CITY OF ABERDEEN

GRAYS HARBOR COUNTY **WASHINGTON**

REGIONAL GENERAL SEWER/WASTEWATER FACILITY PLAN

G&O #18594
AUGUST 2020

Gray & Osborne, Inc.
CONSULTING ENGINEERS

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EXECUTIVE SUMMARY

This *Regional General Sewer/Wastewater Facility Plan* (*Regional Facility Plan*) for the City of Aberdeen addresses the City's planning needs for wastewater collection, transmission, treatment, and disposal for the 20-year planning period. This Plan was prepared in accordance with the provisions of the Revised Code of Washington (RCW), Section 90.48, *Water Pollution Control*, Washington Administrative Code (WAC) Section 173-240-050, *General Sewer Plan*, and WAC 173-240-060, *Engineering Report.* Development of the Plan has been coordinated with the City's 2001 *Comprehensive Plan*, Grays Harbor County planning efforts, and with the City's 2013 *Water System Comprehensive Plan*.

POPULATION AND FLOW FORECASTS

In addition to City of Aberdeen flows, the City's existing wastewater facilities convey and treat flows from the City of Cosmopolis and the Stafford Creek Corrections Center (SCCC). The *Regional Facility Plan* considers the cost effectiveness and environmental benefits of expanding the existing facility or developing a new larger treatment facility to serve additional partners within Grays Harbor County, specifically the City of Hoquiam and Central Park.

Chapter 2 provides detailed information regarding City and County planning and population projections. An annual growth rate of 1.0 percent was selected through a collaboration with the City of Aberdeen to project the future City of Aberdeen service population, for conservatism. (The actual average growth rate has been closer to 0.1 percent for the past 15 years.) This same annual growth rate of 1.0 percent was used for Cosmopolis and Central Park. The growth rate of 0.77 percent used in Hoquiam's *2009 Comprehensive Plan* was applied to Hoquiam in this evaluation. SCCC flows are assumed to be constant based on projections from the Washington State Department of Corrections.

Table E-1 presents population projections for Aberdeen and existing and potential future partners.

TABLE E-1

Projected Population in Aberdeen Wastewater Collection System Service Area (with Hoquiam and Central Park)

Service Area	Population				
	2018	**2023**	**2028**	**2033**	**2038**
City of Aberdeen	16,760	17,615	18,513	19,458	20,450
City of Cosmopolis	1,665	1,750	1,839	1,933	2,032
SCCC(1)	2,150	2,150	2,150	2,150	2,150
Central Park	0	0	1,473 (2)	2,013 (3)	2,603 (4)
Aberdeen Plant Total	**20,575**	**21,515**	**23,976**	**25,553**	**27,235**
Hoquiam	8,560	8,895	9,242	9,604	9,979
Regional Total	**29,135**	**30,410**	**33,218**	**35,157**	**37,215**

(1) Data reported by the City, including full capacity of 1,972 inmates and population equivalent of employees.
(2) It was assumed 50 percent of the total population (2,946) is connected by 2028.
(3) It was assumed 65 percent of the total population (3,096) is connected by 2033.
(4) It was assumed 80 percent of the total population (3,254) is connected by 2038.

Chapter 5 provides a detailed evaluation of past flows and loadings, as well as projections for the future. WWTP records for the period from 2013 through 2018 were reviewed and analyzed to determine current wastewater characteristics and influent loadings. Current wastewater flows and loadings were used in conjunction with projected population data to determine projected future wastewater flows and loadings. In general, infiltration and inflow (I/I) are assumed to be constant throughout the 20-year planning period for much of the service area. (This means ongoing I/I reduction efforts in those areas are assumed to compensate for increased I/I due to growth in the sewer area and deterioration of existing infrastructure.) However, based on our analysis, the completion of the ongoing North Shore Levee project, including an estimated $75 million in levee and stormwater pumping improvements, will significantly reduce ponding and flooding and thus I/I in the low-lying downtown and adjacent areas. The analysis estimates a 12 percent overall reduction in peak day ***inflow***. (Since most of the total peak day flow ***is*** inflow, this results in a 10 percent reduction in total projected peak day flow and a 9 percent reduction in total projected peak hour from what would have otherwise been projected for the future).

Flow and loading projections for Hoquiam were based on values in the *2013 Hoquiam Wastewater Facility Plan*, except for peak day and peak hour flow projections. The *2013 Hoquiam Wastewater Facility Plan* notes that actual peak day and peak hour flows projected to be generated in the Hoquiam system are 14.35 and 15.06 mgd, respectively. However, it is expected that regional wastewater life cycle costs can be minimized by equalization of Hoquiam's flows prior to conveyance to Aberdeen. Per analysis from HDR, peak hour/day flows are assumed to be equalized to 6.5 mgd in an equalization basin constructed near the K Street Pump Station or in the existing Hoquiam WWTP lagoon.

Tables E-2 and E-3 summarize the 10- and 20-year influent flow and loading projections, respectively.

TABLE E-2

Expanded Regional Flow Projections

Flow Type	Projected Flow Rate (mgd)			
	Aberdeen WWTP Total(1)	Hoquiam(2)	Central Park(3)	Expanded Regional Total
2028				
Total Base	2.16	0.92	0.15	3.23
Average Annual	4.15	1.49	0.17	5.81
Maximum Month	7.11	3.35	0.26	10.72
Peak Day	18.73	6.50 (4)	0.43	25.66
Peak Hour	21.34	6.50 (5)	0.68	28.52
2038				
Total Base	2.47	1.10	0.24	3.81
Average Annual	4.46	1.72	0.26	6.44
Maximum Month	7.42	3.73	0.39	11.54
Peak Day	19.05	6.50 (4)	0.66	26.21
Peak Hour	21.97	6.50 (5)	1.05	29.52

(1) Aberdeen total flow including flow from Cosmopolis and SCCC.

(2) Hoquiam flow projections are interpolated from *2013 Hoquiam Wastewater Facility Plan*.

(3) Central Park base flow is calculated based on population projections and a typical wastewater flow rate of 100 gpcd. Annual average, maximum month, peak day, and peak hour flows are calculated using typical peaking factors.

(4) Actual peak day flows projected to be generated in the Hoquiam system in 2028 are 13.17 mgd peak day and 13.82 mgd peak hour. Per the analysis from HDR, these flows will be equalized to 6.5 mgd in an equalization basin.

(5) Actual peak day flows projected to be generated in the Hoquiam system in 2038 are 14.35 mgd peak day and 15.06 mgd peak hour. Per the analysis from HDR, these flows will be equalized to 6.5 mgd in an equalization basin.

TABLE E-3

Expanded Regional Loading Projections

Loading (lb/d)	Aberdeen WWTP Total(1)	Hoquiam(2)	Central Park	Regional Total
2028				
Annual Average BOD_5	7,095	2,325	368	9,788
Annual Average TSS	7,279	2,261	398	9,937
Maximum Month BOD_5	8,412	3,308	437	12,157
Maximum Month TSS	8,871	3,423	485	12,778
2038				
Annual Average BOD_5	8,102	2,785	651	11,537
Annual Average TSS	8,285	2,707	703	11,695
Maximum Month BOD_5	9,569	3,963	769	14,301
Maximum Month TSS	10,028	4,100	851	14,979

(1) Aberdeen total loading including loading from Cosmopolis, SCCC, and hauled septage.
(2) Central Park loadings are calculated based on population projections and typical wastewater loading 0.25 BOD ppcd and 0.27 TSS ppcd. Maximum month and peak day loading are calculated based on the same peaking factor of Aberdeen.

COLLECTION SYSTEM EVALUATION

COLLECTION SYSTEM MODELING

Chapter 4 summarizes the collection system and its condition. Chapter 6 summarizes hydraulic modeling of, and recommended improvements for the collection system. As discussed in Chapter 5, the City has excessive infiltration and inflow, as defined by EPA. However, most of the extraneous flow is due to inflow. The City is currently in the design phase of the North Shore Levee flood control project, which will add additional storm water conveyance and storm water pump stations and is anticipated to further reduce flooding and ponding in the City during storms, and significantly reduce the need for collection system capacity upgrades due to I/I. Hydraulic modeling, conducted with XPSTORM modeling software, identified several areas at risk of sanitary sewer overflows (SSOs) under peak flow conditions. As shown in Figures 6-10, 6-12 and 6-14, several pipes in the vicinity of Grant Street and Market Street have peak flows under all scenarios exceeding 130 percent of capacity. Because of this, as shown in Figure 6-8, there is risk of overflows at several locations near Grant Street, Arthur Street and Chicago Avenue. The risk is exacerbated when the wet well level in the Influent Pump Station is higher. Similarly, manholes along Port Road are at risk of overflows due to capacity limitations (associated with flat slope) in that line, and the risk is increased by high Influent Pump Station wet well levels. In addition, as shown in Figure 6-9, due to capacity limitations, several manholes are at risk of overflows in South Aberdeen to the west of Highway 105 under all scenarios. Finally, as discussed above, acceptance of peak flows from Central Park would cause risk of overflows in the line upstream of Pump

Station 4. As discussed in Chapter 6, it is recommended that the City conduct an I/I Study to identify feasible cost effective means of reducing I/I –related flows, particularly for the areas south of the Chehalis River.

PUMP STATIONS

As described in Chapter 4, the City of Aberdeen owns, operates and maintains 17 pump stations within its sanitary sewer system. Pump Station 1 is the WWTP influent pump station, and Pump Stations 2 through 16 are located throughout the collection system. Additional sewage pump stations are operated and maintained by the City that serve the SCCC and Lemay Landfill. The locations of these pump stations are shown in Figure 4-1. Basic information about the pump stations is included in Table 4-3. All of the collection system pump stations contain two pumps except Pump Station 13, which contains three pumps. Many of the pump station facilities are approaching the end of their useful life and/or require upgrades in the near future. Common deficiencies observed for virtually all the collection system pump stations include lack of security, space not NFPA 820 compliant, and metal corrosion.

In addition, a major deficiency is the lack of piping connections and miscellaneous piping to the force mains near the stations, to allow bypass of the pumps at the stations during power outages or pump failures. Currently, for Pump Stations 2, 4, 5, 6, and 7, if both pumps fail, there is no bypass connection to connect a portable pump. Thus, the City has to pump wastewater from the wet well into trucks and transport the wastewater to a downstream location or to the WWTP. Fortunately, these situations have been rare events and generally occurred in low flow situations. However, if this were to occur during a storm, the result could be massive sanitary sewer overflows in the vicinity of the stations.

Pump Stations 2, 4, 5, 6, 7, 8, 9 and 13 are approaching the end of their useful lives and need to be upgraded in the near future. All electrical at Pump Station 13 is in an underground vault requiring confined space entry and should be raised above grade.

Based on the analysis of pump station flows in Chapter 6, it was concluded that flows to Pump Stations 2 and 7 exceeded capacity (i.e., all pumps, including the redundant pump, were operating) during three recent peak storm events, and Pump Stations 4, 5, 6, 8, 9, 10, 13 and 16 were operating at “full capacity” (with the redundant pump on) during one or two recent peak storm events. Thus, these stations do not have adequate capacity to meet the state and EPA reliability criteria and are in need of a capacity upgrade.

Recommended collection system improvements are summarized in the Capital Improvement Plan section later in this Executive Summary.

REGIONAL CONVEYANCE

A detailed evaluation of conveyance improvement alternatives to serve Hoquiam at the Aberdeen WWTP is provided in the *Expanded Regional Conveyance Alternatives Evaluation Technical Memorandum (Conveyance Memorandum,* HDR, 2020) in Appendix M. The *Conveyance Memorandum i*s summarized at the end of Chapter 6. Due to a lack of available capacity in the Aberdeen's collection system, conveying flow from Hoquiam to the Aberdeen WWTP, bypassing Aberdeen's existing collection system, would be recommended. In order to limit the peak day and peak hour flows conveyed to Aberdeen, flows from Hoquiam would be equalized by providing storage at the existing Hoquiam WWTP site or near the K Street Pump Station.

Four alternatives to serve Hoquiam were considered. In summary, converting from the current wastewater conveyance pattern in the City to conveyance to an expanded regional facility at Aberdeen would require some major and expensive modifications to the Hoquiam conveyance system. Detailed descriptions are provided in the *Conveyance Memorandum*. As shown in Table 6-11, the least expensive capital cost is for Option A1 (Equalization storage at Hoquiam WWTP, force main along Port Industrial Way) at $20.8M.

WWTP CONDITION ASSESSMENT

Table E-4 summarizes the Condition Assessment for the WWTP facilities. The information about the condition of the facilities is taken from the *WWTP and Collection System Condition Assessment* (Condition Assessment) which is provided as Appendix D. In the Condition Assessment, each unit process was assigned an average condition value based on the remaining useful life and the relative cost to restore the unit process to its original physical condition, as well as an importance rating that indicates the relative consequence of specific facility failure with regard to the overall wastewater treatment process. The average condition rating is the mean value for individual ratings of mechanical, electrical, structural, civil, and HVAC. The higher the condition rating, the worse the condition; the higher the importance rating, the higher the consequence of failure. Many of the structures and process units at the Aberdeen WWTP are more than 40 years old and some require replacement or rehabilitation.

TABLE E-4

WWTP Condition Assessment Summary and Necessary Improvements to Address Deficiencies

Item	Process Area	Primary Discipline(s) Deficient	Importance	Average Condition Rating	Weighted Rating	Likely Improvement(s) Necessary
1	Influent Pump Station and Administration Building	Structural/Civil	5	3	15	Rehabilitate wet well and miscellaneous structural improvements, floodproofing
2		HVAC	5	3	15	Ventilation improvements for safety/code compliance
3	Influent Manhole	Mechanical, Piping, and Instrumentation	5	2.75	13.75	Rehabilitate mechanical, piping, and instrumentation sampling system improvements
4	Large Digester	Structural/Civil	5	2.75	13.75	Structural replacement/ rehabilitation (including roof), floodproofing
5		Electrical	5	2.75	13.75	Electrical improvements for classification/compliance
6		Mechanical	5	2.75	13.75	Rehabilitate co-generation system and gas piping replacement
7	Primary Sludge Pump Room	Electrical/Civil	4	3	12	Electrical improvements for safety/code compliance, floodproofing
8		HVAC	4	3	12	Ventilation improvements for safety/code compliance
9	Headworks	Mechanical	4	2.5	10	Improve redundancy and increase capacity
10	Aeration Basins	Structural	4	2.5	10	Miscellaneous structural improvements
11		Electrical	4	2.5	10	Rehabilitate settled conduit
12	Dewatering Facilities	Structural	4	2.4	9.6	Miscellaneous structural improvements

TABLE E-4 – (continued)

WWTP Condition Assessment Summary and Necessary Improvements to Address Deficiencies

Item	Process Area	Primary Discipline(s) Deficient	Importance	Average Condition Rating	Weighted Rating	Likely Improvement(s) Necessary
13	Generator System	Mechanical	3	2.8	8.4	Replace generator and switchgear (w/larger unit to also cover secondary processes)
14		Structural	3	2.8	8.4	Miscellaneous structural improvements
15		HVAC	3	2.8	8.4	Ventilation improvements for code compliance
16	Small Secondary Clarifier	Structural/Mechanical	3	2.75	8.25	Rehabilitate secondary clarifier
17		Electrical	3	2.75	8.25	Rehabilitate settled conduit
18	RAS Pump Room	HVAC	3	2.75	8.25	Ventilation improvements for code compliance, floodproofing
19	WAS Pump Room	HVAC	3	2.75	8.25	Ventilation improvements for code compliance
20	Primary Clarifiers	Structural/Mechanical	3	2.6	7.8	Rehabilitate primary clarifiers and scum pump stations
21	Influent Sampling System	Mechanical	3	2.5	7.5	Establish consistently representative sampling location
22	Effluent Flow Measurement	Mechanical/Electrical	3	2.5	7.5	Replace Parshall flume and rehabilitate settled conduit
23	Gravity Sludge Thickener	Structural/ Mechanical/Electrical	3	2.25	6.75	Rehabilitate gravity sludge thickener
24	Small Digesters	Structural/ Mechanical/Electrical	2	3.25	6.5	Rehabilitate small digester

EXISTING WWTP EVALUATION

Chapter 4 summarizes key performance parameters for the WWTP. The plant has an excellent compliance record and has won numerous State awards for outstanding compliance. Average performance for the plant has been good with average effluent concentrations over the past 5 years of 8.6 mg/L for BOD and 8.1 mg/L for TSS. Percent removal has averaged 94 percent for BOD and 96 percent for TSS. The plant consistently produces Class B biosolids compliant with all applicable criteria.

Chapter 7 presents an evaluation of WWTP hydraulic and treatment capacity for flows and loadings for three scenarios: (1) Aberdeen and Existing Partners (2) Aberdeen and Expanded Regional Partners (Hoquiam and Central Park) and (3) Aberdeen and Expanded Regional Partners with Additional Industrial Flow. The evaluation determined that it would be necessary to increase the capacity of a number of the WWTP treatment plant processes (including the Influent Pump Station, Headworks Screening and Grit Handling, Generator and Aeration Basins) to accommodate 20-year flows and loadings just for Aberdeen and Existing Partners. Significant additional improvements would be needed to treat the additional flows and loads from additional partners.

WWTP ALTERNATIVES

Future WWTP alternatives are evaluated in Chapter 8. Based on a preliminary evaluation, it was recommended that all WWTP alternatives retain the existing basic processes:

- For liquid stream treatment, the major processes include screening, primary clarification, grit removal from primary sludge, conventional activated sludge treatment with multizone aeration basins in a Modified Ludzack-Ettinger (MLE) configuration, secondary clarification, and chlorination.

- For solids treatment, the major processes include sludge thickening, anaerobic digestion, and biosolids dewatering.

Figures 8-3 through 8-6 showing the layouts for the four alternatives considered in Chapter 8. The alternatives include:

1. <u>Serve Existing Regional Partners on Existing Site</u>

   This alternative includes upgrades to the existing WWTP to provide sufficient capacity to serve Aberdeen and the existing regional partners (Cosmopolis and SCCC) for the next 20 years and beyond, and to address issues identified in the Condition Assessment.

2. Serve Expanded Regional Partners on Existing Site

   For this alternative, the existing Aberdeen WWTP would be upgraded to serve Aberdeen, existing partners (Cosmopolis, SCCC), and additional regional partners (Hoquiam and Central Park) through the planning year and beyond. Treating Hoquiam and Central Park flows at the Aberdeen WWTP will necessitate construction of new facilities beyond those required for Alternative 1 including:

   - A new (second) headworks dedicated to screening and flow measurement for Hoquiam's flows (Central Park, Aberdeen, and Existing Partner flows would continue to be screened and measured with the existing upgraded headworks)
   - An additional 65-foot primary clarifier
   - An additional 0.47 MG aeration basin and 1,390 cfm blower
   - An additional 85-foot secondary clarifier
   - Site and piping modifications
   - Effluent pumping and outfall improvements

3. Serve Existing Regional Partners on New Site

   For new site (green field) Alternatives 3 and 4, preliminary conceptual designs were developed based on industry-standard criteria (including the State's *Criteria for Sewage Works Design*, (Orange Book)) for treatment processes, combined with output from CapdetWorks, a software tool provided by Hydromantis for preliminary design and cost estimation of wastewater treatment plant construction projects.

   Serve Expanded Regional Partners on New Site

   For Alternatives 3 and 4, it is assumed that a completely new WWTP is constructed at a new site consisting of the City's property to the west of the existing WWTP and a portion of the parking lot on the adjacent property.

Table E-5 summarizes a comparison of projected life cycle costs for the four alternatives, broken down between Aberdeen, Hoquiam and Central Park. The new-site alternatives (Alternatives 3 and 4) are considered to be cost–prohibitive, with a total capital cost and present worth more than double the cost of the existing-site alternatives (Alternatives 1 and 2). Thus, Alternatives 3 and 4 are rejected.

TABLE E-5

Cost Comparison for Alternatives (20-Year Life Cycle)

Alternative	1. Serve Existing Regional Partners on Existing Site	2. Serve Expanded Regional Partners on Existing Site	3. Serve Existing Regional Partners on New Site	4. Serve Expanded Regional Partners on New Site
Total Project Cost (Capital)	$50,068,000	$80,506,000	$165,705,000	$224,077,000
Aberdeen	$50,068,000	$35,924,000	$165,705,000	$155,184,000
Hoquiam	--	$38,642,000	--	$59,847,000
Central Park	--	$5,940,000	--	$9,046,000
O&M Present Worth Cost	$51,951,000	$62,617,000	$46,938,000	$59,778,000
Aberdeen	$51,951,000	$43,365,000	$46,938,000	$41,399,000
Hoquiam	--	$16,724,000	--	$15,966,000
Central Park	--	$2,528,000	--	$2,413,000
Total Present Worth	$102,019,000	$143,123,000	$212,643,000	$283,855,000
Aberdeen	$102,019,000	$79,289,000	$212,643,000	$196,583,000
Hoquiam	--	$55,366,000	--	$75,813,000
Central Park	--	$8,468,000	--	$11,459,000

(1) 3 percent inflation and discount rate used.
(2) This table is presented as Table 8-15 in Chapter 8.

Table E-6 summarizes life cycle costs for Alternative 1 ("Go It Alone" for both Cities) versus Alternative 2 (Hoquiam Served Along with Existing Partners at Existing Aberdeen WWTP). For Alternative 2, two options are shown; in the first option (Option 2A), Hoquiam pays for all the Regional Conveyance costs associated with conveying their wastewater to the Regional Plant. However, that does not appear to be attractive to Hoquiam, as it would result in a 20-Year Life Cycle for Hoquiam that is significantly more expensive than for Hoquiam to "Go It Alone" (Alternative 1). The only way to reduce the life cycle costs to significantly less than the "Go It Alone" option for both Cities is for Aberdeen to pay for most of the regional conveyance costs ("the other extreme"). In this "other extreme" (Option 2B), Aberdeen would pay the majority ($14 million) of the conveyance costs, an amount that results in significant 20-Year Life Cycle savings for both Cities (about 5 percent in overall life cycle costs). However, this would also make the capital costs for regionalization more expensive for Aberdeen than the "Go It Alone" option. A more attractive cost partitioning option for regionalization is that the share of both capital and operating costs is adjusted so that both capital and operating costs are lower for each City with regionalization.

It should be noted, however, since constructing the regional conveyance system, and additional new facilities on the Aberdeen WWTP site, would be among the first steps of regionalization, it would result in a significant immediate rate increase, and likely opposition to the project, for one or both Cities. It should be noted that this analysis (and costs presented throughout the *Regional Sewer Plan*) are based on <u>planning level</u> (Class 4

AACE) cost estimates, and actual costs could vary significantly from those provided. In addition, the City of Hoquiam is planning on updating their Facility Plan and "Go It Alone" costs, so additional information to update the life-cycle analysis should be available in the near future.

TABLE E-6

20-Year Life Cycle Cost Comparison Alternative 1 vs. Alternative 2 (not including Central Park Costs)

	Alternative 1: “Go It Alone”			**Alternative 2: Hoquiam Served Along with Existing Partners at Existing Aberdeen WWTP**	
Alternative	**Aberdeen “Go It Alone”: Continue to Serve Existing Regional Partners on Existing Site**	**Hoquiam "Go It Alone"**	**Sum of Aberdeen and Hoquiam "Go It Alone" Costs**	**2A: Hoquiam Pays All Regional Conveyance**	**2B: Aberdeen Pays Majority of Regional Conveyance**
Total Project Cost (Capital)	$50,068,000	$49,610,000	$99,678,000	$94,966,000	$94,966,000
Aberdeen	$50,068,000	$0	$50,068,000	$35,924,000	$52,924,000
Hoquiam	$0	$49,610,000	$49,610,000	$59,042,000	$42,042,000
O&M Present Worth Cost	$51,951,000	$12,252,000	$64,203,000	$60,089,000	$60,089,000
Aberdeen	$51,951,000	$0	$51,951,000	$43,365,000	$43,365,000
Hoquiam	$0	$12,252,000	$12,252,000	$16,724,000	$16,724,000
Total Present Worth (20-Year Life Cycle)	$102,019,000	$61,862,000	$163,881,000	$155,055,000	$155,055,000
Aberdeen	$102,019,000	$0	$102,019,000	$79,289,000	$96,289,000
Hoquiam	$0	$61,862,000	$61,862,000	$75,766,000	$58,766,000

(1) Hoquiam pays all regional conveyance costs.
(2) Aberdeen pays the majority of regional conveyance costs
(3) All costs are in 2020 dollars and are planning level, 3 percent inflation and discount rate used.
(4) This table is presented as Table 8-18 in Chapter 8.

Table 8-19 in Chapter 8 summarizes a non-economic evaluation of the two alternatives. For each alternative, a score is provided in the matrix, with 10 being the highest (best) score and 1 being the lowest (worst) score. As shown in that table, Alternative 1 ("Go It Alone") has a slightly higher overall rating. The economies of scale typically associated with regionalization in terms of capital cost do not appear to be as significant for Aberdeen and Hoquiam. The costs of the conveyance and the 20-year life cycle appear to be the most significant factors in realizing a significant economy of scale.

CAPITAL IMPROVEMENT PLAN AND FINANCIAL ANALYSIS

Chapter 9 summarizes the capital improvement plan and financial analysis. The proposed system improvements in the CIP are shown below in Tables E-7 and E-8 for the collection system and WWTP, respectively. Each project cost estimate includes sales tax, construction contingency, and design engineering, construction management and permitting. All project costs are based on 2020 dollars.

To pay for the capital improvements, City Council passed an ordinance in 2019 with a schedule of rate increases that would bring the monthly sewer rates from $46 to $72 by 2024 for both commercial and residential dwelling units.

TABLE E-7

Collection System – 6-Year Capital Improvement Plan

CIP	Project Name	Cost	Year	Description
	Infiltration and Inflow Study	$75,000	2020-2021	I/I Study with smoke testing, flow monitoring, TV inspection
CS-1	Bypass Connections PS 4,6,7	$201.000	2021-2022	Bypass connections to force main to allow bypass of pump stations
CS-2	PS 5 Upgrade	$676,000	2021-2022	Replace force main, all mechanical, electrical and I&C; rehabilitate wetwell concrete surface; add bypass piping connection
CS-3	Fry Creek Pump Stations	$200,000	2020-2021	Small pump stations (project completed by City staff)
CS-4	PS 6 Upgrade	$1,306,000	2021-2022	Replace pumps, all mechanical, electrical. I&C and force main. Rehabilitate wet well concrete surfaces.
CS-5	PS 13 Upgrade	$2,425,000	2021-2022	Construct new above grade control room; replace all mechanical, electrical and I&C; install new generator; rehab wet well concrete surface; add bypass piping connection; Upsize downstream piping
CS-6	PS 10 Upgrade	$580,000	2025-2026	Replace mechanical, electrical and I&C; Rehabilitate wet well concrete surface; add bypass piping connection
CS-7	PS 7 Upgrade	$1,589,000	2021-2023	Upsize pumps to 1,200 gpm, replace mechanical, electrical and I&C; Install new generator; Rehabilitate wetwell concrete surface; Replace forcemain.
CS-8	PS 4 Upgrade	$1,087,000	2021-2023	Upsize pumps to 1,000 gpm, replace mechanical, electrical and I&C; Rehabilitate wet well concrete surface; replace force main.
CS-9	PS 8 Replacement	$1,362,000	2022-2024	Replace Pump Station
CS-10	PS 2 Upgrade	$1,081,000	2024-2025	Add pump, replace all mech., electr. and I&C; rehabilitate wet well concrete surface.
CS-11	PS 9 Upgrade	$865,000	2024-2025	Upsize pumps to 1,000 gpm. Replace mechanical, electrical and I&C; Install new generator; rehab wet well concrete surface; replace discharge force main.
CS-12	PS 11 Upgrade	$606,000	2025-2026	Replace pumps, all mechanical, electrical and I&C; rehab wet well concrete surface; add bypass piping connection.

6-Year CIP only; additional future projects identified in Chapter 8.

TABLE E-8

Wastewater Treatment Plant – 6-Year Capital Improvement Plan

CIP	Project Name	Cost	Year	Description
WW-1	Influent Pump Station Pump Replacement	$42,770	2020	Replace single pump at end of useful life
WW-2	Influent Pump Station VFD Replacement	$67,500	2020	Replace single VFD at end of useful life
WW-3	Disinfection Improvements	$2,482,625	2019-2020	Convert to liquid chlorination/dechlorination, rehabilitate process water system (project to be completed in fall 2020)
WW-4	New WWTP Generator	$3,149,000	2022-2024	New generator, switchgear
WW-5	Influent Pump Station Rehab.	$2,966,000	2021-2023	Rehabilitate wet well, structural improvements, ventilation compliance
WW-6	Existing Digester Rehabilitation	$2,609,000	2021-2022	Fix roof, Replace gas lines, heat exchanger, boiler, electrical code upgrades
WW-7	Primary Sludge Pump Room Rehabilitation	$1,241,000	2021-2023	Electrical and controls, ventilation compliance, process piping improvements, flood hazard mitigation
WW-8	Aeration Basin Improvements	$2,138,000	2023-2025	Miscellaneous structural, mechanical and electrical improvements, including tank surface rehabilitation, remediate settling of yard piping and electrical raceways
WW-9	Headworks Upgrade	$2,558,000	2021-2023	New screens and washer compactors, raise walls, modify stairway access, electrical improvements
WW-10	Secondary Clarifier 1 Improvements	$1,529,000	2023-2025	Replace mechanisms, equipment, surface rehabilitation, remediate settling of yard piping and electrical raceways.
WW-11	Thickener Upgrade	$1,379,000	2023-2025	Replace mechanisms and equipment, surface rehabilitation, Remediate settling of yard piping and electrical raceways, replace yard piping
WW-12	Conduit/Piping Rehabilitation	$250,000	2025-2027	Remediate settled conduit, process piping
WW-13	East Primary Clarifier Rehabilitation(1)	$273,000	2025-2027	Replace mechanisms, equipment, surface rehabilitation, Remediate settling of yard piping and electrical raceways

(1) 6-Year CIP only; additional future projects identified in Chapter 8.

CHAPTER 1

INTRODUCTION

GENERAL

This *Regional General Sewer/Wastewater Facility Plan* (*Regional Facility Plan*) for the City of Aberdeen addresses the City's planning needs for wastewater collection, transmission, treatment, and disposal for the 20-year planning period. This Plan was prepared in accordance with the provisions of the Revised Code of Washington (RCW), Section 90.48, *Water Pollution Control*, Washington Administrative Code (WAC) Section 173-240-050, *General Sewer Plan*, and WAC 173-240-060, *Engineering Report.* Development of the Plan has been coordinated with the City's 2001 *Comprehensive Plan*, Grays Harbor County planning efforts, and with the City's 2013 *Water System Comprehensive Plan*.

The *Regional Facility Plan* provides proposed conceptual designs, cost estimates, schedules, and financing plan for recommended major facility improvements. A State Environmental Policy Act (SEPA) checklist is provided in Appendix A. The projects described in the Regional Facility Plan are consistent with Washington State regulations relating to the prevention and control of discharge of pollutants into waters of the state, anti-degradation of existing and future beneficial uses of ground waters, and anti-degradation of surface waters.

The City of Aberdeen is located within Grays Harbor County in southwest Washington State as shown in Figure 1-1.

SCOPE OF WORK

Since the *Regional Facility Plan* is intended to be both a General Sewer Plan and a Wastewater Facilities Plan, the *Regional Facility Plan* evaluates both the wastewater collection system and the wastewater treatment system in detail. This evaluation includes collection and treatment system modeling, analysis and a capital improvement plan with cost analysis and schedule. In addition to City of Aberdeen flows, the City's existing wastewater system conveys and treats flows from the City of Cosmopolis and the Stafford Creek Corrections Center (SCCC). The *Regional Facility Plan* considers the cost effectiveness and environmental benefits of expanding the existing facility or developing a new larger treatment facility to serve additional partners within Grays Harbor County, specifically the City of Hoquiam and Central Park.

The scope of work for the *Regional Facility Plan* includes the following items:

- Background data
- Service area characteristics
- Population and land use
- Regulatory criteria
- Projected future flow and loadings to the Wastewater Treatment Facility (WWTF)
- Pertinent performance and design criteria for system facilities
- Evaluation of the WWTP
- Computer modeling and evaluation of wastewater collection system
- Identification of system improvements with cost estimates
- Financing plan for capital improvement plan
- Environmental analysis

RELATED PLANNING DOCUMENTS

The following documents were consulted in the preparation of this Regional Wastewater Facilities/General Sewer Plan.

City of Aberdeen Infiltration and Inflow Study, April 1999, Earth Tech Inc.

This report evaluated the capacity of the City's sewage collection system and addressed the issue of Infiltration and Inflow (I/I). A series of technical memoranda were included. Data collection included wastewater flow, precipitation, groundwater levels and tide levels.

The analysis quantified the amount of I/I entering the sewer system, and determined the sources of I/I entering the system.

Some of the conclusions of the study included:

- Flooded yards, basements and crawl spaces had compelled many residents to make illegal surface water connections to the wastewater collection system.

- Staff believed that improvements to the stormwater system would be the most beneficial way to reduce inflow.

- Groundwater levels were often above the sewer main in the lower parts of the City, and above side sewers in wet weather.

- Extreme high tide affected flow rates in the basins on either side of the Chehalis River Bridge (Basins 1 and 19 in the I/I Study, which are named

Legend

- Cities
- Highway
- Street
- Waterbody

N

0 3,000 6,000 12,000 Feet

ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 1-1
LOCATION MAP

Gray & Osborne, Inc.
CONSULTING ENGINEERS

differently in the *Regional Facility Plan*). In addition, high tide affected flow depth but not flow rate in several other basins, due to backwater effects from downstream tidal influences.

- Direct storm inflow as well as the infiltration into manholes and side sewers were the major sources of the extraneous flow.

The study estimated that without rehabilitation to remove extraneous flow, in year 2020, the peak flow would be 25.0 mgd.

The report evaluated the technical feasibility and cost-effectiveness of potential sewer system rehabilitation projects to remove excessive I/I, and also provided a prioritized list of sewer system rehabilitation and treatment plant improvements, including estimated costs.

According to the subsequent 2009 *Comprehensive Sewage Facilities Plan Update*, ("*2009 Facilities Plan Update*"), the City has maintained an ongoing effort to minimize I/I. Annual activities include identifying illegal connections and monitoring progress on their correction, manhole rehabilitation through grouting and epoxy lining, replacement of damaged sewer sections, and hydro cleaning.

<u>*City of Aberdeen Comprehensive Facilities Plan*</u>, January 2000, KCM/Tetra Tech, Inc

This report provides a comprehensive evaluation of City Aberdeen's sewage treatment and discharge facilities, including:

- Assessment of existing condition of plant facilities;
- Study of existing flow and loading and development of projections of future flow and loading;
- Evaluation of plant facilities capacity;
- Recommendations to improve plant performance and accommodate future growth needs.

Several capacity bottlenecks were identified in the treatment process, such as an Influent Pump Station (IPS) capacity deficiency, Headworks (HWs) overflow, poor activated sludge settleability, and overtopped Effluent Parshall Flume.

Plant effluent records also indicated several violations for overflow, fecal coliform, TSS and BOD removal between the years 1994 and 1998.

The capacity evaluation determined some major process units, such as IPS, HWs, aeration basin, secondary clarifier, and chlorine contact tank would have an immediate capacity problem.

The plan evaluated various alternatives and provided the following recommendations:

- Replace the IPS and HWs
- Replace the mechanical aerator with new a fine bubble aeration system
- Add two secondary clarifiers
- Replace the chlorine disinfection system with a UV disinfection system
- Replace the effluent piping
- Add a new gravity belt thicker

According to the *2009 Facilities Plan Update*, the following improvements were made in 2000-2004 improvement project:

- The mechanical aeration system was converted to fine bubble aeration. A new blower building with new centrifugal blower was constructed.

- Anoxic and anaerobic selector zones were created in the aeration tanks to improve sludge settleability.

- The third secondary clarifier was added.

- Two new cloth filter units were installed for effluent filtration.

- A new rotary drum thickener was installed to improve sludge thickening.

- The existing plate and frame sludge filter press was replaced with a screw dewatering press to improve sludge dewatering.

The effluent outfall line and diffusers were reconstructed in the 2014 Outfall Replacement Project.

City of Aberdeen Comprehensive Sewage Facilities Plan Update, September 2009, Carollo Engineering, Inc.

This report was required by the City's NPDES permit, and is an update of the City's *2000 Comprehensive Facilities Plan* limited to discussion of data that had changed since the *2000 Plan*. The plan provided documentation for re-rating the plant capacity based on the upgrades to the plant competed by the City since the *2000 Plan*.

The report addressed the condition of the existing WWTP and evaluated alternatives and recommended a series of capital improvements projects to serve the population growth through 2030. As noted below, many of these recommended improvements have not been constructed.

With the year 2030 chosen as the design target, the projected peak flow was well in excess of the NPDES limit, and projected BOD and TSS limits were also near to or exceeded the NPDES limit.

The EPA SWMM modeling identified the IPS as the major capacity bottleneck.

The 5-year CIP projects recommended are listed below (along with their status):

- PS 7 upgrade (not constructed).
- Upgrade the existing IPS and construct new 1.4 MG equalization basin and flow diversion screening and pumping facilities (not constructed).
- Replace existing outfall diffuser for ammonia discharge mitigation, including the CMP portion of the outfall pipe (constructed).
- Upgrade the existing gas chlorination and gas dechlorination systems with liquid hypochlorite and liquid dechlorination systems (under construction in 2020).
- Add new standby generator for the entire plant including secondary treatment process (not constructed).
- Add heating and mixing system for anaerobic digestion (mixing system constructed).
- Secure backup biosolids disposal contractor (secured).

City of Aberdeen Comprehensive Sewage Facilities Plan Update Evaluation of Leachate Handling, April 2011, Carollo Engineering, Inc.

This report evaluates the feasibility of pretreatment of leachate from the LeMay landfill prior to discharge to the City's sewer and develops a preliminary design for a pump station to deliver leachate into the City's sewer system through the existing force main. It was concluded that pretreatment of leachate would not be a cost-effective alternative. Historic leachate data from this plan was incorporated into the flow and loading analysis of Chapter 5.

City of Aberdeen Water System Plan, March 2013, HDR Inc.

The City of Aberdeen (City) owns and operates a domestic water system serving customers within its City limits, the nearby community of Cosmopolis, and adjacent areas within Grays Harbor County (County).

The *2013 Water System Plan* presents an inventory of existing facilities, evaluates the current and future water demand, describes compliance with the water reservation program and water rights and source reliability, assesses drinking water quality, and recommends capital improvements to meet demand and address system deficiencies. In addition, the Plan provides recommendations for the operation and maintenance of the water system.

Historic water consumption data from the *2013 Water System Plan* was incorporated into the flow and loading analysis of Chapter 5 of the *Regional Facility Plan.*

City of Hoquiam Comprehensive General Sewer Plan, 2009, HDR, Inc.

Hoquiam's wastewater planning is relevant to the preparation of the *Regional Facility Plan,* as Hoquiam is a potential regional partner for the City of Aberdeen.

The capital improvements recommended in the *Hoquiam General Sewer Plan* for the collection system included:

- A second parallel force main crossing the Hoquiam River to serve East Hoquiam.
- Replacing aging force mains prone to leakage, requiring recurring maintenance and repair.
- Replacing aged, inefficient, and difficult to maintain emergency generators at the pump stations.
- Improving daily flow management by installing variable speed drives on the 2nd and Bayview Pump Station.
- Improving wet weather flow monitoring and management by installing flow meters at the pump stations and a rain gauge at the wastewater treatment plant.

The existing WWTP deficiencies identified in the evaluation include:

- Most of the existing facilities are outdated and reaching the end of their useful life.
- Other than lagoon storage, there are no other facilities for handling wastewater solids, and the biosolids have not been removed from the lagoon for over 30 years.
- The existing solids storage lagoon is unlined and has the potential to contaminate groundwater.
- Chlorine gas, a potential hazard, is still used for disinfection.
- There are no redundant treatment units.
- The use of single, large treatment units makes management of both wet and dry weather flow challenging.
- Standby power is limited to disinfection only.

The preferred course of action is to construct a new wastewater treatment plant. Planning-level cost estimates for a new WWTP were provided for budgeting purposes and to assess financial funding implications. A siting study and environmental review process was needed to identify the site for a new facility.

City of Hoquiam Wastewater Facility Plan, Dec 2013, HDR Inc.

The *City of Hoquiam Wastewater Facility Plan* ("*Hoquiam Plan*") provides an evaluation of the collection system and treatment facilities in the City of Hoquiam, adjacent to the City of Aberdeen.

The following conclusions were offered regarding Hoquiam's collection system:

- Three out of the total nine basins contain a large number of defects that are contributing to excessive Infiltration and Inflow (I/I). Like Aberdeen, Hoquiam's extraneous flows are predominantly inflow.
- The pump station and force main analysis suggests that these facilities have sufficient capacity under current (2012) flow conditions.

- Aging infrastructure and deferred renewal and replacements necessitate improvements to existing force mains and replacement of nine existing pump station emergency generators.

The *Hoquiam Facility Plan* included a Condition Assessment that noted that some of the major process units, including the oxidation ditch and clarifier, were in poor condition. The sloped side walls of the oxidation ditch are concrete lined, but the bottom consists of asphalt-coated gravel. This configuration, which was permissible at the time of design, has now been recognized by Ecology as a potential pathway for groundwater contamination and tidal inflow.

The *Plan* noted that the oxidation ditch, clarifier, headworks, and RAS/WAS/lagoon water pump station are expected to reach the end of their service life by 2028. In addition, the *Facility Plan* noted that the aeration basin (ditch) and clarifier failed to meet reliability/redundancy criteria

The plan also developed the projected future flow and loadings that will be incorporated into the *Regional Facility Plan.*

CHAPTER 2

BACKGROUND

The City of Aberdeen is located at the eastern end of Grays Harbor, near the mouth of the Chehalis River and southwest of the Olympic Mountains. The city is the economic center of Grays Harbor County, bordering the cities of Hoquiam and Cosmopolis (see Figure 2-1) According to the U.S. Census Bureau, the City covers an area of about 12.4 square miles (32.0 km2), of which 10.7 square miles (27.6 km2) is land and 1.7 square miles (4.4 km2) is water. In 2018, the population was reported to be 16,760 by the Washington State Office of Financial Management.

SEWER SERVICE AREAS

The City's wastewater treatment plant currently serves a population of about 20,500, with an estimated 6,100 connections. About 10 percent of these connections are commercial or industrial; the rest are residential. The sewer service area includes the City of Aberdeen, the City of Cosmopolis and the Stafford Creek Correctional Center (SCCC). The estimated service area is 3,770 acres for the City of Aberdeen and a total of 4,370 acres for all areas including the City of Cosmopolis and SCCC.

Aberdeen's collection system includes 17 pump stations and approximately 85 miles of sanitary sewers, including a 24-inch force main under the Chehalis River. The wastewater treatment plant provides secondary treatment and has a maximum-month design capacity of 9.9 million gallons per day (mgd) and a peak hour design capacity of 18 mgd. Cosmopolis owns, operates and maintains its wastewater collection system.

NATURAL ENVIRONMENT

TOPOGRAPHY

Ground elevation in the City's sewer service area ranges from 10 to 400 feet above sea level. The topography in the area generally slopes from the higher elevation plateau in the north towards the low elevation river banks along the Chehalis River and Wishkah River. Approximately 2,000 acres within the city has slopes greater than 30 percent. The downtown and older parts of the city are located on relatively flat ground and are within or near the coastal floodplain of the Grays Harbor estuary. Residential areas are built on slopes ranging from 0 to 5 percent. Figure 2-2 is topographic map based on United States Geologic Survey (USGS) showing the varying elevations within the sewer service area.

SOILS AND GEOLOGY

The wastewater service area is on an alluvial terrace that was formed during the late tertiary to quaternary age. Some sedimentary rock of the tertiary age is also present. Much of the low lying Aberdeen-Hoquiam-Cosmopolis areas has been filled with dredged estuary sediment, lumber mill waste, tree stumps, and clays (URS 1976).

The majority of the soils within the Grays Harbor County are classified as Ocosta, Udorthents and Zenker-Elochoman by the U.S Department of Agriculture Soil Conservation Service (SCS).

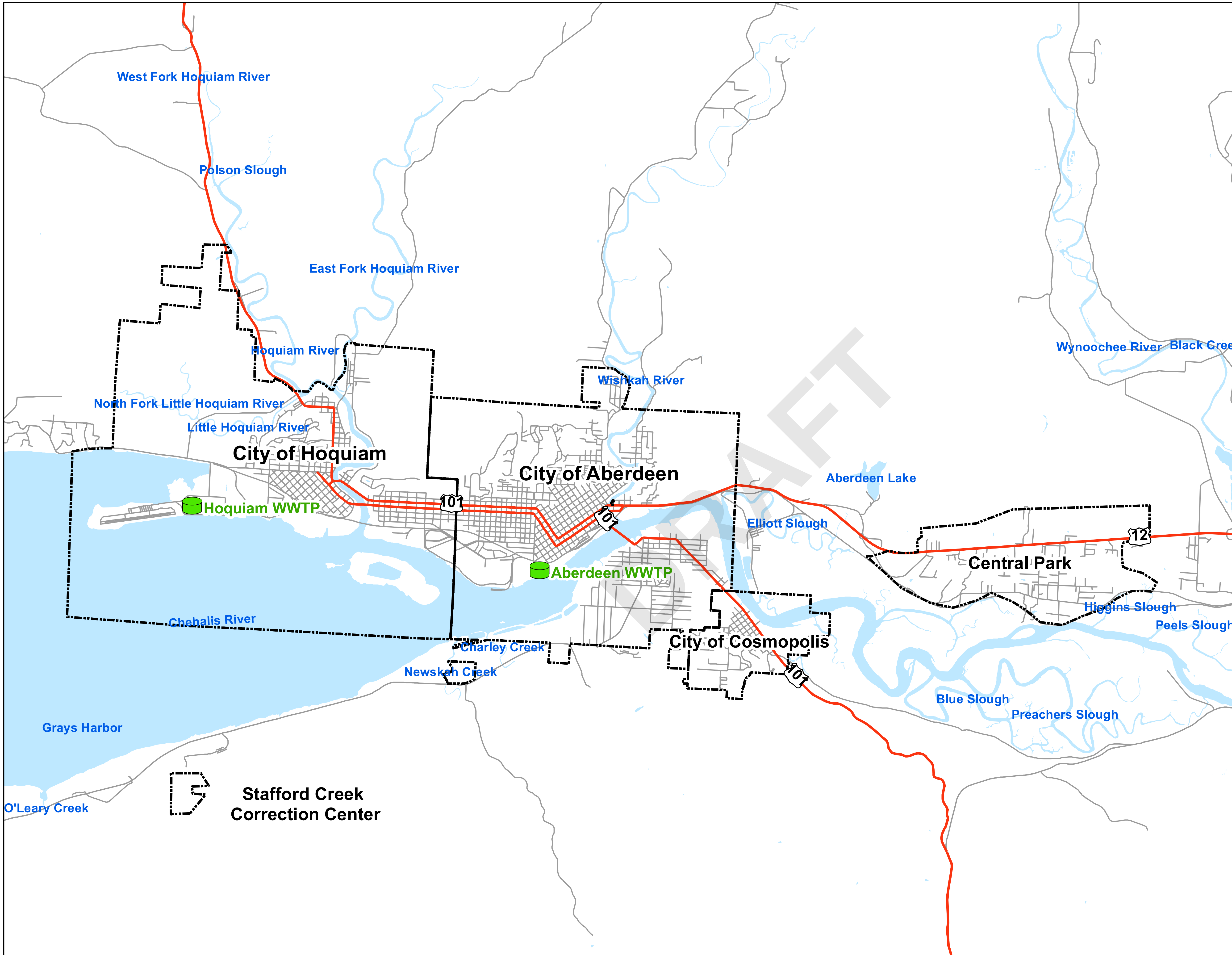
The Ocosta soils are defined as very deep, poorly drained, nearly level soils on flood plains and deltas. The Udorthents series soil consists of sandy and loamy river dredging on diked tidal flats. The Zenker-Elochoman soils are very deep, well drained, and nearly level to extremely steep soils on sandstone uplands.

A map showing locations of the soil classifications within the Wastewater Service Area is presented in Figure 2-3, based on an SCS survey.

CLIMATE

Aberdeen's climate is classified as "maritime" and "Mediterranean," characterized by cool summers and mild winters. Temperatures range from average monthly low of 35.4 degrees F to a high of 69.3 degrees F. Aberdeen's average annual rainfall is 89.5 inches.

Table 2-1 provides precipitation and temperature data measured at the National Oceanic and Atmospheric Administration (NOAA) weather station (USC00450008) in Aberdeen



ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 2-1
VICINITY MAP

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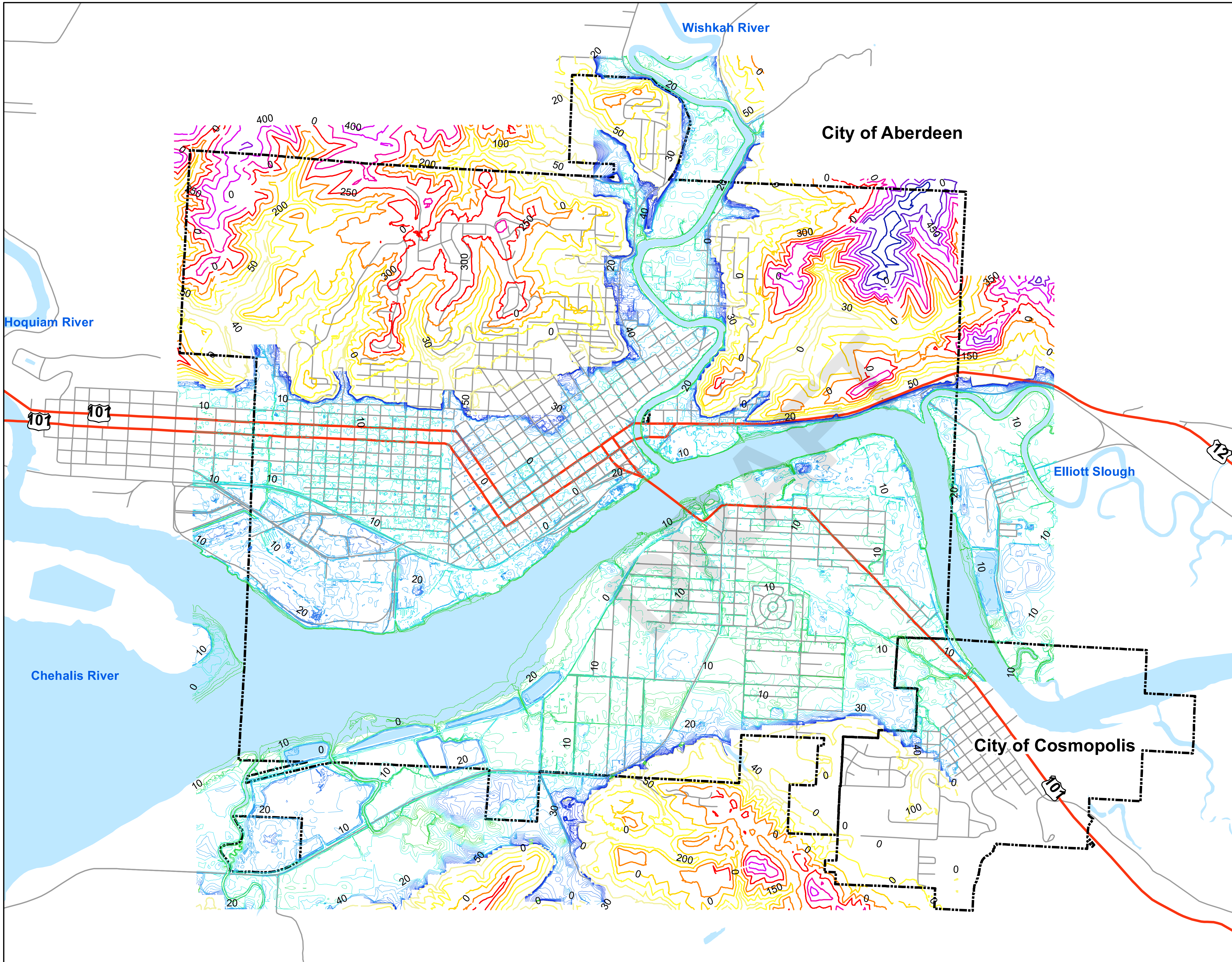

Legend

- CityLimit
- Highway
- Waterbody
- Street

Elevation 0-50 ft

Elevation 50-550 ft

N

0 1,0002,000 4,000 Feet

ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 2-2
TOPOGRAPHIC MAP

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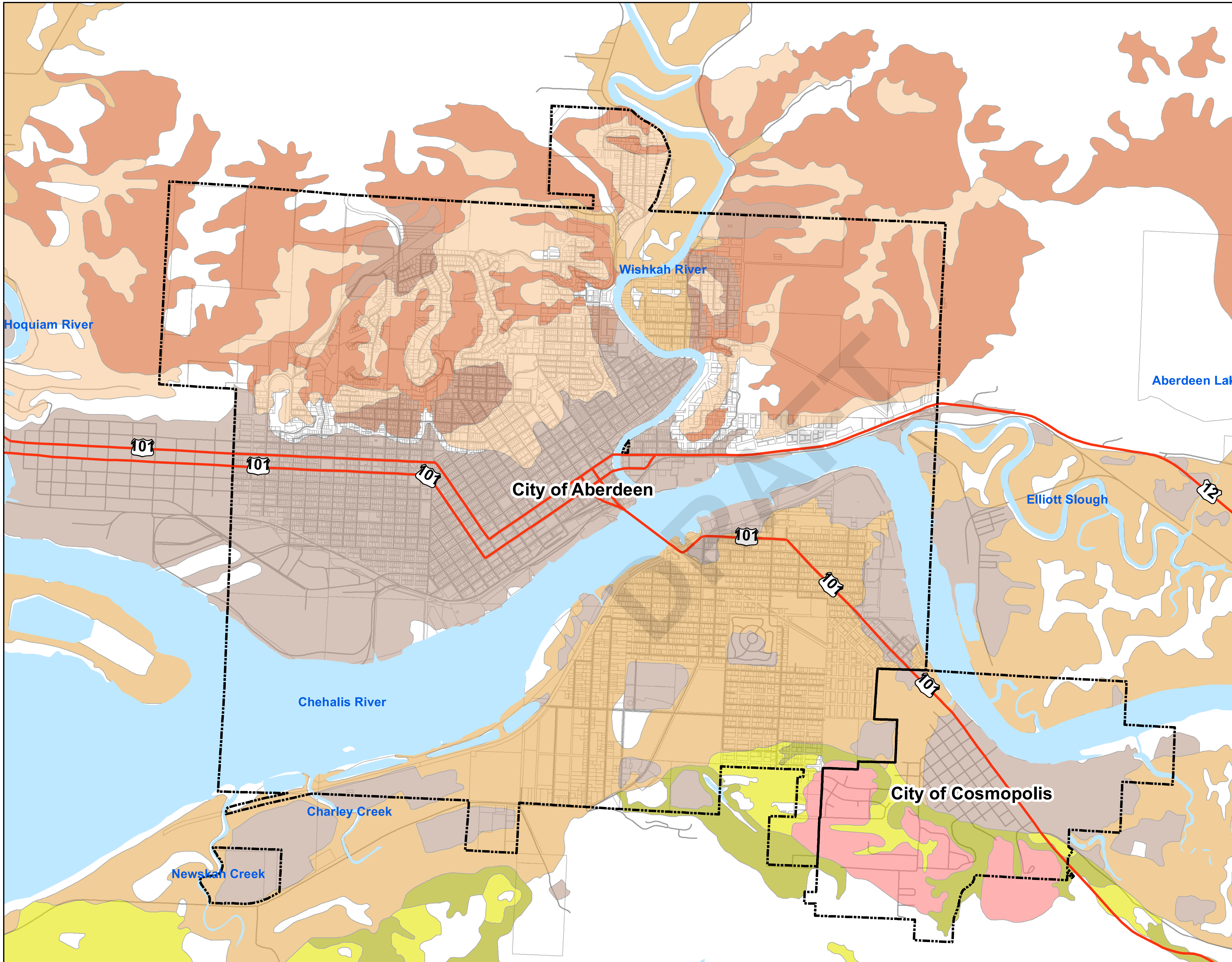

Legend

CityLimit
Highway
Waterbody
Soils
Ocosta Silty Clay Loam
Udorthents
Zenker Silt Loam
Elochoman Silt Loam, 1-8 %
Elochoman Silt Loam, 8-30 %
Calawah Silt Loam, 1-8 %
Mopang silt loam, 5-30 %
Mopang silt loam, 30-65 %
Parcel

N

0 1,000 2,000 4,000 Feet

ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 2-3
SOILS

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CONSULTING ENGINEERS

TABLE 2-1

Aberdeen, WA Station Climate Data 2009-2017

Month	Average Total Precipitation (Inches)	Average Maximum Temperature (°F)	Average Minimum Temperature(°F)
Jan	12.3	47.6	36.4
Feb	8.4	50.0	37.4
Mar	13.0	52.7	38.8
Apr	6.6	56.2	40.8
May	4.0	60.7	46.5
Jun	2.2	64.6	51.2
Jul	0.7	67.5	53.9
Aug	1.1	69.3	54.5
Sep	3.5	69.0	51.8
Oct	10.7	61.4	45.8
Nov	15.1	52.1	39.8
Dec	11.9	46.3	35.4
Annual Average	**90.0**	**58.1**	**44.4**

SOURCE: NOAA, National Virtual Data System.

In the late spring and summer, westerly to northwesterly winds flows over the cold ocean surface forming a high pressure center that contributes to dry conditions over the North Pacific. In late fall and winter, southwesterly and westerly winds off the Pacific Ocean are the source of moisture during the wet season.

SITE-SENSITIVE AREAS

The following section summarizes information regarding site-sensitive/critical areas presented in the *City of Aberdeen Comprehensive Plan*, July 2001. Critical areas within the sewer service area include those classified as streams and watercourses, wetlands, frequently flooded areas, critical aquifer recharge areas, geologically hazardous areas, and fish and wildlife habitat conservation areas.

Surface Water

Lakes and streams are classified as sensitive areas due to the variety of plants and animals that they support. The major surface waters located within the service area include the Chehalis River and Wishkah River. The intent of municipal code 13.70 Storm and Surface Water Management, is to prevent adverse effects to water quality in the Aberdeen area. The major surface waters are shown in Figure 2-1.

Wetlands

The Growth Management Act defines wetlands as areas that have surface or ground water that supports vegetation typically adapted in saturated soil conditions. Wetlands support valuable and complex ecosystems and consequently development is severely restricted if not prohibited in most wetlands and buffer areas around the wetland. There are approximately 740 acres within the City that are classified as wetlands.

The major wetlands within the City are located along Chehalis River and Wishkah River. The intent of the wetland standard in Sections 14.100.200 through 14.100.263 of the City's municipal code is to prevent adverse effects to wetlands and wetland buffers from development effects. Figure 2-4 shows wetland areas within the City.

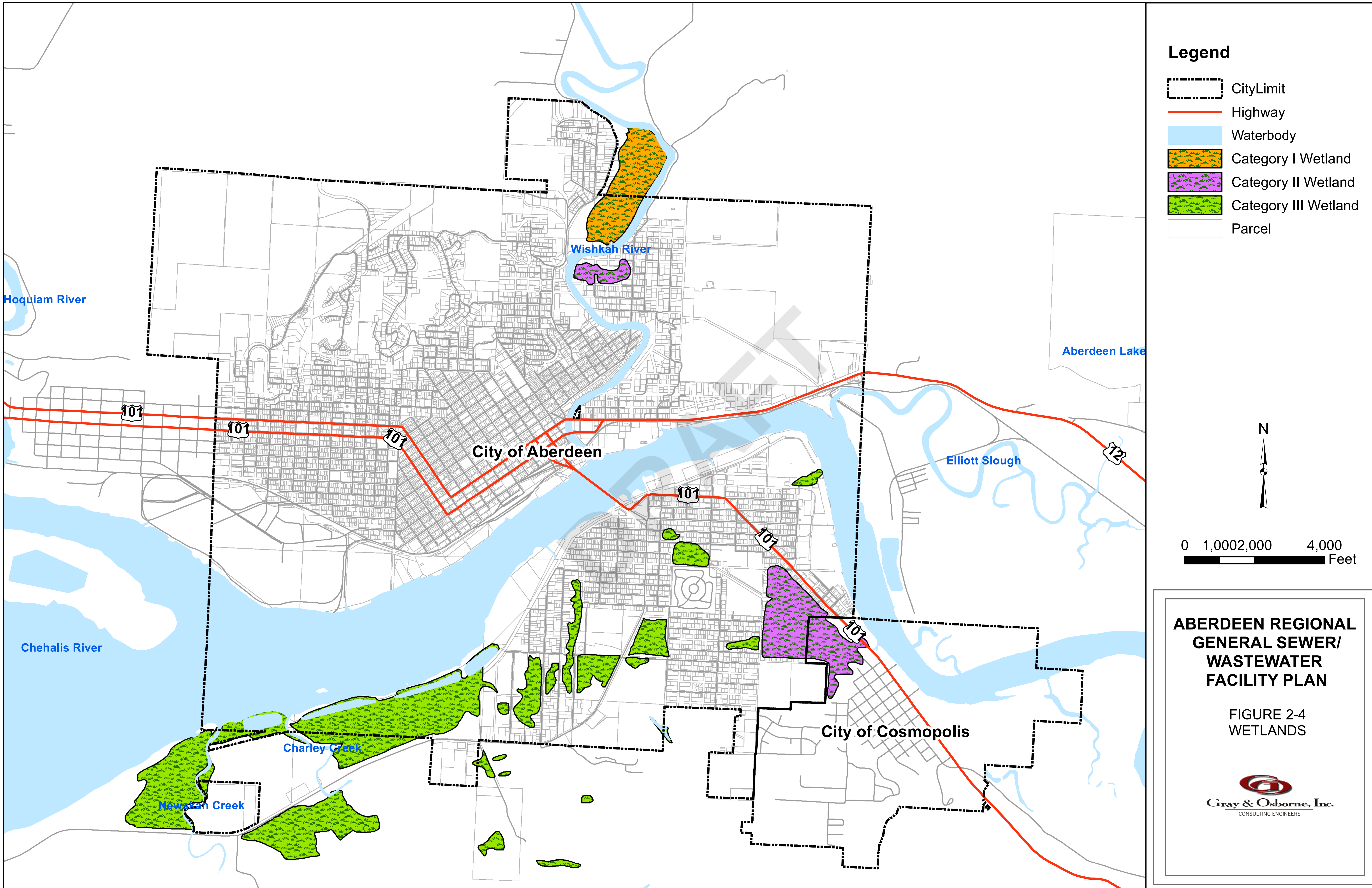
Frequently Flooded Areas

Frequently flooded and flood hazard areas are areas adjacent to lakes, rivers, streams and the ocean that are prone to flooding during peak runoff periods. Construction of buildings and other development in these areas is regulated in accordance with flood hazard construction standards. Significant portions of the City, including several collection system pumps stations and the treatment plant, are located within the 100-year floodplain map (land that has a 1 percent chance of flooding each year) as mapped by the Federal Emergency Management Agency (FEMA). Flood protection will be considered in the planning of the facilities upgrades. The floodplain map is shown in Figure 2-5.

The City has received funding for a project to provide flood protection and flood insurance relief to portions of Aberdeen and Hoquiam (the "North Shore Levee project"). Design and permitting of the project are currently underway. The project includes 5.7 miles of levee between the Wishkah and Hoquiam Rivers to protect against coastal flood events, plus upgrades to and expansion of stormwater pump systems to improve drainage. Once the Levee is constructed and accredited, over 3,100 properties in Aberdeen and Hoquiam, including Downtown Aberdeen, will be removed from the Special Flood Hazard Area (SFHA) due to coastal flood risk. As discussed in Chapter 6, the reduction in flooding and surface water ponding provided from the project is expected to reduce peak, flood-related, flows within the wastewater collection system. Other potential benefits of the levee project include reducing pump station run times, collection system surcharging and attenuating wastewater treatment plant peak influent flow.

Critical Aquifer Recharge Areas

Critical Aquifer Recharge areas (CARA) are those areas with a critical recharging effect on aquifers used for potable water as defined by municipal code Section 14.100.100. CARA have prevailing geologic conditions associated with infiltration rates that create a high potential for contamination of ground water resources or contribute significantly to the replenishment of ground water. The City utilizes both the United States Safe Drinking



Legend
CityLimit
Highway
Waterbody
Category I Wetland
Category II Wetland
Category III Wetland
Parcel
N
0 1,0002,000 4,000
Feet
ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN
FIGURE 2-4
WETLANDS
Gray & Osborne, Inc.
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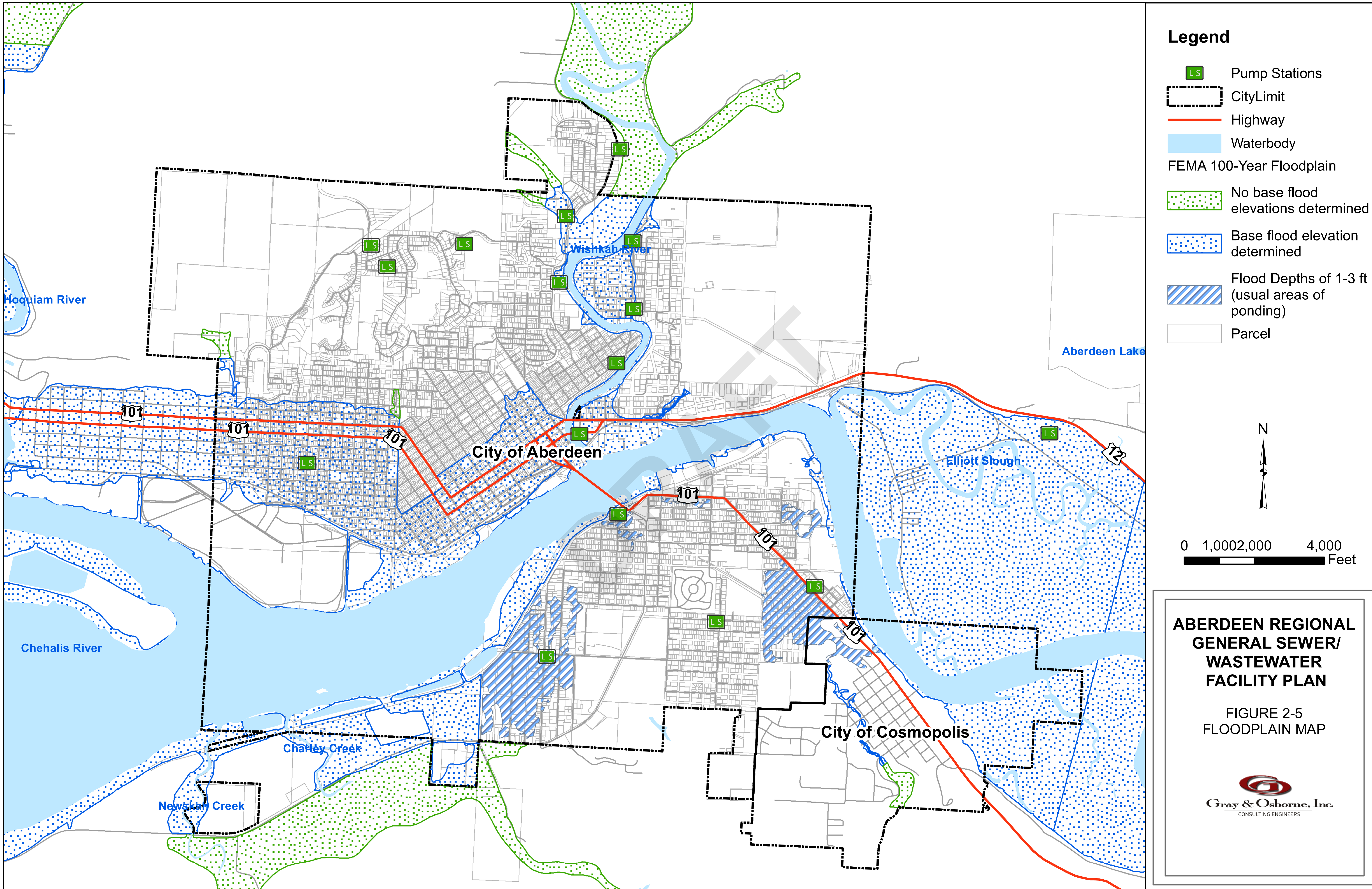
Wishkah River
Hoquiam River
Aberdeen Lake
Elliott Slough
City of Aberdeen
Chehalis River
Charley Creek
Newskah Creek
City of Cosmopolis
101
12



ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 2-5
FLOODPLAIN MAP

Water Act and the Washington State Groundwater Management Program as baseline information sources for regulatory actions involving aquifer recharge area.

Geologically Hazardous Areas

Seismic hazard areas are those with low-density soils that are more likely to experience greater damage due to seismic-induced subsidence, liquefaction, or landslides. Seismic hazard areas are regulated mainly with respect to public safety and with the exception of a severe earthquake, these hazard areas do not impact wastewater facilities. United States is divided into seismic hazard zones based upon historic documents. These zones range from Category 1 to 4, with 4 representing the highest risk. Western Washington falls into Seismic Zone 3. Erosion and landslide hazard areas are regulated under Sections 14.100.400 through 14.100.460 of the City's municipal code. The geologically hazard areas for Aberdeen are shown in Figure 2-6.

Fish and Wildlife Habitat Conservation Areas

Sensitive fish and wildlife habitat is defined as areas which meet the definition of a "Fish and Wildlife Habitat Conservative Area" pursuant to Municipal Code 14.100.500 and are essential for maintaining specifically listed species in suitable habitats. Buffers shall be established for activities adjacent to as necessary to protect the integrity, functions and values of the resource.

WATER SYSTEM

The City of Aberdeen obtains its municipal water supply from the Wishkah River. Surface water is impounded by the Malinowski Dam, diverted to the Water Treatment Plant (WTP), and then routed to the distribution system (see Figure 2-7).

At the WTP, the raw water is filtered by eight membrane microfiltration modules, chlorinated using chlorine gas, held in a clearwell to obtain adequate disinfection contact time, and then treated with caustic soda to control alkalinity and pH. Fluoride is applied at the outlet of the treatment plant. The City built settling ponds to discard backwash water offsite. Backwash water is conveyed into two sequential settling basins that overflow into an infiltration pond. Any water that does not infiltrate will overflow into a small creek adjacent to the property. The City acquired an NPDES permit for this discharge. Twice per year, sludge from the settling basins is pumped into a separate drying basin located onsite. The finished water is then either returned to the transmission main or pumped to the Wishkah tank for distribution to customers along Wishkah River Road. There are two interties with the City of Hoquiam for emergency use. The total system production was 942 million gallons (MG) in 2010. The past 10 years have been stable in production.

Areas outside the City limits currently served water include the City of Cosmopolis and the Stafford Creek Correctional Center, which is approximately 16 percent of the total consumption in the City.

NEARBY WASTEWATER TREATMENT FACILITIES

The City of Hoquiam operates a 4.0-mgd secondary wastewater treatment facility. The Hoquiam facility is on the western margin of the City of Hoquiam adjacent to the airport. Its discharge is to Grays Harbor, approximately 5.2 miles west of the City of Aberdeen's WWTP outfall.

As discussed in Chapter 1, the Hoquiam facility is an extended aeration activated sludge system using an oxidation ditch configuration. The plant includes a headworks, a single secondary clarifier, a gas dechlorination system, a chlorine contact basin, and a 48-acre facultative lagoon for flow equalization and biosolids storage.

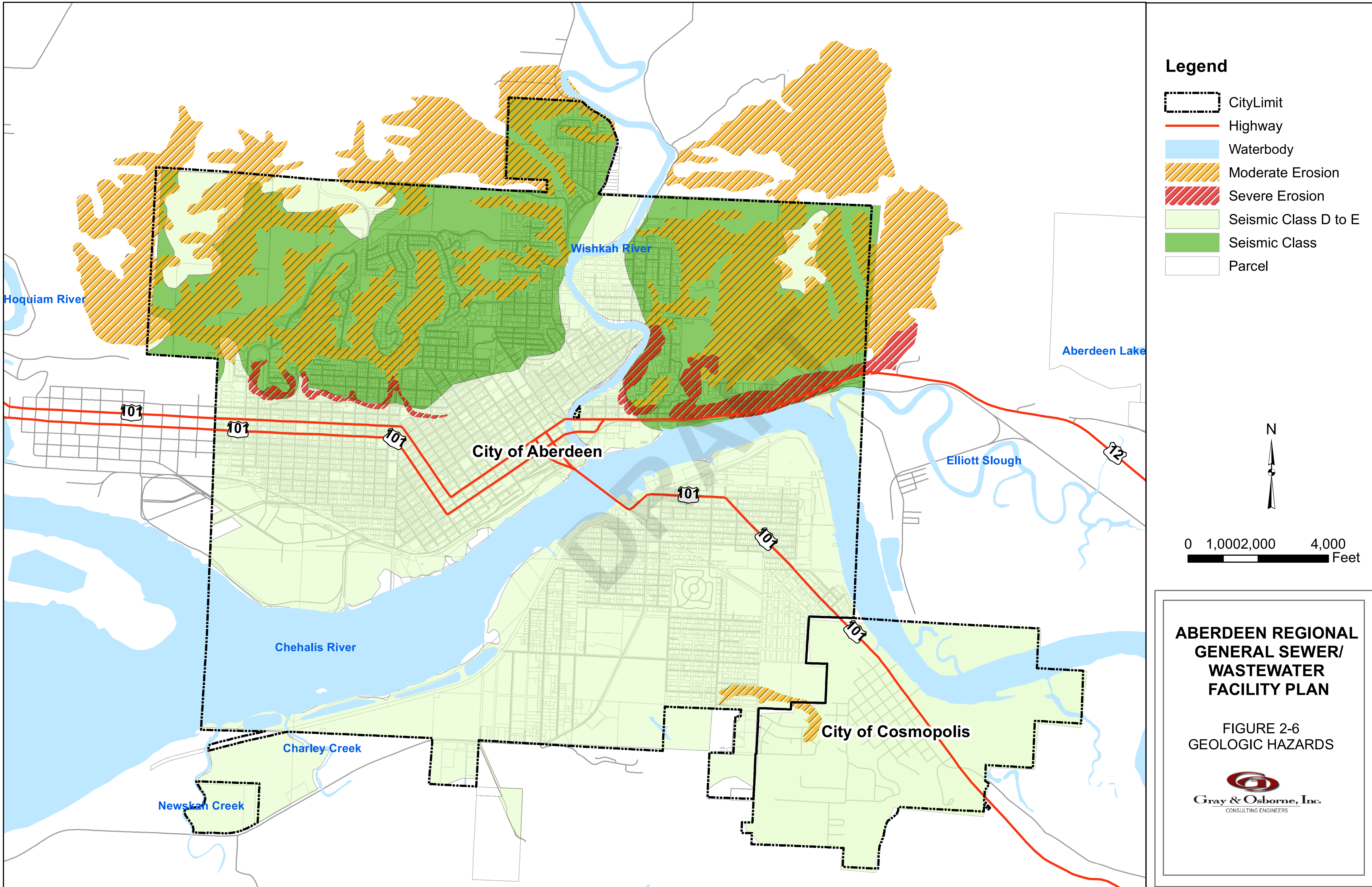
ECONOMIC BASE

Aberdeen and the rest of the Grays Harbor area are relatively dependent on the timber, fishing and tourism industries, and as a regional service center for much of the Olympic Peninsula. Historically the area is dependent on harvesting and exporting natural resources. The Port of Grays Harbor is the largest coastal shipping port north of California. It is still a center for the export of logs on the west coast of the U.S. and has become one of the largest centers for the shipment of autos and grains to Asia.

Major employers in Aberdeen-Hoquiam-Cosmopolis include Sierra Pacific Industries, Grays Harbor Community Hospital, Hoquiam Plywood, Pasha Automotive, Willis Enterprises, Cosmo Specialty Fibers, Ocean Protein Companies, and the Stafford Creek Corrections Center, a state prison that opened in 2000.

Other significant employers include the cranberry-growing cooperative Ocean Spray, worldwide retailer Walmart and Little Hoquiam Shipyard.

In 2007, Imperium Renewables of Seattle invested $40 million in the construction of the biodiesel plant at the Port of Grays Harbor. REG Grays Harbor purchased the biodiesel refinery in 2015. The plant produces 100 million US gallons (380,000 cubic meters) of biodiesel fuel from plants and vegetable material annually. The biodiesel refinery is the second largest capacity biodiesel refinery in the United States. Although the plant is located in the City of Hoquiam, it discharges wastewater to the City of Aberdeen Wastewater Facilities.



ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 2-6
GEOLOGIC HAZARDS

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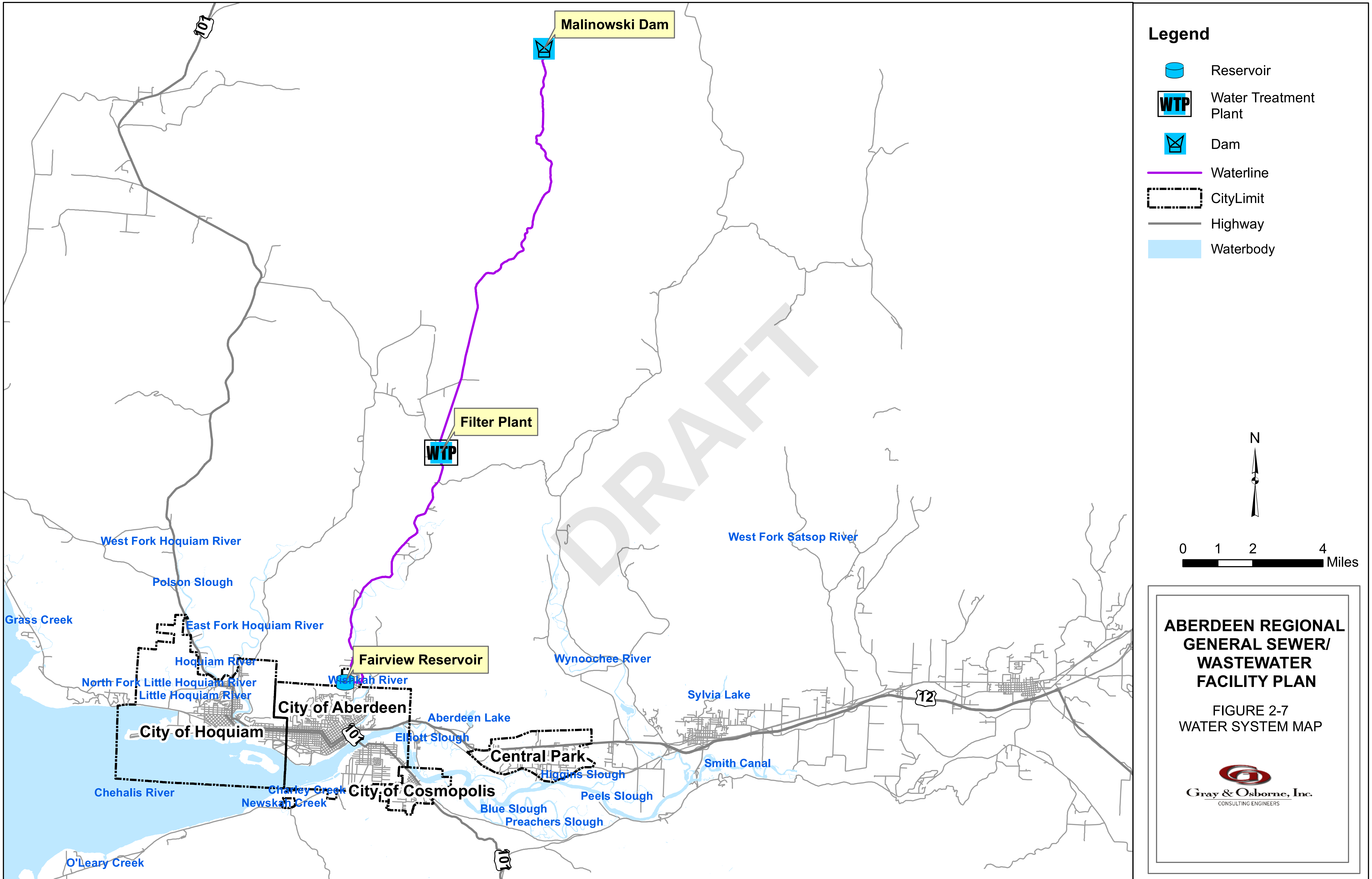


ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 2-7
WATER SYSTEM MAP

The *2001 Comprehensive Plan* noted that the City has been changing from its traditional timber and fishing industrial to more diversified economic progress.

> *The City of Aberdeen, over the last two decades, has been subject to unplanned economic restructuring created by an erosion of the underlying economic base of timber processing and commercial fishing. Land use issues, as a result, became intertwined with economic issues. The economy needed to diversify, with an emphasis placed during this transition period on the retention of existing businesses, relocating existing businesses into the area, and encouraging the start-up of new business. The economy will continue to transition from resource-based activities to those of a regional service and retail provider. Aberdeen's needs for various land uses are substantial, and result in significant changes that reflect this transition period.*

The City is currently in the early stages of developing an update to its Comprehensive Plan.

PLANNING PERIOD

In order to provide wastewater services for future growth, the wastewater system is in need of continuous evaluation and improvement. A planning period for the evaluation of the wastewater utility should be long enough to be useful for an extended period of time, but not so long as to be impractical. The planning period for this *General Sewer/Wastewater Facility Plan* is from 2018 through 2038, coinciding with a 20-year planning interval. In addition, the City's wastewater management needs beyond 20 years (up to 50 years) will be considered.

LAND USE AND ZONING

Existing Land Use

Land uses in the City include residential, commercial, industrial, forest and park, etc. Table 2-2 shows a summary of existing land use in the City. Industrial uses are located along the shorelines of Grays Harbor and the Chehalis and Wishkah Rivers. The central business district is on the north side of the Chehalis River. Residential areas are northwest, east and south of the business district. Residential land use makes up about 20 percent of the City of Aberdeen's total land area. Over 90 percent of this land is occupied by single-family residential units.

TABLE 2-2

City of Aberdeen Current Land Use

Land Use Designation	Acres	Percent
Single-Family	1,124.4	18.4%
Multi-Family	97.9	1.6%
Commercial	618.6	10.1%
Industrial	63.3	1.0%
Forest and Parks	1,336.2	21.9%
Undeveloped Land	2,381.9	39.0%
Public Facilities	382.1	6.3%
School	104.9	1.7%
Total	**6,109.2**	**100.0%**

SOURCE: City of Aberdeen, *2001 Comprehensive Plan*.

Future Land Use

The City's *2001 Comprehensive Plan* indicated the downtown and waterfront areas of Aberdeen have undergone many dramatic changes in recent years. There are several possible future residential development areas in the City, particularly in the area northwest of the City.

Two objectives of the City's Land Use Element, as it relates to accommodating future growth, are restated below:

- *Encourage and provide for growth in economic activity and population while maintaining a balanced and orderly pattern of development and protecting the desirable attributes of the City and its environs.*

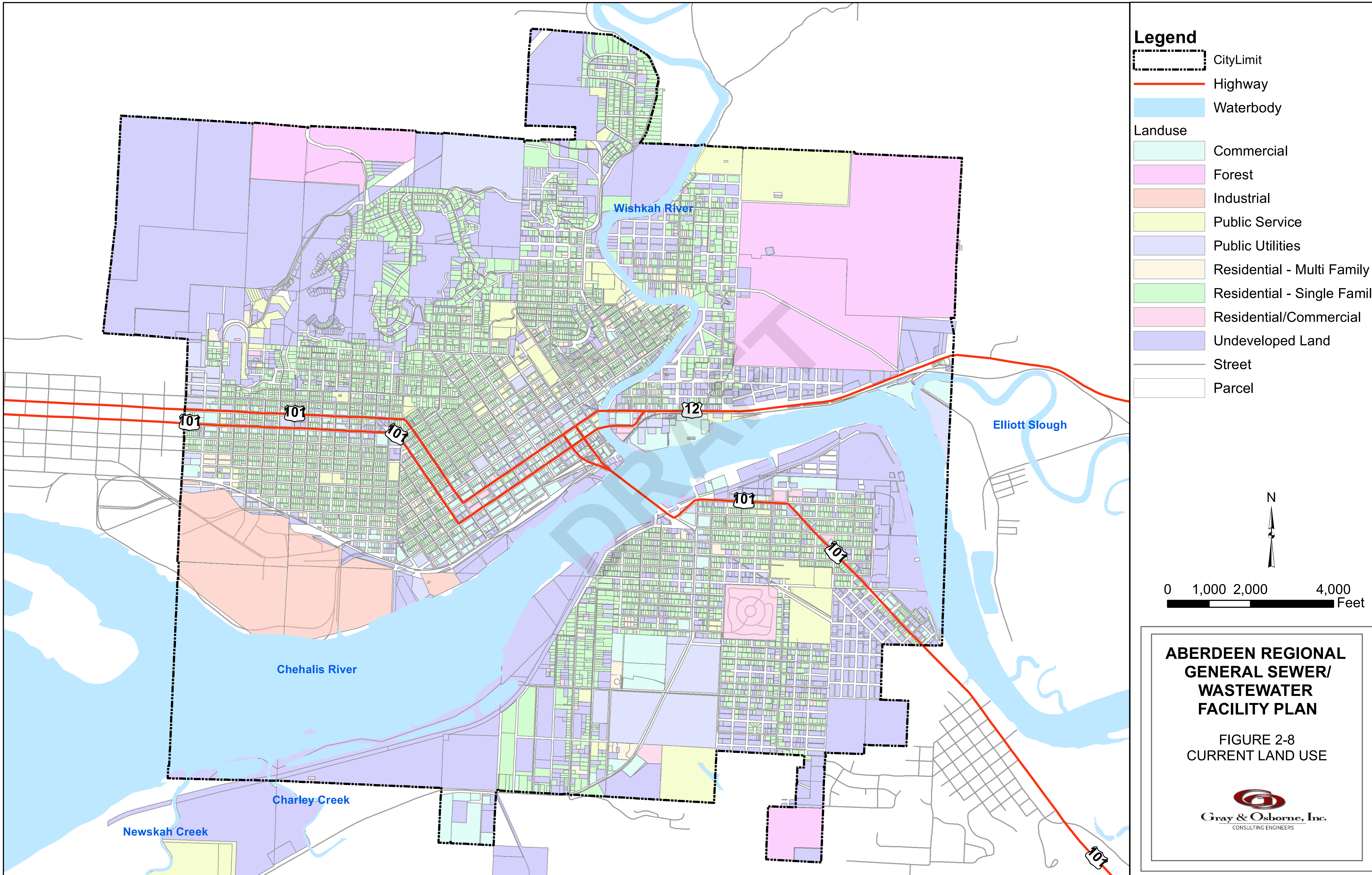
- *Maximize the opportunities provided by waterways and terrain*

Figure 2-8 shows current land use within Aberdeen and Figure 2-9 shows future land use.

ADJACENT JURISDICTIONS

City of Hoquiam

The City of Hoquiam lies within Grays Harbor County and borders Aberdeen to the west. The City of Hoquiam was incorporated in 1890 and has a 2018 population of 8,560. The economic history of Hoquiam has been dominated by the forestry and fishing industries, and the current economics are incorporating sustainable practices and environmentally friendly industries.



ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 2-8
CURRENT LAND USE

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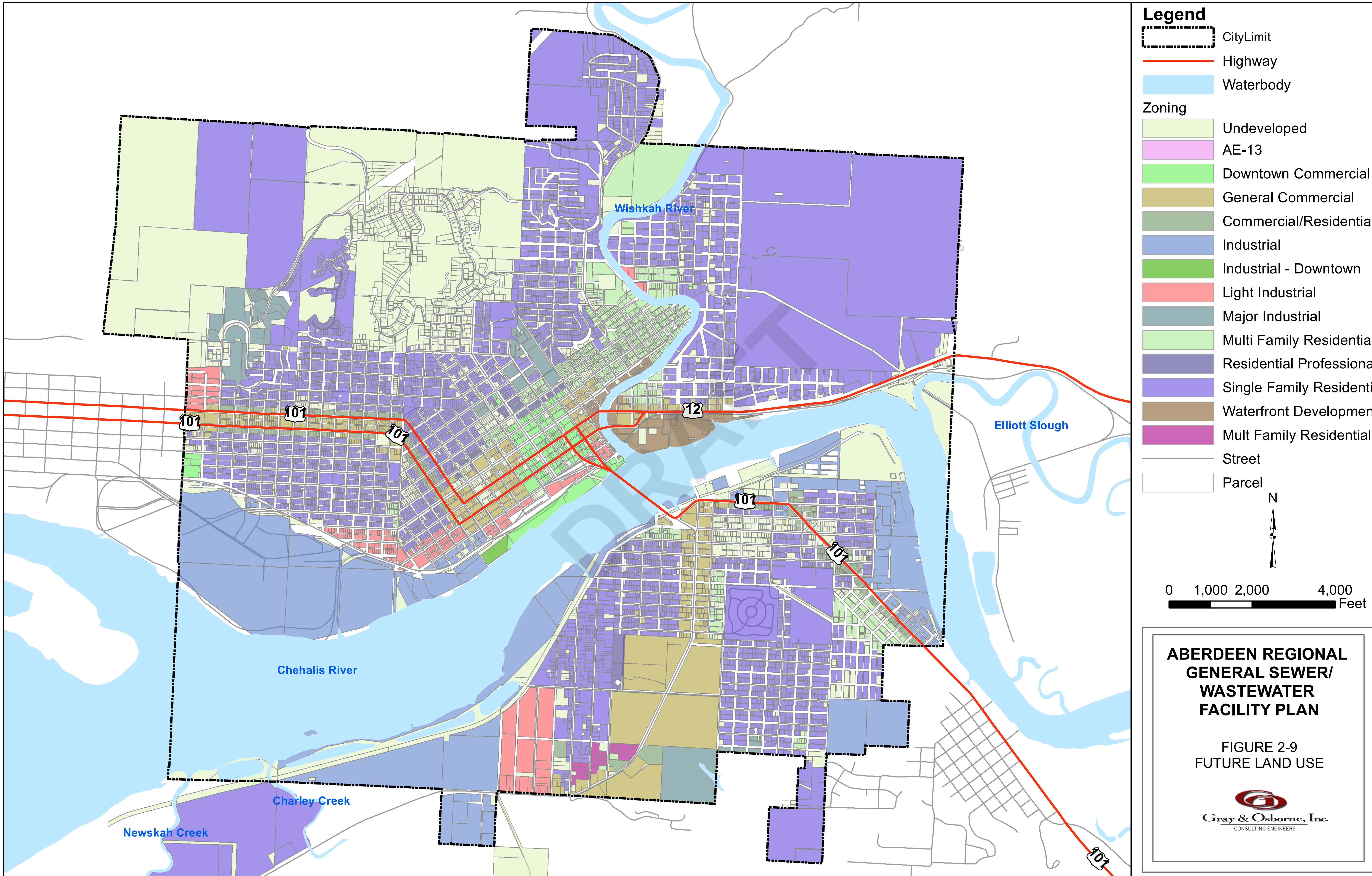


ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 2-9
FUTURE LAND USE

Like Aberdeen, Hoquiam is in a prime position for economic development to occur. The waterfront is an ideal location for mixed industrial and commercial use, and has the potential to be a tourist attraction. The City boundaries are extensive and could easily accommodate an increase in housing demand. The low cost of industrial land and quick access to open ocean, make Grays Harbor attractive to freight-based and industrial development.

The City of Hoquiam provides wastewater collection and treatment service for residential, commercial, and industrial customers within city limits. The sewer system is owned and operated by the City of Hoquiam. Wastewater is collected in gravity piping and conveyed through a series of pump stations and force mains to the wastewater treatment plant (WWTP) located in the southwest area of the City near the Chehalis River and Grays Harbor. The majority of the current collection system and the existing WWTP were constructed in the 1980s.

Hoquiam wastewater flow is taken into account as part of the future regional flow projections in this study.

City of Cosmopolis

The City of Cosmopolis, incorporated in 1891, borders Aberdeen to the southeast. In the 1960s, 1970s, and in the late 1990s and early years of the new century, Cosmopolis developed residential areas on the hills. The population has not risen considerably over the last 20 years. Cosmopolis has a 2018 population of 1,665.

According to the City's Comprehensive Development Plan, the level of growth and development in the City is, in large part, the result of the regional economic base, and planning for economic development should utilize a regional perspective as the costs and benefits of economic growth go beyond jurisdictional boundaries.

In March 2013, City of Aberdeen and Cosmopolis signed a municipal wastewater treatment contract by which Aberdeen agrees to provide wastewater treatment and accept a maximum of 98.3 mgd of annual wastewater until year 2023. Sewage from Cosmopolis is pumped into a force main which discharges into the 24-inch sewer on Decatur Street in South Aberdeen. Cosmopolis is responsible for the operation and maintenance of the wastewater collection system within its service area.

Stafford Creek Correction Center

The Stafford Creek Correctional Center (SCCC) is located about 6.4 miles southwest of the border of City of Aberdeen. The SCCC, owned and operated by the State Department of Corrections (DOC), is another wholesale customer of the City of Aberdeen. SCCC began service in the year 2000. SCCC staff reported that the prison reached its full capacity of 1972 in March 2004. Since then, the population has varied, but remains close to its full capacity. SCCC has a 2018 population of 2,150, including full capacity of 1,972 inmates and a population-equivalent of 178 employees.

The SCCC Pump Station, installed in 2000, transfers sewage from SCCC to the Aberdeen collection system. The sewage is pumped to a gravity line south of the Chehalis River that conveys the sewage to Pump Station 2, which pumps the sewage across the Chehalis River into a line that discharges into the State Street interceptor. The SCCC system also includes a 1.66 mgd aerated equalization storage tank and screening system, which is used to reduce the impact of peak flow which would otherwise be experienced at the treatment plant when peak flow exceeds 13 mgd.

Central Park Community

The community of Central Park is located within Grays Harbor County about 4 miles southeast of the Aberdeen city limits. Central Park has a 2018 population of 2,667. There is currently no sewer service provided in that area, with most of the area using septic systems.

Similar to the City of Hoquiam, the possibility of treating Central Park wastewater is evaluated later in the Plan.

POPULATION

The Washington State Office of Financial Management (OFM) provided a history of population for Aberdeen over a 14-year period, 2005 to 2018, as shown in Table 2-3. The City's population remained relatively stable in the past years. In addition, the populations of the City of Cosmopolis and SCCC have remained relatively unchanged.

TABLE 2-3

Historical Population Data (2005-2018)

Year	Population(1)	Annual Growth Rate
2005	16,450	--
2006	16,470	0.12%
2007	16,450	-0.12%
2008	16,460	0.06%
2009	16,440	-0.12%
2010	16,450	0.06%
2011	16,870	2.55%
2012	16,890	0.12%
2013	16,860	-0.18%
2014	16,850	-0.06%
2015	16,780	-0.42%
2016	16,780	0.00%
2017	16,740	-0.24%
2018	16,760	0.12%
Average	**16,661**	**0.15%**

SOURCE: Washington State Office of Financial Management (OFM).

CHAPTER 3

REGULATORY REQUIREMENTS

Federal and state regulatory requirements were used in developing the design criteria for improvements to the wastewater collection, treatment, and disposal facilities for Aberdeen. The purpose of this chapter is to identify and summarize the regulations that affect the planning, design, and approval of improvements discussed in this plan.

This chapter does not describe each regulation in detail; rather, it addresses important facets of the regulations that affect the planning and design process. Subsequent sections of this report address technical requirements of the regulations at a level of detail appropriate for the evaluation provided by that section. For instance, Chapters 6, 7, and 8 contain more detailed information regarding wastewater collection and treatment system and biosolids management regulations.

FEDERAL AND STATE STATUTES, REGULATIONS, AND PERMITS

This section discusses some of the various federal and state laws that may affect wastewater system construction and operations, as well as other relevant permits, programs, and regulations.

FEDERAL CLEAN WATER ACT

The Federal Water Pollution Control Act is the principal law regulating the water quality of the nation's waterways. Originally enacted in 1948, it was significantly revised in 1972 and 1977, when it was given the common title of the "Clean Water Act" (CWA). The CWA has been amended several times since 1977. The 1987 amendments replaced the Construction Grants program with the Water Pollution Control State Revolving Fund (SRF) that provides low-cost financing for a range of water quality infrastructure projects.

Effluent Discharge Requirements

The National Pollutant Discharge Elimination System (NPDES) program was established by Section 402 of the CWA and its subsequent amendments. The Department of Ecology administers NPDES permits for the U.S. Environmental Protection Agency (EPA). Most NPDES permits have a 5-year term and place limits on the quantity and quality of pollutants that may be discharged to water bodies.

The State of Washington administers the federal effluent limitations through the NPDES program. All wastewater discharges into the waters of the state must be permitted through the Department of Ecology with an NPDES permit. The current Aberdeen

Wastewater Treatment Plant (WWTP) NPDES Permit WA0037192 and fact sheet are attached as Appendix B. The City's current NPDES permit effluent limitations are summarized in Table 3-1. The permit was issued in 2018, and will expire in 2023.

TABLE 3-1

Summary of Aberdeen WWTP NPDES Permit Limits

Parameter	Average Monthly	Average Weekly
Biochemical Oxygen Demand (5-day) (BOD5)	30 milligrams/liter (mg/L) 1,110 pounds/day (lbs/day) 85% removal of influent BOD5	45 mg/L 1,665 lbs/day
Total Suspended Solids (TSS)	30 mg/L 1,335 lbs/day 85% removal of influent TSS	45 mg/L 2,003 lbs/day
Total Ammonia (as N)	The Permittee must operate the facility to minimize Ammonia in the discharge	
Parameter	**Minimum**	**Maximum**
pH	6.0 standard units	9.0 standard units
Parameter	**Monthly Geometric Mean**	**Weekly Geometric Mean**
Fecal Coliform Bacteria	200/100 milliliter (mL)	400/100 mL
Parameter	**Average Monthly**	**Maximum Daily**
Total Residual Chlorine	0.08 mg/L	0.17 mg/L

The permit identifies the following limits for influent flow and load:

- Maximum month flow – 9.9 mgd
- Maximum month BOD loading – 7,400 lbs/day
- Maximum month TSS loading – 8,900 lbs/day

More information about water-quality permitting is provided in the Surface Water Quality Standards discussion later in this chapter.

Industrial Pretreatment/Source Control

Section 307 of the CWA established the National Pretreatment Program; 40 CFR Part 403 lists the federal pretreatment requirements. This program is designed to protect publicly owned treatment works (POTW) from pass-through of pollutants or interference with the treatment process from industrial or other non-residential discharges that is not "domestic-equivalent" (similar in quality to domestic wastewater).

If considered significant, industrial discharges to municipal wastewater collection/treatment systems are typically addressed in State Waste Discharge Permits (SWDPs). There are six current SWDPs issued to facilities in the Aberdeen region, as shown in Table 3-2.

TABLE 3-2

State Waste Discharge Permits Issued

Facility Name	Permit Number	Permit Type
Harold Lemay Enterprises, Inc	ST0006158	Industrial (IU) to POTW/Private SWDP IP
Pasha Automotive Services	ST0006238	Industrial (IU) to POTW/Private SWDP IP
Port of Grays Harbor	ST0006170	Industrial (IU) to POTW/Private SWDP IP
Weyerhaeuser Coastal Timberlands	ST0006257	Industrial (IU) to POTW/Private SWDP IP
Imperium Grays Harbor Biodiesel Pro	ST0006214	Industrial (IU) to POTW/Private SWDP IP
Ocean Protein LLC	ST0006231	Industrial (IU) to POTW/Private SWDP IP

Total Maximum Daily Loads

The CWA requires states to establish (Total Maximum Daily Load) TMDL programs for parameters not meeting applicable surface water quality standards as identified on Section 303(d) water quality impaired lists. A TMDL specifies the maximum amount of a pollutant that a waterbody can receive and still meet the water quality standards. A TMDL also identifies the sum of allowable loads of a single pollutant from all point and nonpoint sources, and determines a margin of safety to ensure protection of the waterbody in case there are unknown pollutant sources or unforeseen events that may impair water quality. There is currently a TMDL Implementation Plan in effect on the lower Chehalis River for controlling fecal coliform.

FEDERAL AND STATE STANDARDS FOR USE OR DISPOSAL OF SLUDGE

The City currently accepts waste sludge from several WWTPs, treats it to Class B standards in the City's anaerobic digestion system, and hauls to third party Beneficial Use Facilities (BUFs, currently Olympic Ag, LLC). An evaluation of alternatives for the City's future biosolids treatment and management is provided in a later chapter.

The generation and use of biosolids, and the disposition of solid waste in general generated from wastewater treatment plants (WWTPs), is subject to both federal and state regulations. The following information is provided to guide the City in its biosolids management efforts.

Federal Basis of Regulations

Based on the 1977 and 1987 amendments to the Clean Water Act, the U.S. Environmental Protection Agency (EPA) established requirements for the final use and disposal of municipal sewage sludge, published in 1993 under 40 CFR 503. These regulations identify three methods for legal disposal or final use of sewage sludge: surface disposal, land application, and incineration. For each of the three methods of disposition, EPA has identified pollutant limits, operational standards, management practices, monitoring, and recordkeeping and reporting requirements. Under the 503 regulations, the EPA placed considerable emphasis on the beneficial use of sludge through a properly managed land application program.

Washington State Regulations

Washington State regulates biosolids under Chapter 70.95J of the RCW. Washington does not have fully delegated authority from the EPA, but has the authority to issue separate state permits for biosolids management. Chapter 70.95J recognizes biosolids as a valuable commodity, and specifies implementation of a program that maximizes beneficial use. The state requirements are found in Chapter 173-308 of the Washington Administrative Code (WAC). The state program meets federal minimum requirements and has added requirements including, but not limited to, the following:

- Biosolids must not contain a significant amount of manufactured inerts (e.g., plastics, debris). Typically, and in Aberdeen's case, this requirement is met by screening the wastewater at the municipality's treatment plant.
- Some of the federal Class A Alternatives are not allowed under state regulations.
- For all practical purposes, the state rule does not allow biosolids to be disposed of (e.g., landfill) on a long-term basis.
- Biosolids generators and all entities managing biosolids must obtain a state permit and pay permit fees.
- The state rule has certain exemptions for research.

Implementation at State Level

In 1998, the State of Washington promulgated WAC 173-308 "Biosolids Management" governing the use and disposal of sewage sludge. Most of the requirements in the federal regulations pertaining to pollutant limits, pathogen reduction, vector attraction reduction, operational standards, and management practice are in essentially the same form within the state regulation. The state regulation requires that any facility generating municipal sewage sludge or material derived from municipal sewage sludge obtain clearance under the State General Permit for Biosolids Management.

Requirements for Land Application

There are three fundamental elements in the federal and state biosolids management regulations that establish minimum criteria for land application of biosolids:

1. Pollutant Concentrations and Application Rates
2. Pathogen Reduction Measures
3. Vector Attraction Reduction Measures

<u>Pollutant Concentrations</u>

Maximum allowable concentrations for nine heavy metals are listed in Table 3-3. If a biosolids sample exceeds the ceiling concentration of any of the nine heavy metals, it cannot be land applied. A lower pollutant threshold concentration is required for Exceptional Quality (EQ) biosolids, as shown in Table 3-3. If biosolids are shown to be within these concentrations, they may be eligible for relatively unrestricted land application, providing they meet the Class A biosolids requirements and vector attraction reduction requirements given below. As shown in Table 3-3, the City's biosolids are well below the biosolids threshold concentrations for all nine metals.

TABLE 3-3

Allowable Biosolids Trace Pollutant Concentrations for Land Application(1)

Element	Symbol	Ceiling Concentration (mg/kg)(1)	EQ Limit (mg/kg)(2)	City of Aberdeen 2016, 4th Quarter	City of Aberdeen 2017, 4th Quarter
Arsenic	As	75	41	< 13	4.11
Cadmium	Cd	85	39	< 4.4	2.03
Copper	Cu	4,300	1,500	500	450
Lead	Pb	840	300	54	40.2
Mercury	Hg	57	17	0.78	0.737
Molybdenum	Mo	75	(3)	6.9	7.94
Nickel	Ni	420	420	22	18.5
Selenium	Se	100	100	< 22	8.2
Zinc	Zn	7,500	2,800	1,200	1,190

(1) WAC-173-308 Table 1.
(2) WAC-173-308 Table 3.
(3) Under review by EPA. Until the EPA completes its review, the effective limit is 75 mg/kg.

Cumulative and annual trace pollutant loading rates are designated for nine heavy metals (Table 3-4). Once a cumulative loading limit is reached for a particular limiting pollutant, the land may no longer receive biosolids containing any level of the limiting pollutant. EQ biosolids are not subject to cumulative loading limits. Assuming that the pollutant concentrations in the City's biosolids are consistent with the concentrations reported in Table 3-4, the cumulative loading limits will not be a concern for the City's land application sites.

TABLE 3-4

Biosolids Pollutant Loading Limits for Land Application[1]

Element	Symbol	Cumulative Loading Limit (lb/ac)	Annual Loading Limit (lb/ac)
Arsenic	As	37	1.8
Cadmium	Cd	35	1.7
Copper	Cu	1,340	67
Lead	Pb	268	13
Mercury	Hg	15	0.76
Molybdenum	Mo	[2]	[2]
Nickel	Ni	375	19
Selenium	Se	89	4.5
Zinc	Zn	2,500	125

(1) 40 CFR Part 503.13 Tables 2 and 4.
(2) Under review by EPA.

Pathogen Reduction Measures

In order for biosolids to be land applied, they must meet specific criteria demonstrating a minimum level of treatment to reduce the density or limit the growth of pathogenic bacteria. By meeting these minimum criteria, a biosolids sample is referred to as meeting Class B pathogen reduction requirements. Class B biosolids must meet one or more of the criteria listed in both Tables 3-5 and 3-6.

A higher level of treatment, known as a process to further reduce pathogens (PFRP), will permit biosolids to meet Class A pathogen reduction requirements. Tables 3-7 and 3-8 provide the pathogen reduction standards for Class A biosolids. Table 3-9 lists the EPA-approved PFRPs. When biosolids meet the Class A standard, they may be eligible for relatively unrestricted land application, provided they meet the EQ trace pollutant limits described above and the vector attraction reduction requirements as described below.

TABLE 3-5

Pathogen Reduction Requirements – Class B Biosolids

Alternative 1	Fecal coliform are less than 2,000,000 most probable number (MPN) or 2,000,000 colony-forming units per gram of total solids. Seven samples are collected at each sampling event. Geometric means are used to determine compliance.
Alternative 2	Use a process to significantly reduce pathogens (PSRP); see Table 3-6.
Alternative 3	Use a process equivalent to a PSRP.

TABLE 3-6

Processes to Significantly Reduce Pathogens

Aerobic Digestion	Biosolids are agitated with air or oxygen to maintain aerobic conditions for a specific time and at a specific temperature, ranging from 40 days at 20 degrees C to 60 days at 15 degrees C.
Air Drying	Biosolids are dried on sand beds or on paved or unpaved basins. The biosolids dry for at least 3 months. During 2 of the 3 months, the ambient average daily temperature is above 0 degrees C.
Anaerobic Digestion	Biosolids are treated in the absence of air for a specific time and at a specific temperature, ranging between 15 days at 35 to 55 degrees C and 60 days at 20 degrees C.
Composting	Using the within-vessel, static aerated pile, or windrow composting methods, the temperature of the biosolids is raised to 40 degrees C or higher and remains at 40 degrees C or higher for 5 days. For 4 hours during the 5 days, the temperature in the compost pile exceeds 55 degrees C.
Lime Stabilization	Enough lime is added to the biosolids to raise the pH to 12 after 2 hours of contact.

TABLE 3-7

Pathogen Reduction Requirements – Class A Biosolids

All Alternatives	Fecal coliform <1,000 MPN per gram total solids, or salmonella <3 MPN per 4 grams total solids.
Alternative 1	Meet specified time/temperature requirements (see Table 3-8).
Alternative 2	Maintain pH above 12 for 72 hours, with temperature during the 72-hour period >52°C for 12 hours. After 72 hours at pH above 12, biosolids are air dried to >50 percent total solids.
Alternative 3	Procedure for documenting that a biosolids treatment process meets Class A standards. Viable helminth ova <1 viable helminth ova per 4 grams total solids and enteric viruses <1 plaque-forming unit per 4 grams total solids. Retesting required when biosolids meet these requirements before the pathogen treatment process. When the treatment process is shown to reduce helminths and viruses and the pathogen treatment conditions are documented, the biosolids are Class A when the documented conditions are used.
Alternative 4	Procedure for documenting that a biosolids product meets Class A standards. Viable helminth ova <1 viable helminth ova per 4 grams total solids and enteric viruses <1 plaque-forming unit per 4 grams total solids.
Alternative 5	Use an approved PFRP, see Table 3-9.
Alternative 6	Use process approved as equivalent to an approved PFRP.

TABLE 3-8

Time and Temperature Requirements – Class A Biosolids

Temperature (°C)	≥7% Solids			<7% Solids		
	Days	Hours	Minutes	Days	Hours	Minutes
50	14			5		
52	7			3		
54	4			2		
56	2			1		
58		24			10	
60		13			5	
62		7			3	
64		4			2	
66		2				41
68			57			30
70			30			30
Above 70			20			30

Note: The table applies to all pathogen reduction processes except when the percent solids of the biosolids are 7 percent or higher and small particles are heated by warmed gases or an immiscible liquid.

TABLE 3-9

Processes to Further Reduce Pathogens(1)

Composting	Using either the within-vessel composting method or the static aerated pile composting method, the temperature of the biosolids is maintained at 55 degrees C or higher for 3 days. Using the windrow composting method, the temperature of the biosolids is maintained at 55 degrees C or higher for 15 days or longer. During the period when the compost is maintained at 55 degrees C or higher, there shall be at least five turnings of the windrow.
Heat Drying	Biosolids are dried by direct or indirect contact with hot gases to reduce the moisture content to 10 percent or lower. Either the temperature of the biosolids particles exceeds 80 degrees C or the wet bulb temperature of the gas in contact with the biosolids as it leaves the dryer exceeds 80 degrees C.
Heat Treatment	Liquid biosolids are heated to a temperature of 180 degrees C or higher for 30 minutes.
Thermophilic Aerobic Digestion	Liquid biosolids are agitated with air or oxygen to maintain aerobic conditions, maintaining 55 to 60 degrees C for 10 days.
Beta Ray Irradiation	Biosolids are irradiated with beta rays from an accelerator at dosages of at least 1.0 megarad at room temperature (approximately 20 degrees C).
Gamma Ray Irradiation	Biosolids are irradiated with gamma rays from certain isotopes, such as Cobalt 60 and Cesium 137, at room temperature (approximately 20 degrees C).
Pasteurization	The temperature of the biosolids is maintained at 70 degrees C or higher for 30 minutes or longer.

(1) Biosolids stabilized to these standards meet Class A pathogen reduction requirements if the end product has:
- Fecal coliform <100 MPN per gram total solids; or
- Salmonella <3 MPN per 4 grams total solids.

<u>Vector Attraction Reduction Measures</u>

The third minimum requirement for biosolids to be land applied is the vector attraction requirement. This measure is designed to make the biosolids less attractive to disease-carrying pests such as rodents and insects. These measures typically reduce the liquid content and/or volatile solids content of the biosolids or make the biosolids relatively inaccessible to vector contact by soil injection or tilling. A total of ten vector attraction reduction alternatives are available for land-applied municipal sewage (see Table 3-10).

If biosolids meet the lower pollutant threshold limits (EQ limits), Class A pathogen reduction requirements, and vector attraction reduction requirements, they are eligible for relatively unrestricted application. Biosolids of this type are referred to as "Exceptional Quality." If biosolids meet the higher pollutant threshold limits, Class B pathogen reduction requirements, and vector attraction reduction requirements, they can then be land applied but are subject to a number of restrictions regarding public contact and ultimate crop use.

TABLE 3-10

Vector Attraction Reduction Alternatives

No.	Description
1.	Biosolids digestion process with >38 percent volatile solids reduction.
2.	Test end product of an aerobic digestion process: 40-day anaerobic test at 30 to 37 degrees C. Acceptable stabilization if <15 percent volatile solids reduction occurs during the test.
3.	Test end product of aerobic digestion process having <2 percent solids: 30-day aerobic test at 20 degrees C. Acceptable stabilization if <15 percent volatile solids reduction occurs during the test.
4.	Facilities with aerobic digestion. Specific oxygen uptake rate (SOUR) test using end product of digestion process. Acceptable stabilization if uptake is <1.5 mg oxygen per total solids per hour at 20 degrees C.
5.	Facilities with aerobic digestion. Time/temperature requirement: 14 days, residence time at digestion temperatures >40 degrees C with average digestion temperature >45 degrees C.
6.	High pH stabilization: biosolids pH >12 for 2 hours and >11.5 for 24 hours.
7.	Treatment by drying. Not to include unstabilized primary wastewater solids. Total solids content >75 percent before mixing with other material.
8.	Treatment by drying. Can include unstabilized primary wastewater solids. Total solids >90 percent before mixing with other materials.
9.	Land application process. Injection into soil. No biosolids on soil surface 1 hour after application (Class B) and septage for 8 hours after application (Class A).
10.	Land application process. Soil incorporation by tillage within 8 hours, Class A biosolids only. Soil incorporation by tillage within 6 hours of application for Class B biosolids and septage.
11.	Sludge monofills only – does not apply to biosolids/septage.
12.	High pH treatment before land application. Acceptable stabilization if pH is >12 for 30 minutes.

(1) When septage has not been previously treated in any process other than a septic system.

Land Application Limitations

For Class B biosolids, waiting periods are required to allow time for pathogens to die off before harvest. For Class B biosolids, the following minimum waiting periods apply:

- Minimum of 30 days for a food crop between biosolids application and harvest.
- Minimum of 14 months between biosolids application and harvest if the biosolids contact the harvested portion of the food crop.
- Minimum of 20 to 38 months between biosolids application and harvest for root crops.

It may not be feasible to raise some food crops (e.g., root crops and low-growing fruits and vegetables) on sites that use Class B biosolids because the waiting period is more than one growing season.

Permitting

WAC-173-308-310 lists permitting requirements for municipalities managing biosolids. The primary permit required for biosolids management activities is *the State General Permit for Biosolids Management*. Treatment works treating domestic sewage that apply for coverage under this permit must submit either a complete permit application, or a notice of intent which is followed at a later date by complete permit information. The contents of a complete permit application are described in WAC 173-308-310(5), and in summary include the following:

- A statement of the applicable activity(ies) for which coverage under the permit is sought.
- The name of the general permit (Biosolids Management).
- Basic facility information including name, name of contacts, location, and relevant jurisdictions.
- Information on other environment permits.
- Maps showing the location of the facility.
- Biosolids data, including pollutant and nitrogen concentrations, and data from existing land application sites.

- A basic description of the applicant's biosolids management practice.
- Information regarding the specific vector attraction reduction and pathogen reduction methods employed.
- Land application plans, as required.
- Information on past, current, and future biosolids production and use.
- Other information the applicant deems helpful or that is required by the department.
- Proof of public notice, as required under proposed WAC 173-308-310(11)(a)(v). Substantiation of public notice is required for the initial application for coverage under the general permit as well as for subsequent site-specific land application plans submitted for approval.

The permittee must carry out public notice as required under WAC 173-308-310(11), and public hearings if required, in accordance with WAC 173-308-310(12), and comply with requirements of the State Environmental Policy Act (SEPA) as stipulated under WAC 173-308-310(030).

Provisional *coverage* under the general permit is effective on receipt of a complete permit application or notice of intent. Provisional coverage allows a permit holder to continue existing practices in compliance with the basic requirements of the rule and permit. Formal coverage is obtained after review and approval of the permit application, including any plans submitted with the application, by Ecology. Review of specific sites proposed at a later date may lead to additional conditions in site-specific land application plans, which become fully enforceable elements of a facility's permit coverage on approval by the department.

Provisional *approval* can be granted under WAC 173-308-310(17). Provisional approval is essentially permission to carry on an existing practice or to engage in a new or altered practice if certain conditions are met. Facilities operating under provisional approval have standing under the permit but are subject to further review and approval at a later time. They must comply with all applicable standards of the rule and permit, including timely submittal of an application or notice of intent. They must comply with requirements of the local health department, and may not obtain provisional approval if Ecology objects. They are not accountable under provisional approval, however, for compliance with additional or more stringent requirements that may eventually be imposed after final review. Provisional approval for new operations or for significant changes to existing operations operates similar to that for existing operations, except that public notice must be carried out and there must be no sustainable objections to a proposal.

Monitoring

Producers of biosolids are required to monitor for pollutant concentrations, pathogen reduction, or vector attraction reduction. The required monitoring frequencies depend on the quantity of biosolids produced. These rates are summarized in Table 3-11. Based on its rate of biosolids production, the City has a minimum monitoring frequency of quarterly.

TABLE 3-11

Minimum Frequency of Monitoring

Annual Biosolids Production (dry tons)	Frequency
Greater than zero but less than 320	Once per year
Equal to or greater than 320 but less than 1,653	Once per quarter
Equal to or greater than 1,653 but less than 16,535	Once per 60 days
Equal to or greater than 16,535	Once per month

In WAC 173-308, municipalities, such as the City, are defined as being responsible for the treatment, transport, use, and disposal of the biosolids produced under its management. Therefore, in addition to monitoring biosolids quality, the City is responsible for the biosolids it produces from the point of production to the point of land application. The Department of Ecology recommends that in addition to meeting the minimum monitoring requirements for biosolids quality, biosolids producers should periodically monitor the storage, transport, and land application of their biosolids to ensure that each step conforms to State regulations, regardless of whether these activities are being contracted to a third party.

Compliance with the State Environmental Policy Act

Treatment works treating domestic sewage that come under this permit must also comply with requirements of the State Environmental Policy Act (SEPA) per WAC 173-308-030. Generally, compliance involves completing an environmental checklist to be reviewed by the lead SEPA agency, which makes a threshold determination of environmental impacts and carries out a public notice of the determination. Potential outcomes are a Determination of Nonsignificance (DNS), Mitigated Determination of Nonsignificance, or Determination of Significance. The latter leads to preparation of an environmental impact statement (EIS). If an EIS must be prepared, approval for the activity in question cannot be obtained under this permit until the EIS is completed. It is expected that most biosolids related proposals will not result in significant adverse environmental impacts, and in most cases a DNS will probably be issued (this has been the bulk of past experience). Mitigation may be appropriate in some cases, but alternatively can probably be addressed as a condition of permit coverage or approval of a general or site-specific land application plan.

When the proponent is a governmental agency (e.g., a municipality operating a wastewater treatment plant) it is expected that lead agency status will fall to the proponent agency in accordance with WAC 197-11-926.

Public Notice

The Department of Ecology requires public notice as a part of the process of issuing a general permit. Public notice requirements for facilities subject to this permit vary depending on the purpose the notice is serving and the quality of biosolids being managed. When a facility applies for initial coverage under the general permit it must carry out public notice for that purpose as specified in WAC 173-308-310(11). Notification must be made to the general public, affected local health departments, and interested parties. Generally, publication in a newspaper is required for initial public notice. Notification of affected local health jurisdictions and interested parties is by direct mail. When biosolids that do not meet the most stringent standards of the rule will be applied to the land, posting of sites is also required. Some facilities may add new sites in accordance with an approved general land application plan after they have received initial approval of coverage under the general permit. If public notice has not been previously carried out for those new sites, it must be done before biosolids can be applied. For sites added at a later date, required notice is limited to posting of the site, notification to Ecology and/or the local health department, and persons on an interested party list maintained by the permit holder. Public notice may also be necessary if a hearing or meeting is required under WAC 173-308-310(12), and to comply with requirements of the State Environmental Policy Act under Chapter 197-11 WAC.

Landfill Disposal of Biosolids

Ecology recognizes that at times circumstances may require that sewage sludge be disposed of in a landfill. Disposal in a monofill, what the federal program calls “placing” of sewage sludge, will remain under the jurisdiction of the state solid waste program and the separate federal sewage sludge program. This permit provides for disposal of sewage sludge in a municipal solid waste landfill as a management option on an emergency, temporary, or long-term basis as defined in WAC 173-308-080 and implemented in WAC 173-308-300. Uses of biosolids as a component of final or intermediate covers, where vegetation will be established, is considered a beneficial use. Use of sewage sludge in daily cover is considered disposal, the same as disposal directly in the landfill cell.

A need to dispose on an emergency basis is generally expected to occur as a result of circumstances largely beyond the control of an operator, and is defined as having duration of less than 1 year. Disposal on an emergency basis is automatically approved under this permit if certain conditions are met. Disposal as a temporary management option may occur for reasons similar to those for an emergency basis, but is expected to require at least one but not more than 5 years to resolve. In these cases, an approved plan is required to demonstrate that disposal is not being sought as a long-term management option. When disposal is contemplated as a management option with no intent to pursue other alternatives, or for a period of more than 5 years, it is considered to be a long-term management option. This option will only be approved if a facility can demonstrate that other management options are economically infeasible. It is important to note that the demonstration must be one of infeasibility, and not simply greater expense.

Sewage sludge that is disposed of in a municipal solid waste landfill must pass a free liquids test – the "paint filter test" – and not be hazardous waste in accordance with WAC 173-308-300(4) and (5). This approach is also consistent with regulations for municipal solid waste landfill management found in WAC 173-351-200(9) and 220(10), and also the requirements of 40 CFR Part 258 for municipal solid waste landfills. Part 503.4 and WAC 173-308-300(3) also require that any landfill receiving sewage sludge be in compliance with the requirements of Part 258.

Record Keeping and Reporting

The general permit implements requirements for record keeping and reporting in accordance with proposed WAC 173-308-290 and –295. Permit holders must keep records of the information used to develop applications for coverage under this permit, and must also keep records, including signed certification statements, regarding on-going biosolids management practices. Annual reports are required of all permit holders. In accordance with requirements of federal rules, annual reports from the larger, what are sometimes called "major" facilities, are required to be more comprehensive. The record-keeping requirement allows for periodic inspection and verification of a facility's performance. The annual reporting function also supports verification of facility practices and allows the collection of information necessary to efficient management of the overall state biosolids program.

Fees

The permit fee system multiplies a basic cost per residential equivalent (the rate) times the number of residential equivalents (the base). WAC 173-308-320 indicates five basic rates for coverage under this permit, dependent on the biosolids management options chosen.

Site Selection Criteria for Land Application

Land application is a commonly employed alternative for the ultimate disposition of biosolids and septage. Once all criteria have been met for pathogen reduction and vector attraction reduction (and additionally for biosolids only, pollutant concentrations), the next step is to select a site suitable for biosolids or septage application.

A biosolids application site must meet certain minimum criteria to meet specific regulatory requirements as well as minimum functional standards. This section will be divided between site criteria that are specifically dictated by regulation and those criteria that are based on agronomic science.

Regulatory Criteria for Siting

The WAC-173-308 and EPA 503 regulations have specific requirements for siting a biosolids application site. There may also be local land use regulations or policies that apply in specific areas of the county. This section deals primarily with those requirements found in the federal 503 and state WAC-173-308 regulations.

Endangered Species Habitat – Biosolids may not be applied to the land if it is likely to enter a wetland area or adversely affect an endangered species or its critical habitat.

Surface Waters Proximity – Biosolids may not be applied within 100 meters of any well or surface water body, including wetlands.

Pathogen Reduction Factor - Unless biosolids meet Class "A" pathogen reduction requirements, biosolids shall only be applied to sites where public access can be restricted. Land immediately adjacent to residential areas, well-traveled roads, parks and recreation areas would not be desirable application sites for anything but Class "A" biosolids.

Recommended Buffers for Biosolids Application Sites

<u>Property Lines and Roads</u>

The *Biosolids Management Guidelines for Washington State* (published by Ecology, July 2000) recommend minimum property buffers for biosolids application sites as shown in Table 3-12.

TABLE 3-12

Recommended Property Buffers for Application Sites for Biosolids and Domestic Septage

Landmark	Distance (ft)
Property Line	5 – 50
Dwelling	50 – 200
Major Arterial or Highway	50 – 100
Minor Road (Dirt or Gravel)	5 – 50

These property buffers do not distinguish between the type of pathogen reduction classification (A or B) under which the biosolids are regulated. For Class "B" biosolids use of the more conservative buffer distance is the recommended goal. Local land use regulations or policies, on a site-specific basis, may require larger buffer areas.

Drinking Water Wells

The *Biosolids Management Guidelines for Washington State* recommend a distance of 2 feet between the top layer of soil and the water table and recommend a 100 to 200 feet. setback distance between biosolids application sites and drinking water wells.

TABLE 3-13

Recommended Property Buffers for Wastewater Land Treatment and Application Sites

Wastewater Land Treatment Sites	
Disinfected Wastewater	500 ft
Non-Disinfected Wastewater	1,000 ft
Wastewater Land Application Sites	
Class "A" Reclaimed Water	50 ft
Class "B" and "C" Reclaimed Water	100 ft
Class "D" Reclaimed Water	300 ft

A wastewater land treatment site is somewhat analogous to a site where Class "B" biosolids are applied, while a land application site is somewhat analogous to a Class "A" biosolid application site. The analogy lies in the role of the soil-crop system. With Class "B" biosolids, just as with a wastewater land treatment system, the soil-crop system is used to provide further treatment. With Class "A" biosolids, as with wastewater land application systems, the land is not required to provide additional treatment to reduce the potential hazard of the waste.

For initial planning purposes the wastewater setback distances shown above may be considered in developing preliminary estimates of distances between biosolids application sites and potable water wells.

Siting Based on Agronomic Criteria

The following criteria are taken from the *Biosolids Management Guidelines* and the *Managing Nitrogen from Biosolids* manual for Washington State. They are intended to provide guidance for site selection based on those characteristics of a site that make it suitable for sustaining a cover crop. Because a primary concern in land application of septage is prevention of leaching of nitrate to groundwater, a key parameter in determining the agronomic rate for land application is the available nitrogen content in the septage. Maintaining a cover crop is absolutely essential for a biosolids or domestic septage application program to be successful. For site-specific cases, it is usually appropriate to consult with a professional soil scientist or agronomist to verify proper application rates or if unique circumstances exist which are not addressed by these general guidelines.

Topography

Land used for biosolids or domestic septage application should generally be well drained, but not excessively. Drainage characteristics are related to soil type, depth to restrictive layer as well as slope. Generally, for agricultural sites, slopes up to 3 percent will be suitable for biosolids or domestic septage application if the depth to the restrictive layer is not too shallow, e.g., less than 20 inches. For slopes between 3 percent and 8 percent, soils should have a deep mantle and be low in silt and clay. Slopes greater than 8 percent are generally not recommended for biosolids or domestic septage applications because of the potential for erosion and runoff.

For land applications in existing forests, sites with steeper slopes may be used. For application in the dry season, the maximum recommended slope is 30 percent for application on a site with good vegetative cover, and 15 percent on a site with poor vegetative cover. For application in the wet season, the maximum recommended slope is 15 percent for application on a site with good vegetative cover, and 8 percent on a site with poor vegetative cover.

Soil Depth

The depth of the soil mantle is important for sustaining a cover crop. Deeper soil depths can retain greater quantities of water, support a better root structure and thereby allow crops to survive long dry weather periods.

Soil depth is important with biosolids and domestic septage application because deeper soils can act as a type of "filter" to prevent nutrients from leaching to groundwater. The processes of nitrification and denitrification remove ammonia nitrogen from wastewater. Both processes are assisted by long detention times in the soil matrix. Denitrification also requires an absence of free oxygen to cause soil bacteria to use nitrate for respiration purposes instead of oxygen. Thus the deeper the soil, the better the environment is for denitrification to occur. A deep soil mantle is beneficial in preventing groundwater pollution.

A soil depth greater than 20 inches is desirable for biosolids or domestic septage application. A depth of 40 inches or more is ideal. A soil that is more shallow than 20 inches will have lower crop yields and limit biosolids application rates.

Soil Texture

Soil textures range from fine to coarse. Finely textured soils are more prone to runoff, whereas coarse soils are well drained. Soil texture by itself is not a selection criterion, but must be considered as a factor in site selection. For example, a sandy soil, though well drained, does not have the ability to retain nutrients while a clay soil has a good capacity for nutrient and water retention. Adding and incorporating biosolids or septage in either type of soil (sandy or clay) would likely prove beneficial because the biosolids can improve porosity in the clay soils and nutrient/moisture retention capacity in the sandy soils.

Soil Structure

Soil structure is the arrangement and stability of soil particles. An ideal soil structure has about half solids and half pore spaces. At maximum moisture holding capacity about half the pore space is filled with water.

Soil Color

Color is an indicator of drainage. Well-drained soils have horizons that are uniformly red, brown or black. Poorly drained soils are gray and may contain brown or red colored mottles. Obviously, well-drained soils are preferred for biosolids domestic application. Poorly drained soils are not good application sites and may be an indication that they are wetlands. Soils suspected of being wetlands should be evaluated by a qualified wetlands or soil scientist to verify they are not wetlands prior to any biosolids or domestic septage land application. State biosolids regulations require a minimum 10-foot buffer between wetlands and biosolids or domestic septage application sites.

Crop Selection and Management

Crop selection is a critical element of designing a biosolids or domestic septage application site. Nutrient uptake rates vary by crop species. Certain crops are capable of nutrient uptake in winter months, while others are not.

In general, perennial grasses, legumes and poplar trees have the highest nutrient uptake rates. However, maintaining these high uptake rates requires proper crop management. By frequently cutting at early stages of growth, nutrient uptake rates are maximized. Table 3-14 is provided as a guide for nutrient uptake rates for different crops.

When the temperature drops, plant growth is curtailed. If biosolids or domestic septage are over-applied in the winter months when nitrogen uptake is low, it is possible that runoff or leaching of nitrogen from the application site could occur. To prevent this from occurring it is necessary to create a plan for biosolids application that ensures that nitrogen loadings match uptake rates for a given period.

Whatever the crop selected, if it is not properly managed the crop will not provide the nutrient uptake targeted in the design of the biosolids or domestic septage application site. This means that the crop must be supplied with proper ratios of all critical nutrients, including phosphorous and potassium, as well as water sufficient to meet crop water requirements. Regular harvesting of crops is needed to maintain the growth process whereby nitrogen is assimilated into the plant biomass. Without including all of these factors in the design and management of a biosolids or domestic septage application program, it is not possible to assume that a given crop will provide the predicted nitrogen uptake rate.

TABLE 3-14

Estimated Nutrient Uptake Rates for Selected Crops (lb/acre*yr)

Crop	Nitrogen	Phosphorus	Potassium
Forage Crops			
Alfalfa	200-480	20-30	155-200
brome grass	116-200	35-50	220
coastal bermuda grass	350-600	30-40	200
Kentucky bluegrass	180-240	40	180
quack grass	210-250	27-41	245
reed canary grass	300-400	36-40	280
rye grass	180-250	55-75	240-290
sweet clover	158	16	90
tall fescue	135-290	26	267
orchard grass	230-250	20-50	225-315
Field Crops			
Barley	63	15	20
Corn	155-172	17-25	96
Cotton	66-100	12	34
grain sorghum	120	14	62
Potatoes	205	20	220-288
Soybeans	94-128	11-18	29-48
Wheat	50-81	15	18-40
Forests			
Hybrid poplar	270-360	--	--
Douglas fir plantation	135-220	--	--

Climate

Climate may be a limiting factor for calculating biosolids application rates. Winter biosolids application is typically impacted by:

- Lower agronomic uptake rates;
- Poor conditions for vehicle traffic on the application site;
- Potential for runoff due to freezing ground.

Each of these factors must be considered in choosing a site for biosolids application if year round use of the site is required. In recent years, Ecology in western Washington has increasingly curtailed winter land application of biosolids unless appliers can demonstrate that the crops/forest take up all the available nitrogen; crop nitrogen uptake is typically at a minimum in the winter.

PROPOSED CAPACITY, MANAGEMENT, OPERATION AND MAINTENANCE REGULATIONS

EPA has proposed a new round of regulations titled Capacity, Management Operation and Maintenance (CMOM). Though the regulations are yet to be formally adopted by EPA, some municipalities are anticipating the adoption and have moved forward with implementation. CMOM focuses on the failure of collection systems and requires a program for long-term financing and repair. Under its authority granted by the federal Clean Water Act, EPA seeks to address sanitary sewer overflows (SSO) under the CMOM program. It is expected that elements of CMOM could be incorporated into NPDES permits.

In general, the CMOM requirements can be summarized in the following elements:

1. General performance standards including system maps, information management, and odor control.

2. Program documentation including the goals, organizational and legal authority of the organization operating the collection system.

3. An overflow response plan that requires response in less than 1 hour and is demonstrated to have sufficient and adequate personnel and equipment, etc. Estimated volumes and duration of overflows must be accurately measured and reported to the regulatory agency.

4. System evaluation requires that the entire system be cleaned on a scheduled basis (for example, once every 5 years), be regularly TV inspected, and that a program for short- and long-term rehabilitation replacement be generated. EPA has proposed, as a rule of thumb, a 1.5 to 2 percent system replacement rate which implies that an entire collection system is replaced somewhere in the range of a 50- to 70-year time period.

5. A capacity assurance plan that will use flow meters to model I&I, ensure lift stations are properly operated and maintained, and that source control is maintained.

6. A self-audit program to evaluate and adjust performance.

7. A communication program to communicate problems, costs, and improvements to the public and decision-makers.

EPA is considering some changes in design standards for collection systems including requiring that sanitary sewer overflows not occur except in extreme storms. They have also decided that they will not predefine the type of storm, leaving that decision to the design engineer.

FEDERAL ENDANGERED SPECIES ACT

Waters of the Chehalis River Basin and Grays Harbor Estuary support a variety of fish and wildlife species, including eight that are currently listed as Threatened or Endangered under the Federal Endangered Species Act.

ESA listings impact activities that affect salmon and trout habitat, such as water uses, land use, construction activities, and wastewater disposal. Impacts to the City may include longer timelines for permit applications and more stringent regulation of construction impacts on in-water work and riparian corridors. The presence of ESA-listed species and associated critical habitat in the vicinity has the potential to impact future WWTP and outfall improvement projects.

NATIONAL ENVIRONMENTAL POLICY ACT

The National Environmental Policy Act (NEPA) was established in 1969 and requires federal agencies to determine environmental impacts on all projects requiring federal permits or funding. Federally delegated activities such as NPDES permits or Section 401 certification are considered state actions and do not require NEPA compliance. If a project involves federal action (through, for example, an Army Corps of Engineers Section 404 permit), and is determined to be environmentally insignificant, a Finding of No Significant Impact (FONSI) is issued; otherwise, an Environmental Assessment (EA) or Environmental Impact Statement (EIS) would be required. NEPA is not applicable to projects that do not include a federal component or nexus. If there is a federal nexus, the City will need to follow NEPA procedures in order to obtain any permits required for upgrades to the WWTP, which are outlined in the Capital Improvement Plan of this document.

When both federal and state licenses or permits are required, then both NEPA and SEPA requirements must be met. WAC 197-11-610 allows the use of NEPA documents to meet SEPA requirements.

FEDERAL CLEAN AIR ACT

The Federal Clean Air Act requires all wastewater facilities to plan to meet the air quality limitations of the region. Aberdeen falls in the jurisdiction of the Olympic Region Clean Air Agency (ORCAA). ORCAA is responsible for enforcing federal, state and local outdoor air quality standards and regulations in Clallam, Grays Harbor, Jefferson, Mason, Pacific and Thurston counties of Washington State.

FOOD AND DRUG ADMINISTRATION – POTENTIAL FUTURE PATHOGEN/DISINFECTION STANDARDS

The Aberdeen WWTP discharges into Grays Harbor, a major shellfish growing region. Per the requirements of the Food and Drug Administration's National Shellfish Sanitation Program (NSSP), a shellfish growing area must meet established bacterial water quality classification standards verified through routine monitoring. The City meets these standards; however, it is possible the standards will change in the near future. A growing body of research indicates that the occurrence of human enteric viruses (including Norovirus, which is thought to be a leading cause of shellfish-related illnesses associated with human fecal contamination), does not correlate well with traditional indicators of fecal pollution, including the fecal coliform limits that are the current standards for Aberdeen. Currently, only bacterial indicators are monitored to assess the sanitary quality of WWTP effluent discharged under the NPDES program for discharge permits. In addition, research demonstrates that most wastewater treatment and disinfection operations do not reduce human enteric viral pathogens as effectively as they reduce bacterial pathogens and bacterial indicator organisms. Research has shown that coliphages (viruses that infect bacteria) may be more reliable than bacterial indicators for monitoring enterovirus load in the wastewater and treatment removal process. EPA and FDA staff have proposed the use of coliphage as a replacement or a supplemental pathogen indicator and basis for effluent permits for shellfish-bearing waters (Montazeri, Goettert, et al, 2015).

WETLANDS

Dredging and Filling Activities in Natural Wetlands (Section 404 of the Federal Water Pollution Control Act)

A U.S. Army Corps of Engineers permit is required when locating a structure, excavating, or discharging dredged or fill material in waters of the United States or transporting dredged material for the purpose of dumping it into ocean waters. Typical projects requiring these permits include the construction and maintenance of piers, wharves, dolphins, breakwaters, bulkheads, jetties, mooring buoys, and boat ramps. If wetland fill activities cannot be avoided, the negative impacts can be mitigated by creating new wetland habitat in upland areas. If other federal agencies agree, the Corps would generally issue a permit.

Wetlands Executive Order 11990

This order directs federal agencies to minimize degradation of wetlands and enhance and protect the natural and beneficial values of wetlands. This order could affect the siting of lift stations and sewer lines.

STATE STATUTES, REGULATIONS, AND PERMITS

STATE WATER POLLUTION CONTROL ACT

The intent of the State Water Pollution Control Act is to "maintain the highest possible control standards to ensure the purity of all waters of the state consistent with public health and the enjoyment the propagation and protection of wildlife, birds, game, fish and other aquatic life, and the industrial development of the state." Under the Revised Code of Washington (RCW) 90.48 and the Washington Administrative Code (WAC) 173-240, Ecology issues permits for wastewater treatment facilities and land application of wastewater under WAC 246-271.

Submission of Plans and Reports for Construction of Wastewater Facilities, WAC 173-240

Prior to construction or modification of domestic wastewater facilities, engineering reports, plans, and specifications must be submitted to and approved by Ecology. This regulation outlines procedures and requirements for the development of an engineering report that thoroughly examines the engineering and administrative aspects of a domestic wastewater facility project. This State regulation defines a facility plan as an engineering report under federal regulations, 40 CFR Part 35.

Key provisions of WAC 173-240 are provided below:

- An engineering report for a wastewater facility project must contain everything required for a general sewer plan unless an up-to-date general sewer plan is on file with Ecology.
- An engineering report shall be sufficiently complete so that plans and specifications can be developed from it without substantial changes.
- A wastewater facility engineering report must be prepared under the supervision of a professional engineer.

Criteria for Sewage Works Design, Washington State Department of Ecology

Ecology has published design criteria for collection systems and wastewater treatment plants. While these criteria are not legally binding, their use is strongly encouraged by Ecology since the criteria are used by the agency to review engineering reports for upgrading wastewater treatment systems. Commonly referred to as the "Orange Book," these design criteria primarily emphasize unit processes through secondary treatment, and also include criteria for planning and design of wastewater collection systems. Any expansion or modification of the City's collection system and/or WWTP will require conformance with Ecology criteria unless the City demonstrates that alternate standards provide similar reliability and efficacy.

Ecology Reliability Requirements

The Orange Book also presents guidelines for the wastewater treatment component design. Including the number of units required for operation during peak flows. These requirements are derived from federal standards developed by the EPA and published in a 1974 document entitled Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability. Table 3-15 presents Ecology criteria for designation of WWTPs into three reliability classes based on the nature or their receiving water. Per the NPDES Permit and fact sheet, the Aberdeen WWTP has a reliability classification of Class II. Reliability criteria for WWTP in Class II are presented in Table 3-16.

TABLE 3-15

Reliability Classifications from the Orange Book

Reliability Class	Guideline
I	These are works whose discharge or potential discharge: (1) is into public water supply, shellfish, or primary contact recreation waters; or (2) as a result of its volume and/or character, could permanently or unacceptably damage or affect the receiving waters or public health if normal operations were interrupted. Examples of Reliability Class I works are those with a discharge or potential discharge near drinking water intakes, into shellfish waters, near areas used for water contact sports, or in dense residential areas.
II	These are works whose discharge, or potential discharge, as a result of its volume and/or character, would not permanently or unacceptably damage or affect the receiving waters or public health during periods of short-term operations interruptions, but could be damaging if continued interruption of normal operations were to occur (on the order of several days). Examples of a Reliability Class II works are works with a discharge or potential discharge moderately distant from shellfish areas, drinking water intakes, areas used for water contact sports, and residential areas.
III	These are works not otherwise classified as Reliability Class I or Class II.

Source: The Orange Book (Ecology, 2008), Paragraph G2-8.

TABLE 3-16

Reliability Requirements for Class II WWTPs

WWTP Component	Class II Requirements
Mechanically Cleaned Bar Screens	A backup bar screen, designed for mechanical or manual cleaning, shall be provided. Facilities with only two bar screens shall have at least one bar screen designed to permit manual cleaning.
Pumps	A backup pump shall be provided for each set of pumps performing the same function. The capacity of the pumps shall be such that, with any one pump out of service, the remaining pumps will have the capacity to handle the peak flow
Comminution Facility	If comminution of the total wastewater flow is provided, an overflow bypass with a manually-installed or mechanically-cleaned bar screen shall be provided. The hydraulic capacity of the comminutor overflow bypass should be sufficient to pass the peak flow with all comminution units out of service.
Primary Sedimentation Basins	The units shall be sufficient in number and size so that, with the largest-flow-capacity unit out of service, the remaining units shall have a design flow capacity of at least 50 percent of the design basin flow.
Final Sedimentation Basins and Trickling Filters	The units shall be sufficient in number and size so that, with the largest-flow-capacity unit out of service, the remaining units shall have a design flow capacity of at least 50 percent of the design basin flow.
Activated Sludge Process Components.	1. Aeration Basin. A backup basin will not be required; however, at least two equal-volume basins shall be provided. (For the purpose of this criterion, the two zones of a contact stabilization process are considered as only one basin.) 2. Aeration Blowers or Mechanical Aerators. There shall be a sufficient number of blowers or mechanical aerators to enable the design oxygen transfer to be maintained with the largest-capacity-unit out of service. It is permissible for the backup unit to be an uninstalled unit, provided that the installed units can be easily removed and replaced. However, at least two units shall be installed. 3. Air Diffusers. The air diffusion system for each aeration basin shall be designed so that the largest section of diffusers can be isolated without measurably impairing the oxygen transfer capability of the system.

TABLE 3-16 – (continued)

Reliability Requirements for Class II WWTPs

WWTP Component	Class II Requirements
Disinfectant Contact Basins	The units shall be sufficient in number and size so that, with the largest-flow-capacity unit out of service, the remaining units shall have a design flow capacity of at least 50 percent of the total design flow.
Electrical Power Supply	Sufficient to operate all vital components and critical lighting and ventilation during peak wastewater flow conditions. Except that the vital components used to support the secondary processes (i.e., mechanical aerators or aeration basin air compressors) need not be operable to full levels of treatment, but shall be sufficient to maintain the biota.

Source: The Orange Book (Ecology, 2008), Paragraph G2-9 and G2-10.

Certification of Operators of Wastewater Treatment Plants, WAC 173-230

Wastewater treatment plant operators are certified by the State Water and Wastewater Operators Certification Board. The operator assigned overall responsibility for operation of a wastewater treatment plant is defined by WAC 173-230 as the "operator in responsible charge." As noted in the NPDES Permit, "this permitted facility must be operated by an operator certified by the state of Washington for at least a Class III plant. This operator must be in responsible charge of the day-to-day operation of the wastewater treatment plant. An operator certified for at least a Class II plant must be in charge during all regularly scheduled shifts."

SURFACE WATER QUALITY STANDARDS (WAC 173-201A)

The Washington State surface water quality standards (Chapter 173-201A WAC) are designed to protect existing water quality and preserve the beneficial uses of Washington's surface waters. Waste discharge permits must include conditions that ensure the discharge will meet the surface water quality standards (WAC 173-201A-510). Water quality-based effluent limits may be based on an individual waste load allocation or on a waste load allocation developed during a basin wide total maximum daily load study (TMDL).

The State adopted revised water quality standards in August 2016. The standards are based on two objectives: protection of public health and enjoyment, and protection of fish, shellfish, and wildlife. For each surface water body in the State, the standards assign specific uses, such as aquatic life, recreation, or water supply. Water quality standards have been developed for each use for parameters such as fecal coliform, dissolved oxygen, temperature, pH, turbidity, and toxic, radioactive, and deleterious substances. The surface water criteria include 29 toxic substances, including ammonia, residual chlorine, several heavy metals, polychlorinated biphenyls (PCBs), and pesticides.

Discharging to surface water requires an NPDES permit issued by Ecology under WAC 173-220. Wastewater treatment plants must generally, at a minimum, meet technology-based limits that include 30 mg/L total suspended solids (TSS) and 30 mg/L 5-day biochemical oxygen demand (BOD_5) (typically termed “30-30 limits”). Additionally, under WAC 173-201A-060, State Water Quality Standards, Ecology is authorized to condition NPDES permits so that the discharge meets water quality standards. Therefore, other permit conditions in addition to or more stringent than the 30-30 limits could be added to ensure that the water quality of the receiving water is not degraded.

It is the policy of the State of Washington to maintain existing beneficial uses of surface water by preventing degradation of existing water quality. However, certain allowances are made by Ecology for discharging treated wastewater into a surface water that enable a temporary or mitigated degradation to occur. These allowances are made by establishing mixing zones and determining the assimilative capacity of the receiving water. Ecology uses modeling to estimate the amount of mixing within the mixing zone. A mixing zone is the defined area in the receiving water surrounding the discharge port(s), where wastewater mixes with the receiving water. Within mixing zones, the pollutant concentrations may exceed water quality numeric standards, so long as the discharge does not interfere with the designated uses of the receiving water body. The pollutant concentrations outside of the mixing zones must meet water quality numeric standards. The Water Quality Standards (WAC 173-201A-400) allow the Washington State Department of Ecology to authorize mixing zones around a point of discharge in establishing surface water quality-based effluent limits. Both “acute” and “chronic” mixing zones may be authorized for pollutants that can have a toxic effect on the aquatic environment near the point of discharge. The concentration of pollutants at the boundary of these mixing zones may not exceed the numerical criteria for that type of zone.

Through modeling, Ecology determines the potential for violating the water quality standards at the edge of the mixing zone and derives any necessary effluent limits. Steady-state models are the most frequently used tools for conducting mixing zone analyses. Ecology determined the dilution factors that occur within these zones at the critical conditions using modeling studies completed in the 2011 Outfall Predesign Report (2011). The dilution factors are listed in Table 3-17. The modeling conducted for the 2017 NPDES Permit showed that no water quality-based permit limits were necessary, so no new permit limits were added; however, the effluent ammonia concentrations were within 20 percent of levels that would trigger a numerical ammonia limit. Similar findings were found in the mixing zone update completed for this *Regional Facility Plan*, discussed in Chapter 7. Thus, it is apparent that the WWTP will need to continue to nitrify.

TABLE 3-17

Mixing Zone Dilution Factors, Aberdeen WWTP

Criteria	Acute	Chronic
Aquatic Life	7.8	36
Human Health, Carcinogen		36
Human Health, Non-carcinogen		36

The State's anti-degradation policy aims to maintain the highest possible quality of water in the State by preventing the deterioration of water bodies that currently have higher quality than the water quality standards require. The revised water quality standards define three tiers of waters in the anti-degradation policy:

- Tier I water bodies are those with violations of water quality standards from natural or human-caused conditions. The focus of water quality management is on maintaining or improving current uses and preventing any further human-caused degradation.

- Tier II water bodies are those of higher quality than required by the water quality standards. The focus of the policy is on preventing degradation of the water quality and to preserve the excellent natural qualities of the water body. New or expanded actions are not allowed to cause a "measurable change" in the water quality unless they are demonstrated to be "necessary and in the overriding public interest."

- Tier III are the highest quality "outstanding resource waters." Tier III(A) prohibits any and all future degradation, or Tier III(B) which allows for de minimis (below measurable amounts) degradation from well-controlled activities.

Per the Fact Sheet of the City's NPDES Permit, the WWTP discharges to the Grays Harbor Estuary at the mouth of the Chehalis River, which is designated as Marine Waters. Based on its designations in WAC-173-201a and the Fact Sheet, the Aberdeen WWTP must meet Tier I requirements:

- Dischargers must maintain and protect existing and designated uses. Ecology must not allow any degradation that will interfere with, or become injurious to, existing or designated uses, except as provided for in chapter 173-201A WAC.

- For waters that do not meet assigned criteria, or protect existing or designated uses, Ecology will take appropriate and definitive steps to bring the water quality back into compliance with the water quality standards.

- Whenever the natural conditions of a water body are of a lower quality than the assigned criteria, the natural conditions constitute the water quality criteria. Where water quality criteria are not met because of natural conditions, human actions are not allowed to further lower the water quality, except where explicitly allowed in Chapter 173-201A WAC.

The applicable criteria noted in the Fact Sheet are summarized in Table 3-18.

TABLE 3-18

Mixing Zone Dilution Factors, Aberdeen WWTP

Parameter	**Value**
Designation	Good Quality
Temperature Criteria – Highest 1D MAX	19 degrees C (66.2 degrees F)
Dissolved Oxygen Criteria – Lowest 1-Day Minimum	5.0 mg/L
Turbidity Criteria	10 NTU over background when the background is 50 NTU or less; or a 20 percent increase in turbidity when the background turbidity is more than 50 NTU.
pH Criteria	pH must be within the range of 7.0 to 8.5 with a human-caused variation within the above range of less than 0.5 units.
Protection of Shellfish Harvesting Criteria	Fecal coliform organism levels must not exceed a geometric mean value of 14 colonies/100 mL, and not have more than 10 percent of all samples (or any single sample when less than ten sample points exist) obtained for calculating the geometric mean value exceeding 43 colonies/100 mL.
Recreational Use Criteria (for Secondary Contact Recreation)	Enterococci organism levels must not exceed geometric mean value of 70 colonies/100 not more than 10 percent of all samples (or single sample when less than ten sample points exist) obtained for calculating the geometric value exceeding 208 colonies/100 mL.

The miscellaneous marine water uses for the receiving water for the Aberdeen WWTP outfall are wildlife habitat, harvesting, commerce and navigation, boating, and aesthetics.

Just upstream of the outfall (although not technically a part of the receiving water) is the fresh-water designated lower Chehalis River, within Water Resource Inventory Area (WRIA 22). The designated uses for the lower Chehalis River include:

- Spawning/Rearing
- Primary Contact for Recreational Uses
- Domestic/Industrial/Agricultural/Stock Water Supply
- Wildlife Habitat
- Harvesting
- Commerce and Navigation
- Boating
- Aesthetics

Additional discussion of the water quality implications of wastewater treatment alternatives is provided in Chapter 7 and 8.

RECLAIMED WATER STANDARDS

Reclaimed water is the effluent derived from a wastewater treatment system that has been adequately and reliably treated, such that it is no longer considered sewage and is suitable for a beneficial use or a controlled use that would not otherwise occur. The legislature has declared that "the utilization of reclaimed water by local communities for domestic, agricultural, industrial, recreational, and fish and wildlife habitat creation and enhancement purposes (including wetland enhancement) will contribute to the peace, health, safety, and welfare of the people of the State of Washington." Consideration of the feasibility of reclaimed water is required in General Sewer Plans, so it is relevant to this *Regional Facility Plan.*

The legislature approved the Reclaimed Water Use Act in 1992 and codified it as chapter 90.46 Revised Code of Washington (RCW). This act initially envisioned treated sanitary wastewater as the source of supply for reclaimed water, and encouraged using reclaimed water for land application and industrial and commercial uses. Legislative amendments to Chapter 90.46 RCW in 2006 required the development of a new Washington Administrative Code (WAC) chapter for reclaimed water. On January 23, 2018, the Department of Ecology adopted a new rule, Chapter 173-219 WAC, Reclaimed Water. The Departments of Ecology and Health cooperatively developed this Rule with significant input from stakeholders and technical advisory groups. The Rule sets forth minimum standards for reclaimed water projects. The agencies may incorporate additional enforceable conditions into a reclaimed water permit issued under the Rule as needed to protect public health and the environment.

The *Water Reclamation and Reuse Standards* define the water quality standards for reclaimed water. The Reclaimed Water Regulations define three classes of reclaimed water: Class A+, Class A, and Class B. The beneficial use of reclaimed water is limited by its classification. Classes of reclaimed water are defined as follows:

> "Class A+ reclaimed water" is the highest quality of reclaimed water and can be used for Class A and Class B uses. Class A can be used for Class A and Class B beneficial uses. Class B water can be used only for Class B beneficial uses. "Class A+ reclaimed water" means a water resource that meets the treatment requirements for Class A reclaimed water and any additional criteria determined necessary on a case-by-case basis by Washington State Department of Health (WDOH) for direct potable reuse. Class A+ reclaimed water is required for direct potable reuse.
>
> "Class A reclaimed water" means a water resource that meets the treatment requirements of this chapter, including, at a minimum, oxidation, coagulation, filtration, and disinfection. Membrane Filtration is acceptable in lieu of coagulation and filtration. Class A reclaimed water may be used for: commercial, industrial, or institutional toilet and urinal flushing, laundry, public water features where public contact may occur; landscape irrigation with direct or indirect public access; irrigation of food crops, trees, and fodder in pastures accessed by milking animals; discharge to Category II wetlands without characteristics provided application rate and supplemental performance standards are met, Category III or IV wetlands, constructed wetlands with public access; direct groundwater recharge; or recovery of reclaimed water stored in an aquifer.
>
> "Class B reclaimed water" means a water resource that meets the treatment requirements of this chapter, including, at a minimum, oxidation, and disinfection. Class B Reclaimed water may be used for: commercial, industrial, and institutional uses with environmental contact or where there is restricted access; landscape irrigation with restricted access and no human contact; frost protection of orchard crops; irrigation of non-food crops, irrigation of orchards, vineyards, process food crops, trees or seed crops in pastures not accessed by milking animals.
>
> The salient performance standards for Class A and Class B reclaimed water are defined in Tables 3-19 and 3-20. Class A+ reclaimed water requirements must be established by jurisdictional health department on a case-by-case basis, and must have approval of the WDOH before reclaimed water can be beneficially used for direct potable reuse.

TABLE 3-19

Minimum Biological Oxidation Performance Standards

Biological Oxidation		
Parameter	**Minimum Biological Oxidation Performance Standard**	
Dissolved Oxygen	Must be measurably present	
Parameter	**Month Average**	**Weekly Average**
BOD_5	30 mg/L	45 mg/L
$CBOD_5$	25 mg/L	40 mg/L
TSS	30 mg/L	45 mg/L
Parameter	**Minimum**	**Maximum**
pH	6 s.u.	9 s.u.
pH (groundwater recharge)	6.5 s.u.	8.5 s.u.

TABLE 3-20

Class A and B Performance Standards

Parameter	Class A Reclaimed Water		Class B Reclaimed Water	
	Monthly Average	**Sample Maximum**	**Monthly Average**	**Sample Maximum**
Coagulation/Filtration				
Turbidity	2 NTU	5 NTU	Not Applicable	Not Applicable
Membrane Filtration				
Turbidity	0.2 NTU	0.5 NTU	Not Applicable	Not Applicable
Disinfection				
Total Coliform	2.2 MPN/100 mL or CFU/100 mL	23 MPN/100 mL or CFU/100 mL	23 MPN/100 mL or CFU/100 mL	240 MPN/100 mL or CFU/100 mL
Virus Removal	See disinfection process standards in WAC 173-219-340		Not Applicable	Not Applicable
Denitrification				
Total Nitrogen	10 mg/L	15 mg/L	Not Applicable	Not Applicable

Note: Numerical values for parameter represent maximum values for monthly average and single sample results.

STATE ENVIRONMENTAL POLICY ACT

WAC 173-240-050 requires a statement in all wastewater comprehensive plans regarding proposed projects in compliance with the State Environmental Policy Act (SEPA), if applicable. The capital improvements proposed in this plan will fall under SEPA regulations. A SEPA checklist is included in Appendix A of this plan for use in the environmental review for the project. In most cases, a Determination of Non-Significance (DNS) is issued; however, if a project will have a probable significant adverse environmental impact, an Environmental Impact Statement (EIS) will be required.

ACCREDITATION OF ENVIRONMENTAL LABORATORIES (WAC 173-050)

The State of Washington established a requirement that all laboratories reporting data to comply with NPDES permits must be generated by an accredited laboratory. This accreditation program establishes specific tasks for quality control and quality assurance (QA/QC) that are intended to ensure the integrity of laboratory procedures. Accreditation requirements must be met for any on-site laboratory or outside laboratory used to analyze samples. Only accredited laboratories may be used for analyses reported for compliance with NPDES permits. In planning for an on-site laboratory, staffing must be sufficient to allow for QA/QC procedures to be performed. The Aberdeen WWTP laboratory is currently accredited for testing the following parameters for TSS, BOD_5, temperature, dissolved oxygen, pH, ammonia, turbidity, and fecal coliform.

MINIMAL STANDARDS FOR SOLID WASTE HANDLING (WAC 173-304)

Grit and screenings are not subject to the sludge regulations in WAC 173-308, but their disposal is regulated under the State solid waste regulations, WAC 173-304. Waste placed in a municipal solid waste landfill must not contain free liquids, nor exhibit any of the criteria of a hazardous waste as defined by WAC 173-303. To be placed in a municipal solid waste landfill, grit, screenings, and incinerator ash must pass the paint filter test. This test determines the amount of free liquids associated within the solids and includes the toxic characteristic leachate procedure (TCLP) test, which determines if the waste has hazardous characteristics.

SHORELINE MANAGEMENT ACT

The Shoreline Management Act of 1971 (RCW 90.58) establishes a broad policy giving preference to shoreline uses that protect water quality and the natural environment, depend on proximity to the water, and preserve or enhance public access to the water. The Shoreline Management Act jurisdiction extends to lakes or reservoirs of 20 acres or greater, streams with a mean annual flow of 20 cubic feet per second (cfs) or greater, marine waters, and any area inland 200 feet from the ordinary high-water mark. Projects are reviewed by local governments according to State guidelines.

The Aberdeen WWTP and portions of the collection systems are located within shoreline areas.

FLOODPLAIN DEVELOPMENT PERMIT

Local governments that participate in the National Flood Insurance Program are required to review projects in a mapped floodplain and impose conditions to reduce potential flood damage from floodwater. A Floodplain Development Permit is required prior to construction, including projects involving wastewater collection facilities.

HYDRAULIC PROJECT APPROVAL

Under the Washington State Hydraulic Code (WAC 220-110), the WDFW requires a hydraulic project approval (HPA) for activities that will "use, divert, obstruct, or change the natural flow or bed" of any waters of the State. For City activities, such as pipeline crossings of streams or WWTP outfall modifications, an HPA will be required. The HPA will include provisions necessary to minimize project-specific and cumulative impacts to fish.

CITY SEWER ORDINANCES AND PLANNING POLICIES

The City has a Municipal Code that regulates sewer services. This chapter of the municipal code has been included in Appendix C. The sewer ordinances address such issues as requirements for connections to sewer system, on-site system requirements, and rates for sewer service. Per Section 13.52.020 of the current code, all structures located on property assessed for sewers shall be required to connect to the City's sewer system where service is available.

REFERENCES

1. Montazeri, Goettert, et al, 2015. *Pathogenic Enteric Viruses and Microbial Indicators during Secondary Treatment of Municipal Wastewater*, Applied and Environmental Microbiology, Vol. 81, No. 18,).

2. City of Aberdeen Wastewater Treatment Plant, National Pollutant Discharge Elimination System (NPDES) Permit WA0037192, 2018.

CHAPTER 4

EXISTING FACILITIES

INTRODUCTION

This chapter describes the existing facilities that compose the City of Aberdeen's wastewater collection and treatment systems, and briefly describes the performance and condition of these facilities. The facilities include pressure and gravity sewers, pump stations, wastewater treatment facilities, and river outfall.

The information about the condition of the facilities is taken from the *WWTP and Collection System Condition Assessment (Condition Assessment)* provided in Appendix D, which provides more detail about the facilities' condition. In the *Condition Assessment,* each facility was assigned a condition value based on the percentage of the value of the facility that would be required to restore each facility to its original physical condition and useful life, as well as an importance rating that indicates the relative consequence of specific facility failure with regard to the overall wastewater treatment process. The condition ranking scale and importance ratings are shown in Tables 4-1 and 4-2. Additional summary information about the condition of facilities utilizing the condition and importance ratings is provided later in this chapter.

TABLE 4-1

Facility Condition Ranking Scale

Ranking	Description	Percentage of Facility Requiring Repair
1	Very Good Condition	0
2	Minor Defects	5
3	Maintenance Required to Return to Acceptable Level of Service	10 to 20
4	Requires Rehabilitation	20 to 40
5	Facility Unserviceable	>50

TABLE 4-2

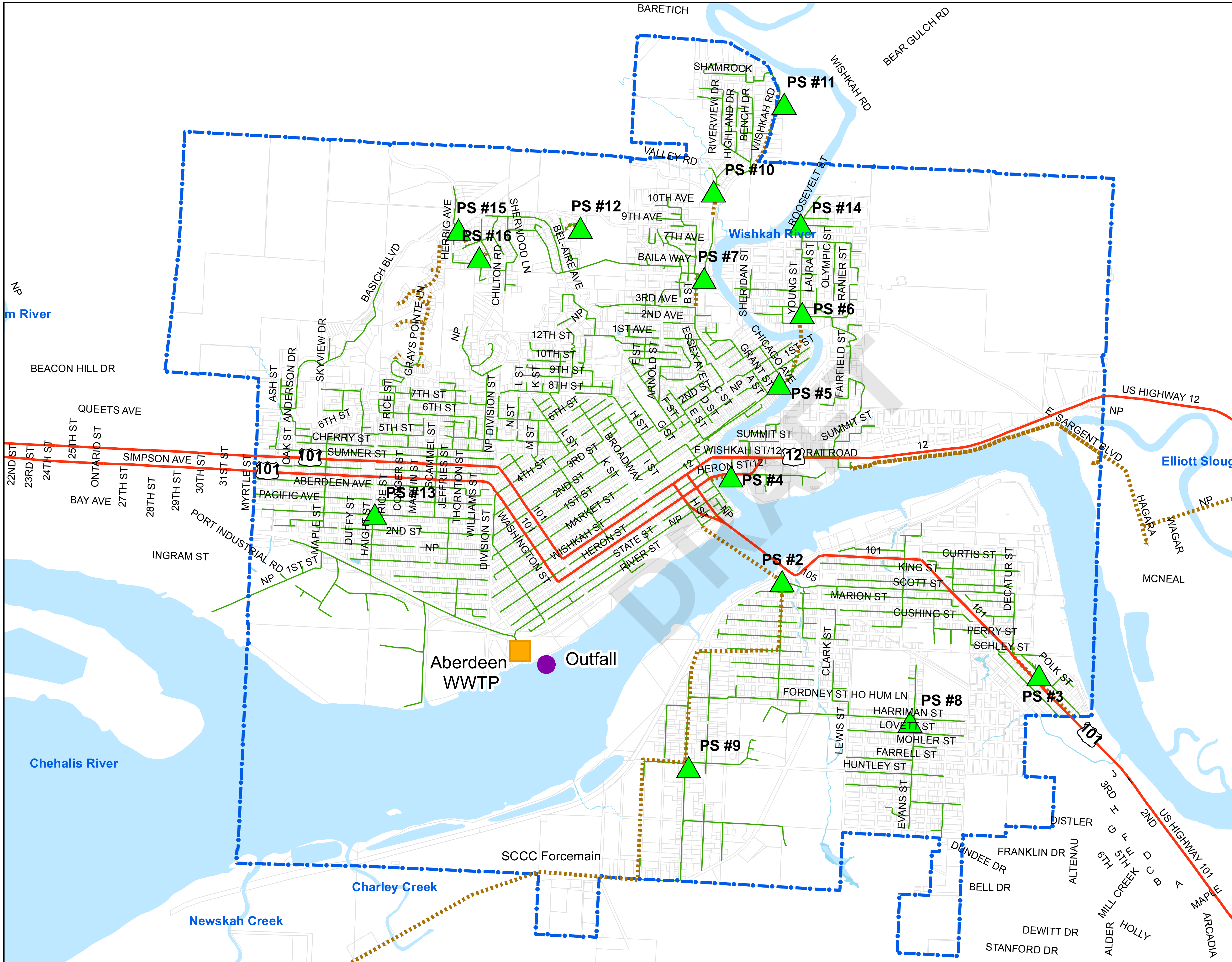
Importance Rating

Importance Rating	Importance Level	Description
5	Very High Importance	Failure would be catastrophic to the City, such as causing significant risk of death or serious injury to staff or the public.
4	High Importance	Failure would have significant impacts to the City, such as causing high risk of permit violation or possible risk to staff or the public health/safety.
3	Moderate Importance	Failure would result in moderate impacts to the City, such as causing moderate risk of permit violation.
2	Low Importance	Failure would likely not result in interruption to the sewer service in the City.
1	No Importance	Failure would have negligible impact to the City, such as process with adequate backup/redundancy.

WASTEWATER COLLECTION SYSTEM

PUMP STATIONS

The City of Aberdeen has 17 pump stations within its sanitary sewer system. Pump Station 1 is the WWTP influent pump station, and Pump Stations 2 through 16 are located throughout the collection system. Additional stations serve the SCCC and Lemay Landfill. The locations of these pump stations are shown in Figure 4-1. Basic information about the pump stations is included in Table 4-3. All of the collection system pump stations contain two pumps except Pump Station 13, which contains three pumps. The pumps range in size from 2 to 150 horsepower (hp). Aberdeen uses a SCADA system to monitor operations. Most of the pumps within each pump station contain individual flow meters connected to the SCADA system. Except for Pump Station 11, each pump station has provisions for emergency power, via a stationary generator or a portable trailer-mounted emergency hookup connection. Each facility is secured with fencing, locked access, or combination thereof. The SCCC system also includes an aerated equalization storage tank and screening system.



Legend

- Pump Stations
- Highway
- CityLimit
- Gravity Sewer
- Force Main
- Waterbody
- Parcel

ABERDEEN GENERAL SEWER/WASTEWATER FACILITIES PLAN

FIGURE 4-1
Collection System
Pump Stations

Gray & Osborne, Inc.
CONSULTING ENGINEERS

TABLE 4-3

Pump Stations

Pump Station	Location	Pump Station Type	Qty. of Pumps	Model	gpm	TDH	Total Station Capacity (gpm, w/one Unit Out of Service)	hp	Flow Meter	Discharge Valve (Force Main Dia.)	Force Main Diameter (in.)	Force Main Length (ft.)
2	930 West Scott Street (South Mill Street and West Scott Street)	Dry Pit	2	Vaughan 8×8×15-612 PE8N10CS	3,650	38'	3,650	75	8"	10"	24	2,138
3	116 East Mead Street (East Mead Street and Southwest Boulevard)	Submersible	2	Barnes Hydromatic S4MX500JB	120	12'	120	5	4"	4"	10	2,084
4	603 East Heron (East Heron Street and South Kansas Street)	Dry Pit	2	Cornell 4NNTL[1]	600	30'	600	7.5	6"	6"	8	670
5	101 Chicago Avenue (Chicago Avenue and East Market Street)	Submersible	2	Vaughan X180TY	340 (#1) 340(#2)	15' (#1) 15' (#2)	340	3	4"	4"	4	84
6	1506 Young Street (Young Street and Lafayette Street)	Dry Pit	2	Cornell 4NNT-CC[1]	650	40'	650	7.5	6"	8"	8	1,607
7	807 5th Avenue (North B Street and 5th Avenue)	Dry Pit	2	Cornell 6NHTA-CC	750 (#1) 600 (#2)	30'	600	30	6"	6"	8	914
8	901 South Evans Street (East Harriman Street and South Evans Street)	Dry Pit	2	Cornell 4NNT-V14[1]	600	30'	600	7.5	8"	8"	8	69
9	1401 West Huntley (SW Front Street and West Huntley Street)	Dry Pit	2	Cornell 4NNT-VM (#1)[1] Cornell 4NNT-CC (#2)	600	30'	600	7.5	8"	8"	8	122
10	2300 North B Street (Victory Way and North B Street)	Submersible	1	Hydromatic Submersibles S4NX500FC	400		0	5	4"	4"	6	618
11	2760 Wishkah Road	Submersible	2	Hydromatic Submersibles H4HX1500JC	500		500	15	6"	6"	6	1,771
12	Wishkah Road (Rognlin Drive and Tolomei Drive)	Submersible	2	Hydromatic Submersibles 5420	400		400	5	4"	4"	3	437
13	1900 Rognlin (Haight Street and Pacific Avenue)	Submersible	3	Hydromatic Submersibles C3126 S6AX1000FB (#1&3) (2) Vaughan Submersible C3126 P25G2707K (#2)	900		1,800	10 (#1 and 3) 15 (#2)	6"	6"	12	35
14	360 North Haight Street (Roosevelt Street and Young Street)	Submersible	2	Hydromatic Submersible S4MX300FB				3	4"	4"	4	48
15	2001 Roosevelt (Herbig Avenue and Judith Court)	Submersible	2	Submersible Vaughan Chopper pump	100	39'	100	5	4"	4"	4	407
16	730 Judith Court	Submersible	2	Submersible Vaughan Chopper pump	100	49'	100	7.5	4"	4"	4	229
SCCC	Stafford Creek Corrections Center (191 Constantine Way)		2	Cornell Centrifugal 6NHTB-VC18DR	1,300	220	1,300	150	10"	10"		

(1) For Pump Stations 4, 6, 8, and 9, the City plans to replace one pump in each station with a Vaughan PE4S6CS-113. 15 hp, 650 gpm, 31 to 39' TDH (40' TDH for Station 6).

(2) For Pump Station 13, one of the pumps will be replaced with a Vaughan SE6W-100. 1,350 gpm, 18' TDH, 15 hp.

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Facilities

There are two types of pump stations within the collection system: Dry Pit and submersible. Prior to 2019, the City had third type; Pump Stations 15 and 16 were the only self-priming pump stations. The self-priming pump stations were originally privately owned and maintained prior to being turned over to the City in the mid-1980s. The City replaced the self-priming pumps in Pump Station 15 and 16 with Vaughan submersible pumps in 2019; electrical upgrades are planned for these stations in 2020.

The dry pit pump stations (2, 4, 6, 7, 8 and 9) illustrated in Figures 4-2 and 4-3 were originally installed in the early 1950s, and initially upgraded in 1981. They are cylindrical, cast-in-place concrete structures. The above-grade structure houses the electrical, instrumentation, and ancillary equipment. The wet well and dry pit make up the below-grade portion of the structure. The dry pit and above-grade structures are pressure ventilated. The dry pit houses two vertical dry pit centrifugal pumps or dry pit submersible pumps, or a combination thereof.

The submersible pump stations illustrated in Figure 4-4 were constructed in the early 1980s. They consist of two below-grade components: wet well and valve vault. The electrical and controls components are located above grade in their respective panel-mounted enclosures. With the exception of Pump Station 13, each wet well houses two submersible pumps. Pump station 13 is outfitted with three submersible pumps.

The newest pump station in the system is the pump station installed in 2000 to transfer sewage from SCCC to the collection system. The SCCC pump station pumps into the force main under the Chehalis River which discharges into the State Street interceptor. The station incorporates a 1.66-mgd storage tank, which is used to reduce the impact of pumping on the system by dampening peak flows. The pump station is routinely operated during the hours of 11:00 p.m. to 5:30 a.m., thus reducing peak flow which would otherwise be experienced at the treatment plant. In addition, the SCCC Pump Station is not operated when instantaneous influent flows to the WWTP exceed 13 mgd.

FIGURE 4-2

Pump Station 4– Typical of Dry Pit Pump Stations

FIGURE 4-3

Pump Station 7 – Typical of Dry Pit Pump Stations

FIGURE 4-4

Pump Station 13 – Typical of Submersible Pump Station

Condition of Pump Stations

Table 4-4 summarizes the condition assessment of the pump stations. As can be seen in this table, Pump Stations 2, 4, 5, 6, 7, 8, 9 and 13 need to be upgraded in the near future. Many of the pump station facilities are approaching the end of their useful life and/or require upgrades in the near future. Common deficiencies observed for virtually all the collection system pump stations are:

1. Lack of security
2. Space not NFPA 820 compliant.
3. Metal corrosion of the structures.

In addition, a major deficiency is the lack of piping connections and miscellaneous piping to the force mains near the stations, to allow bypass of the pumps at the stations during power outages or pump failures. Currently, for Pump Stations 2, 4, 5, 6, and 7, if both pumps fail, there is no bypass connection to connect a portable pump. Thus, the City has to pump wastewater from the wet well into trucks and drive the wastewater to a downstream location or to the WWTP. Fortunately, these situations have generally occurred in low flow situations. However, if this were to occur during a storm, the result could be massive sanitary sewer overflows in the vicinity of the stations.

The electrical and power equipment of Pump Stations 2, 4, 5, 7, 9, and 13 are near the end of their useful life and need to be replaced soon. All electrical at Pump Station 13 is in an underground vault and should be raised above grade.

As noted in Table 4-3, **f**or Pump Stations 4, 6, 8, and 9, the City plans, as maintenance tasks, to replace one pump in each station with a Vaughan PE4S6CS-113 chopper pumps. For Pump Station 13, one of the pumps will be replaced with a Vaughan SE6W-100 chopper pump.

More detailed discussion of the pump stations and recommended improvements is provided in Chapter 6.

TABLE 4-4

Collection System Pump Stations Condition and Weighted Ratings

Pump Station	Importance	Mechanical/ Process/ Piping	Electrical/ I&C/ Power	Structural/ Architectural	Civil/ Site Work	HVAC and Odor Control	Average Rating	Weighted Rating
2	***4***	4	4	3	3	3	**3.4**	**13.6**
3	***2***	3	3	3	2	2	**2.6**	**5.2**
4	***3***	3	4	3	3	3	**3.2**	**9.6**
5	***4***	4	4	3	3	3	**3.4**	**13.6**
6	***3***	3	3	3	3	3	**3.0**	**9**
7	***5***	3	4	3	3	3	**3.2**	**16**
8	***3***	3	4	4	3	3	**3.4**	**10.2**
9	***3***	3	4	4	3	3	**3.4**	**10.2**
10	***3***	2	3	4	2	3	**3.0**	**8.4**
11	***2***	3	3	4	2	3	**2.8**	**5.7**
12	***2***	3	3	4	2	3	**2.8**	**5.7**
13	***4***	3	3	4	2	3	**3.0**	**12**
14	***2***	3	3	4	2	3	**2.8**	**5.7**
15	***2***	1	1	1	1	1	**1.0**	**2.0**
16	***2***	1	1	1	1	1	**1.0**	**2.0**

GRAVITY COLLECTION SYSTEM

The collection system was originally constructed as a combined storm and sewer system. Since the construction of the WWTP in 1950s, the City began separating the combined collection system into separated sanitary and storm system. The replacement of the system piping was completed by the late 1970s, with most of replacement in late 1970s. Improved sewer construction and pipe materials have been used, including non-porous piping materials (PVC pipe) and rubber-gasket type joints to reduce infiltration and improve the condition of the sanitary sewer system.

Figure 4-1 shows the existing sewer system. Wastewater is discharged to the City's secondary wastewater treatment facility, which has an outfall on the Grays Harbor Estuary, near the mouth of Chehalis River. The collection system conveys wastewater from the hilly uplands in the northern portion of the City and the flat lowlands region which comprises the majority of the rest of the City's collection system.

The area generally slopes toward the mouth of the Chehalis River, where the treatment facility is located. Thus, much of the collection system consists of gravity sewers. However, the system also contains pump stations and pressure lines. The current system consists of 4-inch to 48-inch diameter pipe. A summary of the various diameters and the percentage of each within the City's sewer system is provided in Table 4-5. This summary is based on the piping GIS data, which was built by the City based on manhole inspections, review of as-built drawings and previous television inspection. Figure 4-5 shows the sewer system with sewer pipe diameters identified. The City's sanitary sewer system also contains approximately 1,750 manholes. The older brick and concrete block manholes with rigid mortar joints were replaced with newer precast manholes to reduce the infiltration under the collection system reconstruction program around the City in the later 1970s.

TABLE 4-5

Sewer Pipe Summary

<table>
<tr><th colspan="4">Collection Sewers (Gravity)</th></tr>
<tr><th colspan="2">Pipe Diameter (in.)</th><th colspan="2">Pipe Length(ft.)</th></tr>
<tr><td colspan="2">4</td><td colspan="2">139</td></tr>
<tr><td colspan="2">6</td><td colspan="2">11,206</td></tr>
<tr><td colspan="2">8</td><td colspan="2">327,412</td></tr>
<tr><td colspan="2">10</td><td colspan="2">22,594</td></tr>
<tr><td colspan="2">12</td><td colspan="2">10,515</td></tr>
<tr><td colspan="2">14</td><td colspan="2">3,148</td></tr>
<tr><td colspan="2">15</td><td colspan="2">5,590</td></tr>
<tr><td colspan="2">16</td><td colspan="2">423</td></tr>
<tr><td colspan="2">18</td><td colspan="2">935</td></tr>
<tr><td colspan="2">24</td><td colspan="2">585</td></tr>
<tr><td colspan="2">Subtotal</td><td colspan="2">382,547</td></tr>
<tr><th colspan="4">Interceptor Sewers</th></tr>
<tr><th colspan="2">Gravity</th><th colspan="2">Force Main</th></tr>
<tr><th>Pipe Diameter (in.)</th><th>Pipe Length(ft.)</th><th>Pipe Diameter (in.)</th><th>Pipe Length(ft.)</th></tr>
<tr><td>8</td><td>1,567</td><td>2</td><td>8,729</td></tr>
<tr><td>12</td><td>3,795</td><td>3</td><td>1,126</td></tr>
<tr><td>14</td><td>2,735</td><td>4</td><td>1,989</td></tr>
<tr><td>16</td><td>3,200</td><td>5</td><td>6,531</td></tr>
<tr><td>18</td><td>2,592</td><td>6</td><td>2,435</td></tr>
</table>

TABLE 4-5 – (continued)

Sewer Pipe Summary

Interceptor Sewers			
Gravity		Force Main	
Pipe Diameter (in.)	Pipe Length(ft.)	Pipe Diameter (in.)	Pipe Length(ft.)
20	1,316	8	3,177
24	9,486	10	2,084
36	1,898	12	7,470
48	5,378	24	2,138
Subtotal	**31,967**		**35,679**
Total	**450,193 feet (85.26 miles)**		

Condition of Gravity Collection System

Much of the collection system was replaced with polyvinyl chloride (PVC) pipes in the late 1970s, which is expected to be in good condition. However, portions of the system are served with aging asbestos-concrete (AC) or ductile iron pipes, which tend to have defects such as misaligned joints, cracks, fractures, and holes.

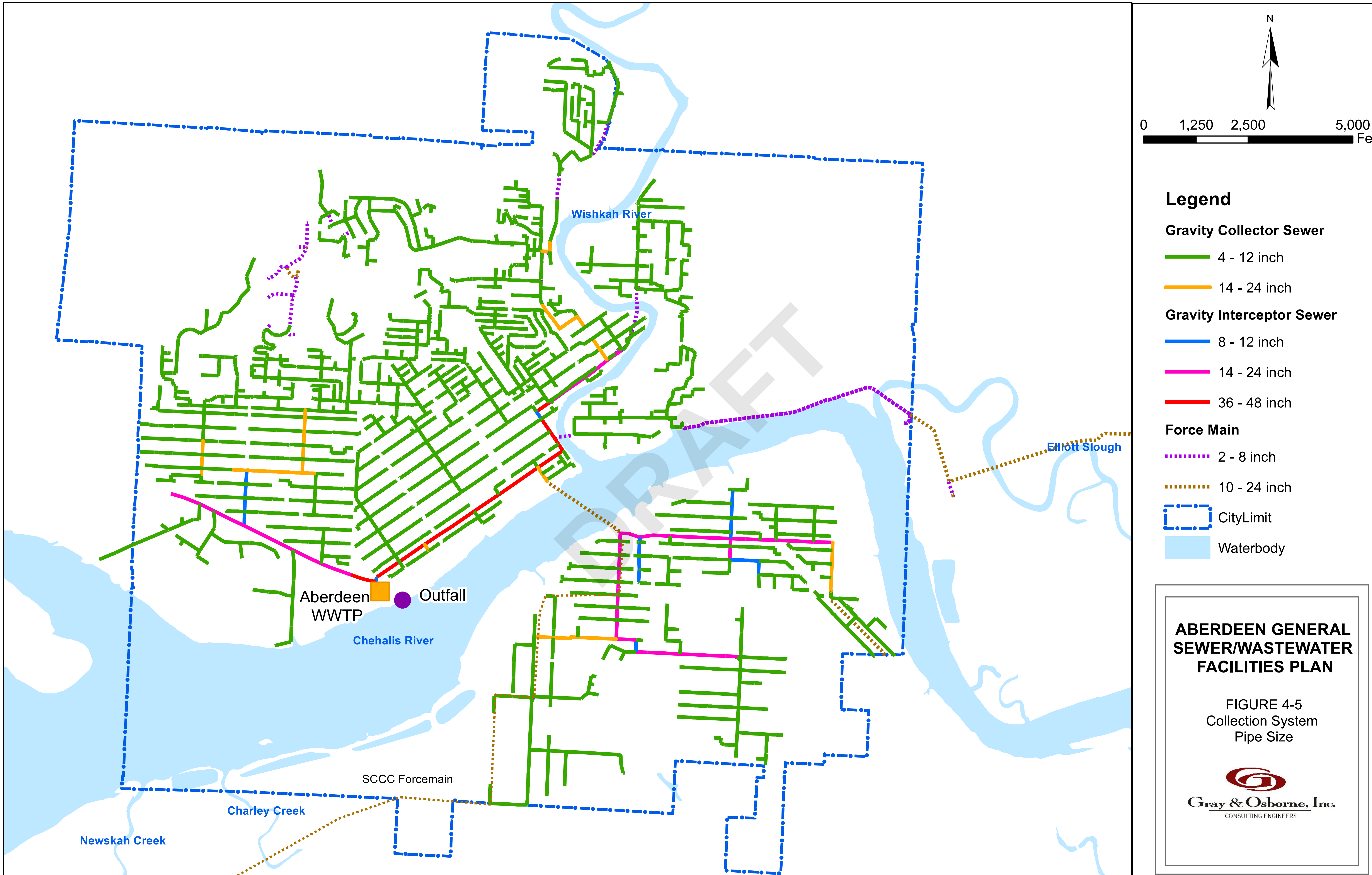
Most of the sewer system service area in Aberdeen is located in low-lying area and subject to I/I, particularly inflow during storms. The storm sewer system can back up when high rain events occur at the same time as high tides. Portions of the system are below the 100-year floodplain, and portions can surcharge during storm events, particularly when the river and/or tide are high during the storm peak.

The City has maintained an ongoing effort to minimize I/I. Annual activities include identifying illegal connections and implementing and monitoring corrective actions, manhole rehabilitation through grouting and epoxy lining, replacement of damage sewer sections, and hydro-cleaning. In addition, the City has completed a major effort to install storm water system pump stations in downtown Aberdeen. City staff note that this has had the effect of reducing the intensity of peak I/I flow. As described in Chapter 2, the City is currently in the design phase of the Northshore Levee flood control project that will add additional storm water pump stations.

More detailed discussion of the gravity collection system and recommended improvements is provided in Chapter 6.

WASTEWATER TREATMENT PLANT

The original City of Aberdeen Wastewater Treatment Plant (WWTP) was constructed in the 1950s as a primary treatment facility with two anaerobic digesters. The upgrade of the original facility was completed by 1981 with a new primary and secondary treatment process upgrade. A new anaerobic digester was added in 1991. Thereafter, the facility



N
0 1,250 2,500 5,000
Feet
Legend
Gravity Collector Sewer
4 - 12 inch
14 - 24 inch
Gravity Interceptor Sewer
8 - 12 inch
14 - 24 inch
36 - 48 inch
Force Main
2 - 8 inch
10 - 24 inch
CityLimit
Waterbody
Wishkah River
Elliott Slough
Aberdeen WWTP
Outfall
Chehalis River
SCCC Forcemain
Charley Creek
Newskah Creek
DRAFT
ABERDEEN GENERAL SEWER/WASTEWATER FACILITIES PLAN
FIGURE 4-5
Collection System
Pipe Size
Gray & Osborne, Inc.
CONSULTING ENGINEERS

underwent several improvements between 2001 and 2005, including the installation of an effluent filter, secondary treatment system improvement, influent pumping and headworks improvement and solid handling equipment upgrade. (The effluent filters proved to not be needed after the installation of the large secondary clarifier in 2004 so the City decommissioned the filters.) A new outfall was installed in 2014. The facility is currently rated at 9.9 mgd maximum month capacity. A process flow diagram of the Aberdeen WWTP is presented in Figure 4-6.

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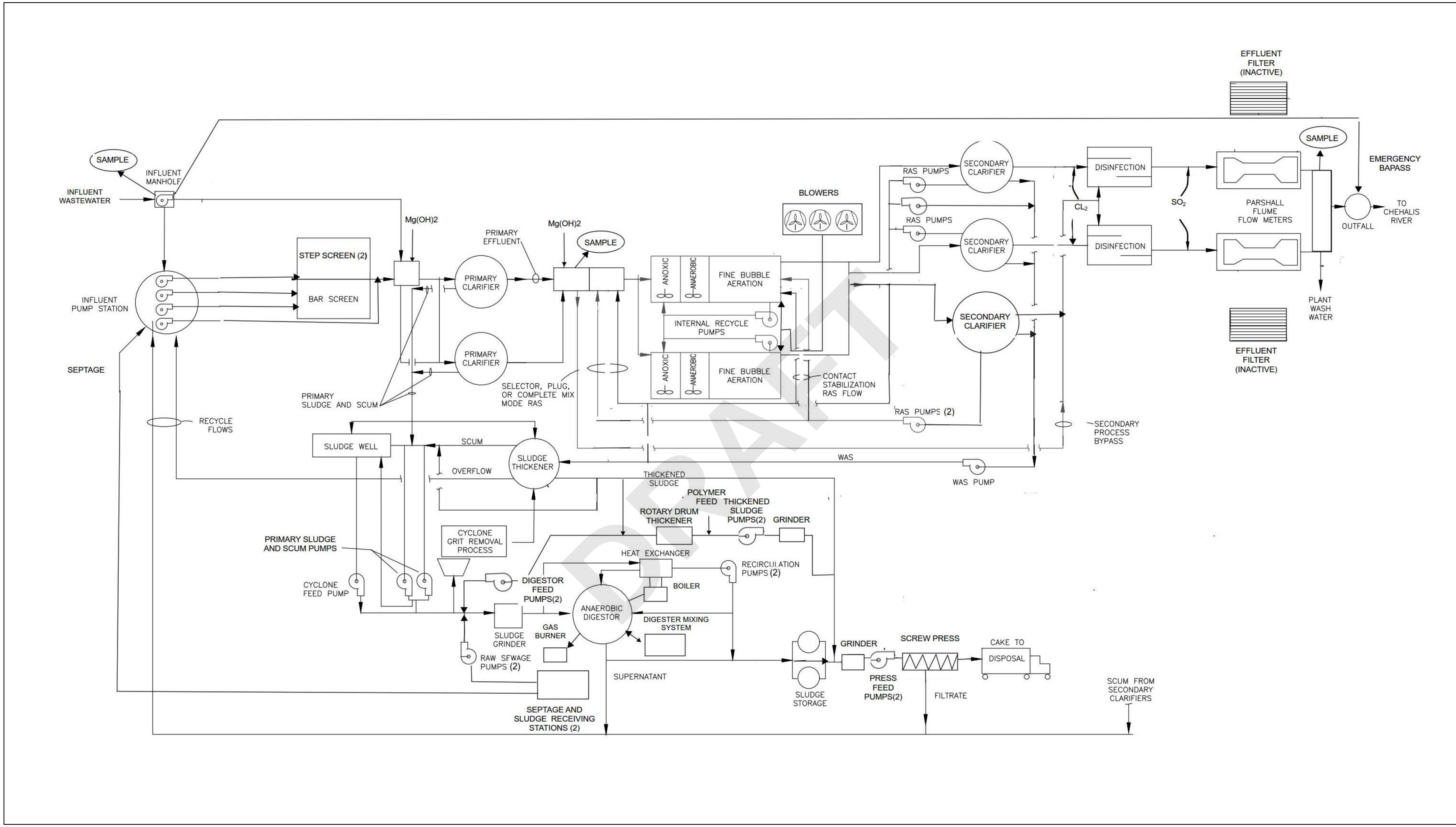

FIGURE 4-6

WWTP Process Schematic

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LIQUID TREATMENT PROCESSES

Influent Pump Station

The Influent Pump Station was constructed during the original WWTP in the 1950s. It consists of four mixed flow centrifugal sewage pumps in a dry well adjacent to a wet well. Both the dry well and wet well are contained in a 30-foot diameter reinforced concrete caisson. The wet well collects flows from the wastewater collection system, wastewater treatment plant recycle flows, and truck-haul waste (including septage). Flows out of the influent pump station (which includes not only WWTP influent but also recycle streams) through the force mains to the headworks are measured with magnetic flow meters. Each of the four pumps has a dedicated force main to the treatment plant headworks. The discharge force main from Pumps 2 and 3 were replaced in 2016 due to leaks from the piping. Pumps 2 and 1 were replaced in 2019. (VFD control will be provided for Pump 1.) Pump data are presented in Table 4-6.

TABLE 4-6

Influent Pump Data

Description	Manufacturer	Capacity (gpm)	Motor Nameplate, Horsepower
Pump 1	Vaughan	4,800	100
Pump 2	Vaughan	3,300	60
Pump 3	Vaughan	3,600	60
Pump 4	Cornell Pump Co.	2,000	30
Pump 5	Flygt	2,000	30

Pressure switches (Druck) are used to start and stop the pumps. Pumps 2 and 3 were provided with a 60-hp variable frequency drive (VFD) between year 2000 and 2001. The other pumps are currently operated in “soft start” mode.

The high water alarm for the pump station is set at elevation -1.0 foot (MLLW datum). This is the level where the influent pipeline from the collection system would be 100 percent submerged. The firm capacity of the influent pump station is approximately 13 mgd with the largest pump out of capacity. Pump 4 is set up to bypass the influent to downstream of the headworks screen. The flow records indicate that Pump 4 is running less than 50 hours per year.

In the addition to the dry well pump station, a fifth pump located in the influent sewage manhole northeast of the laboratory, will operate when necessary and bypass the influent to downstream of the headworks screen, thus increasing the overall capacity of the influent pumping at the WWTP from 18 mgd to 22 mgd with all pumps running. Flow records indicate the submersible bypass pump runs less than 40 hours per year. Figure 4-7 shows Pump 3.

FIGURE 4-7

Influent Pump Station (Pump 3 in Foreground)

Condition of Influent Facilities

Overall, the influent facilities are in moderate condition. In the Influent Pump Station, there is significant degradation of the concrete structure reported. Necessary improvements for HVAC and electrical were identified, including but not limited to upgrades to ensure ventilation and electrical code compliance for the Influent Pump Station.

Headworks

The WWTP headworks structure receives flow through separate force mains from each influent pump. The force main for Pump 1 was replaced in 2017 due to severe internal erosion. The condition of the other force mains is unknown but are thought to be in better condition because flow through those pumps is less frequent. Each force main discharges into the headworks influent box vertically. The headworks structure was put online in 1981. It replaced a grit channel at a lower level that served the original primary clarifier. Headworks data are presented in Table 4-7.

TABLE 4-7

Headworks Data

Description	Value
Bypass Bar Screen	
Type	Manual
Number	1
Channel Width (feet)	3
Bar Depth (feet)	2.55
Bar Spacing (inches)	1
Bar Thickness (inches)	0.38
Step Screen	
Type	Mechanical
Number	2
Capacity (each; mgd)	9
Screenings Conveyor	
Type	Shaftless Screw
Motor Nameplate (hp)	2
Screenings Compactor	
Number	1
Motor Nameplate (hp)	4

Three channels are provided downstream of the influent box. The two outside channels have Huber step screens designed to wash, compact, and dispose of screenings into a dumpster adjacent to the headworks structure (Figure 4-8). The step screens and washer compactor were installed in 2005 to replace the previous comminutor and a channel monster. The inner channel has a manual bar screen. The grit settles in the channels upstream of the screen and is removed by a vactor.

The facilities were designed for a total step screen capacity of 18 mgd. In reality, the capacity is only about 13 mgd. The manual bar screen does not have capacity to pass all the flow from Pumps 1 through 3 (Pumps 4 and 5 bypass screening completely) unless the screen is frequently raked manually. The insufficient capacity of the redundant bar screen poses a risk of overflow. Bypass of flow from Pumps 4 and 5 to the primary clarifier increases the risk of sludge handling system failure due to large debris obstructing the bypass. Gates are provided to permit isolation of any one channel.

The headworks structure also contains a separate chamber to receive primary effluent from the primary sedimentation tanks and an automatic sampler for primary effluent.

FIGURE 4-8

Headworks Step Screen (1 of 2)

<u>Condition of he Headworks</u>

The existing Headworks equipment (fine screens, screenings conveyor, and screenings washer/compactor) has exceeded its useful life. Overall, the Headworks facilities are in poor to moderate condition. The Headworks needs to be upgraded to reduce its vulnerability for failure, and to increase the capacity of the mechanical screening system to screen all of the influent flow, as well as to screen all of the influent flow with one mechanical screen out of service with the manual bar screen.

Primary Sedimentation Tanks

Two, 65-foot diameter tanks provide primary sedimentation for the influent wastewater. Figure 4-9 shows the Primary Sedimentation Tanks. The primary sedimentation tanks were constructed during the 1977-1981 expansion. The grit removal mechanism was rehabilitated after damage from the Nisqually Earthquake in 2001.

FIGURE 4-9

Primary Clarifier (1 of 2)

A single centrifugal pump in the primary clarifier sludge room transfers primary sludge to a hydrocyclone degritter located in the solids building (see Figures 4-10 and 4-11). After grit removal, the degritted sludge is conveyed to the gravity sludge thickener. Two Penn Valley pumps (as well as Vogelsang pumps) are also provided to pump the thickened primary sludge from the gravity thickener and scum from the primary clarifiers directly to the digesters. Primary sedimentation system data are presented in Table 4-8.

TABLE 4-8

Primary Sedimentation System Data

Description	Value
Primary Sedimentation Tanks	
Type	Circular
Number	2
Tank Size (each)	
Diameter (ft)	65
Side Water Depth (ft)	10
Primary Sludge Pump	
Type	Centrifugal
Number	1
Capacity (gpm)	205
Motor Nameplate (hp)	10
Primary Sludge/Scum Pump	
Type	Double Disc
Number	2
Capacity (each, gpm)	85
Motor Nameplate (each, hp)	7.5
Primary Sludge Grit Separator	
Type	Cyclone
Number	1
Capacity (gpm)	200
Primary Sludge Grit Classifier	
Type	Auger
Capacity (tons/day)	23.8
Motor Nameplate (hp)	0.75

FIGURE 4-10

Primary Sludge Pumps

FIGURE 4-11

Hydrocyclone and Degritter

Primary Clarification Performance

Figures 4-12 and 4-13 present removal data for the primary clarifiers from the last 5 years. The charts show removal rates as a percentage of the influent concentration for TSS and BOD. Removal is shown as a function of tank overflow rate in gallons per day per square foot of tank area (gpd/sf). The data vary widely and the correlation between tank overflow rate and removal performance is highly variable. This is likely due to variability in influent characteristics. The average BOD removal rate is 43 percent at an average overflow rate of approximately 530 gallon per day (gpd) per square foot, but the ratio varies widely. The standard deviation in the removal percentage is 18 percent. The average TSS removal rate is 59 percent with standard deviation of 25 percent. The figure shows a best fit line through the data compared to typical levels adapted from the reference *Water Supply and Sewerage* (Steel and McGhee, 1985). The BOD removal rate is above the reference line, while the TSS removal rate is below the reference line. The measured removal rate would be higher if recycle flows and septage were accounted for. These flows go directly to the influent pump station wet well and are not measured in the influent sampler.

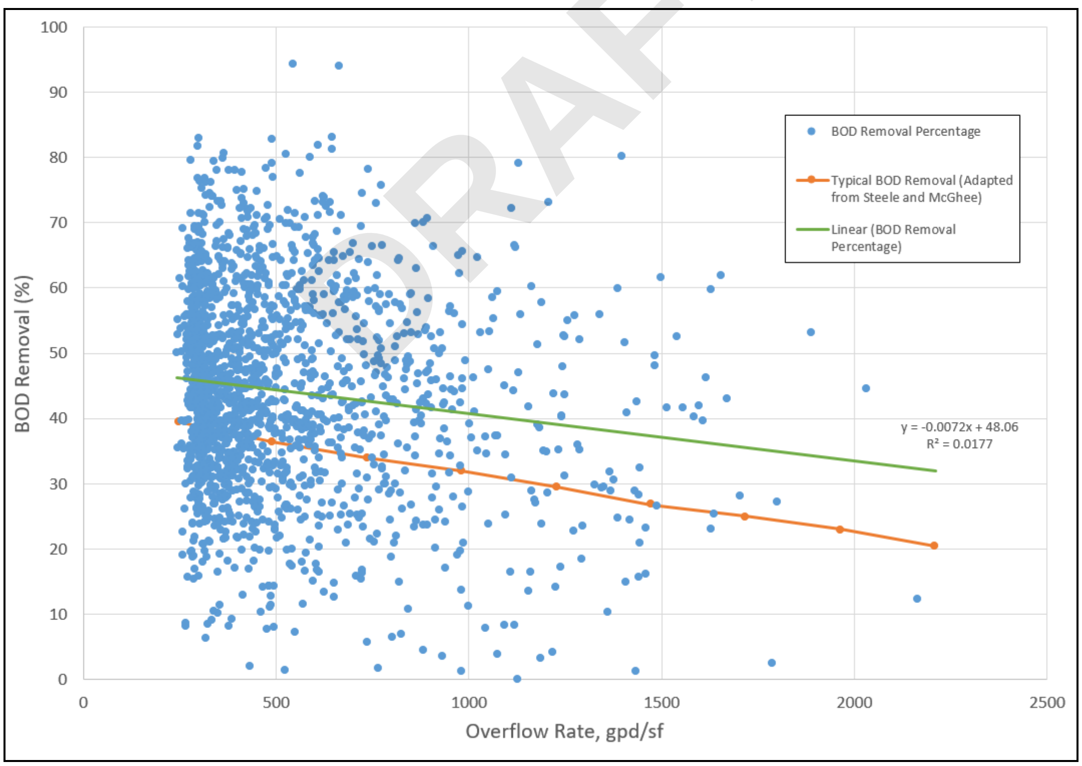


FIGURE 4-12

Primary Sedimentation BOD Removal Performance

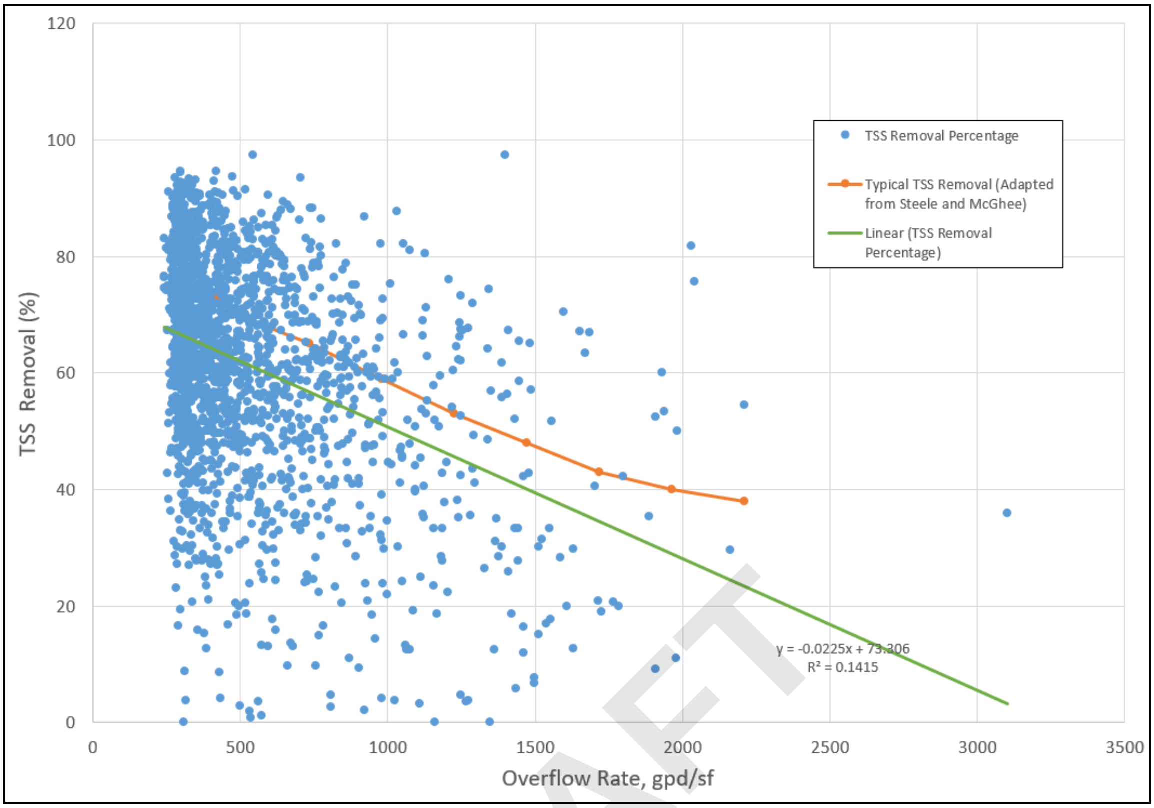


FIGURE 4-13

Primary Sedimentation TSS Removal Performance

<u>Condition of Primary Clarifiers</u>

Overall, the primary clarifiers and associated facilities are in moderate condition. Improvements are needed for HVAC and electrical in the belowground primary sludge pump room, including but not limited to the ventilation compliance, electrical classification/enclosure, and motor control center elevation to 12 inches above the door threshold due to its vulnerability demonstrated during recent events of plant flooding.

Activated Sludge System

The activated sludge system converts influent soluble BOD to waste biological solids, and captures the settled solids in the secondary clarifiers. The aeration tanks, the return activated sludge pumps, and two of the three current secondary tanks were constructed during the 1977-1981 secondary treatment expansion. The system was modified in the 2002-2004 improvements which converted the mechanical aeration system to fine bubble aeration and added the third secondary clarifier tank. In the modification, anoxic and anaerobic selector zones were created in the aeration tanks to improve sludge settleability. As part of the new fine bubble aeration system, a new Blower Building with three centrifugal blowers was constructed. In 2015, one blower was replaced with a high-efficiency small-capacity blower/compressor and an ultrafine bubble diffuser system was added to improve the efficiency and save energy. The two larger capacity blower/compressors were retained, and the replaced retained as a spare.

The aerobic section of the aeration tanks is shown in Figure 4-14. Figure 4-15 shows the blowers.

As noted in the *Condition Assessment*, the Aeration Basin structure also shows signs of concrete degradation and corrosion. It is estimated that the Aeration Basins will be reaching the end of their original useful life (OUL) in 10 years unless rehabilitated, which would be expected to increase the OUL to approximately 30 years. Improvements are needed for the electrical systems including, but not limited to, rehabilitation of settled conduit around the secondary clarifier and aeration basin area.

Activated sludge system data are presented in Table 4-9.

TABLE 4-9

Activated Sludge System Data

Description	Value
Activated Sludge Basins	
Number of Tanks	2
Volume, Mgal (each; mgd)	0.472
Anoxic Zone Volume (each; mgd)	0.126
Aerated Zone Volume (each; mgd)	0.346
Anoxic Tank Mixer	
Type	Submersible
Number	4
Motor Nameplate (each; hp)	5
Internal Recycle Pump	
Type	Centrifugal
Number	2
Capacity (each, gpm)	7,400
Motor Nameplate (each; hp)	20
Aeration System	
Diffuser	
Type	Membrane Panels, Fine Bubble
Number	1,900
Capacity (each, gpm)	85
Motor Nameplate (each, hp)	7.5

TABLE 4-9 – (continued)

Activated Sludge System Data

Description	Value
Blower	
Type	Centrifugal, Multi. Stage VFD
Number	3
Capacity (each; icfm)	2,150 /2,150/1,500
Motor Nameplate (each; hp)	125/125/75

FIGURE 4-14

Activated Sludge Aeration Tank

FIGURE 4-15

Centrifugal Blower

Secondary Clarifiers

Figure 4-16 shows a secondary clarifier tank. Two 85-foot-diameter secondary clarifiers (1 and 2) were constructed in the 1977-1981 expansion. These units have riser pipe sludge withdrawal mechanisms. A new 100-foot-diameter secondary clarifier (3) with spiral scraper and a suction header sludge withdrawal mechanism was constructed as part of the 2002-2004 improvements project.

FIGURE 4-16

Secondary Clarifier 2

As noted in the *Condition Assessment*, Clarifiers 1 and 2 shows signs of widespread effervescence, cracking, and seepage in the clarifier walls. It is estimated that the clarifiers will be reaching the end of their original useful life (OUL) in 10 years unless rehabilitated, which would be expected to increase the OUL to approximately 30 years. The electrical systems require improvements, including but not limited to rehabilitation of settled conduit. Secondary Clarifier data are presented in Table 4-10.

TABLE 4-10

Secondary Clarifier Data

Description	Value
Secondary Clarifiers 1 and 2	
Number of Tanks	2
Diameter (feet)	85
Side Water Depth (feet)	12
RAS Pump	
Type	Centrifugal
Number	3
Capacity (each, gpm)	3,100

TABLE 4-10 – (continued)

Secondary Clarifier Data

Description	Value
Secondary Clarifier 3	
Number of Tanks	1
Diameter (feet)	100
Side Water Depth (feet)	12
RAS Pump	
Type	Submersible
Number	2
Capacity (each, gpm)	3,100
Motor Nameplate (each, hp)	30
WAS Pump	
Type	Double Disc
Number	1
Capacity (gpm)	100
Motor Nameplate (hp)	5

RAS Pumps

Three RAS pumps return activated sludge from the bottom of the Secondary Clarifiers 1 and 2 to the aeration tank. These units are located in a separate building adjacent to the aeration tanks. They were installed in the 1977 to 1981 WWTP expansion. In 2017-2019, the City replaced two of the original RAS pumps with Vaughan chopper pumps of equivalent capacity. The City plans to replace the other pump within the next few years. In the RAS pump station beside Secondary Clarifier 3, two submersible RAS pumps conveying return activated sludge were installed during the 2002-2004 improvement project. Figure 4-17 shows the RAS pumps in the RAS pump building.

FIGURE 4-17

RAS Pumps in RAS Pump Building

FIGURE 4-18

WAS Pump

WAS pump

There is one diaphragm WAS pump to convey waste sludge to the thickener. (An additional pump is recommended for redundancy.) The pump is in the basement of the sludge pump building, shown in Figure 4-18.

Activated Sludge Treatment Performance

Table 4-11 presents data for several process performance variables for the Aberdeen WWTP activated sludge process.

Mixed Liquor Suspended Solids (MLSS) is the concentration of suspended solids in the mixed liquor. If the MLSS content is too high, the process can be prone to bulking and the treatment system becomes overloaded. Conversely, if the MLSS content is too low, the process may not be working efficiently. The typical control band is 1,500 to 4,000 mg/l for the complete mix activated sludge process. At the Aberdeen WWTP, the average MLSS concentration over the 5-year period from 2013 to 2018 was 1,913 mg/L in the north aeration basin and 2,050 mg/L in the south aeration basin. The MLSS in south basin is higher than north basin due to the high loading in the south basin during summers of 2016 and 2017 when the north basin is taken out of service.

Solids Retention Time (SRT) is the average time the activated-sludge solids are in the system. It is an important factor affecting the performance of nutrient removal and sludge characteristics. At the Aberdeen WWTP, the average solids residence time (SRT) was 5.3 days, compared to a typical value range between 3 and 18 days for complete nitrification depending on the mixed-liquor temperature and whether or not nitrification is desired. A typical value range between 3 and 5 days is employed where only BOD removal is required and to discourage nitrification and eliminate the associated oxygen demand.

Another key parameter in the performance of activated sludge treatment facilities is sludge settleability, or the settling rate of activated sludge. The sludge volume index (SVI), a measure of settleability of activated sludge, is the ratio of settled sludge volume after 30 minutes of quiescent settling to the MLSS concentration. The average value for the sludge volume index (SVI) was 129 ml/g with frequent month-long periods of SVI > 200 ml/g. Sludge with good settleability has SVI values in the range of 80 to 120 ml/g. The relatively high SVI values are indicative of some sludge bulking problems caused by excessive growth of filamentous organisms in the aeration tank mixed liquor.

MLSS, SVI and SRT history is presented in Figure 4-19, 4-20 and 4-21, respectively.

TABLE 4-11

Activated Sludge Performance Data (2013-2018)

Description	MLSS (mg/L)		SVI (ml/g)		SRT (Days)
	North Aeration Basin	South Aeration Basin	North Aeration Basin	South Aeration Basin	Both Basins
2013 Average	1,966	2,021	115	115	6.5
2014 Average	1,716	1,754	107	107	5.5
2015 Average	1,827	1,811	130	128	6.2
2016 Average	1,862	2,127	173	160	4.5
2017 Average	2,282	2,522	133	132	4.5
2018 Average (Jan-Aug)	1,896	2,130	134	134	4.5
Average	1,913	2,050	129	129	5.3
Max	3,720	4,420	621	641	10.0
Min	760	780	13	39	2.0

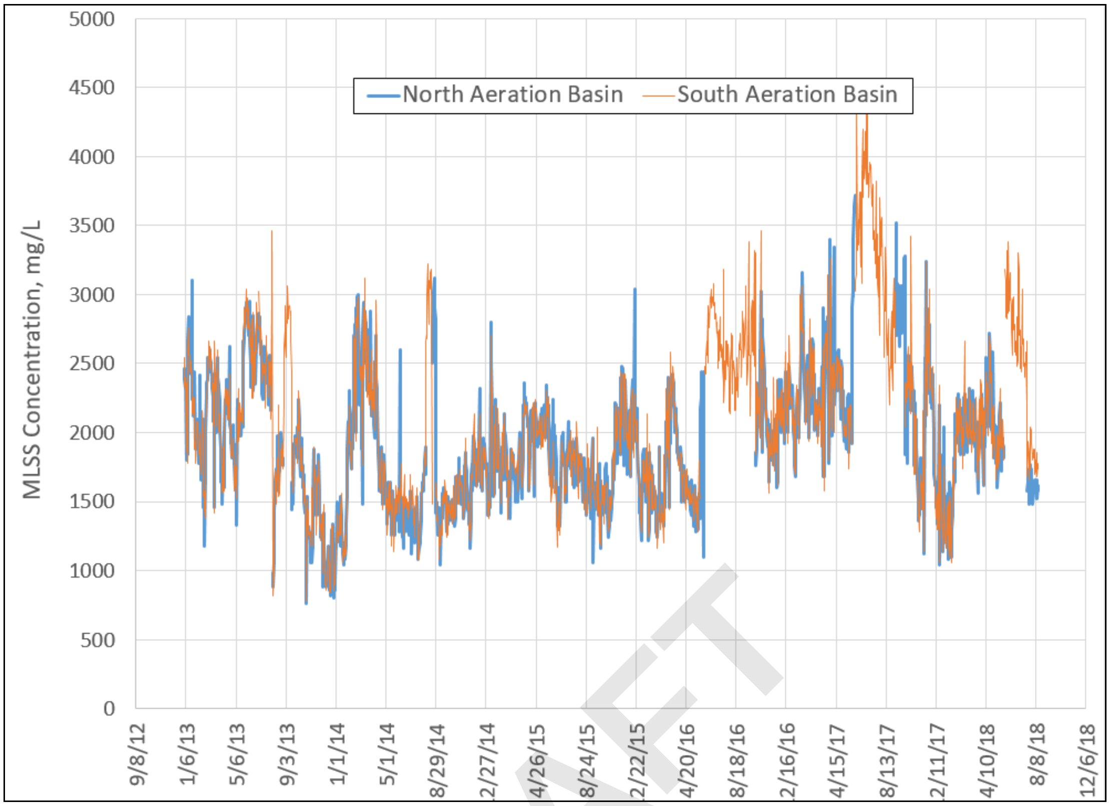


FIGURE 4-19

Aeration Tank Mixed Liquid Solids (MLSS) Concentrations

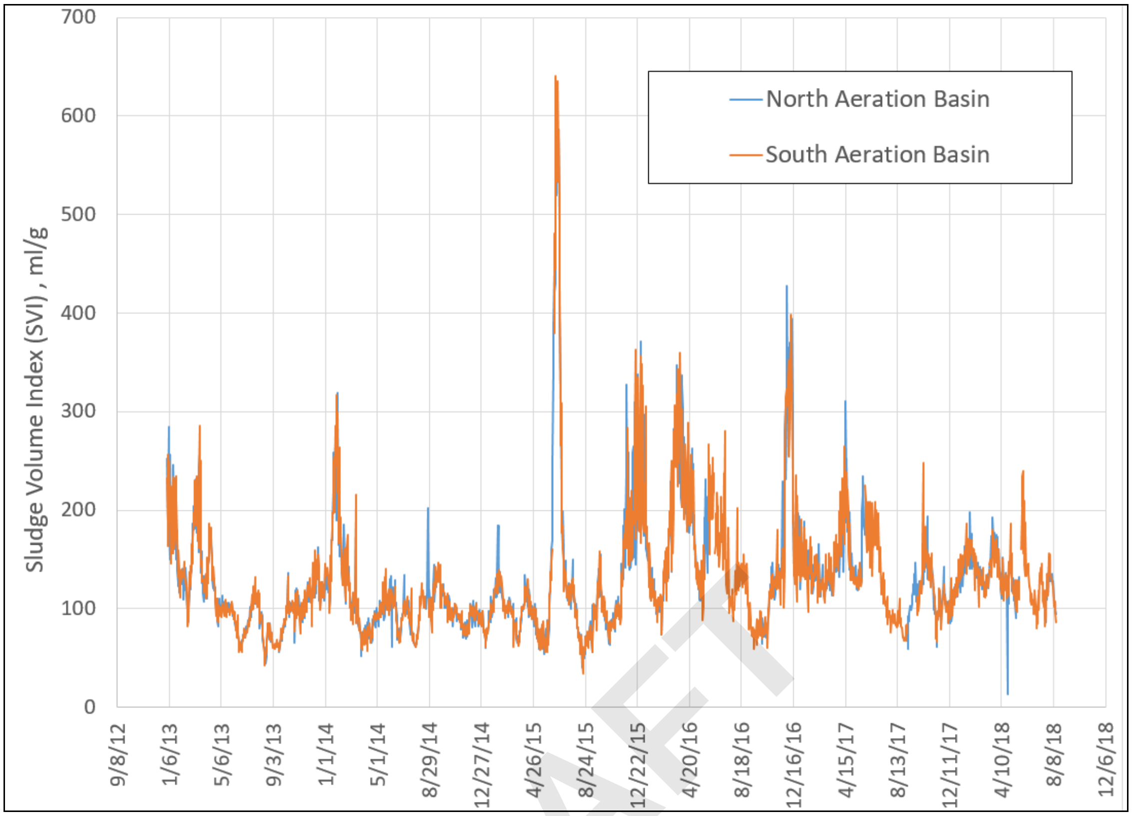


FIGURE 4-20

Aeration Tank Sludge Volume Index (SVI) Concentrations

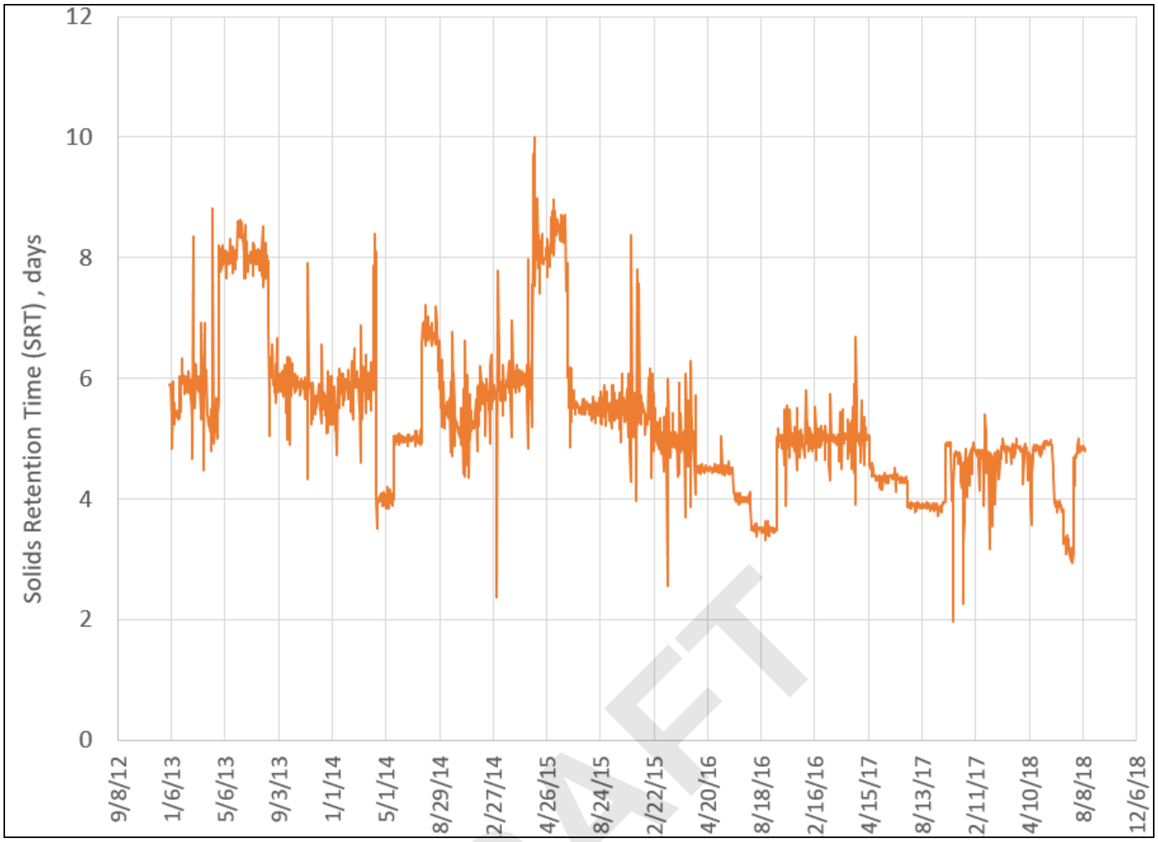


FIGURE 4-21

Aeration Tank Solids Residence Time (SRT)

<u>Effluent BOD and TSS</u>

As shown in Figure 4-22, effluent BOD and TSS concentrations have been compliant with effluent permit limits, averaging approximately 8.6 mg/L for BOD and 5.0 mg/L for TSS over the last 5 years of record. Some of the higher BODs appear to have been the result of pass through of soluble BOD to the effluent, or ammonia oxidation in the BOD test. The figure shows values for the 30-day moving average of effluent BOD and TSS concentration from January 2013 through August 2018.

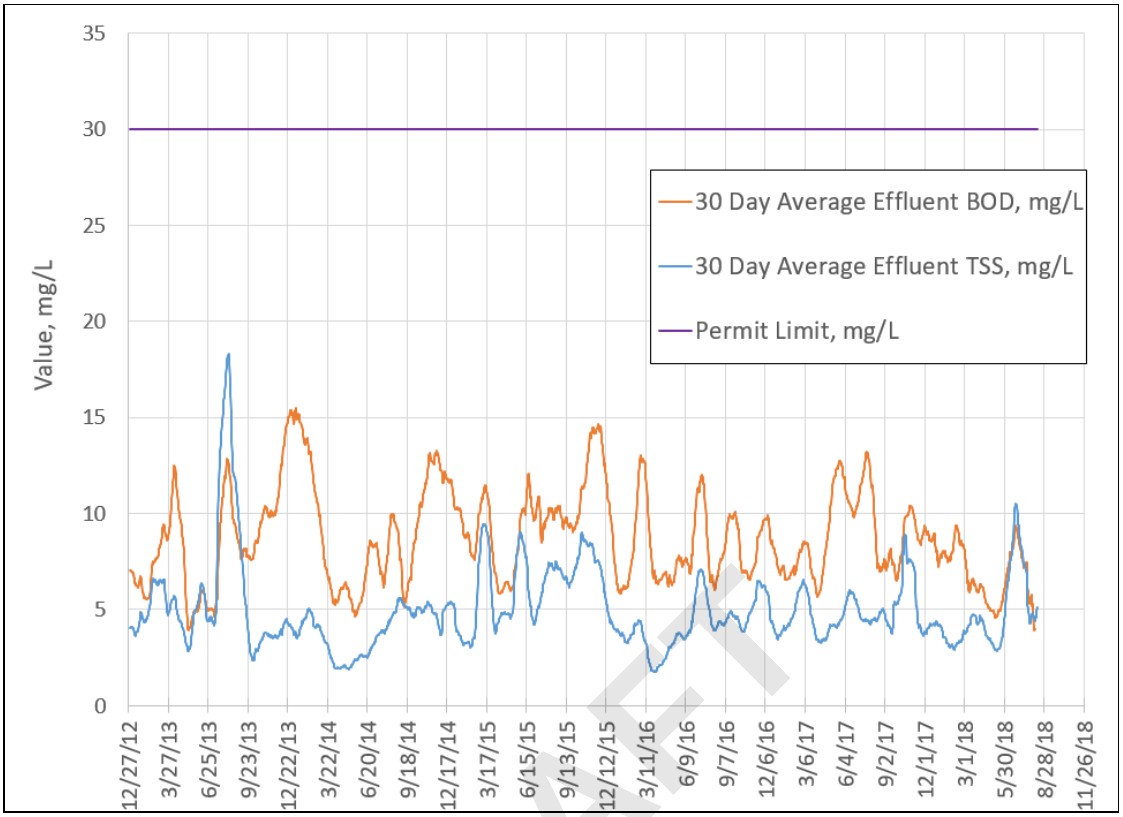


FIGURE 4-22

30-Day Average Effluent Concentrations

These effluent records indicate good solids capture by the secondary sedimentation tanks. This may be partly the result of the relatively filamentous activated sludge that is grown at Aberdeen under current conditions. Activated sludge plants with poor (high) values for SVI often have low effluent concentrations of suspended solids. This appears to be due to entrapment of fine material in the sludge floc. So, although poor settleability can reduce plant capacity significantly, it can result in low effluent BOD and TSS at low clarifier hydraulic loading rates.

Figure 4-23 shows daily BOD and TSS removal as a percentage of influent values from January 2013 through August 2018. The removal rate is low during the wet season of each year, especially during high flows. The removal rate appears to correlate inversely with rainfall. This provides further evidence of degree of impact of inflow and infiltration in the City's wastewater collection system.

Figure 4-24 shows that the running 30-day average BOD removal failed to achieve the permit limit of 85 percent in February 2014, November 2014 and November 2015. The average for the month, however, was not below 85 percent, so there was no violation. The measured removal rate would be higher if septage was accounted for. These flows go directly to the influent pump station wet well and are not measured in the influent sampler.

Figures 4-25 and 4-26 show that the plant has been in compliance with the monthly and weekly effluent permit limits throughout the period of record for both BOD and TSS.

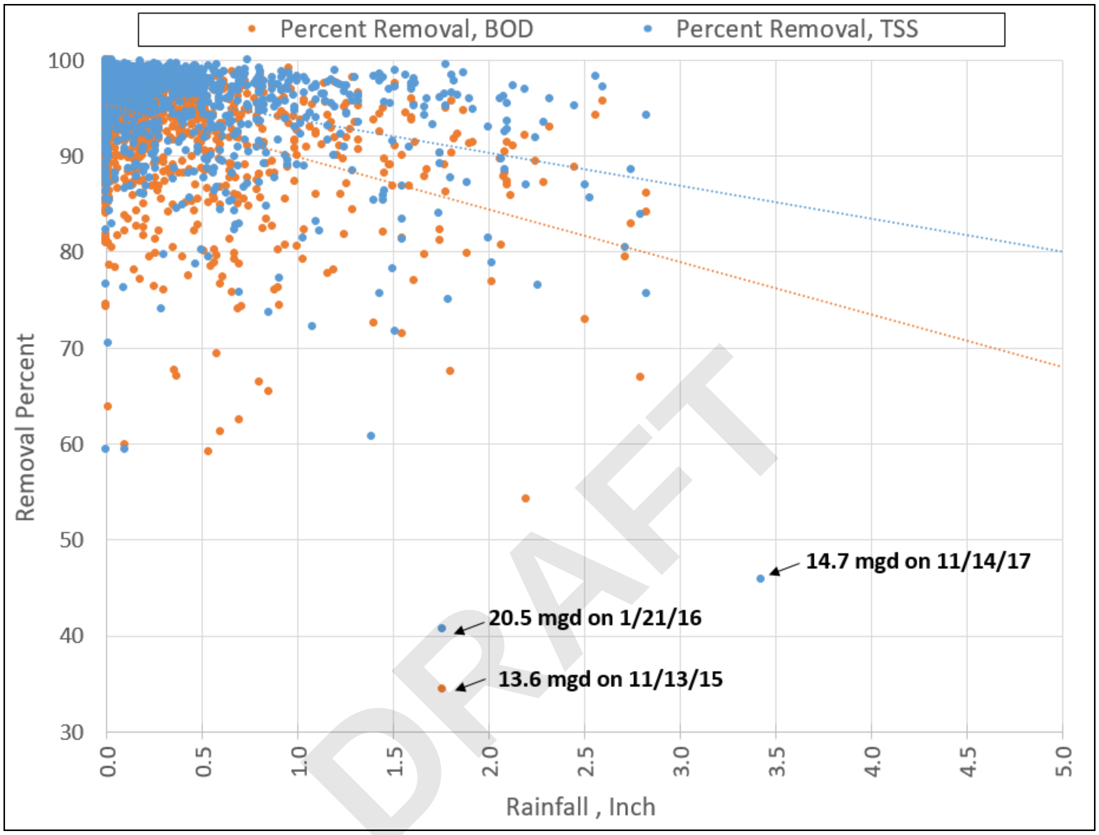


FIGURE 4-23

Daily Removal Percentages

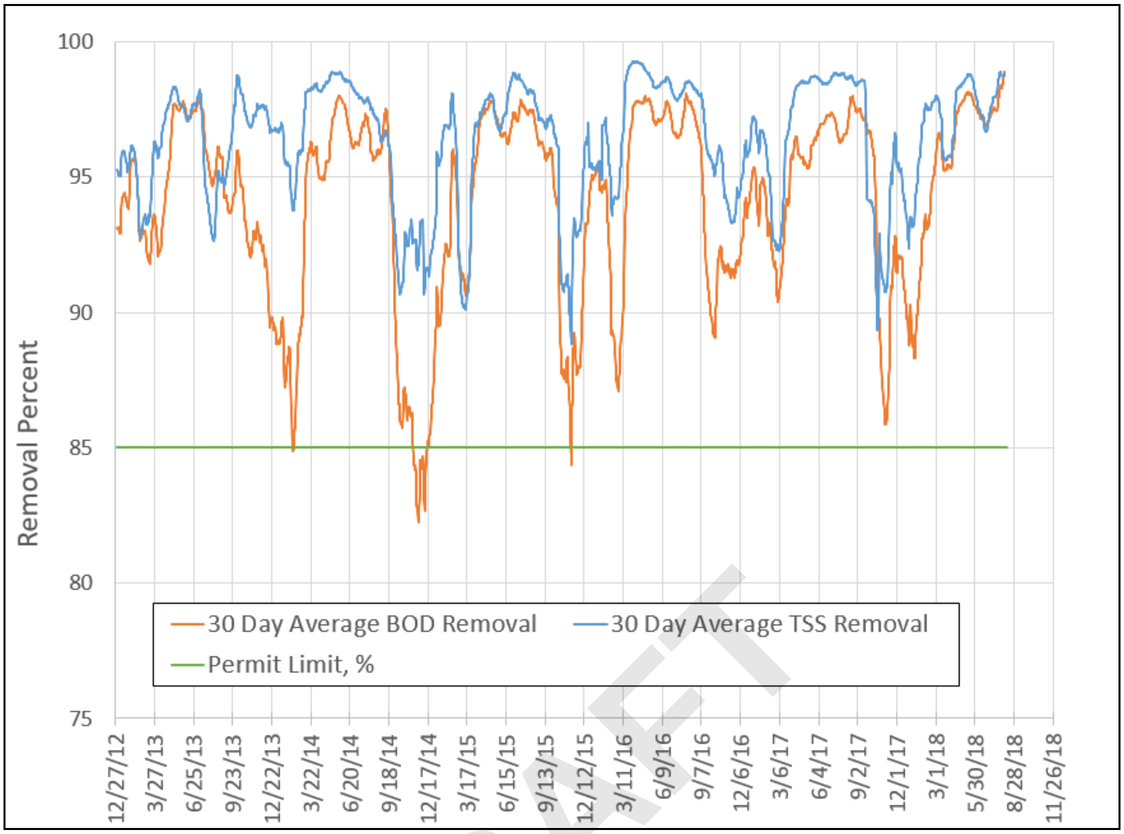


FIGURE 4-24

30-Day Running Average Removal Percentages

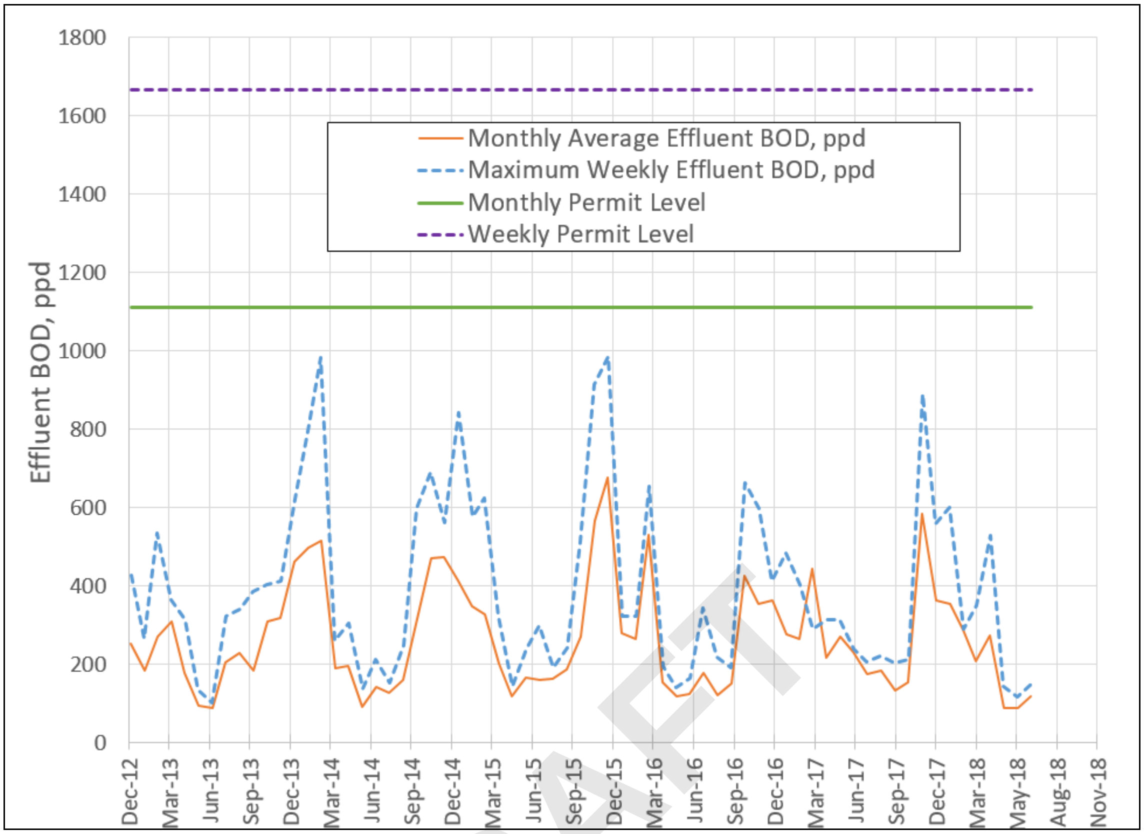


FIGURE 4-25

Effluent BOD Loading

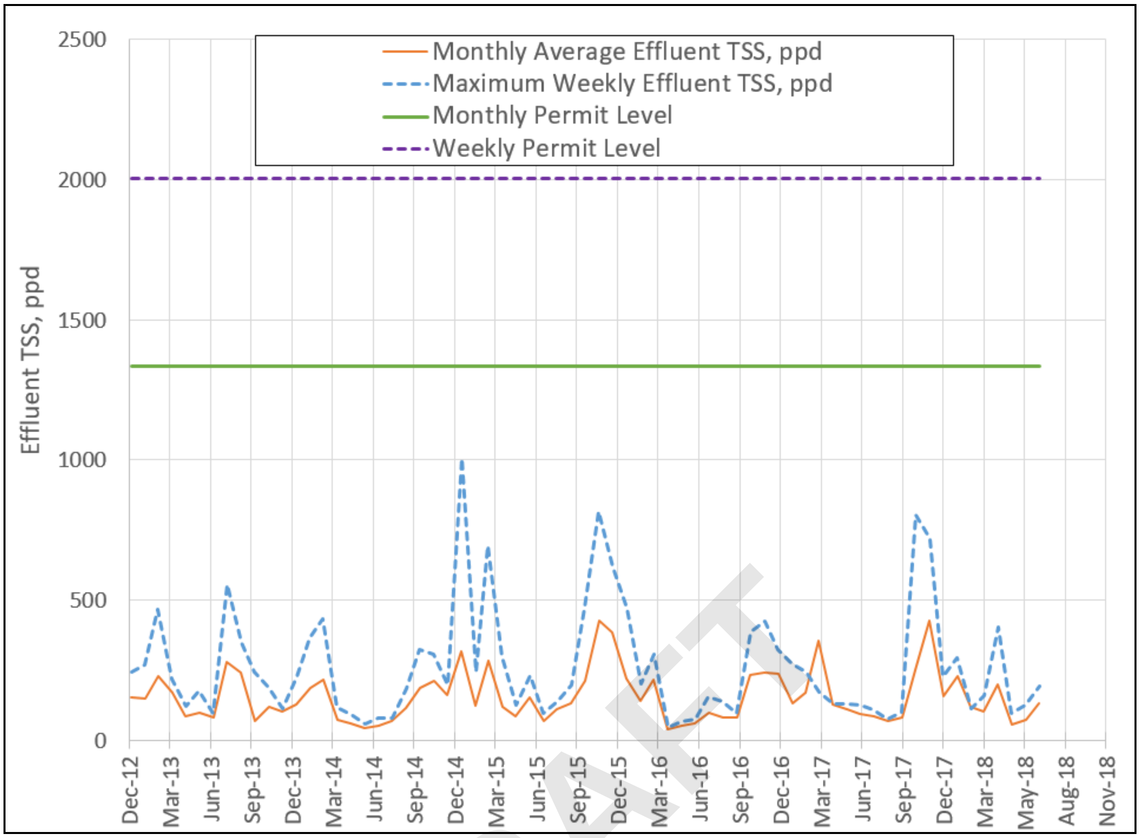


FIGURE 4-26

Effluent TSS Loading

<u>Ammonia Removal</u>

The City's NPDES Permit states that "the Permittee must operate the facility to minimize ammonia in the discharge." In addition, the modeling conducted for the 2017 NPDES Permit showed that effluent ammonia concentrations were within 20 percent of levels that would trigger a numerical ammonia limit. Thus, it is apparent that the WWTP will need to continue to nitrify.

Figure 4-27 presents a comparison of influent, primary effluent, and final effluent ammonia concentrations for the period from 2013 to 2018. Numerical values are presented in Table 4-12. Since septage trucks dump on-site downstream of the influent sampler, these loadings show up in the primary effluent concentrations. The data show that the WWTP has been reducing ammonia. However, it does not completely nitrify on a year-round basis. Limited nitrification occurred in summers of 2016 and 2017. This may be because one aeration tank was taken out of service during that period of time, reducing SRT. Nitrification normally occurs more readily with increasing SRT and increasing temperature.

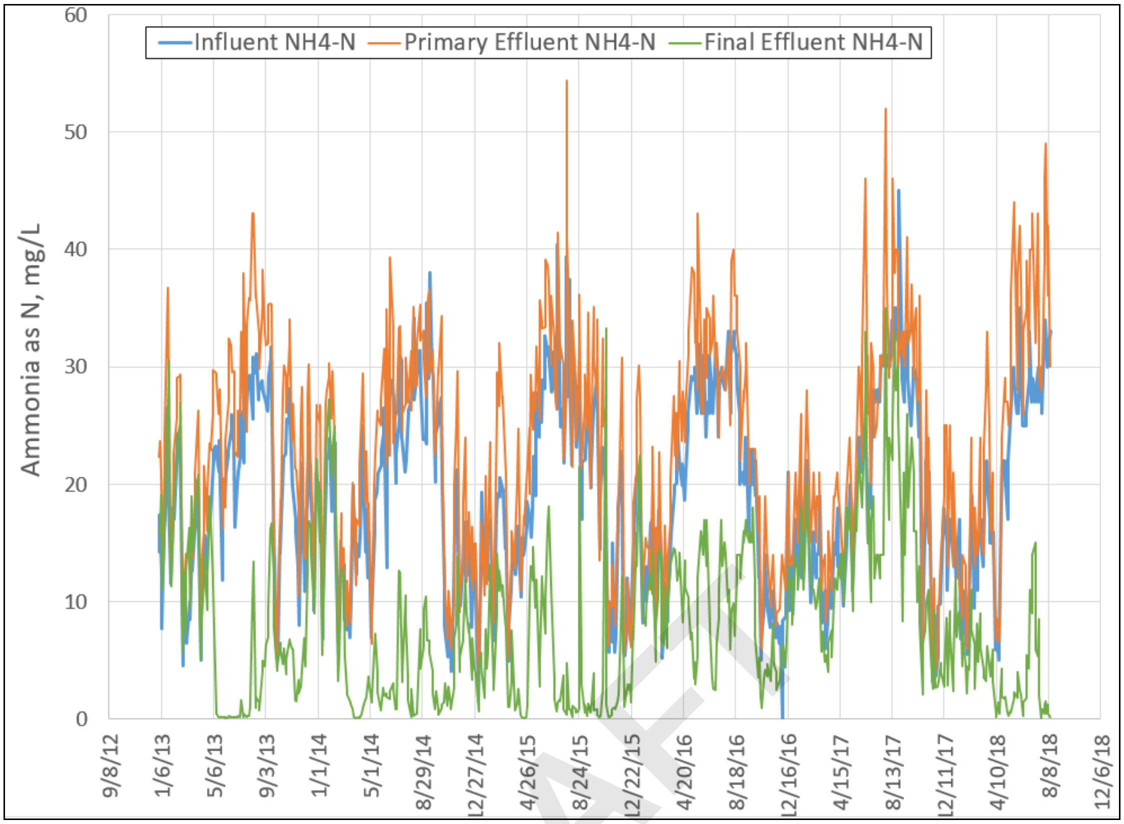


FIGURE 4-27

Ammonia (NH3-N) Concentrations

TABLE 4-12

WWTP Ammonia Concentrations (2013-2018)

Description	Influent NH3-N (mg/L)	Primary Effluent NH3-N (mg/L)	Final Effluent NH3-N (mg/L)
Average	19.4	23.4	8.1
Max	45.0	54.4	35.0
Min	1.6	2.3	0.1
2013 Average	19.3	24.2	9.0
2014 Average	19.1	22.1	5.8
2015 Average	19.3	22.7	4.9
2016 Average	19.3	23.1	9.9
2017 Average	19.3	23.3	13.6
2018 Average (Jan-Aug)	20.4	25.6	4.1

Disinfection

The WWTP currently disinfects with chlorine using liquid/gas chlorine and dechlorinates using sulfur dioxide. The chlorination equipment (Figure 4-28) was installed during the 1977 to 1981 expansion. Dechlorination equipment was installed in a subsequent upgrade. Chorine contact is provided in channels forming an annular ring around Secondary Clarifiers 1 and 2. Chlorine flash mixers are installed in chambers immediately upstream of the contact tanks. Sulfur dioxide is added at the effluent Parshall flume.

FIGURE 4-28

Chlorinators (Currently in the Process of Being Demolished)

The disinfection system is in poor condition and is in the process of being replaced in 2020 with a system utilizing liquid sodium hypochlorite as the disinfectant and calcium thiosulfate as the dechlorinating agent. Data for the new disinfection system are presented in Table 4-13.

TABLE 4-13

Disinfection System Data

Description	Value
Existing Chemical Feed System	
Chlorinator	
Number	2
Capacity (each, ppd)	2,000
Upgraded Chemical Feed System	
Feed Pump	
Type	Peristaltic
Number	
Sodium Hypochlorite	5
Calcium Thiosulfate	3
Capacity (each, ppd)	792
Motor Nameplate (each, hp)	0.25
Storage Tank	
Sodium Hypochlorite	
Number	2
Capacity (gal)	2,500
Calcium Thiosulfate	
Number	2
Capacity (gal)	2,000
Chlorine Contact Tank	
Number	2
Baffled Length (each)	306
Width (each)	5.5
Water Depth (feet)	9.5
Flash Mixer	
Type	Mechanical Turbine Vertical
Number	2
Motor Nameplate (each, hp)	Three existing/one upgraded

The NPDES permit limits for fecal coliform bacteria are 200 per 100 ml on a monthly average basis and 400 per 100 ml on a maximum week. Effluent records for 2013 through 2018 are shown in Figure 4-29. The plant has been in compliance with the monthly and weekly permit limits throughout the period of record.

Figure 4-30 shows that the plant has been consistently in compliance with its chlorine residual permit limit of maximum daily 0.17 mg/L and average monthly 0.08 mg/L. Only one daily exceedance has occurred in 5 years.

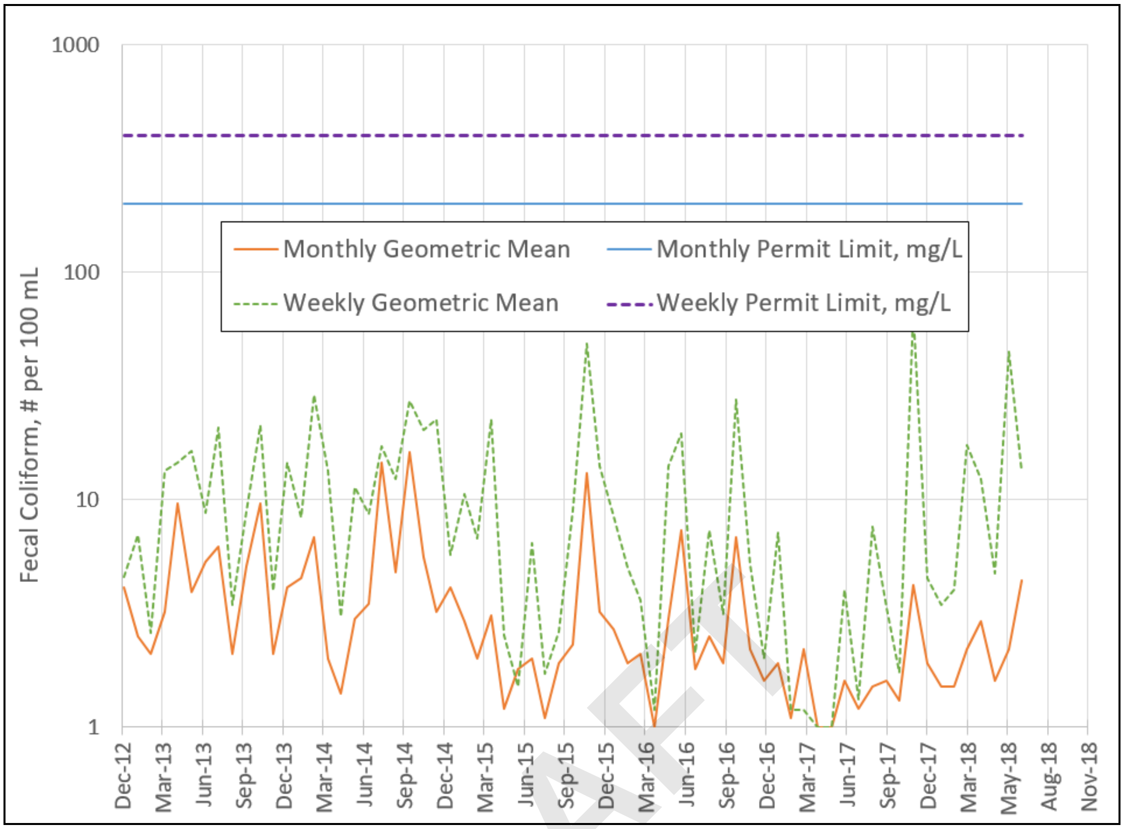


FIGURE 4-29

Effluent Fecal Coliform History – Monthly and Weekly Average

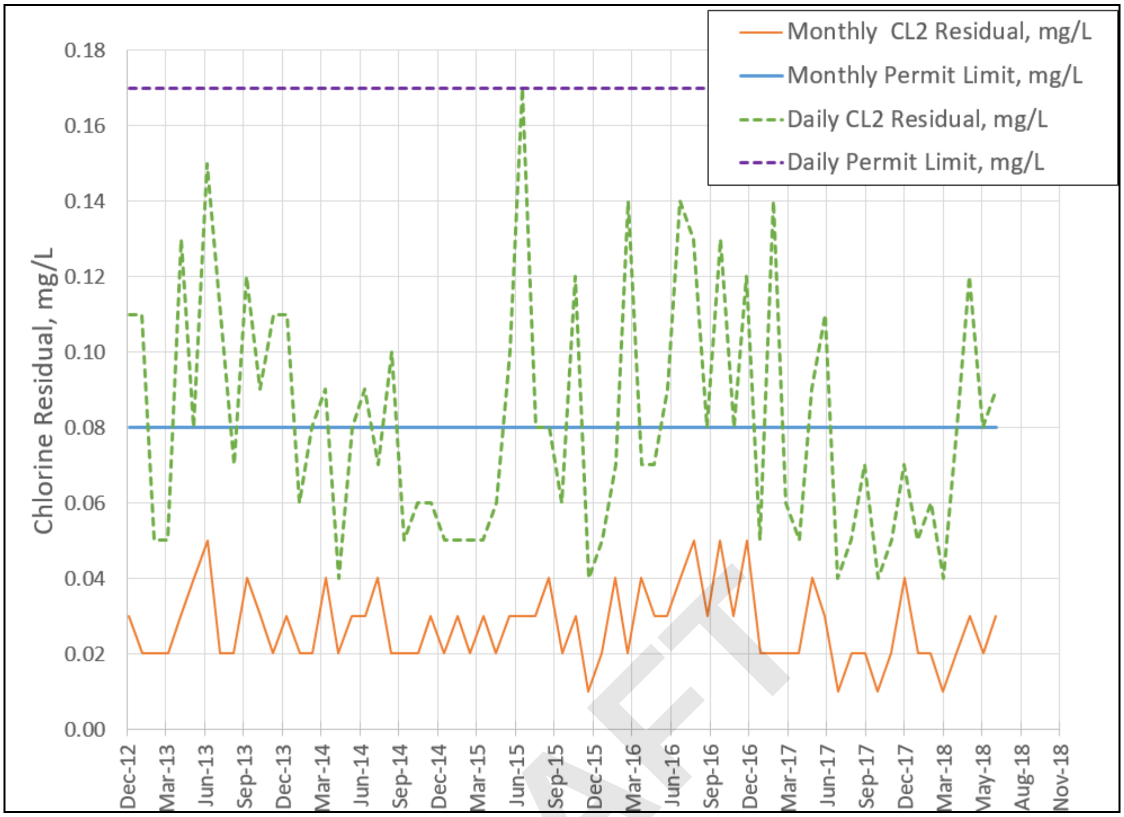


FIGURE 4-30

Effluent Chlorine Residual History – Monthly Average and Daily

Wash Water System

The Wash Water System (also known as the Non-potable Water System, Figure 4-31), including two centrifugal pumps and one upstream simplex basket strainer, is in poor condition, inoperable and has never worked properly. Instead, the backup potable water is used as wash water for plant operations. That makes the WWTP the largest potable water consumer in the City. The wash water system is in the process of being replaced (as of 2020). The upgraded wash water system data are presented in Table 4-14.

TABLE 4-14

Wash Water System Data

Description	Value
Plant Wash Water Pump	
Type	Multi Stage Centrifugal
Number	2
Capacity (each, gpm)	200
Motor Nameplate (each, hp)	15
Plant Wash Water Filter	
Type	Auto Clean
Number	1
Capacity	500 gpm at 1 psi

FIGURE 4-31

Wash Water Pumps (Inactive)

Effluent Flow Monitoring

Two Parshall flumes with ultrasonic level detection provide effluent flow monitoring. The flumes, shown in Figure 4-32, were installed in the 1977 to 1981 upgrade.

FIGURE 4-32

Effluent Parshall Flume

Effluent Outfall

The treatment plant discharges secondary-treated effluent through an outfall at the mouth of the Chehalis River in Grays Harbor. The outfall originally consisted of a 36-inch concrete pipe from the effluent flow monitoring structure to the outfall manhole, 450 feet of 48-inch corrugated metal pipe (CMP) pipe, 155 feet of 36-inch CMP connected to a 36-foot length of ductile iron pipe (DIP), and a diffuser section. The diffuser was a pile-supported 24-inch diameter ductile iron pipe attached perpendicular to the end of the outfall pipe. In 2014, the City conducted the Outfall Replacement Project, in which the CMP outfall was capped and abandoned, the diffuser was demolished and removed, and the new 36-inch ductile iron/HDPE outfall pipe was constructed.

Septage Receiving Station

Septage and sludge from outside the City area is delivered by septage trucks and vactor trucks, and dumped at the hauled waste receiving stations. There are two hauled waste receiving stations: one at the west of the digester control building that conveys both septage to the influent pump station and sludge to the digester, and the other at the south of the secondary clarifier. There is no screening or rock trap or holding tank for received septage.

SOLIDS HANDLING FACILITIES

Sludge Thickening

Primary sludge and WAS may be combined for co-thickening in a 40-foot diameter gravity sludge thickener. The gravity thickener was constructed in the 1977 to 1981 upgrade to the WWTP. The thickener is shown in Figure 4-33.

FIGURE 4-33

Gravity Sludge Thickener

Under some conditions, the primary sludge is transferred to the gravity sludge thickener, while the WAS is transferred to the rotary drum thickener, as described later. Normally, the gravity sludge thickener primarily receives the degritted sludge from the hydrocyclone.

The gravity thickener shows significant age-related deterioration, including corrosion of the drive, mechanism, and electrical conduit. The Gravity Sludge Thickener will be reaching the end of its OUL in 10 years and should be replaced by then. Sludge from the bottom of the thickener is withdrawn using two Vaughan positive displacement vane/rotary lobe pumps (Figure 4-34) located in the solids handling building. A sludge grinder is provided upstream of the thickened sludge feed pumps. These pumps convey sludge to a flocculation tank upstream of the rotary drum thickener or pump directly to the digester bypassing the rotary drum thickener. Polymer solution is added to the sludge immediately upstream of the flocculation tank to promote the solids thickening process.

The sludge thickening system data are presented in Table 4-15.

TABLE 4-15

Sludge Thickening System Data

Description	Value
Gravity Sludge Thickener	
Number of Unit	1
Diameter (feet)	20
Thickened Sludge Pump (To RDT or Digester)	
Type	Positive Replacement
Number	2
Capacity (each, gpm)	75
Motor Nameplate (each, hp)	5
Rotary Drum Thickener	
Number of Unit	1
Capacity (dry ton /day)	7.5
Motor Nameplate (hp)	1.5
Polymer Feed System	
Pump	
Number	3
Capacity (each, gpm)	3.3
Motor Nameplate (each, hp)	1.0
Mixer	
Number	1
Motor Nameplate (hp)	1.0
Thickened Sludge Pump (to Digester)	
Type	Progressing Cavity
Number	2
Capacity (each, gpm)	35
Motor Nameplate (each, hp)	5

FIGURE 4-34

Raw Feed Pumps

Figure 4-35 shows the rotary drum thickener configured to receive the co-thickened primary and waste activated sludge from the gravity sludge thickener or digestate from the digester for recuperative thickening. The rotary drum thickener is capable of producing sludge in the range of 6 to 8 percent solids concentration. Another rotary drum thickener is installed as pretreatment upstream of the screw press, but it has not proven to improve performance of dewatering, so its use has been abandoned.

FIGURE 4-35

Rotary Drum Thickener

Currently there are two progressing cavity pumps receiving thickened sludge from the rotary drum thickener or directly from the gravity sludge thickener and pump to the digester for anaerobic digestion (see Figure 4-36).

FIGURE 4-36

Thickened Sludge Pump

As shown in Figure 4-37, thickened sludge concentrations have been quite variable, averaging about 3.4 percent solids. The high removal rates in 2017 were likely due to the co-thickening with the rotary screen thickener. Since primary sludge concentrations are not measured, it is impossible to determine the solids capture ratio for the thickener. Raw sludge volatile solids concentration has averaged approximately 84 percent (see Figure 4-38).

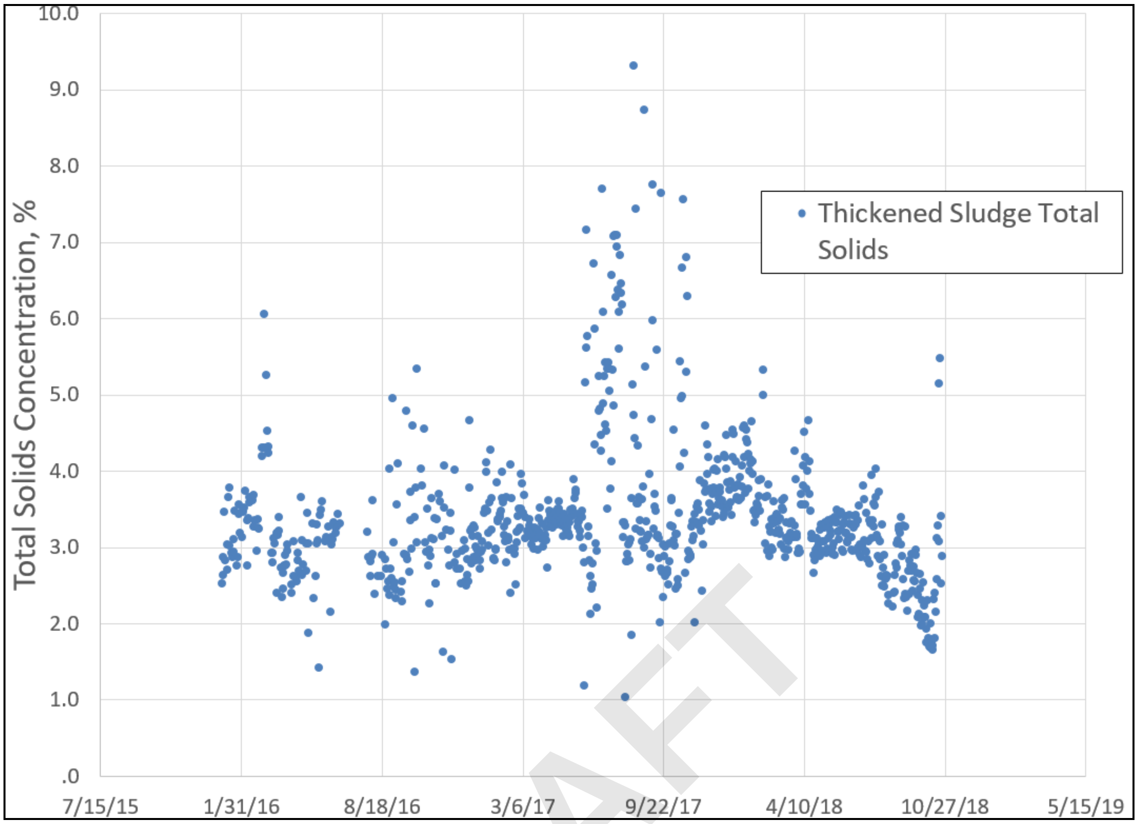


FIGURE 4-37

Thickened Sludge Solids Concentrations

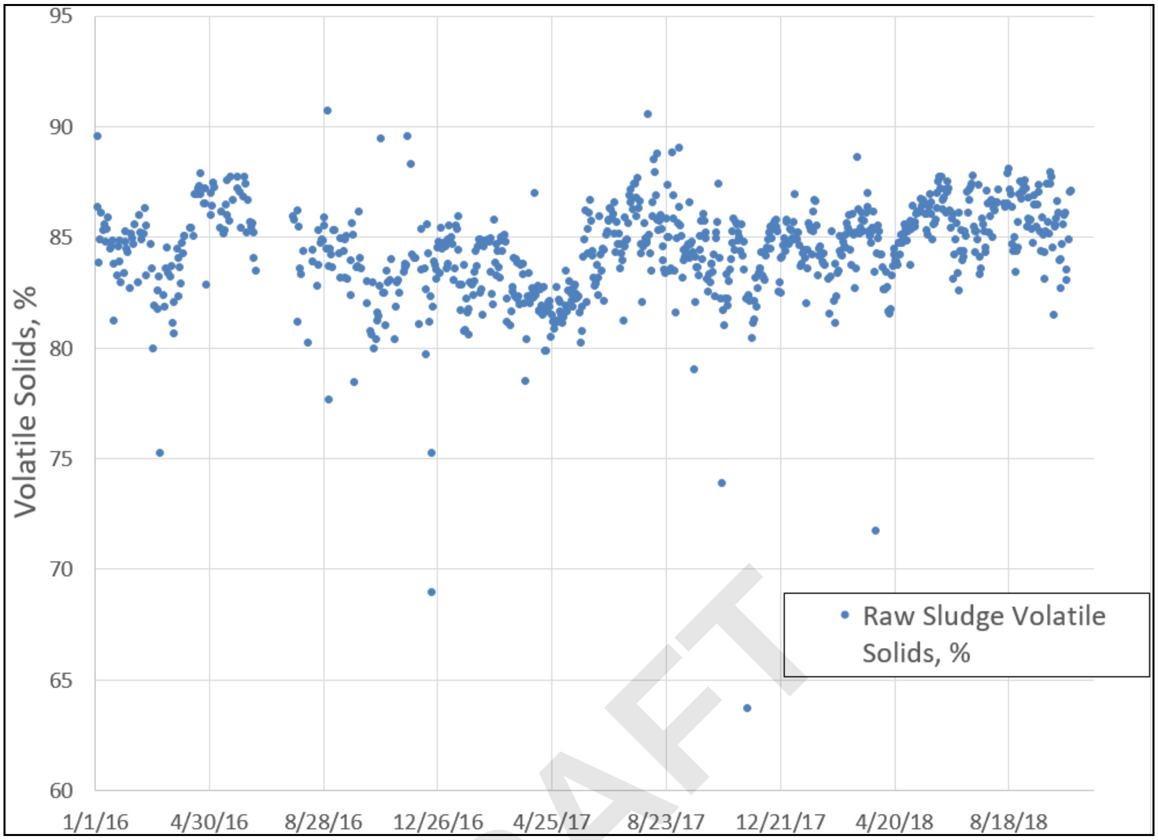


FIGURE 4-38

Raw Sludge Volatile Solids Concentrations

Sludge Digestion

Anaerobic digestion is used to stabilize sludge prior to off-site disposal. The plant has one anaerobic digester (a 50-foot diameter unit, with a 40-foot side water depth), and two older, smaller tanks, each 30-foot in diameter, with a 25-foot side water depth. The two small tanks were constructed and used as digesters in the late 1950s. In the 1977-1981 upgrade, sludge incinerators were built, and these smaller digesters were converted to sludge holding tanks for storage prior to dewatering. In 1990, the large digester was added and the two smaller tanks were converted to storage for digested sludge. The digester is equipped with spiral heat exchanger, boiler, recirculation pumps, waste gas burner, and pumped mixing system (Vaughan Rotamix system). The Vaughan Rotamix digester mixing system replaced the gas sparge mixing system in 2016. Figure 4-39 shows the fixed cover of the digester.

The anaerobic digester exhibits structural deterioration, including cracks in the roof and slab settlement. Although the cracks have been repaired, the risk of additional cracks and other structural issues is considered significant, and could trigger methane leakage and related safety concerns. The gas piping between the Digester building and the flare is vulnerable to failure. In addition, co-generation is not functional due to inadequate cleanup of digester gas.

The digester data are presented in Table 4-16.

TABLE 4-16

Digester Data

Description	Value
Type	Anaerobic
Number of Tanks	1
Geometry	Circular
Diameter (feet)	50
Side Water Depth (max, feet)	43
Rotamix Mixing System	
Pump	
Type	Horizontal Chopper
Number	1
Capacity (each, gpm)	2300 gpm at 36’ TDH
Motor Nameplate (each, hp)	30
Heat Exchanger	
Type	Spiral
Boiler	
Type	Gas Fired
Capacity (mBTU/hr)	1,357
Gas Generator	
Type	Gas Fired
Capacity (kW)	75
Sludge Pump (to Dewatering Screw Press)	
Type	Rotary Lobe
Number	2
Capacity (each, gpm)	100
Motor Nameplate (each, hp)	7.5

FIGURE 4-39

Dome Cover of Large Digester

Plant records for the digester between 2016 through 2018 are shown in Figures 4-40 through 4-43.

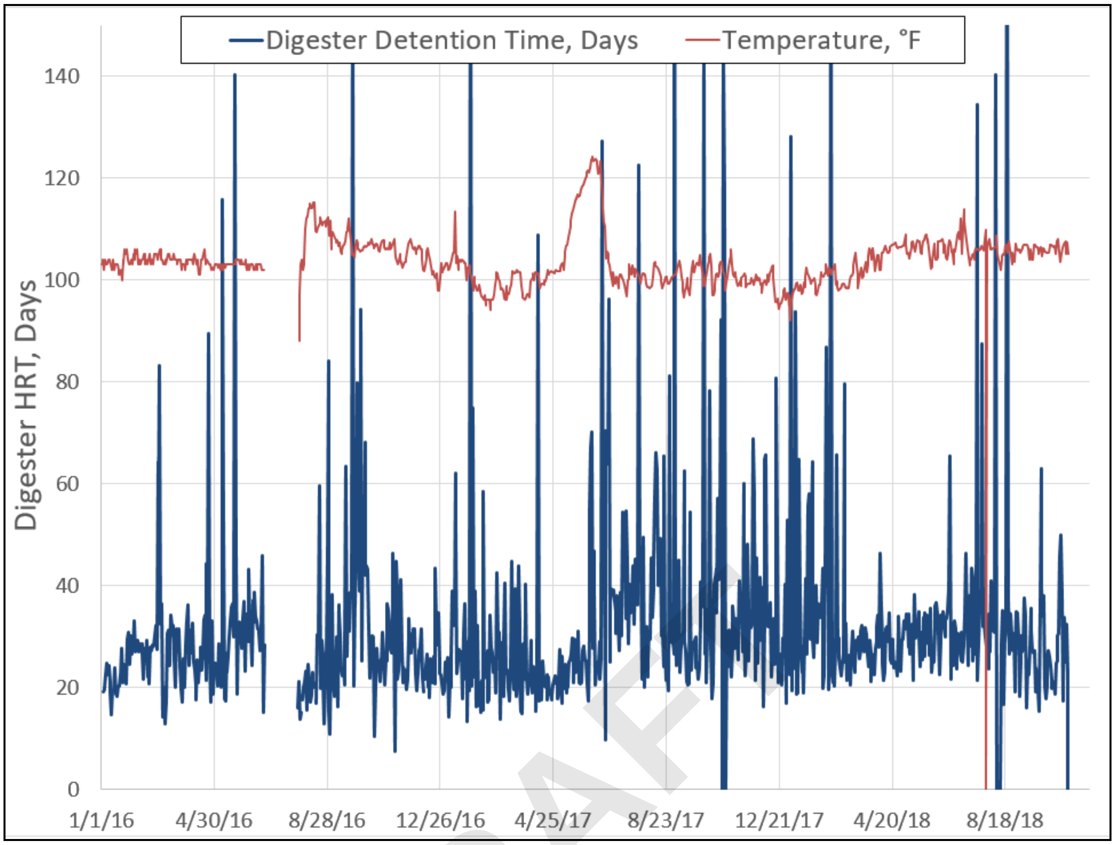


FIGURE 4-40

Digester Hydraulic Detention Time and Temperature

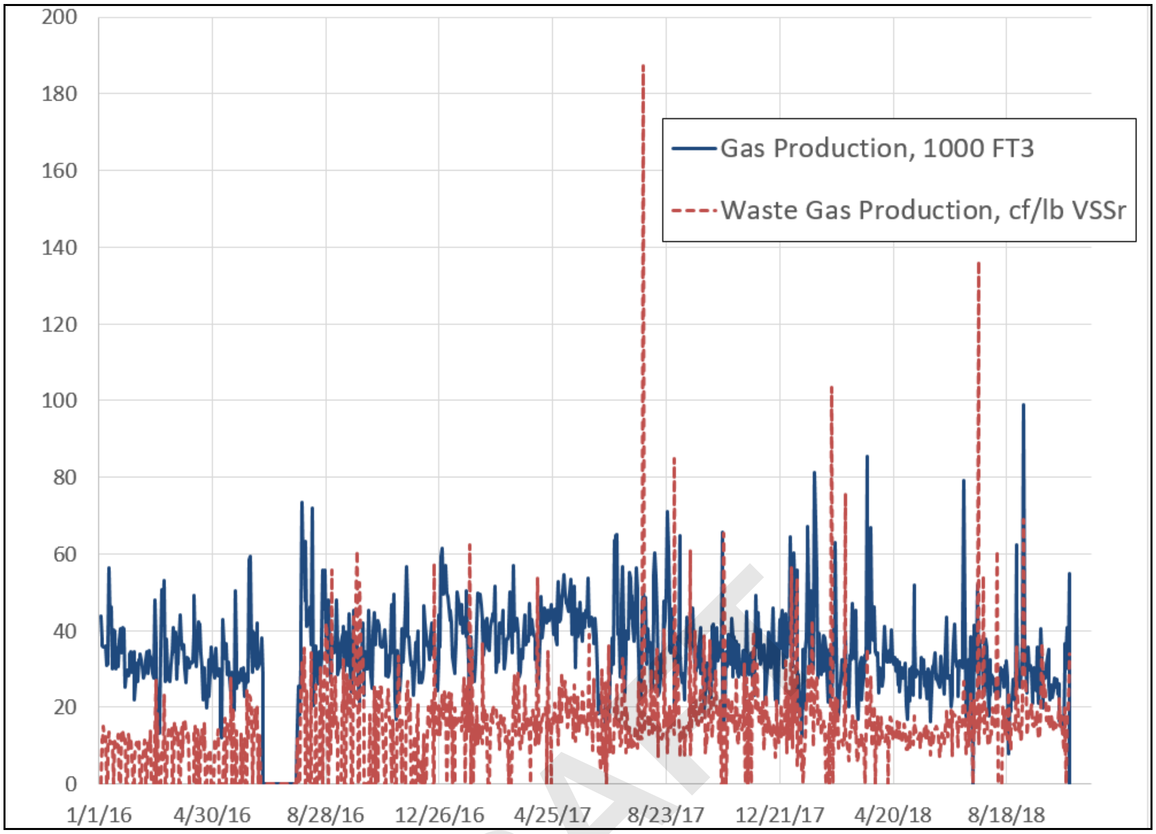


FIGURE 4-41

Digester Gas Production

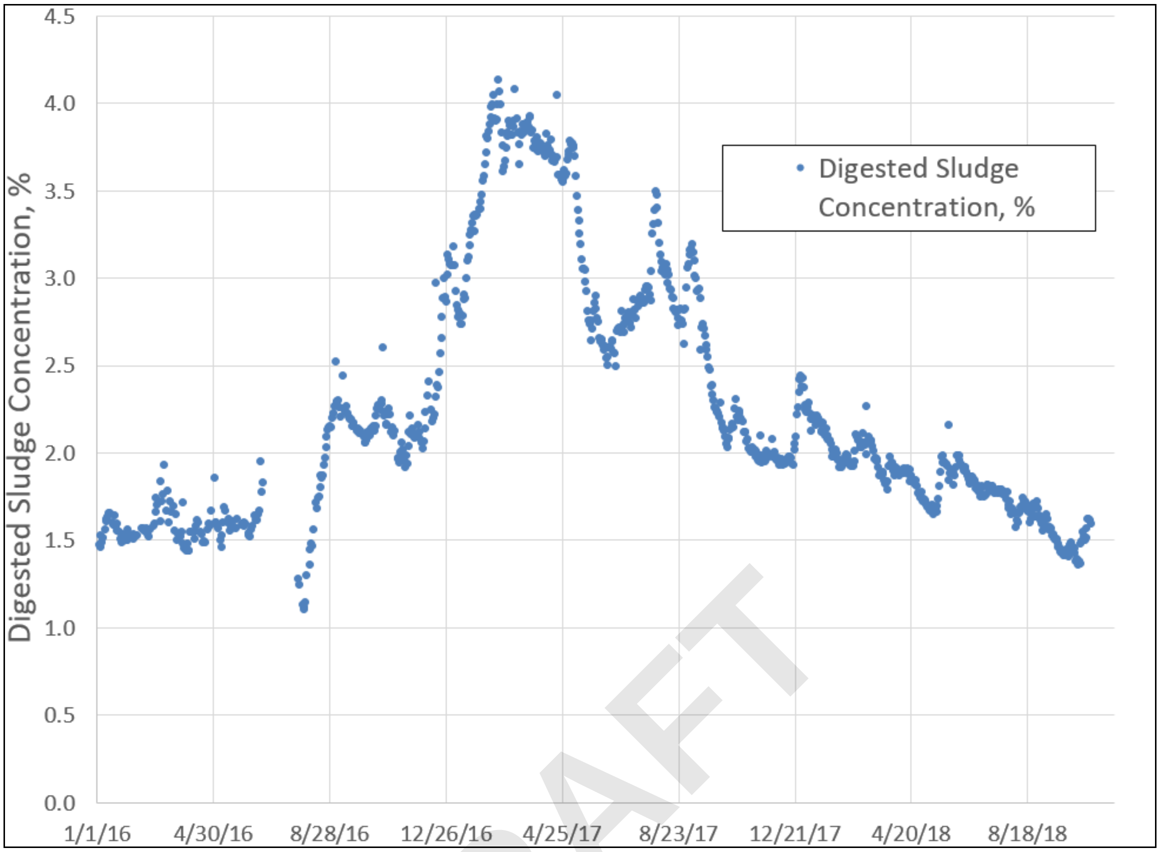


FIGURE 4-42

Digested Sludge Concentrations

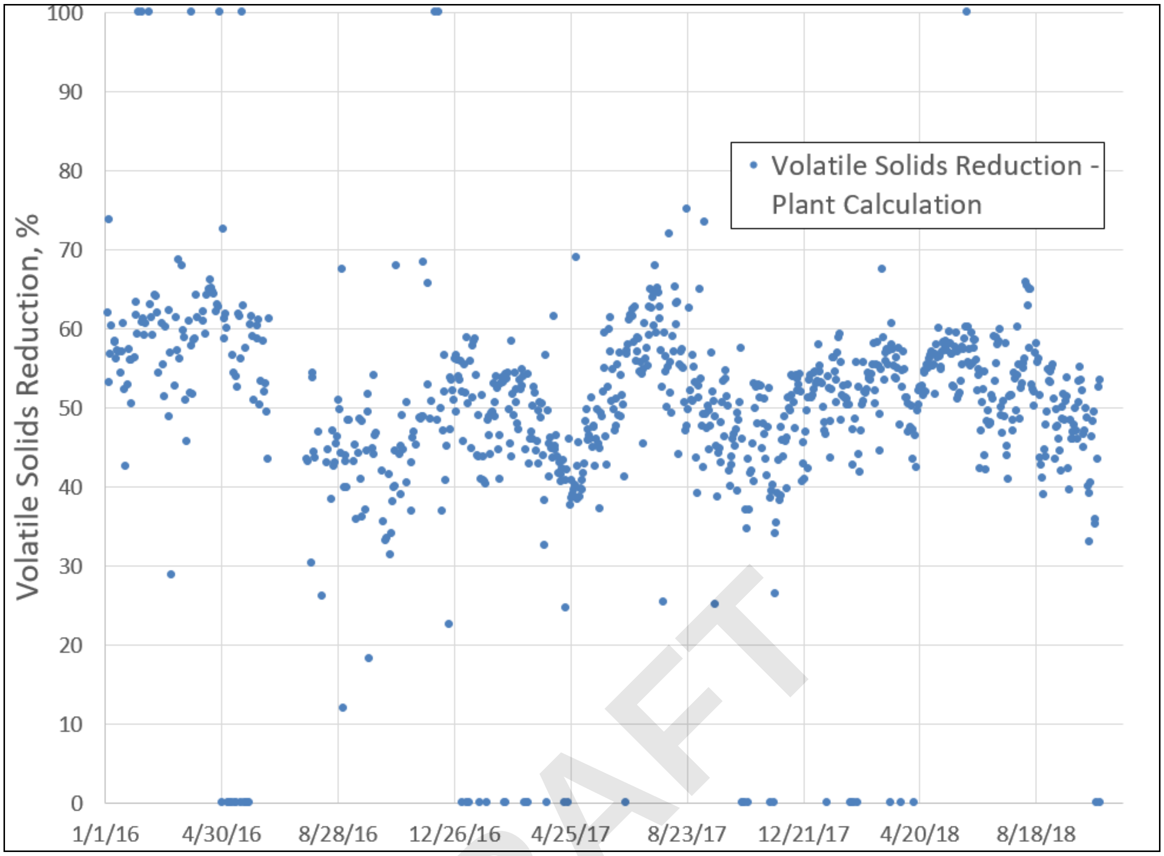


FIGURE 4-43

Digester Volatile Solids Reduction

The measured hydraulic detention time in the digesters has approached an average of 32 days with an average temperature at 103.7 degrees F. Gas production has averaged about 33,700 cubic feet per day. Compared to the apparent volatile solids destruction, this amounts to about 17.3 cubic feet per pound of volatile solids destroyed. This ratio can vary widely in anaerobic digesters and is within the typical range of 10 to 20 cubic feet per pound of volatile solids destroyed.

The EPA sludge treatment manual indicates that the primary cause of variation in the volatile solids reduction is digester temperature, with an optimal value around 95 degrees F (EPA,1979, Figure 6-14). Digested sludge concentrations have averaged approximately 2.3 percent.

Digester volatile solids reduction has typically ranged from 40 to 60 percent, with an average of approximately 51 percent. The plant calculation of volatile solids reduction uses the following equation:

$$VSS_r = [V_{raw}/(V_{raw} - 100)- V_{dig}/(V_{dig} - 100)] / (V_{raw}/(V_{raw} - 100))*100$$

where

VSS_r = Volatile solids reduction, %
V_{raw}= Volatile solids concentration of mixed raw sludge, %
V_{dig} = Volatile solids concentration of digested sludge, %

Figure 4-44 show the relationship of volatile solids reduction to digester hydraulic residence time (HRT)

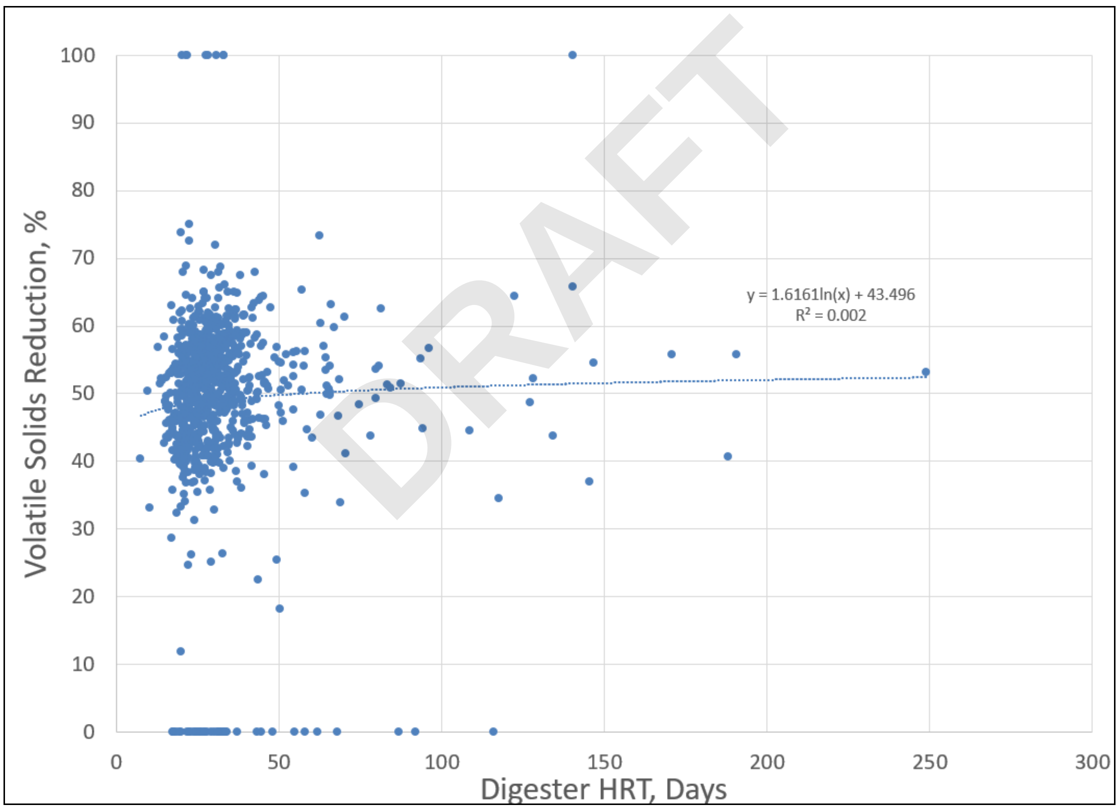


FIGURE 4-44

Digester Volatile Solids Reduction versus HRT, Days

In the past, the digestion system incorporated a cogeneration system with a 75-kW engine-driven generator, allowing the engine to operate on either digester gas or propane. A heat exchanger on the generator exhaust stack captured heat to heat the sludge in the digester. The cogeneration system was abandoned due to the poor quality of the digester gas, and lack of payback in investment to clean up the digester gas.

Regulatory Compliance

Washington State regulates biosolids under Chapter 70.95J of the RCW. The state requirements are found in Chapter 173-308 of the Washington Administrative Code (WAC). Pollutants, pathogen reduction, and vector attraction reduction are the major monitoring parameters for biosolids permits. For the Aberdeen WWTP, the anaerobic digested Class B biosolids are obligated to comply with the following requirements discussed below: (1.) Biosolids pollutant limits, (2.) Pathogen reduction requirements, and (3.) Vector attraction reduction requirements.

1) WAC 173-308-160, Biosolids pollutant limits

WAC-173-308 Table 1 and 3 set, respectively, *(1) the maximum allowable concentration (ceiling limit) of pollutants in biosolids that are applied to the land, (2) the lower pollutant concentration threshold which, when achieved, relieves the person who prepares biosolids and the person who applies biosolids, from certain requirements related to recordkeeping, reporting, and labeling.*

These limits are provided in Table 4-17 along with recent test results for biosolids from Aberdeen. No exceedances of Table 1 or 3 criteria were reported between 2013 and 2017.

TABLE 4-17

Pollutant Test Results (2013-2017) and Regulatory Limits

		Arsenic (As)	Cadmium (Cd)	Copper (Cu)	Lead (Pb)	Mercury (Hg)	Molybdenum (Mo)	Nickel (Ni)	Selenium (Se)	Zinc (Zn)	Lab
WAC-173-308 Table 1		**75**	**85**	**4,300**	**840**	**57**	**75**	**420**	**100**	**7,500**	
WAC-173-308 Table 3		**41**	**39**	**1,500**	**300**	**17**	**75**	**420**	**100**	**2,800**	
2013	1st quarter	ND	ND	330	47	1.6	5.5	18	ND	910	TestAmerica Seattle
	2nd quarter	ND	ND	340	40	0.7	5	16	ND	900	TestAmerica Seattle
	3rd quarter	ND	ND	490	39	1.5	7.6	19	ND	1,100	TestAmerica Seattle
	4th quarter	ND	ND	500	45	1.9	6	18	ND	1,100	TestAmerica Seattle
2014(1)											
2015	1st quarter	ND	ND	420	66	0.82	6.1	20	ND	1,100	TestAmerica Seattle
	2nd quarter	ND	ND	400	37	0.97	4.9	22	ND	1,100	TestAmerica Seattle
	3rd quarter	ND	ND	450	36	1.1	6	20	ND	1,100	TestAmerica Seattle
	4th quarter	ND	ND	410	45	0.87	5.2	19	ND	1,100	TestAmerica Seattle
2016	1st quarter	ND	ND	360	47	0.94	ND	17	ND	1,000	TestAmerica Seattle
	2nd quarter	ND	ND	94	8.7	0.17	1.2	3.3	ND	260	TestAmerica Seattle
	3rd quarter	ND	ND	520	44	0.83	7.2	17	ND	1,300	TestAmerica Seattle
	4th quarter	ND	ND	500	54	0.78	6.9	22	ND	1,200	TestAmerica Seattle
2017	1st quarter	5.1	2.75	540	54	0.763	8.5	22.9	6.8	1,160	ALS Group USA, Corp
	2nd quarter	3.84	2.06	399	36.2	1.000	6.23	17.4	5.9	895	ALS Group USA, Corp
	3rd quarter	3.54	1.79	434	33.6	0.481	7.38	17	7.4	1,100	ALS Group USA, Corp
	4th quarter	4.11	2.03	450	40.2	0.737	7.94	18.5	8.2	1,190	ALS Group USA, Corp

(1) Not Available.

2) *WAC 173-308-170, Pathogen reduction*

Anaerobic digestion. The biosolids must be treated in the absence of air for a specific mean cell residence time at a specific temperature. Values for the mean cell residence time and temperature must be between fifteen days at 35 to 55°C (95 to 131°F) and sixty days at 20°C (68°F).

The 2016-2018 biosolids data reported average residence time of 32 days with temperature of 103.7 degrees F, which meet the 15 days at 95 degrees F pathogen reduction requirements.

Table 4-18 lists the suggested solids retention time from *Metcalf & Eddy* (2002).

TABLE 4-18

Suggested Solids Retention Time for Anaerobic Digestion

Operating Temperature, ºF	SRT (Minimum)	SRT (Desired)
64.4	11	28
75.2	8	20
86	6	14
95	4	10
104	4	10

3) *WAC 173-308-180, Vector attraction reduction*

Volatile Solids Reduction: The mass of volatile solids in the biosolids must be reduced by a minimum of thirty-eight percent. Bench-scale test for anaerobically digested solids: When the thirty-eight percent volatile solids reduction requirement in this subsection cannot be met for anaerobically digested biosolids, vector attraction reduction can be demonstrated by digesting a portion of the previously digested biosolids anaerobically in the laboratory in a bench-scale unit for forty additional days at a temperature between 30 and 37°C (86 and 98.6°F). After the forty-day period, the vector attraction reduction requirement is met if the volatile solids in the biosolids at the beginning of that period are reduced by less than seventeen percent.

The 2016-2018 biosolids data reported volatile solids reduction varies between 40 to 60 percent and with the average value of 51 percent, which is in compliance with the 38 percent vector attraction reduction requirement.

Sludge Dewatering

Located in the solids handling building, two rotary lobe pumps (Figure 4-45) transfer digested sludge from the sludge holding tank to the sludge dewatering units.

The dewatering system data are presented in Table 4-19.

TABLE 4-19

Dewatering System Data

Description	Value
Sludge Pump (from Digester)	
Type	Rotary Lobe
Number	2
Capacity (each, gpm)	100
Motor Nameplate (each, hp)	7.5
Dewatering Unit	
Type	Screw Press
Number of Units	1
Capacity (dry ton/day)	5.1
Sludge Belt Conveyor	
Type	Shaftless Screw

FIGURE 4-45

Press Feed Pumps

The 2005 solids handling upgrade replaced the plant and frame sludge filter press with an FKC screw-dewatering press (Figure 4-46). The capacity of the screw press is 5.1 dry tons per day.

FIGURE 4-46

FKC Screw Press

As shown in Figure 4-47, the dewatered sludge concentration has varied from 16 to 28 percent, with an average of 21 percent. Total digester cake solids production is shown in Figure 4-48.

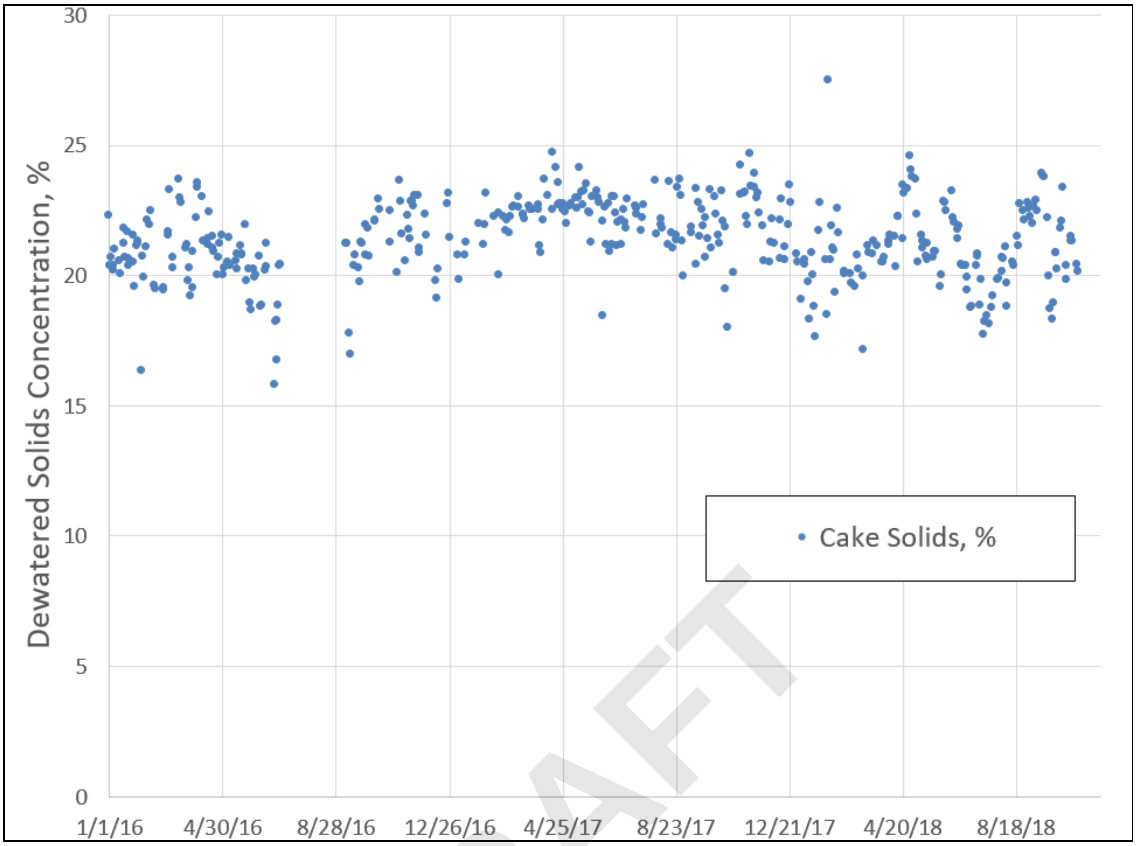


FIGURE 4-47

Dewatering Sludge Concentrations

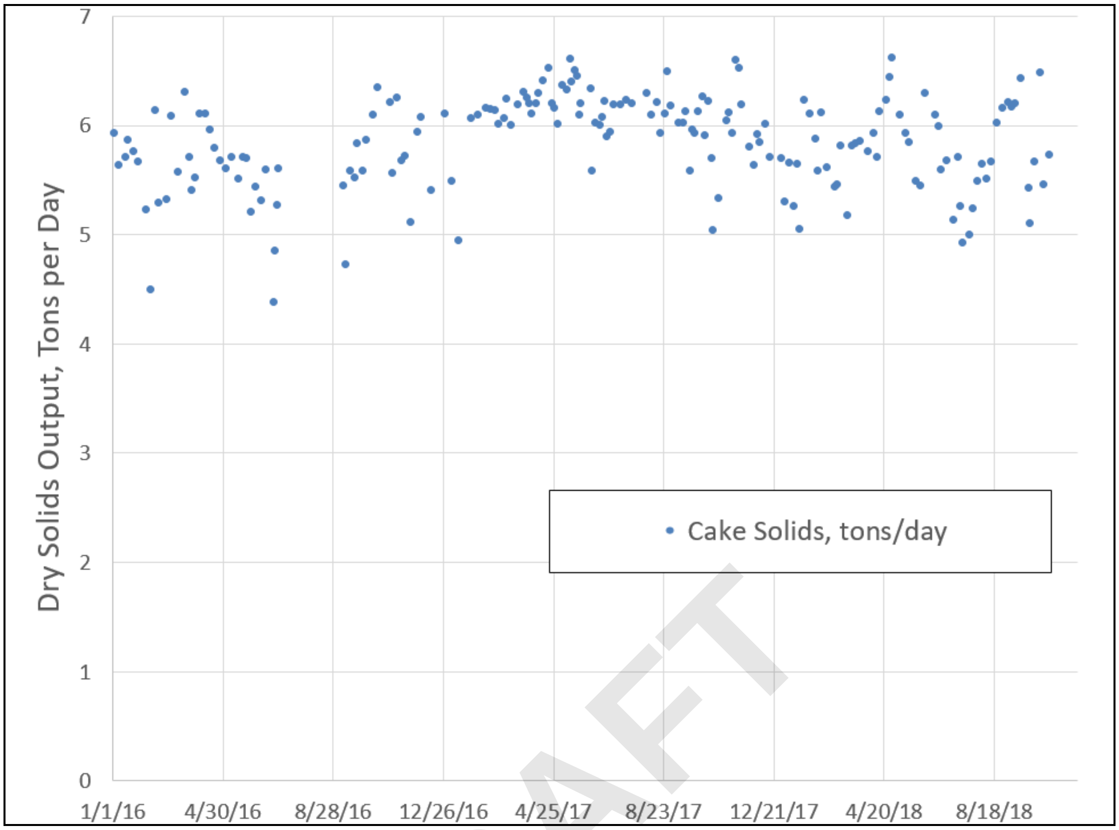


FIGURE 4-48

Dewatering Sludge Cake Production

AUXILIARY FACILITIES

Plant Control System

The treatment facility largely uses local process control typical of facilities designed in the 1970s. Many of the control systems in the plant used relay logic and timer-based control. The plant has a hard-wired intercom system to help operators make manual operational decisions on equipment status. The system was modified in the 2002 to 2004 improvements to install the computer-based central programmable logic control (PLC).

A central panel in the electrical room monitors each of the City's collection system pump stations by telemetry. Parameters monitored at the pump stations include the following:

- Telephone line loss
- Power Failure
- High wet-well level
- Low wet-well level

- Station flooding
- Emergency generator failure
- Intruder entry

If any of these conditions occur, a single red light is energized on the face of the central monitoring panel at the treatment plant and an audible alarm is triggered. The system was installed in 1980 and is dependent on commercial telephone lines. The City installed an automated dialing alarm system that will call a predetermined list of telephone numbers to notify appropriate staff when an unacknowledged alarm event occurs.

The influent and effluent flows are measured at influent manhole and effluent chamber. Two Parshall flumes installed in parallel measure effluent flow. As is typical with Parshall flumes, flow rate is determined by measuring the depth at the upstream end of the flume. An ultrasonic meter mounted over the flume upstream of the throat measures the water depth. The ultrasonic flow transmitters indicate flow locally. Each flow transmitter sends a signal to the chlorine building to pace chlorine feed in proportion to flow. Discrete flow signals are sent to a ratio flow controller that controls the speed of the return activated sludge pumps. The effluent flow signals are combined and recorded.

RAS pumped from each secondary clarifier is monitored by Environtech T500 magnetic flow meters. WAS flows are monitored using a portable clamp-on magnetic flow meter. There is no flow monitoring on the recycle flow from sludge thickening and dewatering.

Standby Power Generator

The plant has a single 500-KW generator set for standby power (Figure 4-49). This is adequate to maintain primary treatment and disinfection, but not the aeration system. The generator is approaching the end of its useful life, and should be replaced within 6 years.

The City also has three portable generators, with a capacities of 60 kW, 75 kW, and 125 kW.

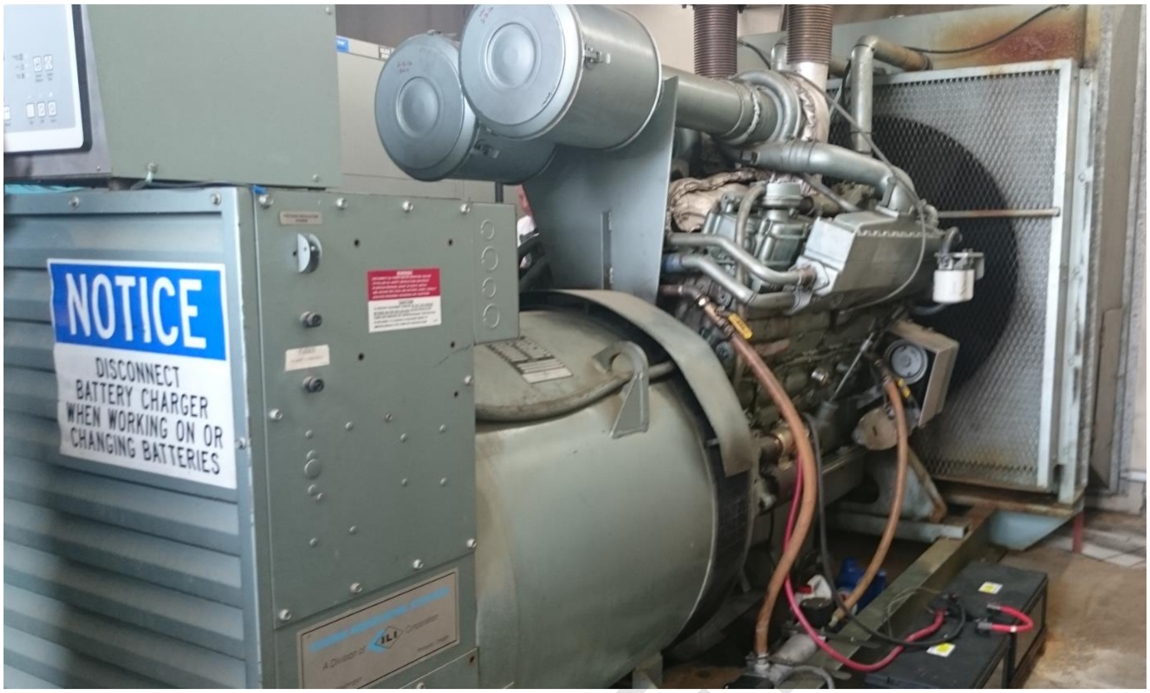


FIGURE 4-49

Standby Power Generator

Laboratory

Most laboratory analyses are performed onsite. The treatment plant's lab is state-certified for general analyses, including BOD, TSS, pH, and chlorine. Other analyses are sent to offsite laboratories. These include: metals, pesticides, and the required biosolids tests.

Staffing (O&M)

Staffing and labor organization at the WWTP is summarized below. 13 employees total work in the Sewer Department (including the WWTP and collection system), with additional assistance provided as needed by the public works maintenance pool.

- Chief Operator
- Administrative Coordinator
  - Maintenance Supervisor
  - Laboratory Supervisor
    - Operator III
      - Operator IV
      - Operator II
      - Operator II

- Operator I
- Operator in Training
- Equipment Tech
- Collection System Supervisor
- Maintenance Worker IV
- Public Works Maintenance Pool

OVERALL CONDITION SUMMARY

Table 4-20 summarizes the condition assessment and necessary improvements to address deficiencies for the WWTP. For more detail, see the Condition Assessment in Appendix D.

TABLE 4-20

WWTP Condition Assessment Summary and Necessary Improvements to Address Deficiencies

	Project Name	Importance	Condition Average Rating	Weighted Rating
1	Influent Pump Station: Rehab Wet Well and Miscellaneous Structural Improvements	5	3	15
2	Influent Pump Station: Ventilation Compliance	5	3	15
3	Influent Manhole: Rehabilitate Mechanical, Piping and Instrumentation	5	2.75	13.75
4	Large Digester: Structural Rehabilitation	5	2.75	13.75
5	Large Digester: Electrical Classification/Compliance	5	2.75	13.75
6	Large Digester: Co-generation System Upgrade	5	2.75	13.75
7	Large Digester: Gas Piping Replacement	5	2.75	13.75
8	Disinfection System: Upgrade (underway)	5	2.6	13
9	Disinfection System: Rehabilitate Settled Conduit	5	2.6	13
10	Primary Sludge Pumps Room: Electrical Improvement	4	3	12
11	Primary Sludge Pumps Room: Ventilation Compliance	4	3	12
12	Headworks: Improve Redundancy Capability	4	2.5	10
13	Aeration Basins: Miscellaneous Structural Improvements	4	2.5	10
14	Aeration Basins: Rehabilitate Settled Conduit	4	2.5	10
15	Dewatering Facilities: Miscellaneous Structural Improvements	4	2.4	9.6
16	Generator: Replace	3	2.8	8.4
17	Generator Room: Miscellaneous Structural Improvements	3	2.8	8.4
18	Generator Room: Ventilation Compliance	3	2.8	8.4
19	Small Secondary Clarifier: Rehabilitate Secondary Clarifier	3	2.75	8.25
20	Small Secondary Clarifier: Rehabilitate Settled Conduit	3	2.75	8.25
21	RAS Pump Room: Ventilation Compliance	3	2.75	8.25
22	WAS Pump Room: Ventilation Compliance	3	2.5	7.5

TABLE 4-20 – (continued)

WWTP Condition Assessment Summary and Necessary Improvements to Address Deficiencies

	Project Name	Importance	Condition Average Rating	Weighted Rating
23	Primary Clarifiers: Rehabilitate Primary Clarifiers & Scum Pump Stations	3	2.5	7.5
24	Sampling System: Sampling Location and Mechanism Optimization	3	2.5	7.5
25	Parshall Flume: Upgrade	3	2.5	7.5
26	Parshall Flume: Rehabilitate Settled Conduit	3	2.5	7.5
27	Gravity Sludge Thickener: Upgrade	3	2.25	6.75
28	Small Digesters: Upgrade	2	3.25	6.5

OVERALL WWTP PERFORMANCE

Table 4-21 presents effluent data for four main performance parameters for the WWTP: BOD, TSS, Fecal Coliform, and ammonia (NH_3-N). Table 4-13 presents data for percent removal of BOD, TSS and ammonia. Average performance for the plant has been quite good with average BOD concentrations of 8.6 mg/L and TSS concentrations of 8.1 mg/L over the 5-year period. Percent removal has averaged 94 percent for BOD and 96 percent for TSS. The average removal percentage for ammonia was 63 percent over the 5-year period. For ammonia removal, the percentages have been calculated based on both influent ammonia and primary effluent ammonia, since the influent sample does not include septage and recycle loading, which add a significant load of ammonia.

TABLE 4-21

Effluent Concentration Data

Description	BOD (mg/L)	TSS (mg/L)	NH_3-N (mg/L)	Description	Fecal Coliform (1/100 ml)
Permit Requirement	*30*	*30*	*N/A*	*Permit Requirement*	*200*
Geometric Mean	N/A	N/A	N/A	Geometric Mean	2.7
Average	8.6	5.0	8.1	Average	46
Maximum Value	23.2	41.4	35.0	Maximum Value	16,400
Minimum Value	1.8	0.3	0.1	Minimum Value	1
2013 Average	8.1	5.8	9.0	2013 Geometric Mean	3.8
2014 Average	9.2	3.8	5.8	2014 Geometric Mean	4.7
2015 Average	9.8	6.3	4.9	2015 Geometric Mean	2.3
2016 Average	8.2	4.2	9.9	2016 Geometric Mean	2.5
2017 Average	9.0	4.8	13.7	2017 Geometric Mean	1.6
2018 Average	6.9	4.8	4.1	2018 Geometric Mean	2.2

TABLE 4-22

Effluent Percentage Removal Data

Description	BOD (%)	TSS (%)	NH_3-N (%) (Based on Influent)	NH_3-N (%) (Based on Primary Effluent)
Permit Requirement	85	85	*N/A*	*N/A*
Average	94	96	54	63
Maximum Value	100	100	100	100
Minimum Value	34	11	-187	-25
2013 Average	95	96	46	58
2014 Average	92	96	66	71
2015 Average	94	96	71	78
2016 Average	95	97	44	53
2017 Average	95	96	27	41
2018 Average	96	97	74	80

REFERENCES

EPA, Process Design Manual, Sludge Treatment and Disposal, EPA 625/1-79-011, September 1979.

Steel and McGhee, Water Supply and Sewerage, McGraw-Hill, 1985.

Metcalf and Eddy, Tchobanoglous, Burton, and Stensel, Wastewater Engineering: Treatment and Reuse, 5th Edition, 2013.

CHAPTER 5

WASTEWATER FLOW AND LOADING PROJECTIONS

INTRODUCTION

Proper design of wastewater treatment and conveyance facilities requires the determination of the quantity and quality of wastewater generated by the users of the City's sanitary sewage collection system.

In this chapter, the existing wastewater characteristics for the service area will be analyzed and projections made for future conditions.

DEFINITIONS OF TERMS

The terms and abbreviations used in the analysis are described below, listed in alphabetical order.

Average Annual Flow

Average annual flow (AAF) is the average daily flow over a calendar year. This flow parameter is used to estimate annual operation and maintenance costs for treatment and lift station facilities.

Average Dry Weather Flow

Average dry weather flow (ADWF) is wastewater flow during periods when the groundwater table is low and precipitation is at its lowest of the year. The dry weather flow period in western Washington normally occurs during June through September. During this time, the wastewater strength is highest, due to the lack of dilution with the ground and surface water components of infiltration and inflow. The higher strength coupled with higher temperatures and longer detention times in the sewer system create the greatest potential for system odors during this time. The average dry weather flow is the average daily flow during the three lowest consecutive flow months of the year. For this study, average flows for July, August, and September are used.

Biochemical Oxygen Demand

Biochemical oxygen demand (BOD) is a measure of the oxygen required by microorganisms in the biochemical oxidation (digestion) of organic matter. BOD is an indicator of the organic strength of the wastewater. If BOD is discharged untreated to the environment, biodegradable organics will deplete natural oxygen resources and result in the development of septic (anaerobic) conditions. BOD data together with other parameters are used in the sizing of the treatment facilities and provide a measurement for determining the effectiveness of the treatment process. BOD is typically expressed as a concentration in terms of milligrams per liter (mg/L) and as a load in terms of pounds per day (lb/d). The term BOD typically refers to a 5-day BOD, often written BOD_5, since the BOD test protocol requires 5 days for completion. BOD_5 of a wastewater is composed of two components – a carbonaceous oxygen demand ($CBOD_5$) and a nitrogenous oxygen demand ($NBOD_5$). The use of $CBOD_5$ as a parameter for evaluating wastewater strength removes the influence of nitrogenous components, including ammonia and organic nitrogen.

Domestic Wastewater

Domestic wastewater is wastewater generated from single- and multi-family residences, permanent mobile home courts, and group housing facilities such as nursing homes. Domestic wastewater flow is generally expressed as a unit flow based on the average contribution from each person per day. The unit quantity is expressed in terms of gallons per capita per day (gpcd).

Equivalent Residential Unit

An equivalent residential unit (ERU) is a baseline wastewater generator that represents the average single-family residential household. An ERU can also express the average annual flow contributed by a single-family household in units of gallons per day, or an annual average loading (of 5-day biochemical oxygen demand or total suspended solids) contributed by a single-family household in units of pounds per day.

Infiltration

Infiltration is groundwater entering a sewer system by means of defective pipes, pipe joints, or manhole walls. Infiltration quantities exhibit seasonal variation in response to groundwater levels. Storm events or irrigation trigger a rise in the groundwater levels and increase infiltration. The greatest infiltration is observed following significant storm events after prolonged periods of precipitation. Since infiltration is related to the total amount of piping and appurtenances in the ground and not to any specific water use component, it is generally expressed in terms of the total land area being served. The unit quantity generally used is gallons per acre per day.

Inflow

Inflow is surface water entering the sewer system from yard, roof and footing drains, from cross connections with storm drains, and through holes in manhole covers. Peak inflow occurs during heavy storm events when storm sewer systems are taxed beyond their capacity, resulting in hydraulic backups and local ponding. Inflow, like infiltration, can be expressed in terms of gallons per capita day or gallons per acre per day.

WWTP flow records are utilized to characterize infiltration and inflow (I/I) in the Aberdeen system in terms of peak hour, peak day, maximum month, and average annual I/I.

Maximum Month Flow (Treatment Design Flow)

Maximum month flow (MMF) is the highest monthly flow during a calendar year. It typically occurs in months with maximum rainfall. In western Washington, the maximum month flow normally occurs in the winter due to the presence of more I/I. This wintertime flow is composed of the normal domestic, commercial, and public use flows with significant contributions from inflow and infiltration. The predicted maximum month flow at the end of the design period is used as the design flow for sizing treatment processes and selecting treatment equipment.

Non-Residential Wastewater

Non-residential wastewater is wastewater generated from commercial activities, such as restaurants, retail and wholesale stores, service stations, office buildings, and industrial flow (process wastewater, rinse water, and other industrial activities). Non-residential wastewater quantities for commercial and industrial wastewater are expressed in this Plan in terms of equivalent residential units (ERUs).

Peak Hour Flow

Peak hour flow (PHF) is the highest hourly flow during a calendar year. The peak hour flow in western Washington usually occurs in response to a significant storm event preceded by prolonged periods of rainfall which have previously developed a high groundwater table in the service area. Peak hour flows are used in sizing the hydraulic capacity of wastewater collection, treatment, and pumping components. Peak hour flow is typically determined from treatment facility flow records and projected future flows.

Total Suspended Solids

Total suspended solids (TSS) is a measure of the solid matter carried in the waste stream. The total suspended solids in a wastewater sample is determined by filtering a known volume of the sample, drying the filter paper, and measuring the increase in weight of the filter paper. TSS is expressed in the same terms as BOD; milligrams per liter for concentration and pounds per day for mass load. The amount of TSS in the wastewater is used in the sizing of treatment facilities and provides another measure of the treatment effectiveness. The concentration of TSS in wastewater affects the treatment facility biosolids production rate, treatment and storage requirements, and ultimate disposal requirements.

Total Kjeldahl Nitrogen

Total Kjeldahl nitrogen (TKN) is the combination of organically bound nitrogen and ammonia in wastewater. The organically bound nitrogen must be released from the organic matter by a process of digestion prior to analysis. This form of nitrogen is usually much higher on influent (untreated waste) samples then effluent samples. In most domestic wastewater facilities, the biological activity breaks down the organic matter releasing and/or consuming the nitrogen as energy in the process. Total nitrogen is the combination of organic nitrogen and inorganic nitrogen (NH_3, NO_3-, NO_2-).

Wastewater

Wastewater is water-carried waste from residential, business, industry, and public use facilities together with quantities of groundwater and surface water which enter the sewer system through defective piping and direct surface water inlets. The total wastewater flow is quantitatively expressed in millions of gallons per day (mgd).

POPULATION

EXISTING POPULATION

The City of Aberdeen WWTP receives wastewater from the Cities of Aberdeen, Cosmopolis, and the Stafford Creek Corrections Center (SCCC). The population in these communities between 2013 and 2019 has been relatively stable as indicated in Table 5-1.

The Washington State Office of Financial Management develops population forecasts for each county in the state. While the State of Washington experienced substantial population growth in the past 7 years, the population in the City's sewer service area has remained essentially constant for the past 7 years.

TABLE 5-1

Historical Population Data (2013 to 2019)

	Population						
	2013	**2014**	**2015**	**2016**	**2017**	**2018**	**2019**
City of Aberdeen (1)	16,860	16,850	16,780	16,780	16,740	16,760	16,880
City of Cosmopolis (1)	1,650	1,645	1,640	1,650	1,660	1,665	1,680
SCCC (2)	2,150	2,150	2,150	2,150	2,150	2,150	2,150
Total	**20,660**	**20,645**	**20,570**	**20,580**	**20,550**	**20,575**	**20,710**

(1) Source: Washington State Office of Financial Management.
(2) Source: Reported by the City, including full capacity of 1,972 inmates and population equivalent of employees.

POPULATION FORECASTS

The OFM data implies an annual growth rate for Aberdeen of 0.21 percent. However, per discussion with City staff, an annual growth rate of 1.0 percent was used to project future City population, for conservatism.

The City of Cosmopolis and the SCCC are wholesale customers that are served by Aberdeen. In March 2013, the City of Aberdeen and Cosmopolis signed a municipal wastewater treatment contract by which Aberdeen agrees to provide wastewater treatment and accept a maximum of 98.3 mgd of annual wastewater until 2023. A copy of this contract is included in Appendix E.

The Stafford Creek Correctional Center, owned and operated by the Washington State Department of Corrections (DOC), is another wholesale customer of the City. SCCC began service in 2000. SCCC staff reported that the prison reached its full capacity of 1,972 in March 2004. Since then, the population has varied, but remains close to its full capacity. There was a proposal to expand the facility by 300 additional beds, but funding limitations have reduced the priority for this action. As such, staff reports that there is no anticipated expansion of capacity at SCCC in the near future.

One area outside Aberdeen's city limits that may be added to the City's sewer service area is the Central Park area, which has a designated population of 2,667 by July 2018 according to the United States Census Bureau. If the sewer system was extended to convey Central Park wastewater to the Aberdeen WWTP, the flow rate may be limited by the existing pump station and force main capacity (which will be discussed later in the chapter). Aberdeen city staff anticipate that the Central Park extension plan would not be completed for 20 years. For this analysis, it is assumed that portions of Central Park will be connected to the City's collection system by 2028 (50 percent connected by 2028, 65 percent by 2033, and 80 percent by 2038).

The City of Cosmopolis and the community of Central Park were assumed to have the same 1.0 percent growth rate as the City of Aberdeen.

For consideration of the possibility of wastewater treatment regionalization, flow and loadings from the City of Hoquiam are presented. A growth rate of 0.77 percent was estimated in the City's 2009 Comprehensive Plan and is used for these projections.

Table 5-2 presents 5-, 10-, 15-, and 20-year population projections for both the City-only and Regional plans.

TABLE 5-2

Projected Population in Aberdeen Wastewater Collection System Service Area (with Hoquiam)

Service Area	Population				
	2018	**2023**	**2028**	**2033**	**2038**
City of Aberdeen	16,760	17,615	18,513	19,458	20,450
City of Cosmopolis	1,665	1,750	1,839	1,933	2,032
SCCC (1)	2,150	2,150	2,150	2,150	2,150
Central Park	0	0	1,473 (2)	2,013 (3)	2,603 (4)
Aberdeen Plant Total	**20,575**	**21,515**	**23,976**	**25,553**	**27,235**
Hoquiam	8,560	8,895	9,242	9,604	9,979
Regional Total	**29,135**	**30,410**	**33,218**	**35,157**	**37,215**

(1) Data reported by the City, including full capacity of 1,972 inmates and population equivalent of employees.
(2) It was assumed 50 percent of the total population (2,946) is connected by 2028.
(3) It was assumed 65 percent of the total population (3,096) is connected by 2033.
(4) It was assumed 80 percent of the total population (3,254) is connected by 2038.

EXISTING WASTEWATER FLOWS AND LOADING

WWTP records for the 7-year period from 2013 through 2019 were reviewed and analyzed to determine current wastewater characteristics and influent loadings. Current wastewater flows and loadings were then used in conjunction with projected population data to determine projected future wastewater flows and loadings.

WASTEWATER FLOWS AT CITY OF ABERDEEN WWTP

Table 5-3 summarizes reported WWTP flows for the 7-year period of 2013 to 2019. The average dry weather flow was reducing over that period, indicating decreasing infiltration. The monthly average WWTP flows ranged from 1.80 to 6.83 mgd. The peak day flow (PDF) typically occurs between December and March. The comparison of plant influent and rainfall on Figure 5-3 shows that wastewater flow is strongly influenced by rainfall. The peak day flow of 20.60 mgd occurred during a major storm event on January 5, 2015. Aberdeen does not record peak hourly flows. A peak hour flow of 22.99 mgd was reported on October 20, 2016.

TABLE 5-3

Historical WWTP Influent Flows (2013 to 2019)

Flow Type	2013	2014	2015	2016	2017	2018	2019
Average Dry Weather Flow(1)	2.56	2.64	2.05	2.15	2.02	1.93	2.14
Annual Average Flow	3.25	3.62	3.45	3.99	3.87	3.52	2.96
Maximum Monthly Flow	4.30	5.50	5.54	6.60	6.83	6.00	4.77
Peak Day Flow	10.98	13.15	20.60	20.50	14.67	12.86	17.60
Peak Hour Flow (2)	N/A	N/A	N/A	22.99	N/A	N/A	23.0
Annual Rainfall	64.24	91.01	84.38	106.39	106.26	79.26	50.88

(1) Average of July, August, and September.
(2) Peak hour flow is only available for certain days.

Monthly discharge monitoring report (DMR) data for this period are provided in Appendix F and summarized in Table 5-4. Some unusually high loading days were recorded. It is possible that these extreme values were the results of an extraordinary loading from one or more sources, but there is evidence that the unusually high values were the result of unrepresentative sampling. The mixed liquor inventories in the aeration basins for these days were not unusually high. If the loading had been the result of an extraordinary loading, the inventory would be expected to increase dramatically. Because of this, these high unrepresentative concentrations (>500 mg/L) will not be included in the analysis.

Graphical representations of daily, peak day of month, and average monthly WWTP flows for the period from 2013 through 2019 are shown on Figures 5-1 through 5-3. As shown on Figure 5-3, the data indicate that the permit limit of 9.9 mgd for the existing facility has not been exceeded as a monthly average over the period of 2013 to 2019.

TABLE 5-4

Summary of Discharge Monitoring Reports WWTP Influent Monthly Averages

	Flow			Average Monthly					
	Avg. Monthly	Max. Daily	Min. Daily	BOD_5		TSS		NH_3-N	
Month	(mgd)	(mgd)	(mgd)	(mg/L)	(lb/d)	(mg/L)	(lb/d)	(mg/L)	(lb/d)
Jan-13	4.30	10.82	2.57	129	4,045	123	3,689	15.42	515
Feb-13	3.83	8.60	2.62	129	4,048	122	3,671	17.12	446
Mar-13	4.19	7.88	2.70	129	4,244	118	3,953	12.63	430
Apr-13	4.11	7.88	2.47	154	4,910	152	4,916	13.96	433
May-13	2.82	4.49	2.15	192	4,366	188	4,318	20.73	477
Jun-13	2.59	3.81	1.99	218	4,572	213	4,515	21.64	452
Jul-13	2.14	2.70	1.75	212	3,768	217	3,863	25.76	457
Aug-13	2.03	3.27	1.83	249	4,247	288	4,536	28.56	463
Sep-13	3.20	10.98	1.82	211	4,362	267	4,535	20.92	482
Oct-13	2.99	6.63	2.04	189	4,353	195	4,553	21.93	488
Nov-13	3.50	9.07	2.35	154	4,219	142	4,083	17.04	459
Dec-13	3.36	6.30	2.36	158	4,491	173	4,538	17.26	511
Jan-14	3.85	12.19	2.45	154	4,644	148	4,477	18.08	444
Feb-14	4.57	10.87	2.40	147	4,849	141	4,833	14.97	485
Mar-14	5.50	12.53	2.81	147	5,844	169	6,704	12.17	526
Apr-14	3.63	6.95	2.51	217	4,293	320	4,260	17.08	433
May-14	3.31	8.21	2.20	165	4,163	154	3,923	16.85	458
Jun-14	2.06	2.36	1.85	247	4,212	216	3,691	24.68	437
Jul-14	1.93	2.41	1.71	229	3,695	179	2,883	25.78	428
Aug-14	2.05	2.90	1.71	249	4,075	211	3,368	29.43	475
Sep-14	2.33	3.93	1.68	225	4,251	162	3,150	27.70	520
Oct-14	4.47	9.57	2.06	133	3,616	110	3,167	16.59	453
Nov-14	4.67	9.70	2.22	102	3,565	84	2,997	12.16	420
Dec-14	5.12	13.15	2.84	92	3,365	77	2,888	13.01	455
Jan-15	4.92	20.60	2.61	119	3,818	109	3,610	11.33	372
Feb-15	3.92	9.70	2.41	167	4,795	176	4,567	15.10	408
Mar-15	3.68	9.95	1.94	148	4,063	120	3,314	13.52	379
Apr-15	2.99	4.25	2.34	164	4,024	133	3,262	15.19	376
May-15	2.13	3.09	1.87	265	4,667	244	4,302	21.37	392
Jun-15	1.97	2.19	1.61	289	4,742	280	4,568	29.76	484
Jul-15	1.92	2.52	1.69	394	5,240	471	4,917	29.14	468
Aug-15	1.89	2.90	1.62	399	5,358	467	5,197	26.55	410
Sep-15	2.34	2.83	2.07	334	4,532	416	3,917	24.18	471
Oct-15	3.39	13.48	2.00	235	5,348	297	6,799	21.10	502
Nov-15	5.54	13.64	2.52	164	5,619	169	6,264	12.93	470
Dec-15	6.74	16.31	2.95	155	6,580	151	6,906	9.09	464

TABLE 5-4 – (continued)

Summary of Discharge Monitoring Reports WWTP Influent Monthly Averages

Month	Flow			Average Monthly					
	Avg. Monthly (mgd)	Max. Daily (mgd)	Min. Daily (mgd)	BOD_5 (mg/L)	BOD_5 (lb/d)	TSS (mg/L)	TSS (lb/d)	NH_3-N (mg/L)	NH_3-N (lb/d)
Jan-16	5.82	20.50	2.53	155	5,590	150	5,930	15.42	457
Feb-16	4.87	8.45	3.13	156	5,889	164	6,147	14.12	511
Mar-16	6.00	14.34	2.90	125	4,687	116	4,606	12.36	576
Apr-16	2.71	3.25	2.30	272	5,785	278	5,573	21.33	502
May-16	2.16	2.82	1.93	316	5,179	318	5,287	27.98	507
Jun-16	2.30	2.89	1.99	275	4,969	255	4,388	27.38	502
Jul-16	2.00	2.84	1.87	367	5,179	369	5,047	28.75	476
Aug-16	2.06	2.49	1.83	350	5,247	381	5,146	29.20	498
Sep-16	2.38	3.57	2.09	236	4,540	264	4,430	20.88	467
Oct-16	6.01	14.36	2.29	111	4,299	141	5,170	12.27	420
Nov-16	6.60	18.93	3.83	87	4,403	99	4,726	8.36	431
Dec-16	5.01	7.30	3.20	121	4,723	117	4,582	11.34	503
Jan-17	4.05	11.31	2.74	214	6,041	195	5,663	16.70	512
Feb-17	4.58	9.86	2.68	153	5,288	147	5,146	13.86	486
Mar-17	6.53	10.52	4.58	107	5,247	114	5,771	9.25	462
Apr-17	4.73	6.46	3.46	147	5,562	147	5,708	13.51	511
May-17	3.39	5.75	2.22	260	6,519	307	7,788	18.11	477
Jun-17	2.53	3.81	2.22	369	7,396	402	7,949	21.00	437
Jul-17	2.02	2.34	1.87	410	6,108	406	6,070	29.20	494
Aug-17	1.97	2.12	1.86	359	5,332	416	5,262	33.88	555
Sep-17	2.06	2.69	1.81	313	5,094	347	5,452	28.63	492
Oct-17	3.31	12.68	1.98	269	5,114	261	5,143	22.61	461
Nov-17	6.83	14.67	2.66	113	4,833	114	5,160	8.84	475
Dec-17	4.47	11.90	2.56	139	4,514	131	4,161	14.85	471
Jan-18	6.00	11.85	3.04	94	3,973	79	3,494	11.15	466
Feb-18	4.51	8.24	2.96	124	4,363	126	4,411	11.70	406
Mar-18	3.27	5.98	2.59	209	5,559	189	5,206	17.38	433
Apr-18	5.06	12.86	2.68	159	5,844	156	5,380	13.75	446
May-18	2.25	2.75	1.91	228	4,297	226	4,246	25.50	467
Jun-18	2.05	2.37	1.85	274	4,543	289	4,515	28.75	497
Jul-18	1.90	2.44	1.66	306	4,855	334	5,311	28.85	463
Aug-18	1.85	2.09	1.61	346	5,321	360	5,542	32.83	502
Sep-18	2.03	2.49	1.76	304	5,064	380	6,349	31.92	536
Oct-18	3.17	7.34	2.00	198	4,700	223	5,268	21.91	505
Nov-18	4.44	12.40	2.44	142	4,433	135	4,561	13.65	405
Dec-18	5.69	12.02	2.63	124	5,288	121	5,116	9.15	379

TABLE 5-4 – (continued)

Summary of Discharge Monitoring Reports WWTP Influent Monthly Averages

Month	Flow Avg. Monthly (mgd)	Flow Max. Daily (mgd)	Flow Min. Daily (mgd)	Average Monthly BOD$_5$ (mg/L)	BOD$_5$ (lb/d)	TSS (mg/L)	TSS (lb/d)	NH$_3$-N (mg/L)	NH$_3$-N (lb/d)
Jan-19	4.77	8.78	2.73	139	4,973	156	5,654	11.91	394
Feb-19	3.65	7.37	2.66	150	4,337	132	3,865	14.92	429
Mar-19	2.55	3.20	2.23	231	4,879	223	4,748	20.46	424
Apr-19	3.23	5.11	2.24	190	4,892	173	4,503	18.71	456
May-19	2.28	2.54	2.00	258	4,906	239	4,563	25.42	482
Jun-19	2.05	2.69	1.81	284	4,864	255	4,342	27.75	480
Jul-19	2.08	2.53	1.75	272	4,692	255	4,463	26.47	453
Aug-19	1.95	2.58	1.75	289	4,686	296	4,804	27.42	435
Sep-19	2.39	3.69	1.74	234	4,491	239	4,559	23.29	459
Oct-19	3.33	7.09	2.11	186	4,593	185	4,608	18.36	463
Nov-19	2.87	5.87	2.14	203	4,754	219	5,142	17.03	415
Dec-19	4.37	17.60	2.33	190	6,039	240	7,567	16.41	408
Average	**3.53**	**7.46**	**2.24**	**202**	**4,823**	**198**	**4,811**	**19.59**	**462**
Maximum	**6.83**	**20.60**	**3.83**	**361**	**7,396**	**380**	**7,949**	**33.88**	**576**
Minimum	**1.85**	**2.09**	**1.61**	**87**	**3,365**	**77**	**2,883**	**8.36**	**372**

(1) Data with BOD$_5$ or TSS concentration greater than 500 mg/L are excluded from the analysis.

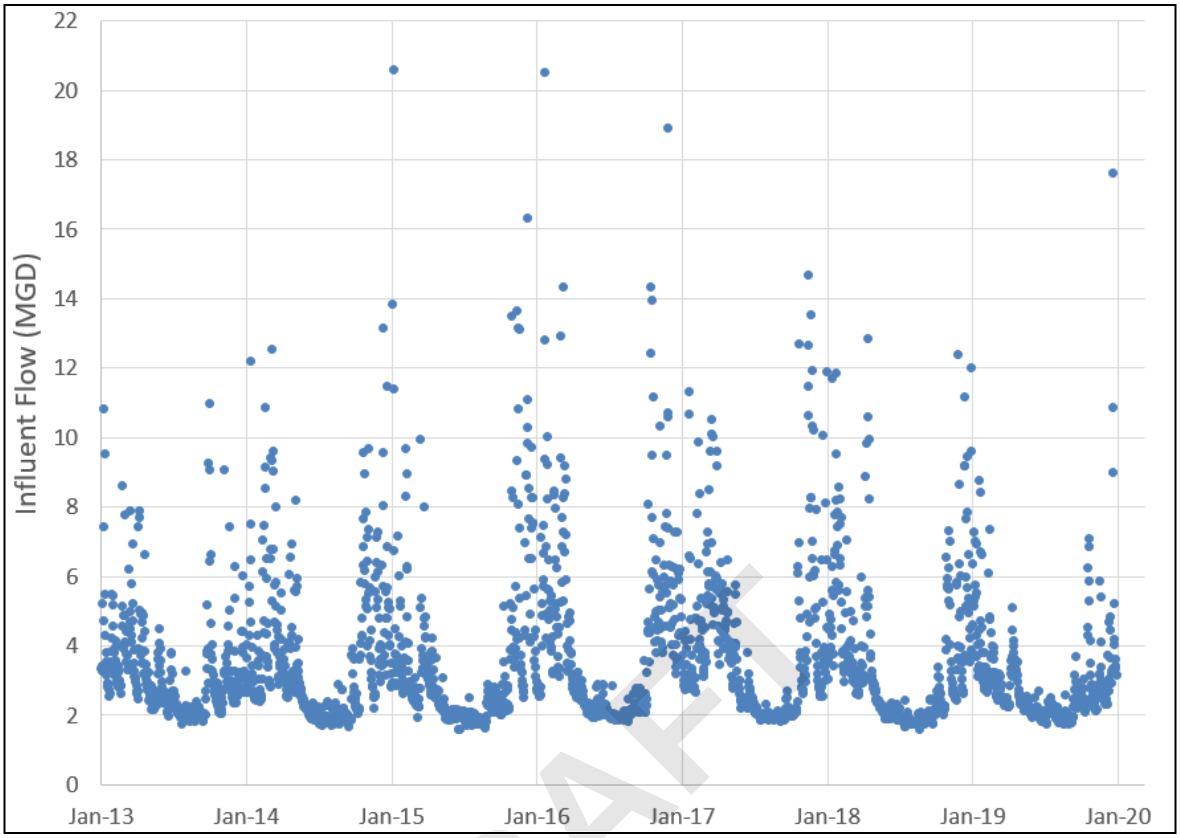


FIGURE 5-1

Daily Influent Flow

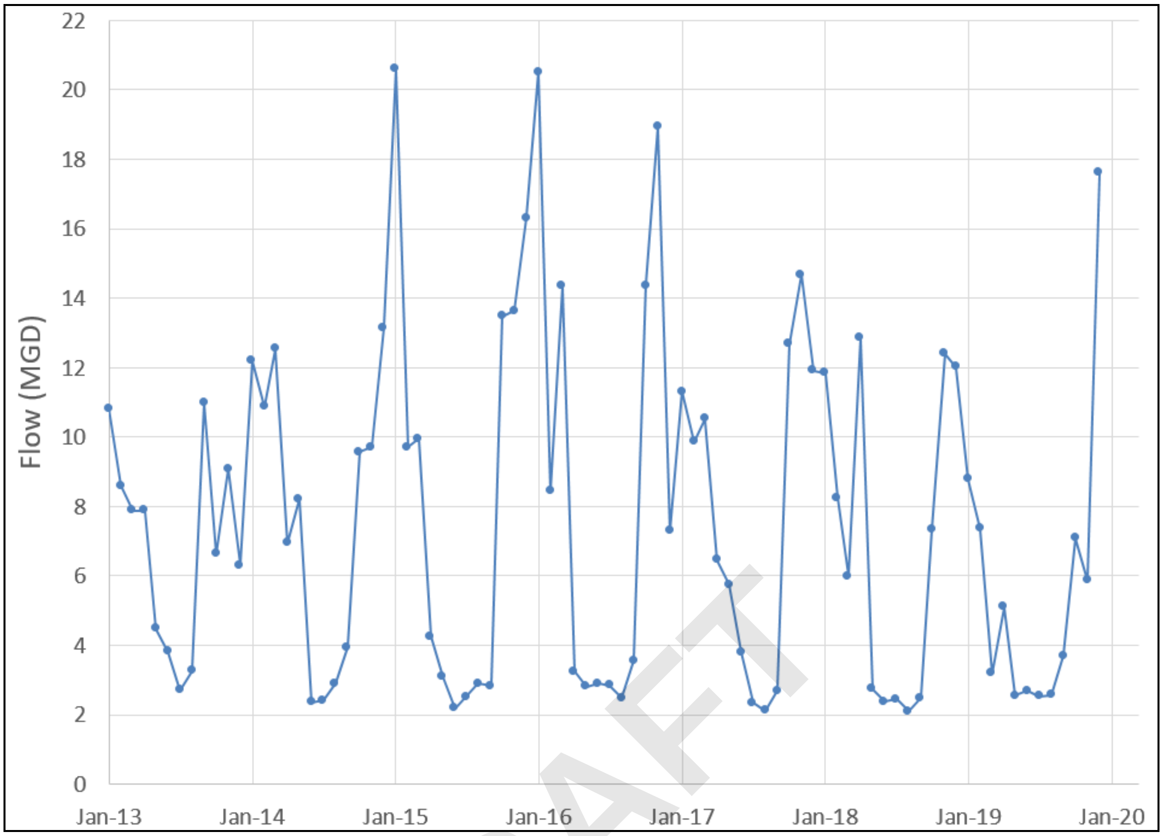


FIGURE 5-2

Monthly Peak Day Influent Flow

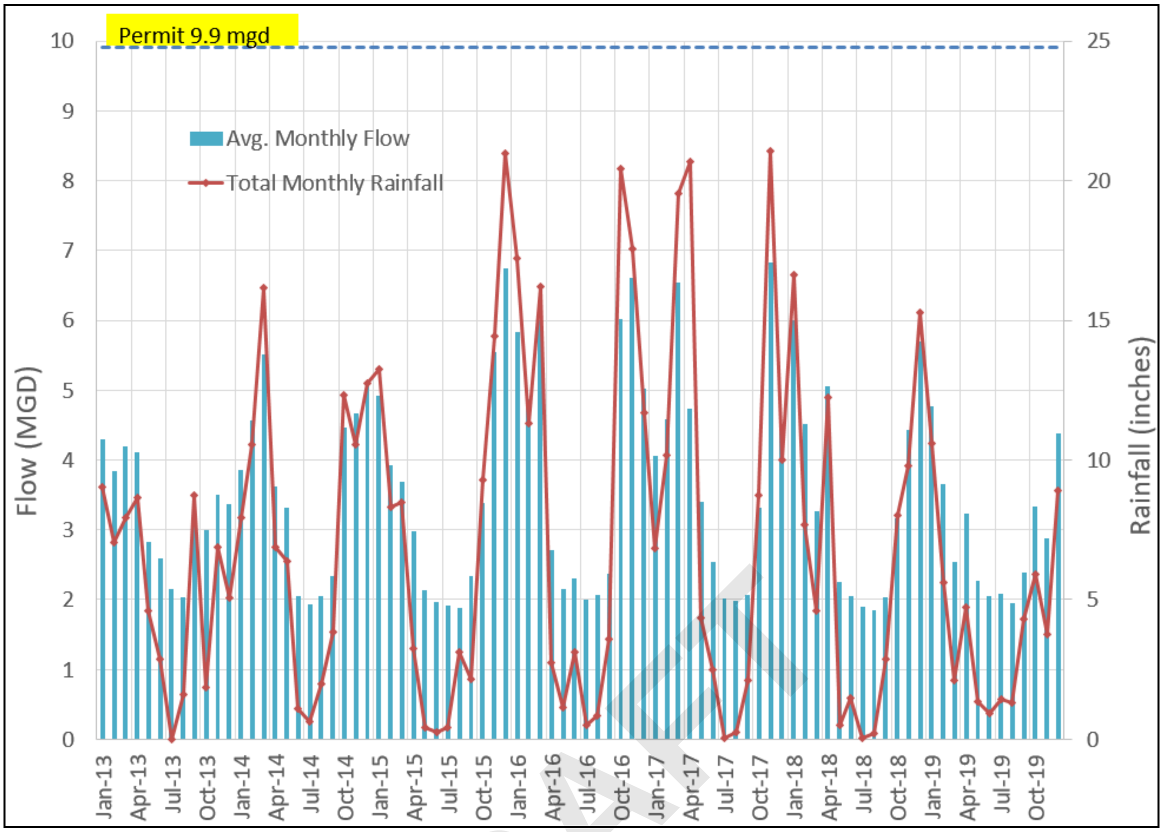


FIGURE 5-3

Monthly Average Influent Flow

INDUSTRIAL FLOWS

As required by its NPDES permit, each year the City has conducted an industrial user survey and reported the findings of the survey to Ecology. Table 5-5 lists the major industrial users. The 2017 Industrial User Surveys are presented in Appendix G.

TABLE 5-5

Major Significant Industrial Users (SIUs)

Industry	Source of Wastewater	Estimated Flow(1)
Cosmo Specialty Fibers	Domestic Discharge	N/A
Renewable Energy Group Grays Harbor	Cooling Water and Boiler Water	Range: 32,000–34,000 gpd Average: 33,300 gpd
Lemay Landfill	Landfill Leachate	Range: 10–138,000 gpd Average: 30,000 gpd
Stafford Creek Correctional Center	Domestic Discharge	Range: 39,000–958,000 gpd Average: 200,000 gpd

(1) Data provided by City staff.

The industrial flow could contribute to some variability in the character of wastewater treated by the City and the residual sludge generated by the treatment plant.

Additional information about the industrial flows of some of the current and potential major Significant Industrial Users (SIUs) is provided below, based on information in NPDES permit fact sheets and other sources.

Lemay Landfill, Inc.

The Lemay landfill site is located east of the City adjacent to State Highway 12. The Lemay landfill was closed on December 31, 1994. The leachate is currently conveyed through a force main from the landfill site to a manhole in the City's sanitary sewer system, designated SSMH 1. Leachate flow rate varies dramatically throughout the year, with minor discharge in summer and significant discharge in winter.

Table 5-6 summarizes leachate flows and loadings that were estimated in the *Comprehensive Sewage Facilities Plan Update Evaluation of Leachate Handling 2010*. As shown, the leachate flows and loadings are relatively minor.

TABLE 5-6

Summary of 2010 WWTP and Leachate Flows and Loadings Evaluation of Leachate Handling City of Aberdeen(1)

Parameter	WWTP Average	Leachate Average	Leachate (%)
Flow (mgd)	3.9	0.03	0.70%
BOD (lb/d)	4,574	2.38	0.10%
TSS (lb/d)	4,450	16.57	0.40%
NH_3-N (lb/d)	399	23.41	5.90%

(1) From Table 4.2, City of Aberdeen Comprehensive Sewage Facilities Plan Update Evaluation of Leachate Handling.

Renewable Energy Group (REG) Grays Harbor

REG Grays Harbor is a 100 million-gallon annual production capacity biorefinery in Hoquiam, Washington. The facility was originally engineered and owned by Imperium Renewables and began production in August 2007. REG acquired the biorefinery in August 2015.

The major waste stream discharged to the WWTP is from its closed-loop contact cooling water and boiler water. A glycerin byproduct is present in the discharge. The biodiesel production is expanding and expected to increase its waste discharge to the WWTP.

Cosmo Specialty Fibers, Inc.

Cosmo Specialty Fibers, Inc. (CSF), an affiliate of The Gores Group, was created in 2011 to operate the former Weyerhaeuser Specialty Cellulose Mill in Cosmopolis, Washington. This mill uses virgin hemlock to make up to 550 tons of dissolving pulp (acetate, viscose, and ether grades) each day. It discharges treated wastewater to Grays Harbor and the Chehalis River.

Currently, only domestic wastewater from the Cosmo Specialty Fibers is discharged to the City of Aberdeen WWTP. There is the potential that the company may request in the future that a portion or all of their industrial wastewater will be delivered to the WWTP for treatment, but for the purpose of this Plan that is assumed not to occur during the 20-year planning period. DMR data provided by Ecology's Water Quality Permitting and Reporting Information System (PARIS) indicated an average treated industrial wastewater discharge of 10.5 mgd, with post-treatment strength of 11.7 mg/L BOD and 114.2 mg/L TSS in 2018. No data was available for the characteristics of the untreated discharge. PARIS reported several effluent violations on residual solids, BOD_5, pH, and 2,3,7,8-Tetrachlorodibenzofuran (TCDF) between 2015 and 2018. Exceedance of fecal coliform effluent limits is also a major concern that triggered the closure of downstream shellfish beds.

BHP Billiton Canada, Inc. (BHP)

BHP, the worldwide mining company, is considering building a potash storage and export facility in Hoquiam using Grays Harbor as the terminal to export product to Asia and Brazil. If Hoquiam is chosen as the preferred site for the potash facility, construction on the $440 million project is not scheduled to be completed for 3 to 4 years. The company has said the facility would have a lifespan of at least 50 years. Considering the limitation of the Hoquiam WWTP and the potential for wastewater treatment regionalization, the facility could discharge to the Aberdeen WWTP and become a major industrial user for the Aberdeen WWTP.

EXISTING EQUIVALENT RESIDENTIAL UNITS

To assist with the determination of the number of residential units with sewer service, the *2013 City of Aberdeen Water System Plan* was reviewed. The report utilized 2005 to 2010 data. Since the population in the City has been relatively constant between 2005 and 2019, as shown in Table 5-7, it was assumed the characteristics of the water demand such as the distribution of the customer class and the winter water use used to develop the wastewater ERU value is applicable for the current study.

TABLE 5-7

Historical Population Data (2005 to 2019)

Year	City of Aberdeen (1)	City of Cosmopolis(1)	SCCC(2)	Total
2005	16,450	1,600	2,150	**20,200**
2006	16,470	1,635	2,150	**20,255**
2007	16,450	1,645	2,150	**20,245**
2008	16,460	1,650	2,150	**20,260**
2009	16,440	1,640	2,150	**20,230**
2010	16,450	1,645	2,150	**20,245**
2011	16,870	1,645	2,150	**20,665**
2012	16,890	1,640	2,150	**20,680**
2013	16,860	1,650	2,150	**20,660**
2014	16,850	1,645	2,150	**20,645**
2015	16,780	1,640	2,150	**20,570**
2016	16,780	1,650	2,150	**20,580**
2017	16,740	1,660	2,150	**20,550**
2018	16,760	1,665	2,150	**20,575**
2019	16,880	1,680	2,150	**20,710**

(1) Source: Washington State Office of Financial Management.
(2) Source: Reported by City, including full capacity of 1,972 inmates and population equivalent of employees.

SERVICE CONNECTIONS

The number of service connections from 2005 to 2010 is provided in Table 5-8. At the end of 2010, the City had 6,033 connections. The vast majority (78 percent) were single family.

The City's industrial water is supplied by the Wynoochee Industrial Water System. The Industrial Water System is not connected to the City's potable distribution system and is not being counted in the City's service connections or water analysis. It is assumed that any large-scale industrial development in the future will be served by the Industrial Water System.

TABLE 5-8

City of Aberdeen Water Service Connections by Customer Class (2005 – 2010)

	Service Connections						
	2005	**2006**	**2007**	**2008**	**2009**	**2010**	**Yearly Avg.**
Single-Family	4,711	4,704	4,701	4,738	4,718	4,685	4,710
Duplex	189	190	190	186	182	185	187
Multi-Family	199	195	193	192	188	185	192
Commercial(1)	585	573	596	597	597	583	589
Outside City Limits	310	311	318	323	324	324	318
City Facilities	11	12	12	25	45	47	25
Irrigation	3	5	3	8	8	23	8
Metered Fire Service	1	1	1	1	1	1	1
Total	**6,009**	**5,991**	**6,014**	**6,070**	**6,063**	**6,033**	**6,030**

(1) Cosmopolis and Stafford Creek Correctional Center included in Commercial.

WINTER WATER CONSUMPTION

Winter water use is used to estimate wastewater volumes entering the collection system because the amount of winter water consumption is typically equal to wastewater base flow except for a minor amount of water that does not enter the sewer system (such as winter irrigation flows, spills and evaporation).

The annual winter water consumption by customer category and by winter month (November through February) for 2005 to 2010 is provided in Table 5-9.

TABLE 5-9

Average Monthly Winter Water Use by Customer Class (2005 to 2010)

Customer Type	Winter Water Use by Customers (gpd)				
	November	December	January	February	Monthly Average
Single Family	773,333	774,194	767,742	803,509	779,694
Duplex	50,000	51,613	51,613	52,632	51,464
Multifamily	176,667	203,226	193,548	185,965	189,851
Commercial (1)	826,667	877,419	890,323	943,860	884,567
Outside City Limits	66,667	64,516	64,516	63,158	64,714
City Facilities	183,333	161,290	135,484	140,351	155,115
Irrigation	3,333	3,226	3,226	—	2,446
Metered Fire Service	3,333	—	—	—	833
Total	**2,083,333**	**2,135,484**	**2,106,452**	**2,189,474**	**2,128,686**

(1) Cosmopolis and Stafford Creek Correctional Center included in Commercial.

EQUIVALENT RESIDENTIAL UNITS

Use of ERUs is a way to express the amount of water or sewer use by non-residential customers as an equivalent number of residential customers. It is estimated that 15 percent of the winter water consumption does not enter the wastewater collection system (such as winter irrigation flows, spills, and evaporation), so the wastewater ERU value is calculated by dividing the winter water use for single-family residential (SFR) units by the number of single-family units and multiplying by 0.85. Thus, the wastewater ERU value is 141 gpd/ERU:

$$141\ gpd/ERU = \frac{779{,}694\ gpd}{4{,}710\ SFR\ Service\ Connections} * 0.85$$

Table 5-10 summarizes wastewater ERUs based on an analysis of winter water use during the winters of 2005 to 2010. As previously discussed, each wastewater ERU is defined as 141 gpd/ERU.

As indicated at the beginning of the section, the population between 2005 and 2018 has been stable. However, to adjust the average 2005 to 2010 ERUs to 2017 ERUs, the ratio of year 2017 population to average of years 2005 to 2010 population (1.0153 = 20,550/20,239) was applied to the total ERU of 12,447 of average of years 2005 to 2010. The conversion resulted in an estimated 2017 ERU of 12,638 (12,447 ∗ 1.015) and base flow of 1.78 mgd (1.752 mgd ∗ 1.0153).

TABLE 5-10

Wastewater ERUs (2005 to 2010)

Customer Type	Average Winter Water Use (mgd)	85% of Average Winter Water Use = Base Wastewater Flow (mgd)	Sewer ERUs	% of Total ERUs
Single-Family	0.780	0.663	4,710	37.8%
Duplex	0.051	0.044	311	2.5%
Multi-Family	0.190	0.161	1,147	9.2%
Commercial(1)	0.885	0.752	5,343	42.9%
Outside City Limits(2)	0.065	N/A	N/A	N/A
City Facilities	0.155	0.132	937	7.5%
Irrigation (2)	0.003	N/A	N/A	N/A
Metered Fire Service(2)	0.001	N/A	N/A	N/A
Total	**2.129**	**1.752**	**12,447**	**100%**

(1) Cosmopolis and Stafford Creek Correctional Center included in Commercial.
(2) Customer type of Outside City Limits, Irrigation, and Metered Fire Service are excluded during converting water to wastewater flow, since water consumption in these customer types is considered not to return back to the wastewater system.

As indicated previously, the two largest industrial users are Renewable Energy Group Grays Harbor and Lemay Landfill, discharging an average of about 64,000 gpd of flow, which equals 454 ERUs. It was estimated that there is a total of 100,000 gpd of industrial flow in the City, which equals about 700 industrial ERUs.

The City's totalized 2017 ERUs for all class types is 13,338 (12,638 + 700) and base flow is 1.88 mgd (1.78 + 0.10 mgd).

INFILTRATION AND INFLOW

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

For this report, infiltration and inflow is expressed in units of gallons per acre per day (gpad). The total collection area of the City of Aberdeen is measured at approximately 4,370 acres.

Table 5-11 summarizes the infiltration/inflow analysis for current conditions. The data contained in this table is useful as a baseline for evaluating changes in infiltration and inflow in the future. This data is also used to estimate future flows.

TABLE 5-11

Estimated Infiltration and Inflow

Flow Type	Influent Flow at WWTP (mgd)	Base Flow (mgd)	I/I (mgd)	Service Area (acre)[(1)]	I/I (gpad)
Annual Average	3.87[(2)]	1.88	1.99	4,370	455
Maximum Month	6.83[(2)]	1.88	4.95	4,370	1,133
Peak Day	20.60[(3)]	1.88	18.72	4,370	4,284
Peak Hour	22.99[(4)]	3.76[(5)]	19.23	4,370	4,400

(1) Estimated developed areas in the Aberdeen sewer service area unchanged since 2000 due to negligible growth.
(2) Annual average and maximum month flows were for 2017.
(3) Peak day flow was the highest daily flow between 2013 and 2018.
(4) Reported flow of 22.99 mgd at 9:00 a.m. on October 20, 2016, was selected to represent peak hour flow.
(5) Reported flow of 3.76 mgd at 8:00 a.m. on July 12, 2016, was selected to represent peak hour base flow, with a peaking factor of 2 (3.76/1.88).

INFILTRATION AND INFLOW ANALYSIS USING EPA CRITERIA

Analysis of infiltration and inflow was performed to compare estimates of per capita I/I to EPA criteria. These infiltration and inflow rates are summarized in Table 5-12.

The U.S. EPA manual entitled *I/I Analysis and Project Certification* provides recommended guidelines for determining if infiltration and/or inflow is excessive.

1. To determine if excessive *infiltration* is occurring, a threshold value of 120 gallons per capita per day (gpcd) is used. This includes domestic wastewater flow, infiltration, and nominal industrial and commercial flows. This infiltration value is based on an average daily flow over a 7- to 14-day non-rainfall period during seasonal high groundwater conditions.

2. To determine if excessive *inflow* is present in a collection system, the U.S. EPA uses a threshold value of 275 gpcd. If the average daily flow (excluding major commercial and industrial flows greater than 50,000 gpd each) during periods of significant rainfall exceeds 275 gpcd, the amount of inflow is considered excessive. This calculation should exclude major commercial and industrial flows (greater than 50,000 gpd each).

TABLE 5-12

Per Capita Infiltration and Inflow Based on EPA Criteria

Parameter	EPA Criteria for Excessive I/I (gpcd)	Estimated Aberdeen I/I Value (gpcd)
EPA Excessive Infiltration Criteria	120	135
EPA Excessive Inflow Criteria	275	1,001

Infiltration

Rainfall records from the Aberdeen WWTP DMR data show a 7-day period, December 8 through 14, 2017, during which only trace amounts of rainfall were measured. This would also be a period of relatively high groundwater. The average daily flow recorded during this time period is 2.78 mgd. With a total population of sewer users in 2017 of 20,550, the "EPA I/I Infiltration Value" for Aberdeen is estimated at 135 gpcd which is slightly greater than the EPA guideline of 120 gpcd and therefore indicates excessive infiltration.

Inflow

The maximum day influent flow at the WWTP over the period of 2013 to 2018 was 20.6 mgd (recorded on January 5, 2015), as shown in Table 5-1. With a total population of sewer users in 2015 of 20,570, the "EPA I/I Inflow Value" for Aberdeen is estimated at 1,001 gpcd. Because this value is much higher than the EPA guideline of 275 gpcd, even excluding the major commercial and industrial flows, Aberdeen is considered to have excessive inflow by EPA criteria.

I/I REDUCTION

The City has maintained an I/I reduction program for many years. Figure 5-4 shows average monthly flows from 2003 through 2019 as a function of total monthly rainfall.

If the I/I reduction program resulted in significant I/I reductions, a decrease in the slope of the linear regression line would be expected throughout the years. 2003 to 2017 data is divided in three 5-year groups, 2003 to 2007, 2008 to 2012, 2013 to 2017, and 2018 to 2019 to generate the linear regression lines. The minor slope decrease indicates the I/I reduction has been modest. However, the decrease of the y-intercept value, which represents the "no rain" day flow from 2003 to 2019 indicate there is a reduction in base flow and perhaps infiltration.

The R^2 value of linear regression lines indicate there is a strong correlation between the rainfall and WWTP inflow. There is a severe inflow issue that is evident during periods of consistent heavy rainfall.

The potential impact of the North Shore Levee project and other I/I reduction evaluated in later chapters is incorporated into the projections developed in this chapter.

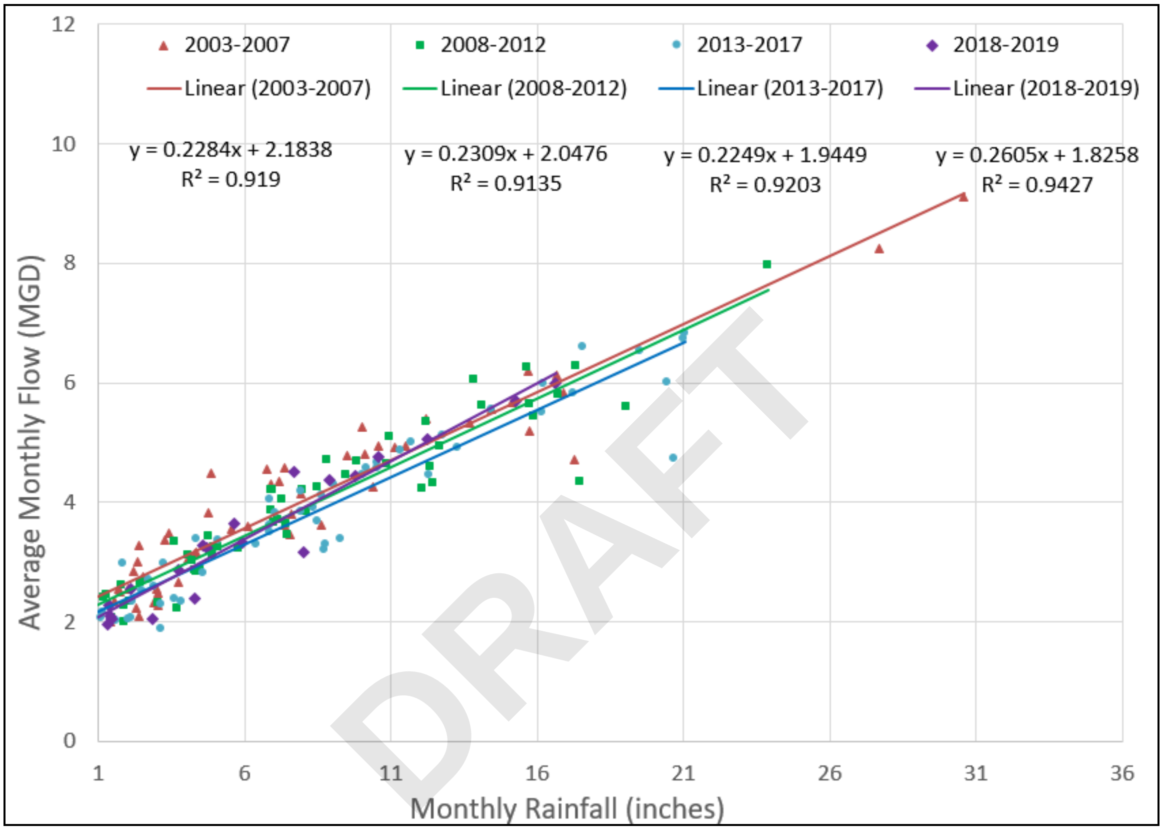


FIGURE 5-4

Influent Flow as a Function of Rainfall

HISTORICAL INFLUENT LOADING AT WWTP

Influent samples are taken at the influent manhole, which does not include flows from the hauled septage from the communities outside the City dumped at the hauled waste receiving station which conveys the waste to the influent pump station. There is no screening or rock trap or holding tank for received septage. The WWTP also receives third-party sludge from other facilities at the digester control building that is pumped to the digester.

Septage and sludge flow data obtained from City are summarized in Table 5-13. Using typical characteristics of septage and sludge, loadings were calculated and are presented in the table. The septage and sludge contributed modestly to the BOD, TSS, and nitrogen loading, while the contribution to flow was relatively negligible. As shown, septage volumes increased every year over the 5-year period. In addition, there was a significant increase of the amount of hauled sludge in 2017. (Note: Later in the chapter, the impact of hauled septage on the influent loadings is quantified. The third-party sludge, which only impacts the liquid stream through recycle streams, is not addressed in this chapter, but is in Chapter 7.)

TABLE 5-13

Annual Hauled Septage and Third-Party Sludge Received

Type	2013	2014	2015	2016	2017
Annual Septage (gallons)	490,075	714,560	1,024,440	1,039,160	1,259,474
Average Daily Flow(1) (gpd)	1,343	1,958	2,807	2,847	3,451
BOD(2) (lb/d)	72.6	105.8	151.7	153.9	186.5
TSS(2)(lb/d)	258.7	377.2	540.7	548.5	664.8
Ammonia N(2) (lb/d)	1.1	1.6	2.3	2.3	2.8
TKN(2) (lb/d)	6.6	9.6	13.8	14.0	16.9
Annual 3rd-Party Sludge (dry tons)	16.54	17.71	12.38	20.025	104.21
Average Daily Flow(3) (gpd)	596	638	446	721	3,754
BOD(4) (lb/d)	90.6	97.0	67.8	109.7	571.0
TSS (lb/d)	90.6	97.0	67.8	109.7	571.0
Ammonia N(5) (lb/d)	0.60	0.64	0.45	0.72	3.75
TKN(6) (lb/d)	4.67	5.00	3.50	5.66	29.45

(1) Average day septage flow is based on hauling frequency based on City-reported data.
(2) Loading calculation based on average value of septage: 6,480 mg/L BOD, 12,862 mg/L TSS, 97 mg/L ammonia nitrogen, and 588 mg/L TKN per *Guide to Septage Treatment and Disposal*, U.S. EPA.
(3) Sludge of 5 percent solids concentration and hauling frequency of every 2.75 days are based on the City-reported data.
(4) Assume BOD/TSS = 1.0 in sludge.
(5) Assume 1,000 mg/L ammonia nitrogen in sludge.
(6) Assume TKN = (85% TSS) ∗ 5% + ammonia nitrogen in sludge.

Influent BOD_5/TSS/ammonia loadings as sampled at the influent manhole for the period from 2013 through 2019 are shown on Figures 5-5 through 5-10. The annual average, maximum month, and peak day BOD_5, TSS, and ammonia mass loadings for 2013 through 2019 are summarized in Table 5-14. As indicated previously, some unrepresentative sample data were removed from the analysis. Due to the sampling issues, the annual average and maximum month data are considered to be more reliable than the peak day data. Reported annual average and maximum monthly loadings increased from 2013 to 2019.

Given the issues with variability and representativeness with the reported influent data (including the aforementioned evidence that the unusually high values were the result of unrepresentative sampling), the calculated values (based on 0.2 pounds per capita per day) of 6,024 lb/d annual average and 6,928 lb/d maximum month are used for the existing BOD5 and TSS loadings. Detailed analysis is presented in the technical memorandum in Appendix H. The annual average loading was multiplied by the design a peaking factor of 2.8 (from Metcalf & Eddy) to calculate an expected peak day influent TSS loading of 16,867 lb/d. The peak day BOD5 of 12,929 lb/d was obtained from the historic data, since the Metcalf & Eddy BOD5 peaking factor appear to overestimate the peak day. In this Plan, these values will be the base values for BOD and TSS that will be increased to project future loadings incorporating assumptions of growth in population, commercial, and service area and potential loadings from additional regional partners."

The influent sampling does not include plant recycle flow, which contains significant loading from solids handling units such as the screw press and anaerobic digester. For example, the concentration of ammonia nitrogen in the primary effluent has exceeded 40 mg/L. Monitoring indicates ammonia nitrogen in the influent wastewater is less than 25 mg/L on the same day. The high ammonia concentration is expected to have come from pressate or from another recycle stream.

TABLE 5-14

WWTP Influent Annual Average Loadings

	Annual Average				Maximum Month			Peak Day		
Year	**Flow (mgd)**	**BOD_5 (lb/d)**	**TSS (lb/d)**	**NH_3-N (lb/d)**	**BOD_5 (lb/d)**	**TSS (lb/d)**	**NH_3-N (lb/d)**	**BOD_5 (lb/d)**	**TSS (lb/d)**	**NH_3-N (lb/d)**
2013	3.25	4,302	4,264	468	4,910	4,916	515	12,912	15,563	758
2014	3.62	4,214	3,862	461	5,844	6,704	526	12,929	15,242	733
2015	3.45	4,899	4,802	433	6,580	6,906	502	12,284	22,151	678
2016	3.99	5,041	5,086	484	5,889	6,147	576	10,818	18,912	1,371
2017	3.87	5,587	5,773	486	7,396	7,949	555	11,510	17,122	750
2018	3.52	4,853	4,950	459	5,844	6,349	536	12,353	21,078	768
2019	2.96	4,842	4,901	441	6,039	7,567	482	14,157	28,628	757
Average(1)	**3.52**	**4,820**	**4,805**	**462**	**6,072**	**6,648**	**528**	**12,423**	**19,814**	**831**
Calc. 2017		**6,024**	**6,024**		**6,928**	**6,928**		**12,929**	**16,867**	

(1) Average of monthly averages.

As indicated previously, the loading from the hauled septage will be taken into account for the influent loading. Since the third-party sludge enters into the treatment process at the solids handling units, its contributions to loading are reflected in the recycle loading which is the WWTP Evaluation chapter. Table 5-15 summarizes the 2017 WWTP BOD, TSS, and ammonia loadings including the hauled septage. TKN loading is included to represent the combined total of ammonia and organic nitrogen.

TABLE 5-15

WWTP Influent Annual Average Loadings Including Hauled Septage

Type	Influent	Hauled Septage(1)	Total Influent Plus Hauled Septage	Ratio Total Influent Plus Hauled Septage to Influent
Annual Average Flow (mgd)	3.87		3.87	1.00
Annual Average BOD_5 (lb/d)	6,024	186	6,210	1.03
Annual Average TSS (lb/d)	6,024	370	6,394	1.06
Annual Average NH_3-N (lb/d)	486	2.8	489	1.01
Annual Average TKN (lb/d)(2)	694	16.9	711	1.02
Maximum Month BOD_5 (lb/d)	6,928	466	7,394	1.07
Maximum Month TSS (lb/d)	6,928	925	7,853	1.13
Maximum Month NH_3-N (lb/d)	555	7.0	562	1.01
Maximum Month TKN (lb/d)(2)	793	42.3	835	1.05
Peak Day BOD_5 (lb/d)	12,929	932	13,861	1.07
Peak Day TSS (lb/d)	16,867	1,851	18,718	1.11
Peak Day NH_3-N (lb/d)	750	14.0	764	1.02
Peak Day TKN (lb/d)(2)	1,071	84.6	1,156	1.08

(1) Negligible septage flow contribution. Loading peaking factors: maximum month/annual average = 2.5 and peak day/annual average = 5, calculated from reported septage flow data.

(2) TKN is calculated based on typical NH_3-N/TKN ratio of 0.7 in the influent sewage.

NPDES PERMIT LOADING LIMITS

Table 5-16 presents a summary of current loadings compared to the loading limits listed in the current NPDES permit. The table shows the influent flow and loading, as well as the adjusted loading that includes the hauled loading. In 2017, the maximum month flow was approximately 69 percent of the NPDES limit, BOD loading was 100 percent with and 94 percent without third-party loading, while TSS was 88 percent with and 78 percent without third-party loading, respectively, of the NPDES permit limit.

TABLE 5-16

NPDES Permit Flow Loading Limits – City of Aberdeen

Parameter	Units	2017 Influent	2017 Influent Plus Septage	NPDES Permit Limit	2017 Influent Percent of Limit	2017 Influent Plus Septage Percent of Limit
Maximum Month Flow	mgd	6.83	—	9.9	69%	—
Maximum Month BOD	lb/d	6,928	7,394	7,400	94%	100%
Maximum Month TSS	lb/d	6,928	7,853	8,900	78%	88%
Peak Day Flow	mgd	20.60(1)	—	18.0	114%	—

(1) Peak day flow was the highest daily flow between 2013 and 2018.

The loading characteristics of the influent wastewater received from the City's collection system are discussed below. The analysis is combined with the hauled septage data to develop the future loading projection.

BOD_5 LOADING

There is high degree of variability in the concentrations of BOD_5 in the influent wastewater. Influent BOD_5 concentrations ranged from 22 to 490 mg/L. As illustrated on Figure 5-5, the average monthly BOD_5 concentration appears to correlate inversely with rainfall. This provides further evidence of the significant inflow and infiltration in the City's wastewater collection system.

The historical record indicates that the BOD_5 load to the wastewater treatment facility has been more consistent than the concentration. Monthly average influent BOD_5 loadings ranged from 3,365 to 7,396 lb/d (6,928 lb/d maximum month, calculated) for the 7-year period of analysis, with no apparent correlation with season or rainfall, as shown on Figure 5-6. The monthly average influent BOD_5 rated loading of 7,400 lb/d was exceeded once during the 7-year period of analysis based on the sampling data, although this exceedance is considered to be anomalous and the calculated data is used.

The average influent BOD_5 concentration for the 7-year period is 202 mg/L, which would be considered moderate-strength domestic wastewater. The average BOD_5 loading for the 7 years, as summarized in Table 5-4, was 4,823 lb/d.

With a service population of 20,550 for 2017 and an annual average BOD_5 loading of 5,587 lb/d, the 2017 annual average BOD_5 loading was 0.272 lb/cap/d. This value is higher than the *Wastewater Engineering* (Metcalf & Eddy) of 0.2 lb/cap/d, likely due to the presence of commercial/industrial loading.

To convert the maximum month BOD_5 loading to a per capita and an ERU basis, the 2017 service population of 20,550 and number of ERUs (13,338) and calculated maximum month BOD_5 of 6,928 lb/d were used to calculate a maximum month per capita and ERU BOD_5 loading of 0.337 lb/cap/d and 0.519 lb/ERU/d, respectively. The ratio of the maximum month BOD_5 loading to the annual average BOD_5 loading is 6,924:6,024 or 1.15:1. The ratio of the peak day BOD_5 loading to the annual average BOD_5 loading is 2.15:1. These ratios are used in the development of future loadings to the WWTP later in the chapter.

TOTAL SUSPENDED SOLIDS LOADING

Similar to BOD_5, there is a high degree of variability in the concentration of TSS in the influent wastewater. Daily influent TSS concentrations from January 2013 through 2019 ranged from 14 to 500 mg/L. As shown on Figure 5-7, the average monthly concentration of TSS, like that of BOD_5, appears to correlate inversely with rainfall.

A review of Figure 5-8 shows that monthly average TSS loadings ranged from 2,883 to 7,949 lb/d (6,928 lb/d, calculated). Similar to BOD_5, the mass loading of TSS appears to be more consistent on a monthly basis. The monthly average influent rated TSS loading of 8,900 lb/d was not exceeded during the 7-year period of analysis.

The average influent TSS concentration for the 7-year period is 198 mg/L, which would be considered moderate domestic wastewater. The average TSS loading for the 7 years, as summarized in Table 5-4, was 4,811lb/d.

The calculated 2017 maximum month loading of 6,928 lb/d and 2017 service population of 20,550 and number of ERUs (13,338) translate to a maximum month TSS loading for 2017 of approximately 0.337 lb/cap/d and 0.519 lb/ERU/d, respectively. The annual average loading of 0.281 lb/cap/d is higher than the *Wastewater Engineering: Treatment and Resource Recovery* (Metcalf & Eddy) value of 0.2 lb/cap/d.

The ratios of the maximum month and peak day TSS loading to the annual average TSS loading are 1.15:1 and 2.8:1, respectively. These ratios are used in the development of future loadings to the WWTP later in the chapter.

AMMONIA LOADING

The annual average NH_3-N loading for 2017 was calculated as 0.024 lb/cap/d, which is close to the typical ammonia loading of 0.023 lb/cap/d cited by *Wastewater Engineering: Treatment and Resource Recovery* (Metcalf & Eddy).

With the same methodology described in the paragraphs of existing BOD_5 and TSS loading, the maximum month NH_3-N loading for 2017 was calculated as 0.027 lb/cap/d and 0.042 lb/ERU/d, respectively. The ratios of the maximum month and peak day to the annual average NH_3-N loading are 1.14:1 and 1.54:1, respectively. These ratios are used in the development of future loadings to the WWTP later in the chapter.

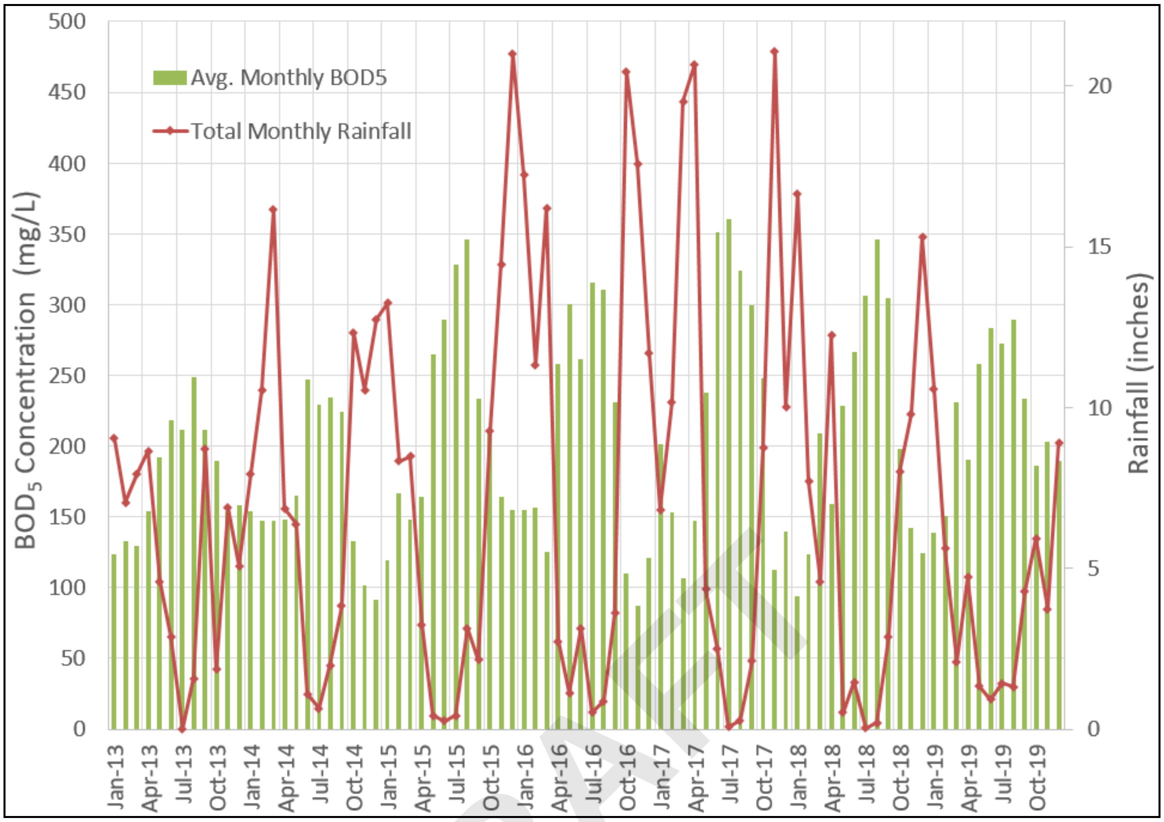


FIGURE 5-5

Monthly Average Influent BOD_5 Concentrations

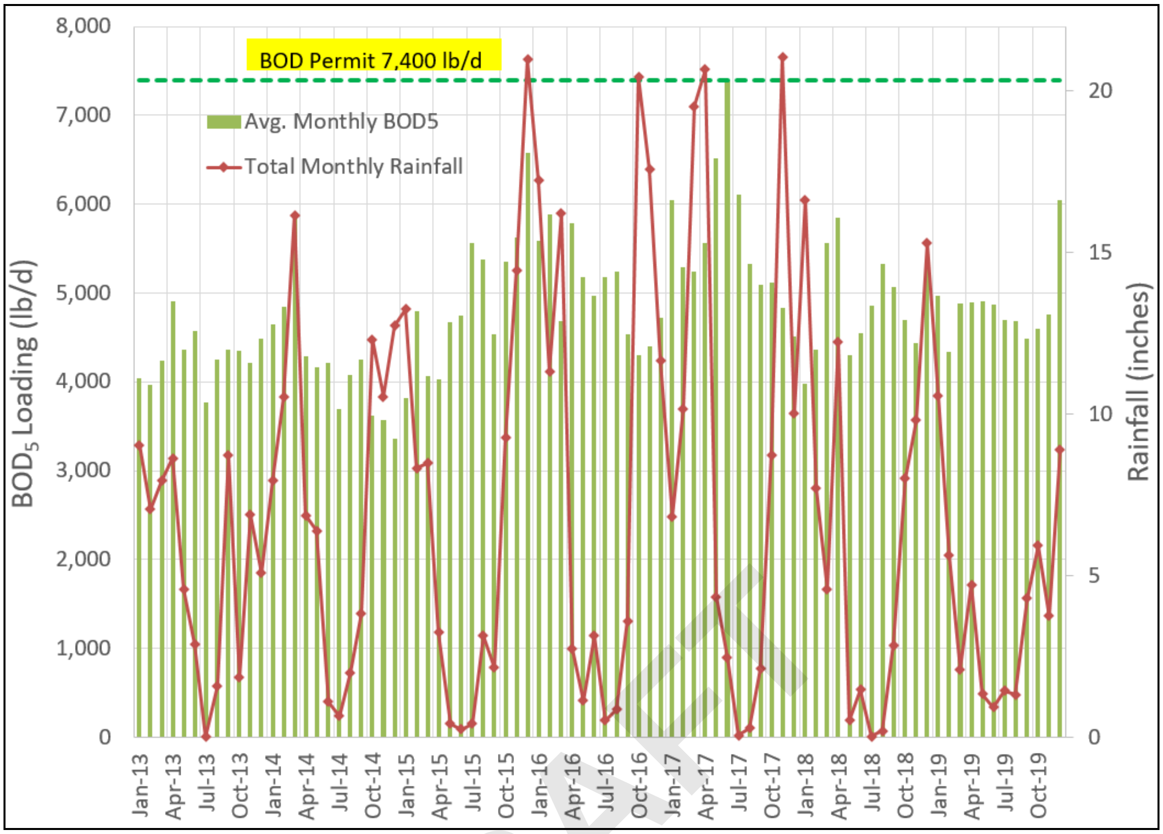


FIGURE 5-6

Monthly Average Influent BOD_5 Loadings

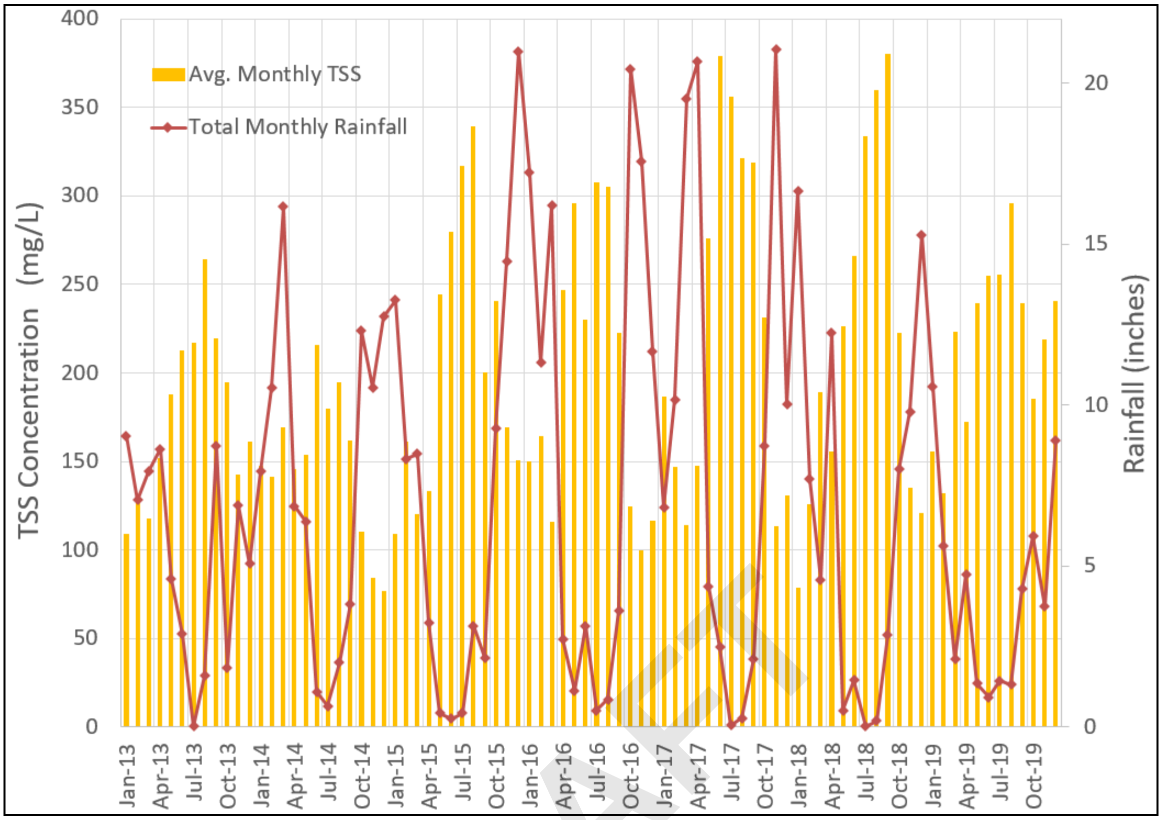


FIGURE 5-7

Monthly Average Influent TSS Concentrations

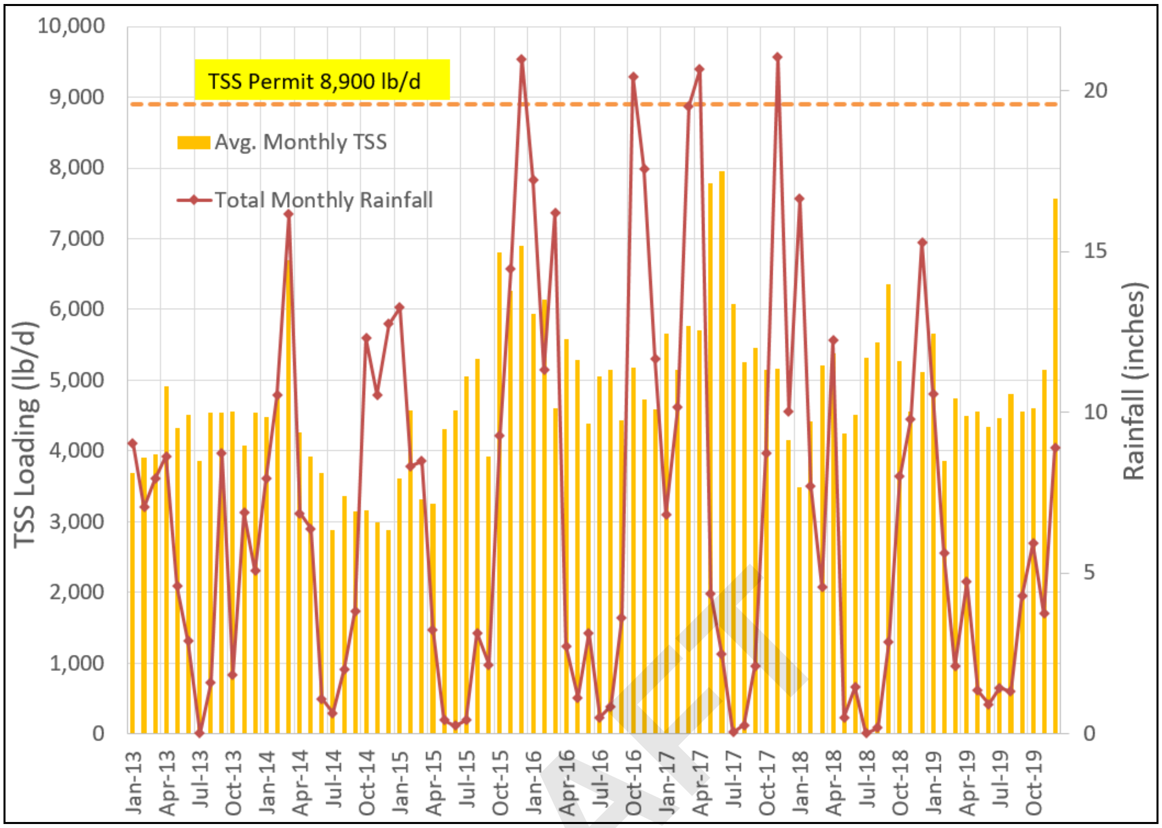


FIGURE 5-8

Monthly Average Influent TSS Loadings

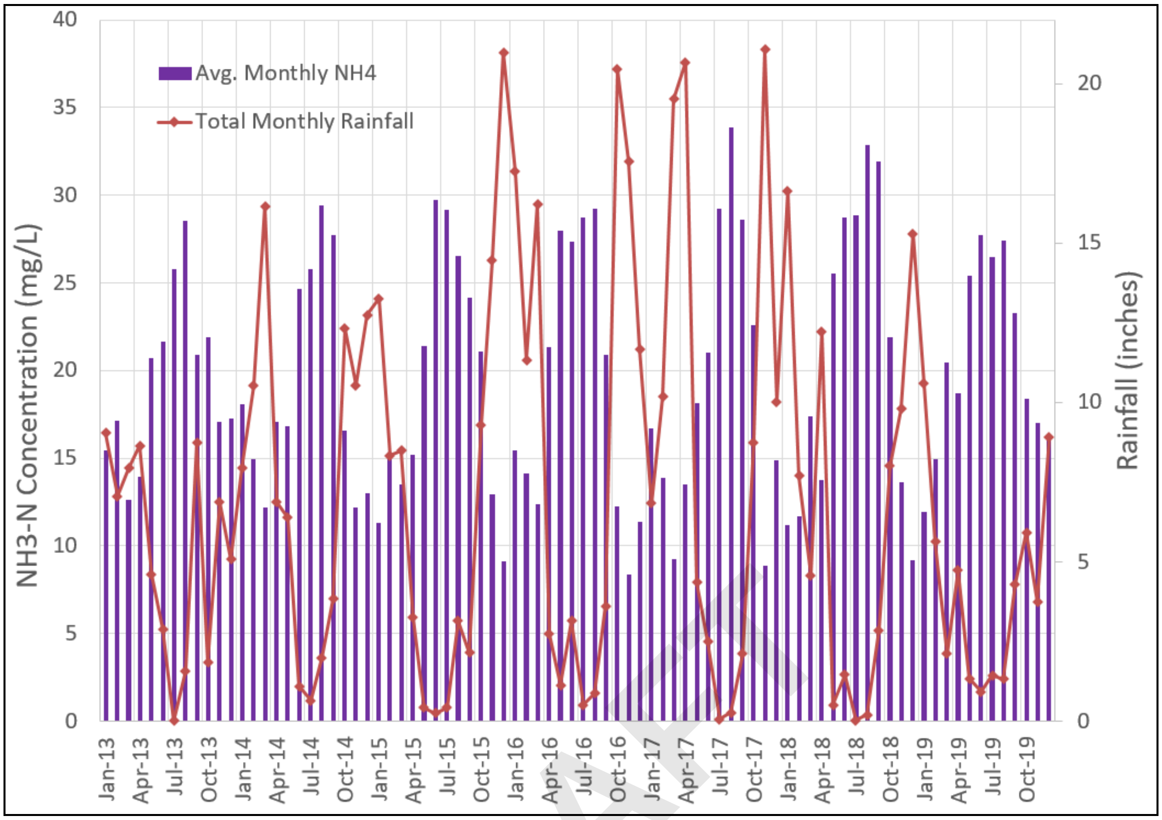


FIGURE 5-9

Monthly Average Influent NH_3-N Concentrations

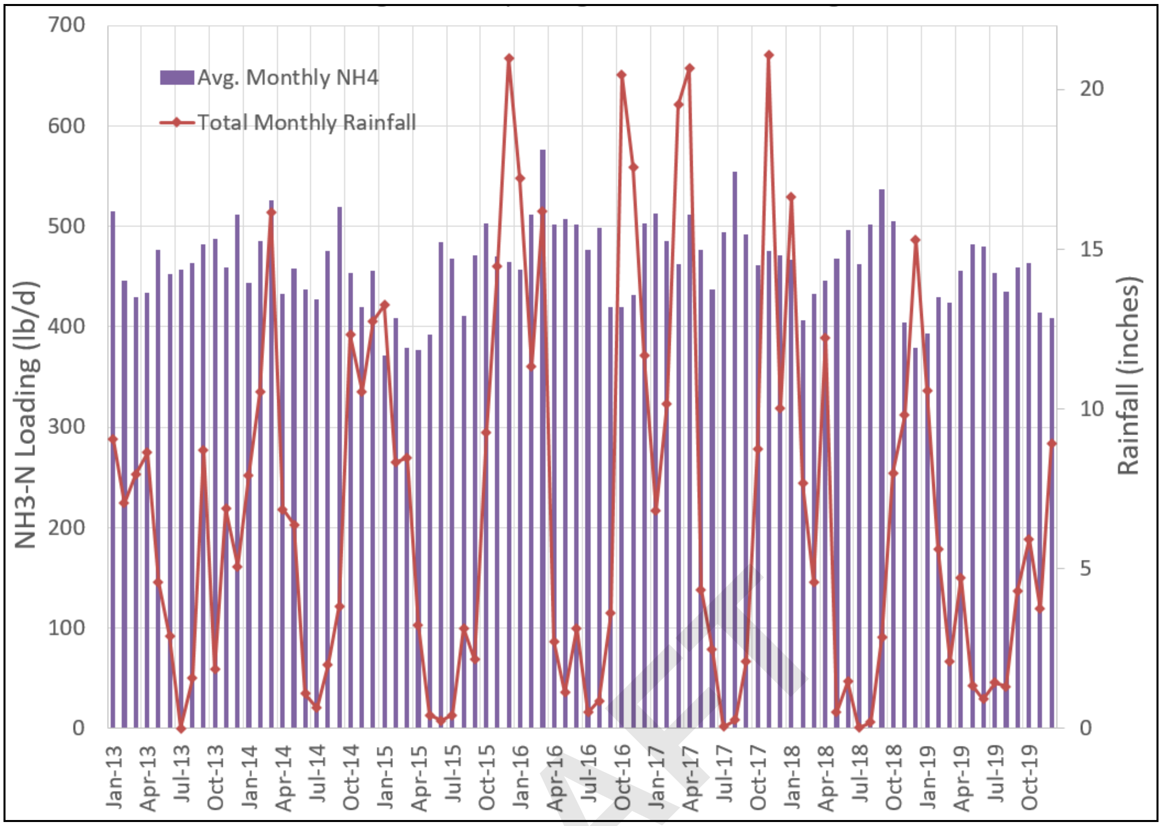


FIGURE 5-10

Monthly Average Influent NH_3-N Loadings

PROJECTED SEWER SERVICE ERUS AND FLOWS

The current and projected 20-year ERUs are summarized in Table 5-17. The projected flows and ERUs are based on use of the 1 percent growth assumptions applied to all customer classes except the industrial category.

Projected future industrial flows are estimated to grow at a faster rate (5 percent) than domestic flows based on discussion with the City.

Infiltration and inflow are assumed to be constant throughout the 20-year planning period for much of the service area. (In other words, ongoing I/I reduction efforts are assumed to compensate for the increased I/I due to growth in the sewer area and deterioration of infrastructure.) However, as discussed in the Collection System Capacity Evaluation, it is estimated that completion of the ongoing North Shore Levee project, including stormwater pumping improvements, will reduce ponding and flooding and thus I/I, significantly in some areas, resulting in a 12.2 percent overall reduction in peak inflow. Projected I/I flow is summarized in Table 5-18.

TABLE 5-17

Projected Future ERUs

Customer Type	Sewer ERUs				
	2017	**2023**	**2028**	**2033**	**2038**
Single-Family	4,782	5,076	5,335	5,607	5,893
Duplex	316	335	352	370	389
Multi-Family	1,164	1,236	1,299	1,365	1,435
Commercial(1)	5,425	5,759	6,053	6,361	6,686
City Facilities	951	1,010	1,061	1,115	1,172
Industrial	700	938	1,197	1,528	1,950
Total	**13,338**	**14,354**	**15,297**	**16,347**	**17,525**

(1) Cosmopolis and Stafford Creek Correctional Center included in Commercial.

TABLE 5-18

Current and Projected Future I/I

	2017 I/I(1) (mgd)	**Projected 2023–2038 I/I (mgd)**
Annual Average	1.99	1.99
Maximum Month	4.95	4.95
Peak Day	18.72	16.61
Peak Hour	19.23	17.03

(1) From Table 5-11.

Future WWTP flows are projected based on a dry weather flow of 141 gpd/ERU. To estimate future annual average, maximum month, and peak day flows, the I/I flow rates were added to the base level wastewater flows derived from the ERU projections to obtain the respective future WWTP influent flow rates.

TABLE 5-19

Current and Projected Future Flow

Projected Flows (mgd)					
Flow Type	**2017**	**2023**	**2028**	**2033**	**2038**
Total Base	1.88	2.02	2.16	2.30	2.47
Average Annual	3.87	4.01	4.15	4.29	4.46
Maximum Month	6.83	6.97	7.11	7.25	7.42
Peak Day	20.60(1)	18.60	18.73	18.88	19.05
Peak Hour	22.99(1)	21.08(2)	20.34(2)	21.64(2)	21.97(2)

(1) Peak flows were selected from 5 years of flow data between 2013 and 2018.
(2) A peaking factor of 2 was used to calculate the peak hour base flow; refer to Table 5-11 Note 5 for data source.

As part of the wastewater facility planning effort, the City of Aberdeen will evaluate the possibility of a new larger regional treatment facility to serve the City of Hoquiam and the Central Park community in addition to the City's service area and customers. The 2013 *Wastewater Facility Plan* for the City of Hoquiam developed by HDR Engineering, Inc. was reviewed for the current and projected flow and loading. The projected Hoquiam influent flow quantities are summarized in Table 5-20.

TABLE 5-20

City of Hoquiam Current and Projected Future Flow

Projected Flows (mgd)					
Year	**Average Dry Weather**	**Average Annual**	**Maximum Month**	**Peak Day**	**Peak Hour**
2012	0.69	1.19	2.83	11.47	12.04
2032	0.99	1.58	3.5	13.63	14.3
Buildout	1.41	2.27	5.11	20.04	21.03

However, it is expected that regional wastewater life cycle costs can be minimized by equalization of Hoquiam's flows prior to conveyance to Aberdeen. Per analysis from HDR, peak hour/day flows are assumed to be equalized to 6.5 mgd in an equalization basin constructed in the existing Hoquiam lagoon.

The future regional flow projections including Hoquiam (equalized and total) and Central Park are summarized in Table 5-21.

The projected annual average flow (AAF), maximum month flow (MMF), peak day flow (PDF), and peak hour flow (PHF) for both Aberdeen and Regional options are shown on Figure 5-11.

It was indicated in the *Comprehensive Sewage Facilities Plan Update – Evaluation of Leachate Handling Report* that the conveyance force main from the Central Park area to the Aberdeen WWTP is between 4 and 6 inches in diameter. With typical peak velocity of 8 fps, the capacity of the force main is calculated as 1.8 mgd, which is more than the projected peak flow for Central Park expected in 2038. Based on the analysis, it was assumed the force main capacity would not limit the flow from Central Park in the 20-year planning period.

TABLE 5-21

Expanded Regional Flow Projections

	Projected Flow Rate (mgd)					
Flow Type	**Aberdeen WWTP Total(1)**	**Hoquiam Total(2)**	**Hoquiam Equalized(3)**	**Central Park(4)**	**Expanded Regional Total**	**Expanded Regional Total, Equalized**
2023						
Total Base	2.02	0.84	0.84	0.00	2.87	2.87
Average Annual	4.01	1.39	1.39	0.00	5.40	5.40
Maximum Month	6.97	3.18	3.18	0.00	10.15	10.15
Peak Day	18.60	12.61	6.50	0.00	31.22	25.10
Peak Hour	21.08	13.23	6.50	0.00	34.31	27.58
2028						
Total Base	2.16	0.92	0.92	0.15	3.23	3.23
Average Annual	4.15	1.49	1.49	0.17	5.81	5.81
Maximum Month	7.11	3.35	3.35	0.26	10.72	10.72
Peak Day	18.73	13.17	6.50	0.43	32.33	25.66
Peak Hour	21.34	13.82	6.50	0.68	35.85	28.52
2033						
Total Base	2.30	1.01	1.01	0.20	3.51	3.51
Average Annual	4.29	1.60	1.60	0.22	6.12	6.12
Maximum Month	7.25	3.54	3.54	0.34	11.13	11.13
Peak Day	18.88	13.75	6.50	0.56	33.19	25.94
Peak Hour	21.64	14.42	6.50	0.89	36.96	29.04
2038						
Total Base	2.47	1.10	1.10	0.26	3.83	3.83
Average Annual	4.46	1.72	1.72	0.29	6.47	6.47
Maximum Month	7.42	3.73	3.73	0.39	11.54	11.54
Peak Day	19.05	14.35	6.50	0.66	34.06	26.21
Peak Hour	21.97	15.06	6.50	1.05	38.08	29.52

(1) Aberdeen total flow including flow from Cosmopolis and SCCC.
(2) Hoquiam flow is interpolated by flow rate presented in Table 5-19.
(3) Hoquiam peak day and peak hour flows are based on 6.5 mgd equalized.
(4) Central Park base flow is calculated based on population projections in Table 5-2 and a typical conservative wastewater flow rate of 100 gpcd. Annual average, maximum month, peak day, and peak hour flows are calculated using typical peaking factors.

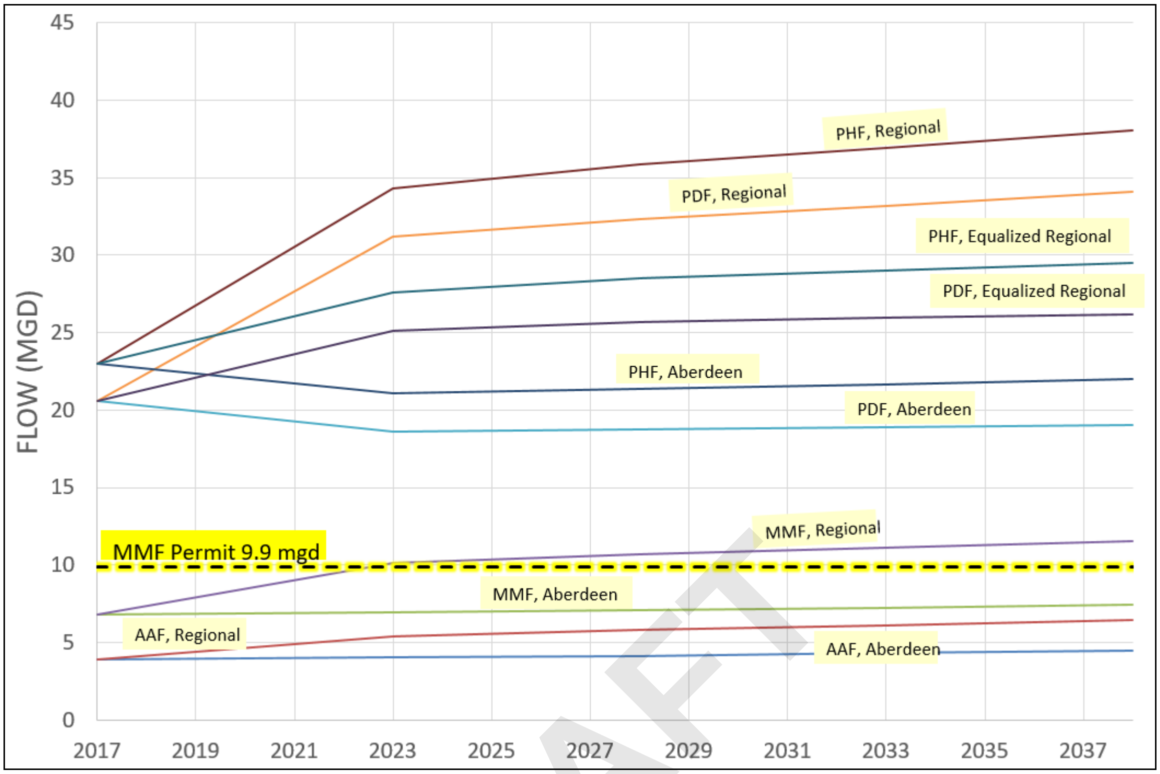


FIGURE 5-11

Regional Flow Projections

PROJECTED WASTEWATER LOADING

Future WWTP maximum month BOD_5, TSS, ammonia nitrogen, and TKN loadings are estimated by multiplying the projected number of ERUs by the respective ERU-based loadings.

The strength of the combined industrial wastewater with regard to BOD_5, TSS, ammonia nitrogen, and TKN for the industrial ERUs indicated in Table 5-17 discharged to the City is assumed to be that of domestic wastewater for this analysis. (It is likely that the combined industrial wastewater is more dilute than domestic, but due to a lack of information regarding BOD_5, TSS, ammonia nitrogen, and TKN concentrations for current and future industries, use of domestic concentrations is appropriate and conservative.)

Future ERU-based annual average loadings are estimated using the ratio of the maximum month to annual average loadings of these parameters. As calculated in the previous section, the current maximum month BOD_5, TSS, and ammonia nitrogen loadings are

0.519 lb BOD_5/ERU/d, 0.519 lb TSS/ERU/d, and 0.042 NH_3-N lb/ERU/d, respectively. TKN loading is calculated as 0.059 TKN lb/ERU/d based on typical NH_3-N/TKN ratio of 0.7 of wastewater. The ratio of the maximum month to annual average BOD_5 is 1.15:1. The ratio of the maximum month to annual average TSS is 1.15:1. The ratio of the maximum month to annual average NH_3-N and TKN is 1.14:1.

The ratio of the peak day to annual average BOD_5 is 2.15:1. The ratio of the peak day to annual average TSS is 2.8:1. The ratio of the peak day to annual average NH_3-N and TKN is 1.54:1. Table 5-22 provides a summary of projected future WWTP influent loadings.

TABLE 5-22

Current and Projected WWTP Loadings (Including Existing Partners) (Not Including Hauled Septage)

ERUs and Loadings (lb/d)	2017	2023	2028	2033	2038
Total ERUs	13,338	14,354	15,297	16,347	17,525
Annual Average BOD_5	6,024	6,483	6,909	7,383	7,915
Maximum Month BOD_5	6,928	7,455	7,946	8,491	9,103
Peak Day BOD_5	12,929	13,913	14,828	15,846	16,988
Annual Average TSS	6,024	6,483	6,909	7,383	7,915
Maximum Month TSS	6,928	7,455	7,946	8,491	9,103
Peak Day TSS	16,867	18,151	19,345	20,672	22,162
Annual Average NH_3-N	486	523	557	596	639
Maximum Month NH_3-N	555	597	637	680	729
Peak Day NH_3-N	750	807	860	919	985
Annual Average TKN	694	747	796	851	912
Maximum Month TKN	793	853	909	972	1,042
Peak Day TKN	1,071	1,153	1,229	1,313	1,408

It is assumed that the hauled septage will continue to be hauled to the WWTP at the current rate for the 20-year period based on the City's projection. The projected total loading including outside hauled loading is summarized in Table 5-23.

TABLE 5-23

Current and Projected WWTP Loadings Including Hauled Septage Loading

Loadings (lb/d)	2017	2023	2028	2033	2038
Annual Average BOD_5	6,210	6,669	7,095	7,569	8,102
Maximum Month BOD_5	7,394	7,922	8,412	8,957	9,569
Peak Day BOD_5	13,861	14,846	15,760	16,778	17,920
Annual Average TSS	6,394	6,853	7,279	7,753	8,285
Maximum Month TSS	7,853	8,381	8,871	9,416	10,028
Peak Day TSS	18,718	20,002	21,195	22,523	24,013
Annual Average NH_3-N	489	526	560	598	641
Maximum Month NH_3-N	562	604	643	687	736
Peak Day NH_3-N	764	821	874	933	999
Annual Average TKN	711	764	813	868	929
Maximum Month TKN	835	896	952	1,014	1,084
Peak Day TKN	1,156	1,238	1,313	1,398	1,492

The expanded regional option is evaluated with additional loading from Hoquiam and Central Park. Hoquiam projected loading from the 2013 *Hoquiam Facility Plan* is summarized in Table 5-24.

TABLE 5-24

City of Hoquiam Current and Projected Future Loading

	Annual Average			Maximum Month		
Year	BOD_5 (lb/d)	TSS (lb/d)	TKN (lb/d)	BOD_5 (lb/d)	TSS (lb/d)	TKN (lb/d)
2012	1,523	1,276	230	2,478	2,564	374
2032	2,186	1,830	330	3,556	3,679	537
Buildout	3,109	2,603	469	5,057	5,231	763

The future loading projections for the expanded regional alternative are summarized in Table 5-25. Loading projection for both Aberdeen and Regional options are shown on Figures 5-12 through 5-15.

TABLE 5-25

Projected Regional Loadings

Loading (lb/d)	Aberdeen Plant Total[(1)]	Hoquiam[(2)]	Central Park[(3)]	Regional Total
2023				
Annual Average BOD_5	6,669	2,124	-	8,793
Annual Average TSS	6,853	2,066	-	8,919
Annual Average NH_3-N	526	224	-	750
Annual Average TKN	764	321	-	1,085
Maximum Month BOD_5	7,922	3,023	-	10,944
Maximum Month TSS	8,381	3,127	-	11,508
Maximum Month NH_3-N	604	319	-	924
Maximum Month TKN	896	456	-	1,352
Peak Day BOD_5	14,846	7,572	-	22,418
Peak Day TSS	20,002	7,911	-	27,913
Peak Day NH_3-N	821	800	-	1,621
Peak Day TKN	1,238	1,143	-	2,381
2028				
Annual Average BOD_5	7,095	2,325	368	9,788
Annual Average TSS	7,279	2,261	398	9,937
Annual Average NH_3-N	560	246	29	835
Annual Average TKN	813	351	42	1,206
Maximum Month BOD_5	8,412	3,308	437	12,157
Maximum Month TSS	8,871	3,423	485	12,778
Maximum Month NH_3-N	643	350	34	1,027
Maximum Month TKN	952	500	48	1,499
Peak Day BOD_5	15,760	8,288	818	24,866
Peak Day TSS	21,195	8,657	1,158	31,011
Peak Day NH_3-N	874	876	46	1,796
Peak Day TKN	1,313	1,251	66	2,630
2033				
Annual Average BOD_5	7,569	2,544	503	10,617
Annual Average TSS	7,753	2,474	543	10,770
Annual Average NH_3-N	598	269	40	908
Annual Average TKN	868	384	58	1,309
Maximum Month BOD_5	8,957	3,621	595	13,173
Maximum Month TSS	9,416	3,746	660	13,822
Maximum Month NH_3-N	687	383	46	1,116
Maximum Month TKN	1,014	547	66	1,627
Peak Day BOD_5	16,778	9,071	1,115	26,965

TABLE 5-25 – (continued)

Projected Regional Loadings

Loading (lb/d)	Aberdeen Plant Total(1)	Hoquiam(2)	Central Park(3)	Regional Total
Peak Day TSS	22,523	9,474	1,579	33,576
Peak Day NH_3-N	933	959	63	1,955
Peak Day TKN	1,398	1,369	90	2,857
2038				
Annual Average BOD_5	8,102	2,785	651	11,537
Annual Average TSS	8,285	2,707	703	11,695
Annual Average NH_3-N	641	294	52	988
Annual Average TKN	929	420	74	1,424
Maximum Month BOD_5	9,569	3,963	769	14,301
Maximum Month TSS	10,028	4,100	851	14,979
Maximum Month NH_3-N	736	419	60	1,215
Maximum Month TKN	1,084	599	85	1,768
Peak Day BOD_5	17,920	9,929	1,440	29,289
Peak Day TSS	24,013	10,368	2,037	36,418
Peak Day NH_3-N	999	1,049	81	2,130
Peak Day TKN	1,492	1,499	116	3,107

(1) Aberdeen total loading including loading from Cosmopolis, SCCC, and hauled septage.

(2) Hoquiam loading is interpolated by loading rate presented in Table 5-25. TKN is converted from ammonia nitrogen by TKN/NH_3-N ratio of 0.7.

(3) Central Park base flow is calculated based on population projection in Table 5-2 and typical wastewater loading 0.25 BOD ppcd, 0.27 TSS ppcd, and 0.02 NH_3-N ppcd. TKN is converted from ammonia nitrogen by TKN/NH_3-N ratio of 0.7. Maximum month and peak day loading are calculated based on the same peaking factor as Aberdeen.

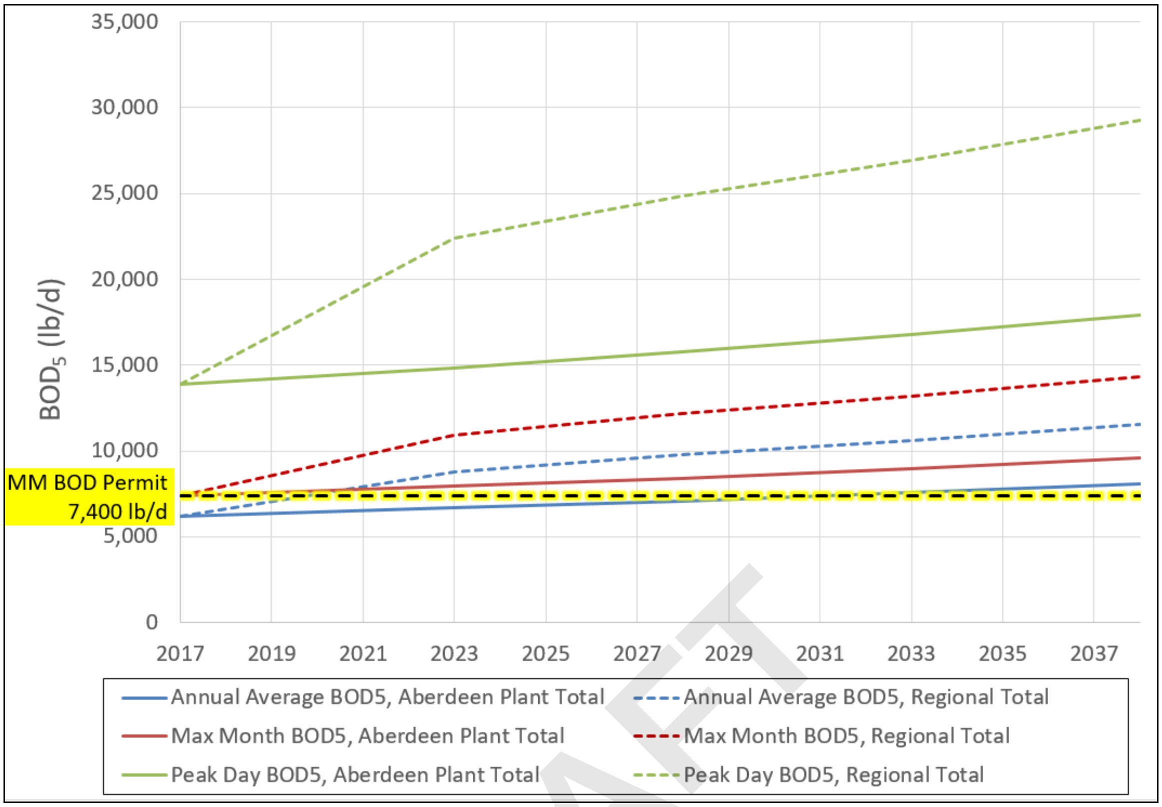


FIGURE 5-12

Projected BOD_5 Loading

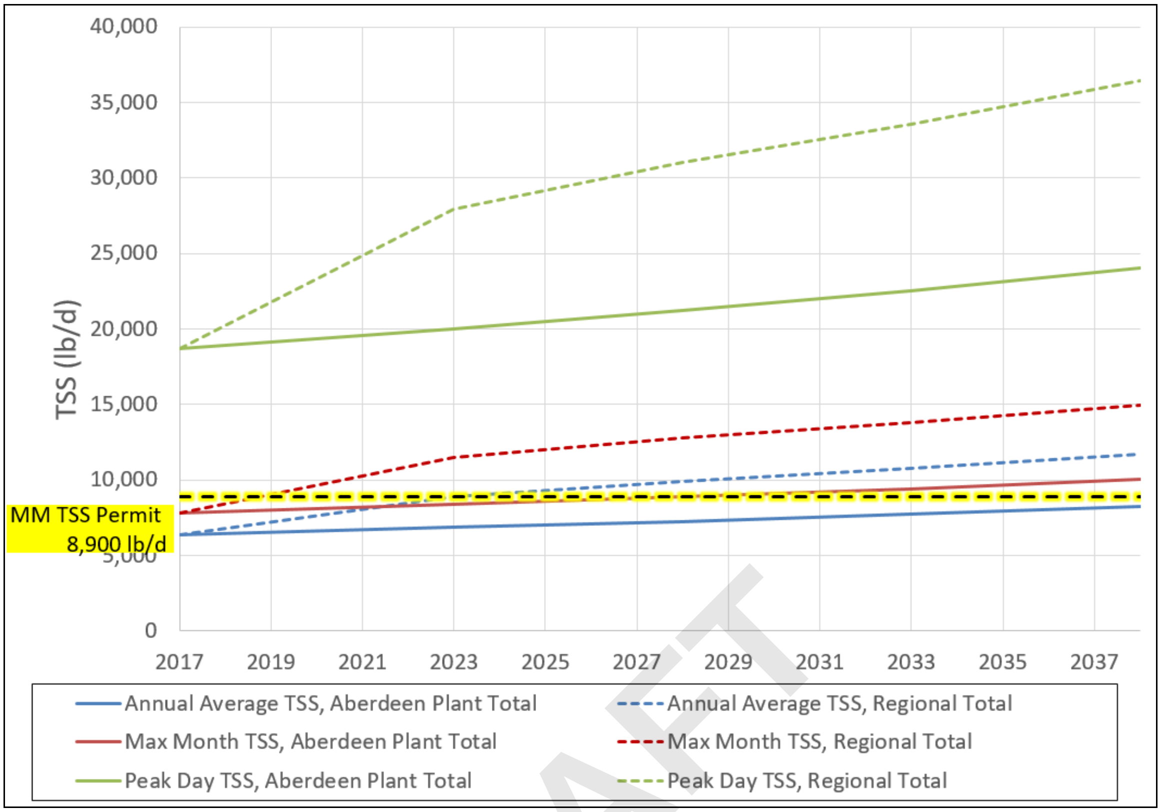


FIGURE 5-13

Projected TSS Loading

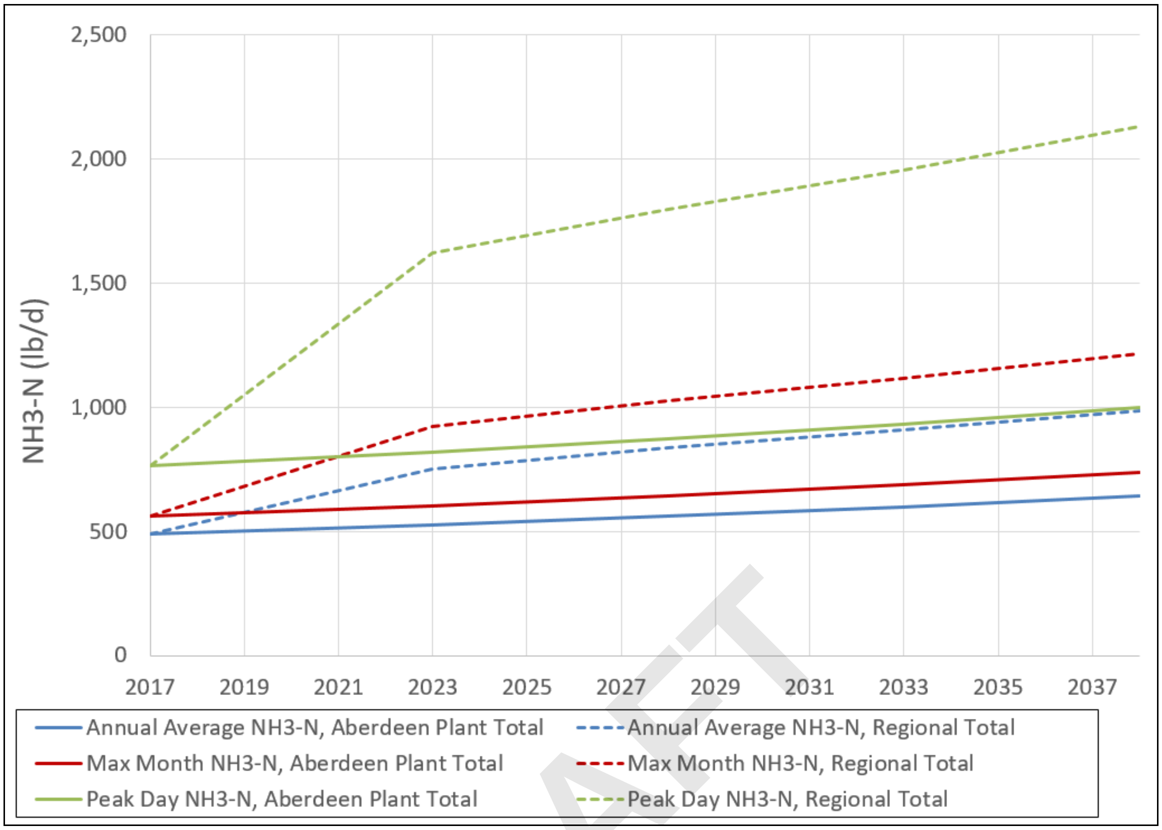


FIGURE 5-14

Projected Ammonia Loading

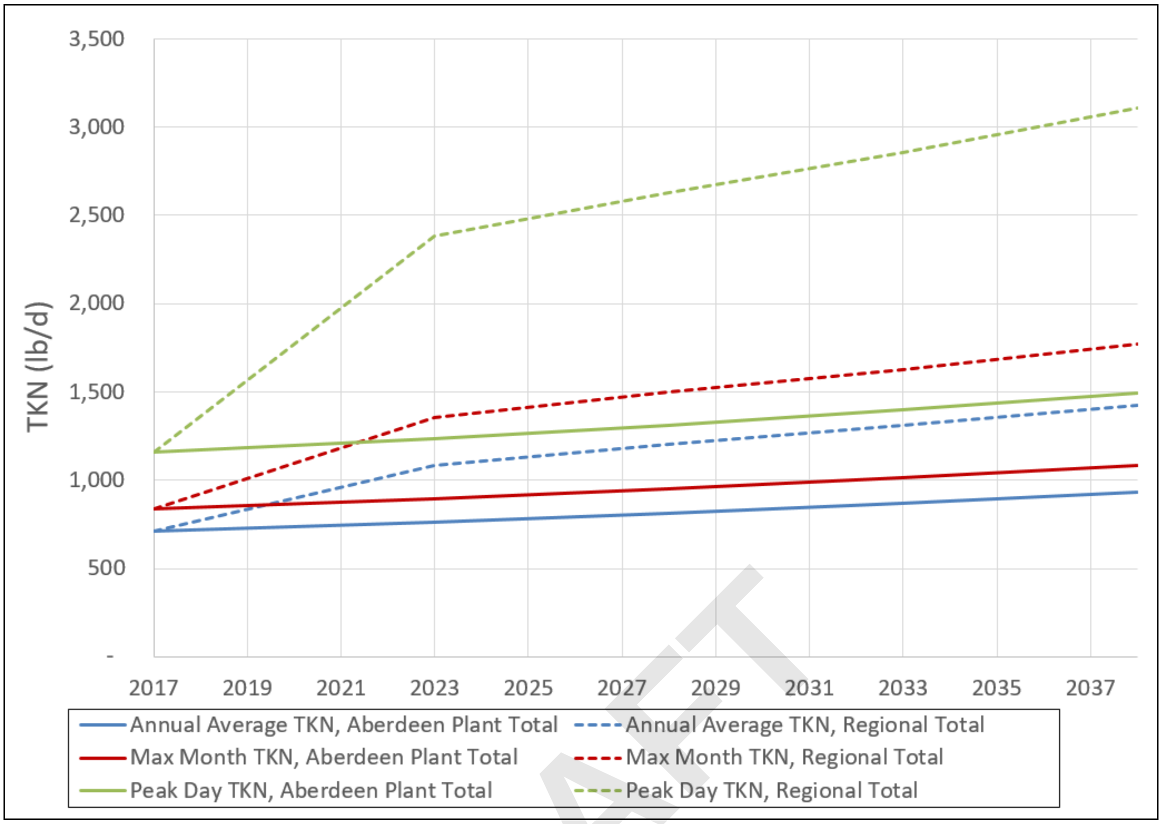


FIGURE 5-15

Projected TKN Loading

REFERENCES

U.S. EPA, 1994. *Guide to Septage Treatment and Disposal*.

Tchobanoglous, Stensel, Tsuchihashi, Burton, 2013. *Wastewater Engineering: Treatment and Resource Recovery* (Metcalf & Eddy), 5th edition.

CHAPTER 6

COLLECTION SYSTEM EVALUATION

This chapter presents an evaluation of the City's wastewater collection system. The system currently serves Aberdeen and Existing Partners (Cosmopolis, SCCC, and the County landfill). Following the evaluation, potential improvements necessary to serve Aberdeen and Existing Partners are considered, followed by an evaluation of improvements necessary to serve the Expanded Regional Partners (Hoquiam and Central Park) at the end of the chapter. Recommended improvements are provided based on the evaluation of capacity, condition, operation and maintenance, and reliability.

The City's collection system is comprised of over 106 miles of pipe ranging from 2 to 48 inches in diameter. There are 17 pump stations within the collection area including the influent pump station at the WWTP and the Stafford Creek Correction Center (SCCC) pump station. The City owns, operates and maintains these facilities in accordance with WAC 173-240-105.

The collection system was originally built as a combined sewer system. Starting in the 1930s, the system was rebuilt with a separate system. During the late 1970s and early 1980s, the City undertook a major sewage collection system upgrade project, which replaced the majority of the old concrete sewer lines with new PVC pipe. However, the State Street trunk sewer has not been replaced, rehabilitated or thoroughly inspected since its installation in 1958. The State Street sewer line is constructed of reinforced concrete and is the largest diameter sewer in the City's wastewater collection system. Attempts to evaluate the condition of the State Street sewer have been hampered by the depth of the sewer and the cost of diverting existing sewage flows to allow a thorough inspection. Because of the age of the sewer, it is suspected that there may be significant deterioration of the concrete pipe, particularly at the joints, which may be allowing excessive inflow and infiltration.

INFLOW AND INFILTRATION CONTROL EFFORTS

Sewage flow rates that are much higher during wet-weather periods than during dry-weather periods indicate the presence of infiltration and inflow. Infiltration is *groundwater* that enters a sewer system through locations such as cracks in pipes and manholes, loose pipe joints, foundation drains, and basement sump pumps. Inflow is *surface water* that enters the system through sites such as cross-connections with storm drains and downspouts, area drains, unplugged and leaking cleanouts, and ponding on manhole covers. High volumes of I/I consume the capacity of pipes, pump stations, and treatment facilities, requiring that larger facilities be designed to accommodate the increased flow in the wastewater system.

The City has maintained an ongoing effort to minimize I/I. Annual activities include identifying illegal connections and implementing and monitoring corrective actions, manhole rehabilitation through grouting and epoxy lining, replacement of damaged sewer sections, and hydro-cleaning. In addition, the City has completed construction of storm water system improvements including pump stations in downtown Aberdeen. Operation of the stormwater pumping systems has been effective in reducing the duration of peak I/I flow. In addition, as described in Chapter 2, the City is currently in the design phase of the North Shore Levee flood control project, which will add additional storm water conveyance and pump stations and is anticipated to further reduce I/I to the sanitary sewer system.

BASIN FLOWS

Pump station data and WWTP flow and precipitation records collected during the study period were evaluated to determine the volume of I/I being generated throughout the collection system. The pump station flow data were calculated primarily based on pump run times instead of pump station flow meter data, due to issues with the flow meters at pump stations and SCADA system recording.

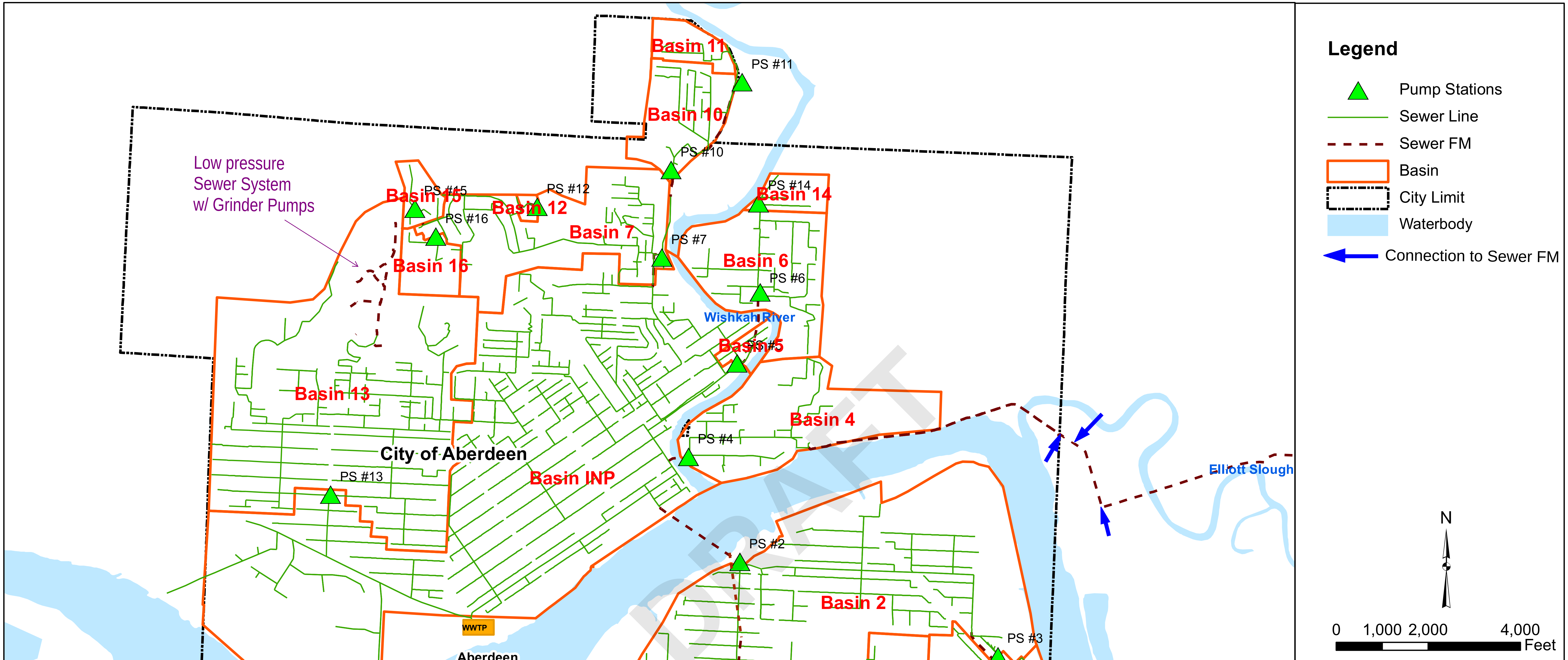
The City's wastewater collection system has 17 drainage basins (including one for SCCC); each was delineated to correspond to, and named after, the tributary area of each pump station. Figure 6-1 shows the location of basins and pump stations and Figure 6-2 shows schematically how they are arranged throughout the collection system. Pump Stations 2, 6, 7, 10, along with the WWTP Influent Pump Station (called INP in this chapter), receive flow from more than one basin. Flows for each basin were calculated from pump data measured directly at each pump station, and subtraction of upstream basin flows where applicable.

In addition to flows generated within the City, there are also flow contributions from outside of the City, including Cosmopolis flow to Pump Station 2, and County (formerly LeMay) Landfill flow to Pump Station 4. These flows were subtracted from Pump Stations 2 and 4 in the City basin flow analysis and quantified separately, and identified as Cosmo, and Landfill. Flows from SCCC are discharged to the 18-inch force main downstream of discharge from Pump Station 2.

PEAK DAY FLOW AND UNIT FLOWS

For this analysis, peak day I/I is defined as the difference between the recorded peak day flow and the average dry weather flow. As such, the I/I quantities do not include the (the expected small) level of dry weather infiltration.

A summary of I/I flows is presented in tabular and graphic form in Figure 6-3. As shown, the I/I response to precipitation within Aberdeen's sanitary system varies substantially from area to area. Some portions of the system are known to respond very quickly to rainfall, some react more slowly, while others appear to have only minimal response.



ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 6-1
DRAINAGE BASINS

Gray & Osborne, Inc.
CONSULTING ENGINEERS

Because flow varies with the size of the basin, unit I/I flow rates for each basin were calculated by dividing the peak-day I/I by the basin area. These unit I/I rates, in gallon per day per acre (gpad), allow the comparison of the level of I/I among basins of different sizes.

Basin flows from three major storm events (1/4/2015, 1/5/2015 and 1/21/2016) were assessed. Because the storm generating the highest peak flow was not the same for all basins, the peak flow was represented as the composite of three storm events. The data collected was evaluated to determine I/I rates. SCCC did not contribute to the peak I/I, since it is not allowed to discharge during the peak wet weather event, and holds flow in an equalization basin. I/I in the Aberdeen system appears to be predominantly inflow (about 20 times the infiltration levels on a peak day basis), so the focus in this chapter is on inflow.

As shown in Table 5-12, *infiltration* in the Aberdeen collection system is considered, by a small margin, to be excessive by EPA criteria; the infiltration is 135 gallons per capita day (gpcd) and the EPA criterion is 120 gpcd. However, *inflow* in the Aberdeen system exceeds the criterion for excessive inflow by a large margin; the inflow is 1,001 gallons per capita day (gpcd) and the EPA criterion is 120 gpcd. Thus, reduction in inflow is the key objective for I/I removal.

Flow discharged from each respective basin or group of sewage collection basins during the storm events is shown on Figure 6-3. Estimated I/I flows generated within each basin are presented for 24-hour average periods. I/I flows were adjusted slightly to match the total I/I to the WWTP.

Basins 12 and 5 are the smallest basins in the study, with areas of 5.2 acre and 18.9 acre, respectively. Because they are such small basins, Basins 12 and 5 produce an inflow of only 167,187 gpd and 171, 281 gpd (a small volume compared to Basins INP, 2 and 13); however, the unit inflow rates for these basins are 31,845 gpad and 9,049 gpad, which are the highest among all the basins.

Basins INP, 2, 9, and 13 generate the highest volume of inflow. They are also ranked the third to sixth highest inflow per acre of all basins within the City's collection system. Basin INP and 13 represent much of the west part of Aberdeen, and were reported to have the highest I/I levels in the 1999 Aberdeen I/I study. Basin INP includes large-diameter (24-inch, 36-inch, 48-inch) pipes, including the State Street trunk sewer constructed in 1958, which may be deteriorated and contributing to significant I/I. Basin 2 and INP collectively contribute approximately 50 percent of the total inflow to the wastewater collection system.

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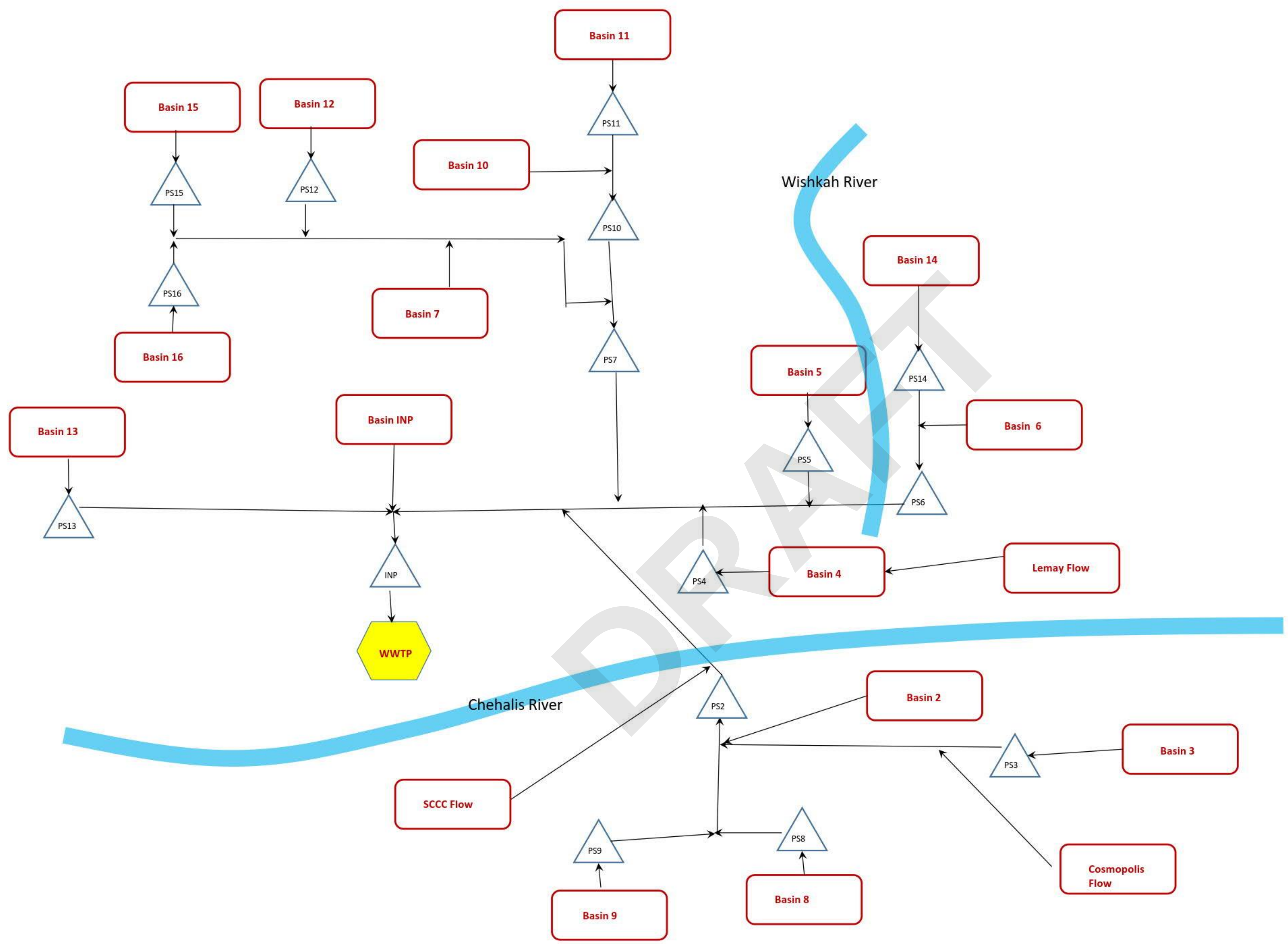


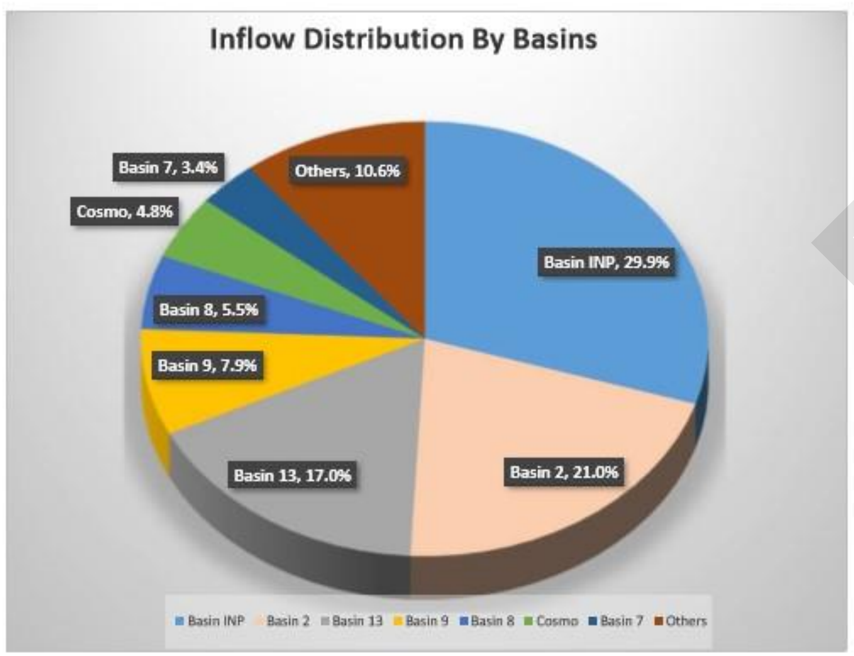
FIGURE 6-2

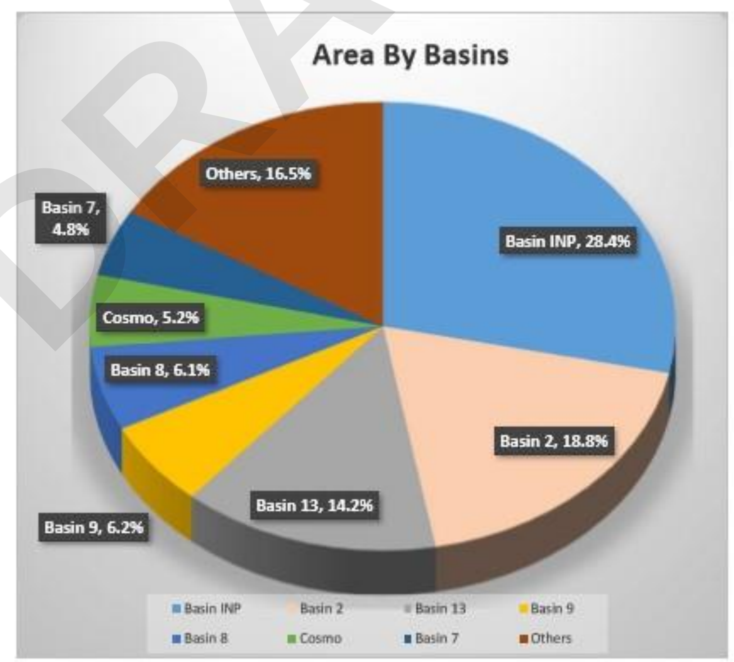
City of Aberdeen Collection System Schematic

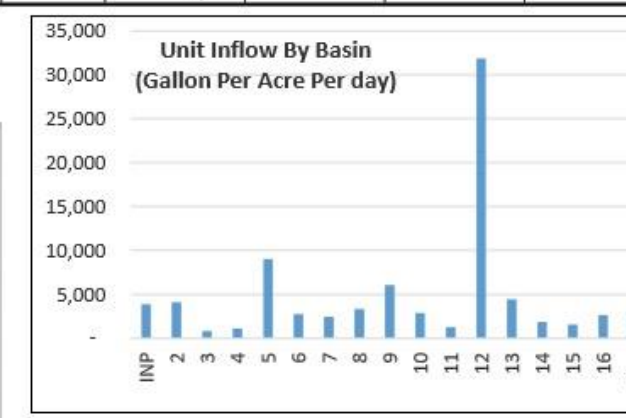
Basin	Derived by Pump Station Flow	Peak Day Flow, 1/4/2015 event	Peak Day Flow, 1/5/2015 event	Peak Day Flow, 1/21/2016 event	Average Peak Day Flow	Average Dry Weather Flow	Wet Weather Infiltration	Peak Day Inflow, Adjusted	Area	Unit Infiltration	Peak Day Unit Inflow	Percent of Inflow by Basin	Percent of Inflow to Peak Flow	Percent of Area by Basin
		gpd	gpd	gpd	gpd	gpd	gpd	gpd	Acres	gpad	gpad			
INP	EFF-PS2-PS4-PS5-PS6-PS7-PS13	1,977,575	7,894,948	8,344,747	6,072,424	1,037,131	134,015	4,631,708	1189.0	113	3,895	29.9%	79.8%	28.4%
2	PS2-PS3-PS8-PS9-Cosmo	4,958,099	3,761,256	3,600,736	4,106,697	253,643	413,760	3,250,133	786.9	526	4,130	21.0%	83.0%	18.8%
3	PS3	88,951	69,984	138,024	98,986	1,857	2,571	89,357	103.9	25	860	0.6%	95.3%	2.5%
4	PS4-Landfill	322,623	252,083	379,620	318,109	63,357	20,186	221,665	193.4	104	1,146	1.4%	72.6%	4.6%
5	PS5	102,000	201,000	388,000	230,333	19,286	29,798	171,281	18.9	1,574	9,049	1.1%	77.7%	0.5%
6	PS6-PS14	892,350	352,020	332,140	525,503	31,643	27,087	441,101	159.7	170	2,762	2.8%	88.2%	3.8%
7	PS7-PS10-PS12-PS15-PS16	630,340	818,280	327,520	592,047	8,041	20,702	532,323	216.4	96	2,460	3.4%	94.9%	5.2%
8	PS8	449,550	1,535,760	764,640	916,650	7,143	5,571	854,219	255.2	22	3,347	5.5%	98.5%	6.1%
9	PS9	-	-	1,362,000	1,362,000	46,000	21,114	1,223,667	201.6	105	6,070	7.9%	94.8%	4.8%
10	PS10-PS11	250,560	91,800	542,160	294,840	11,571	8,781	259,391	89.5	98	2,897	1.7%	92.7%	2.1%
11	PS11	75,600	37,800	32,400	48,600	5,714	286	40,257	31.4	9	1,283	0.3%	87.0%	0.7%
12	PS12	293,760	220,320	133,920	216,000	11,657	27,429	167,184	5.2	5,225	31,845	1.1%	81.1%	0.1%
13	PS13	2,710,975	4,066,462	2,688,753	3,155,396	241,071	122,529	2,638,248	594.4	206	4,439	17.0%	87.9%	14.2%
14	PS14	93,960	30,570	53,960	59,497	2,500	1,628	52,324	28.2	58	1,856	0.3%	92.7%	0.7%
15	PS15	26,460	56,700	79,800	54,320	11,700	3,500	36,968	23.3	150	1,589	0.2%	70.9%	0.6%
16	PS16	125,280	116,100	103,200	114,860	6,686	806	101,463	38.2	21	2,653	0.7%	93.1%	0.9%
Cosmo	Cosmo	789,000	1,032,000	1,133,000	984,667	119,500	84,640	737,598	258.0	328	2,859	4.8%	78.3%	6.2%
Landfill (1)	Landfill	62,917	62,917	95,380	73,738	1,500	5,600	62,973				0.4%	89.9%	
SCCC (2)	SCCC					222,000	20,000							
Sum		13,900,000	20,600,000	20,500,000	18,300,000	1,880,000	930,000	15,500,000	4,193					
Average										519	4,891			

(1) Unit I/I is not calculated for Landfill, since it's not area based

(2) SCCC does not discharge flow when WWTP influent > 13 mgd







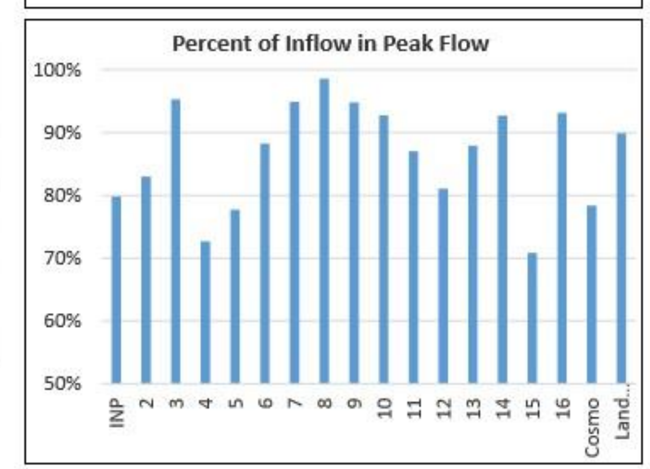


FIGURE 6-3

City of Aberdeen I/I Evaluation

EFFECT OF TIDES ON WASTEWATER FLOWS

The possibility that significant amounts of tidal water could be entering the wastewater collection system and contributing to I/I was considered during the assessment. One of the methods used in making this assessment was to compare recorded pump run times and flows against the tide depth in Grays Harbor. This was done during the dry weather period to see if increased flow coincided with high tides, while excluding the influence of rainfall. Figure 6-4 shows the wastewater flows plotted against tide levels. Based on the evaluation, it is concluded that tides have, at the most, only a minor direct impact on wastewater flows. However, as discussed later in this chapter, it is possible that tide levels have some influence on wastewater flows, when high tides occur during peak precipitation periods of major storms, causing backups at stormwater outfalls, exacerbating flooding and increasing the probability of I/I to the sewage collection system particularly through manholes.

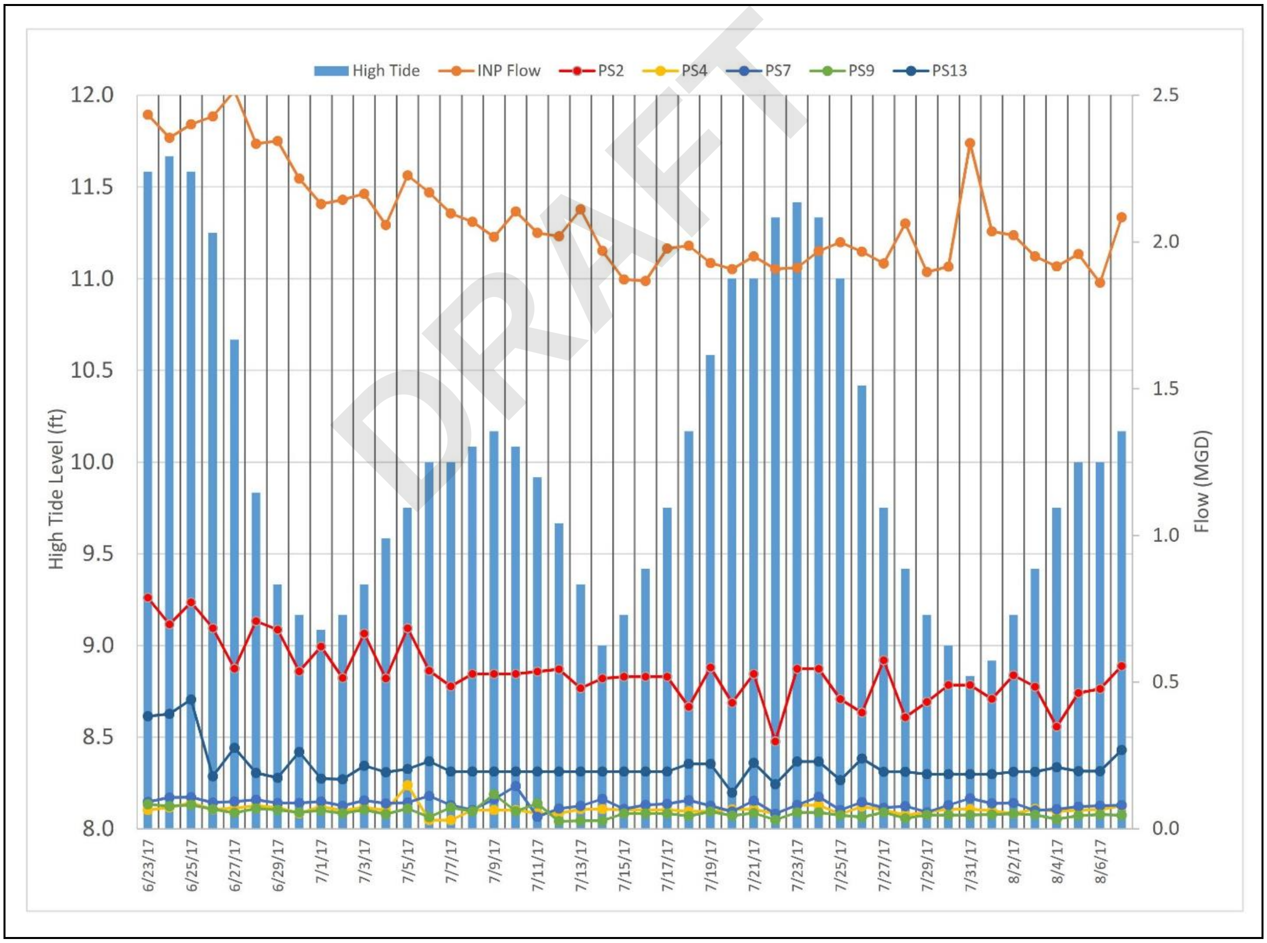


FIGURE 6-4

City of Aberdeen Tide vs. Flow during Dry Weather Period

IMPACT OF NORTH SHORE LEVEE AND STORMWATER IMPROVEMENTS

The City, along with Hoquiam and other regional partners, is proceeding with the design and permitting of the $75 million North Shore Levee project, which is planned to include 5.7 miles of levee between the Wishkah and Hoquiam Rivers to protect against coastal flood events; the project also includes upgrades to and expansion of several stormwater pump stations and associated piping to improve drainage and reduce ponding. The construction of both the levee and stormwater improvements will effectively reduce wastewater flows during wet weather.

IMPACT OF RIVER ELEVATIONS AND LEVEE CONSTRUCTION

To evaluate the possible contribution of river water to the collection system peak flows, the river water elevation gauge data from USGA Montesano, WA station were compared with several high wastewater flow events, as shown in Table 6-1. It was concluded that, with the exception of the extreme, 50-year event on January 5, 2015, the peak sewage flow events do not appear to directly correspond to high river elevations. This conclusion is based on the fact that the peak sewage flow typically precedes, and does not coincide with, the maximum river water levels. (Similar to the impact of tides, however, river elevations can likely exacerbate I/I during peak precipitation events due to increased ponding of surface water.)

Based on the analysis of past storms, it is concluded that, instead of river flooding being the primary driver of high influent flows, ponding caused by precipitation within the City (as well as running down from the hills on the north side of the City) is the primary cause. This can be exacerbated by high river levels, which may interfere with or preclude the flow of stormwater into the river.

A 2017 hydrology study, *North Shore Levee, Aberdeen and Hoquiam, WA - Hydraulic Analysis and Floodplain Mapping - Memorandum* modeled the extreme 100-year tide plus 10-year storm event and concluded that the levee will significantly reduce flooding in the City from the river, as shown in Figure 6-5. This will greatly reduce the possibility of river flooding in the City causing high I/I flows, as occurred in the January 5, 2015 event.

IMPACT OF PONDING AND STORMWATER IMPROVEMENTS

Figure 6-6 shows 100-year ponding maps (from North Shore Levee, 100-Year Rainfall Ponding Depth Site Plans, KPFF, 2018) with and without the levee in place. The proposed storm drain system improvement will significantly reduce the depth and area of surface water ponding in Basins INP, 5 and 13. The inflow in these basins is expected to be reduced accordingly, with Basin INP receiving the biggest reductions. Additional figures showing the impact of the levee and stormwater improvements on ponding are included in Appendix J.

TABLE 6-1

Comparison of River Elevation with Peak Flow

Date	Rainfall	Max Gage Height (ft.)	Flow (mgd)	High Flow Attributed Primarily to:
January 3-7, 2015				
1/3/2015	0.23	10.10	2.9210	
1/4/2015	8.57	10.44	13.8550	Rain
1/5/2015	0.10	13.05	20.6010	River Elevation/Rain
1/6/2015	0.01	12.28	11.3850	
1/7/2015	0.00	11.65	6.7400	
December 7-11, 2015				
12/7/2015	1.79	12.16	10.2930	
12/8/2015	3.04	12.87	16.3140	Rain
12/9/2015	0.66	14.58	11.0910	
12/10/2015	0.58	15.3	9.8390	
12/11/2015	0.67	14.59	7.6810	
January 20-23, 2016				
1/20/2016	2.32	10.84	9.3660	
1/21/2016	3.43	12.10	20.5030	Rain
1/22/2016	0.10	13.60	12.8160	
1/23/2016	0.18	12.75	6.8450	
November 23-27, 2016				
11/23/2016	2.25	9.72	9.5050	
11/24/2016	2.12	11.26	18.9330	Rain
11/25/2016	0.99	12.17	10.7140	
11/26/2016	0.50	12.11	10.5900	
11/27/2016	0.56	12.24	7.3780	
November 13-23, 2017				
11/13/2017	2.26	10.59	12.6600	
11/14/2017	2.20	10.81	14.6680	Rain
11/15/2017	0.70	11.75	11.4600	
11/16/2017	0.46	11.39	7.9790	
11/17/2017	0.12	10.88	5.9910	
11/18/2017	0.01	10.59	4.6730	
11/19/2017	2.08	10.79	8.2870	
11/20/2017	0.56	11.36	8.2630	
11/21/2017	2.02	11.55	13.5510	Rain
11/22/2017	1.45	13.32	11.9340	
11/23/2017	0.57	13.21	10.3140	

(1) Blue shading indicates peak gage height during period.
(2) Yellow shading indicates highest flows during period.

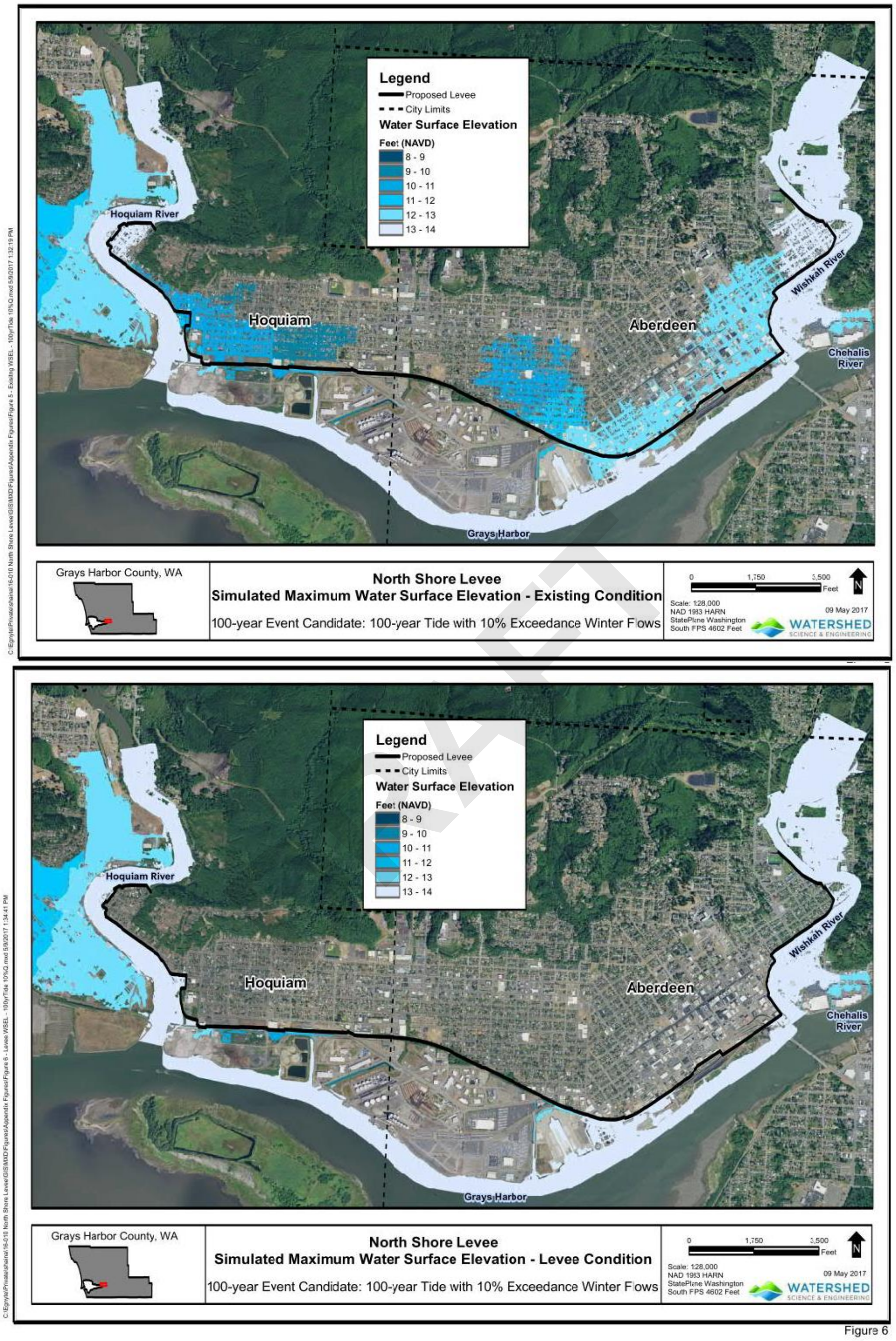


FIGURE 6-5

Simulated Water Surface Elevation – with and without Levee
Watershed Science and Engineering, 2017

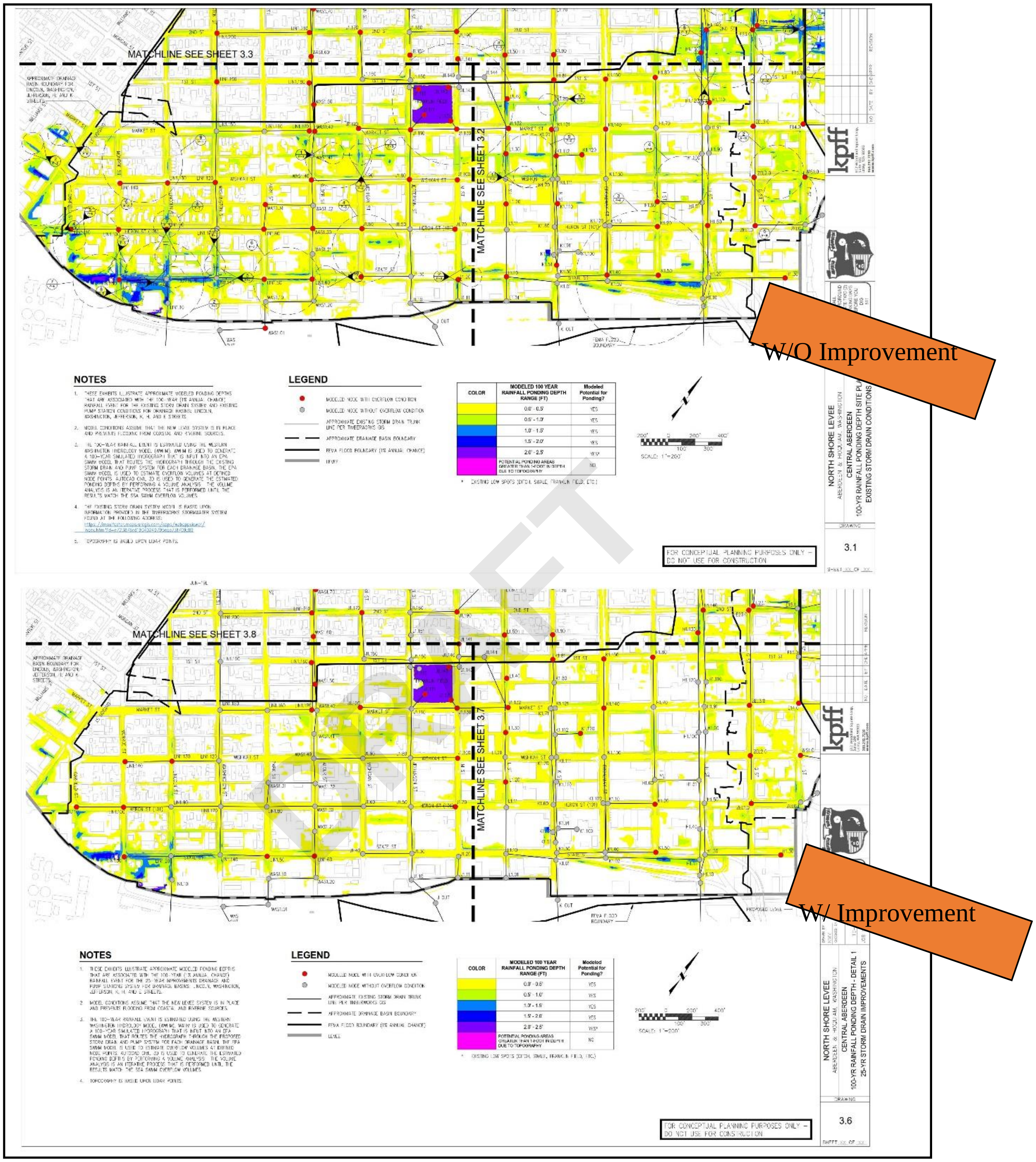


FIGURE 6-6

Simulated Ponding Condition – with and without Storm System Improvements
KPFF, 2018

IMPACT ON PEAK FLOWS

Based on this analysis, it was concluded that construction of the North Shore Levee will significantly reduce inflow to the sanitary sewer due to the reduction in potential of river water from entering the City during extreme high river flood events. In addition, the stormwater pumping and conveyance improvements will reduce peak flows caused by precipitation-induced ponding. It is estimated that these improvements will lead to inflow reductions of 25 percent, 15 percent and 15 percent within Basins INP, 5 and 13, respectively. This corresponds to reductions of 12.2 percent peak inflow or 11.4 percent to overall peak I/I in the entire collection system (and 10.1 percent peak day flow, or 9.1 percent peak hour flow reduction at the WWTP in year 2038). The reduction of the peak flow is incorporated into the future flow projection in Chapter 5, the WWTP capacity evaluation in Chapter 7 and collection system improvement later in this chapter.

CITY OBSERVATIONS

City operations staff have reported significant surcharging, with the potential for sanitary sewer overflows, occurs in the manholes immediately downstream of Pump Station 13 particularly during peak storms when all three pumps are in operation. In addition, similar surcharging and overflows have been reported in recent years during peak wet weather events near Pump Station 5 at manholes at the intersections of Arthur Avenue and B Street, Grant Street and Arthur Street, and Cleveland Street and Grant Street.

HYDRAULIC MODEL

A hydraulic model of the City's collection system was developed using XPSTORM software. The modeling was limited to the major trunk sewers only. The output from this model was used to evaluate the capacity of the existing collection system and to identify improvements that will be necessary to provide adequate service in the future. The model is intended to be updated and maintained periodically and to be used as a tool to aid in future planning and design efforts. The modeling outputs from this effort are included in Appendix G.

MODEL ELEMENTS

The sewage inventory data (i.e., manholes, pipes, pump stations, and force mains) was obtained from the City's 1999 I/I report and GIS database. Following the import of data to XPSTORM, the model network map was visually inspected to correct geographic errors such as mismatched crowns, missing data, or reverse slopes. Where invert elevations of manholes were missing, the invert elevations were linearly interpolated between known inverts. Where manhole rim elevations were unknown, LIDAR contours were used to estimate the ground elevations at the manhole locations.

For simplicity, all pump stations were modeled as constant-discharge pumps, so that the pump stations produce a constant discharge at full pump flow regardless of head conditions. (This is generally conservative, as Variable Frequency Drives will reduce pump output from peak capacity, depending on actual flows and wet well levels.)

Modeled network data for pipes, manholes, and pump stations are included in Appendix H.

When possible, identifying names for pipes and manholes were taken directly from the City's GIS database, in order to be consistent with the City's identifications. The model requires each pipe and manhole to be designated with a unique identifier. Pump station wet wells, pumps, and force mains were assigned labels based on the pump station's name.

FLOW DISTRIBUTION

The model was developed to assess the collection system flows during peak flow events. The peak hour flow was calculated using a peak hour/peak day peaking factor of 1.15, based on historical flow records. Peak hour flows for each basin, which is the total of base-flow, infiltration and inflow, are summarized in Table 6-2.

The base flow and infiltration levels developed for the individual basins were then distributed evenly across each particular basin on a per-node basis. The inflow was also distributed evenly across each basin, except for basins on low-land areas that would be affected by rainfall ponding, such as Basins INP and 13 in North Aberdeen and Basin 2 in South Aberdeen. For Basins INP and 13, different magnitudes of inflow, identified as Levels 1 through 4, were assigned to each node depending on the ponding depth indicated in the *North Shore Levee, 100-Year Rainfall Ponding Depth Site Plans* (KPFF Consulting Engineers, 2018). In Basin INP, loadings were also allocated to reflect the surcharging in the Grant Street area, in accordance with City observations. Ponding in Basin 2 (South Aberdeen) was not evaluated by KPFF, so the inflow rate was varied based on the topography and depth of manhole. Figure 6-7 shows the estimated distribution of ponding levels. Table 6-3 summarizes the hydraulic loading per node by basin and ponding level.

Regional conveyance was also evaluated with the model. However, due to a lack of available capacity in the Aberdeen pipes that could convey flow from Hoquiam to the Aberdeen WWTP, it is assumed that, if Hoquiam is connected, a new force main would be constructed to convey Hoquiam flows all the way to the Aberdeen WWTP. Central Park flows are presumed to be conveyed to the Aberdeen WWTP through the force main that serves the county landfill east of the city limits. This force main discharges to the gravity sewer upstream of Pump Station 4, which conveys wastewater under the Wishkah River to the State Street gravity interceptor, which conveys the wastewater to the WWTP. Assuming Central Park is sewered and connected, peak flow from Central Park is projected to be 1.14 mgd by 2038.

MODELING SCENARIOS

Three sewer modeling scenarios were evaluated, with model runs within each scenario representing different Influent Pump Station wet well conditions. Preliminary analysis showed that Influent Pump Station wet well affected surcharging due to backwater effects in a number of runs, necessitating the runs at the various wet well levels. The scenarios included:

1. Current peak hour flows from Aberdeen and Existing Partners, with separate model runs for various depths in the Influent Pump Station wet well

   These scenarios simulated the current base and peak hour I/I flows to the City's sewer system as it currently exists. The peak flow represents an approximately 10-year storm event, which was the average of the three largest storm events that occurred during the past 5 years. The return period of the rainfall was identified using a Washington State Isopluvials Map created by MGS Engineering, Inc.

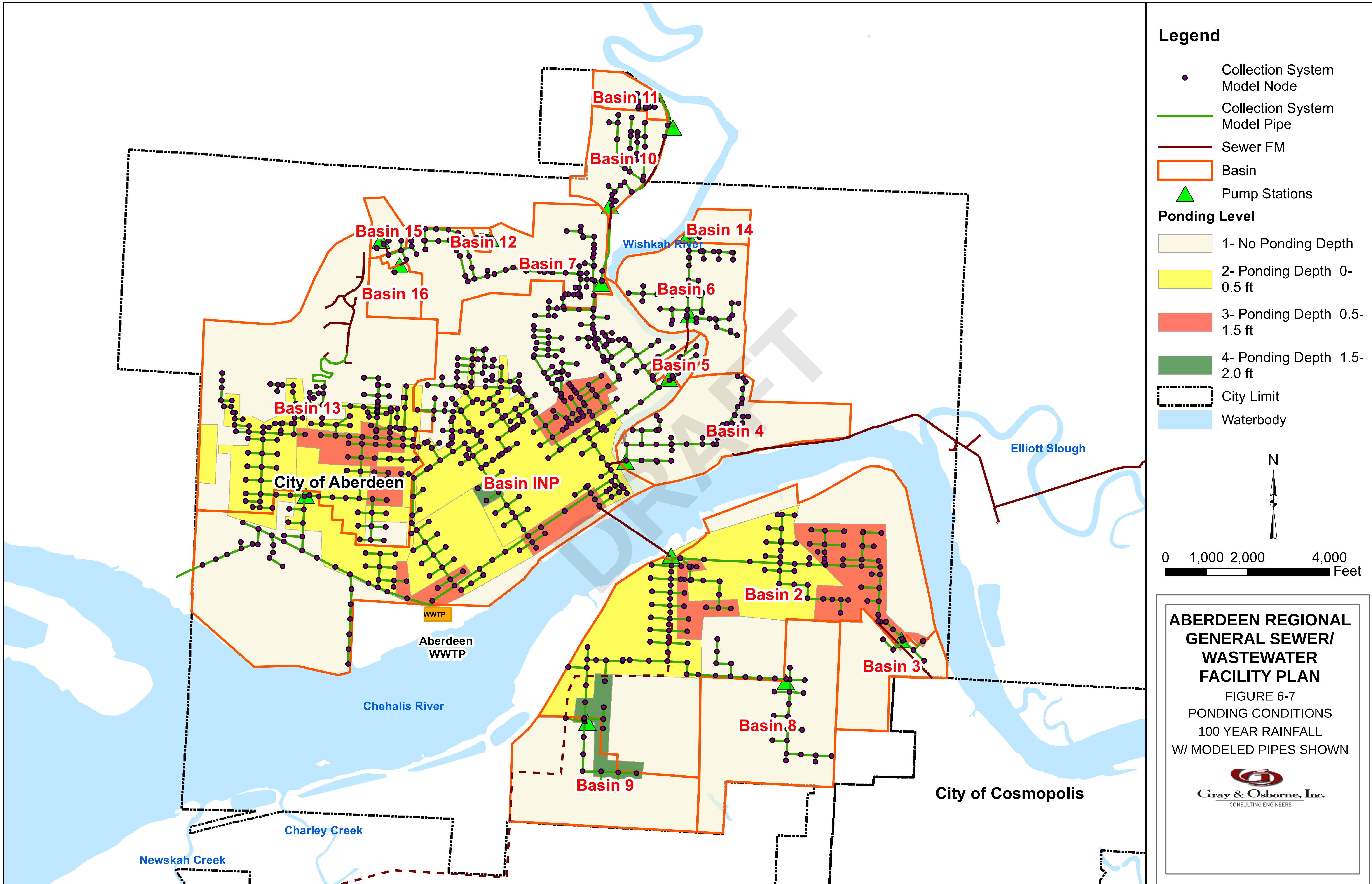
2. Year 2038 flows from Aberdeen and Existing Partners, with separate model runs for various depths in the Influent Pump Station wet well

   This scenario simulated (1.) the increased base flow due to growth in ERUs, and (2.) the same peak hour I/I flow as in the current peak hour scenarios, except that the inflow in Basins INP, 5 and 13 were reduced, as described above, due to the reduction in ponding caused by the North Shore Levee project.

3. Year 2038 Expanded Regional flows, with separate model runs for various depths in the Influent Pump Station wet well.

Gravity pipes throughout the system were assigned a Manning's roughness coefficient of 0.014, which reflects the roughness of old concrete pipe. This coefficient is considered to be a conservative parameter in the model, as some of the City's newer sewers consists of plastic pipes which are smoother and generally have greater capacity.

All pump stations were assumed to be operating at their rated full capacity.



ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 6-7

PONDING CONDITIONS

100 YEAR RAINFALL

W/ MODELED PIPES SHOWN

TABLE 6-2

Projected Peak Hour Flows by Basin

Basin	Base-Flow (gpd)		Infiltration (gpd)	Inflow(gpd)			Total (gpd)	
	Current	Projected 2038(1)	Current and Projected 2038(2)	Current	Projected 2038(3)	2038 Inflow Reduction Provided by Levee	Current	Projected 2038
INP	1,037,131	1,362,613	134,015	4,597,292	3,447,969	25%	5,768,438	4,944,597
2	31,643	41,573	393,760	4,481,799	4,481,799		4,907,202	4,917,133
3	1,857	2,440	2,571	168,371	168,371		172,800	173,383
4	63,357	83,241	20,186	717,484	717,484		801,027	820,910
5	19,286	25,338	29,798	1,390,917	1,182,279	15%	1,440,000	1,237,415
6	31,643	41,574	27,087	742,432	742,432		801,162	811,093
7	8,041	10,564	20,702	187,258	187,258		216,000	218,523
8	7,143	9,384	5,571	851,286	851,286		864,000	866,242
9	46,000	60,436	21,114	1,315,286	1,315,286		1,382,400	1,396,836
10	11,571	15,203	8,781	51,648	51,648		72,000	75,631
11	5,714	7,508	286	498,000	498,000		504,000	505,793
12	11,657	15,316	27,429	536,914	536,914		576,000	579,658
13	241,071	316,727	122,529	3,524,400	2,995,740	15%	3,888,000	3,434,995
14	2,500	3,284	1,628	427,873	427,873		432,000	432,784
15	11,700	15,372	3,500	172,000	172,000		187,200	190,872
16	6,686	8,784	806	179,709	179,709		187,200	189,298
Cosmo	119,500	157,003	84,640	533,458	533,458		737,598	775,100
Landfill	1,500	1,971	5,600	55,873	55,873		62,973	63,444

(1) Projected base flow of each basin is calculated by multiplying the current basin baseflow by 1.31, the ratio of future to current City base flow, (1.31 = 2.47 mgd/1.88 mgd).

(2) Infiltration is assumed to stay the same throughout the planning period.

(3) Inflow is assumed to stay the same throughout the planning period, except for Basin INP, Basin 5 and Basin 13 which are expected to generate less inflow due to the North Shore Levee project.

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TABLE 6-3

Loading Per Node by Basin and Ponding Level(1)

Basin	No. of Loading Nodes	Ponding Level 1 Nodes			Ponding Level 2 Nodes			Ponding Level 3 Nodes			Ponding Level 4 Nodes		
		Node Count	Current Loading Per Node (gpd/node)	2038 Loading Per Node	Node Count	Current Loading Per Node (gpd/node)	2038 Loading Per Node	Node Count	Current Loading Per Node (gpd/node)	2038 Loading Per Node	Node Count	Current Loading Per Node (gpd/node)	2038 Loading Per Node
INP (Influent PS) Basin(2)	386	150	4,483	1,778	178	5,073	3,457	54	7,848	5,136	4	14,785	9,333
Grant St adjacent area in INP Basin(2)	2	2	1,855,000	1,855,000									
2	140	9	11,202	11,273	70	27,529	27,600	56	43,856	43,927	5	84,674	84,745
3	8	8	21,600	21,673									
4	42	42	19,072	19,545									
5	13	13	110,769	95,186									
6	46	46	17,417	17,632									
7	80	80	2,700	2,732									
8	21	21	41,143	41,250									
9	13	13	106,338	107,449									
10	38	38	1,895	1,990									
11	20	20	25,200	25,290									
12	5	5	115,200	115,932									
13	162	37	9,775	9,113	97	24,837	21,915	28	39,898	34,717			
14	5	5	86,400	86,557									
15	4	4	46,800	47,718									
16	1	1	187,200	189,298									
Cosmo	1	1	737,598	775,100									
Landfill	1	1	62,973	63,444									

(1) The relative magnitudes of inflow for Ponding levels 1, 2, 3, 4 are 1x, 3x, 5x and 10x, respectively.
(2) Besides considering ponding level, loadings were also adjusted to reflect the Grant St surcharging.

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MODEL RESULTS

Current gravity main capacity deficiencies identified in the modeling effort are discussed below. The locations of projected sanitary sewer overflows under various model runs under the three scenarios are shown in Figures 6-8 and 6-9, while the percentages of peak hour flow to full pipe capacities are summarized in Figures 6-10 through 6-15. As described above, water levels in the Influent Pump Station wet well affected surcharging at distant manholes. In Figures 6-8 and 6-9, manholes are shown as follows:

- **Yellow** if they surcharge to the point of an overflow with a >**2-foot** water depth in the Influent Pump Station wet well and **2038** peak hour flows from **Aberdeen and Existing Partners**.

- **Blue** if they surcharge to the point of an overflow with a >**4-foot** water depth in the Influent Pump Station wet well and **2038** peak hour flows from **Aberdeen and Existing Partners**.

- **Purple** if they surcharge to the point of an overflow with a >**6-foot** water depth in the Influent Pump Station wet well and **2038** peak hour flows from **Aberdeen and Existing Partners**.

- **Red** if they surcharge to the point of an overflow with **any** water depth in the Influent Pump Station wet well and **2038** peak hour flows from **Aberdeen and Expanded Regional Partners**.

- **A concentric circle** if they surcharge to the point of an overflow with a **>6-foot** depth in the Influent Pump Station wet well and **current** peak hour flows from **Aberdeen and Existing Partners**. (Six feet, we understand is the approximate depth in the Influent Wet Well under peak flow conditions.)

In Figures 6-10 through 6-15, modeled pipes are shown as follows:

- **Blue** if they were **<85 percent** capacity at peak hour flows under the specified conditions.

- **Purple** if they are **85 to 100 percent** of capacity at peak hour flows under the specified conditions.

- **Gold** if they are **100 to 130 percent** of capacity at peak hour flows under the specified conditions.

- **Red** if they are **>130 percent** of capacity at peak hour flows under the specified conditions.

Model Scenario 1: Current Flows from Aberdeen and Existing Partners

Under Model Scenario 1 (Current Flows from Aberdeen and Existing Partners), several pipelines with insufficient capacity were noted in the modeling results. In North Aberdeen, the capacity deficiencies are primarily in pipes downstream of Pump Station 11 and Pump Station 13, the pipe section along Market Street and State Street. In South Aberdeen, the deficiencies are primarily in pipes receiving flow from Pump Station 3 and the Cosmopolis discharge, as well as pipes downstream of Pump Station 8 and Pump Station 9. Some capacity issues are identified in areas with low pipe slopes.

The two main interceptors, the Railroad interceptor, between Pump Station 13 and the WWTP, and the State Street interceptor, conveying flow from the rest of the City to WWTP, have capacities of 3.7 mgd and 17.8 mgd, respectively. Some pipe segments in the Railroad interceptor and State Street need to be upsized to convey the peak flow, if I/I cannot be reduced sufficiently.

Model Scenario 2: Projected Year 2038 Flows from Aberdeen and Existing Partners

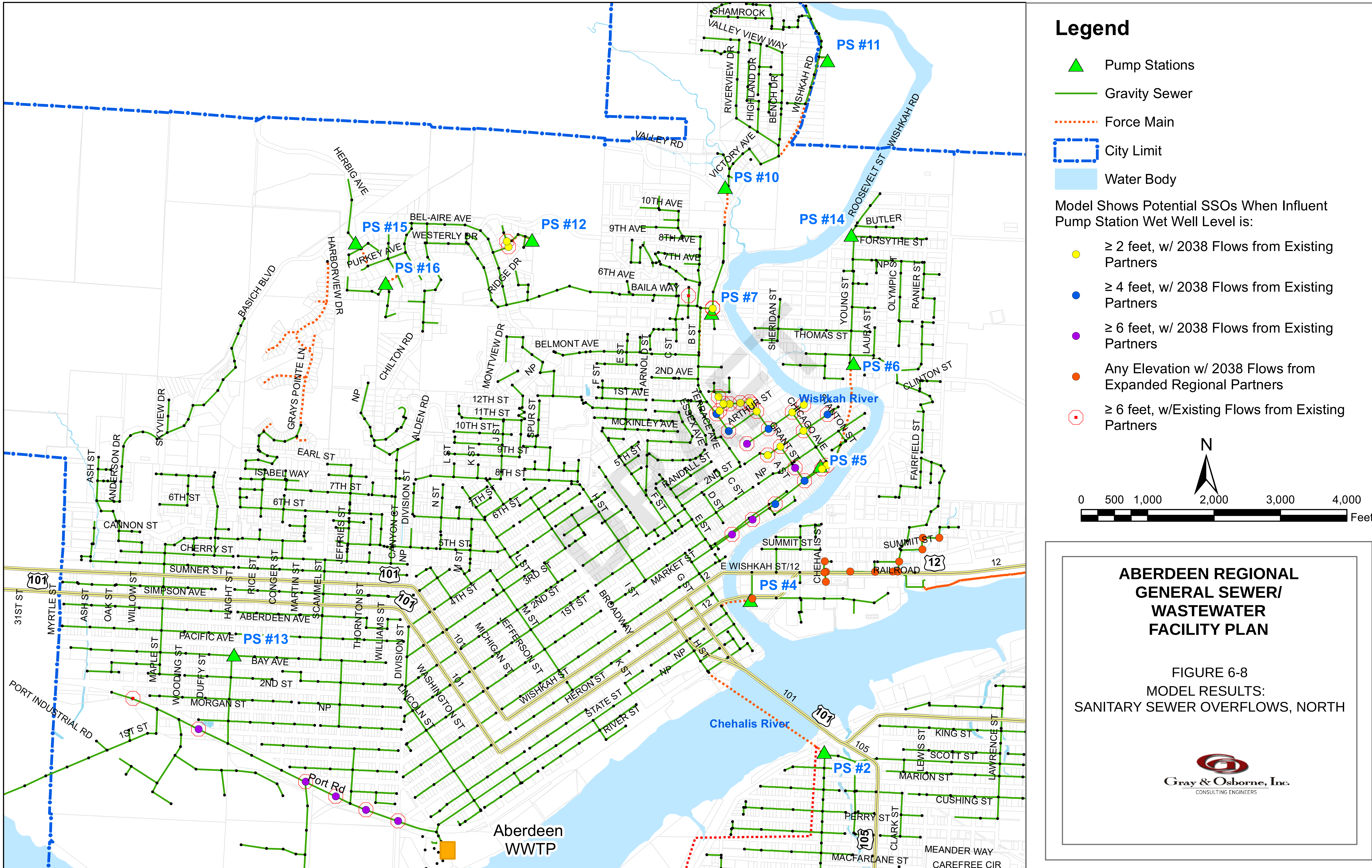
The flows in Basins INP, 5 and 13 in North Aberdeen are projected to be reduced due to the North Shore Levee project. The degree of flow reduction due to completion of the levee project is expected to eliminate the need for upgrade of some existing pipes identified as over-capacity in Model Scenario 1. However, most of the deficiencies will be alleviated but not all and a few existing pipes will need to be replaced to achieve the needed conveyance capacity. It is possible that flow reductions due to the North Shore Levee project could exceed expectations, and/or the City can reduce other I/I sources, in order to address these capacity concerns. Future monitoring and evaluation are recommended upon the completion of the Levee project.

Model Scenario 3: Projected Year 2038 Flows, Expanded Regional Flows

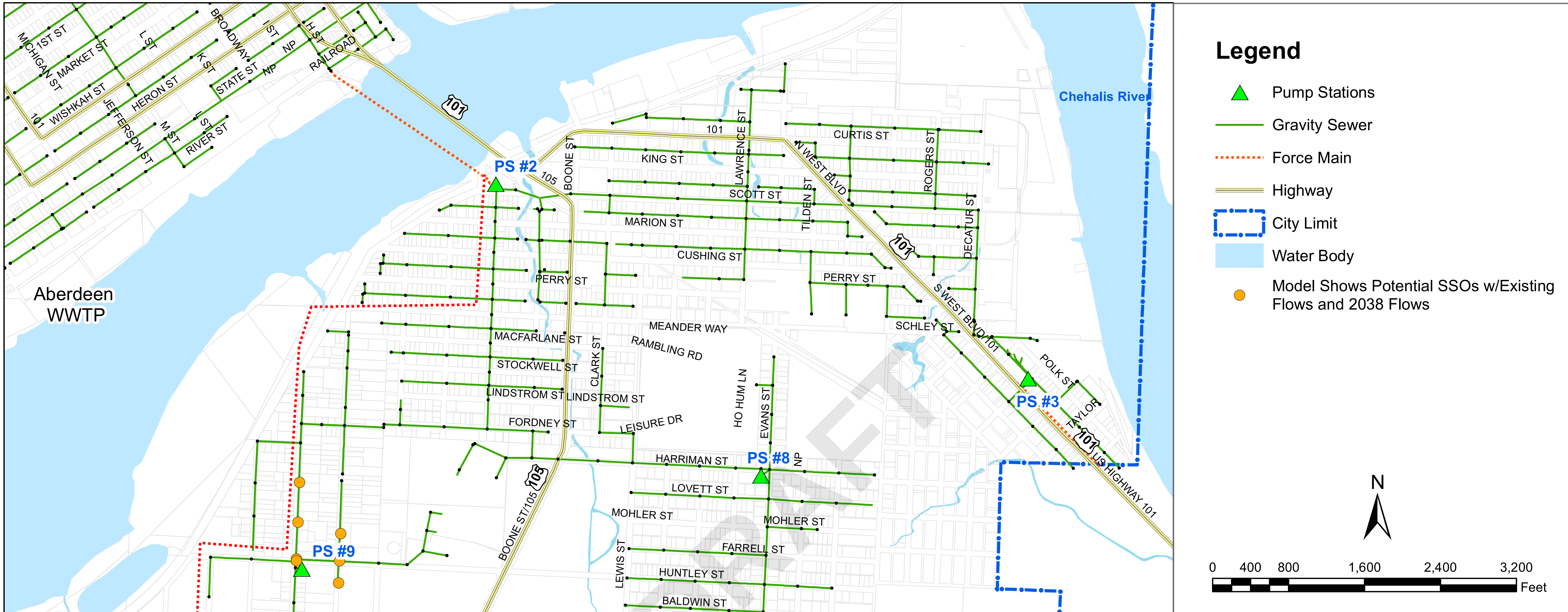
As noted earlier, if Aberdeen accepts flows from Hoquiam, it is assumed they will be conveyed all the way to the Aberdeen WWTP, due to the low slopes and capacity limitations in the pipelines in west Aberdeen pipelines. However, it is assumed that Central Park flows would be conveyed through the force main serving the landfill, and through a gravity section, before being conveyed by Pump Station 4. Under these conditions, flows would exceed the capacity of the lines along Highway 12.

Projected Sanitary Sewer Overflows

The modeling identified several areas at risk of sanitary sewer overflows (SSOs) under peak flow conditions. As shown in Figures 6-10, 6-12 and 6-14, several pipes in the vicinity of Grant Street and Market Street have peak flows under all scenarios exceeding 130 percent of capacity. Because of this, as shown in Figure 6-8, there is risk of overflows at several locations near Grant Street, Arthur Street and Chicago Avenue, and



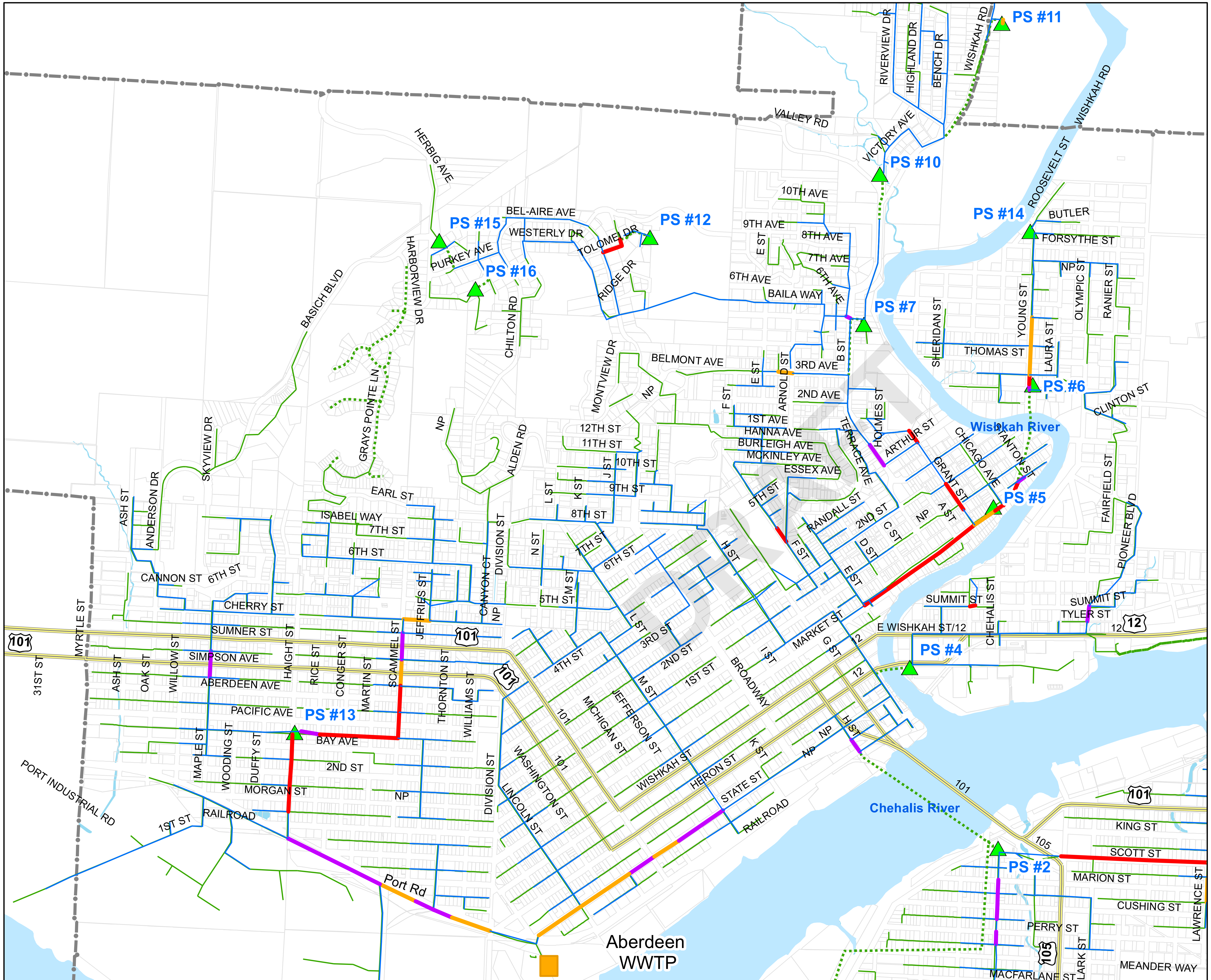
Legend
Pump Stations
Gravity Sewer
Force Main
City Limit
Water Body
Model Shows Potential SSOs When Influent Pump Station Wet Well Level is:
≥ 2 feet, w/ 2038 Flows from Existing Partners
≥ 4 feet, w/ 2038 Flows from Existing Partners
≥ 6 feet, w/ 2038 Flows from Existing Partners
Any Elevation w/ 2038 Flows from Expanded Regional Partners
≥ 6 feet, w/Existing Flows from Existing Partners
N
0 500 1,000 2,000 3,000 4,000
Feet
ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN
FIGURE 6-8
MODEL RESULTS: SANITARY SEWER OVERFLOWS, NORTH
Gray & Osborne, Inc.
CONSULTING ENGINEERS
PS #2
PS #4
PS #5
PS #6
PS #7
PS #10
PS #11
PS #12
PS #13
PS #14
PS #15
PS #16
Wishkah River
Chehalis River
Aberdeen WWTP



ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 6-9
MODEL RESULTS:
SANITARY SEWER OVERFLOWS, SOUTH

Gray & Osborne, Inc.
CONSULTING ENGINEERS



Legend

- Pump Stations
- Gravity Sewer
- Force Main
- City Limit
- Water Body

Percent of Free Flow Gravity Pipe Capacity at Projected Peak Hour Flow

- < 85 %
- 85 % - 100 %
- 100 % - 130 %
- > 130 %

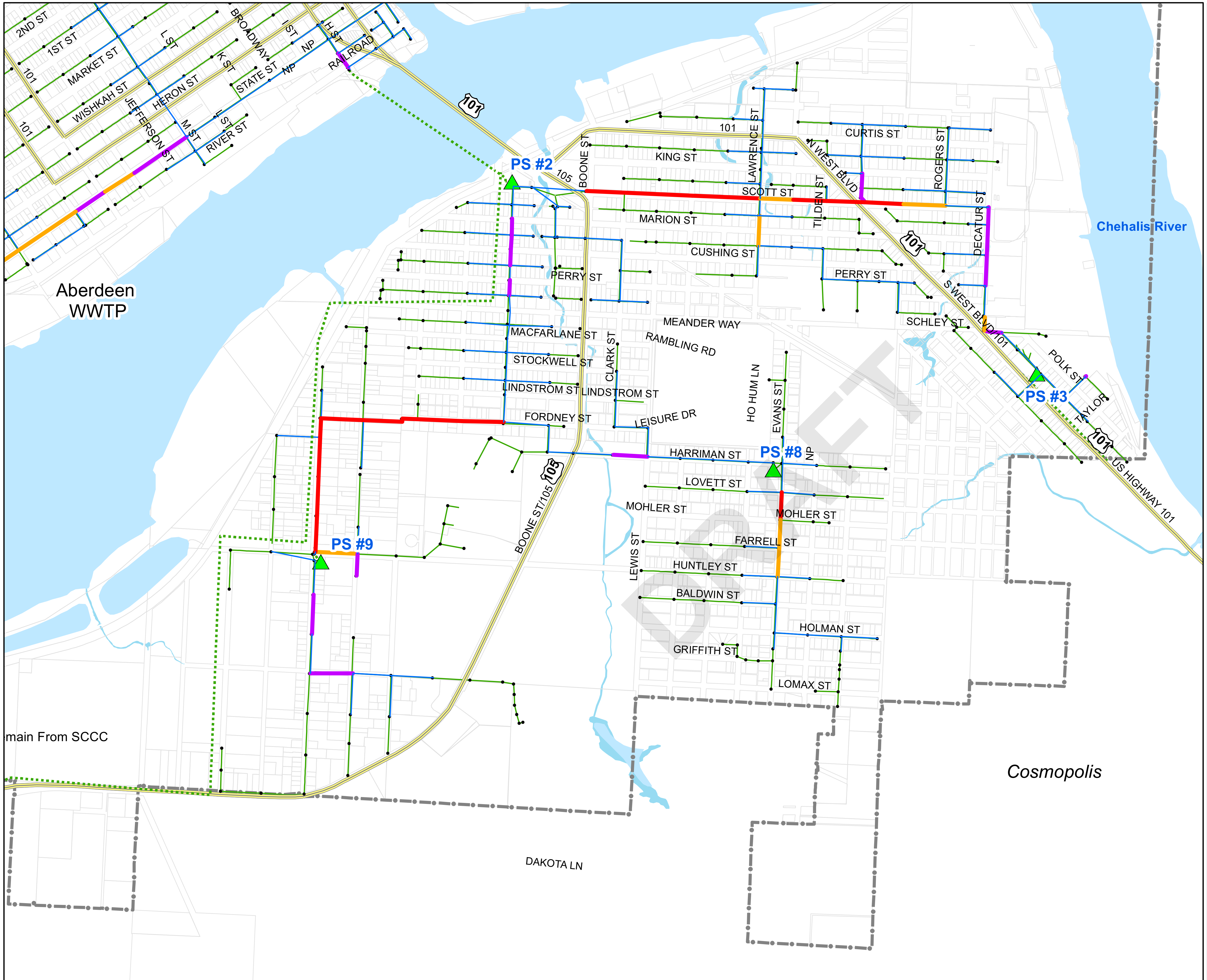
N

0 400 800 1,600 2,400 3,200 Feet

ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 6-10
MODEL RESULTS:
EXISTING FLOWS, NORTH

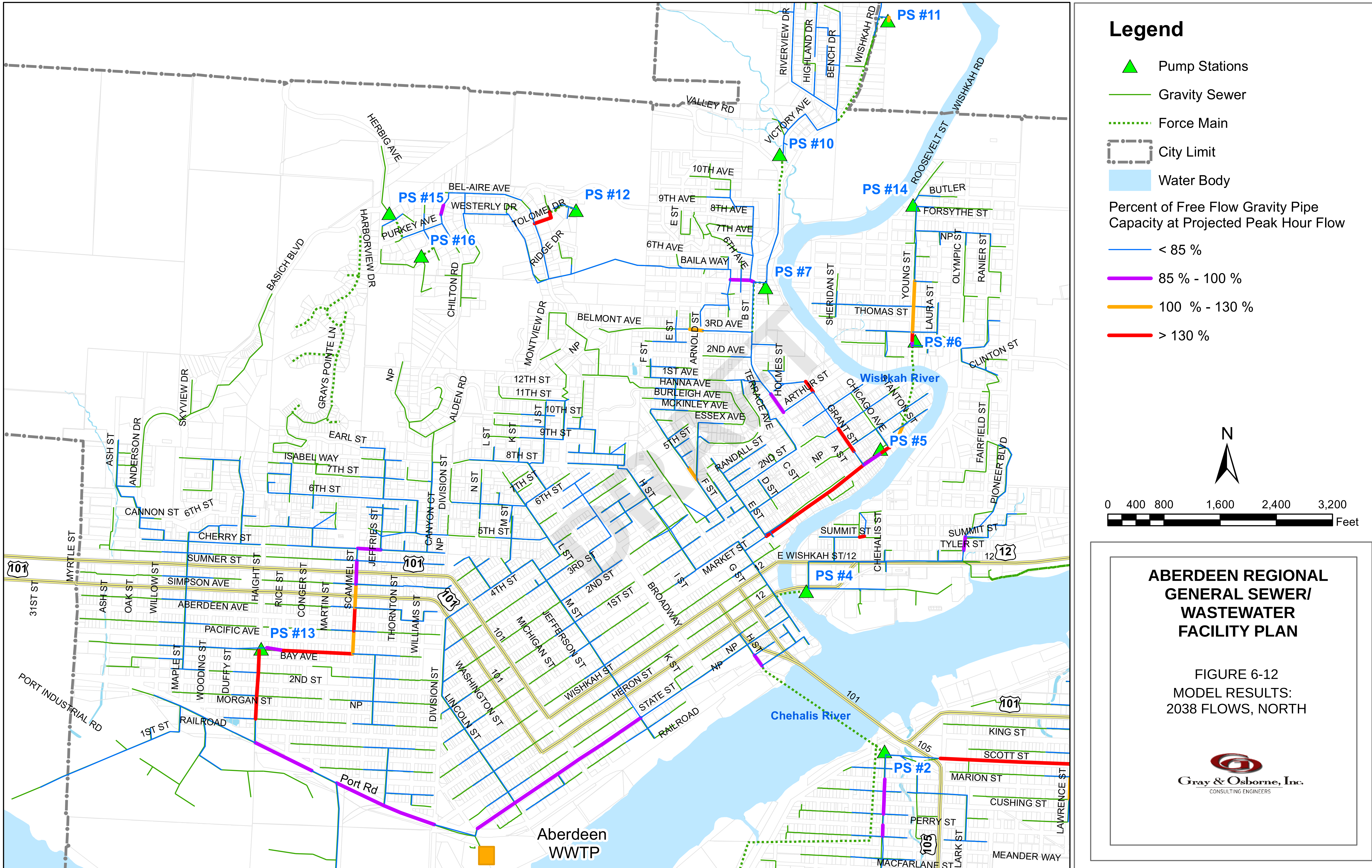
Gray & Osborne, Inc.
CONSULTING ENGINEERS



ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

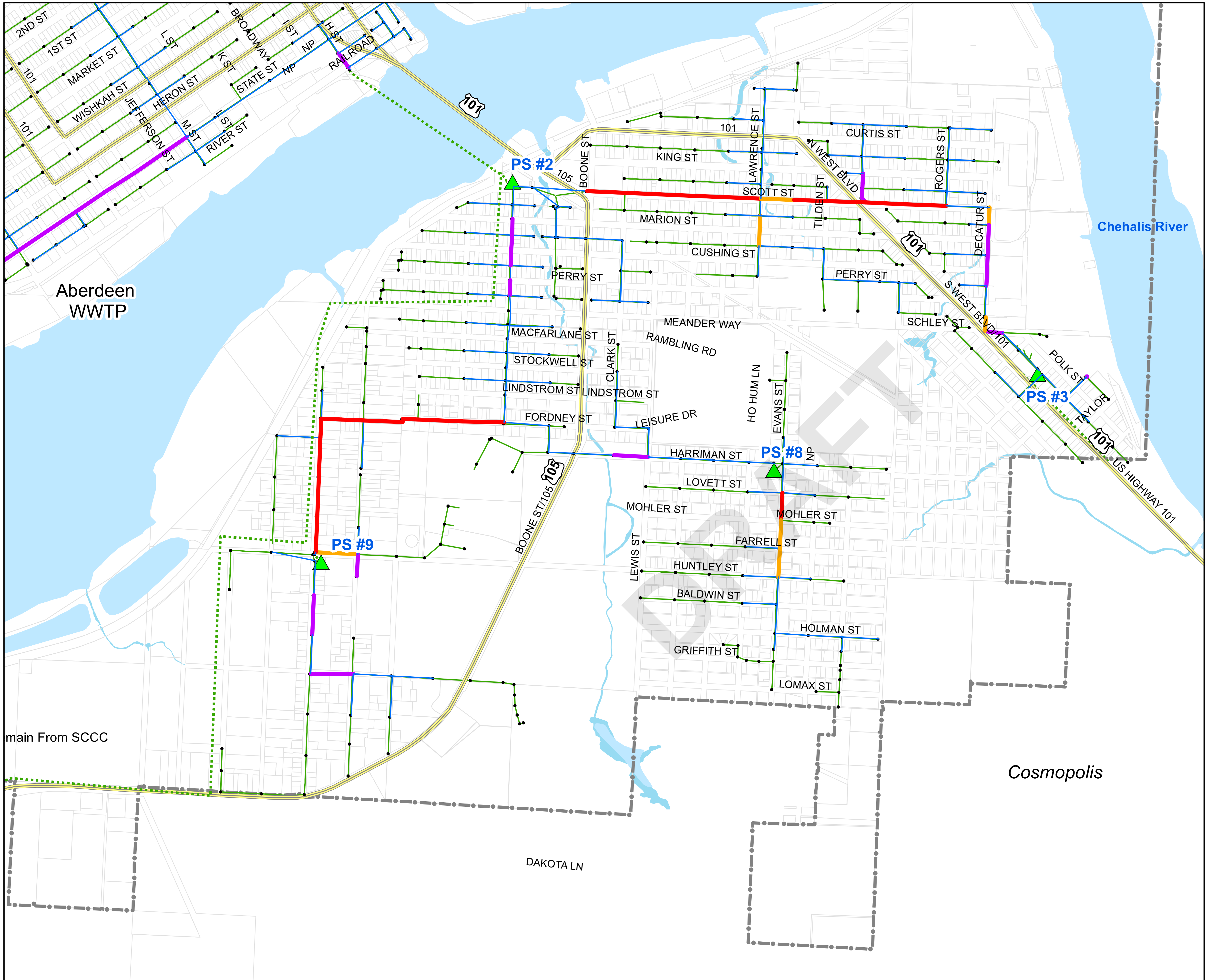
FIGURE 6-11
MODEL RESULTS:
EXISTING FLOWS, SOUTH

Gray & Osborne, Inc.
CONSULTING ENGINEERS



ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 6-12
MODEL RESULTS:
2038 FLOWS, NORTH



Legend

- Pump Stations
- Highway
- Gravity Sewer
- Force Main
- City Limit
- Water Body

Percent of Free Flow Gravity Pipe Capacity at Projected Peak Hour Flow

- < 85 %
- 85 % - 100 %
- 100 % - 130 %
- > 130 %

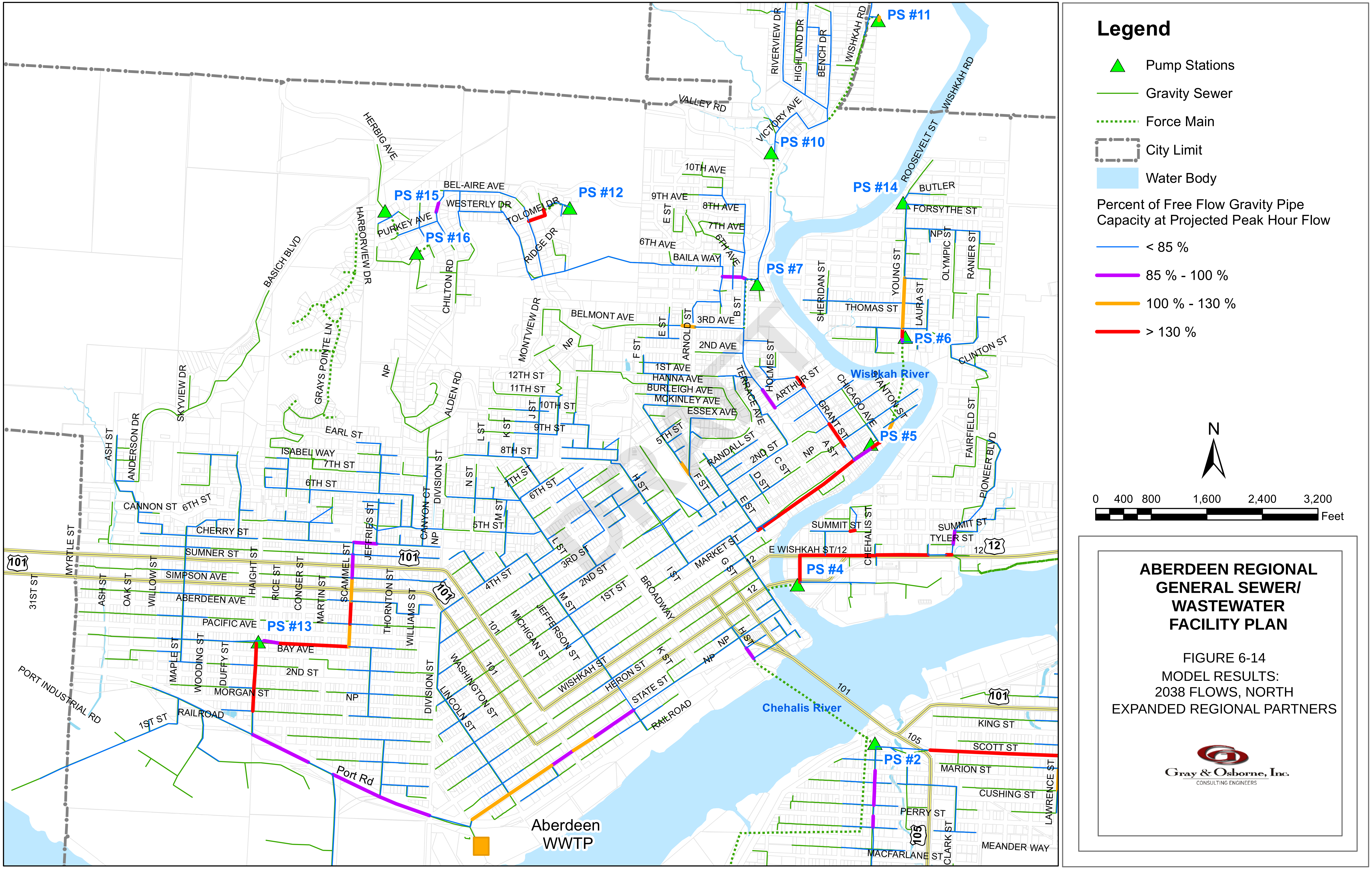
N

0 400 800 1,600 2,400 3,200 Feet

ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 6-13
MODEL RESULTS:
2038 FLOWS, SOUTH

Gray & Osborne, Inc.
CONSULTING ENGINEERS



ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 6-14
MODEL RESULTS:
2038 FLOWS, NORTH
EXPANDED REGIONAL PARTNERS

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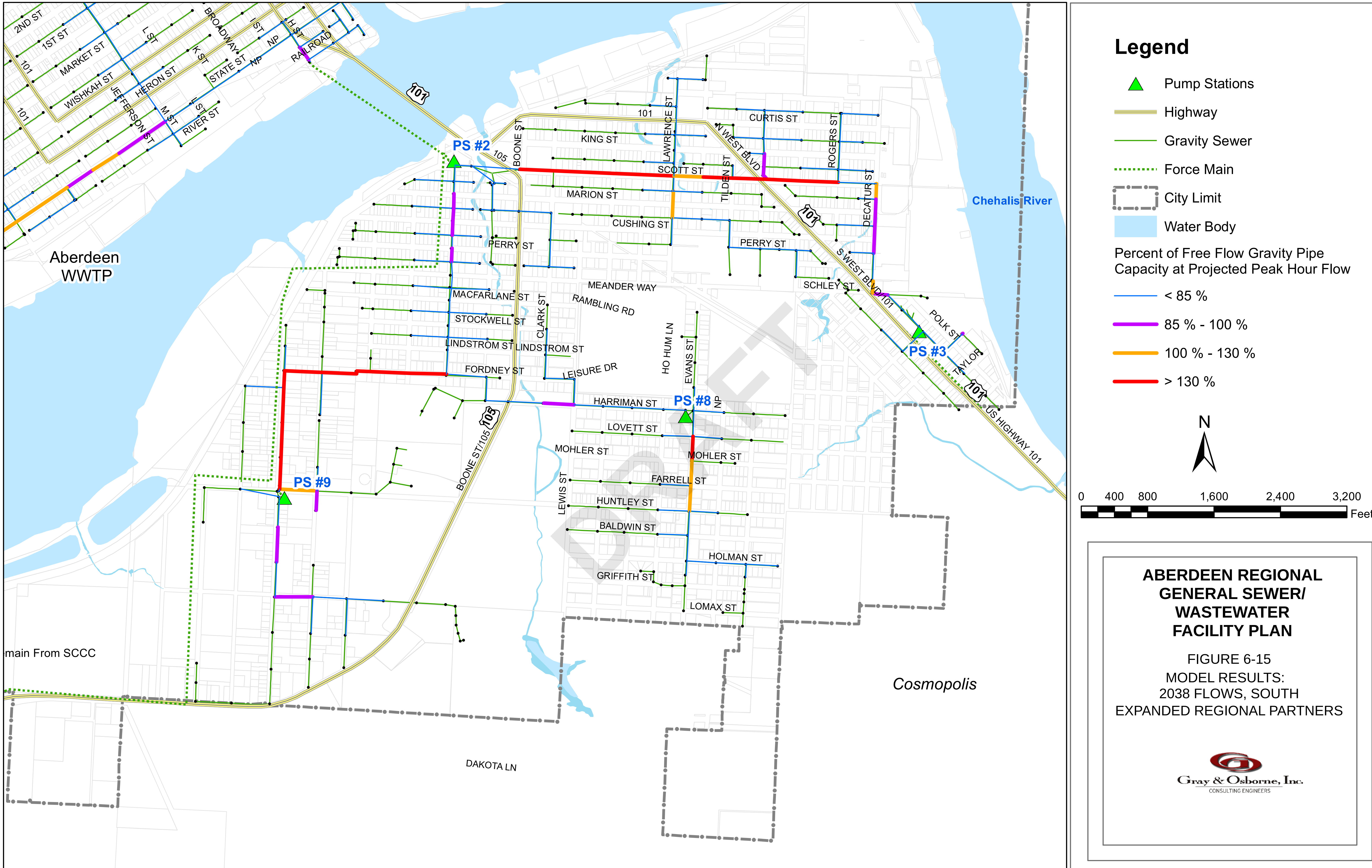
CONSULTING ENGINEERS



ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 6-15
MODEL RESULTS:
2038 FLOWS, SOUTH
EXPANDED REGIONAL PARTNERS

Gray & Osborne, Inc.
CONSULTING ENGINEERS

the risk is exacerbated when the wet well level in the Influent Pump Station is higher. Similarly, manholes along Port Road are at risk of overflows due to capacity limitations (associated with flat slope) in that line, and the risk is increased by high Influent Pump Station wet well levels. In addition, as shown in Figure 6-9, due to capacity limitations, several manholes are at risk of overflows in South Aberdeen to the west of Highway 105 under all scenarios. Finally, as discussed above, acceptance of peak flows from Central Park would cause risk of overflows in the line upstream of Pump Station 4.

As most of the peak day and peak hour flows are due to excessive I/I, the City might be able to reduce I/I in the upstream basins sufficiently to eliminate the need for improvements to increase the flow capacity

COLLECTION SYSTEM CAPACITY EVALUATION

PIPING CAPACITY EVALUATION

Potential system capacity deficiencies were identified based on the modeling results. Maximum pipe surcharge conditions and the ratio of maximum flow to full capacity are summarized in Table 6-4. The under-capacity gravity segments are generally the root cause of the surcharging. The segments with capacity issues that were shown are categorized in Table 6-4 and indicated in Figure 6-16. The potential improvements typically involve increasing pipe sizes, or reducing peak flows. It should be noted that the modeling was completed assuming steady-state conditions with a 10-year storm, which is considered to be quite conservative.

The capacity evaluation for the force mains is tied directly to the pump station capacity evaluation. The capacity of each force main is based on a maximum design velocity of 8 feet per second (fps). The results of this evaluation are shown in Table 6-6.

TABLE 6-4

Sewer System Hydraulic Model Results: Surcharging Pipes Identified

Pipe Section	Basin	Location	Issue		Length (ft)
			Surcharge Status(1)	Pipe Maximum Flow/Full Capacity	
1	13	Scammel Street, between Cherry Street and Bay Avenue; Bay Avenue, between Scammel Street and Haight Street	Allowable Surcharge	1.6	3,100
2	INP	Haight Street, between Pacific Avenue and 1st Street	Allowable Surcharge	1.5	700
3	INP	Port Road, between Haight Street and Division Street	No Surcharging	0.9	2,400
4	INP	Between Cleveland Street, Arthur Street and B Street	Allowable Surcharge	1.9	800
5	8	Evans Street, between Lovett Street and Huntley Street	Exceeds Allowable Surcharge	1.7	900
6	6	Young Street, between Hayes Street and Lafayette Street	Allowable Surcharge	2.5	800
7	INP	State Street, between M Street and Port Road	No Surcharging	1.0	2,200
8	5, INP	Market Street, between Chicago Avenue and E Street	Allowable Surcharge	2.1	2,000
9	7	Tolomei Drive, between Monterey Lane and Bel-Aire Avenue	Exceeds Allowable Surcharge	2.4	300
10	2	Scott Street, between Boone Street and Decatur Street Decatur Street, between Scott Street and Schley Street	Exceeds Allowable Surcharge	1.7	5,700
11	2,9	Fordney Street, between Mill Street and Harding Road; Harding Road, between Fordney Street and Highway 105	Exceeds Allowable Surcharge	2.2	5,000
12	2	Harding Road and Coolidge Road	Exceeds Allowable Surcharge	1.6	700

(1) Allowable Surcharging: For Pipe depth >10 ft, the allowable surcharge is 3 ft
For Pipe depth < 10 ft, the allowable surcharge is 1 ft
Shallow: Surcharge is at a depth < 2 ft from the ground surface

PUMP STATION CAPACITY EVALUATION

Table 6-5 summarizes pump run time during three of the larger storm events over the past 6 years. All of the collection system pump stations have two pumps except Pump Station 13, which has three pumps. Per Ecology redundancy requirements, pump stations must be able to convey peak flows with the largest pump out of service, so, generally, if all the pumps have come on at once, the pump station is likely over capacity. (This is clearly the case if peak day combined run times exceed 24 hours for a 2-pump station or 48 hours for a 3-pump station, and may be the case at lower combined run times. The higher the combined run times, the more likely it is that more than one pump was in operation at one time.) It is concluded that Pump Stations 2 and 7 were operating with all pumps running during all three events, since the sum of the run times for those stations was more than 24 hours. Pump Stations 4, 5, 6, 8, 9, 10, 13 and 16 were operating at "full capacity" (with the redundant pump on) during one or two large storm events. Priorities for capacity increases are indicated based on a review of the run time data; if the pump station clearly had all pumps in service during at least two or more of the three largest events between year 2013 and 2018, or if the pump station has insufficient capacity to convey the projected flow from modeling, the pump station capacity is considered to be insufficient, and the pump station is listed as High or Highest priority. To accommodate the peak flow, these pump stations need to be upgraded to increase the capacity so that the firm capacity (with one pump out of service) will be sufficient to handle the peak flow. (Alternatively, another approach is to aggressively resolve the inflow problems causing the excessive amounts of extraneous flow to enter the collection system, as discussed later in this chapter.)

Table 6-6 shows a summary of existing pump station capacity and recommended design flows for pump station upgrades. Also shown are recommended force main sizes for upgraded pump stations, and the capacity of the pipes downstream of the force mains of the pump stations. By comparing the design flows with the downstream gravity pipe capacity, it is noted that the pipes downstream of Pump Stations 6, 7, 8, 9 and 13, highlighted in yellow in table, need to be upsized to accommodate the peak discharge flow. As noted earlier, PS 13 is expected to see a significant I/I reduction; however, due to the combination of the PS 13 discharge with other flows into the downstream sewer system, it is recommended that the downstream pipes be upsized.

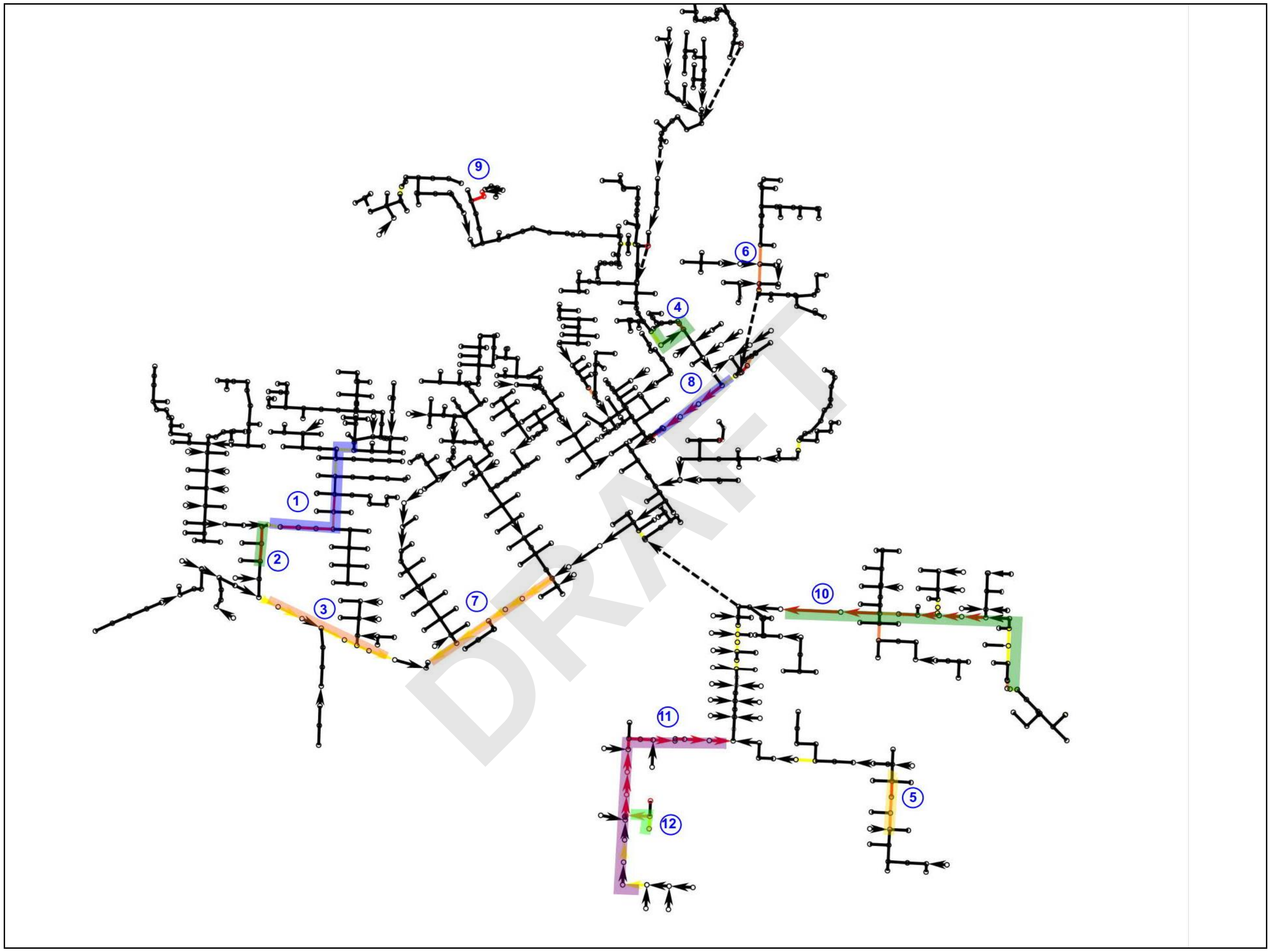


FIGURE 6-16

Summary of Major Pipes with Modeled Flows Exceeding Capacity

TABLE 6-5

City of Aberdeen Collection System Pump Run-Time During Peak Flow Events

PS	1/4/2015 Event Pump Run Hour			1/5/2015 Event Pump Run Hour			1/21/2016 Event Pump Run Hour			Percentage of Peak Flow Events with All Pumps in Operation	Priority for Capacity Increase	Peak Pumped Flow (gpm)
	No. 1	No. 2	No. 3	No. 1	No. 2	No. 3	No. 1	No. 2	No. 3			
2	*18.2*	*20.6*		*19.7*	*19.8*		*21*	*22.2*		100%	Highest	5,310
3	7.3	6.5		6	4.8		12.8	8.5		Possibly 33%		120 or 240
4	8.1	6.6		*23.9*	*8.7*		6.8	7.1		33%	High	700
5	1	1.6		0.1	3.9		*19.4*	*20.8*		33%	High	440
6	12	16.1		5.6	5.3		5.2	5.8		33%	Medium	856
7	*20.1*	*19.9*		*17.6*	*16.5*		*19.3*	*19.4*		100%	Highest	1,000
8	8.4	5.4		*23.7*	*23.7*		14	9.6		66%	High	800
9	-(1)	-(1)	-(1)	-(1)	-(1)		*15.6*	*17.4*		100%	High	800
10(6)	7	8.1		2.8	3.2		*24.4*	*2.2*		33%	Medium	580
11	1.6	1.2		0.7	0.7		0.5	0.7		0%		500
12	2.9	10.7		9	1.2		2.5	3.7		0%		400
13	*12.3*	*14.0*	*15.0*	*18.5*	*21.0*	*22.6*	-(1)	*18.5*	*18.6*	100%	-(5)	2,700
14	2.1	1.4		0.7	0.3		0.8	0.8		0%		200
15	1.5	3.4		3.6	6.9		-(1)	13.3		0%		100
16(6)	*15.9* (6)	*7.3*		*16.6*	*4.9*		-(1)	17.2		Possibly 33- 66%		130

(1) Not Documented

(2) Lowest pipe capacity downstream of the pump station

(3) Yellow highlighting indicates all pumps operated simultaneously. Peach highlighting indicates high probability that all pumps operated simultaneously.

(4) SCCC stores flows and pump station is not in operation during peak flows.

(5) I/I reduction is assumed to reduce flows significantly.

(6) Run times noted were all measured prior to upgrade in 2019.

TABLE 6-6

Pump Station Capacity Summary

Pump Station	Existing Firm Capacity (gpm)	Basis for Firm Pump Station Capacity	Existing Full Capacity (gpm)	Basis for Full Pump Station Capacity	Priority for Capacity Increase	Recommended Design Flow, Firm Capacity (gpm) (1)	Downstream Gravity Piping Capacity (gpm)	Force Main Upsize Needed (in.)
2	3,000	Measured	5,310	Measured	Highest	5,310	5,760	--
3	120	Nameplate	240	Estimated	--	120	208	--
4(3)	500	Measured	700	Estimated	High	1,000	9,647	10
5(3)	400	Measured	440	Measured	High	Pumps replaced	1,943	6
6	650	Nameplate	856	Estimated	Medium	650	416	--
7	600	Measured	1,000	Measured	Highest	1,200	694	10
8(3)	550	Measured	800	Measured	High	800	763	--
9(3)	700	Measured	800	Measured	High	1,000	555	8
10	440	Measured	580	Measured	Medium	Pumps replaced	486	--
11	500	Nameplate	700	Estimated	--	500	555	--
12	400	Nameplate	800	Estimated	--	400 (4)	208	--
13(3)	1,800	Nameplate	2,700	Estimated	-- (5)	1,800	1,800	--
14	200	Nameplate	400	Estimated	--	200	278	--
15	100	Nameplate	130	Estimated	--	Pumps replaced	763	--
16(2)	100	Nameplate	130	Estimated	--	Pumps replaced	278	--

(1) Design flows are based on past run-time data (with adjustments to convert to peak hour) and modeling of 2038 flows.
(2) Previous run time data showed both pumps running simultaneously. Pumps upsized in 2018-2019; limited run time available since upgraded.
(3) Capacity increase for PS 13 not recommended at this time since it is expected to see a significant I/I reduction in its tributary area.
(4) As discussed later in this chapter, it may be feasible to replace PS 12 with a gravity line.

INFILTRATION AND INFLOW REDUCTION

The remaining sections of this chapter focus on development of a Capital Improvement Plan for the collection system. Many of these projects are needed to convey excessive infiltration and inflow. Prior to completing these projects, it is recommended that the City complete a more detailed sewer system evaluation, to verify bottlenecks and identify any "low hanging fruit" of infiltration and inflow. As noted in Chapter 5, EPA's criterion for excessive inflow is 275 gallons per capita day (gpcd); Aberdeen significantly exceeds that level, with inflow, as defined by EPA, exceeding 1,000 gpcd.

The evaluation of the collection system for this Plan did not include review of television inspection and other detailed assessments of the physical condition of the sewers, since current information in this regard was lacking. A cost of $75,000 (including $25,000 in flow meter rental costs has been included in the CIP for this I/I evaluation, but the actual cost may be lower or higher based on the level of City involvement in the assessment. The City has recently procured sewer system television inspection equipment, which can be utilized for this assessment. In addition, the City should physically evaluate the actual degree of surcharging during peak flow conditions. The projects identified to increase capacity are estimated to cost millions of dollars, and some of them may, in fact, be determined to be unnecessary after completion of the sewer system evaluation, and accompanying I/I reduction efforts deemed to be cost-effective.

The North Shore Levee project certainly appears to have benefit to reducing inflow particularly for the INP basin, which is estimated to currently contribute 29 percent of the total inflow to the collection system. South Aberdeen Basin 2 is a next highest contributor with regard to contribution of extraneous flows; inflow within Basin 2 accounts for 83 percent of the total wastewater flow from Basin 2 and 22 percent of the total inflow to the entire collection system. Any future capacity that the City may need to accommodate growth could be achieved by aggressively reducing **inflow** in Basin 2 or elsewhere in South Aberdeen.

There appears to be a lot of ponding of surface water in Basin 2 that appears to be coincident with the "over capacity" pipelines that are recommended for upsizing. It seems likely that the ponding is the driving force responsible for the high rate of inflow into the collection system and the cause for existing pipes to be surcharged to the point of overflow. The City should verify that manhole covers are sealed in the areas subject to ponding of surface water. When water tight manhole covers are installed it is often necessary to install supplemental gas venting systems at manholes to prevent gas binding, and increased potential for foul odor complaint. Without addressing the surface water ponding issue, upsizing pipelines could increase flows to PS2.

COLLECTION SYSTEM IMPROVEMENTS

Collection system improvements were identified and prioritized based on the collection system capacity analysis, the condition assessments of the collection system summarized in Chapter 4 and Appendix K, problematic gravity mains and pump station issues identified by City operational staff, on-going programs intended to reduce infiltration and inflow, and projects previously scheduled by the City.

The costs presented in this chapter are in 2020 dollars and have been prepared for guidance in project evaluation from the information available at the time of preparation. The final costs for projects will depend on the actual labor and material costs, actual site conditions, competitive market conditions, final project scope, final project schedule, and other variable factors. As a result, the final project costs will vary from the costs presented below. Because of these factors, funding needs must be carefully reviewed prior to making specific financial decisions or establishing final budgets.

Each project cost estimate includes an 8.93 percent sales tax, a 30 percent construction contingency, 13 percent for engineering services, 12 percent for construction administration and 5 percent for legal, permitting and project administration.

Pump Station Improvements

Based on the capacity analysis conducted previously in this chapter, considerations for pump station capacity improvements included:

- The prioritization of pump stations for capacity improvements noted in Tables 6-5 and 6-6. Prioritization is dependent on the frequency of the pump station having all pumps in service during the three largest events between year 2013 and 2018, and the projected flow to the pump station capacity versus current capacity.

- If a pump station has insufficient capacity, upgrades will be recommended to provide sufficient capacity in the station upgrade to pump the peak hour design flow (based on the largest flow between year 2013 and 2018), with the largest unit out of service (firm capacity).

- If a pump station is upgraded, the discharge piping and force mains were evaluated for the need to upgrade to match the full capacity of the pump stations.

As described in Chapter 4, many of the pump station facilities are approaching the end of their useful life and/or require upgrades in the near future. Common deficiencies observed for all collection system pump stations are:

1. Lack of security to the facility access.

2. Space not NFPA 820 compliant.

3. Metal corrosion of the structures.

In addition, the electrical and power equipment of Pump Stations 2, 4, 5, 7, 9, and 13 are near the end of their useful life and need to be replaced soon.

Based on the analysis of the remaining pump stations and in discussions with City staff, it is recommended that the City prioritize Pump Stations: 2, 4, 5, 7, 8 and 9 and 13 due to capacity insufficiency issues. Additional consideration of design flows and total system head should be provided in predesign evaluations conducted before implementation

Pump Station Capital Improvement Projects

The following pump station projects are designed to increase capacity, improve operations, or rehabilitate existing pump stations.

<u>CIP CS-1 – Bypass connections and Miscellaneous Piping Improvements for PS 2, 4, 6 and 7</u>

Project Details: This project will install new piping connections and miscellaneous piping to the force mains near the stations, to allow bypass of the pumps at the stations during power outages or pump failures. The bypass piping would include connection tees with plug valves and flanged camlocks that the bypass pump(s) would connect to. It is anticipated that the bypass pump suction will be directly placed into the pump wet well during the bypass events.

Estimated Project Cost: $201,000

<u>CIP CS-2 – Pump Station 5 Upgrades</u>

Project Details: Pump Station 5 collects flow through an area just west of the Wishkah River and conveys it to the gravity sewer through a short force main. The existing mechanical and electrical components of this station are in very poor condition. The City has purchased new, higher capacity pumps for the station; however, the force main diameter is too small, causing the backup pump to come on in violation of Department of Ecology criteria. The proposed upgrades include new electrical and instrumentation and controls (I&C), new ladder and hatch, new piping and valves, new force main, and rehabilitation of the wet well. A bypass connection similar to that as in CIP CS-1 will also be constructed.

Estimated Project Cost: $630,000

CIP CS-3 Fry Creek Pump Stations

The City of Aberdeen alongside the City of Hoquiam seeks to restore Fry Creek, with the goal of reducing flooding, restoring habitat, and improving public open space. In accordance with the goal of improving public open space, the City of Aberdeen is planning a project to remove the existing above-ground sections of the sewer lines crossing Fry Creek at three locations between Simpson Ave. and the Grocery Outlet Bargain Market, and bore an alternate sewer line below the proposed creek bed. The project will include the addition of three small pump stations (essentially pumps installed in new manholes) serving 13, 11 and 3 homes respectively. The City has estimated a cost of $200,000 for this project.

Estimated Project Cost: $200,000

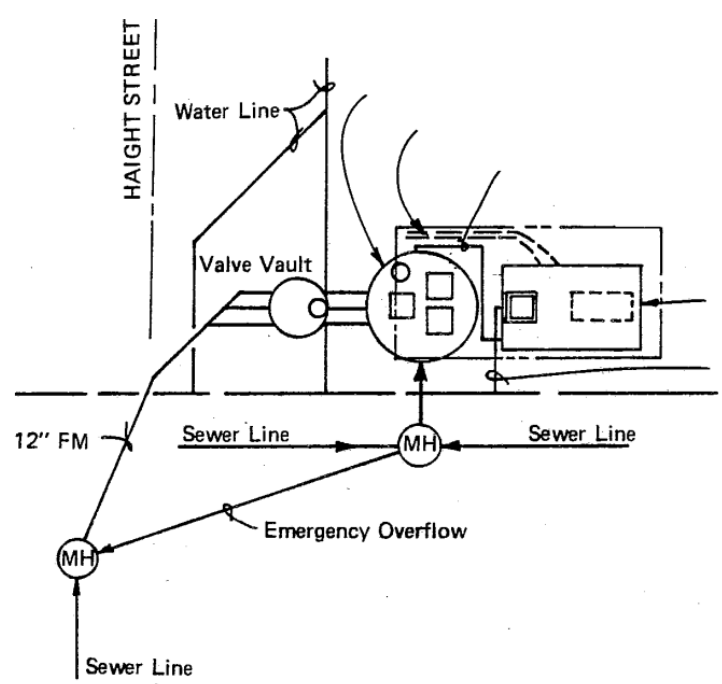
CIP CS-4 – Pump Station 6 Upgrades

Project Details: Like Pump Station 4, Pump Station 6 collects flow on the east side of the Wishkah River (including flow from Pump Station 14) and conveys it to the west side. The upgrades include new electrical and I&C, new pumps on rails (equivalent to existing 650-gpm capacity), new ladder and hatch, new piping and valves, rehabilitation of the wet well, and replacing the existing 1,700 ft AC force main with HDPE. (Note: This project may be completed in phases, as the existing Hydronix pumps plug frequently and are scheduled to be replaced in 2020.) Installation of bypass pumping connections, an immediate major priority, has been included in CIP CS-1.

Estimated Project Cost: $1,306,000

CIP CS-5 – Pump Station 13 Upgrades

Project Details: Pump Station 13 collects flow from a large area in west Aberdeen. This project would include a new above grade control building to replace the existing underground one. The upgrades include new electrical and I&C, new ladder and hatch, new piping and valves, new trailer mounted generator and ATS, and rehabilitation of the wet well. A bypass connection similar to CIP 1 will be constructed.



The City recently replaced one of the (frequently ragging) Hydromatic pumps with a Vaughan chopper pump, and will purchase a larger Vaughan pump (1,600 gpm) this year. As part of this project, a third Vaughan pump (1,350 gpm, 15 hp) will be purchased and both of the new pumps will be installed.

All three of the force mains discharge into a common 12-inch diameter force main that is less than 100 feet long which discharges into the gravity conveyance that is over capacity. Because of this, this project is recommended to be completed at the same time as upsizing of the downstream sewer system.

Estimated Project Cost: $2,425,000 (w/replacement of 1,300 feet of downstream piping)

<u>CIP CS-6 – Pump Station 10 Upgrades</u>

Project Details: Pump Stations 10 and 11 are in the far north part of Aberdeen. Pump Station 11 pumps its flow to Pump Station 10. The pumps from PS 5 were recently relocated to PS 10; however, the remaining mechanical and electrical components of the station need to be upgraded. The upgrades also include new I&C, new ladder and hatch, new piping and valves, and rehabilitation of the wet well. A bypass connection same as in CIP 1 will be constructed.

Estimated Project Cost: $580,000

<u>CIP CS-7 – Pump Station 7 Upgrades</u>

Project Details: Pump Station 7 is located west of the Wishkah River and receives flow from Pump Stations 10, 11, 12, 15 and 16. This project would upgrade the existing 750 gpm pumps to two 1,200 gpm pumps and add a new generator and ATS. The upgrades also include new electrical and I&C, new ladder and hatch, new piping and valves, rehabilitation of the wet well, and replacing the existing 900 ft AC force main with a new HDPE line. (Installation of bypass pumping connections, an immediate major priority, has been included in CIP CS-1.)

Estimated Project Cost: $1,589,000

<u>CIP CS-8 – Pump Station 4 Upgrades</u>

Project Details: This pump station serves the landfill (which recently requested an increase in authorized discharge) and much of East Aberdeen, and pumps flow under the Wishkah River. This project would include upgrading the existing two 600-gpm pumps to 1,000 gpm each. The upgrades also include new electrical and I&C, new ladder and hatch, new piping and valves, rehabilitation of the wet well, and replacing the existing 700 ft of AC force main crossing the river with an HDPE pipeline, extended to a point downstream in the gravity system with more capacity. (Installation of bypass pumping connections, an immediate major priority, has been included in CIP CS-1. In addition, a new Vaughan chopper pump, 15 hp, 650 gpm, will replace one of the existing pumps as an interim measure in 2021.)

Estimated Project Cost: $1,087,000

CIP CS-9 – Pump Station 8 Upgrades

Project Details: Pump Station 8 collects flow in the central portion of South Aberdeen. Per discussion with the City, due to the age and condition of the pump station, it is recommended that a replacement pump station be constructed. (Note: As described earlier, it is recommended that an I/I Study be completed in South Aberdeen prior to completion of this project.) A new Vaughan chopper pump (15 hp, 650 gpm) will replace one of the existing pumps as an interim measure in 2021

Estimated Project Cost: $1,362,000

CIP CS-10 – Pump Station 2 Upgrades

Project Details: This pump station serves all of South Aberdeen and Cosmopolis, receiving flow from Pump Stations 3, 8 and 9, and pumps flow under the Chehalis Estuary through a combined force main with SCCC flow. As currently conceived, this project would include installation of one new 3,650-gpm submersible pump as the third pump in the existing station to increase the capacity. However, as described earlier, it is recommended that the City conduct an I/I evaluation in South Aberdeen prior to design of the upgrade to this pump station.

The upgrades also include new electrical and I&C, new ladder and hatch, new piping and valves, and rehabilitation of the wet well. (Installation of bypass pumping connections, an immediate major priority, has been included in CIP CS-1.)

Estimated Project Cost: $1,081,000

CIP CS-11 – Pump Station 9 Upgrades

Project Details: Pump Station 9 collects flow in the west portion of South Aberdeen. This project includes replacing the two 600-gpm pumps with 1,000 gpm pumps. (Note: As described earlier, it is recommended that an I/I Study be completed in South Aberdeen prior to completion of this project. This is a project for which costs could potentially be reduced significantly if I/I can be reduced.) The upgrades also include new electrical and I&C, new ladder and hatch, new piping and valves, and rehabilitation of the wet well, replacement of the force main and installation of a new bypass connection. A new Vaughan chopper pump (15 hp, 650 gpm) will replace one of the existing pumps as an interim measure in 2021

Estimated Project Cost: $865,000

<u>CIP CS-12 – Pump Station 11 Upgrades</u>

Project Details: The upgrades to Pump Station 11, the farthest north pump station in the City, include replacement of pumps with identical capacity new pumps, new electrical and I&C, new ladder and hatch, new piping and valves, and rehabilitation of the wet well. A bypass connection as described for CIP 1 will be constructed.

Estimated Project Cost: $606,000

<u>CP CS- 13 – Pump Station 12 Upgrades</u>

*Project Details***:** Due to the age and condition of the pump station, the pump station should either be replaced with a new pump station or replaced with a new gravity line. The cost estimate to replace the station is approximately $1.3 million. Based on a preliminary analysis, it appears that it is feasible, and about 50 percent cheaper, to connect the existing wet well piping to the manhole BEL36 about 1,000 feet to the south. However, additional investigation should be completed, including environmental /cultural resource review, and extensive clearing and potential procurement of easements will likely be required. Figure 6-16b shows an aerial photo of the area, and plan and profile view.

Estimated Project Cost: $1,281,000 (if PS 12 replaced), or
$600,000 (if gravity line constructed)

<u>CIP CS-14 – SCCC Facilities Upgrades</u>

Project Details: This project would include wetwell HVAC repair, storage tank steel bridge and platform painting, storage tank concrete coating, storage tank rebar repair, settling surface restoration, pressure reducing valve, electrical code compliance, odor control (Bioxide) chemical system rehabilitation, generator battery box repair, settling fuel tank piping rehabilitation, and new VFDs for the pumps. In addition, eight of the air relief valves for the force main need to be replaced.

Estimated Project Cost: $1,015,000

<u>CIP CS-15 – Pump Station 3 Upgrades</u>

Project Details: Pump Station 3 collects wastewater on the east side of South Aberdeen. The upgrades include new pumps, new electrical and I&C, new ladder and hatch, new piping and valves, and rehabilitation of the wet well. A bypass connection as described for CIP 1 will be constructed.

Estimated Project Cost: $595,000

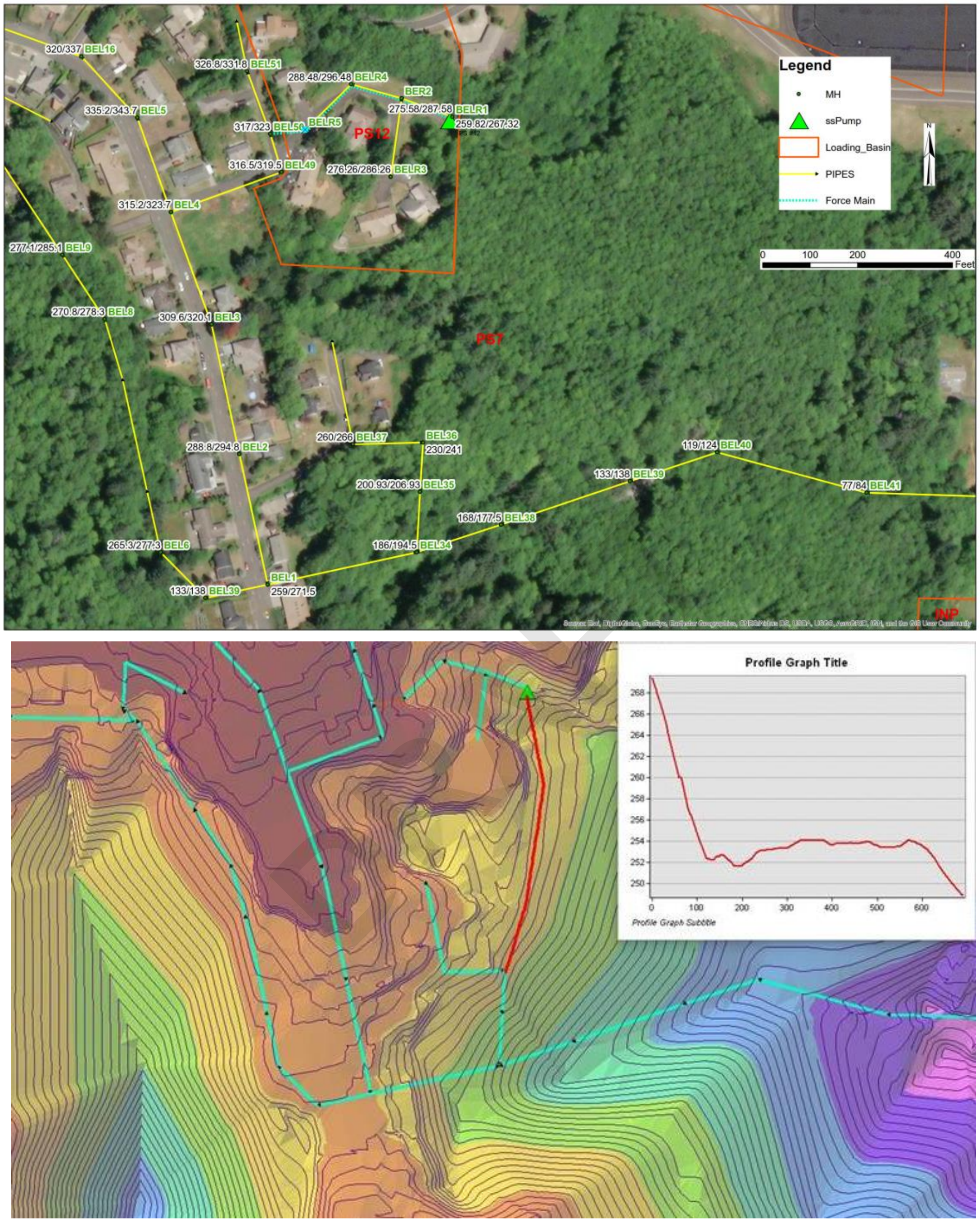


FIGURE 6-16b

Potential Gravity Line Replacement for PS 12

Table 6-7 presents a summary of recommended capital improvements for the pump stations. The costs of any recommended force main replacements, and the downstream sewer replacement specifically for PS 13, are included in this table; additional downstream gravity pipe upsizing is summarized in Table 6-9.

TABLE 6-7

Cost Summary of Pump Station Upgrades

Pump Station	Components	Capital Cost
PS 2, 4, 6 and 7	Bypass connections to piping	$201,000
PS 2	Add pump, replace all mechanical, electrical and I&C; Rehabilitate wet well concrete surface.	$1,081,000
PS 3	Replace pumps, all mechanical, electrical and I&C; Rehabilitate wet well concrete surface; Add bypass piping connection.	$595,000
PS 4	Upsize pumps to 1,000 gpm, replace mechanical, electrical and I&C; Rehabilitate wet well concrete surface; Replace discharge force main.	$1,087,000(1)
PS 5	Replace force main, all mechanical, electrical and I&C; Rehabilitate wet well concrete surface; Add bypass piping connection.	$676,000
PS 6	Replace pumps, all mechanical, electrical and I&C; Rehab wet well concrete surface.	$1,306,000
PS 7	Upsize pumps to 1,200 gpm, replace mechanical, electrical and I&C; Install new generator; Rehabilitate wet well concrete surface; Replace discharge force main.	$1,589,000
PS 8	Replace entire PS.	$1,362,000
PS 9	Upsize pumps to 1,000 gpm. Replace mechanical, electrical and I&C; Install new generator; Rehab wet well concrete surface; Replace discharge force main; Add bypass piping connection.	$865,000
PS 10	Replace mechanical, electrical and I&C; Rehabilitate wet well concrete surface; Add bypass piping connection.	$580,000
PS 11	Replace pumps, all mechanical, electrical and I&C; Rehab wet well concrete surface; Add bypass piping connection.	$606,000
PS 12	Replace entire PS.	$1,281,000(2)
PS 13	Construct new above grade control room; Replace all mechanical, electrical and I&C; Install new generator; Rehab wet well concrete surface; Add bypass piping connection; Upsize downstream piping.	$2,425,000
SCCC	Structural, mechanical and electrical rehabilitation.	$1,015,000
Fry Creek (New)	Three new small submersible pump stations.	$200,000 (3)

(1) Cost does not include the cost of increasing the capacity to accommodate 2038 Central Park flows. That cost is addressed in the Expanded Regional Conveyance Evaluation later in this chapter.

(2) Pump Station upgrade cost is presented. However, replacing station with gravity line may be feasible and save 50 percent of the cost. Additional evaluation of feasibility recommended.

(3) Preliminary estimate provided by City.

Pipeline Capacity Improvements

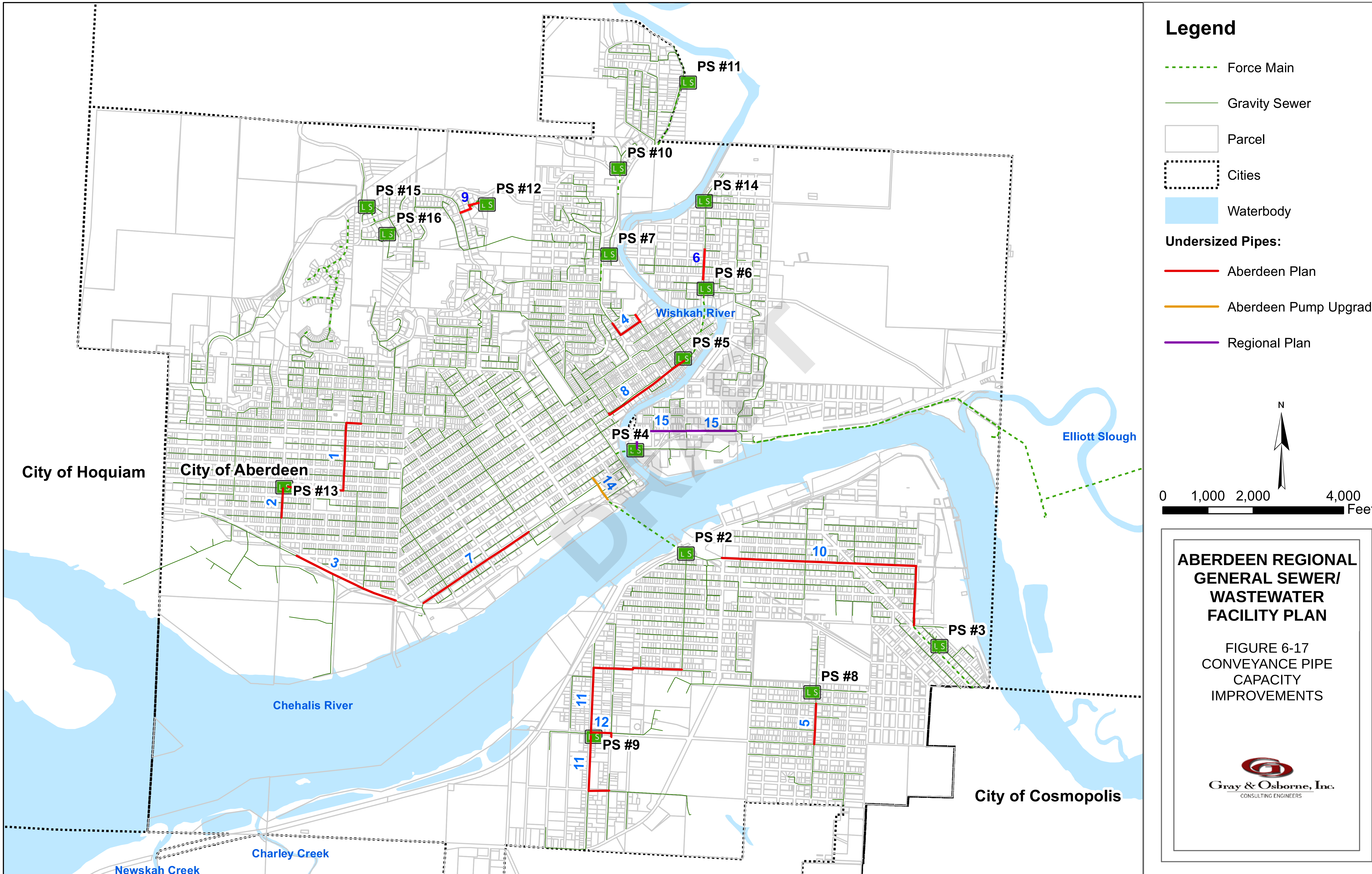
A list of potential improvements to increase pipe capacity was assembled to address deficiencies identified with the hydraulic modeling discussed earlier in the chapter. The potential improvements, summarized in Table 6-8, would increase the capacity of the pipes in order to accommodate 2038 flows.

The selection criteria for the improvements included:

- If the pipe capacity is less than the full new capacity of the upstream pump station that is upsized (PS 2, 7, and 9), the pipe capacity is insufficient.
- If the pipe capacity is less than the tributary basin peak flow, the pipe capacity is insufficient.
- If the pipe is causing surcharging, the pipe capacity is insufficient.
- If the pipe capacity is insufficient, it is recommended that the pipe is upsized to accommodate 120% the peak flow condition without surcharge.

Costs have not been included in Table 6-8, but are included in Appendix L. Prior to completing these projects, it is recommended that the City complete a more detailed sewer system I/I Evaluation to identify any "low hanging fruit" of infiltration and inflow, as described previously, and confirm system bottlenecks.

Figure 6-17 identifies the pipe sections needing capacity improvements, under both Aberdeen and Existing Partners and Expanded Regional scenarios. It should be emphasized that the model was based on available limited GIS information and uncertain estimates and projections; thus, it is recommended that supplemental predesign analysis consisting of field investigation and analysis be conducted to validate the need for implementation of each project identified in this plan prior to initiating design and construction.



ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 6-17
CONVEYANCE PIPE CAPACITY IMPROVEMENTS

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TABLE 6-8

Potential Piping Capacity Upgrades

Pipe Section	Length (lf)	Improvements
1	3,100	Increase pipe to 24-inch
2	700	Increase pipe to 27-inch
3	2,400	Increase pipe to 27-inch
4	800	Increase pipe to 18-inch
5	900	Increase pipe to 24-inch
6	800	Increase pipe to 18-inch
7	2,200	Increase pipe to 54-inch
8	2,000	Increase pipe to 36-inch
9	300	Increase pipe to 12-inch
10	5,700	Increase pipe to 24-inch
11	5,000	Increase pipe to 24-inch
12	700	Increase pipe to 12-inch
13	500	Increase pipe to 15-inch
14	400	Increase pipe to 36-inch

(1) In addition to these upgrades, if the Expanded Regional Alternative is implemented, Pipe Section 15 would need to be increased from 12-inch to 15-inch to accommodate Central Park 2038 flows as discussed later in this chapter.

Pipeline Condition Improvements

As noted above, the City has not been actively performing CCTV assessment of existing pipes and does not have reliable information with regard to current condition in many areas. This results in a significant gap in data as the condition rating should be the backbone of a renewal/replacement program. The current pipe scoring and prioritization are primarily based on pipe material and I/I condition.

As indicated in the Condition Assessment in Appendix D, portions of the City's collection system piping are asbestos-concrete (AC) or ductile iron pipes which were constructed in the 1950s or earlier. This pipe, along with the concrete pipe, is approaching or has exceeded its expected service life span. These aging pipes may be deteriorating and need to be replaced. Recognizing that I/I is more excessive in certain basins, potential piping upgrades were identified and prioritized based on the severity of I/I – from low-lying areas, such as Basin INP and Basin 2 identified in the I/I analysis earlier in the chapter with the worst I/I to Basin 15 that has the least I/I as determined by the analysis.

Figure 6-18 illustrates the location of aging pipe that is recommended for additional evaluation, and, if confirmed to be in deteriorated condition, replacement.

Table 6-9 summarizes the aging pipe replacement projects by basin, which are prioritized based on I/I in the basin. These improvements are not capacity-related, and the implementation should be based on further condition assessment including field investigation. Similar to the improvements to address pipeline capacity deficiencies, costs are not provided in this table, but are in the millions of dollars, and are included in Appendix L.

TABLE 6-9

Summary of Potential Piping Condition Upgrades

Basin	Length (lf)	Priority
INP	30,200	1
PS2	23,200	2
PS13	10,800	3
PS9	5,000	4
PS7	21,700	6
PS6	2,500	7
PS10	2,300	8
PS4	400	9
PS5	1,000	10
PS16	1,400	11
PS3	1,200	12
PS11	100	13
PS15	1,400	14

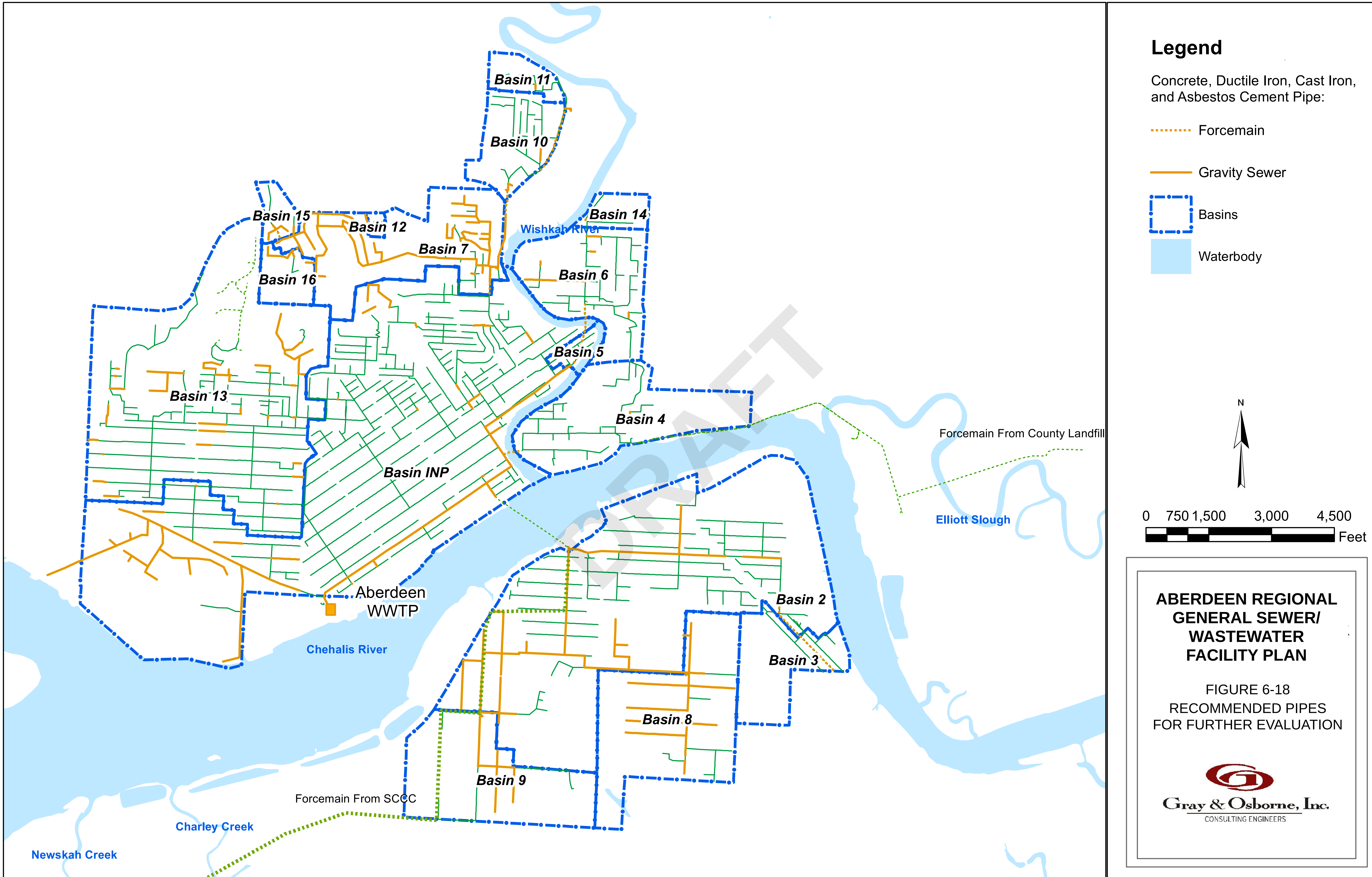
EXPANDED REGIONAL CONVEYANCE EVALUATION

Additional conveyance upgrades will be necessary if the Aberdeen WWTP serves the Expanded Regional Partners, Hoquiam and/or Central Park.

CENTRAL PARK

As noted earlier in this chapter, Central Park flows are presumed to be conveyed to the Aberdeen WWTP through the force main that serves the county landfill east of the city limits. Preliminary analysis indicates there is sufficient capacity in this line. This force main discharges to the gravity sewer upstream of Pump Station 4, which conveys wastewater under the Wishkah River to the State Street gravity interceptor, which conveys the wastewater to the WWTP.

Assuming Central Park is sewered and connected, peak flow from Central Park is projected to be 1.05 mgd by 2038. The existing interceptors were evaluated; segments identified with insufficient capacity are shown in Figure 6-19. PS 4 would require an



ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 6-18
RECOMMENDED PIPES FOR FURTHER EVALUATION

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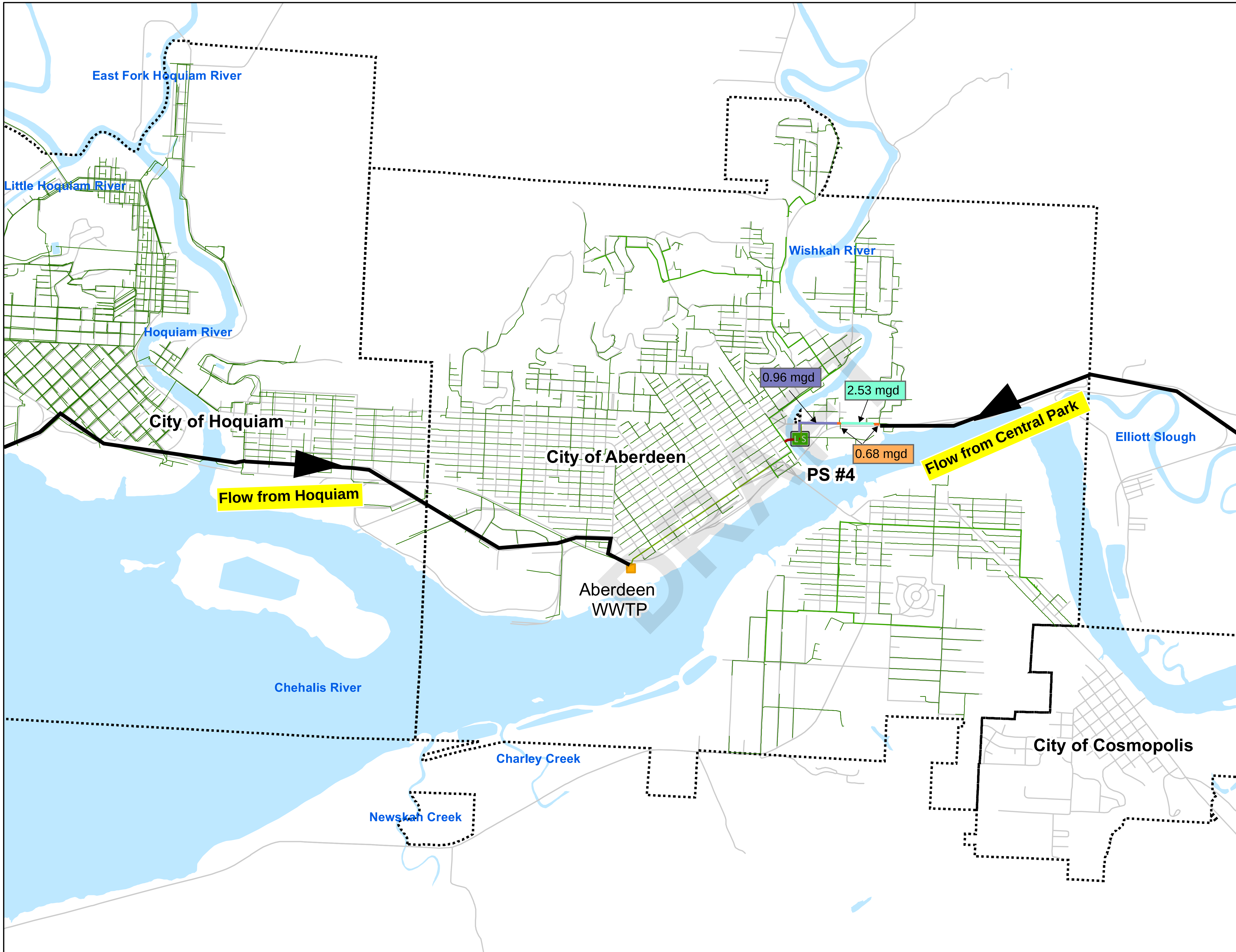

ABERDEEN REGIONAL GENERAL SEWER/ WASTEWATER FACILITY PLAN

FIGURE 6-19 CONVEYANCE PIPE CAPACITIES – EXPANDED REGIONAL ALTERNATIVE

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upgrade to a firm capacity of 1,700 gpm for each of the two pumps to accommodate flow from Central Park. Table 6-10 summarizes the additional conveyance costs to accommodate Central Park.

TABLE 6-10

Additional Conveyance Costs to Accommodate Central Park

Project	Description	Capital Cost ($)
Pipe Capacity Increase	Increase 2,200 feet of pipe from 12 inch to 18 inch	$900,000
PS 4 Upgrade	Pumps, all mechanical and electrical, generator, force main	$1,500,000(1)

(1) *Additional* cost to upgrade the pump station beyond that identified in Table 6-7 in order to increase the capacity to accommodate Central Park. Total upgrade cost is $2,600,000.

HOQUIAM

A detailed evaluation of conveyance improvement alternatives to serve Hoquiam is provided in the *Expanded Regional Conveyance Alternatives Evaluation Technical Memorandum (Conveyance Memorandum,* HDR, 2020) in Appendix M. The *Conveyance Memorandum i*s summarized here; for more detailed information, see the appendix.

Due to a lack of available capacity in the Aberdeen pipes, conveying flow from Hoquiam to the Aberdeen WWTP, bypassing the Aberdeen collection system, is recommended. In order to limit the peak day and peak hour flows conveyed to Aberdeen, flows from Hoquiam can be equalized by providing storage at the existing Hoquiam WWTP site or near the K Street Pump Station. To evaluate these alternatives, a model relating precipitation to influent flows to the Hoquiam WWTP was constructed, as described in the *Conveyance Memorandum.*

WWTP influent data from February 2018 to January 2020, recorded at 6-minute intervals, were utilized to calibrate the model. Future projected flows from Hoquiam were evaluated with the model. In addition, additional flows from a potential future industry with a peak hour flow of 2.4 mgd and an average day flow of 2.0 mgd was also evaluated.

Because Hoquiam's sewer system is, like Aberdeen's, heavily influenced by I/I with substantial peaking during rain events, flow equalization is necessary so that sewers and pump stations conveying wastewater to Aberdeen are not prohibitively large, as well as to reduce the size of flow-related treatment processes at the Aberdeen WWTP. It was decided to complete the analysis assuming that flow from Hoquiam to Aberdeen is limited to 6.5 mgd without a potential future 2-mgd industry, and to 8.5 mgd with a potential future 2-mgd industry. The system model predicts flows for the entire sewer system. However, not all basins in the sewer system will be directly routed to the equalization basin. The 28th and Bay, Riverside, and K Street basins (and future buildout basins that would flow to these basins) are assumed to flow directly without equalization to Aberdeen. The remainder of the system could flow to equalization, as necessary. The 2038 required equalization storage was 10.6 million gallons including a 10 percent factor of safety (about the same size with the addition of the 2-mgd industry, but peak flows to Aberdeen would increase to 8.5 mgd). However, additional storage would be required for the buildout condition.

Four conveyance scenarios were developed based around changing two variables.

The variables changed are regarding the location of equalization storage as well as the alignment of the force main to Aberdeen. This includes the following options:

- Option A: Equalization located at the Hoquiam WWTP (Figure 6-20)
- Option B: Equalization located at the K Street Pump Station (Figure 6-21)
- Option 1: Force main alignment along Port Industrial Way
- Option 2: Force main alignment along Pacific Avenue/Division Street

These variables combine into four scenarios: A1, A2, B1, and B2.

For all scenarios, it is assumed that significant modifications are needed at the K Street Pump Station. For all scenarios, a new 18-inch-diameter force main would need to be constructed from the K Street Pump Station to the Aberdeen WWTP. An 18-inch diameter force main is necessary to keep velocities below 8 feet per second when flowing at 8.5 mgd, which is the maximum allowable peak flow to Aberdeen if considering the additional demand for a potential future industry. Option A will require (among other improvements) reconfiguring and lining the lagoon, and a major new pump station with a capacity of 6.5 mgd (or 8.5 mgd with the 2-mgd industry) at the Hoquiam WWTP. Option B would require the construction of a 10.6 million gallon above-ground storage tank.

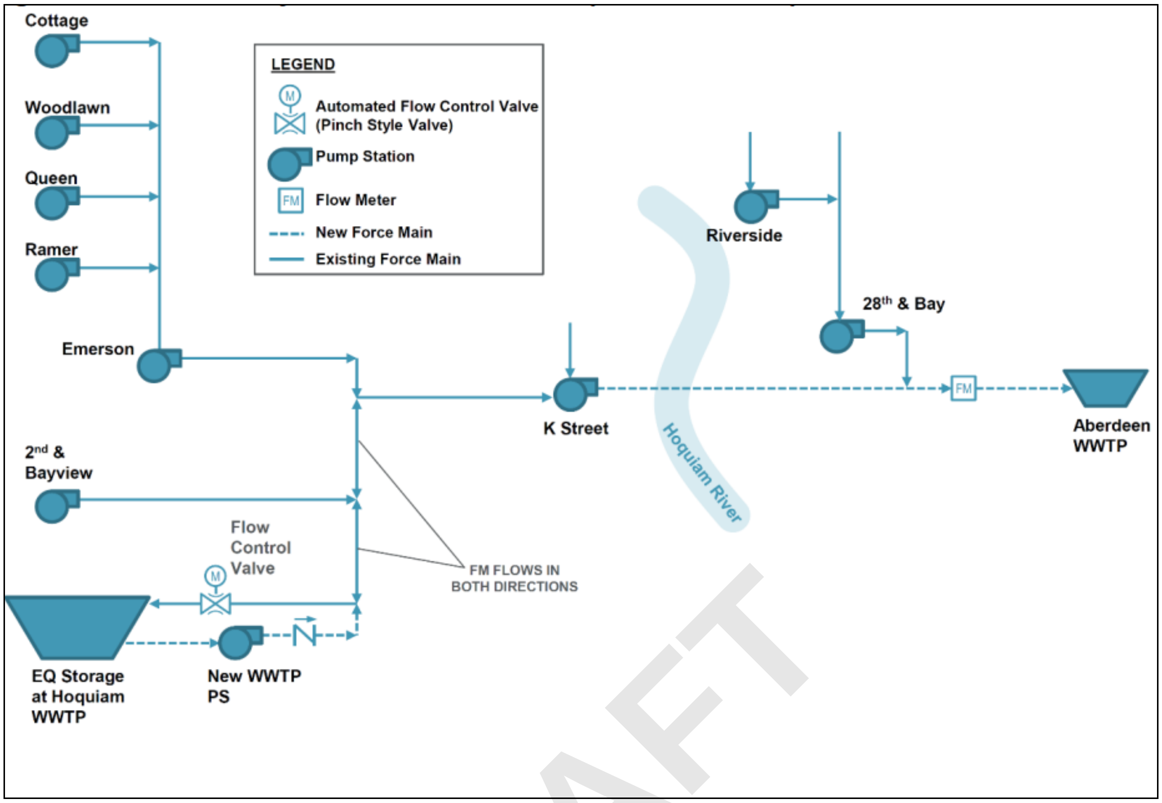

FIGURE 6-20

Schematic of System Modifications for Equalization at Hoquiam WWTP (courtesy of HDR)

Two alignments were considered (Options 1 and 2), shown in Figure 6-21. For both alignments, a new horizontal directional drilled (HDD) crossing of the Hoquiam River is necessary. For the Port Industrial Road Option (Option 1), after crossing the Hoquiam River from the K Street Pump Station, the force main would continue along Port Industrial Road all the way to the Aberdeen WWTP. This is the most direct path to the Aberdeen WWTP but would require four railroad crossings between the Hoquiam River and the Aberdeen WWTP as well as being located along a roadway with higher traffic.

For the Pacific Avenue/Division Street Option (Option 2), after crossing the Hoquiam River from the K Street Pump Station, the force main would continue along Pacific Avenue followed by Division Street to reach the Aberdeen WWTP. Although this route is not as direct as going along Port Industrial Way, this alignment would place the force main on residential streets with lower traffic volumes and would require only one railroad crossing between the Hoquiam River and the Aberdeen WWTP.

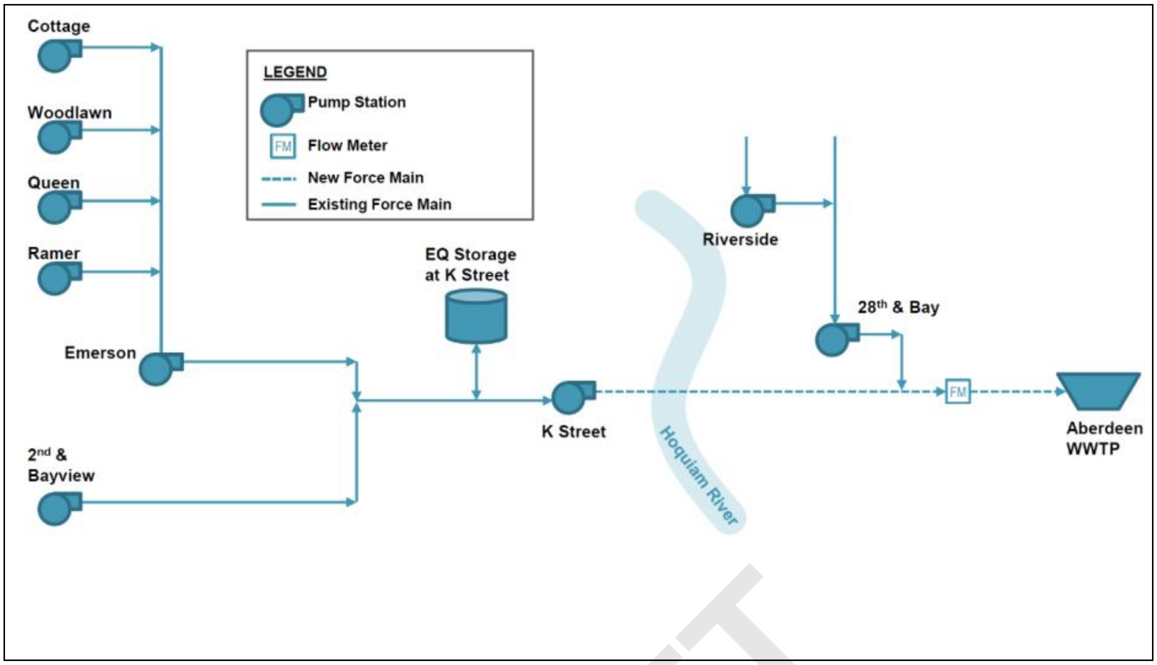

FIGURE 6-21

Schematic of System Modifications for Equalization at K Street Pump Station (courtesy of HDR)

Converting from the current wastewater conveyance pattern in the City to conveyance to an expanded regional facility at Aberdeen will require some major and costly modifications to the Hoquiam conveyance system. Detailed descriptions are provided in the *Conveyance Memorandum*. As shown in Table 6-11, the least expensive capital cost is for Option A1 (Equalization storage at Hoquiam WWTP, force main along Port Industrial Way) at $20.8M.

TABLE 6-11

Conveyance Alternatives and Estimated Capital Costs

Scenario	Description	Capital Cost
A1	Equalization storage at Hoquiam WWTP, force main along Port Industrial Way	$20.8 M
A2	Equalization storage at Hoquiam WWTP, force main along Pacific Avenue and Division Street	$21.4 M
B1	Equalization storage at K Street Pump Station, force main along Port Industrial Way $34.8	$34.8 M
B2	Equalization storage at K Street Pump Station, force main along Pacific Avenue and Division Street	$35.4 M

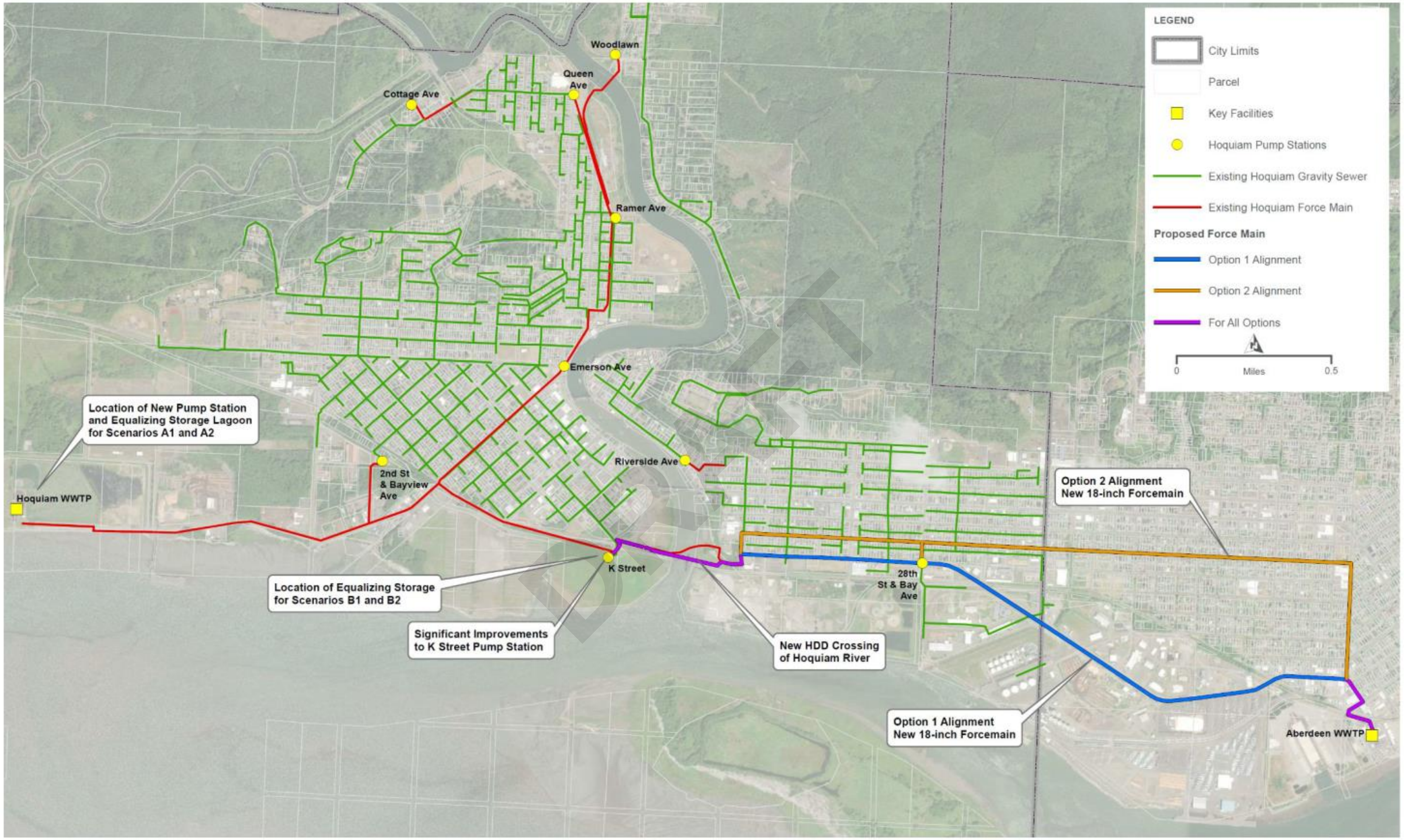

FIGURE 6-22

Possible Force Main Options from Hoquiam to Aberdeen (courtesy of HDR)

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REFERENCES

1. Watershed Science and Engineering, North Shore Levee, Aberdeen and Hoquiam, WA - Hydraulic Analysis and Floodplain Mapping - Memorandum, May 9, 2017.

2. KPFF, North Shore Levee, 100-Year Rainfall Ponding Depth Site Plans, 2018.

3. HDR, Expanded Regional Conveyance Alternatives Evaluation TM, 2020.

CHAPTER 7

WASTEWATER TREATMENT PLANT EVALUATION

This chapter provides an evaluation of the existing Aberdeen WWTP, including:

1. A comparison of WWTP NPDES permit limits and design criteria to projected flows and loadings for both Existing Partners (Aberdeen, Cosmopolis and Stafford Creek Correctional Center) and Expanded Regional Partners (Existing partners plus Hoquiam and Central Park).

2. An update to the WWTP Mixing Zone Analysis and projection of future NPDES Permit Limits.

3. Evaluation of the hydraulic capacity of the WWTP.

4. Evaluation of the process capacity of the WWTP.

This chapter also builds on the evaluation of condition and performance summarized in Chapter 4. Alternatives for WWTP Upgrades, both for Existing Partners and Expanded Regional Partners, are discussed in Chapter 8.

Hydraulic capacity is the ability of each unit of the treatment plant to pass the process flow. Process capacity is each unit's ability to effectively treat the flows passing through it. Some discussion of appropriate sizing is provided in this chapter; however, detailed evaluation of alternatives to provide the necessary capacity and level of treatment is provided in Chapter 8.

ANALYSIS OF WWTP FLOW AND LOADING PROJECTIONS

Chapter 5 presented a detailed analysis of existing, and projections of future, flows and loadings for Aberdeen and existing and potential expanded regional partners. A summary of the projected flows and loadings is provided in Tables 7-1 and 7-2, along with the applicable NPDES Permit limits and/or design criteria.

FLOWS

As shown in Table 7-1, the projected maximum month influent flows to the WWTP for Aberdeen with existing partners do not exceed the rated capacity of the WWTP (9.90 mgd) for the 20-year period. However, projected peak hour flows do exceed design values, requiring an evaluation of the capacity of the plant to treat the higher flows.

As discussed in Chapter 8, the existing plant does not have available hydraulic capacity to accommodate flows from the expanded regional partners without significant expansion and upgrade.

LOADINGS

As noted in Chapter 5, there are concerns that unrepresentative sampling of influent potentially may have caused high bias in some of the historical reported BOD_5 and TSS loadings. (Recommendations to improve the representativeness of sampling are discussed in Chapter 8.) However, based on the analysis of data in Chapter 5, and summarized in Table 7-2, it is concluded that maximum month BOD_5 loadings have exceeded 99 percent of rated design capacity. Therefore, the WWTP does not have significant available capacity to accommodate additional BOD loading without significant expansion and upgrade, and loadings are projected to exceed the rated capacity before planning year 2028.

With regard to TSS loading, the plant is currently operating at 88 percent of the rated design capacity for the maximum month TSS loading and is projected to reach 100 percent of the plants rated design capacity for TSS removal by planning year 2028. The existing plant would need to be expanded and upgraded to accommodate the TSS loadings from the expanded partners.

The addition of current flows and loadings from the Expanded Regional Partners to current flows and loadings from Aberdeen and Existing Partners would result in exceedances of the maximum month TSS and BOD loading limits.

TABLE 7-1

Comparison of NPDES Permit Limits/Design Criteria and Projected Future Flows

Flow Type	Projected Flow Rate (mgd)						
	NPDES Permit Limit/ Design Criteria	Aberdeen and Existing Partners Total(1)	Hoquiam(2)	Central Park(3)	Expanded Regional Total(4)	Potential Additional Industrial Flow	Total
Current							
Total Base	--	1.88	0.76	--	2.64	2.00	4.64
Average Annual	--	3.87	1.28	--	5.15	2.00	7.15
Maximum Month	9.90	6.83	2.98	--	9.81	2.00	**11.81**
Peak Day	--	20.60	6.50 (5)	--	27.10	2.20	29.30
Peak Hour	20.4(8)	**22.99**	6.50 (6)	--	**29.49**	2.40	**31.89**
2028							
Total Base	--	2.16	0.92	0.15	3.23	2.00	5.23
Average Annual	--	4.15	1.49	0.17	5.81	2.00	7.81
Maximum Month	9.90	7.11	3.35	0.26	**10.72**	2.00	**12.72**
Peak Day	--	18.73	6.50 (5)	0.43	25.66	2.20	27.86
Peak Hour	20.4(8)	**21.34**	6.50 (6)	0.68	**28.52**	2.40	**30.92**
2038							
Total Base	--	2.47	1.10	0.24	3.81	2.00	5.81
Average Annual	--	4.46	1.72	0.26	6.44	2.00	8.44
Maximum Month	9.90	7.42	3.73	0.39	**11.54**	2.00	**13.54**
Peak Day	--	19.05	6.50 (5)	0.66	26.21	2.20	28.21
Peak Hour	20.4(8)	**21.97 (9)**	6.50 (6)	1.05	**29.52**	2.40	**31.92**

(1) Aberdeen total flow including flow from Cosmopolis and SCCC.

(2) Hoquiam flow projections are interpolated from *2013 Hoquiam Wastewater Facility Plan*.

(3) Central Park base flow is calculated based on population projections and a typical wastewater flow rate of 100 gpcd. Annual average, maximum month, peak day, and peak hour flows are calculated using typical peaking factors. For this analysis, it is assumed that portions of Central Park will be connected to the City's collection system by 2028.

(4) Sum of Aberdeen and Existing Partners, Hoquiam and Central Park.

(5) Flows projected to be generated in the Hoquiam system in 2028 are 13.17 mgd peak day and 13.82 mgd peak hour. Per the analysis from HDR, these flows will be equalized to limit the discharge to the city of Aberdeen WWTP to 6.5 mgd by Hoquiam providing an equalization basin.

(6) Flows projected to be generated in the Hoquiam system in 2038 are 14.35 mgd peak day and 15.06 mgd peak hour. Per the analysis from HDR, these flows will be equalized in an equalization basin constructed by the city of Hoquiam to limit the transfer of flow from Hoquiam to the City of Aberdeen to 6.5 mgd.

(7) **Bold** values exceed applicable criteria.

(8) Not an NPDES Permit limit, but the effective peak hour design criteria as discussed later in this chapter.

(9) Projected 20-year peak hour flow is less than existing peak hour flow of 22.99 mgd due to the impact of $75 million in flood control improvements, as discussed in Chapter 5 and 6. Since the 22.99 mgd current flow is higher than the 21.97 mgd projected flow, the 22.99, rounded to 23 mgd is called the "design peak flow."

TABLE 7-2

Comparison of NPDES Permit Limits/Design Criteria and Projected Future Loadings (Including Hauled Septage Loading)

Loading (lb/d)	NPDES Permit Limit/Design Criteria	Aberdeen and Existing Partners Total 1)	Hoquiam	Central Park(2)	Expanded Regional Total
Current					
Annual Average BOD_5	--	6,210	1,906	--	8,116
Annual Average TSS	--	6,394	1,854	--	8,248
Maximum Month BOD_5	7,400	7,394	2,712	--	**10,106**
Maximum Month TSS	8,900	7,853	2,806	--	**10,659**
Maximum Month TKN	1,768	793	409	--	1,202
2028					
Annual Average BOD_5	--	7,095	2,325	368	9,788
Annual Average TSS	--	7,279	2,261	398	9,937
Maximum Month BOD_5	7,400	**8,412**	3,308	437	**12,157**
Maximum Month TSS	8,900	8,871	3,423	485	**12,778**
Maximum Month TKN	1,768	952	500	48	1,499
2038					
Annual Average BOD_5	--	8,102	2,785	651	11,537
Annual Average TSS	--	8,285	2,707	703	11,695
Maximum Month BOD_5	7,400	**9,569**	3,963	769	**14,301**
Maximum Month TSS	8,900	**10,028**	4,100	851	**14,979**
Maximum Month TKN	1,768	1,084	599	85	1,768

(1) Aberdeen total loading including loading from Cosmopolis, SCCC, and hauled septage.
(2) Central Park loadings are calculated based on population projections and typical wastewater loading 0.25 BOD ppcd and 0.27 TSS ppcd. Maximum month and peak day loading are calculated based on the same peaking factor of Aberdeen. For this analysis, it is assumed that portions of Central Park will be connected to the City's collection system by 2028
(3) **Bold** values exceed applicable criteria.

There is a requirement in the City's NPDES permit that when actual monthly average influent flow or loading to the WWTP exceed 85 percent of design criteria for 3 consecutive months, or if the City has *projected* increases in wastewater flow or loading that would cause exceedance of design capacity within 5 years, the City must submit a "Plan and schedule to Maintain Adequate Capacity" (PMAC) to Ecology. The 85 percent and 5-year requirements are needed to provide sufficient time for municipalities to plan, design and construct sufficient capacity, if it is needed. Based on projections, the City will exceed its BOD loading limit prior to 2028, even without additional regional partners. This Plan constitutes the PMAC as required by the NPDES Permit.

MIXING ZONE ANALYSIS

It is the policy of the State of Washington to maintain existing beneficial uses of surface water by preventing degradation of existing water quality. However, certain allowances are made by Ecology for discharging treated wastewater into a surface water that enable a temporary or mitigated degradation to occur. These allowances are made by establishing mixing zones and determining the assimilative capacity of the receiving water. WAC 173-201A-100 has provisions for mixing zones around the point of discharge for permitted discharges. Water quality standards must be met at the boundary of the mixing zone, but may be exceeded inside the mixing zone. Before a mixing zone is granted by Ecology, the discharger is required to apply all known, available and reasonable technology (AKART) prior to discharge. Mixing zones may be granted for a constituent because the water quality criteria are too stringent for traditional wastewater treatment technology to meet the criteria on an end-of-pipe basis. Under WAC 173-201A-060, State Water Quality Standards, Ecology is authorized to condition NPDES permits when there is a "reasonable potential" to exceed water quality standards at the mixing zone boundary.

In a mixing zone study, dilution is modeled at mixing zone boundaries, assimilative capacity is determined, and potential NPDES permit limits are projected. A copy of the updated mixing zone study completed for the projected Existing Partner and Expanded Regional Partner flows is provided in Appendix N. Based on the Mixing Zone Study, it is expected that no new permit limits (i.e., for pollutant concentrations) will be necessary for either the Existing Partner or Expanded Regional WWTP, ***assuming the WWTP continues to nitrify***. For the Existing Regional WWTP, effluent ammonia needs to remain below 40 mg/L to avoid triggering a permit limit for ammonia. For an Expanded Regional WWTP, effluent ammonia would need to remain below 33 mg/L to avoid triggering a permit limit for ammonia. However, it is recommended that the WWTP nitrify to achieve significantly lower levels than these triggers. The City's NPDES permit requires the City to "operate the facility to minimize ammonia in the discharge" and the MLE process is capable of removing ammonia and nitrogen to low levels.

Consistent nitrification will provide for a more reliable, stable process due to the higher SRT. In addition, ensuring low effluent ammonia levels will reduce the chance that the effluent would show a reasonable potential for ammonia in the future, which would potentially result in imposition of more stringent, water-quality based permit limits.

It is expected that new effluent pathogen limits may be imposed in the next NPDES Permit, as Ecology is transitioning from fecal coliform to Enterococci as the primary pathogen indicator of recreational water quality. (Fecal coliform would likely be retained in the new permit for protection of shellfish harvesting areas.) Based on the experience of other facilities, it is anticipated the WWTP can easily meet the Enterococci limits.

HYDRAULIC CAPACITY

A spreadsheet-based mathematical model was developed to evaluate the hydraulic capacity of the Aberdeen WWTP for projected flows from the Existing Regional Partners. (A similar analysis was not conducted for flows from the Expanded Regional Partners, since the regional expansion would require the construction of an effluent pump station, the replacement of some of the major pipelines in the WWTP and the addition of several others.) The analysis starts with establishing the level of the receiving water and then proceeds upstream through the plant. When the hydraulic capacity of conveyance or treatment facilities is exceeded, flows can back up and increase the water level in upstream facilities, impacting their performance and potentially causing overflows.

Four different receiving water scenarios were evaluated with the spreadsheet model:

- The annual extreme high tide (AEHT) condition. The AEHT was determined to be approximately 12.3 feet at mean lower low water (MLLW) datum, based on the evaluation of the National Oceanic and Atmospheric Administration's predicted tides for Grays Harbor Estuary at Aberdeen from 2017 through 2019.

- The AEHT plus 1 ft of storm surge, assuming the storm is coinciding with the peak high tide. The allowance of 1 foot accounted for local effects of wind, waves and storm water river flows.

- The extreme high 100-year flood (EHF) condition, which is 13.00 ft NAVD 88 according to FEMA National Flood Hazard Map (area number 53027C0904D, effective 02/03/2017). This translates to 14.75 feet MLLW based on the datum conversion between NAVD 88 and the local Aberdeen MLLW.

- The higher high water (MHHW) condition of 10.07 feet MLLW plus 1 foot of storm surge.

Evaluation of these four conditions, which include the coincidence of flood, tide and storm surge, is considered to be conservative; however, the scenarios do not include an explicit allowance for sea level rise due to climate change. Depending on the magnitude of the sea level rise, it may be necessary to add effluent pumping during peak tide/river flow conditions for Aberdeen and Existing Regional Partners. The current estimates for sea level estimates for sea level rise in Aberdeen from the Washington Coastal Hazards Resilience Network suggest a sea level rise of between 1 and 4 inches by 2040, and between 2 and 8 inches by 2060. It is recommended that the City re-evaluate sea level rise every 10 years to assess impacts to system hydraulics during extreme events.

METHODS OF CALCULATION

Hydraulic profiles are a graphic representation of water surface elevation as wastewater flows through each unit process of the treatment plant. The water surface elevation changes from process to process because of frictional losses, changes in the elevation, type, and location of hydraulic control structures, and mechanical energy added to the system by pumping.

Hydraulic profiles for the Aberdeen WWTP were developed based on the equipment, structures, locations, piping configuration and piping sizes shown on the record drawings and supplemental documentation provided by the City of Aberdeen Sewer Department. The profiles presented in this chapter utilized the projected 20-year peak hour flow for Aberdeen and existing regional partners of 21.9 mgd. Additional discussion is provided regarding the hydraulic analysis that includes the both Existing and Expanded Regional Partners with additional industrial flows for a combined flow of 31.92 mgd.

The water surface elevation through the treatment plant was calculated using Bernoulli's equation for conservation of energy. Hydraulic head losses in piping system were determined using Hazen-Williams equation. Losses in open channels were calculated using Manning's equation. Calculations of head conditions at hydraulic control structures such as weirs conformed to University of Michigan formulas documented in *Handbook of Hydraulics* (Brater and King 1976). Calculations of head conditions for critical flow at other submerged and free discharged control structures conform to methodologies set forth by Benefield, et al. (1984). Minor head losses through pipe fittings and valves were calculated using a fitting-specific constant times the velocity head. Detailed data and results for the hydraulic profile are presented in Appendix O.

RESULTS

Under each scenario of receiving water conditions, the following critical hydraulic limitations (summarized in Table 7-3) at the existing facilities were evaluated:

1. The flow rate at which the manhole on the outfall pipeline inside the plant fence line will overflow.

2. The flow rate at which the effluent flow metering Parshall flume starts to become submerged. At this level of submergence, it is possible to measure the flow with the flume; however, to do so requires measurement of the depth of water both upstream and downstream of the throat of the flume ("two-point measurement"). The existing single point measurement will not be able to accurately measure the flow.

3. The flow rate that will cause the Parshall flume to submerge 95 percent, which will make flow measurement inaccurate.

4. The flow rate at which the weir plate of the secondary clarifiers will be overtopped, causing excessive backwater and hydraulically interfering with the flow distribution characteristics in the secondary clarifier.

5. The flow rate at which the scum baffle will overtop in the secondary clarifiers and lose containment of the scum to the effluent.

6. The flow rate that will require full operation of all five influent pumps (including the pump in the influent manhole).

7. The flow rate that will cause the weir plate of the primary clarifiers to be overtopped.

8. The flow rate that will cause the bypass of the headworks step screens to the manual bar screen, which will cause retainage of a large volume of screenings in the liquid stream.to the secondary treatment process.

9. The flow rate that the step screens (2) can accommodate, according to plant operation records.

10. The flow rate that will cause the water surface elevation in the headworks influent box to be within 6 inches of overflowing the wall of the structure, assuming all the influent is processed through the headworks. (However, Pump 4 in the influent pump station and the submersible pump in the influent manhole, Pump 5, are set up so they can bypass the influent to a point downstream of the headworks, as noted in hydraulic limitation no. 11 below.)

11. The limiting flow rate through the headworks under the partial influent bypass condition with Pumps 4 and 5 conveying wastewater to a point downstream of the screens.

Flow through the largest secondary clarifier (Secondary Clarifier 3) was used to develop the hydraulic model because it is assumed that this path has the highest headloss. In addition, it is considered the most vulnerable among the three clarifiers due to the lower level of the weir plate and scum baffle compared to the other two secondary clarifiers.

TABLE 7-3

Hydraulic Capacity Summary

	AEHT	AEHT + 1' Surge	MHHW + 1' Surge	FEMA 100-Yr Flood	
Receiving Water Elevation[1] (ft)	12.3	13.3	11.07	14.7	14.7
	Limiting Flow Rate (mgd)				
Hydraulic Limitation	**Existing Facilities**	**Existing Facilities**	**Existing Facilities**	**Existing Facilities**	**Improved Facilities[3]**
1. Outfall manhole overflow	24.2	22.5	29.8	16.4	23.0
2. Effluent Parshall flume submergence	18.3	14.2	22.3	5.1	14.6
3. Effluent Parshall flume 95 percent submergence	26.5	23.1	29.9	17.5	23.0
4. Secondary 2nd Clarifier weir plate overtopped	24.6	22.3	25.3	n.a. [2]	16.0
5. Secondary Clarifier launder scum baffle overtopped	26.7	22.7	26.7	n.a. [2]	17.6
6. Operation of all five influent pumps (4)	21.9	21.9	21.9	21.9	21.9
7. Primary Clarifier weir plate overtopped	23.3	23.3	23.3	n.a. [2]	23.3
8. HWs Maximum screening capacity before bypass begins around existing mechanical screens	4.2	4.2	4.2	4.2	4.2
9. HWs Step Screen Capacity	13	13	13	13	13
10. HWs channels (3) within 6" of overflowing	19.8	19.8	19.8	19.8	19.8
11. HWs channels (3) within 6" of overflowing when influent partially bypasses the HWs	26.3	26.3	26.3	26.3	26.3

(1) All elevations are City of Aberdeen datum.
(2) Head loss cannot be assessed as effluent floods without improvements.
(3) Limiting flow rate was assessed assuming the downstream hydraulic obstacles will be eliminated by facilities improvements, e.g., (a) elevating the effluent manhole cover from 17 feet to 18.6 feet, so it will accommodate the current peak hour flow of 23 mgd without overflowing. (b) raising the Parshall flume invert elevation from 15.5 feet to 17.0 feet, so the flume will be less than 95 percent submerged under the design peak hour flow of 23 mgd.
(4) Capacity with all pumps running; firm capacity (with 1 pump out of service) is 15.3 mgd.

Annual Extreme High Tide

Under the Annual Extreme High Tide (AEHT) condition, a flow rate greater than 18.3 mgd will cause the submergence of the Parshall flume, which requires converting the existing single point measurement approach to dual point measurement to ensure accurate effluent flow measurement. The principal hydraulic limitation is the influent pump station, which only has a firm capacity of 15.3 mgd. With a 26.3 mgd flow, freeboard in the headworks will be less than the 6-inch required minimum when a portion of influent is bypassed downstream of the headworks. A flow rate greater than 4.2 mgd will cause the flow to start to bypass the headworks step screens to the manual bar screen. Even though this situation is not considered as a hydraulic limitation, it will affect the performance of the headworks and downstream treatment process. The above critical flow rates exist also for the other hydraulic scenarios. Figure 7-1 illustrates the hydraulic profile of the existing treatment facilities at the projected peak hour flow rate of 21.9 mgd under AEHT condition at the receiving water.

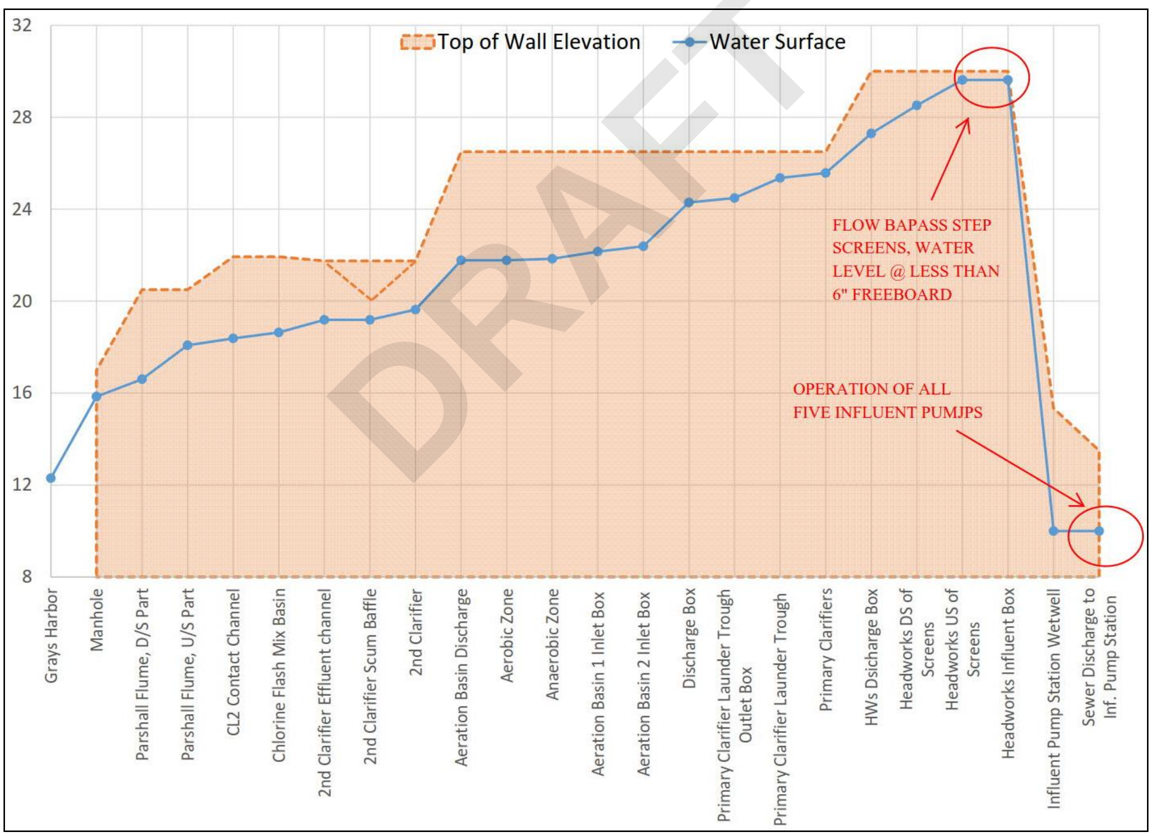

FIGURE 7-1

Hydraulic Profile at 21.9 mgd with Annual Extreme High Tide (12.3 feet) Boundary Condition in Grays Harbor Estuary

Annual Extreme High Tide Plus 1-Foot Storm Surge

Under the condition of AEHT plus 1-foot storm surge, the maximum effluent that can be accurately measured using the existing single point flume measurement is approximately 14.2 mgd. The hydraulic limitation is the influent pump capacity and the corresponding hydraulic profile is shown in Figure 7-2.

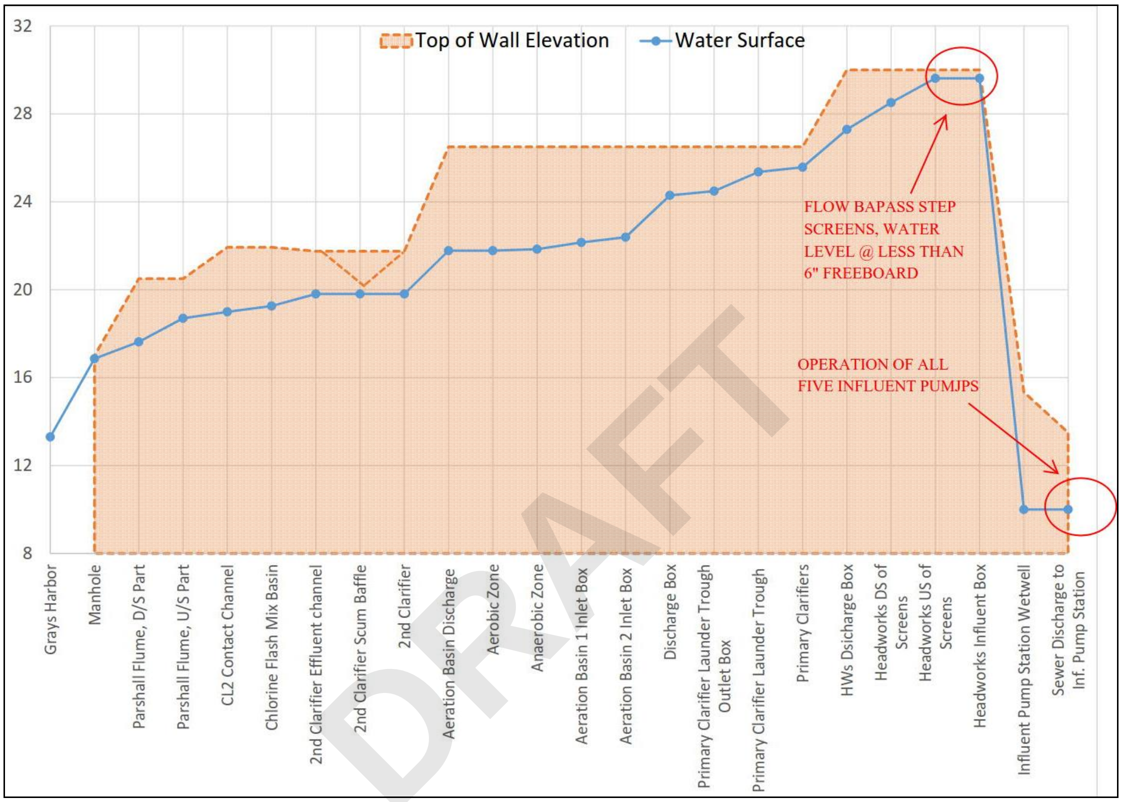

FIGURE 7-2

Hydraulic Profile at 21.9 mgd with Annual Extreme High Tide Plus 1 Foot (13.3 feet) of Storm Surge in Grays Harbor Estuary

Mean Higher High Water Plus 1-Foot Storm Surge

The result of the hydraulic analysis under the condition of Mean Higher High Water (MHHW) tide plus 1 foot of storm surge is illustrated in Figure 7-3. The effluent flow measurement starts losing accuracy at 22.3 mgd. Besides the limitation of the influent pump station, there would be no other hydraulic difficulties until the flow reaches 23.3 mgd, at which the primary clarifier weir will be overtopped.

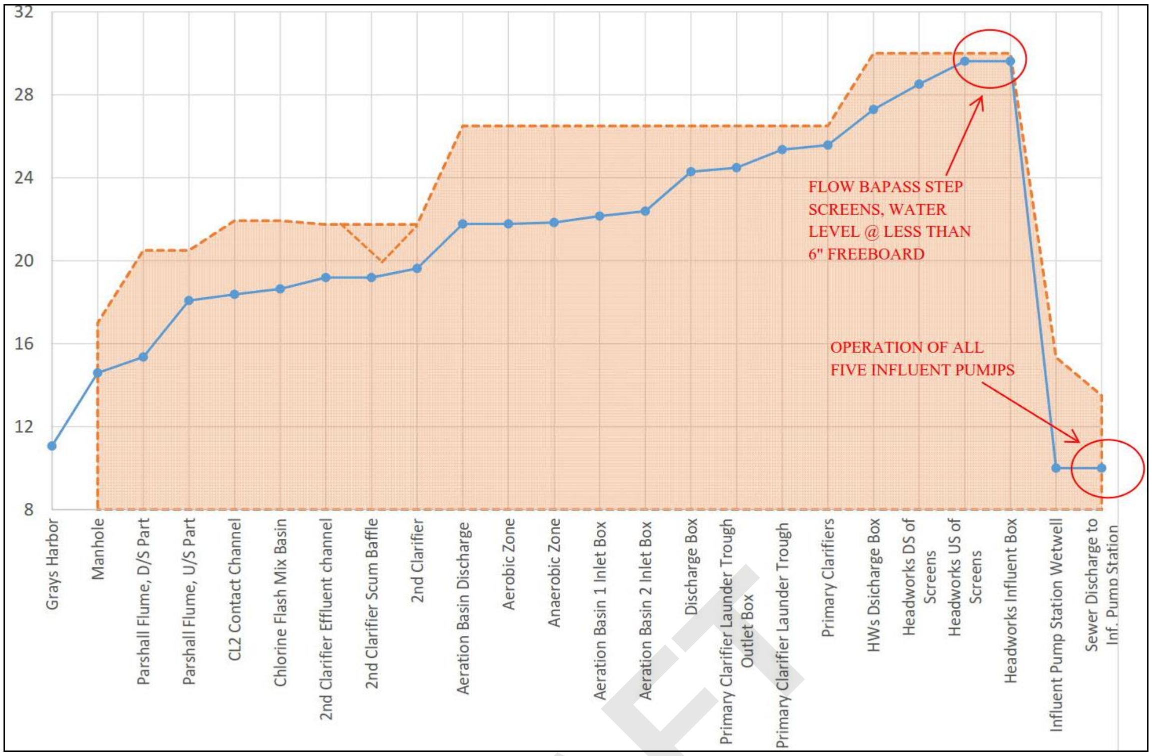

FIGURE 7-3

Hydraulic Profile at 21.9 mgd with Mean Higher High Water Tide Plus 1 Foot (11.07 feet) of Storm Surge in Grays Harbor Estuary

Extreme High Flooding

Under the condition of 100-year extreme high flooding, the Parshall flume becomes 95 percent submerged when the flow exceeds 17.5 mgd. The outfall manhole will overflow when the flow exceeds 16.4 mgd. The backwater will flood the treatment process due to the high tailwater condition.

To be able to assess the flow limitations in the upstream process units, it is assumed the hydraulic deficiencies will be eliminated by facilities improvements, including (a) elevating the effluent manhole cover from 17 ft to 18.6 ft, so it will accommodate the design peak hour flow of 23 mgd without overflowing. (b) raising the Parshall flume invert elevation from 15.5 ft to 17.0 ft, so the flume will be less than 95 percent submerged under the design peak hour flow of 23 mgd.

Figure 7-4 illustrates the hydraulic profile of the existing treatment facilities at the limiting flow rate of 21.9 mgd. However, the extreme high flooding condition is not considered typical in the plant capacity evaluation due to its low frequency.

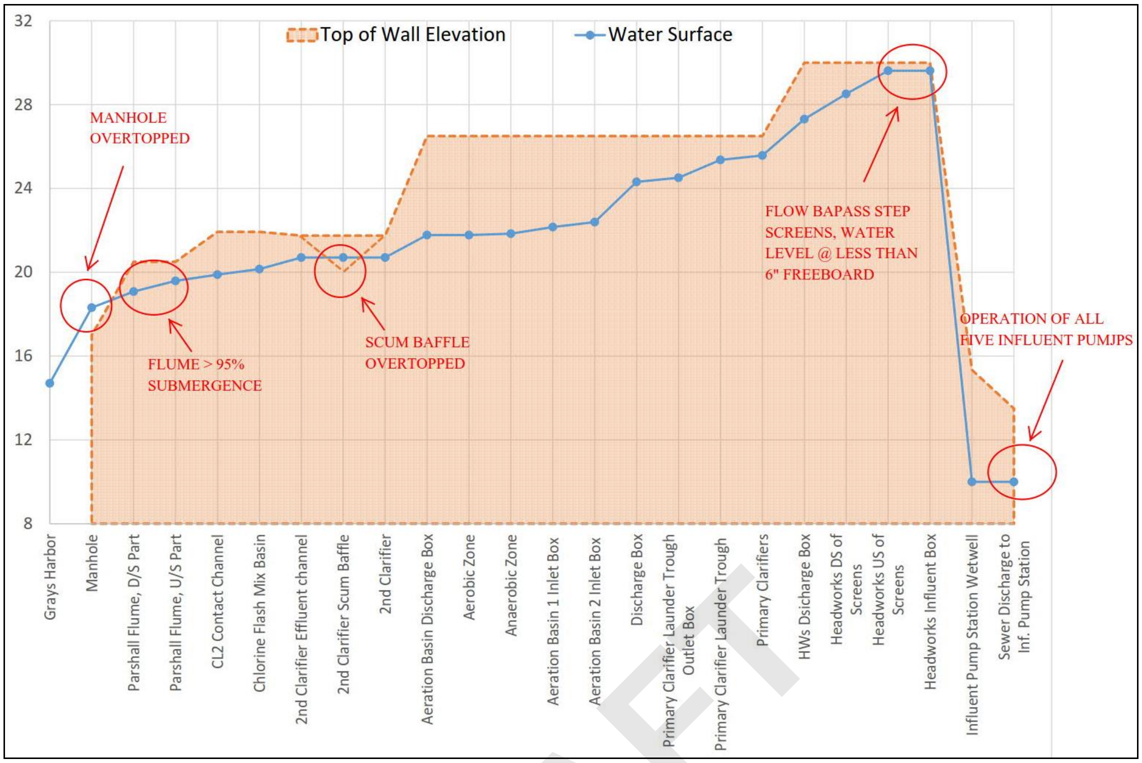

FIGURE 7-4

Hydraulic Profile at 21.9 mgd with 100-Year Extreme High Flooding (14.7 feet) Boundary Condition in Grays Harbor Estuary

CONCLUSIONS

The existing treatment facilities have several hydraulic deficiencies. The full capacity of 21.9 mgd of the existing influent pump station is sufficient to convey the projected influent sewage flow from the Aberdeen and its current partners (21.97 mgd) within the margin of error for the analysis. However, this analysis was conducted with the assumption that all pumps are in service; with the assumption that the largest pump is not in service, as required by Department of Ecology criteria, the Influent Pump Station capacity is insufficient. As discussed in Chapter 8, it is recommended that the capacity of the influent pump station be increased. However, since substantial reductions in flow are expected due to the decrease in flooding expected through completion of the $75-million North Shore Levee project, it is recommended that the influent pump station capacity upgrade be deferred until after completion of the North Shore Levee project and flows are further evaluated.

There are limitations with the effluent flow metering system depending on the receiving water elevations. Once the flume starts to become submerged, an additional measuring point is necessary to ensure measurement accuracy. Once the flume submergence exceeds 95 percent, the effluent flow measurement is considered to have failed. Thus, the submergence below 95 percent is assumed acceptable in this study assuming the dual point measurement will be in place in the near future.

Under the AEHT plus 1-foot surge condition, which is considered representative for this capacity evaluation, as flows increase, the failure to meet design criteria occurs in sequence at the following locations:

1. Influent Pump Station (pump capacity exceeded)
2. Secondary clarifier (weir overtopped)
3. Effluent manhole overflow (overtopped)
4. Secondary clarifier (scum baffle overtopped)
5. Effluent Parshall flume. (95 percent submerged)
6. Primary clarifier (weir overtopped)
7. Headworks (< 6 inches freeboard)

According to the City's municipal code, critical facilities (including wastewater treatment facilities) are to be protected from the 100-year flood with an additional 3 feet of freeboard. While this does not affect the hydraulic profile in this section, it does affect existing structures that do not conform to this requirement. Many of the buildings on the WWTP site were designed to withstand a flood elevation of 13.6 feet NAVD 88 (or 15.3 feet, City of Aberdeen datum).

The current FEMA 100-Year Flood Elevation is 14.7 feet (City of Aberdeen datum). With the 3 feet of freeboard, the elevation required is 17.7 feet (MLLW or City of Aberdeen datum). Flood proofing of the following buildings is required and discussed in Chapter 8 for the following facilities: emergency generator building, the blower building, the chlorine disinfection building, the headworks structure and the administration building.

PROCESS CAPACITY

This section summarizes an evaluation of the capacities of the major WWTP components at current and 2038 projected flows and loadings listed above in Chapter 5, and where applicable, compares them to accepted design criteria, such as those published in the Ecology's Criteria for Sewage Works Design (Orange Book, 2008), WEF Design of Wastewater Treatment Plants, Manual of Practice No. 8 (2010) and Wastewater Engineering (Metcalf and Eddy, 2014). The detailed calculations are presented in Appendix P.

INFLUENCE OF SOLIDS HANDLING RETURN FLOWS

In addition to influent flow, the treatment process must accommodate recycle flows from solids handling processes such as thickening and dewatering. These solids handling return flows typically contain 10 to 20 percent of a plant's influent loading. The assumptions for solids capture efficiency for thickening and dewatering are presented in

Table 7-4. Design flows and loading to the plant including these return streams are presented in Table 7-5.

TABLE 7-4

Solids Handling Assumptions

Description	Solids Capture (Existing)	Solids Capture (Planning Period)
Thickening	85%	85%
Dewatering	90%	90%

TABLE 7-5

Summary of Plant Flows and Loadings Including Solids Handling Return Streams

Description	Existing (2018)	Planning Period (2038)
Aberdeen and Existing Regional Partners		
Influent + RF Flow (mgd)		
Average Annual	3.92	4.52
Maximum Month	6.89	7.49
Maximum Day	20.72	19.20
Peak Hour	23.11	22.12
Influent + RF BOD (ppd)		
Average Annual	7,124	9,243
Maximum Month	8,483	10,894
Maximum Day	16,179	20,743
Influent + RF TSS (ppd)		
Average Annual	7,668	9,896
Maximum Month	9,404	11,953
Maximum Day	22,122	28,196
Influent + RF TKN[(1)] (ppd)		
Average Annual	816	1,060
Maximum Month	958	1,234
Maximum Day	1,349	1,727

TABLE 7-5 – (continued)

Summary of Plant Flows and Loadings Including Solids Handling Return Streams

Description	Planning Period (2023)[2]	Planning Period (2038)
Aberdeen and Expanded Regional Partners		
Influent + RF Flow (mgd)		
Average Annual	5.40	6.42
Maximum Month	10.15	11.54
Maximum Day	25.10	26.55
Peak Hour	27.58	29.63
Influent + RF BOD (ppd)		
Average Annual	10,026	13,093
Maximum Month	12,474	16,220
Maximum Day	21,689	27,495
Influent + RF TSS (ppd)		
Average Annual	10,657	13,914
Maximum Month	13,727	17,812
Maximum Day	28,436	36,188
Influent + RF TKN[1] (ppd)		
Average Annual	1,237	1,616
Maximum Month	1,541	2,005
Maximum Day	2,113	2,636

(1) TKN was obtained from Chapter 5 Flow and Loading based on typical NH3-N/ TKN ratio of 0.7

(2) It is assumed in this analysis that the Expanded Regional Partners will start contributing flow to Aberdeen WWTP in year 2023.

(3) RF indicates Recycle Flows

INFLUENT PUMPS

With five active pumps installed (include the submersible bypass pump), only four are available as "firm" capacity by state and federal reliability criteria, as discussed in Chapter 3. However, this station is operating currently with all pumps on during peak flows. The Influent Pump Station firm capacity of 15.3 mgd is inadequate for current and 2038 plant peak flow conditions.

As discussed earlier in this chapter and in more detail in Chapter 8, it is recommended that the capacity of the influent pump station be increased incrementally by replacing Pumps 4 and 5 with 4,000 gpm pumps to bring the Influent Pump Station capacity to 22 mgd.

The Influent Pump Station cannot accommodate flows from Hoquiam associated with the Expanded Regional Option. It is assumed that the pump stations conveying Hoquiam flows would convey the wastewater all the way to a new dedicated headworks at the Aberdeen WWTP.

HEADWORKS

The existing Aberdeen plant headworks features two Huber step screens, with a bypass channel containing a manual bar screen. The upstream portion of the bypass channel has an adjustable weir gate installed at 2 feet above the channel floor. The channel immediately downstream of the screens is highly restricted in low flow conditions as a consequence of maintaining adequate screen submergence. The step screens use a single (common) washer compactor. The capacity of the manual bar screen with the upstream weir gate installed is limited to about 6.1 mgd when there is 6 inches of freeboard at the most up-gradient part of the headworks and 8.3 mgd when the headworks is overtopped. If the City removes the weir gate entirely then the capacity of the bypass channel is about 12.5 mgd with a clean bypass screen and 6 inches of freeboard.

The existing Headworks equipment (fine screens, screenings conveyor, and screenings washer/compactor) was installed in 2005 and has exceeded its useful life. In addition, peak flows to the Headworks exceed the design capacity of the screening facilities. Bypassing to the manual bar screen, which is designed as an emergency backup, has occurred when the flow is as low as 4.2 mgd. Due to the lack of capacity, reliability and redundancy of the Headworks, not all of the incoming sewage is adequately screened, which is not in compliance with Biosolids regulations (WAC-173-308) and Orange Book criteria. The biosolids regulations require that the sewage, sludge or biosolids be screened to treated by other process to remove manufactured inert material so that it is minimized in the final treated biosolids. Adequate screening is defined by the performance of the bar screen with 3/8-inch bar spacing. The existing step screens meet this requirement but the manual bar screen does not. Only three of the five influent pumps convey wastewater to a location *upstream* of the headworks screens. In addition, overflow events have occurred, including an event on November 4, 2018 in which an estimated 780,000 gallons of untreated sewage was discharged to the Chehalis estuary.

The existing Headworks structure has some degradation of concrete, necessitating some concrete repair. Other than that, it is in reasonably good condition and not in need of replacement due to physical deterioration of the structure.

An upgrade to the Headworks is a high priority due to its condition and capacity issues. The facilities were designed for a total step screen capacity of 18 mgd. In reality, the capacity is only about 13 mgd. With current plant peak hour flows approaching 23 mgd, these facilities are operating beyond their intended design range. In addition to the screens, additional redundancy is needed for the conveyors (or addition of sluiceways) and washer-compactors. If a conveyor fails, there is only a single bypass channel with a capacity of 8.3 mgd, if personnel rake the screen continuously. The City does not have a backup conveyor on the shelf. The City does not have a spare washer compactor and handling raw screening during maintenance and repair would be problematic. (Note: The conveyor is a single point of failure, plus housekeeping to prevent debris from accumulating on the conveyor is usually a challenge. Another option that can be considered during the design phase is implementation of sluiceways for screenings handling, which can make it easier to control debris. Also, a sluiceway requires only one mechanical component: a pump to sustain flow, and a redundant unit can be provided. With a sluiceway, the washer/compactor does have to be specified with adequate hydraulic capacity.)

TABLE 7-6

Preliminary Treatment Capacity Summary

Unit Process	Limiting Criterion	Criteria			Aberdeen and Existing Partners Operating Condition		Aberdeen and Expanded Regional Partners Operating Condition		Aberdeen and Expanded Regional Partners w/Industrial Flow Operating Condition	
		Criteria	Units	Source	Existing	Planning Period (2038)	Planning Period (2023)	Planning Period (2038)	Planning Period (2023)	Planning Period (2038)
Influent Pumps	Capacity	15.3	mgd	Equipment Capacity	23.1	22.1	27.6	29.6	29.6	32.1
Influent Step Screen	Capacity	13	mgd	Equipment Capacity	23.1	22.1	27.6	29.6	29.6	32.1

The headworks cannot accommodate flows from Hoquiam associated with the Expanded Regional Option. It is assumed that a new headworks dedicated to Hoquiam flows would be constructed at the Aberdeen WWTP.

The capacity evaluation of the preliminary treatment is summarized in Table 7-6.

PRIMARY TREATMENT

Primary Clarifiers

The recommended surface overflow rate for primary clarifiers in the Orange Book (Ecology 2008) is 2,000 to 3,000 gallons per day per square foot (gpd/sf) at “Peak Design Flow,” and 800 to 1,200 gpd/sf at “Average Design Flow.” The Orange Book also says “Surface overflow rates higher than those recommended above for primary settling tanks may be acceptable if the secondary treatment process, including the waste activated sludge system, is able to satisfactorily treat the greater amount of organic loading that passes through the primary treatment process.” In this evaluation, an overflow rate criterion of 3,500 gpd/sf at peak hour flow is used. Based on this criterion, the tanks are approaching their capacity limit under existing flow conditions. The peak hour overflow rate is projected to decline in the planning period since the Levee project is expected to significantly reduce inflow and thus the peak hour flow in the collection system.

The recommended weir loading rate for primary clarifiers in the Orange Book (Ecology 2018) is 10,000 to 40,000 gpd/lf. Although the upper bound loading rate has been exceeded during peak flow events, there has been little impact on the performance of the solids removal by the primary clarifiers. The only concern would be excessive water velocities that can entrain solids from the tank floor and possibly cause solids carryover in the effluent. Since there is no evidence of such situation during plant operation, the weir loading rate during existing peak flows is not considered a capacity restriction in this study.

The Ecology-required 2.5-hour hydraulic detention time is met throughout the 20-year planning period for the Aberdeen and Existing Partners scenario.

The primary clarifiers meet the requirements for Reliability Class II. The reliability criteria require that primary clarifiers should be sufficient in number and size so that, with the largest flow capacity unit out of service, the remaining units have a design flow capacity of at least 50 percent of the design flow.

Overall, the existing two primary clarifiers have adequate capacity for planning year flows for Aberdeen and existing partners. An additional 65-foot primary clarifier would be needed to accommodate flows from Expanded Regional Partners (Hoquiam and Central Park).

Primary Sludge Pump

The existing primary sludge pump has a capacity of 205 gpm, which is more than adequate for the planning period to accommodate Aberdeen and existing partners. There is one primary sludge pump that conveys flow to the hydrocyclone. If that pump fails or the cyclone fails then the City can use the scum pumps to pump primary sludge and scum to the digester. Additional primary sludge pumping facilities would be needed to accommodate Expanded Regional Partners.

Primary Sludge Grit Removal System

Grit removal for the plant is through a cyclone and classifier system on the plant primary sludge. Sludge pumped from the clarifiers needs to be synchronized to same flow rate of feeding to the cyclone. The 200-gpm capacity is more than adequate for the planning period provided only one clarifier is pumped at a time. The City receives occasional moderately large grit loads when high flows scour the collection system. Normally, these grit volumes are manageable. However, recently (December 20, 2019), the City received two dumpsters worth of grit (more than 20 times the normal daily volume) in one day. Volumes of this magnitude can overwhelm the system and cause deterioration of the degritting system and allow grit to interfere with the operation of downstream sludge handling and processes. It is anticipated that I/I reduction will decrease the magnitude of the grit slug loads. However, it is recommended that grit volumes and impacts be closely monitored; if necessary, it may be necessary to expand the grit handling system through the addition of an additional hydrocyclone or a grit removal at the headworks.

The capacity evaluation of the primary treatment system is summarized in Table 7-7.

TABLE 7-7

Primary Treatment Capacity Summary

Unit Process	Limiting Criterion	Criteria			Aberdeen and Existing Partners		Aberdeen and Expanded Regional Partners		Aberdeen and Expanded Regional Partners w/Industrial Flow	
					Operating Condition		Operating Condition		Operating Condition	
		Criteria	Units	Source	Existing	Planning Period (2038)	Planning Period (2023)	Planning Period (2038)	Planning Period (2023)	Planning Period (2038)
Primary Sedimentation Tank	PHF Overflow Rate	≤ 3500	gpd/sf	Ecology Orange Book, 2008	3484	3334	4158	4467	4459	4844
Primary Sedimentation Tank	AAF Overflow Rate	≤ 1200	gpd/sf	Ecology Orange Book, 2008	591	682	815	967	1116	1269
Primary Sedimentation Tank	AAF Detention Time	≥ 2.5	hr	Metcalf & Eddy, 2014	3.0	2.6	2.2	1.9	1.6	1.4
Primary Sedimentation Tank	Alternative unit serves 50 percent of the PHF with primary unit out of service	≥ 50	%	EPA4-430-99-74-001, Reliability Criteria	50(1)	52(1)	42.1(1)	39.2(1)	39.2(1)	36.1(1)
Primary Sludge Pump	Capacity	205	gpm	Equipment Capacity	113	144	131	168	157	201
Hydrocyclone and Girt Classifier	Capacity	200	gpm	Equipment Capacity	113	144	131	168	157	201(2)

(1) Based on 3,500 gpd/sf overflow criterion
(2) Process is marginally overloaded during peak flow condition. The capacity is considered adequate.

ACTIVATED SLUDGE SYSTEM

Aeration tanks, secondary clarifiers, and return activated sludge pumps are components of the overall activated sludge system for removal of oxygen-demanding pollutants. To adequately model system capacity, these components must be considered together. The evaluation approach includes the following steps:

- Calculate sludge production
- Calculate mixed-liquor suspended solids (MLSS)
- Calculate limiting clarifier solids flux

To meet the permit requirement for ammonia removal, it is assumed that full nitrification in the biological treatment process is provided.

Projected sludge production for the planning period was used to predict MLSS concentrations in the aeration tank, based on the plant-reported solids residence time (SRT) of 5 days for the existing condition and 9.3 days for BOD removal with full nitrification. This, in turn, was used to calculate limiting solids flux values for the secondary clarifiers. The limiting solids flux is the maximum loading of solids in pounds per day per square foot (ppd/sf) of surface area that can be applied to the clarifier without excessive solids loss over the effluent weir.

Loadings

Projected secondary treatment loadings are shown in Table 7-8.

TABLE 7-8

Secondary Treatment Loading

Description	Existing (2018)	Planning Period (2038)
Aberdeen and Existing Regional Partners		
Flow (mgd)		
Average Annual	3.92	4.52
Maximum Month	6.89	7.49
Maximum Day	20.72	19.20
Peak Hour	23.11	22.12
BOD_5 (ppd)		
Average Annual	3,918	5,084
Maximum Month	4,666	5,991
Maximum Day	8,413	10,786
TSS (ppd)		
Average Annual	3,027	3,907
Maximum Month	3,713	4,719

TABLE 7-8 – (continued)

Secondary Treatment Loading

Description	Existing (2018)	Planning Period (2038)
Maximum Day	8,318	10,602
TKN (ppd)		
Average Annual	449	583
Maximum Month	527	679
Maximum Day	702	898
Description	**Planning Period (2023)(1)**	**Planning Period (2038)**
Aberdeen and Expanded Regional Partners		
Flow (mgd)		
Average Annual	5.35	6.35
Maximum Month	10.09	11.46
Maximum Day	24.98	26.40
Peak Hour	27.46	29.48
BOD (ppd)		
Average Annual	6,069	7,902
Maximum Month	7,550	9,785
Maximum Day	12,689	16,044
TSS (ppd)		
Average Annual	5,484	5,844
Maximum Month	7,052	7,481
Maximum Day	13,988	14,475
TKN (ppd)		
Average Annual	749	975
Maximum Month	933	1,210
Maximum Day	1,237	1,538

(1) It is assumed in this analysis that the expanded regional partners will start contributing flow to Aberdeen WWTP in year 2023.

Biological Selectors

The selectors at the inlet end of each aeration basin provide compartmentalization to create an environment with a high food/mass (F/M) ratio to favor the growth of floc-forming (readily settling) organisms, and produce a low sludge volume index (SVI). According to M&E recommended F/M of at least 2 lb BOD/lb MLSS, is achieved with current facilities, however, the year 2038 F/M is inadequate for Aberdeen with existing regional partners.

The hydraulic detention times are in compliance with the Ecology and M&E recommended 10 to 30 minutes throughout the 20-year period.

A new aeration basin, including selectors is necessary for the Expanded Regional Alternative.

Aeration Basin

Common empirical design criteria for aeration tank design are hydraulic residence time (HRT), BOD loading, and Solids Retention Time (SRT).

The BOD_5 removal capacity of aeration basins can be adjusted by operational parameters, such as mixed liquor suspended solids, solids wasting rate, solids retention time, and food-to-microorganism ratio. Therefore, as the BOD_5 removal capacity of the aeration basins is approached, the operational parameters should be reviewed and analyzed to establish appropriate expansion schemes. Although the City of Aberdeen does not have ammonia limits at the present time, future requirements for nitrification in the aeration basin will affect the nominal BOD_5 removal capacity of the aeration basins.

Activated sludge systems can be operated over a wide range of loadings. The Department of Ecology's Orange Book provides a range of design values for aeration tank organic (BOD) loading of 20 to 60 pounds of BOD per day per 1,000 cubic feet of tank volume, with an HRT of 3 to 5 hours, based on the average design flow and loading for the complete-mix loading pattern. M&E recommends a maximum BOD loading limit of 100 ppd/1,000 cu.ft. and minimum HRT of 0.5 hr under peak flow conditions.

Currently, Aberdeen is in the medium to high range of loading for typical activated sludge processes. Based on current projections, the capacity will be exceeded about 2038 (the planning year).

A higher mixed liquor suspended solids (MLSS) may reduce the settleability of the sludge, and result in greater solids loading on the secondary clarifier, which may cause the suspended solids concentration in the effluent to increase during periods of reduced settleability. The Orange Book specifies a design range for MLSS of 1,500 to 3,500 mg/L. The MLSS will exceed the desirable value within the 20-year planning period due to the longer SRT to meet the nitrification requirements.

Based on the analysis, throughout the planning period, at projected 2038 flow and loadings for Aberdeen and the Existing Regional Partners, the existing aeration basin will be marginally overloaded for carbonaceous and limited ammonia removal, while inadequate for sustainable ammonia removal. An additional (third) aeration basin is required before 2038 to accommodate Aberdeen and its existing partners.

For the Expanded Regional Alternative, a second new aeration basin is required, bringing the total number of aeration basins at the plant to four.

Aeration System

The aeration system includes fine bubble membrane strip diffusers and centrifugal blowers. As shown in Table 7-9, the firm capacity of existing process air blowers is 3,650 ICFM. The maximum TKN the WWTP can treat while treating the current maximum month secondary CBOD loading of 4,666 lb/d is 564 lb/d. The maximum secondary CBOD loading the WWTP can treat without TKN removal is 7,410 lb/d. The analysis shows that the existing aeration system will be out of capacity by 2038 with Aberdeen and Existing Partners.

It is expected that the aeration basin oxygen demand is decreased by the process of denitrification and by the periodic wasting of biomass growth. (However, the ability to denitrify might be limited by low concentrations of readily biodegradable BOD in the influent.) It is difficult to predict the level of denitrification without plant operation data, so the credit of the oxygen demand that might be provided by denitrification is not counted in this study. The existing anoxic selector configuration does not have adequate detention time for denitrification except during dry weather flows. The process of nitrification consumes 7.1 lbs of alkalinity per pound of ammonia nitrified. This causes the pH to be lower. About half of the alkalinity required by the nitrification process can be recovered through a biologically medicated denitrification process provided there is: adequate detention time, adequate biodegradable carbon available in the influent, an adequate population of denitrifying organisms and the redox potential of the mixed liquor is in the range between +50 mV and -150 mV. Nitrification of ammonia in the secondary treatment process at Aberdeen has caused the pH of the mixed liquor and the effluent to fall below the effluent discharge requirement and has required supplemental alkalinity addition using sodium bicarbonate and magnesium hydroxide. The City uses an adaptive management process control strategy to maximize ammonia removal to the extent possible without routine supplemental addition of alkalinity.

TABLE 7-9

Evaluation of Oxygen Demand for Activated Sludge Treatment

Airflow	$BOD_{influent}$	$BOD_{removed}$	$TKN_{oxidized}$
icfm	lb/d	lb/d	lb/d
3,650	2,000	1,800	1,113
3,650	2,500	2,250	1,010
3,650	3,000	2,700	907
3,650	3,500	3,150	804
3,650	4,000	3,600	701
3,650	4,500	4,050	599
3,650	5,000	4,500	496
3,650	5,500	4,950	393
3,650	6,000	5,400	290
3,650	6,500	5,850	187
3,650	7,000	6,300	85
3,650	7,410	6,669	0

Secondary Clarifiers (Sedimentation Tanks)

The peak hour flows produce overflow rates at the clarifiers approaching the Orange Book design criterion of 1,200 gpd/sf with all three clarifiers operating under projected planning year (2038) conditions. Based on the overflow rate, the peak hour capacity of the existing secondary clarification process is 23 mgd with all existing clarifiers in service, which is marginally adequate for Aberdeen and Existing Partners. Per the Reliability Criteria, the secondary clarifiers must be able to meet the treat 50 percent of the design peak hour flow rate. With the largest clarifier out of service, the overflow rate is 968 gpd/sf, so there is adequate hydraulic capacity for Aberdeen and Existing Partners through the planning period.

Based on the WEF criterion of 50 ppd/sf as the allowable peak hour solids loading, the clarifiers will have adequate capacity, with the construction of a new aeration basin for projected 2038 flows and loadings. (The construction of a third aeration basin train will allow operation at lower MLSS, but with a larger overall solids inventory, which will be needed for reliable ammonia removal.) Without the additional aeration basin, the solids loading rate would significantly exceed the WEF criterion (78 ppd/sf vs. the 50 ppd/sf criterion) at 2038 flows and loadings.

For the Expanded Regional Alternative, additional clarification capacity would be needed. This could be accomplished by either expanding the existing 85-foot clarifiers to 100 feet (by removing the chlorine contact tanks on the periphery and replacing the collector mechanisms), or building a new secondary clarifier.

Return Sludge Pumps

The return sludge pumps have a firm capacity (capacity with one unit out of service) of 4.9 mgd for the small clarifiers and 3.3 mgd for the large clarifier. This firm capacity limits the RAS rate to 37 percent at the planning period peak hour flow if no modifications are made to the existing pumping system.

WAS Pump

The existing WAS pump station has a capacity of 100 gpm, which is more than adequate for the planning period.

Additional RAS facilities will be needed for the Expanded Regional Alternative.

The capacity evaluation of the secondary treatment system is summarized in Table 7-10.

TABLE 7-10

Secondary Treatment Capacity Summary

Unit Process	Limiting Criterion	Criteria			Aberdeen and Existing Partners		Aberdeen and Expanded Regional Partners		Aberdeen and Expanded Regional Partners w/Industrial Flow	
		Criteria			Operating Condition		Operating Condition		Operating Condition	
		Criteria	Units	Source	Existing	Planning Period (2038)	Planning Period (2023)	Planning Period (2038)	Planning Period (2023)	Planning Period (2038)
Aeration Tank	MMF Biological Selector Detention Time	≥ 30	min	Ecology Orange Book, 2008	36.3	33.4	24.8	21.9	20.8	18.6
Aeration Tank	PHF Biological Selector Detention Time	≥ 10	min	Ecology Orange Book, 2008	10.8	11.3	9.1	8.5	8.5	7.8
Aeration Tank	PHF Biological Selector F/M	≥ 2	lb BOD/lb MLSS	Metcalf & Eddy, 2014	2.0	1.1	2.0	1.2	2.0	1.2
Aeration Tank	AAF HRT	≥ 3	hr	Ecology Orange Book, 2008	4.0	3.5	4.2	3.6	3.1	2.7
Aeration Tank	PHF HRT	≥ 0.5	hr	Metcalf & Eddy, 2014	0.7	0.7	0.8	0.8	0.8	0.7
Aeration Tank	MMF Aerobic SRT	≥ 5	days	Metcalf & Eddy, 2014	5.0	9.3	5.0	9.3	5.0	9.3
Aeration Tank	MMF MLSS	≤ 3500	mg/l	Ecology Orange Book, 2008	2176	5584	3522	8207	4224	9844
Aeration Tank	AAF Unit BOD Loading	≤ 60	ppd/kcf	Ecology Orange Book, 2008	42	55	65	85	78	102
Aeration Tank	PHF Unit BOD Loading	≤ 120	ppd/kcf	Metcalf & Eddy, 2014	91	116	137	173	164	207
Aeration Blower	Capacity	5800	cfm	Equipment Capacity	4115	7179	6304	13295	7563	15951

TABLE 7-10 – (continued)

Secondary Treatment Capacity Summary

Unit Process	Limiting Criterion	Criteria			Aberdeen and Existing Partners Operating Condition		Aberdeen and Expanded Regional Partners Operating Condition		Aberdeen and Expanded Regional Partners w/Industrial Flow Operating Condition	
		Criteria	Units	Source	Existing	Planning Period (2038)	Planning Period (2023)	Planning Period (2038)	Planning Period (2023)	Planning Period (2038)
Secondary Sedimentation Tank	MMF Overflow Rate	≤ 800	gpd/sf	Ecology Orange Book, 2008	359	390	525	596	628	699
Secondary Sedimentation Tank	PHF Overflow Rate	≤ 1200	gpd/sf	Ecology Orange Book, 2008	1204(1)	1152	1429	1534	1532	1663
Secondary Sedimentation Tank	MMF Solids Loading Rate	≤ 30	ppd/sf	WEF, 2010	9.4	26.3	22.3	59.0	32.0	83.0
Secondary Sedimentation Tank	PHF Solids Loading Rate	≤ 50	ppd/sf	WEF, 2010	31.6	77.6	60.7	152	78.0	197
Secondary Sedimentation Tank	Alternative unit serves 50 percent of the PHF with primary unit out of service	≥ 50	%		58.9(2)	61.5(2)	49.4(1)(2)	45.9(2)	46.0(2)	42.4(2)
RAS Pump	Capacity	8.2	mgd	Equipment Capacity	10.4(3)	10.0(3)	12.3	13.3	13.2	14.4
WAS Pump	Capacity	100	gpm	Equipment Capacity	36.8	40.4	47.5	55.1	57.0	66.1

(1) Process is marginally overloaded during peak flow condition. The capacity is considered adequate.
(2) Based on 1200 gpd/sf overflow criterion.
(3) The equipment capacity is based on the firm capacity (one unit out of service). The backup unit could cover the shortage during the peak flow condition. The capacity is considered adequate.

DISINFECTION

The existing gas-based chlorination and dechlorination systems are in the process of being replaced with new systems using liquid sodium hypochlorite and calcium thiosulfate. The project will be completed in 2020. The systems will provide ample capacity throughout the planning period for both the Existing Partner and Expanded Partner scenarios.

Chlorine Contact Tank

Orange Book criteria for chlorine contact residence time are 15 minutes at peak day flow and 60 minutes at average flow. For Aberdeen and Existing Partners, the tanks have adequate capacity, with a residence time of 16 minutes at peak day flow and 76 minutes at annual average flows. Maximum month residence time is 46 minutes.

For Aberdeen and Expanded Regional Partners, additional chlorination contact capacity would be necessary.

Effluent Parshall Flumes

The existing Parshall flumes have adequate capacity for Aberdeen and Existing Partners. However, the twin Parshall flume capacity is limited by the current setting on the flow meter transmitter. The flume recorder should be recalibrated or rescaled. Two-point measurement should be provided to compensate for flume submergence during extreme high tide and high flow events. The current strategy involves using the influent flow to the headworks as the flow to flow pace the disinfection system and dechlorination system and to account for the effluent discharge when the plant flows exceed approximately 16 mgd. When effluent flows fall below 16 mgd then the disinfection and dechlorination system and effluent discharge recording revert to the sum of the flow rates measured at the effluent Parshall flume structure.

The Parshall flumes would need to be modified to accommodate the additional flows contributed by the Expanded Regional Partners.

The capacity evaluation of the disinfection system is summarized in Table 7-11

TABLE 7-11

Disinfection Capacity Summary

Unit Process	Limiting Criterion	Criteria			Aberdeen and Existing Partners Operating Condition		Aberdeen and Expanded Regional Partners Operating Condition		Aberdeen and Expanded Regional Partners w/Industrial Flow Operating Condition	
		Criteria	**Units**	**Source**	**Existing**	**Planning Period (2038)**	**Planning Period (2023)**	**Planning Period (2038)**	**Planning Period (2023)**	**Planning Period (2038)**
Chlorine Contact Tank	PDF Contact Time	≥ 15	min	Ecology Orange Book, 2008	14.9[1]	15.6	12.5	11.7	11.7	10.8
Chlorine Contact Tank	AAF Contact Time	≥ 60	min	Ecology Orange Book, 2008	87.8	76.1	64.5	54.3	47.0	41.4
Disinfection Chemical Feed System	Capacity	3168	ppd	Equipment Capacity	1641	1934	1990	2524	2165	2809

(1) Process is marginally overloaded during peak flow condition. The capacity is considered adequate.

SOLIDS HANDLING FACILITIES

Analysis of the Solids Handling and Biosolids Management facilities and operations is provided in the technical memorandum in Appendix Q. The following section is provided to summarize and supplement that analysis.

The solids treatment system design flow and loading to the gravity sludge thickener for the existing and planning are presented in Table 7-12

TABLE 7-12

Solids Handling Loadings

Description	Existing (2018)	Planning Period (2038)
Aberdeen and Existing Regional Partners		
Combined Sludge Amount (ppd)		
Average Annual	6,471	8,115
Maximum Month	7,750	9,471
Maximum Day	16,779	20,455
Combined Sludge Flow (gpd)		
Average Annual	51,255	63,786
Maximum Month	60,991	73,731
Maximum Day	127,640	153,386
Combined Sludge Concentration	1.5%	1.5%
Description	**Planning Period (2023)(1)**	**Planning Period (2038)**
Aberdeen and Expanded Regional Partners		
Combined Sludge Amount (ppd)		
Average Annual	8,778	11,121
Maximum Month	10,955	13,796
Maximum Day	21,300	26,389
Combined Sludge Flow (gpd)		
Average Annual	67,035	84,011
Maximum Month	82,788	103,122
Maximum Day	155,086	189,952
Combined Sludge Concentration	1.6%	1.6%

(1) It is assumed in this analysis that the expanded regional partners will start contributing flow to Aberdeen WWTP in year 2023.

Sludge Thickener

At projected planning year (2038) flows and loadings, the maximum month and maximum day solids loadings to the gravity thickener are 7.5 ppd/sf and 16.3 ppd/sf, respectively. The recommended loading rates for gravity thickener treating a mixture of primary and waste activated sludge are in the range of 5.0 to 15.0 ppd/sf (EPA 1979), and for only primary sludge, the loading limit is 24 ppd/sf (WEF 2010). It is expected that, in the future, the primary sludge will be thickened in gravity sludge thickener, and the WAS will be thickened by rotary drum thickener.

The pumped flow rate to the gravity thickener results in a surface overflow rate of 235 gpd/sf in planning year with existing regional partners which is acceptable.

Overall, the capacity of the gravity sludge thickener is sufficient in the planning period for both the Existing Partners and Expanded Regional Partner scenarios. The thickened sludge pumps, each with a firm capacity of 75 gpm at projected planning year (2038) flows and loadings.

Rotary Screen Thickener

The rotary screen thickener is occasionally used to co-thicken the sludge from gravity sludge thickener. It has adequate capacity at projected planning year (2038) flows and loadings. The RDT has also been used to recuperatively thicken the digestate, however this practice is rarely utilized because it has not improved gas production or volatile solids destruction.

Sludge Digestion

The Aberdeen WWTP receives outside sludge and grease at the inlet of the sludge digester. Table 7-13 summarizes the outside sludge loading. Besides the impact of the sudden loading increase on the digestion system, the additional volatile fatty acids will increase the acidity of sludge and potentially affect methanogenesis (methane production), thus decreasing the volatile solids reduction rate and the overall efficacy of digestion.

TABLE 7-13

Outside Sludge and Grease Data

Description	Existing	Planning Period
Sludge Flow (gpd)		
Average Annual	3,000	3,000
Maximum Month	4,500	4,500
Maximum Day	8,000	8,000
Concentration	5%	5%
Solids Loading (ppd)		
Average Annual	1,251	1,251
Maximum Month	1,877	1,877
Maximum Day	3,336	3,336
Volatile Solids %	84%	84%
Volatile Solid (ppd)		
Average Annual	1,051	1,051
Maximum Month	1,576	1,576
Maximum Day	2,802	2,802

Based on the reported plant sludge data, the outside hauls load the plant at a frequency of average 3 days per week. In the digestion process evaluation, the average annual outside sludge loading is averaged when added to the plant average annual and maximum month loading. The maximum day loading is the maximum month plant loading plus maximum day outside loading or the maximum day plant loading, whichever is higher.

Based on the evaluation, the average digester SRT is estimated at 25 days under current conditions, and decreases to 21 days under future (year 2038) maximum month loadings. Based on the SRT requirement of 20 days to meet Class B quality, the plant has adequate SRT throughout the planning period with the existing regional partners.

At the planning year design condition with existing regional partners, the volatile solids loading of the existing digester is 0.17 lb VS/d/cu.ft, which is in compliance with the WEF and M&E recommended loadings for the anaerobic digestion process.

As shown in Chapter 3, one of the criteria to show compliance with vector attraction reduction requirements in the State Biosolids regulations (WAC 173-308) for sludge disposal is a minimum volatile solids destruction (VSR) of 38 percent. The digester meets the minimum digester VSR requirements almost all the time, and this is expected to continue through the planning period. However, the WWTP does not currently have a backup digester. As discussed in the Solids Handling Analysis in Appendix Q, it is recommended that an additional digester be provided for redundancy. This additional digester will be necessary to accommodate Expanded Regional flows, if that alternative is selected.

Because solids are not dewatered on a daily basis, maintaining storage capacity in the digester can become problematic when the volume of sludge fed to the digester is greater than the rate at which the digestate is withdrawn.

Sludge Holding Tanks

The sludge holding tanks are the old 1950s digesters which are currently in poor condition and will be demolished. There are no mixers in the tanks and the digestate does not stay homogenous when stored in an unagitated tank. The tanks are not used for normal plant operations. One of the tanks has been used in past to store sludge to be used to seed the large anaerobic digester following digester cleaning.

Dewatering

The screw press receives anaerobically digested sludge. The screw press is rated at 425 dry pounds of sludge per hour. At the 2038 maximum month design loading, the screw press will operate continuously for about 3 days (75 hours) during each 7-day week, including startup and shutdown time.

Since the dewatering system operates most efficiently when running continuously, and the weekly operating time at maximum month conditions allows over 4 days each week for shutdown, the existing screw press is adequate for 2038 design loading. Therefore, no expansion of the dewatering facility is required for both the Existing Partner and Expanded Regional Partner scenarios.

Press feed pump, with the designed hours of operation of the screw press, will have adequate capacity for future solids handling needs for Aberdeen and existing partners and with expanded partners.

The capacity evaluation of the solids handling system is summarized in Table 7-14.

Pressate generated during dewatering contains relatively high concentrations of ammonia as a consequence of the sludge digestion process. Strategies to manage the impact of this ammonia load is discussed in the analysis of alternatives.

TABLE 7-14

Solid Handling Capacity Summary

Unit Process	Limiting Criterion	Criteria			Aberdeen and Existing Partners		Aberdeen and Expanded Regional Partners		Aberdeen and Expanded Regional Partners w/Industrial Flow	
					Operating Condition		Operating Condition		Operating Condition	
		Criteria	Units	Source	Existing	Planning Period (2038)	Planning Period (2023)	Planning Period (2038)	Planning Period (2023)	Planning Period (2038)
Gravity Sludge Thickener[1]	PHF Solids Loading Rate	≤ 24	ppd/sf	WEF, 2010	13.3	16.3	16.9	21.0	20.3	25.2
Gravity Sludge Thickener	MMF Overflow Rate	≤ 500	gpd/sf	WEF, 2010	235	235	235	235	235	235
Rotary Drum Thickener	MMF Hours of Operation	11	hrs	Equipment Capacity	8.8	11.0	16.7	21.2	20.1	25.4
Thickened Sludge Pump	Capacity	75	gpm	Equipment Capacity	33.9 [2]	41.4 [2]	43.1 [2]	53.4 [2]	51.7 [2]	64.0 [2]
Anaerobic Digester	MMF Solids Residence Time	≥ 15	days	EPA Biosolid Permit, 2007	24.9	20.5	17.8	14.3	15.0	11.9
Anaerobic Digester	PDF Volatile Solids Loading	≤ 0.2	lb/cf	WEF, 2010	0.14	0.17	0.18	0.22	0.21	0.26
Anaerobic Digester	PDF Volatile Solids Reduction	≥ 38	%	EPA Biosolid Permit, 2007	45.0	42.7	41.2	38.8	39.2	36.8
Dewatering Screw Press	MMF Hours of Operation per Week	≤ 80	hr/wk	Equipment Capacity	57.6	74.8	85.3	113.2	104.4	138.0

(1) It is assumed only primary sludge is thickened by the gravity sludge thickened.in the future.
(2) Based on 11 hr/day operation time.

CHAPTER 8

WASTEWATER TREATMENT PLANT ALTERNATIVES ANALYSIS

In this chapter, alternatives are considered for future treatment of City of Aberdeen wastewater. The Aberdeen Wastewater Treatment Plant (WWTP) treats flows from the City's existing wastewater system, plus the City's Existing Partners: the City of Cosmopolis, and the Stafford Creek Corrections Center (SCCC). This alternatives analysis considers the feasibility and cost effectiveness of expanding the existing facility to serve future projected flows from Aberdeen and its Existing Partners, or expanding the existing WWTP to serve the "Expanded Regional Partners," which include the existing partners and the City of Hoquiam (which has its own plant) and Central Park (currently unsewered), or developing a new larger facility to serve either the Existing Regional Partners or Expanded Regional Partners.

WWTP ALTERNATIVES

The following future WWTP alternatives were evaluated:

1. Serve Existing Regional Partners on Existing Site
2. Serve Expanded Regional Partners on Existing Site
3. Serve Existing Regional Partners on New Site
4. Serve Expanded Regional Partners on New Site

As discussed in Chapter 7, based on updates to the Mixing Zone Study, it is expected that no new permit limits (i.e., for pollutant concentrations) will be necessary for either the Existing Partner or Expanded Regional WWTP, ***assuming the WWTP continues to nitrify***. For an Existing Regional WWTP, effluent ammonia would need to remain below 40 mg/L to avoid triggering a permit limit for ammonia. For an Expanded Regional WWTP, effluent ammonia would need to remain below 33 mg/L to avoid triggering a permit limit for ammonia. However, it is recommended that the WWTP nitrify to achieve significantly lower levels than these triggers. The City's NPDES permit requires the City to "operate the facility to minimize ammonia in the discharge".

Within each of the four main alternatives, there are potential permutations such as equalization, headworks screening and conveyance alternatives, process alternatives, and alternative solids treatment/biosolids management options that are discussed below. Selection of the recommended alternative is provided in Chapter 9.

ALTERNATIVE 1 – EXISTING REGIONAL PARTNERS ON EXISTING SITE

This alternative includes upgrades to the existing WWTP to provide sufficient capacity to serve Aberdeen and the existing regional partners (Cosmopolis and SCCC) for the next 20 years and beyond, and to address issues identified in the Condition Assessment.

The current Aberdeen biological treatment process, the Modified Ludzack-Ettinger (MLE) process, is capable of removing ammonia and nitrogen to low levels, and can meet the nitrification needs. The MLE process is a modified conventional activated sludge process in which an anoxic (low-oxygen) zone is configured upstream of the aerobic zone. An internal recycle pump system returns nitrate-rich mixed liquor created through nitrification in the aerobic zone where it is mixed with the influent in the anoxic zone, where denitrification (conversion of nitrate to nitrogen gas, which exits the system) occurs. In the Aberdeen WWTP, like many plants with design flows > 5 mgd, primary clarification is provided upstream of the MLE process.

There are several feasible alternatives to the primary clarification/MLE combination in the future, including:

1. Extended Aeration

   Extended Aeration is a type of activated sludge process with no primary clarification and a longer aerobic detention time. Although this process can be more economical for smaller flows and can generate less waste sludge overall, it is not favored for Aberdeen with Existing or Expanded Regional Partners, due to the presence of useful existing primary clarification/MLE infrastructure, and the fact that primary clarification/MLE will pair better with anaerobic digestion.

2. Integrated Fixed Film Activated Sludge

   Integrated Fixed Film Activated Sludge (IFAS) systems add fixed or free floating media to an activated sludge basin to increase the amount of biomass and enhance the treatment process. Since the attached biomass is retained in the activated sludge basin, and not sent to the clarifiers, use of IFAS technology can increase the capacity of the activated sludge system in the same tank volume, and thus can be a good option when available space is limited. Although the benefits can be significant, IFAS systems also come with several physical requirements that can add capital and operating costs, including additional mixing, aeration, effluent screening and foam mitigation. For these reasons, and the presence of useful existing primary clarification / MLE infrastructure, and adjacent land, IFAS is not favored for the Aberdeen WWTP.

3. Membrane Bioreactors

Membrane bioreactors for wastewater treatment use a combination of a suspended growth biological treatment method, usually activated sludge, with membrane filtration equipment, typically low-pressure microfiltration (MF) or ultrafiltration (UF) membranes. The membranes are immersed in the wastewater and are used, instead of secondary clarifiers, to perform the critical solid-liquid separation function. MBRs typically are used when there is a need to generate very high quality effluent, such as for water reuse, or, when available space is limited.

Based on the need for ammonia removal, the flows involved, and the presence of existing facilities on the Aberdeen WWTP site, and the additional adjacent space, it is recommended that primary clarification and the MLE process be continued in the future.

- For liquid stream treatment, the major processes include screening, primary clarification, grit removal from primary sludge, conventional activated sludge treatment with multizone aeration basins in a Modified Ludzack-Ettinger (MLE) configuration, secondary clarification, and chlorination.

- For solids treatment, the major processes include sludge thickening, anaerobic digestion, and biosolids dewatering. As discussed in the *Regional Biosolids Management Alternatives Evaluation* in Appendix Q, it is recommended that sludge thickening, anaerobic digestion, and biosolids dewatering be continued in the future.

As described later in this chapter, similar processes are also assumed for the new site alternatives.

The following section provides a discussion of the infrastructure that needs to be upgraded, alternatives considered and the recommended Capital Improvement Plan projects to treat planning year 2038 flows and loadings with existing regional partners.

INFLUENT SCREENING AND CONVEYANCE

A *City of Aberdeen WWTP Influent Screening and Conveyance Improvements Engineering Report,* was completed in December 2019 and is provided as Appendix R. The Engineering Report includes an evaluation of alternatives for improvements to the headworks screens, influent pump station and primary sludge pump room. (The primary sludge pump room is located below the headworks and includes electrical and controls for the headworks, so it should be upgraded with the headworks.)

As noted in the Engineering Report, the influent screening and conveyance improvements were identified in the *2018 Aberdeen WWTP and Collection System Condition*

Assessment as high priority needs based on both their condition and importance ratings. The overall goals of the proposed improvements are to:

1. Eliminate overflows of untreated wastewater to the Chehalis River estuary and shellfish-growing areas in Grays Harbor, and protect key equipment from the risk of flooding.

2. Rehabilitate the aging WWTP Influent Screening and Conveyance facilities and to increase the capacity, redundancy and resilience of those facilities to ensure effective, consistent conveyance and screening of influent wastewater.

3. Rehabilitate the influent pump station wet well to repair the corroded and spalled concrete. The project will also provide a durable and reliable corrosion resistant protective coating to prevent degradation in the future.

4. Upgrade the existing pump station wet and dry well ventilation systems to comply with current electrical and ventilation codes and regulations.

5. Flood proof the sludge pump room to prevent the intrusion of floodwaters into the primary sludge and scum pump room and to prevent backwater of the plant drain system into the room. Provide a system to monitor the building floor for flooding and to activate the SCADA alarm. Upgrade piping and pipe supports in the sludge pump room to reduce the risk of process piping system failure.

6. Increase the capacity of Pumps 5 and 6 so that the influent pump station can meet the capacity and reliability needs with the largest pump out of service.

7. Meet the above goals at a reasonable cost to the communities, while ensuring that the upgraded facilities are compatible with future plans and alternatives for the site, including the possibility of accommodating additional regional partners (Hoquiam and Central Park).

In addition, the Influent Screening and Conveyance Improvements will provide the first phase of increasing the firm capacity of the Influent Pump Station.

Headworks Screen System

The existing Headworks equipment (fine screens, screenings conveyor, and screenings washer compactor) was installed in 2005 and has exceeded its useful life. In addition, peak flows to the Headworks exceed the design capacity of the screening facilities. Due to the lack of capacity, reliability and redundancy of the Headworks, not all of the incoming sewage is screened, which is not in compliance with State Biosolids regulations

(WAC-173-308) and Orange Book criteria. (Only three of the five influent pumps convey wastewater to a location *upstream* of the Headworks.) The mechanical bar rack (a backup in case of power outages, and also used for peak flows) does not have sufficient capacity for all the flow with three influent pumps running. In addition, overflow events have occurred, including an event on November 4, 2018 in which an estimated 780,000 gallons of untreated sewage was discharged to the Chehalis River estuary.

The concrete in the existing Headworks structure has some moderate degradation of concrete, necessitating some repair for continued use. Other than that, it is in reasonably good condition and not in need of reconstruction.

As described in the Engineering Report in Appendix R, several alternatives for screening technologies (including step, perforated plate, and multiple rake bar screens) and configurations were evaluated. The current configuration, with two channels with mechanical step screens on either side of a middle channel manual bar screen, was rejected since it has inadequate capacity with one screen out of service, and since the bar screen will not be able accommodate the peak flow when one of the step screens is out of service. As described in the Engineering Report, one proposed configuration of the headworks is three step screens in the existing parallel channels. However, this option does have a serious drawback in that is does not provide a means for screening if there is a power outage, including the emergency power system. Sufficient capacity can also be provided at reduced capital cost and operational complexity with a 2-screen system by raising the walls of the existing headworks upstream of the screens to allow an increase in hydraulic screening capacity (i.e., by adding additional wetted screen surface area larger screens as compared to the existing screens). The improvements would increase the channel depth effectively 2 feet above that in the existing design. This is the preferred option and shown in Figure 8-1.

The new system would have two separate system control panels for the screens, and separate conveyors and washer compactors, for improved reliability and redundancy. Washing and compaction of removed screenings are critical functions of the screening process especially for fine screens. Washing removes organic material from the screenings, which is returned to the wastewater flow. Compaction reduces the volume of screenings, thereby reducing the costs of storage and disposal. Two conveyors will be constructed with sluice gate controlling the opening to each of the washer/compactor unit.

The upgraded screening system will have increased hydraulic capacity, since it will provide additional wetted area with 3/8-inch openings compared to the 1/4-inch openings of the existing screens. The upgrade will provide sufficient capacity for maximum day flows with one screen out of service.

As shown in a schematic of the proposed screening configuration in Figure 8-2, the ability to bypass the screens will be provided in case of an interruption of power to the screening system. Both new side-channel screens would be installed 10 feet in front of the new middle channel manually raked bar screen to provide adequate weir length for screening bypass through the middle channel to each of the side channels in case of a power failure. The perimeter walls of the headworks would need to be raised 2 feet to provide the additional necessary freeboard.

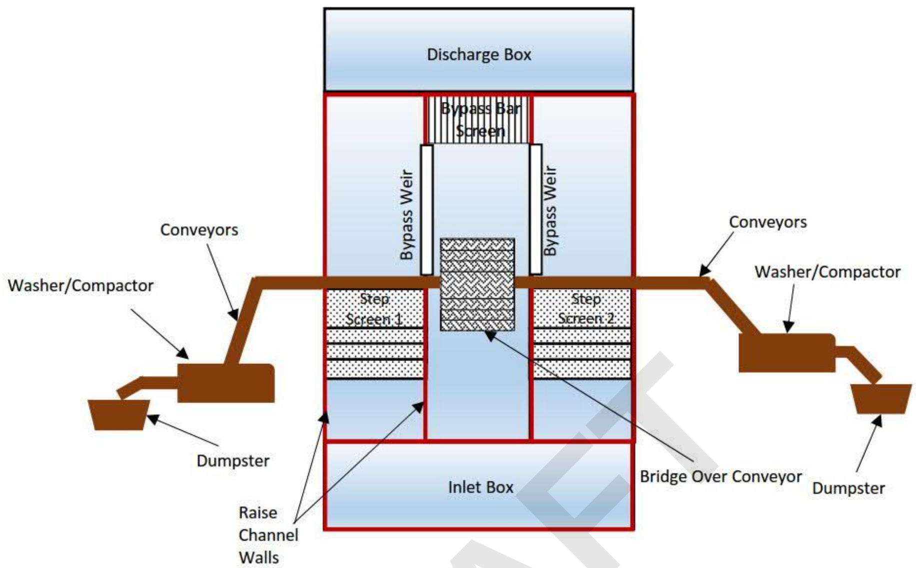

FIGURE 8-1

Schematic of the Upgraded Headworks

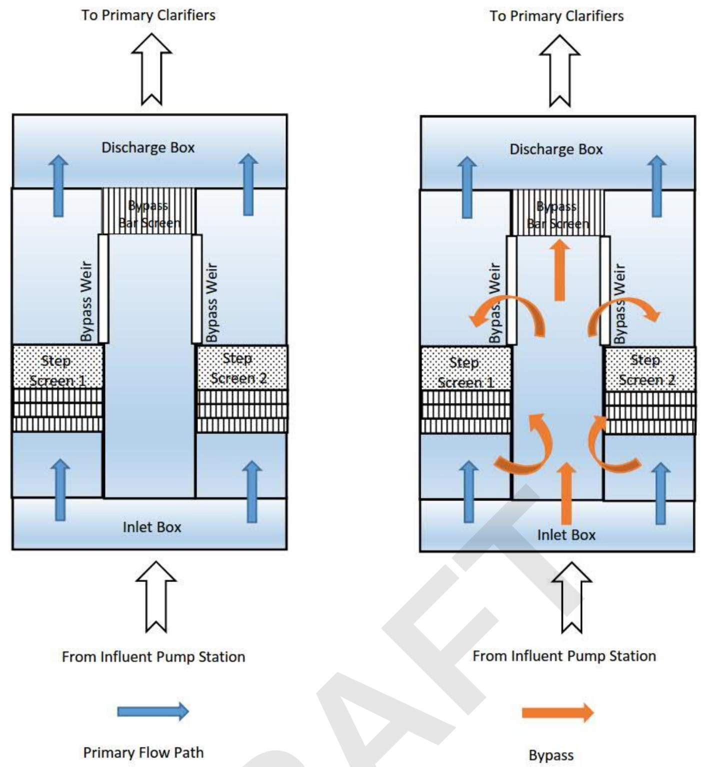

FIGURE 8-2

Flow Schematic of the Upgraded Headworks (Normal Operation on the Left and Bypass on the Right, Conveyors and Washer/Compactors Not Shown for Clarity)

Primary Sludge Pump Room

The Primary Sludge Pump Room, in the lower level of the Headworks structure, houses some electrical/controls for the headworks as well as for the sludge pumps. This room is below the 100-year flood elevation, and has flooded due to (1) several piping failures caused by old and inadequately supported or restrained piping; and (2) overflows of the Headworks above caused by failure of the existing screens. In addition, the fact that only a gravity drain, instead of sump pumps, is present for the room, allows the back up of floodwater (whether it emanates from an overflow or river flood) into the room. The sludge pump room is a classified space and most of the electrical installation in the room is not in compliance with the National Electric Code. The existing ventilation system is not functional and needs to be replaced with a code-compliant system. This room needs to be flood-proofed and brought into compliance with the NEC, NFPA and the City Flood Management codes.

Another issue is that only one primary sludge pump is capable of conveying flow to the hydrogritter. The other two pumps (Penn Valley Pumps) are used to convey scum and thickened sludge from the gravity thickener to the digester normally. Both the scum and primary sludge pumps operate on interval timers.

As described in the Engineering Report in Appendix R, several alternatives were considered to upgrade the Primary Sludge Pump Room. The proposed improvements will include flood proofing of the access doorway to the pump room to prevent intrusion of floodwaters into the primary sludge pump room and to prevent backwater of plant drain system into the room. In addition, a monitoring system would be installed to monitor the building floor for flooding and to activate a SCADA alarm. Upgrades of piping and pipe supports in the sludge pump room to eliminate process piping system failures are also recommended for the improvements.

This project will also be designed for compliance with the National Electrical Code (NEC) and ventilation requirements of NFPA 820. The HVAC should be installed with a 500-cfm supply/exhaust system and a 10-kW duct heater. The replacement NEMA MCC control panel must be positioned at least 3 feet above the flood elevation of 14.7 ft in NAVD 88.

Influent Pump Station

The Influent Pump Station was constructed in the 1950s, and the wet well shows signs of significant deterioration of the concrete, as evidenced by significant spalling of the concrete in the well. For continued use, the wet well must be rehabilitated, and the pump station needs to be brought into electrical and ventilation code compliance. The pump station currently has an inadequate ventilation system that is not code-compliant, and has no odor control, despite the fact that the station sits directly below the WWTP Administration Building. In addition, there are two force mains that convey sewage from the pump station to a location downstream of the influent screens, leading the unscreened raw sewage into the primary treatment process. One of these force mains conveys sewage from a large submersible pump installed in a manhole upstream of the Influent Pump Station in about 2000. The other forcemain conveys flow from an additional pump installed in the dry well of the Influent Pump Station.

As discussed in Chapter 7, the full capacity of 21.9 mgd of the existing influent pump station is sufficient to convey the projected influent sewage flow from the Aberdeen and its current partners (21.97 mgd) within the margin of error for the analysis. However, this analysis was conducted with the assumption that all pumps are in service; with the assumption that the largest pump is not in service, as required by Department of Ecology and EPA reliability criteria, the Influent Pump Station capacity is insufficient and would have a reliable capacity of 15.3 mgd. As discussed in Chapter 7, it is recommended that the capacity of the influent pump station be increased incrementally, by replacing Pumps 4 and 5 with 4,000 gpm pumps to bring the total Influent Pump Station capacity to 22 mgd. After completion of the Northshore Levee project, influent flows should be further evaluated. The capacity of the electrical equipment will be increased near-term; however, the two pumps will be upsized to provide additional capacity in subsequent years as they are replaced.

As described in the Engineering Report in Appendix R, several alternatives were considered to upgrade the Influent Pump Station. The proposed improvements to the Influent Pump Station include replacement of existing access hatches for the wet well, and repair of the corroded and spalled concrete and installation of a durable and reliable corrosion resistant protective coating to prevent degradation in the future.

To comply with the ventilation requirement of NFPA 820, the wet well and the wet well access room will be upgraded with a 2,100-cfm exhaust fan system and a gravity supply vent. The wetwell will be provided with new gas-tight access hatches for separation from the adjacent administration area.

The Influent Pump Station dry well will be provided with a larger gas-tight access hatch that will allow the largest pump to be removed and replaced without major pump disassembly. To comply with NFPA 820, the ventilation system for the dry well will be upgraded with a 1,100-cfm supply fan, 20 kW duct heater, a 1,200-cfm exhaust fan, and a gas-tight separation for the entrance to the drywell.

TABLE 8-1

Design Criteria – Recommended WWTP Influent Screening and Conveyance Improvements

Design Criteria	Value
Headworks	
Influent In-Channel Fine Screens	
No.	2
Channel Dimensions, each	3-ft width (Depth increased 2 feet from existing)
Type of Screens	In Channel Step Screen, 3/8" openings
Capacity, each	12 mgd
Grit Washer/Compactor	
No.	2
Drive motor	5 hp
Influent Pump Station	
Ventilation System Capacity	
Drywell	1100 cfm supply and 1200 cfm exhaust, based on six air changes per hour
Wetwell and Access Room	2100 cfm, based on 12 air changes per hour
Primary Sludge Pump Room	
Ventilation System Capacity	500 cfm, based on six air changes per hour

Recommended Improvements

Detailed cost estimates for the proposed improvements are provided in Appendix L; the cost estimates are summarized in Table 8-2.

TABLE 8-2

WWTP Influent Screening and Conveyance Improvements – Cost Estimate Summary

Improvements	Capital Cost ($)
Screening Improvements	$2,528,000
Primary Sludge Pump Room Improvements	$1,242,000
Influent Pump Station Improvements	$2.967,000

PRIMARY TREATMENT

As indicated in Chapter 7, the overflow rate for the primary clarifiers is near the typical design limit during peak hour flows from Aberdeen and Existing Partners. However, given the expected reduced peak flow with completion of the Northshore Levee project (as well as the proposed offline storage equalization, if needed) the primary clarifier capacity is considered adequate for the planning year. However, the existing sludge and scum collection mechanisms for the two clarifiers are almost 40 years old and are at the end of the useful life, and need to be replaced.

Excess screened raw sewage flow from the headworks would be conveyed to a new 65-foot diameter circular storage tank with 0.3 MG storage capacity. It is sized to maintain a peak flow less than 21.4 mgd to prevent the flow overtopping the clarifier weirs during the peak flow event. Diverted flow would be returned to the IPS by gravity after the peak flow event has passed. (Given the reduction in flooding expected with the completion of the $75 million Northshore Levee project, this project has been scheduled for after the completion of the project. It is possible that this project will not be necessary if robust flow reduction is provided.)

Recommended Improvements

Table 8-3 presents an estimate cost of the recommended improvements.

TABLE 8-3

Primary Treatment Improvements – Cost Estimate Summary

Improvements	Capital Cost ($)
New Offline Equalization Tank	3,017,000
Primary Clarifier Mechanism Replacement	1,820,000

ACTIVATED SLUDGE TREATMENT

For biological treatment without nitrification, the projected flows and loadings are expected to reach the capacity of the existing activated sludge system (aeration basins and blowers) by the end of the planning period, with all units operating. It is recommended that a new aeration basin be constructed by 2038, for the following reasons:

1. To maintain desirable solids retention time for ammonia removal, improve BOD removal efficiency during high flow events and to enhance solids settleability.
2. For added reliability/redundancy so one aeration tank can be taken out of service during the summer months for maintenance, with the WWTP still able to effectively remove ammonia.

The existing aeration basins show signs of concrete degradation and corrosion during the condition assessment. In addition, the air diffuser membranes are nearing the end of useful life and due for replacement in the next few years.

Recommended Improvements

The near-term improvements include aeration basin rehabilitation including epoxy injection, concrete spalling repair and coating of the internal surface of the tanks, replacement of electrical/controls and instrumentation, submersible mixers and membrane diffusers. In addition, the existing tank submersible mixers are nearly 15 years old so they will likely need to be replaced in the next 5 to 10 years.

The recommended longer-term improvements include the construction of one additional aeration basin including associated basin equipment (air diffusers, air supply piping and valves, air flow meters, dissolved oxygen meters, internal recycle pump and pipe, submersible mixers), new air supply header pipe from the blowers, and expansion of the aeration basin splitter influent box. In addition, additional blowers with 6,060 cfm capacity are recommended to increase the aeration system firm capacity to 9,710 scfm to meet the needs for peak day ammonia removal (i.e., nitrification).

Table 8-4 presents an estimated cost of the recommended improvements.

TABLE 8-4

Activated Sludge Treatment Improvements - Cost Estimate Summary

Improvements	Capital Cost ($)
Rehabilitate Two Existing Aeration Basins(1)	2,141,000
New Aeration Basin with Blowers	6,388,000

(1) Diffuser membrane and other mechanical equipment replacement cost after 10 years is assumed to be covered by the O&M budget and not included here.

SECONDARY CLARIFICATION

As with the secondary clarifiers, it is projected that the peak hour overflow rate will actually be reduced within the 20-year planning period with completion of the Levee project and equalization storage tank.

The mechanisms of the two small secondary clarifiers are nearing 40 years of service, and exhibiting signs of corrosion, and are due for replacement. In addition, the concrete is showing significant deterioration.

Recommended Improvements

It is recommended that the City replace the clarifier mechanisms for each of the existing 85-foot diameter secondary clarifiers and rehabilitate the tanks in the near future. The work will include surface preparation, floor grouting and field coating for the tanks, and replacement of aging electrical and controls.

The existing 100-foot diameter secondary clarifier is approximately 15 years old and will be due for mechanism replacement and concrete rehabilitation toward the end of the 20-year planning period.

Table 8-5 presents the estimated cost of the recommended improvements.

TABLE 8-5

Secondary Clarification Improvements - Cost Estimate Summary

Improvements	Capital Cost ($)
Small Secondary Clarifier Improvements	3,056,000
Large Secondary Clarifier Improvements	2,294,000

DISINFECTION SYSTEM

The existing gas-based chlorination and dechlorination systems are in the process of being replaced with new systems using liquid sodium hypochlorite and sodium thiosulfate. The project will be completed in 2020. The systems will provide ample capacity throughout the planning period.

As discussed in Chapter 7, the chlorine contact tank peak detention time is below the Orange Book criterion. No modifications are recommended, however, as the system is meeting disinfection limits at current peak flows, and future peak flows are projected to be reduced through I/I rehabilitation, the Levee project, and if implemented, flow equalization.

EFFLUENT FLOW MEASUREMENT AND OUTFALL

The existing effluent flumes are subject to submergence at high flows and tides which causes the metering system to overstate the actual effluent flow. The City's experience is that 18 mgd is the maximum rate of flow that they can reliably use the effluent flume to determine the plant flow. Above 18 mgd, the influent flow meter reading is used because the effluent flow meter is not considered to be reliable. The effluent flow meter is likely reliable above 18 mgd if the flume is not submerged due to tidal influence.

The recommended improvement would be to use two-point measurement on each flume so the effect of flume submergence can be taken into account when determining the actual flow rate through the flume. The problem with using the influent flow meters is that all of the plant recycle flow is included in the influent flow measurement; consequently, measurement of the pumped flow from the influent pump station will always over-state the actual effluent flow.

The existing outfall has adequate capacity throughout the planning period.

SOLIDS PROCESS

An analysis of the WWTP Biosolids Treatment and Management is provided in Appendix Q. As discussed in Chapter 7, there is adequate capacity through the planning year for gravity sludge thickener (GST), anaerobic digester and dewatering screw press. The City normally operates the GST for thickening comingled primary and waste activated sludge. If waste activated sludge is sent to the rotary drum thickener (RDT) for treatment, the lighter, solely primary sludge loading to the GST would improve its thickening efficiency. Similar to the primary and secondary clarifiers, the mechanism of the GST is approaching the end of its useful life and due for a replacement, and the tank needs to be rehabilitated.

As discussed in the Condition Assessment and the memo in Appendix Q, the existing anaerobic digester is aging and requires major structural, mechanical and electrical rehabilitation. In addition, a backup anaerobic digester is recommended for redundancy to keep up the stabilization process when the existing digester is taken out of service for maintenance.

The backup anaerobic digester improvement could be a newly constructed digester with the similar equipment as the existing one (tank mixer, sludge recirculation pump and piping, sludge/water heat exchanger, instrumentation).

Alternatively, the existing 30-foot diameter sludge storage tanks constructed in the 1950s as anaerobic digesters, could be rehabilitated for use as digesters. However, based on the age and condition of the existing tanks, the construction of a new digester is listed as the recommended improvement in this chapter.

Recommended Improvements

The following recommendations are provided for the solids handling system, as discussed in the memo in Appendix Q,

1. Send waste activated sludge directly to the existing RDT for solids thickening treatment.
2. Rehabilitate the GST tank and replace the mechanism and electrical, which are at the end of their useful life.
3. Rehabilitate the existing 50-foot diameter anaerobic digester structure.
4. Construct a new 50-foot diameter anaerobic digester
5. Rehabilitate the existing anaerobic digester.

Table 8-6 presents an estimate cost of the recommended improvements.

TABLE 8-6

Solids Process Improvements - Cost Estimate

Alternative	Capital Cost ($)
Gravity Sludge Thickener Mechanism Replacement	1,375,000
New Anaerobic Digester and Control Building	13,500,000
Rehabilitate Existing Anaerobic Digester	2,608,000

ELECTRICAL

The existing 500-kW generator is in good condition, but it has inadequate capacity for the entire WWTP, as it only serves the primary treatment and disinfection facilities. The standby power is insufficient to power the two of the 125-hp aeration blowers for the secondary treatment process.

Recommended Improvements

Upgrade standby power generation facilities by installing a 1000-kW generator.

Table 8-7 presents an estimate cost of the recommended improvement.

TABLE 8-7

Electrical Improvement Cost Estimate

Alternative	Capital Cost ($)
New 1000 kw Generator	3,146,000

MISCELLANEOUS

The soils at the WWTP are poor, and facilities that are not pile-supported are settling, creating differential settlement versus the pile-supported structures. This has caused conduit and electrical and control wires to be broken and process piping to fail in the past, requiring emergency remediation. In addition, some of the process piping is also at risk of failure due to age and wear. One of the forcemains from the lead influent pump was recently replaced due to erosion from grit. It is expected that the primary sludge pumping between the primary sludge pump room and the hydrocyclone could be next on the list for catastrophic failure since it can be expected to contain a high concentration of grit. The original design did not provide for differential settlement.

As noted earlier, several of the structures at the WWTP are vulnerable to flooding and not in compliance with City code. City code requires critical facilities to be protected to 3 feet above Base Flood Elevation BFE but only 0.75 ft of freeboard is provided for some of the facilities. Flood protection has been included in the estimates for several of the facilities that are receiving upgrades mentioned above. In addition, the chlorination/sludge pumping building will need to be flood-proofed.

Recommended Improvements

It is recommended that deteriorated and settled piping and conduit and associated wiring be rehabilitated to reduce the risk of failure which can cause consequential damage many times the value of the cost of timely and controlled rehabilitation. In addition, it is recommended that critical facilities be flood-proofed.

Table 8-8 presents an estimate cost of the recommended improvements.

TABLE 8-8

Site Improvements Cost Estimate

Alternative	Capital Cost ($)
Remediation of Deteriorated and Settled Conduit and Process Piping, and Flood-Proofing of Additional Critical Facilities	2,000,000

SUMMARY

Table 8-9 summarizes the 8-year capital improvement program to upgrade the WWTP. Table 8-10 summarizes the additional improvements that will be needed within the next 20 years. Figure 8-3 shows the recommended near-term improvements in orange and those improvements recommended for 2028 to 2038 in green. The capital costs provided are total project costs inclusive of contingency (30 percent), sales tax (8.93 percent), engineering (13 percent), construction administration (12 percent), and legal, City administration, and permitting (5 percent).

The majority of the capital improvement projects within the 20-year period are driven by condition, age, reliability, and/or redundancy.

Based on an examination of the City's sewer department budget, existing annual operation and maintenance costs for the WWTP are estimated to be $2,873,000. It is expected that the implementation of more consistent nitrification and the addition of the recommended additional infrastructure (an additional digester and aeration basin) within the next 20 years will increase (in 2019 dollars) annual operation and maintenance costs to $3,190,000.

TABLE 8-9

Alternative 1 – Existing Partners on Existing Site - Capital Improvement Plan, 2019 – 2027

Design Year/ Construction Year	CIP ID	Project	Components	Justification	Projected Total Project Cost (2019 $)
Current Projects 2019/2020	WW-1	Influent Pump Station – Incremental Upgrade	Replace pump	Age, condition, capacity	$50,000
	WW-2	Influent Pump Station – Incremental Upgrade	Replace VFDs	Age, condition, capacity	$67,500
	WW-3	Disinfection Improvements	Convert to liquid chlorination/dechlorination, Rehabilitate process water system	Age, condition, safety	$2,500,000 (under construction)
Phase 1 2021/2022	WW-4	New WWTP Generator	New generator, switchgear	Age, condition, capacity, reliability	$3,146,000
	WW-5	Influent Pump Station	Rehabilitate wet well, structural improvements, ventilation compliance	Age, condition, safety, capacity needs	$2,967,000
	WW-6	Rehabilitate Existing Digester	Fix roof, Replace gas lines, heat exchanger, boiler, Electrical code upgrades	Age, condition, safety	$2,608,000
	WW-7	Primary Sludge Pump Room	Electrical and controls, Ventilation compliance, Process piping Improvements, Flood hazard mitigation	Safety, reliability	$1,242,000
	WW-8	Aeration Basins	Miscellaneous structural, mechanical and electrical improvements, including tank surface rehabilitation, remediate settling of yard piping and electrical raceways	Age, condition, Avoid potential failure due to conduit settlement	$2,141,000
	WW-9	Headworks Screens Replacement	New screens and washer compactors, raise walls, Modify stairway access, Electrical improvements	Age, condition, capacity needs.	$2,528,000

TABLE 8-9 – (continued)

Alternative 1 – Existing Partners on Existing Site - Capital Improvement Plan, 2019 – 2027

Design Year/ Construction Year	CIP ID	Project	Components	Justification	Projected Total Project Cost (2019 $)
Phase 2 2023/2024	WW-10	Secondary Clarifier No 1	Replace mechanisms, equipment, Surface rehabilitation, remediate settling of yard piping and electrical raceways,	Age, condition	$1,528,000
	WW-11	Gravity Sludge Thickener Upgrade	Replace mechanisms and equipment, Surface rehabilitation, remediate settling of yard piping and electrical raceways, Replace yard piping	Age, condition	$1,375,000
	WW-12	Throughout WWTP Site	Remediation of settled conduit and process piping	Potential failure due to settlement	$2,000,000
Phase 3 2025/2026	WW-13	East Primary Clarifier	Replace mechanisms, equipment, Surface rehabilitation, remediate settling of yard piping and electrical raceways	Age, condition	$910,000
	WW-14	New Offline Equalization Tank (if required)	65-foot diameter storage with submersible pumps and piping	Age, condition	$3,017,000
Phase 4 2026/2027	WW-15	West Primary Clarifier	Replace mechanisms, equipment, surface rehabilitation, address Settling	Age, condition	$910,000
	WW-16	Construct Additional Digester	Additional digester with all appurtenances, and digester control building	Reliability	$13,500,000
	WW-17	Secondary Clarifier 2	Replace mechanisms, equipment, Surface rehabilitation, address settling	Age, condition	$1,528,000
Total (2019 to 2027)					**$39,400,000[(1)]**

(1) Excludes current projects WW1 through WW3.

TABLE 8-10

Alternative 1 – Existing Partners on Existing Site Capital Improvement Plan, 2028 – 2038

Design Year/ Construction Year	CIP ID	Project	Components	Justification	Projected Total Project Cost (2019 $)
Phase 5 2028 to 2038	WW-18	New Aeration Basin	Construct one new 0.47 MG basin and 1,390 cfm blower Equipment replacement at existing basins: diffuser membranes, mixers, recycle pumps	Additional capacity needed, especially for nitrification	\$6,388,000 \$1,566,000(1)
	WW-19	Large 100-Foot Secondary Clarifier Rehabilitation	Replace mechanisms, equipment, surface rehabilitation, Construct launder trough cover	Age, condition	\$2,294,000
	WW-20	Disinfection System	Replace chemical dosing pumps	Age, condition	\$120,000(1)
	WW-21	Rotary Thickener	Replace equipment	Age, condition	\$150,000(1)
	WW-22	Solids Dewatering	Replace feed pumps, polymer system, drive motor, VFDs	Age, condition	\$150,000(1)
Total (2028 to 2038)					**\$10,668,000**

(1) To be conservative, the equipment replacement cost is included in the capital improvement cost summary. However, it could be covered by the plant O&M budget.

ALTERNATIVE 2 – EXPANDED REGIONAL PARTNERS ON EXISTING SITE

For this alternative, the existing Aberdeen WWTP would be upgraded to serve Aberdeen, existing partners (Cosmopolis, SCCC), and additional regional partners (Hoquiam and Central Park) through the planning year and beyond. This alternative is shown on Figure 8-4, and described with costs in Table 8-10. Treating Hoquiam and Central Park flows at the Aberdeen WWTP will necessitate the construction of new facilities beyond those required for Alternative 1 including:

- A new (second) headworks dedicated to screening and flow measurement for Hoquiam's flows (Central Park, Aberdeen, and Existing Partner flows would continue to be screened and measured with the existing upgraded headworks)
- An additional 65-foot primary clarifier
- An additional 0.47 MG aeration basin and 1,390 cfm blower
- An additional 100-foot secondary clarifier (or expansion of the existing 85-foot secondary clarifiers to 100 feet)
- Site and piping modifications
- Additional chlorine contact tank capacity
- Effluent pumping and minor outfall improvements

New infrastructure required to treat additional flows from Hoquiam and Central Park is shown in blue on Figure 8-4 and in Table 8-11. (Also, as on Figure 8-4, the recommended near-term improvements *required irrespective of acceptance of the Hoquiam and Central Park flows* through 2019 to 2027 are shown in orange and those improvements recommended for 2028 to 2038 are shown in green.)

Table 8-10 also includes a potential capital cost apportionment scenario for discussion purposes. In Table 8-10, costs are split based on projected peak day flows. There are several legitimate methods to allocate costs for wastewater regionalization and cost allocation should be further evaluated if Aberdeen, Hoquiam, and/or Central Park feel that regionalization merits more detailed consideration. Facilities driven solely by treatment of Hoquiam and Central Park flows are apportioned between those two entities based on flow projections. Other improvements (*not* including those currently under design/construction, such as the current disinfection improvements and influent pump station incremental upgrades) are apportioned between Hoquiam, Central Park, and Aberdeen and Existing Partners. Based on this approach, the capital cost breakdown is:

- Hoquiam: $38,642,000
- Central Park: $5,940,000
- Aberdeen and Existing Partners: $35,924,000

Also shown in Figure 8-4 (in purple) are the additional facilities that will be required if an additional industry discharging 2 mgd (of assumed domestic strength wastewater) were sited in the region. For purposes of cost estimates, it is assumed that the additional 2 mgd is conveyed to the WWTP through the force main from Hoquiam.

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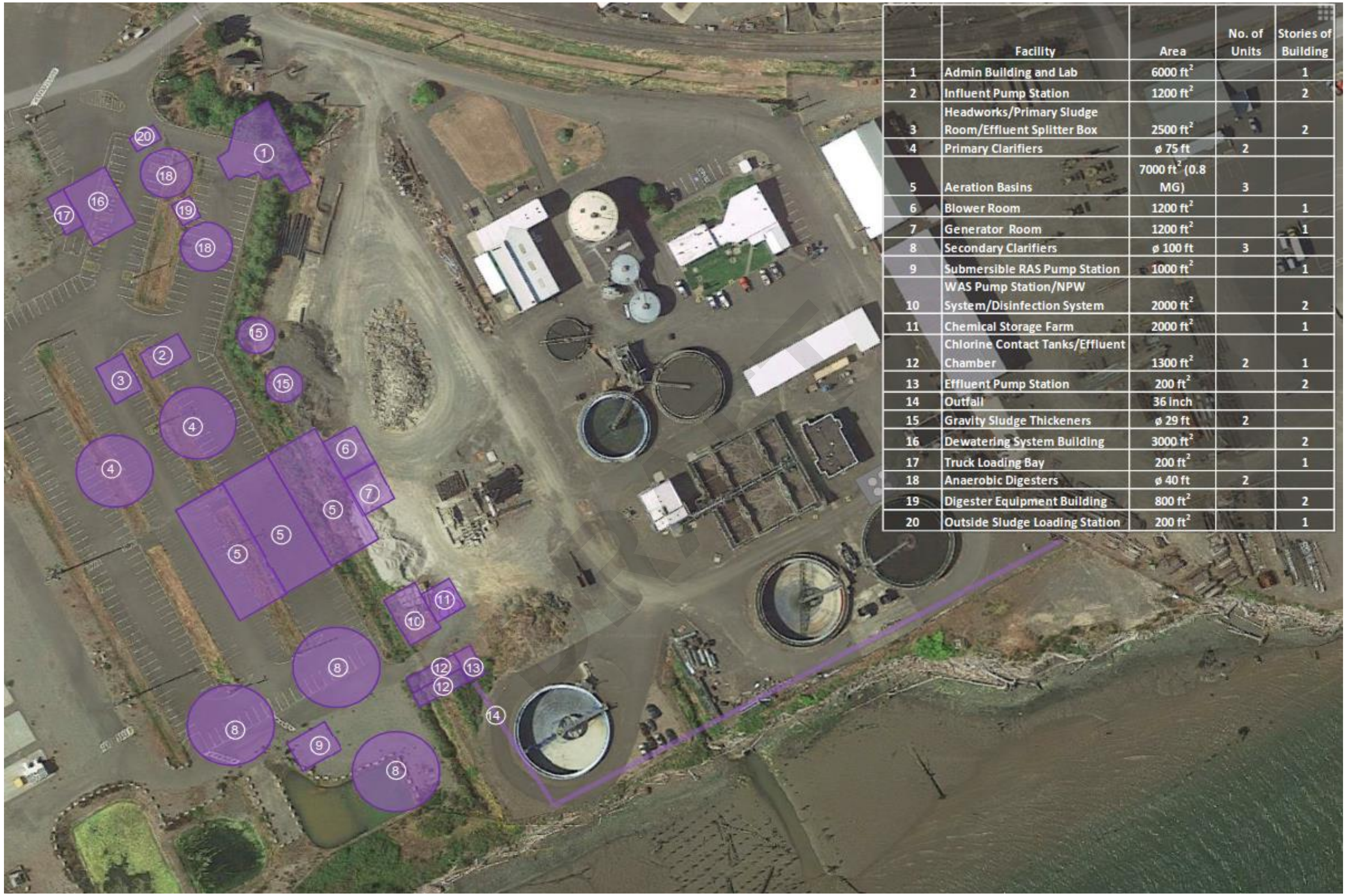
FIGURE 8-3

Alternative 1 – Existing Partners on Existing Site

FIGURE 8-4

Alternative 1 – Expanded Regional Partners on Existing Site



	Facility	Area	No. of Units	Stories of Building
1	Admin Building and Lab	6000 ft^2		1
2	Influent Pump Station	1200 ft^2		2
3	Headworks/Primary Sludge Room/Effluent Splitter Box	2500 ft^2		2
4	Primary Clarifiers	ø 75 ft	2	
5	Aeration Basins	7000 ft^2 (0.8 MG)	3	
6	Blower Room	1200 ft^2		1
7	Generator Room	1200 ft^2		1
8	Secondary Clarifiers	ø 100 ft	3	
9	Submersible RAS Pump Station	1000 ft^2		1
10	WAS Pump Station/NPW System/Disinfection System	2000 ft^2		2
11	Chemical Storage Farm	2000 ft^2		1
12	Chlorine Contact Tanks/Effluent Chamber	1300 ft^2	2	1
13	Effluent Pump Station	200 ft^2		2
14	Outfall	36 inch		
15	Gravity Sludge Thickeners	ø 29 ft	2	
16	Dewatering System Building	3000 ft^2		2
17	Truck Loading Bay	200 ft^2		1
18	Anaerobic Digesters	ø 40 ft	2	
19	Digester Equipment Building	800 ft^2		2
20	Outside Sludge Loading Station	200 ft^2		1

FIGURE 8-5

Alternative 2 – Existing Partners on New Site

	Facility	Area	No. of Units	Stories of Building
1	Admin Building and Lab	8000 ft^2		1
2	Influent Pump Station	1200 ft^2		2
3	Headworks/Primary Sludge Room/Effluent Splitter Box	3000 ft^2		2
4	Primary Clarifiers	ø 60 ft	4	
5	Aeration Basins	10000 ft^2 (1.2 MG)	3	
6	Blower Room	1500 ft^2		1
7	Generator Room	1500 ft^2		1
8	Secondary Clarifiers	ø 100 ft	4	
9	Submersible RAS Pump Station	1500 ft^2		1
10	WAS Pump Station/NPW System/Disinfection System	3000 ft^2		2
11	Chemical Storage Farm	2000 ft^2		1
12	Chlorine Contact Tanks/Effluent Chamber	1800 ft^2	2	1
13	Effluent Pump Station	200 ft^2		2
14	Outfall	48 inch		
15	Gravity Sludge Thickeners	ø 35 ft	2	
16	Dewatering System Building	3600 ft^2		2
17	Truck Loading Bay	200 ft^2		1
18	Anaerobic Digesters	ø 45 ft	2	
19	Digester Equipment Building	1000 ft^2		2
20	Outside Sludge Loading Station	200 ft^2		1

FIGURE 8-6

Alternative 4 – Expanded Partners on New Site

TABLE 8-11

Alternative 2 – Expanded Partners on Existing Site

Description	New Facilities Driven by Expanded Regional Partners		2020 to 2027 Capital Improvement Plan		2027 to 2038 Capital Improvement Plan	
	Capital Cost (2019 $)	Description	Capital Cost (2019 $)	Description	Capital Cost (2019 $)	Description
Equalization		No additional	$3,017,000	New offline equalization tank		
Influent Pump Station		In Hoquiam	$2,967,000	Structural improvements and ventilation compliance		
Headworks	$5,280,000	Construct separate screen system at Aberdeen for 6.5 mgd	$3,770,000	Replace screens Headworks structure upgrade raise channel walls Rehabilitate primary sludge room		
Primary Clarifier	$5,182,000	Construct one new 65 ft diameter clarifier unit	$1,820,000	Replace mechanisms, equipment, surface rehab.		
Activated Sludge	$6,695,000	Construct one new 0.47 MG aeration basin and 1,390 cfm blower	$2,141,000	Miscellaneous structural improvements and replace diffuser membrane	$7,954,000	Construct one new 0.47 MG basin and 1,390 cfm blower and equipment replacement at existing basins: diffuser membrane, mixers, recycle pumps
Secondary Clarifiers	$1,903,000	Demolish existing chlorine contact tanks circling secondary clarifiers, convert existing 85 ft diameter secondary clarifiers to 100 ft secondary clarifiers	$3,056,000	At two existing 85 ft. diameter secondary clarifiers: replace mechanisms, equipment, surface rehab., and construct launder trough cover	$2,294,000	At existing 100 ft. diameter clarifier: replace mechanisms, equipment, surface rehab., and construct launder trough cover
Chlorination Disinfection	$3,228,000	Add 0.45 MG contact tank			$120,000	Replace chemical dosing pumps every 10 years
Gravity Sludge Thickener	$2,104,000	Additional Gravity Sludge Thickener	$1,375,000	Replace mechanisms, equipment, surface rehab.		
Rotary Screen Thickener		No additional			$150,000	Replace equipment
Anaerobic Digestion		No additional	$16,108,000	Structural improvements and replace equipment at existing digester Construct additional digester		
Solids Dewatering		No additional			$150,000	Replace feed pumps, polymer system, drive motor, VFD
Administration Building and Laboratory		No additional				
Sitework	$3,976,000	Based on Hoquiam Flow	$2,000,000	Remediation of settled conduit and process piping		

TABLE 8-11 – (continued)

Alternative 2 – Expanded Partners on Existing Site

	New Facilities Driven by Expanded Regional Partners		2020 to 2027 Capital Improvement Plan		2027 to 2038 Capital Improvement Plan	
Description	**Capital Cost (2019 $)**	**Description**	**Capital Cost (2019 $)**	**Description**	**Capital Cost (2019 $)**	**Description**
Generator		No additional	$3,146,000	New upsized generator and switchgear		
Outfall	$2,075,000	Effluent Pumping: 20 hp for 30 mgd pumping @ 2 ft TDH, and outfall extension				
TOTAL PROJECT COSTS	**$30,440,000**		**$39,400,000**		**$10,668,000**	
Allocation to Hoquiam	87%	Hoquiam/Total MDF (6.5/7.5) in 2038	24.5%	Hoquiam/Total MDF (6.5/26.55) in 2038	24.5%	Hoquiam/Total MDF (6.5/26.55) in 2038
Allocation to Central Park	13%	Central Park/Total MDF (1/7.5) in 2038	3.8%	Central Park/Total MDF (1/26.55) in 2038	3.8%	Central Park/Total MDF (1/26.55) in 2038
Capital Costs for Hoquiam	**$26,380,000**		**$9,650,000**		**$2,612,000**	
Capital Costs for Central Park	**$4,060,000**		**$1,480,000**		**$400,000**	
Capital Costs for Aberdeen and Existing Partners	**$0**		**$28,270,000**		**$7,660,000**	

It is assumed that two additional FTEs are required to operate the Expanded Regional Plant (Alternative 2) relative to Alternative 1; in addition, there would be increases in all other operation and maintenance costs, but with some economies of scale. The estimated *additional* annual operational and maintenance costs for 2038 for Alternative 2 over Alternative 1 are $751,000, for a total annual O&M cost (for 2038) of $3,941,000.

ALTERNATIVES 3 AND 4 – EXISTING PARTNERS ON NEW SITE AND EXPANDED REGIONAL PARTNERS ON NEW SITE (GREEN FIELD ALTERNATIVES)

For new site (green field) alternatives, preliminary conceptual designs were developed based on industry-standard criteria (including the State's *Criteria for Sewage Works Design*, (Orange Book)) for treatment processes, combined with output from CapdetWorks, a software tool provided by Hydromantis for preliminary design and cost estimation of wastewater treatment plant construction projects.

Table 8-12 summarizes the criteria used to size the facilities, based on Ecology's Orange Book, *Manual of Practice No. 8, Design of Water Resource Recovery Facilities* (WEF), *Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability* (EPA), and engineering judgment.

TABLE 8-12

Alternatives 3 and 4 – Criteria for Green Field WWTPs

Unit Process	Limiting Criterion	Limit	Unit
Primary Sedimentation Tank	Peak Hour Overflow Rate	2,500	gpd/sf
Primary Sedimentation Tank	Maximum Month HRT (minimum)	2.5	hour
Primary Sedimentation Tank	Alternative Unit Serves 50 percent of the PHF with Primary Unit Out of Service		
Aeration Tank	Maximum Month HRT	7	hour
Aeration Tank	Maximum Month SRT (minimum)	8	day
Aeration Tank	Maximum Day Unit BOD Loading	100	ppd/kcf
Secondary Sedimentation Tank	Peak Overflow Rate	1,200	gpd/sf
Secondary Sedimentation Tank	Maximum Month Solids Loading Rate	30	ppd/sf
Secondary Sedimentation Tank	PHF Solids Loading Rate	50	ppd/sf
Secondary Sedimentation Tank	Alternative Unit Serves 50 percent of the PHF with Primary Unit Out of Service		
Chlorine Contact Tank	Peak Hour Contact Time	20	min
Gravity Sludge Thickener	Maximum Solids Loading Rate	24	ppd/sf
Anaerobic Digester	Maximum Month Solids Residence Time	15	day
Anaerobic Digester	Maximum Day Volatile Solids Loading	0.2	lb/cf
Anaerobic Digester	Volatile Solids Reduction (minimum)	38	%

For Alternatives 3 and 4, it is assumed that a completely new WWTP is constructed at a new site consisting of the City's property to the west of the existing WWTP and a portion of the parking lot on the adjacent property. Figures 8-5 and 8-6 show WWTP site layout schematics for Alternatives 3 and 4, respectively, and Tables 8-13 and 8-14 provide estimated capital and operation and maintenance costs.

(Note: It is unknown if the parking lot on the adjacent property is available. Property acquisition costs have not been included in the estimates provided in this memo.)

TABLE 8-13

Alternatives 3 and 4 – Expanded Partners on New Site Capital Cost Estimates

Item	Alternative 3 Existing Partners	Alternative 4 Expanded Regional
Influent Pump Station	$3,980,000	$4,428,000
Headworks	$3,213,000	$4,279,000
Primary Clarifier	$6,031,000	$8,586,000
Activated Sludge	$15,162,000	$21,641,000
Secondary Clarification	$11,903,000	$16,256,000
Disinfection	$2,912,000	$4,303,000
Gravity Sludge Thickener	$1,486,000	$1,813,000
Rotary Screen Thickener	$942,000	$1,056,000
Anaerobic Digestion	$5,760,000	$8,010,000
Solids Dewatering	$4,285,000	$5,186,000
Administration Building and Laboratory	$4,067,000	$5,423,000
Sitework	$4,069,000	$5,439,000
Outfall/Effluent Pumping Improvements	$4,000,000	$5,000,000
Subtotal	**$67,810,000**	**$91,421,000**
Construction Contingencies (30%)	$20,343,000	$27,426,300
Subtotal	**$88,153,000**	**$118,847,300**
Sales Tax (8.9%)	$7,872,000	$10,613,000
Total Estimated Construction Cost	**$96,025,000**	**$129,460,000**
Engineering Services (13%)	$12,483,000	$16,830,000
Construction Administration (12%)	$11,523,000	$15,535,000
Legal, County Administration, Permitting (5%)	$4,801,000	$6,473,000
Total Project Cost	**$124,832,000**	**$ 168,298,000 (1)**

(1) Capital costs without Hoquiam equalization are estimated to be $182,470,000.

TABLE 8-14

Alternatives 3 and 4 – Expanded Partners on New Site WWTP O&M Estimates(1)

Item	Alternative 3 Existing Partners	Alternative 4 Expanded Regional
Operational Labor Cost	$931,000	$1,083,000
Maintenance Labor Cost	$487,000	$564,000
Material and Supply Cost	$1,060,000	$1,441,000
Chemical Cost	$359,000	$490,000
Energy Cost	$318,000	$440,000
Total Annual Cost	**$3,155,000**	**$4,018,000** (2)

(1) Costs are for WWTP operation, maintenance, and administration only and do not include collection system operation, maintenance, and administration costs.

(2) WWTP O&M costs without Hoquiam equalization are estimated to be $4,246,000.

ANTI-DEGRADATION ANALYSIS

Federal regulations (40 CFR 131.12) and the Water Quality Standards for Surface Waters of the State of Washington (WAC 173-201A-300, 310, 320, 330) establish a water quality antidegradation program for surface waters. The federally-mandated program establishes three tiers of protection for water quality. These three tiers function to protect existing and designated in-stream uses, to limit the conditions under which water of a quality higher than the state standards can be degraded, and to provide a means to set the very best waters of the state aside from future sources of degradation entirely. WAC 173-201A-320 contains the Tier II antidegradation provisions for the State's surface water quality standards. Consistent with the federal water quality antidegradation regulations, Washington's Tier II program functions as a pollution-prevention program to provide an extra measure of protection for water quality.

Appendix N provides, along with the Mixing Zone Study, an anti-degradation analysis for the recommendations in this Regional General Sewer/Wastewater Facility Plan. The analysis demonstrate that additional improvements beyond those discussed in this Plan would not be affordable to the communities and that there are no water quality triggers that would cause a reasonable potential to exceed water quality limits. In fact, the improvements will reduce the potential associated for water quality impacts relative to the existing plant.

With the Expanded Regional Partners scenario, the additional loading at the outfall may cause more localized effects around the outfall but not as a whole in regard to Grays Harbor. The conversion of septic tanks in Central Park to sewer service would provide pollutant reductions to Grays Harbor since high groundwater conditions likely lead to discharge of nutrients, organics and pathogens from septic tank drain fields to surface water and shallow groundwater that discharge to Grays Harbor.

REGIONAL BIOSOLIDS COOPERATION

Two memoranda evaluating consolidation of regional solids handling facilities at Aberdeen is provided in Appendix Q. The memorandum evaluates the alternate (called Alternative 5, with subalternatives 5-1, 5-2 and 5-3) to continue to operate separate treatment facilities at Aberdeen and Hoquiam, but to consolidate solids processing at Aberdeen as a potential collective cost savings for both municipalities that could result from the economy of scale for operating a single, larger solids processing facility. Solids processing typically accounts for half the total cost of treating wastewater. The following are several drivers for consolidated solids processing:

1. The Hoquiam WWTP does not currently include dedicated solids processing facilities. Generated solids are now stored temporarily in a facultative sludge lagoon for later harvesting and beneficial use.

2. The two WWTPs are located approximately 5 miles apart, a short distance for transporting solids from Hoquiam to Aberdeen, with probable low costs for solids transfer.

3. The fist memorandum prepared on solids processing alternatives at Aberdeen shows that there are economic advantages of jointly treating Hoquiam and Aberdeen solids. The Aberdeen WWTP uses anaerobic sludge digestion, which is highly effective at stabilizing municipal solids, and also generates usable fuel in the form of biogas. Joint solids processing would provide funds that could be used to replace outdated facilities, and allow the construction of redundant facilities, primarily a second digester, that would promote system reliability. Aberdeen has also contracted for a local site on which to beneficially use biosolids at a relatively low cost.

The second memorandum focuses on the feasibility of conveying solids from Hoquiam to Aberdeen.

SOLIDS CONVEYANCE

The following three solids conveyance alternatives were identified:

- **Alternative 5-1:** truck transport of unthickened sludge from Hoquiam to Aberdeen

- **Alternative 5-2:** truck transport of thickened sludge from Hoquiam to Aberdeen

- **Alternative 5-3:** pumping of unthickened sludge from Hoquiam to Aberdeen

A present-worth cost analysis found that Alternative 5-3, pumping of unthickened sludge from Hoquiam to Aberdeen, had the lowest cost, and also scored highest in the overall evaluation (including economic and non-economic factors).

Among the three sub-alternatives, the most favorable is for a sludge pump station to be sited at the Hoquiam WWTP and a force main be constructed to connect with the Aberdeen sewer system. Unthickened solids would then be transferred from Hoquiam to the Aberdeen collection system. Because dilute sludge would be conveyed, the force main would be a conventional wastewater configuration. The pump station could be equipped with either standard nonclog wastewater pumps or positive-displacement sludge pumps.

More information is provided in Appendix Q. This alternative (and sub-alternatives) will be further compared to Alternatives 1 through 4 in a subsequent draft of this Plan.

ECONOMIC EVALUATION OF ALTERNATIVES

The following four WWTP alternatives are compared and evaluated in this section:

1. Serve Existing Regional Partners on Existing Site
2. Serve Expanded Regional Partners on Existing Site
3. Serve Existing Regional Partners on New Site
4. Serve Expanded Regional Partners on New Site

Table 8-15 summarizes a comparison of projected life cycle costs for the four alternatives, broken down between Aberdeen, Hoquiam and Central Park. The new-site alternatives (Alternatives 3 and 4) are considered to be cost–prohibitive, with a total capital cost and present worth more than double the cost of the existing-site alternatives (Alternatives 1 and 2). Aberdeen and Hoquiam are financially challenged communities; over 50 percent of Aberdeen households are considered low to moderate income. In addition, like many communities, Aberdeen and Hoquiam may suffer from the recession predicted to be caused by the COVID-19 pandemic that may also result in affordability challenges for many of the ratepayers, as well as a possible reduction in available grant funding.

With the elimination of Alternatives 3 and 4, the focus is on a comparison of Alternatives 1 and 2, the existing-site alternatives – whether to expand the Aberdeen WWTP to treat Hoquiam's, and possibly Central Park's, wastewater. Table 8-14 shows that Aberdeen would save over $14 million in capital costs ($50,068,000 vs. $35,924,000) $22 million in 20-year life cycle costs ($102,019,000 vs. $79,289,000) with regionalization, based on the previous assumptions used for cost partitioning. The question, then, is: does Hoquiam have an incentive to regionalize? Aberdeen could potentially share a portion of their savings with Hoquiam (in other words, change the cost partitioning assumptions) to make regionalization more attractive, if necessary. As long as the total combined life cycle costs for the two Cities to regionalize (Alternative 2) is cheaper than the "Go It

Alone" option (Alternative 1), both Cities could potentially share cost savings and have an economic incentive to regionalize.

The 2013 *City of Hoquiam Wastewater Facility Plan* ("*2013 Hoquiam Facility Plan*") completed in 2013 for the City of Hoquiam identified a number of deficiencies at the WWTP, and recommended four phases of improvements. Table 8-16 summarizes the costs of the four phases; the costs have been updated and escalated from 2013 to 2020 using the ENR (Engineering News Record) Construction Cost Index.

TABLE 8-15

Cost Comparison for Alternatives (20-Year Life Cycle)

Alternative	1. Serve Existing Regional Partners on Existing Site	2. Serve Expanded Regional Partners on Existing Site	3. Serve Existing Regional Partners on New Site	4. Serve Expanded Regional Partners on New Site
Total Project Cost (Capital)	$50,068,000	$80,506,000	$165,705,000	$224,077,000
Aberdeen	$50,068,000	$35,924,000	$165,705,000	$155,184,000
Hoquiam	--	$38,642,000	--	$59,847,000
Central Park	--	$5,940,000	--	$9,046,000
O&M Present Worth Cost	$51,951,000	$62,617,000	$46,938,000	$59,778,000
Aberdeen	$51,951,000	$43,365,000	$46,938,000	$41,399,000
Hoquiam	--	$16,724,000	--	$15,966,000
Central Park	--	$2,528,000	--	$2,413,000
Total Present Worth	$102,019,000	$143,123,000	$212,643,000	$283,855,000
Aberdeen	$102,019,000	$79,289,000	$212,643,000	$196,583,000
Hoquiam	--	$55,366,000	--	$75,813,000
Central Park	--	$8,468,000	--	$11,459,000

(1) 3 percent inflation and discount rate used.

TABLE 8-16

City of Hoquiam WWTP
Projected Capital Costs and O&M Costs for "Go It Alone" Option

Implementation Phase	Assumed Year of Implementation	Capital Costs	Annual O&M Cost
Phase 1	2022	$4,880,000	$518,000
Phase 2	2026	$21,400,000	$518,000
Phase 3	2030	$12,870,000	$549,800
Phase 4 - Biosolids	2034	$10,480,000	$870,900
Total		**$49,610,000**	

As shown, in Table 8-17, based on the 2013 *Hoquiam Facility Plan,* Hoquiam is facing $44,960,000 in capital costs for the four phases over the next 20 years. Table 8-17 shows a comparison of the combined Aberdeen/Hoquiam "Go It Alone" capital costs compared to those for regionalization (Alternative 2 WWTP shares for Hoquiam and Aberdeen combined with regional conveyance costs from Hoquiam to Aberdeen). As shown, the total capital costs are **approximately 5 percent less expensive for regionalization**.

TABLE 8-17

Comparison of Capital Costs for Aberdeen and Hoquiam – Alternative 1 vs. Alternative 2

Alternative 1 ("Go It Alone")	Alternative 2 (Regionalization including Hoquiam)
Aberdeen – $50,068,000 WWTP	Regional – $74,166,000 WWTP (Hoquiam and Aberdeen shares)
Hoquiam – $49,610.000 WWTP	Hoquiam – $20,800,000 Conveyance
Total – $99,678,000	**Total – $94,066,000**

All costs are in 2020 dollars and based on planning level cost estimates. Central Park not included.

Table 8-18 summarizes life cycle costs for Alternative 1 ("Go It Alone" for both Cities) versus Alternative 2 (Hoquiam Served Along with Existing Partners at Existing Aberdeen WWTP). For Alternative 2, two options are shown; in the first option (Option 2A), Hoquiam pays for all the Regional Conveyance costs. However, that does not appear to be attractive to Hoquiam, as it would result in a 20-Year Life Cycle for Hoquiam that is significantly more expensive than Alternative 1. The only way that both Cities' life cycle costs are significantly less than the "Go It Alone" option is for Aberdeen to pay for most of the regional conveyance costs. In this "other extreme" (Option 2B), Aberdeen would pay the majority ($14 million) of the conveyance costs, an amount that results in significant 20-Year Life Cycle savings for both Cities (about 5 percent in overall life cycle costs). However, this would also make the capital costs for regionalization more expensive for Aberdeen than the "Go It Alone" option. A more attractive cost partitioning option for regionalization is that the share of both capital and operating costs is adjusted so that both capital and operating costs are lower for each City with regionalization.

It should be noted, however, since constructing the regional conveyance system, and additional new facilities on the Aberdeen WWTP site, would be among the first steps of regionalization, it would result in a significant immediate rate increase, and likely opposition to the project, for one or both Cities. It should be noted that this analysis (and costs presented throughout the *Regional Sewer Plan*) are based on planning level (Class 4 AACE) cost estimates, and actual costs can vary significantly from those provided. In addition, the City of Hoquiam is planning on updating their Facility Plan and "Go It

Alone" costs, so additional information to update the life-cycle analysis should be available in the near future.

TABLE 8-18

20-Year Life Cycle Cost Comparison Alternative 1 vs. Alternative 2
(not including Central Park Costs)

	Alternative 1: “Go It Alone”			Alternative 2: Hoquiam Served Along with Existing Partners at Existing Aberdeen WWTP	
Alternative	**Aberdeen “Go It Alone”: Continue to Serve Existing Regional Partners on Existing Site**	**Hoquiam "Go It Alone"**	**Sum of Aberdeen and Hoquiam "Go It Alone" Costs**	**Hoquiam Pays All Regional Conveyance**	**Aberdeen Pays Majority of Regional Conveyance**
Total Project Cost (Capital)	$50,068,000	$49,610,000	$99,678,000	$94,966,000	$94,966,000
Aberdeen	$50,068,000	$0	$50,068,000	$35,924,000	$52,924,000
Hoquiam	$0	$49,610,000	$49,610,000	$59,042,000	$42,042,000
O&M Present Worth Cost	$51,951,000	$12,252,000	$64,203,000	$60,089,000	$60,089,000
Aberdeen	$51,951,000	$0	$51,951,000	$43,365,000	$43,365,000
Hoquiam	$0	$12,252,000	$12,252,000	$16,724,000	$16,724,000
Total Present Worth (20-Year Life Cycle)	$102,019,000	$61,862,000	$163,881,000	$155,055,000	$155,055,000
Aberdeen	$102,019,000	$0	$102,019,000	$79,289,000	$96,289,000
Hoquiam	$0	$61,862,000	$61,862,000	$75,766,000	$58,766,000

(1) Hoquiam pays all regional conveyance costs.
(2) Aberdeen pays the majority of regional conveyance costs
(3) All costs are in 2020 dollars and are planning level, 3 percent inflation and discount rate used.

It should be noted that this analysis (and costs presented throughout the *Regional Sewer Plan*) are based on planning level (Class 4 AACE) cost estimates, and actual costs can vary significantly from those provided. In addition, the City of Hoquiam is in the process of updating their "Go It Alone" costs, so additional information to update the life-cycle analysis should be available in the near future.

NON-MONETARY COMPARISON

In this section, Alternatives 1 and 2 are compared with regard to non-monetary considerations, including Treatment Process Quality/Adaptability, Public Concerns, Local Control, Risk and Environmental Benefits.

TREATMENT PROCESS QUALITY/ADAPTABILITY

When comparing for treatment process quality, emerging drivers within the wastewater treatment industry that could affect wastewater management for the region should be considered. Although it is impossible to predict future wastewater treatment needs and regulations, it is expected that the following drivers observed over recent years will increasingly influence wastewater treatment in upcoming years:

- Increasing Levels of Treatment for Contaminant and Pathogen Removal

  Consistent with the goals of the National Pollutant Discharge Elimination System (NPDES), discharge limits for contaminants and pathogens have become more stringent over time. There has been a trend toward lower pathogen limits for some effluents, particularly for water reuse and discharge to shellfish-bearing waters. In addition, research continues regarding the environmental significance and fate of ultra-trace levels of organic compounds such as pesticides and pharmaceuticals, often found in parts per billion or trillion in treated effluent. Finally, the State continues to update its surface water standards, which may result in more stringent limits for discharges, including regulating pollutants down to lower levels.

- Energy Conservation

  Wastewater treatment plants and water reclamation facilities are increasingly designed with energy conservation as a major consideration. Energy conservation efforts have been boosted by significant national and state funding. It is expected that this trend will continue as energy costs increase in the future.

- Climate Change (reducing contributions to climate change and protection from its effects)

  Climate change is expected to have a significant impact on water resources, and thus human consumption/ waste disposal patterns, in the coming century. In

Washington State, these impacts are expected to include increasing ambient temperatures, reductions in snowpack and some streamflows, lower pH in oceans and higher sea levels.

Increasingly, consideration is provided to minimizing greenhouse gas emissions and the "carbon footprint" from wastewater treatment plants, including reducing carbon dioxide (including through energy minimization), methane and nitrous oxide emissions from "cradle to grave" analyses for (1) wastewater from generation to reuse; and (2) treatment /conveyance infrastructure, from manufacture, use and disposal.

- Resource Recovery

  Consistent with the goals of minimizing energy consumption and carbon footprint, plants are increasingly designed for maximal recovery of useful products – not only reclaimed water and fertilizer (biosolids), but also, in some cases, nitrogen and phosphorus.

  Aberdeen's WWTP provides high quality effluent and Class B Biosolids for reuse providing resource recovery. Energy conservation is provided through the use of high efficiency blowers, fine bubble diffusers, variable frequency drives, and the use of recycle to anoxic zones to reduce oxygen consumption. The WWTP's processes can relatively easily be adapted to meet new standards. If Hoquiam's wastewater would be treated at Aberdeen, a similar evaluation would apply.

  Based on the 2013 City of Hoquiam Facility Plan, the upgraded Hoquiam WWTP would certainly be designed to meet the same objectives as the Aberdeen WWTP, including resource recovery, energy efficiency and high quality effluent. Thus, neither alternative has a significant advantage in meeting these criteria.

PUBLIC CONCERNS

Proposals for siting new wastewater treatment facilities near existing land uses often result in the "NIMBY response" ("Not In My Back Yard"). Locating an equalization facility at the K Street pump station night engender significant opposition from neighbors; however, equalization at the Hoquiam WWTP appears to be much more cost-effective and would likely result in less opposition.

Another typical significant public concern is sewer rates. Implementation of Alternative 2 will result in a significant near-term increase in sewer rates to pay for the conveyance improvements.

LOCAL CONTROL

With Alternative 1, each City will have control over current and future decisions regarding conveyance and treatment. In addition, each City will have control over the means of financing the costs associated with future operations and capital expenditures.

A key means to mitigate the reduction in local control for Hoquiam with the implementation of Alternative 2 is through the Interlocal Agreement with Aberdeen. The Interlocal Agreement should be crafted to ensure that costs are fairly distributed, and that Hoquiam has input in the decision making and management of the facility.

RISK

Construction and operation of wastewater treatment facilities is an important and necessary aspect of modern society, but entails significant risks, including the risk of construction cost overruns, future operating/capital cost increases, and regulatory/operations risk.

- Potential for Cost Over-Runs for Construction of WRF

  Like any large construction project, building new sewer lines and new or upgraded treatment facilities has the potential for cost overruns. The inclusion of a significant contingency fund and the completion of appropriate geotechnical and archeological studies can reduce the potential; however, as has been observed with construction of other treatment facilities (Brightwater, Belfair, etc.), unforeseen costs for treatment plants can exceed contingencies and raise project costs and rates. The risk of cost overruns is similar for both alternatives.

- Risk of Future Operating Cost Increases

  As energy prices increase and discharge standards become more stringent, the need for more highly trained operators and overall costs for wastewater treatment increase. Finding economies of scale through regionalization can mitigate future cost increases, and is often favored by funding agencies and regulators. Many communities in Washington have decided to regionalize, including in the last few years, Raymond –South Bend, Sequim-Carlsborg, and Ridgefield – Clark County/Salmon Creek.

- Regulatory, Operations Risk

  There is significant legal and financial risk to operating a WWTP. Dozens of facilities within Washington State have been sued by third-party groups for such violations, resulting in expensive legal bills, settlements and additional costs. There is some benefit to regionalization in reducing risk, as larger facilities can typically afford to employ more experienced operations staff.

ENVIRONMENTAL BENEFITS

Implementation of either alternative will have significant environmental benefits to the region. Higher levels of WWTP reliability will reduce risks to the Chehalis River estuary and Grays Harbor. There is some environmental advantage to decommissioning the Hoquiam WWTP, as it is closer to the shellfish beds, and an upset there is therefore more likely to cause shellfish contamination.

Climate change is expected to result in increasing temperatures, reductions in snowpack and some streamflows and lower pH in oceans and higher sea levels. It is difficult to predict whether these changes will favor Alternative 1 or 2. Both WWTP locations are expected to be impacted by rising sea levels. Since both alternatives utilize variations on the activated sludge process, neither is expected to have a significant advantage with regard to causing additional climate change.

SUMMARY AND RECOMMENDATIONS

Table 8-19 summarizes the evaluation of the two alternatives. For each alternative, a score is provided in the matrix, with 10 being the highest (best) score and 1 being the lowest (worst) score. As shown, Alternative 1 ("Go It Alone") has a slightly higher overall rating. The economies of scale typically associated with regionalization in terms of capital cost do not appear to be as significant for Aberdeen and Hoquiam. Alternative 1 is the one for which a CIP and financial analysis are provided in Chapter 9. However, the economic and overall ratings are very close between the two alternatives, and the Cities do have some additional time to further consider regionalization. Aberdeen has received funding for improvements to the WWTP Influent Pump Station, Headworks and Primary Sludge Pump Room, projects that are fully compatible with either Alternative 1 or Alternative 2. It is understood that Hoquiam will in the near future update its Facility Plan, and when complete, costs for upgrading the Hoquiam WWTP could be updated for the life cycle comparison.

TABLE 8-19

Comparison for Aberdeen and Hoquiam – Alternative 1 vs. Alternative 2

Criteria	Alternative 1: "Go It Alone"	Alternative 2: Hoquiam Served Along with Existing Partners at Existing Aberdeen WWTP
Capital Costs	5	6
Operating Costs	5	6
Public Concerns	7	5
Local Control	7	3
Risk	5	5
Environmental Benefits	4	6
TOTAL	**33**	**31**

(1) 10 is the highest (best) score and 1 is the lowest (worst) score.

CHAPTER 9

CAPITAL IMPROVEMENT PLAN AND FINANCIAL ANALYSIS

INTRODUCTION

This chapter summarizes the City of Aberdeen's Captial Improvement Plan (CIP) and provides a financial program for City of Aberdeen sewer utility that is consistent with the implementation of the recommended capital improvments for the wastewater collection system and treatment plant and operating expenses outlined in the previous chapters. The financial status of the sewer utility, the funding required to pay for the scheduled improvements, potential funding sources, and the financial impact of wastewater improvements and operating expenses on sewer rates are presented.

The information in this chapter includes excerpts from a Utility Rate Study conducted in 2019-2020 for the City of Aberdeen. More information from the rates study is included in Appendix T.

CAPITAL IMPROVEMENT PLAN

Wastewater capital improvements have been identified and prioritized based on the collection and plant hydraulic analyses, regulatory requirements, condition assessment, operation and maintenance considerations, system benefit, and costs. For all proposed projects identified in this chapter, detailed preliminary project cost estimates are presented in Appendix L. Figures illustrating the conceptual locations of the proposed improvement projects are included in Chapters 6 (for the collection system) and 8 (for the WWTP). The WWTP CIP assumes implementation of Alternative 1 (Chapter 8).

Other capital improvement projects may arise in the future that are not identified as part of the City's CIP presented in this chapter. Such projects may be deemed necessary for remedying an emergency situation, assessing growth in other areas, accommodating improvements proposed by other agencies or land development, or addressing unforeseen problems with the City's wastewater system. Due to budgetary constraints and/or addressing growth scenarios that differ from that which was modeled in this Plan, the construction of these projects may require changes in the proposed completion date for projects in the CIP. When new information becomes available, the Plan should remain flexible to allow rescheduling, addition to, or deletion of proposed projects or to expand or reduce the scope of the projects, as best determined by the City. Additionally, future planning efforts may affect land use zoning and service requirements within the City. Developments may create streets or provide alignments and locations of facilities that are different than shown on the Plan. Each capital improvement project should be reevaluated to consider the most recent planning efforts as the proposed completion date for the project approaches.

PROPOSED SYSTEM IMPROVEMENTS

The proposed system improvements in the CIP are shown below in Tables 9-1 and 9-2 for the collection system and WWTP, respectively. Each project cost estimate includes sales tax, construction contingency, and design, engineering, and permitting. All project costs are based on 2020 dollars with no adjustments made for inflation in future years.

TABLE 9-1

Collection System – 6-Year Capital Improvement Plan

CIP #	Project Name	Cost	Year	Description
	Infiltration and Inflow Study	$75,000	2020-2021	I/I Study with smoke testing, flow monitoring, TV inspection
CS-1	Bypass Connections PS 4,6,7	$201.000	2021-2022	Bypass connections to force main to allow bypass of pump stations
CS-2	PS 5 Upgrade	$676,000	2021-2022	Replace force main, all mechanical, electrical and I&C; rehabilitate wetwell concrete surface; add bypass piping connection
CS-3	Fry Creek Pump Stations	$200,000	2020-2021	Small pump stations (project completed by City staff)
CS-4	PS 6 Upgrade	$1,306,000	2021-2022	Replace pumps, all mechanical, electrical. I&C and force main. Rehabilitate wet well concrete surfaces.
CS-5	PS 13 Upgrade	$2,425,000	2021-2022	Construct new above grade control room; replace all mechanical, electrical and I&C; install new generator; rehab wet well concrete surface; add bypass piping connection; Upsize downstream piping
CS-6	PS 10 Upgrade	$580,000	2025-2026	Replace mechanical, electrical and I&C; Rehabilitate wet well concrete surface; add bypass piping connection
CS-7	PS 7 Upgrade	$1,589,000	2021-2023	Upsize pumps to 1,200 gpm, replace mechanical, electrical and I&C; Install new generator; Rehabilitate wetwell concrete surface; Replace forcemain.
CS-8	PS 4 Upgrade	$1,087,000	2021-2023	Upsize pumps to 1,000 gpm, replace mechanical, electrical and I&C; Rehabilitate wet well concrete surface; replace force main.
CS-9	PS 8 Replacement	$1,362,000	2022-2024	Replace Pump Station
CS-10	PS 2 Upgrade	$1,081,000	2024-2025	Add pump, replace all mechanical, electrical and I&C; rehabilitate wet well concrete surface.
CS-11	PS 9 Upgrade	$865,000	2024-2025	Upsize pumps to 1,000 gpm. Replace mechanical, electrical and I&C; Install new generator; rehab wet well concrete surface; replace discharge force main.
CS-12	PS 11 Upgrade	$606,000	2025-2026	Replace pumps, all mechanical, electrical and I&C; rehab wet well concrete surface; add bypass piping connection.

6-Year CIP only; additional future projects identified in Chapter 8.

TABLE 9-2

Wastewater Treatment Plant – 6-Year Capital Improvement Plan

CIP #	Project Name	Cost	Year	Description
WW-1	Influent Pump Station Pump Replacement	$42,770	2020	Replace single pump at end of useful life
WW-2	Influent Pump Station VFD Replacement	$67,500	2020	Replace single VFD at end of useful life
WW-3	Disinfection Improvements	$2,482,625	2019-2020	Convert to liquid chlorination/dechlorination, rehabilitate process water system (project ro be completed in fall 2020)
WW-4	New WWTP Generator	$3,149,000	2022-2024	New generator, switchgear
WW-5	Influent Pump Station Rehabilitation	$2,966,000	2021-2023	Rehabilitate wet well, structural improvements, ventilation compliance
WW-6	Existing Digester Rehabilitation	$2,609,000	2021-2022	Fix roof, Replace gas lines, heat exchanger, boiler, electrical code upgrades
WW-7	Primary Sludge Pump Room Rehabilitation	$1,241,000	2021-2023	Electrical and controls, ventilation compliance, process piping improvements, flood hazard mitigation
WW-8	Aeration Basin Improvements	$2,138,000	2023-2025	Miscellaneous structural, mechanical and electrical improvements, including tank surface rehabilitation, remediate settling of yard piping and electrical raceways
WW-9	Headworks Upgrade	$2,558,000	2021-2023	New screens and washer compactors, raise walls, modify stairway access, electrical improvements
WW-10	Secondary Clarifier 1 Improvements	$1,529,000	2023-2025	Replace mechanisms, equipment, surface rehabilitation, remediate settling of yard piping and electrical raceways.
WW-11	Thickener Upgrade	$1,379,000	2023-2025	Replace mechanisms and equipment, surface rehabilitation, Remediate settling of yard piping and electrical raceways, replace yard piping
WW-12	Conduit/Piping Rehabilitation	$250,000	2025-2027	Remediate settled conduit, process piping
WW-13	East Primary Clarifier Rehabilitation (1)	$273,000	2025-2027	Replace mechanisms, equipment, surface rehabilitation, Remediate settling of yard piping and electrical raceways

(1) 6-Year CIP only; additional future projects identified in Chapter 8.

FINANCIAL STATUS OF EXISTING ABERDEEN SEWER UTILITY

CURRENT SEWER RATES

The sewer rates for the City of Aberdeen Sewer Utility are defined in the City Code Chapter 13.48.020. The current rates and the rates for 2021 are shown in Table 9-3. On November 13, 2019, City Council has passed Ordinance 6655, which established a series of annual sewer rate increases beginning January 1, 2020 and extending through 2024 at which time the sewer rate would be $72 per month for both commercial and residental dwelling units. Beginning January 1, 2025 the ordinance provides for an annual increase of sewer rates based on the average rate of increase in the consumer price index for antecedent 12 month period July through June.

TABLE 9-3

Aberdeen Current Sewer Utility Rates

	2020		2021	
	MonthlyBasic Residental	**Volume Charge(1)**	**MonthlyBasic Residental**	**Volume Charge(1)**
Commercial	$46.00	$0.06/cf	$53.00	$0.07/cf
Dwelling Units	$46.00	$0.06/cf	$53.00	$0.07/cf

(1) Volume charge: Additional fee per cubic foot of metered water use over 1000 cubic feet per month.

CURRENT SEWER CONNECTION CHARGES

Connection charges are defined in the Aberdeen City Code Chapter 13.52.030. There is no connection charge or inspection fee for a new connection, residential or commercial/governmental.

HISTORICAL FINANCIAL OPERATIONS

Sewer utility operating revenue and expenses for the years 2018 through 2020 are summarized in Table 9-4.

TABLE 9-4

Historical Operating Revenues and Expenses

Revenue Summary	**2018**	**2019**	**2020 Budget**
Sewer service – retail	$3,315,742	$3,358,610	$4,162,400
Sewer service – contract	$555,433	$554,667	$667,920
Swr Industrial treatment	$104,673	$110,076	$211,750
Septage and Sludge Trt	$253,822	$250,623	$181,500
Impact Fees	$0	$180	$0
Investment Interest	$2,847	$1,740	$0
Loan Proceeds	$0	$0	$1,288,878
Misc. Revenue	$7,393	$783	$0
Transfer In 413 Cum Res	$0	$1,000,000	$0
Total Sewer Revenue	$4,239,910	$5,276,679	$6,512,448
Operating Expense	**2018**	**2019**	**2020 Budget**
Admin/Cust Svc/Billing	$1,015,462	$1,030,805	$1,098,778
Allocate Admin 50/50 to Collect and Treatment			
Collection System	$1,317,753	$1,362,155	$1,591,234
Treatment System	$2,327,569	$2,191,741	$2,517,608
Total Sewer Oper Exp	$3,645,322	$3,553,895	$4,108,842
Sewer Debt	$449,493	$447,718	$453,172
Sewer Capital	$288,600	$1,638,715	$1,885,449
Transfer to Reserves	$0	$50,000	$0
Total Sewer Expense	$4,383,415	$5,690,328	$6,447,463

Table 9-5 summarizes the net operating revenue (operating revenue minus operating expenses) from 2018 to 2020.

TABLE 9-5

Historical Net Operating Revenue

Net Operating Revenue	**2018**	**2019**	**2020 Budget**
Operating Revenue	$4,239,910	$5,276,679	$6,512,448
Operating Expenses	$4,383,415	$5,690,328	$6,447,463
Net Operating Revenue	($143,505)	($413,649)	$64,985

The historical utility expenses between year 2016 and 2020 are presented in Figure 9-1.

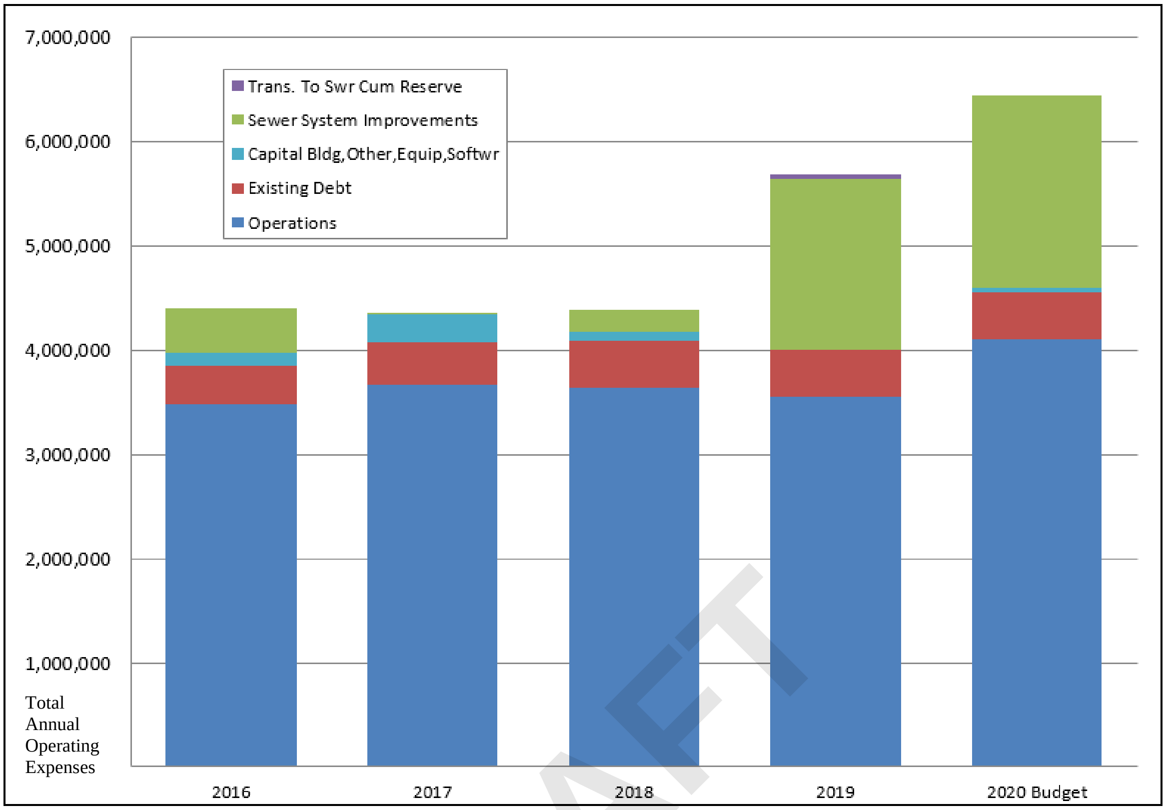


FIGURE 9-1

Sewer Utility Expense History

The composition of the latest year 2020 expense budget is presented in Figure 9-2.

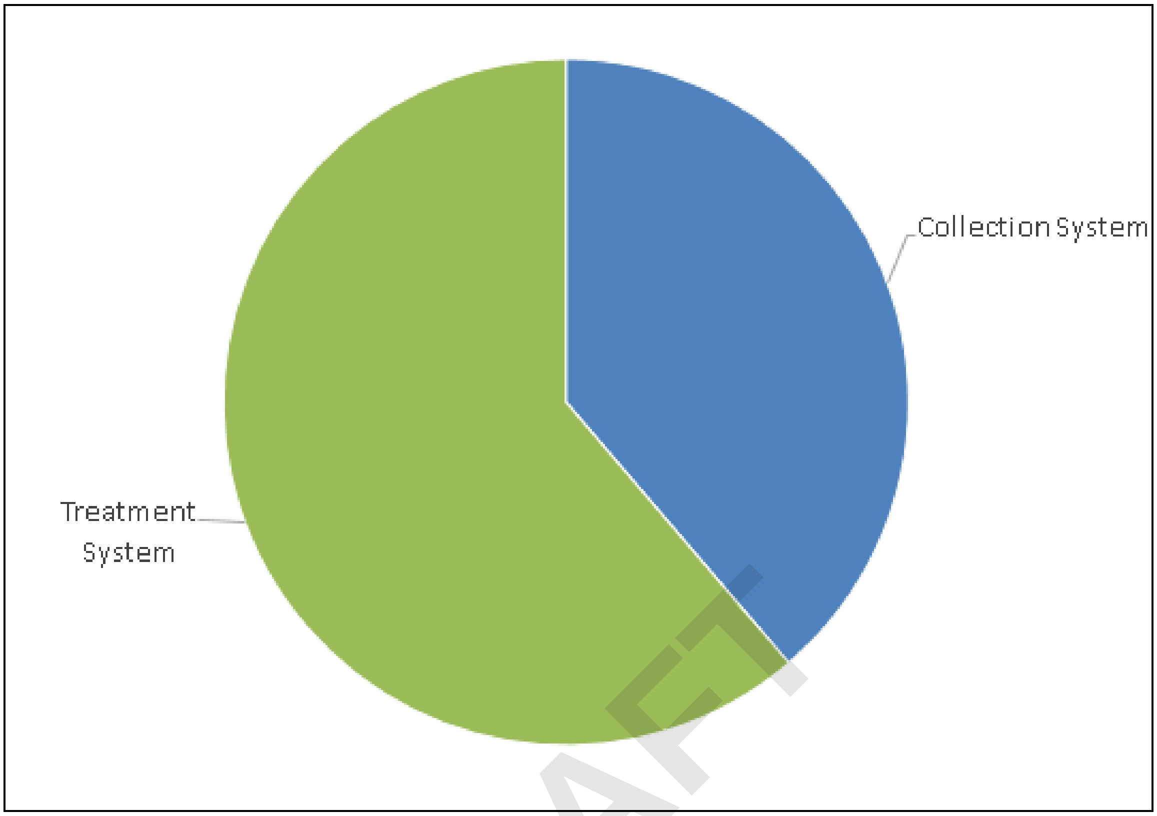


FIGURE 9-2

Sewer Operations - 2020 Budget

PROJECTED GROWTH

In order to project future revenues, the growth in the number of customers must be estimated. Chapter 5 included a discussion of the projected population for the City of Aberdeen. Current population in the City is estimated to be 16,880. The projected population growth is 1.0 percent per year which results in a 2038 population of 20,450. This is a conservative projection for planning infrastructure, because the City has actually had much lower growth (average annual growth of 0.15 percent) over the last 15 years. However, this is not a conservative projection for the financial analysis. The financial analysis in this chapter assumes no growth in Aberdeen residential, commercial or industrial customers or in Cosmopolis or SCCC, in order to be conservative and ensure adequate funding.

PROJECTED EXPENSES, REVENUES, AND CAPITAL RESERVES

FUTURE OPERATING REVENUES AND EXPENSES

Tables 9-6 and 9-7 show the background data upon which the operating revenue and expenses projections developed below are based. Assumptions used in determining the projections are shown in Table 9-6.

Table 9-6 presents the projected operating revenues for the sewer utility. Revenues and expenses for 2020 – 2025 are projected based on the 2019 actual revenue and expenses.

TABLE 9-6
Projected Operating Revenues

	2019	2020	2021	2022	2023	2024	2025
Assumptions							
Growth – New Homes per year		0	0	0	0	0	0
Cost Escalation – General		3.0%	3.0%	3.0%	3.0%	3.0%	3.0%
Cost Escalation – Construction/CIP		4.0%	4.0%	4.0%	4.0%	4.0%	4.0%
Residential Monthly Sewer Rate	$38.62(1)	$46.00(1)	$53.00(1)	$60.00(1)	$66.00(1)	$72.00(1)	$74.16(2)
Sewer Service Connection Fee	$0	$0	$0	$0	$0	$0	$0
Side Sewer Inspection Fee	$0	$0	$0	$0	$0	$0	$0
Sewer General Facility Charge – GFC	$0	$0	$0	$0	$0	$0	$0
Sewer Fund – 403							
Revenue	**2019**	**2020**	**2021**	**2022**	**2023**	**2024**	**2025**
Sewer Service – Retail	$3,358,610	$4,162,400	$4,795,800	$5,429,200	$5,972,100	$6,515,000	$6,710,500
Sewer Service – Contract	$554,667	$667,920	$769,560	$871,200	$958,320	$1,045,440	$1,076,803
Industrial Treatment	$110,076	$211,750	$243,973	$276,196	$303,815	$331,435	$341,378
Septage and Sludge Treatment	$250,623	$181,500	$186,900	$192,500	$198,300	$204,200	$210,300
Impact Fees	$180	$0	$0	$0	$0	$0	$0
Investment Interest	$1,740	$0	$0	$0	$0	$0	$0
Loan Proceeds	$0	$2,482,625	$0	$0	$0	$0	$0
Misc. Revenue	$783	$0	$0	$0	$0	$0	$0
Transfer From 413 Swr Cum. Reserve	$1,000,000	$0	$0	$0	$0	$0	$0
Total Operating Revenue	**$5,276,679**	**$7,706,195**	**$5,996,233**	**$6,769,096**	**$7,432,535**	**$8,096,075**	**$8,338,981**

(1) The sewer rate value shown is stipulated in the City of Aberdeen Municipal Code.
(2) The value shown is the estimated sewer rate beginning January 1, 2025 based on the methodology specified in Ordinance 6655 and the assumed CPI of 3 percent.

The projected operating expenditures for the sewer utility are presented in Table 9-7 and Figure 9-3.

TABLE 9-7

Projected Operating Expenditures

Expenses	2019	2020	2021	2022	2023	2024	2025
Sewer Collection	$1,362,155	$1,591,234	$1,639,000	$1,688,200	$1,738,800	$1,791,000	$1,844,700
Sewer Treatment	$2,191,741	$2,517,608	$2,593,100	$2,670,900	$2,751,000	$2,833,500	$2,918,500
Existing Debt	$447,718	$453,172	$557,489	$580,290	$277,098	$277,098	$277,098
Comp Equip, Mach and Equip, Bldg, Other	$0	$40,000	$41,200	$42,400	$43,700	$45,000	$46,400
Sewer CIP Funded by Rates	$1,638,715	$2,005,719	$1,064,900	$1,626,700	$2,195,700	$1,750,100	$1,907,700
New Debt for CIP	$0	$0	$0	$0	$409,600	$839,500	$1,035,500
Transfer To 413 Swr Cum. Reserve	$50,000	$0	$0	$0	$0	$0	$0
Total Operating Expenses	$5,690,328	$6,607,733	$5,895,689	$6,608,490	$7,415,898	$7,536,198	$8,029,898

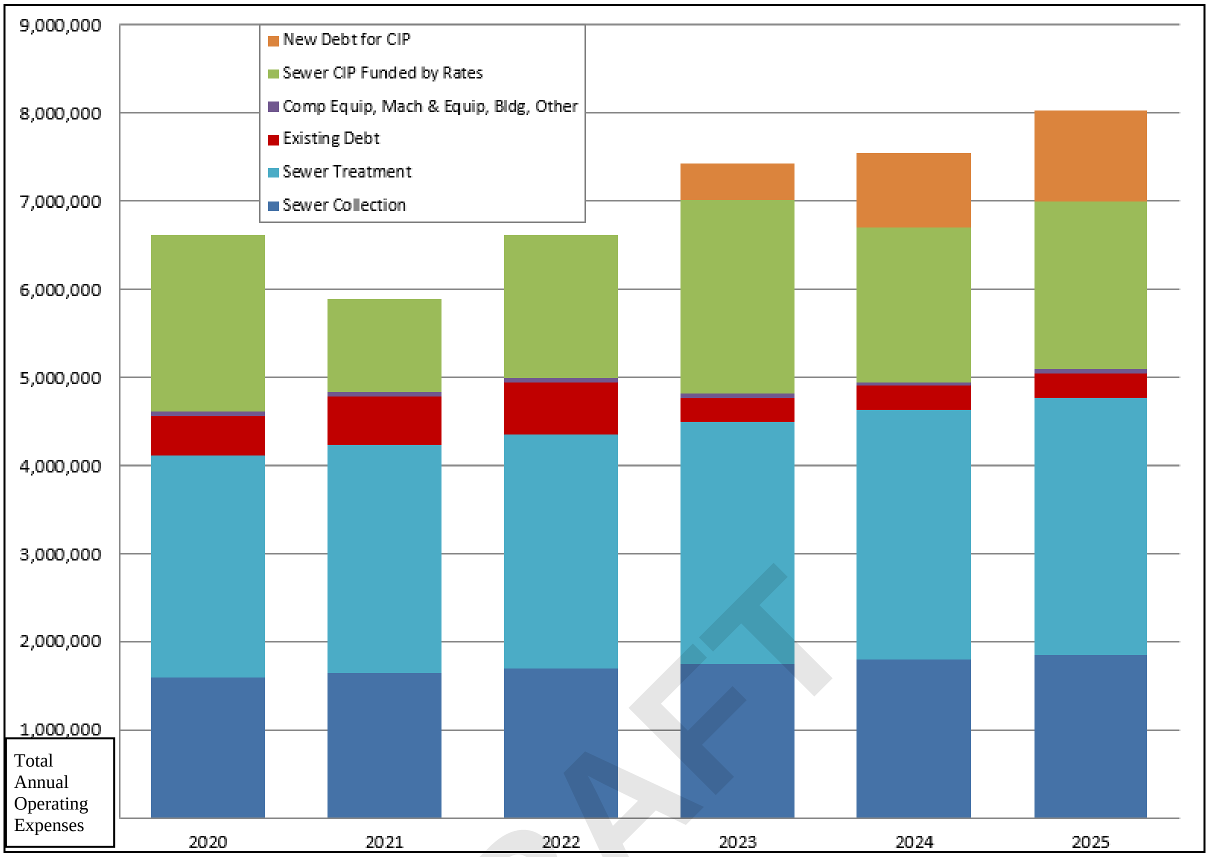


FIGURE 9-3

Sewer Program Outlook

CAPITAL EXPENDITURES AND RESERVES

Maintaining reserves at an appropriate level to provide for operations, revenue stabilization, emergency repair or replacement of essential equipment and for capital maintenance is an element of sound utility management.

Capital improvement projects to be funded over the period 2019 – 2025 are summarized in Table 9-8. Collection system and WWTP improvements are described in Chapter 6 and 8, respectively, and are summarized below. These projects will be debt financed.

TABLE 9-8

Projected Capital Expenditures

CIP #	Project Name	2019	2020	2021	2022	2023	2024	2025
Sewer WWTP		**Project cost estimates are shown in 2020 dollars, with yearly totals escalated at 4 percent per year**						
	Flow Meter, Repair and Replace		$75,000					
WW-1	Influent Pump 1 Replace Pump		$42,770					
WW-2	Influent Pump 1 Pump VFD and Install		$67,500					
WW-3	Disinfection Improvements	$1,137,176	$1,345,449					
WW-4	New WWTP Generator				$409,000		$2,740,000	
WW-5	Influent Pump Station, Wet Well, Structural, Ventilation			$386,000		$2,580,000		
WW-6	Rehabilitate Existing Digester			$339,000	$2,270,000	$0		
WW-7	Primary Sludge Pump Room			$161,000		$1,080,000		
WW-8	Aeration Basin Improvements					$278,000		$1,860,000
WW-9	Headworks Upgrade			$298,000		$2,260,000		
WW-10	Secondary Clarifier 2 Improvements					$199,000		$1,330,000
WW-11	Thickener Upgrade					$179,000		$1,200,000
WW-12	Remediate Settled Conduit, Process Piping(1)							$250,000
WW-13	East Primary Clarifier Rehabilitation(1)							$273,000
	Flow Meter, Repair and Replacement		$75,000					
	PS 15 Rehabiliation	$106,164						
	PS 16 Rehabilitation	$106,164						
	Misc Pump Improv. (Budget)		$200,000					
	Infiltration and Inflow Study		$75,000					
CS-1	Bypass Connections PS 4,6,7			$27,000	$174,000			
CS-2	PS 5 Upgrade			$90,000	$586,000			

TABLE 9-8 – (continued)

Projected Capital Expenditures

CIP #	Project Name	2019	2020	2021	2022	2023	2024	2025
Sewer Collection – (continued)		**Project cost estimates are shown in 2020 dollars, with yearly totals escalated at 4 percent per year**						
CS-3	Fry Creek Pump Stations			$200,000				
CS-4	PS 6 Upgrade			$208,000	$1,353,000			
CS-5	PS 13 Upgrade			$313,000	$2,040,000			
CS-6	PS 10 Upgrade							$88,000
CS-7	PS 7 Upgrade			$226,000	$560,000	$912,000		
CS-8	PS 4 Upgrade			$160,000		$1,040,000		
CS-9	PS 8 Replace Entire PS				$184,000		$1,199,000	
CS-10	PS 2 Upgrade						$174,000	$1,133,000
CS-11	PS 9 Upgrade						$123,000	$799,000
CS-12	PS 11 Upgrade							$681,000
Total Costs								
Total 6-Year CIP - S ($2020)		**$1,349,504**	**$2,005,700**	**$2,208,000**	**$7,576,000**	**$8,528,000**	**$4,236,000**	**$7,614,000**
6-Year CIP S (Escalated)			**$2,005,700**	**$2,296,300**	**$8,194,200**	**$9,592,800**	**$4,955,500**	**$9,263,600**
Total 2020-25		**$32,167,719**	**$36,308,100**					
Average 2020-25		**$5,361,287**	**$6,051,350**					
Subtotal WWTP ($2020)		**$1,137,200**	**$1,530,700**	**$1,231,400**	**$2,897,600**	**$7,397,100**	**$3,205,400**	**$5,977,400**
Subtotal Collection ($2020)		**$212,300**	**$475,000**	**$1,065,000**	**$5,296,600**	**$2,195,700**	**$1,750,100**	**$3,286,200**
Construction Cost Escalator			***1.00***	***1.04***	***1.08***	***1.12***	***1.17***	***1.22***

(1) For projects WW-12 and WW-13, year 2025 costs only include engineering design cost; construction of these projects, with costs $1,750,000 and $910,000 for WW-12 and WW-13, repectively, will start after year 2025.

CAPITAL IMPROVEMENTS FINANCING

The sewer utility is not adequate to self-finance the schedule of projects. The City needs to consider a combination of grants, debt financing along with monthly service rate adjustments.

PUBLIC FINANCING SOURCES

The following section describes several financing sources available to the City. Each funding source has different eligibility dependent upon the programs goals and mission. In addition, each funding source has different requirements for how they may calculate the City's grant and loan eligibility. Some programs rely on median household income statistics while other programs look at the low-to-moderate income status for the region. Following is a description of each program.

DEPARTMENT OF COMMERCE CDBG GENERAL PURPOSE GRANT

The Department of Commerce administers the Community Development Block Grant (CDBG) General Purpose Grant Program. This program makes funds available annually through a competitive application process to assist Washington cities, towns, and communities. Eligible activities include "public facilities such as water, wastewater, and streets." A main emphasis of this program is to provide services to low- and moderate-income (LMI) persons. To be eligible for the program a community must be over 51 percent LMI. The maximum grant available is $900,000, the applications are typically due in June of each year. Aberdeen qualifies for funding from the CDBG program.

DEPARTMENT OF ECOLOGY STATE REVOLVING FUND

Ecology administers the State Revolving Fund (SRF) program, which makes no- and low-interest loans available to communities with qualifying projects. Application for these funds is through Ecology's annual funding cycle in the fall. SRF loans are available for planning, design, and construction projects. Loans are available for terms up to 30 years at interest rates that are calculated at 60 percent of the average municipal bond interest rate. For qualifying low-income communities, zero percent loans can be made available.

Grant and loan eligibility is based on what Ecology terms as hardship. Hardship is when a community's monthly rate exceeds 2 percent of the median household income (MHI). Ecology has four levels of hardship as shown in Table 9-9.

TABLE 9-9

Ecology Hardship Funding

Sewer Fee/MHI	<2%	>/=2%but<3%	>/=3% but <5%	>/=5%
Hardship Designation	Non-hardship	Moderate hardship	Elevated hardship	Severe hardship
20-Year Loan Rates	2.00%	1.30%	0.70%	0.00%
Grant Eligibility	Not Eligible	50% up to $5M	75% up to $5M	100% up to $5M

PUBLIC WORKS TRUST FUND (PWTF)

The Legislature created the Public Works Board in partnership with local governments to assist in addressing infrastructure needs. They use a dedicated funding pool to offer low-interest financing in a revolving loan program. A citizens' board of infrastructure representatives manage the program. Cities, counties, special purpose districts, public utility districts, and quasi-municipal governments are eligible to receive loans from the PWTF (now typically called Public Works Board financing). Eligible projects include repair, replacement, and construction of infrastructure for domestic water, sanitary wastewater, stormwater, solid waste, road, and bridge projects. The standard loan is for terms between 5 and 20 years. All loan terms are subject to negotiation and Board approval. Interest rates for PWTF loans have generally been in the 0.75 percent to 1.5 percent range. This funding has been available intermittently in recent years.

USDA-RURAL DEVELOPMENT

The USDA Rural Development agency (RD) has a loan program which, under certain conditions, includes a limited grant program. Grants may be awarded when the average user rate exceeds 1.5 percent of the median household income. Loans are offered at interest rates of around 2.0 to 3.0 percent at terms up to 40 years. Because RD is a federal funding program, an environmental report meeting the requirements of NEPA is required.

In general, Aberdeen has too large a population to be considered for RD funding. However, some of its current and future regional partners may qualify for RD funding.

INFRASTRUCTURE ASSISTANCE COORDINATING COUNCIL

The Infrastructure Assistance Coordinating Council (IACC) is comprised of state and local agencies whose function is to provide funding for infrastructure repair and development. Its purpose is to assist local governments in coordinating funding efforts for infrastructure improvements and can be a valuable resource to provide awareness of any new funding opportunities. The IACC holds an annual fall conference.

REVENUE BONDS

Another source of funds for construction of major utility improvements is the sale of revenue bonds. The City would issue the tax-free bonds. The major source of funds for debt service on these revenue bonds is from sewer service rates. In order to qualify to sell revenue bonds, the City must show that its net operating income (gross income less expenses) is equal to or greater than a debt coverage factor times the annual principal and interest payments due for all outstanding bonded indebtedness. The debt coverage factor is applicable to revenue bonds sold on the commercial market. The City's bond writer will typically set the debt coverage factor and it may vary from 1.2 to 1.4.

GENERAL OBLIGATION BONDS

The City, by special election, may issue general obligation bonds to finance almost any project of general benefit to the community. Assessments levied against all privately owned properties within the community will pay for the bonds. This includes vacant property that otherwise would not contribute to the cost of such general improvements. This type of bond issue is usually reserved for municipal improvements that are of general benefit to the public, such as arterial streets, bridges, lighting, municipal buildings, firefighting equipment, parks, and water and wastewater facilities. Because the money is raised by assessment levied on property values, the business community also provides a fair share of funds to pay off such bonds.

General obligation bonds have the best market value and carry the lowest interest rate of all types of bonds available to the City.

Disadvantages of general obligation bonds include the following:

- Voter approval is required which may be time-consuming, with no guarantee of successful approval of the bond.

- The City would have a practical or legal limit for the total amount of general obligation debt. Financing large capital improvements through general obligation debt reduces the ability of the utility to issue future debt for projects such as parks and community facilities that cannot be directly funded through enterprise funds.

UTILITY LOCAL IMPROVEMENT DISTRICTS

Another potential source of funds for improvements comes through the formation of Utility Local Improvement Districts (ULIDs) involving an assessment made against properties benefited by the improvements. ULID bonds are further guaranteed by revenues and are financed by issuance of revenue bonds. ULID financing is frequently applied to sewer system extensions into previously unserved areas. Typically, ULIDs are formed by the municipality at the written request (by petition) of the property owners within a specific area of the municipality. Upon receipt of a sufficient number of signatures on petitions, the local improvement area is defined, and a sewer system is designed for that particular area in accordance with the municipality's general comprehensive plan. Each separate property in the ULID is assessed in accordance with the special benefits the property receives from the sewer system improvements.

DEVELOPER FINANCING

Developers may fund the construction of extensions to the sewer system to property within new plats. The developer extensions are turned over to the City for operation and maintenance when completed.

It may be necessary, in some cases, to require the developer to construct more facilities than those required by the development in order to provide either extensions beyond the plat and/or larger pipelines for the ultimate development of the sewer system. The City may, by policy, reimburse the developer through direct outlay, latecomer charges, or reimbursement agreements for the additional cost of facilities, including increased size of pipelines over those required to serve the property under development. Construction of any pipe in commercial or industrial areas that is larger than the size required to serve the development should also be considered as an oversized line possibly eligible for compensation.

COMMUNITY ECONOMIC REVITALIZATION BOARD (CERB)

CERB is administered through the Department of Commerce and provides funding to local governments for public infrastructure which supports private business growth and expansion. Eligible projects for CERB funding include domestic and industrial water, stormwater, wastewater, public buildings, telecommunications and port facilities, among others. CERB can provide funding for the following opportunities:

- <u>Committed Private Partner Program</u>: A private business or development is ready to occur and is contingent on approval of CERB funds. The project will create a significant number of permanent jobs or generate significant private capital investment. The median hourly wage of private sector jobs created after the project is completed must exceed the City-wide median wage. Up to $300,000, or 50 percent of the total award, whichever is less, may be awarded as grant, with a 20 percent cash match required.

- <u>Planning Projects</u>: Limited funds are available to fund studies which evaluate high priority economic development projects. Priority is given to applications which could ultimately result in a type of project eligible for CERB construction funds. Up to $50,000 may be awarded as grant and a 25 percent cash match is required.

- Prospective Development Construction Program: Rural communities may receive loans and grants for public infrastructure to enable future business development. The Aberdeen communities would be eligible for this program if an economic feasibility study demonstrated that private business development is likely to occur as a result of the public improvements. As with the Committed Private Partner Program, the development would need to lead to significant job creation, and it must be demonstrated that the applicant has no other feasible funding alternative. Up to $300,000, or 50 percent of the total award may be awarded as grant, with a 50 percent cash match required.

GENERAL FACILITY CHARGE

A General Facility Charge (GFC) is a charge to connect to and purchase capacity in the sanitary sewer system and to address the added demand placed upon the system. A GFC is intended to cover the cost of developing the necessary capital facilities to support the expanded capacity. A GFC is typically charged when a new development connects to a City's system or expansions of a development necessitates additional capacity. A GFC study can be performed to evaluate recommended GFC charges for expansions to the system.

FUNDING SUMMARY

Many of the funding programs described above, such as the CERB program, are specific to a community's particular needs. The most likely funding programs for the capital improvements necessary in Aberdeen are Ecology Centennial/SRF Program, and the Department of Commerce's PWTF, and CDBG.

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