the more cost effective approach. The initial costs to implement phased capacity upgrades including full capacity upgrade for PS #4 are estimated to approach $1.5 million less than the cost of the full length of gravity parallel pipe and PS #4 flow bypass, and reduced capacity. Considering the present value of future costs to complete the phasing work indicates there is a potential for overall savings of about $0.5 million or about 15%. However, this analysis is sensitive to the costs of upgrading PS #4 for each scenario. The 2009 evaluation was based on a higher forecast of future flows and a larger parallel pipe at minimal slope. At that slope the bypass project was anticipated to be marginally feasible. Larger gravity pipe would likely be necessary to match the potentially feasible conditions. An updated pre-design evaluation is recommended to confirm the preliminary cost comparison and feasibility. For planning purposes, the CIP presents the phased interceptor approach for the upgrades from PS #3 to #4.

Improvements upstream of PS #7 are not anticipated to be necessary till the end of the planning period, or beyond. Other capacity improvements, particularly along Alderson Road, are anticipated in response to the pace of development in areas served by that interceptor.

TABLE 7.2

CAPITAL PROJECTS

ID	Capital Improvement	Project Description	Estimated Project Cost (2019 $)	Recommended Year of Completion
		I. WWTP		
T-1	Toxicity Testing	Acute and Chronic Toxicity Testing	$ 36,000	2024, 2029, 2034
T-2	Evaluation	Outfall Evaluation and Effluent Mixing Study	$ 50,000	2024 & 2034
T-3	NPDES	NPDES Permit Renewal	$ 3,000	2024, 2029, 2034
T-4	WWTP Eng. Report	Engineering Report Update, including evaluation in support of Projects T-7, T-8 and T-9	$ 120,000	2020 & 2030
T-5	UV Upgrade	Replace UV Modules with high output modules (4 total) and replace appurtenances	$ 373,000	2019
T-6	Headworks Odor Control Upgrades	Implement upgrades, per findings of Project T-4, for odor and moisture containment and additional blower with scrubber (see note 1)	$ 170,000	2020
T-7	Biosolids Management Upgrade	Implement first phase of upgrades, per findings of Project T-4, for improved solids thickening, digester cover and replacement south basin diffusers (see note 1)	$ 200,000	2019 & 2020
T-8	WWTP Site Work	Misc. site improvements to address paving and drainage deficiencies (Consider coordinating timing to follow access road water main work)	$ 84,000	2021
T-9	WWTP Upgrades	Aeration/Clarification Upgrades based on 2012 WWTP Engineering Report, as refined by Project T-4 (see note 1)	$ 6,800,000	2022-2024
		Subtotal for WWTP Projects	**$ 7,836,000**	

TABLE 7.2

CAPITAL PROJECTS

ID	Capital Improvement	Project Description	Estimated Project Cost (2019 $)	Recommended Year of Completion
		II. PUMPING		
P-1	PS #8 Structure	Replace top slab of generator vault.	$ 268,000	2019
P-2	PS #3 Pump Upgrade Phase 1	Evaluate/replace ex. pumps to restore to nameplate capacity/increase capacity to 3,800 gpm, for full use of 16" force main, including replacement generator (approx. 10 years of growth capacity, anticipated to be accommodated in ex. structures and piping)	$ 520,000	2020-2021
P-3	PS #4 and FM Upgrade Phase 1	Upgrade PS #4 to 3,400 gpm (approximately 10 years of growth capacity), including 16" replacement force main crossing of Terrel Creek and replacement generator (Configuration to be confirmed with Project O-1 but additional wet well and generator vault not anticipated)	$ 556,000	2021-2022
P-4	PS BR Upgrade	Upgrade PS BR to 1,000 gpm capacity (upgrades will require larger generator and motor controls centers, but upgrade is anticipated to be accommodated in ex. structures and piping)	$ 312,000	2022-2023
P-5	PS #5 and FM Upgrade	Upgrade PS #5 to 2,600 gpm, including replacement 12" force main split to two interceptors. Include manual transfer switch and receptacle for portable standby power connection.	$ 337,000	2024
P-6	PS #6 and FM Upgrade	Upgrade PS #6 to 2,200 gpm, including replacement generator and replacement 12" force main split to two interceptors.	$ 480,000	2026
P-7	PS #7 and FM Upgrade	Upgrade PS #7 to 1,600 gpm, including replacement 10" force main and split to two interceptors.	$ 303,000	2028
P-8	PS A & 4" FM (& 8" GS)	Construct a 150 gpm submersible pump station with standby generator about 700 ft. south of Lincoln Road and 2,000 ft. west of Harbor View Road, with 2,300 ft. of 4" force main and 300 ft. of 8" gravity sewer to Anderson Road.	$ 1,483,000	>2028
P-9	PS #3 Upgrade Ph 2	Replace pumps to increase capacity to 4,650 gpm, for full capacity use of the 18" force main	$ 360,000	2029
P-10	PS #4 Upgrade Ph 2	Replace pumps to increase capacity to 4,150 gpm	$ 320,000	2031
		Subtotal for Pumping Projects	**$ 4,939,000**	

TABLE 7.2

CAPITAL PROJECTS

ID	Capital Improvement	Project Description	Estimated Project Cost (2019 $)	Recommended Year of Completion
		III. COLLECTION		
C-1	Collection System Evaluation & Repair - I/I	Continue evaluation of sanitary sewer system to identify sources of extraneous wastewater flow and continue program for remediation	$ 665,000	2020-2038
C-2a	24" - PS #3 to #4 - Ph I	Install 2,201 ft. of 24" parallel gravity sewer interceptor from MH743-040 to 743-034 (consider 36" diameter to support feasibility of gravity bypass of PS #4 from Alderson Rd. interceptor, to be confirmed by Project O-1)	$ 1,363,000	2020-2021
C-3a	18" - PS #4 to #5 - Ph I	Install 1,242 ft. of 18" parallel gravity sewer interceptor from MH 743-117A to 743-095. Modify piping at PS #5 to allow gravity overflow from wet well to new interceptor	$ 703,000	2023
C-4a	15" - PS #5 to #6 - Ph I	Install 1,931 ft. of 15" parallel gravity sewer interceptor from MH 743-126 to 743-121	$ 966,000	2025
C-5a	15" - PS #6 to #7 - Ph I	Install 1,551 ft. of 15" parallel gravity sewer interceptor from MH 742-050A to 742-037. Modify piping at PS #7 to allow gravity overflow from wet well to new interceptor	$ 776,000	2027
C-6	12" - PS #8 to MH 742-105	Install 1,742 ft. of 12" parallel gravity sewer between PS #8 and MH 742-105. Alternatively extend PS #8 force main to MH 742-105 and upgrade pumps to maintain adequate capacity	$ 819,000	2028
C-7	12" - Alderson Road (Parallel)	Install 2,401 ft. of 12" parallel gravity sewer between MH 743-080 to MH 743-042 (or as adjusted to integrate with the findings of Project O-1). May omit 492 ft. along existing steep segments, if diameter restriction is acceptable.	$ 1,128,000	2028
C-8	8" Jackson Rd. (Parallel)	Install 587 ft. of 8" parallel gravity sewer from MH 024-002 to MH 743-016	$ 258,000	2038

TABLE 7.2

CAPITAL PROJECTS

ID	Capital Improvement	Project Description	Estimated Project Cost (2019 $)	Recommended Year of Completion
C-9	15" Alderson Rd.	Construct 850 ft. of 15" gravity sewer from MH 743-080 east along Alderson Road to replace the temporary gravity tightline receiving discharge from the Blaine Road PS force main and provide local service.	$ 425,000	2020
C-10	12" Alderson Rd.	Construct 850 ft. of 12" gravity sewer from Project C-9 continuing east along Alderson Road, to replace the temporary gravity tightline receiving discharge from the Blaine Road PS force main and to provide local service.	$ 400,000	2024
C-2b	24" - PS #3 to #4 - Ph II	Install 668 ft. of 24" parallel gravity sewer interceptor from MH 743-034 to 743-028 (consider 36" diameter to support feasibility of gravity bypass of PS #4 from Alderson Rd. interceptor, to be confirmed by Project O-1)	$ 407,000	2029
C-3b	18" - PS #4 to #5 - Ph II	Install 1,239 ft. of 18" parallel gravity sewer interceptor from MH 743-095 to 743-085.	$ 681,000	2030
C-4b	15" - PS #5 to #6 - Ph II	Install 698 ft. of 15" parallel gravity sewer interceptor from MH 743-121 to 743-119.	$ 349,000	2031
C-5b	15" - PS #6 to #7 - Ph II	Install 1,507 ft. of 15" parallel gravity sewer interceptor from 742-037 to 742-001	$ 754,000	2032
		Subtotal for Collection Projects	**$ 9,694,000**	
		IV. OTHER		
O-1	PS #4 Pre-design Update	PS #4 and Alderson Rd. Gravity Sewer Bypass Pre-design Evaluation Update	$ 25,000	2020
O-2	SCADA	SCADA System Upgrades	$ 45,000	2019
O-3	Phones	Phone System Upgrades	$ 15,000	2019
O-4	Vehicles	Replacement Vehicles	$ 298,000	2019, 2022, 2023, 2024, 2027, 2028
O-5	CSP	Comprehensive Sewer Plan Update	$ 300,000	2027, 2037
O-6	Reclaim Water ER	Water Reclamation Engineering Report	$ 60,000	TBD, before 2029
O-7	Record	Digital Records Project	$ 9,000	2019
O-8	Facility	Facility Upgrade/Building Upgrades	$ 29,000	2019
O-9	Financial Management Policy	Update Financial Management Policy	$ 7,500	2019
		Subtotal for Other Projects	**$ 788,500**	
		Grand Total	**$ 23,257,500**	

1 Estimated project cost is a preliminary estimate for budget purposes. Project cost is subject to refinement of project objectives, scope and more detailed cost estimate.

TABLE 7.3

10-YEAR CAPITAL IMPROVEMENT PLAN (2019-2028)

ID	Capital Improvement	Funding Source*	Estimated Project Cost - Thousands (2019 $)									
			2019	2020	2021	2022	2023	2024	2025	2026	2027	2028
	I. WWTP											
T-1	Toxicity Testing	A,C						12				
T-2	Evaluation	A,C						25				
T-3	NPDES	A,C						1				
T-4	WWTP Eng. Report	A,C		60								
T-5	UV Upgrade	A,C	373									
T-6	Headworks Odor Control Upgrades	A,C		170								
T-7	Biosolids Management Upgrade	A,C	41.5	158.5								
T-8	WWTP Site Work	A,C			84							
T-9	WWTP Upgrades	A,C				204	2,312	4,284				
	Subtotal for WWTP Projects		**415**	**389**	**84**	**204**	**2,312**	**4,322**	**-**	**-**	**-**	**-**
	II. PUMPING											
P-1	PS #8 Structure	A,C	268									
P-2	PS #3 Pump Upgrade Phase 1	A,C		52	468							
P-3	PS #4 and FM Upgrade Phase 1	A,C			55.6	500.4						
P-4	PS BR Upgrade	A,C				31.2	280.8					
P-5	PS #5 and FM Upgrade	A,C						337				
P-6	PS #6 and FM Upgrade	A,C								480		
P-7	PS #7 and FM Upgrade	A,C										303
	Subtotal for Pumping Projects		**268**	**52**	**524**	**532**	**281**	**337**	**-**	**480**	**-**	**303**

TABLE 7.3

10-YEAR CAPITAL IMPROVEMENT PLAN (2019-2028)

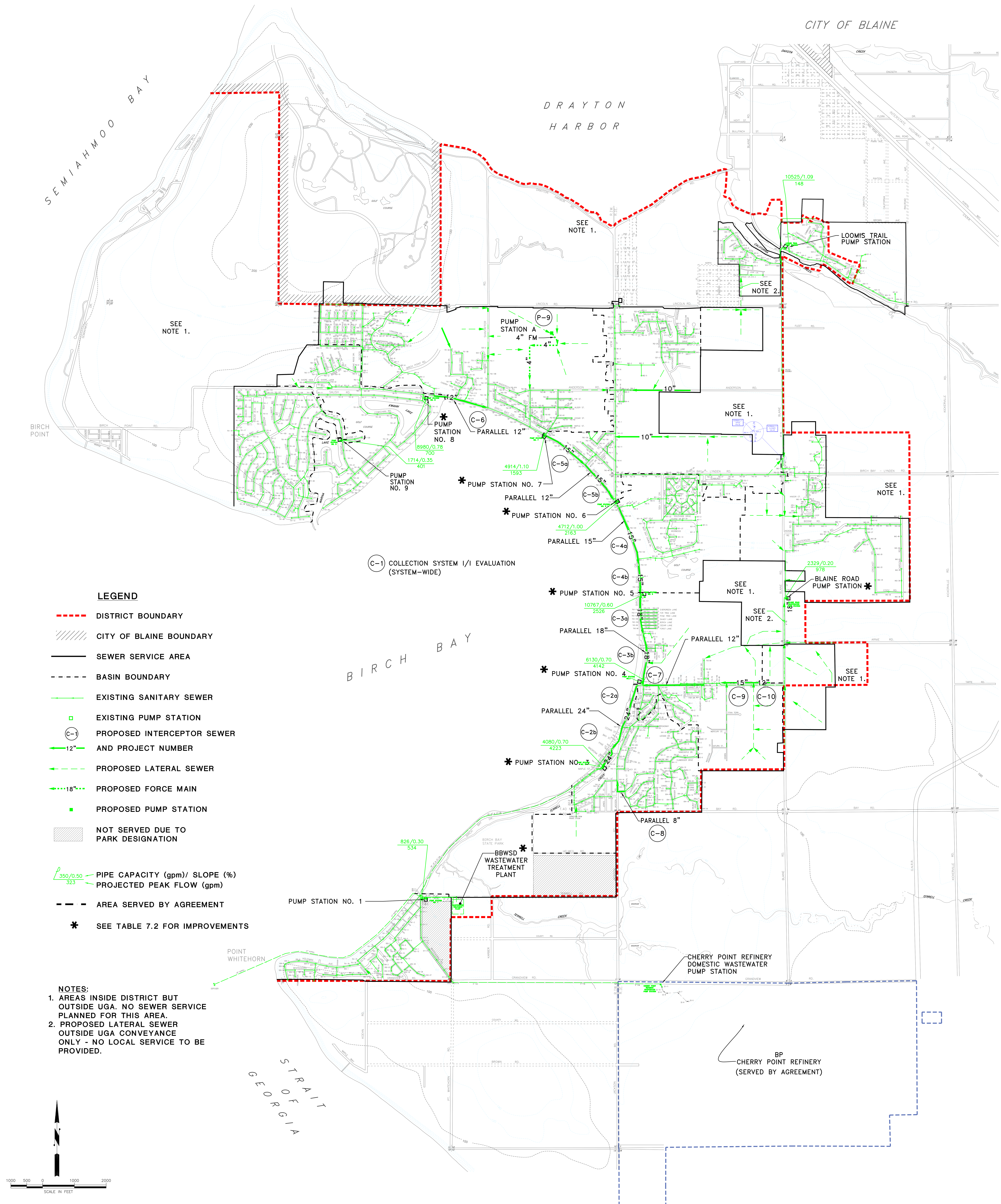
ID	Capital Improvement	Funding Source*	Estimated Project Cost - Thousands (2019 $)									
			2019	2020	2021	2022	2023	2024	2025	2026	2027	2028
						III. COLLECTION						
C-1	Collection System Evaluation & Repair - I/I	C		35	35	35	35	35	35	35	35	35
C-2a	24" - PS #3 to #4 - Ph I	A,C		136.3	1,227							
C-3a	18" - PS #4 to #5 - Ph I	A,C					703					
C-4a	15" - PS #5 to #6 - Ph I	A,C							966			
C-5a	15" - PS #6 to #7 - Ph I	A,C									776	
C-6	12" - PS #8 to MH 742-105	A,C										819
C-7	12" - Alderson Road (Parallel)	A,C										1,128
C-9	15" Alderson Rd.	A, B		425								
C-10	12" Alderson Rd.	A, B						400				
Subtotal for Collection Projects			**-**	**596**	**1,262**	**35**	**738**	**435**	**1,001**	**35**	**811**	**1,982**
						IV. OTHER						
O-1	PS #4 Pre-design Update	A,C	25									
O-2	SCADA	A,C	45									
O-3	Phones	A,C	15									
O-4	Vehicles	A,C	53			100	50	45			25	25
O-5	CSP	A,C									150	
O-6	Reclaim Water ER	A,C										60
O-7	Record	A,C	9									
O-8	Facility	A,C	29									
O-9	Financial Management Policy	A,C	8									
Subtotal for Other Projects			**184**	**-**	**-**	**100**	**50**	**45**	**-**	**-**	**175**	**85**
Grand Total			**866**	**1,037**	**1,869**	**871**	**3,381**	**5,139**	**1,001**	**515**	**986**	**2,370**

*Funding Sources:

A - GFC Funded from District General Facilities Charges revenue

B - LFC Constructed by developer extension, or by District with subsequent cost recovery thru Local Facilities Charge revenue

C - Rates System improvements projects funded by sewer service charges

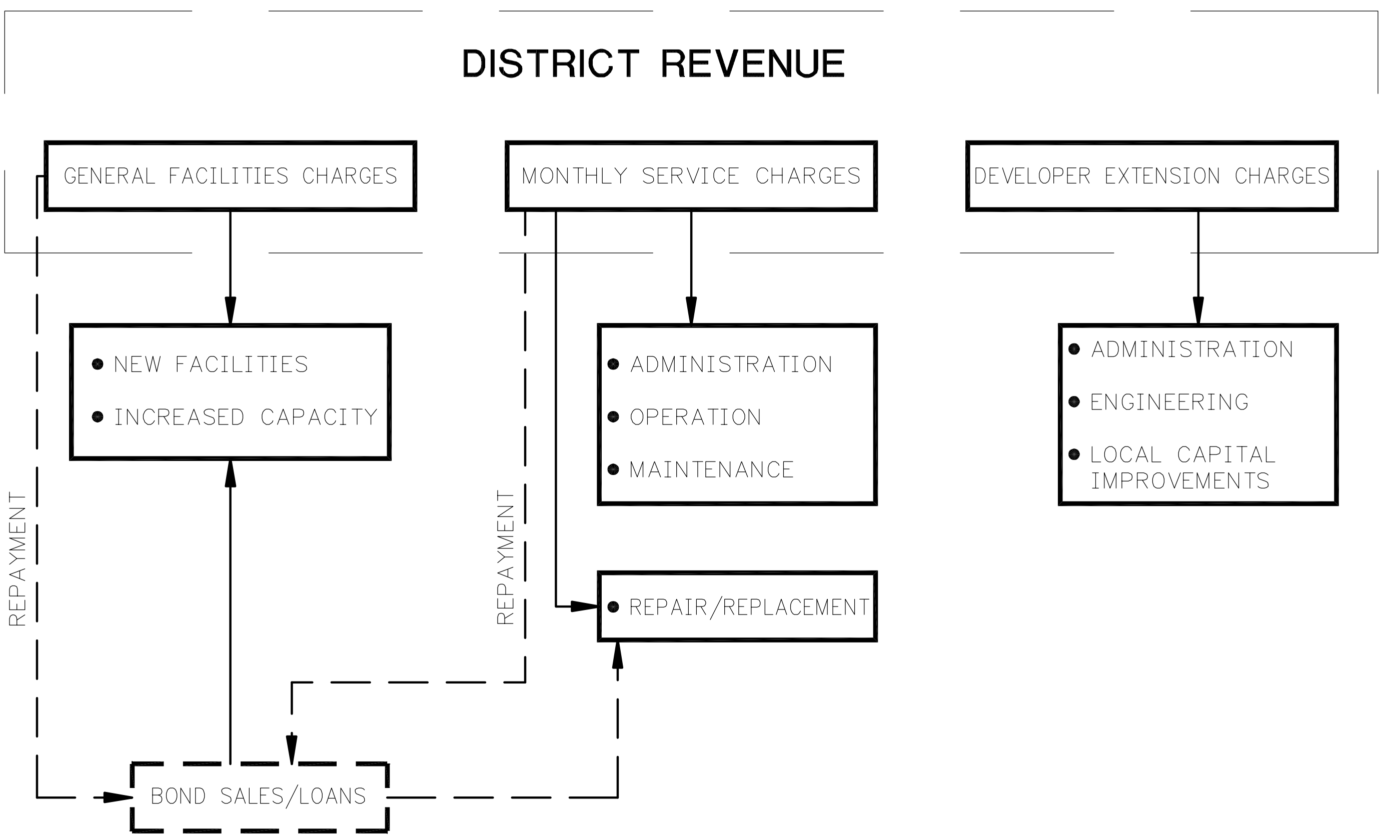


CHS ENGINEERS

Birch Bay Water and Sewer District

2019 Comprehensive Sewer System Plan

FIGURE 7.1
CAPITAL IMPROVEMENT PLAN



USER: Scott Stcherbinine PLOTTING DATE: 8/28/2018 FILE LOCATION: K:\19-BBWSD\2015\191502 Comp Sewer Plan Update\191502 CAD Figures\191502 CSP - FIG 7-2.dwg

CHS ENGINEERS

Birch Bay Water and Sewer District

2019 Comprehensive Sewer System Plan

FIGURE 7.2

SCHEMATIC OF FINANCES

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CHAPTER 8

DEVELOPER PROJECT STANDARDS

Birch Bay Water and Sewer District has had, and will continue to have, developer extensions for sewer service to residential and commercial development.

The Board of Commissioners of Birch Bay Water and Sewer District has established certain standards for the extension of existing main line sewers within the District. These standards are published in a bound volume titled *Developer Project Manual*. This manual is updated periodically with the last revision dated May, 2019.

The *Developer Project Manual* is available for reference at the Birch Bay Water and Sewer District office.

The current Design Criteria for sewer extensions is reprinted below:

BIRCH BAY WATER AND SEWER DISTRICT
DESIGN AND CONSTRUCTION STANDARDS – SEWER

1. GENERAL

All extensions to the sewer system must conform to the design standards of the District. In general, the Developer is required to construct the sewer lines through his property in order to allow for future extension, expansion and continuation of the District's collection system or for conformance with the Comprehensive Sewer System Plan. The following items are necessary to meet the conditions.

The District and its consultants do not insure the correctness of the information supplied to the Developer from the District's records. The Developer shall verify by survey any information provided by the District prior to using the information in design or construction.

A. Plans and Specifications

The installation of sewer extensions shall be made in accordance with these Conditions and Standards. The scale shall be: horizontal 1" = 50' or other scale as appropriate for the specific project, subject to the approval of the District; vertical 1" = 5' on 22" x 34" mylar. The minimum text height shall be 0.12 inch. The plans shall be sealed by a Professional Engineer licensed in Washington. Enclosed is a sample plan showing a typical sewer design. Drafting of plans for the District shall conform to this example. The sewer extension shall be shown on a sheet separate from the sewer, storm drainage and roadway plans. If the project is part of a phased development, a plan of the entire development shall be included, with the current phase clearly indicated.

The construction plans shall be reviewed or prepared by the District's Engineer. The developer shall submit three (3) sets of plans for review by the District. When the plans have been determined to meet the District standards, then a final set of reproducible plans shall be submitted to the District. These reproducible plans shall receive the District's "Plan Review" approval stamp. The District shall submit the plans to the regulatory agencies for approval. After approvals have been received, a set of plans stamped "Issued for Construction" shall be made available to the developer.

When the Contractor completes the main line sewer work and the manholes have been adjusted to the finish grade, the mylars of the sewer plans shall be revised to conform with construction records, and then sent to the District. Prior to submitting revised plans, manhole inverts and horizontal alignment shall be verified by a professional land surveyor. Photomylars are required for the District record drawings.

B. Right-of-Way and Monuments

All rights-of-way in which the sewer extension is to be made shall be improved prior to preparation of construction plans and installation of the sewers. Permanent private easements shall be not less than twenty feet (20') in width. Public rights-of-way shall be cleared, grubbed and graded in accordance with the requirements of Whatcom County. Monuments disturbed or destroyed shall be replaced at the Developer's expense.

2. DESIGN STANDARDS

A. Unless otherwise called for by the District's Engineer in the specification and plans, gravity sewers shall be PVC pipe.

Plastic-PVC	ASTM D3034-SDR 35 or F789
Ductile Iron (Polyethylene Encased)	AWWA C151
Concrete	ASTM C-14 Class 2

B. Manholes shall be precast, shall be 48" I.D. in accordance with the specifications and Detail Nos. 13 and 14 of the Specifications and shall conform to ASTM C478. Manhole covers shall be locking type in accordance with the specifications and Detail No. 12 and shall be supplied with stainless steel allen head cap screws.

C. Pressure mains shall be ductile iron, high density polyethylene (HDPE), or PVC, of a pressure class suitable for the anticipated working and surge pressure.

D. All joints for sewers or pressure mains shall be of the rubber gasket type for ductile iron or PVC pipe, or butt-fusion for HDPE pipe.

E. The sewer system extension shall be sized and routed to meet the following criteria:

   i. Provide sewer line to serve all the frontage of all lots or structures in the proposed development, so as to limit the length of side sewers to 150 feet.

   ii. Connect between the sewer system in the proposed development and the District's existing sewer collection system and in the manner indicated by the District

   iii. Extend sewerline through the property for potential future connection or extension in accordance with the District's Comprehensive Sewer System Plan or as required by the District.

   iv. Pipe diameter shall be minimum of 8 inches, or larger as required for future service in accordance with District's Comprehensive Sewer System Plan.

F. Minimum grade for 8-inch mains shall be 0.5% and the minimum grade for end sewer mains that will not be extended shall be 0.75% unless otherwise approved by the District. Minimum grade and design criteria, unless District criteria is more stringent, shall be in accordance with "Criteria for Sewage Works Design", Washington State Department of Ecology (DOE); latest edition. However, grades between 0.40% and 0.50% may be approved by the District for use on non-terminal 8-inch sewer runs if topographic or other conditions prevent use of the minimum slope. If a slope of less than 0.50% is approved, the sewer must be constructed at no les than minimum slope approved. Slopes shall be expressed to two decimal places if shown as a percentage, or to four decimal places if expressed as a ratio (i.e., feet of rise per feet of run). DOE minimum slopes for sewer mains larger than 8-inch diameter may only be used upon approval of the District.

G. Manholes shall be placed at each grade and direction change. Distances between manholes shall not exceed 400 feet. Manholes shall be a minimum of six (6) feet deep and shall be seven (7) feet deep where possible and shall be used at the termination of each sewer, unless otherwise approved by District. Joints on manhole sections shall be rubber gasket type and shall be externally sealed per the Details and Specifications.

H. The grade for 6” side sewer stubs shall be a minimum of 2 percent (2%).

I. A tight line bypass shall be required to separate existing flows from the new connection until final acceptance of the sewer extension. A grouted in-place plug shall be required at the connection of a new system to a dead end existing manhole until final acceptance of the sewer extension.

J. Sewer lines shall generally be located five (5) feet on either side of centerline in accordance with Whatcom County Development Standards.

K. Terminal manholes where future connection/extension may occur shall not be channeled. A grouted bottom sloping to the outlet shall be constructed.

L. Pump stations shall have at least two submersible wastewater pumps with a standby power generator and telemetry system. All equipment shall meet the requirements and approval of the District.

M. All-weather vehicular access suitable for use by District sewer maintenance vehicles, and access right of way or easement granted to the District, shall be provided to all manholes and/or cleanouts on the public sewer extension, unless waived by the District.

3. EASEMENTS

Legal descriptions for easement to be dedicated to the District for all portions of the sewer system which lie outside of public street rights-of-way shall be signed and stamped by a professional land surveyor and transmitted to the District. Easements shall be twenty (20) feet in width or as required by the District. An easement may coincide with another utility easement, except that all sanitary sewer lines must be ten feet or more from water lines and other utilities must be a minimum of five feet from the water lines. Sewer shall be located no closer than ten (10) feet from the easement edge. There shall be a separate easement provided for each lot that a sewer line crosses. These easements are required by the District regardless of easements recorded with property deeds or plats.

Easements must be approved by the District prior to sewer service connection.

4. CONSTRUCTION AND INSPECTION

A. Installation and Inspection

No work on the sewer system shall be performed without a District Inspector being present. The District may refuse acceptance of any portion of the work installed without the Inspector having reviewed the work. The District must be notified a minimum of two full working days in advance of a firm starting date and time to arrange for and schedule the Inspector. Work must proceed in a continuous manner. If there are breaks in construction, there must be two working days' notice before beginning work again.

The Developer or Contractor shall furnish cut sheets to the Engineer for review two work days prior to the start of construction. Cut sheets shall include offset hubs at manholes (new and/or existing) and at 25' and 50' upstream of manholes (unless the distance to the next manhole is shorter) and every 100 feet thereafter. Construction shall not commence until Contractor has received the cut sheets stamped "issued for construction" from the Engineer.

The approved construction plans and specifications shall be followed. No deviations will be allowed without request for change and approval received from the District. The District reserves the right to order changes in the event of conditions or circumstances discovered during construction; such changes could result from the ability or care shown by the Contractor, natural and man-made conditions, or any other reason.

The Contractor shall exercise extreme care in checking and cleaning all pipes and fittings of dirt, debris, and/or any foreign matter during installation. All material shall be kept clean. Plugs shall be used to seal system installed when it is to be left for any period of time, including lunch breaks, coffee breaks, and overnight. Pipe and fittings will be cleaned before installation if contaminated by dust, smoke, exhaust or any other material. Material contaminated by petroleum products or questionable chemical will be rejected. No trench water is to be allowed to enter installed system.

All connections to existing District mains and manholes must be performed while the District Inspector is present.

B. As-Built Drawings

When the Contractor completes the sewer system work, the mylars of the sewer plans shall be revised to conform with construction records and then sent to the District. Photomylars are required for the District's record drawings. Prior to submitting revised plans, manhole and cleanout location elevation and horizontal alignment shall be verified by a professional land surveyor.

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REFERENCES

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