CITY OF LA CENTER

CLARK COUNTY **WASHINGTON**

GENERAL SEWER PLAN UPDATE

G&O #22607
MARCH 2024

Gray & Osborne, Inc.
CONSULTING ENGINEERS

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CHAPTER 1

INTRODUCTION

This *General Sewer Plan Update* for the City of La Center 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*, and Washington Administrative Code (WAC) Section 173-240-050, *General Sewer Plan*. Development of the Plan has been coordinated with the *City of La Center Comprehensive Plan*.

The Plan provides proposed conceptual designs, cost estimates, schedule, and a financing plan for recommended facility improvements. A State Environmental Policy Act (SEPA) checklist is provided in Appendix A. The projects described in the 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 groundwater, anti-degradation of surface water and reuse of biosolids. An adopted water quality plan exists for the receiving water (East Fork Lewis River). Further discussion on permit limits for the WWTP is provided in Chapter 3.

The City of La Center is located in Clark County, along the East Fork Lewis River (see Figure 1-1).

SCOPE OF WORK

The Plan addresses the wastewater collection system and the wastewater treatment systems for the City of La Center. This evaluation includes a collection system/wastewater treatment plant analysis, a capital improvement plan and a cost analysis with an associated schedule. The Plan is organized into the following chapters:

- Chapter 1 – Introduction
- Chapter 2 – Land Use, Population Projections, and Service Area
- Characteristics
- Chapter 3 – Regulatory Requirements
- Chapter 4 – Existing Facilities
- Chapter 5 – Wastewater Flow and Loading Projections
- Chapter 6 – Collection System Evaluation
- Chapter 7 – Wastewater Treatment Plant Evaluation
- Chapter 8 – Biosolids Treatment and Management
- Chapter 9 – Capital Improvement Plan

RELATED PLANNING DOCUMENTS

The following documents were consulted in the preparation of this Plan.

GROWTH MANAGEMENT ACT (GMA) RELATED PLANS, POLICIES AND DEVELOPMENT REGULATIONS

<u>La Center Comprehensive Plan 2016-2036</u>, Adopted by the Board of City Commissioners

The City's 2016 Comprehensive Plan provided growth projections to have a population of 7,642 and employment of 2,876 by 2036.

<u>Clark County Comprehensive Plan 2016-2036</u>, Adopted by the Board of County Commissioners

The County's 2016 Comprehensive Plan provided growth projections for the urban growth areas and rural areas within the County.

WATER AND WASTEWATER SYSTEM PLANNING

<u>Wastewater Facility Plan</u>, July 2008, Kennedy/Jenks Consultants

The *2008 Wastewater Facility Plan* included an evaluation of the City's Wastewater Treatment Plant (WWTP). The *2008 Plan* provided recommendations for WWTP improvements, which were broken down into three phases. Currently, Phase 1A has been completed.

The Phase 1A expansion, completed in 2011, converted the plant from a sequencing batch reactor to a membrane bioreactor system with capacity of 0.69 mgd maximum monthly flow and 1.29 mgd peak day flow

Phase 1B would upgrade the blower system and add additional membrane units, increasing capacity to 1.04 mgd maximum monthly flow and 1.94 mgd peak day flow.

Phase 2 would increase sludge handling capacity.

Phase 3 would increase liquid stream capacity to 3.0 mgd maximum monthly flow and 6.0 mgd peak day flow.

<u>General Sewer Plan</u>, March 2013, Wallis Engineering

The *2013 General Sewer Plan* provided an evaluation of the City's sewerage system and its ability to accommodate 20-year projected flows and loadings. The *2013 Plan* provided recommendations for collection system improvements. In addition, the *2013*

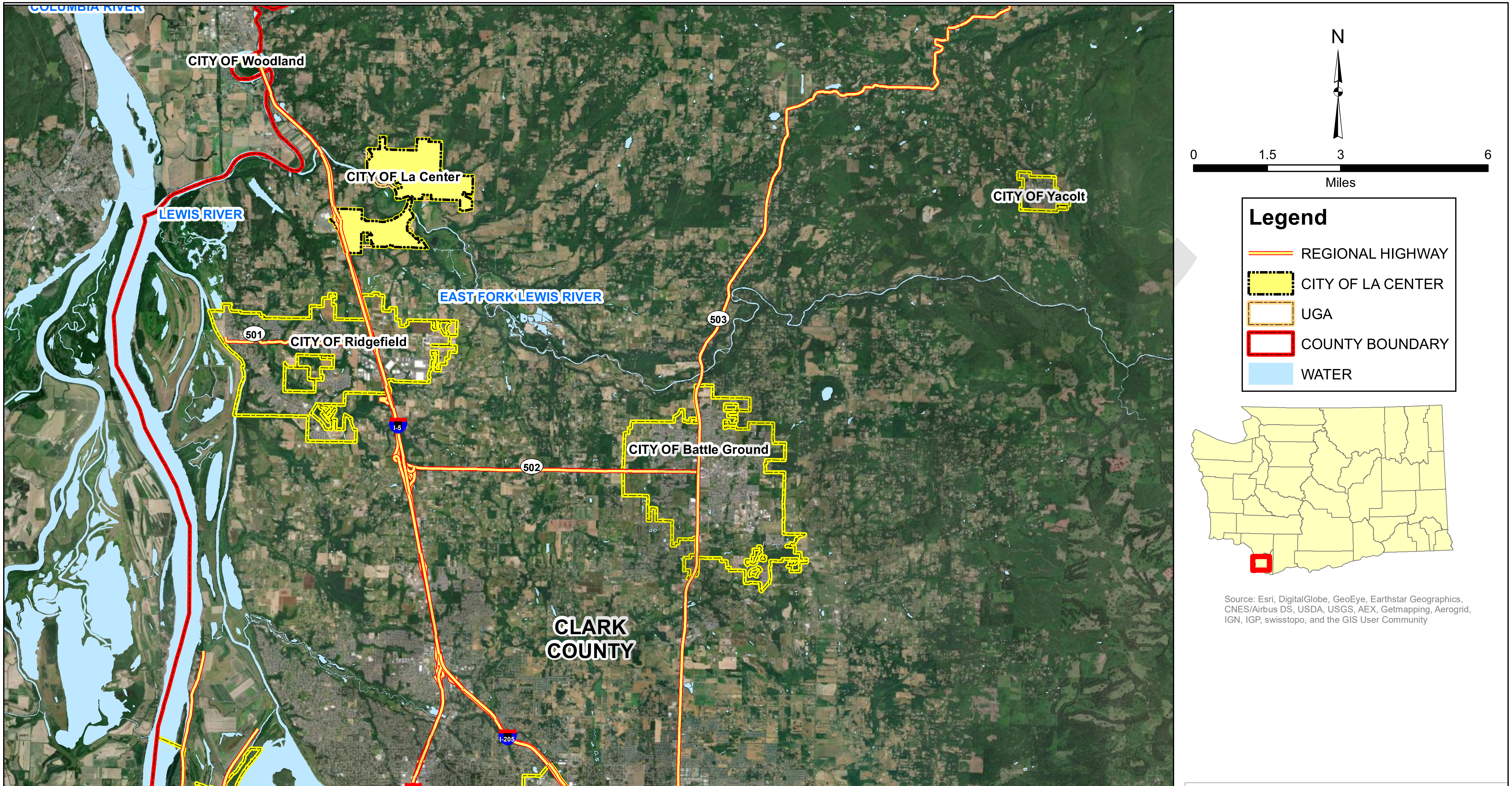

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FIGURE 1-1
VICINITY MAP

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Plan reevaluated the timeline for the WWTP improvements based on the updated growth projection

<u>Draft General Sewer Plan Update</u>, July 2019, City of La Center

The *2019 Draft General Sewer Plan update* was intended to address the Ecology's comments on the *2013 General Sewer Plan.* The *2019 Draft Plan* was prepared to establish the future service area, estimate flow and loading projections, analyze the sewer collection and treatment systems and their operation, and recommend improvements to correct deficiencies and meet future service needs. The analysis and recommended improvements in the *2019 Draft Plan* are primarily based on the previous work in *2008 Facility Plan* and *2013 General Sewer Plan.* This Plan was never completed or approved by Ecology.

<u>Clark County Coordinated Water System Plan Update</u>, November 2011, Clark County Water Utility Coordinating Committee

The *2011 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.

CHAPTER 2

LAND USE, POPULATION PROJECTIONS AND SERVICE AREA CHARACTERISTICS

INTRODUCTION

This chapter defines the sewer service area evaluated in this Plan and presents population projections and land use. Various natural features of the service area are discussed, including topography, geology, soils, climate, sensitive areas, floodplains, wetlands, air quality, and surface and ground water resources. Information regarding the public utilities available in the area is also presented.

The planning period for this Plan is from 2023 through 2043 to provide consistency with population projections and other planning documents.

SERVICE AREA

The City of La Center is located approximately 15 miles north of Vancouver, Washington, 3 miles east of Interstate 5, along the north bank of East Fork of the Lewis River.

The sewer service area for the City of La Center (study area for this Plan) consists of the City's Urban Growth Area (UGA) and City limits as shown on Figure 2-1. The City uses the term UGA to specifically refer to the areas outside of the current city limits that are within the UGA. Clark County currently has zoning and land use jurisdiction over these unincorporated areas. The City limits encompass approximately 1,697 acres and the UGA consists of an additional 146 acres, for a total service area of 1,843 acres.

NATURAL FEATURES OF THE SEWER SERVICE AREA

Various natural features of the service area are discussed below, including climate and precipitation, soils, geology, steep slope and site-sensitive areas, such as floodplains, and wetlands. The natural features of the service area will have an impact on the design and siting of wastewater collection and treatment facilities.

TOPOGRAPHY

The topography is dominated by the East Fork Lewis River (the River), which essentially splits the study area into distinct north and south sections. The study area is well defined by drainage ways flowing to the river. In general, the area north of the river is less fragmented by the drainage ways, the most significant is Brezee Creek. The area on the south side of the river is extremely fragmented by McCormick Creek and its side drainage ways. Figure 2-2 shows the topography of the La Center area based on United States Geological Survey (USGS) maps. Elevations vary from approximately 40 feet at the Wastewater Treatment Plant to 400 feet within the Urban Growth Area.

SOILS AND GEOLOGY

La Center lies in the Portland basin physiographic region, near the junction of major geologic units of sedimentary rocks of the Upper Tertiary and volcanic rocks of the Lower Tertiary periods. The basin is layered with well-sorted sand, clay and gravel from the Missoula floods.

Surface geology will determine the stability, strength, and permeability of soils, which impacts the suitability of land for building construction and on-site sewage systems. Alluvial deposits composed of sand and gravel have been identified in the Columbia and East Fork Lewis River floodplain. Figure 2-3 provides a map of the soil types, based on the United States Department of Agriculture, Soils Conservation Service. As shown, there are a wide variety of soils in the area, predominantly Gee Silt Loam or Hillsboro Silt Loam. Silt Loams are a mixture of silt and clay. They are moderately stable cohesive soils, medium-textured, moderately well-drained, and of moderate permeability in the upper layer, and lower permeability in the lower layers. These soils have slow surface runoff and generally low erosion hazards.

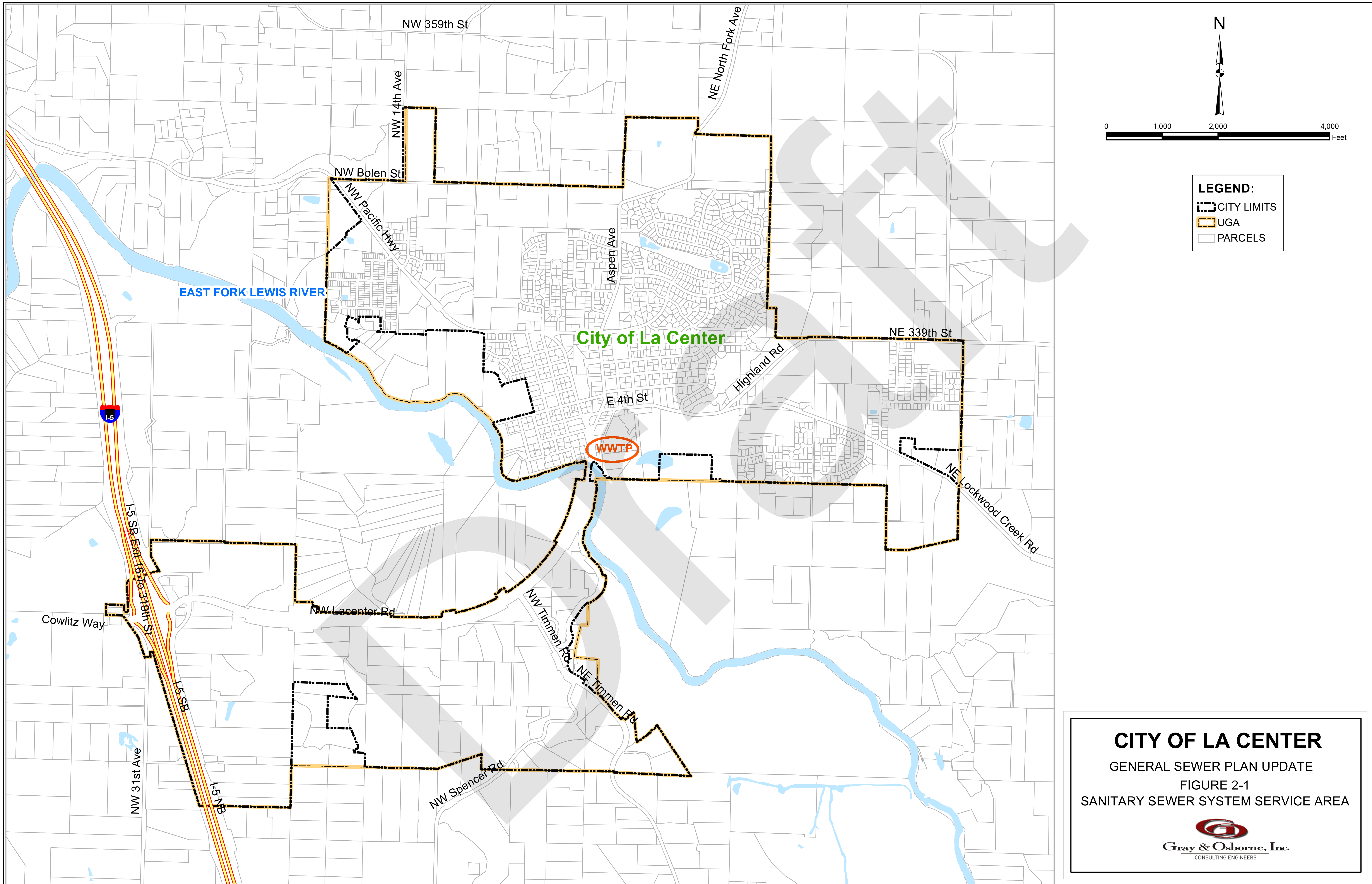
WETLANDS

Wetlands are defined by EPA as areas that are inundated with water for at least part of the year. The U.S. Fish and Wildlife Service defines wetlands as those areas that have characteristics such as hydrophyte plants, hydric soils, and frequent flooding. Wetlands support valuable and complex ecosystems and, consequently, development is severely restricted if not prohibited in most wetlands. Figure 2-4, Wetlands Map, shows that the majority of the wetlands are along the East Fork of the Lewis River and along the area called “La Center Bottoms.”

SENSITIVE AREAS

Areas susceptible to erosion and landslides are shown in Figure 2-5. They are generally steeply sloped areas associated with the various drainage ways to the river, or areas that are susceptible to landslides due to topographic, geologic, and/or hydrologic conditions. These areas are designated by La Center and the County as unsuitable for most structures.

The existing treatment plant is located just above the 100-year floodplain of the river. The 100-year flood elevation for the river is at an elevation 33 feet above mean sea level.



CITY OF LA CENTER

GENERAL SEWER PLAN UPDATE

FIGURE 2-1

SANITARY SEWER SYSTEM SERVICE AREA

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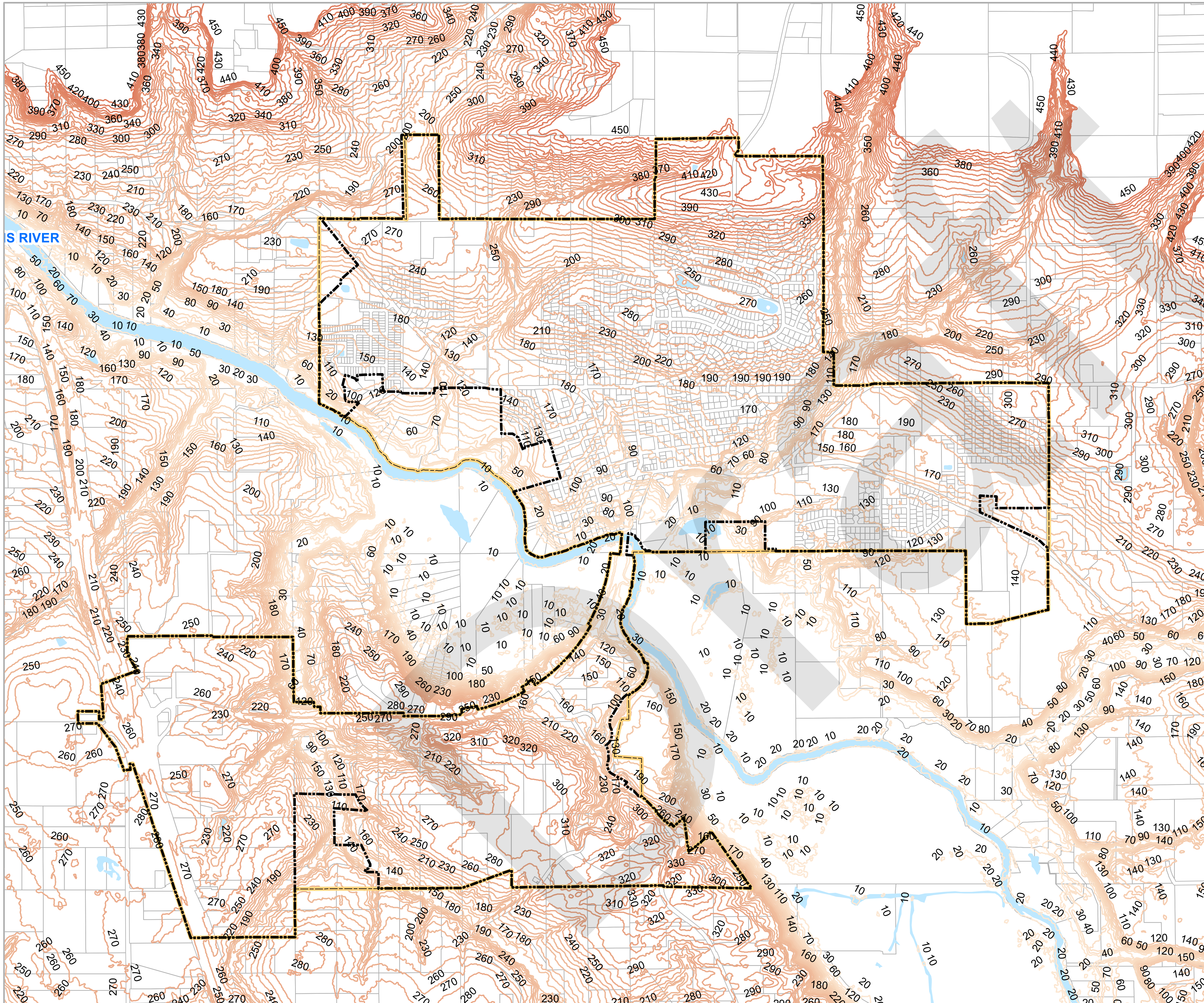

N

0 1,000 2,000 4,000 Feet

LEGEND

- CITY LIMITS
- UGA
- 10' CONTOURS - USGS (INTERPOLATED)
- WATER
- PARCELS

Source: USGS 10m DEM 2019

CITY OF LA CENTER

GENERAL SEWER PLAN UPDATE

FIGURE 2-2

TOPOGRAPHY

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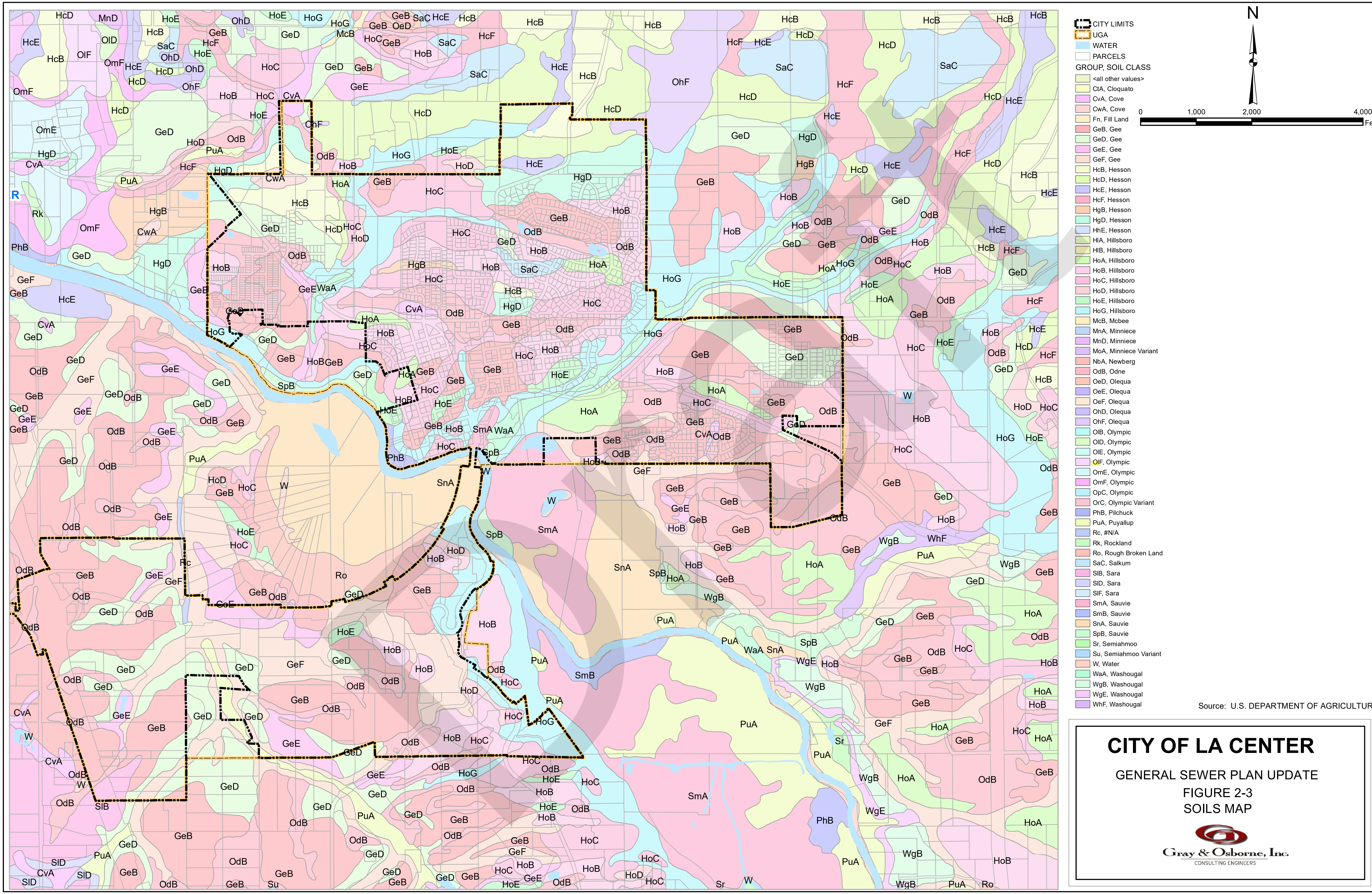

CITY OF LA CENTER

GENERAL SEWER PLAN UPDATE
FIGURE 2-3
SOILS MAP

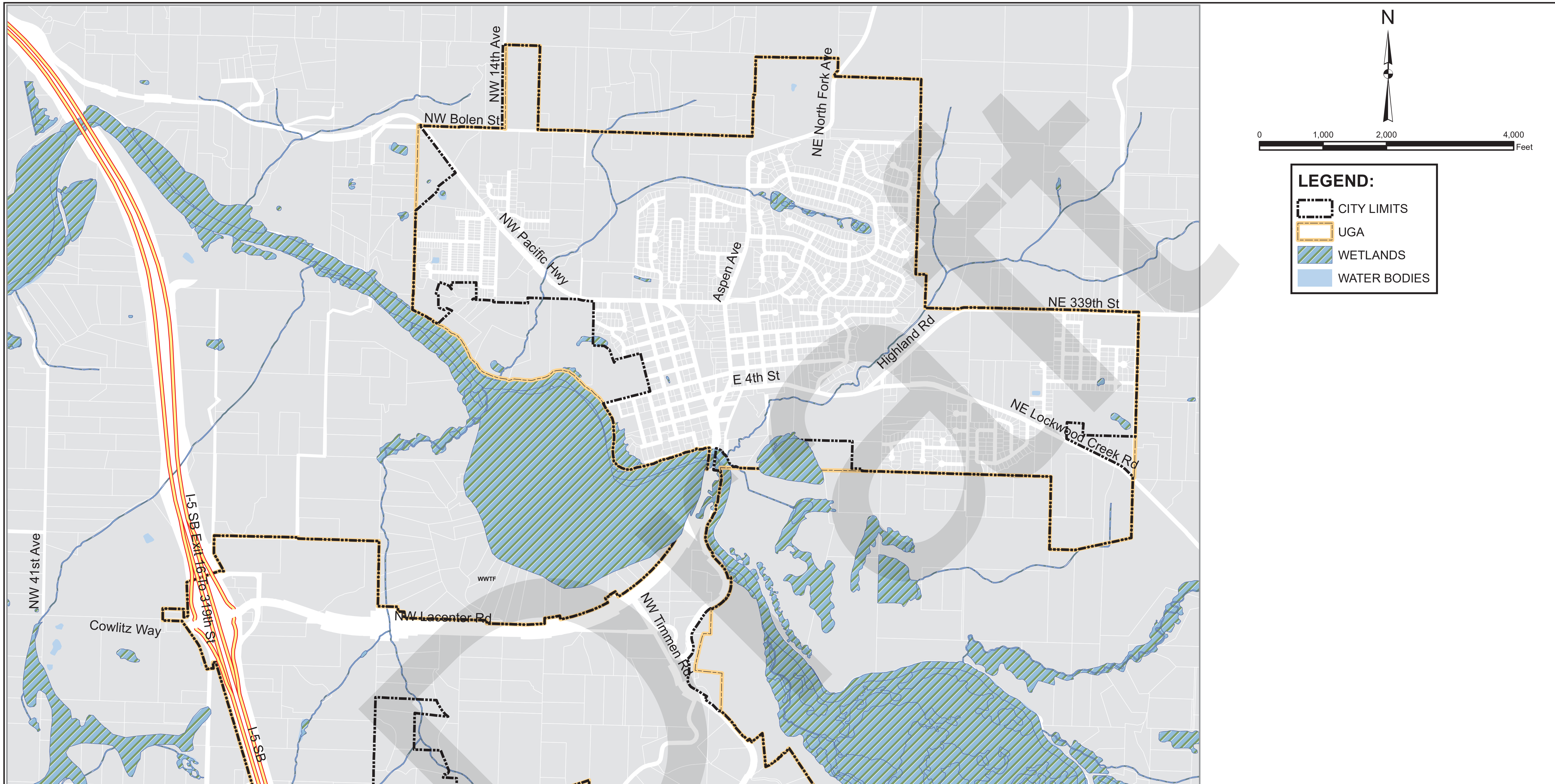

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FIGURE 2-4

WETLANDS

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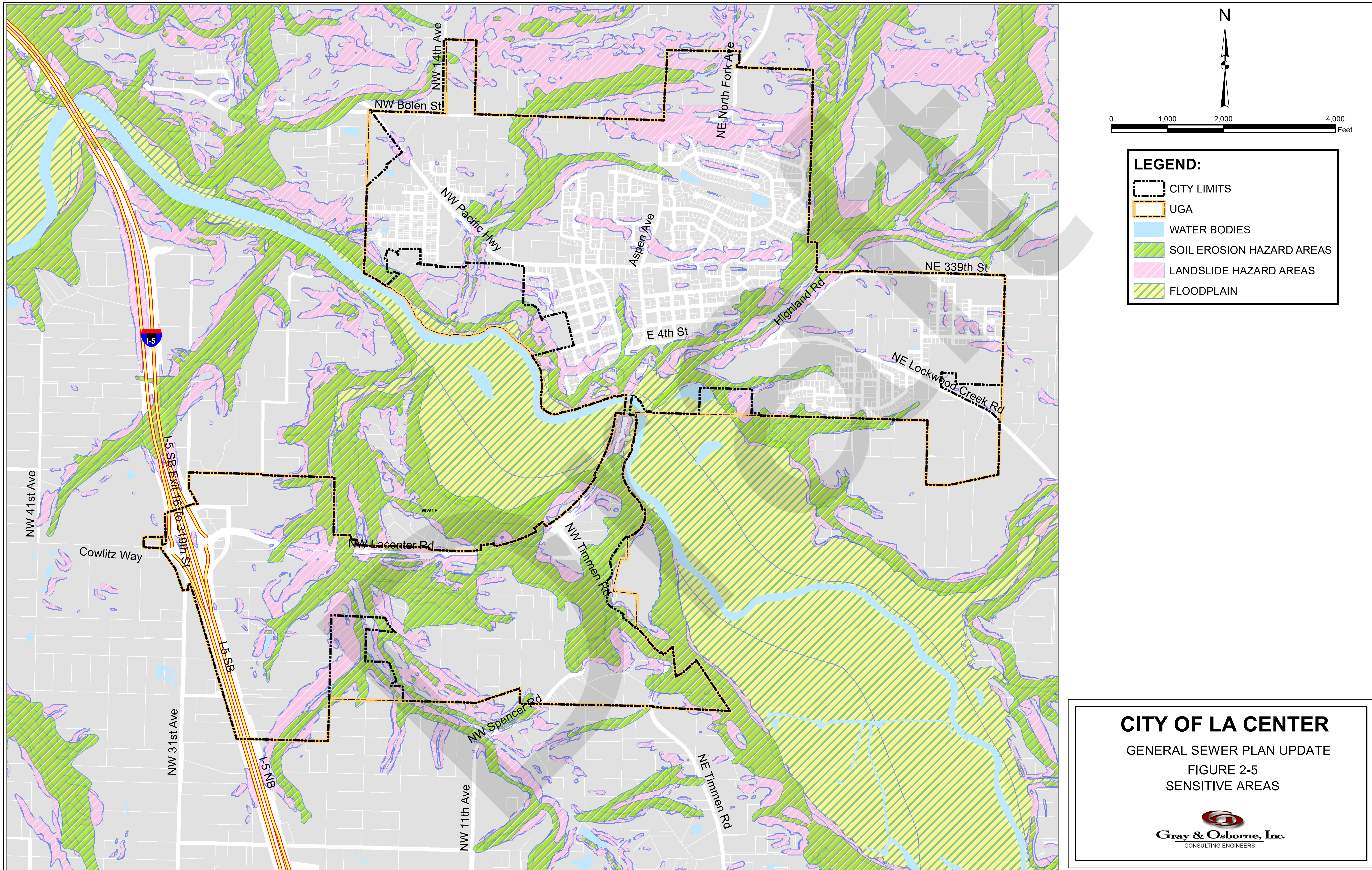

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GENERAL SEWER PLAN UPDATE

FIGURE 2-5
SENSITIVE AREAS

CLIMATE

La Center has a mild climate typical of the valleys between the coast range and Cascade Range in Oregon and Washington, with local weather occasionally influenced by the effects of the Columbia River Gorge, bringing in heat and cold from the East. The National Oceanic and Atmospheric Administration (NOAA) collects data from a nearby weather station in La Center. Climate data from this station averaged over a 40-year period is summarized in Table 2-1. Winters are wet and mild. Snow falls occasionally, but usually melts within a few days. Precipitation averages approximately 70 inches annually, most of which falls in the 6-month period between November and April.

TABLE 2-1

Average Precipitation and Temperature(1)

Month	Average Precipitation (inches)	Average Temperature (°F)
January	9.9	40.2
February	7.7	42.8
March	7.7	47.0
April	5.7	51.9
May	3.8	57.3
June	2.7	62.3
July	0.9	65.2
August	1.1	63.3
September	2.8	56.7
October	5.9	48.6
November	11.0	41.8
December	9.3	39.1
Annual Total	68.5	**N/A**
Annual Average	**N/A**	**51.3**

(1) Climate data is from the *Merwin Dam weather station, NOAA Climatological Data,* for the years 1982 through 2022.

GROUNDWATER

Groundwater levels in the study area are generally very high. During wet weather, the groundwater elevation is only a few feet below the ground surface. One result is numerous springs discharging in the drainages flowing to the East Fork Lewis River. The high groundwater levels result in infiltration and inflow.

SURFACE WATER

The City of La Center and its UGA are located in the East Fork Lewis River Drainage Basin which is a portion of the Water Resource Inventory Area 27 (WRIA 27). The East Fork Lewis River flows from the southwest portion of the Gifford Pinchot National Forest through Clark County past the town of Yacolt and City of La Center before joining the Lewis River main stem below the City of Woodland. The La Center WWTP discharges effluent to the East Fork Lewis River at latitude 45º51'34" N and longitude 122º40'13" W, at approximately River Mile 3.2.

WATER SYSTEM

La Center's water system is shown in Figure 2-6. The Clark County PUD operates a water system that supplies potable water to the City. The PUD has added wells and reservoirs and continue to increase the capacity of the water system to provide service to over 200 square miles including the La Center UGA. The source of the water is from wells that are both inside and outside of the City's UGA and are part of a regional system. Chlorination was provided for disinfection.

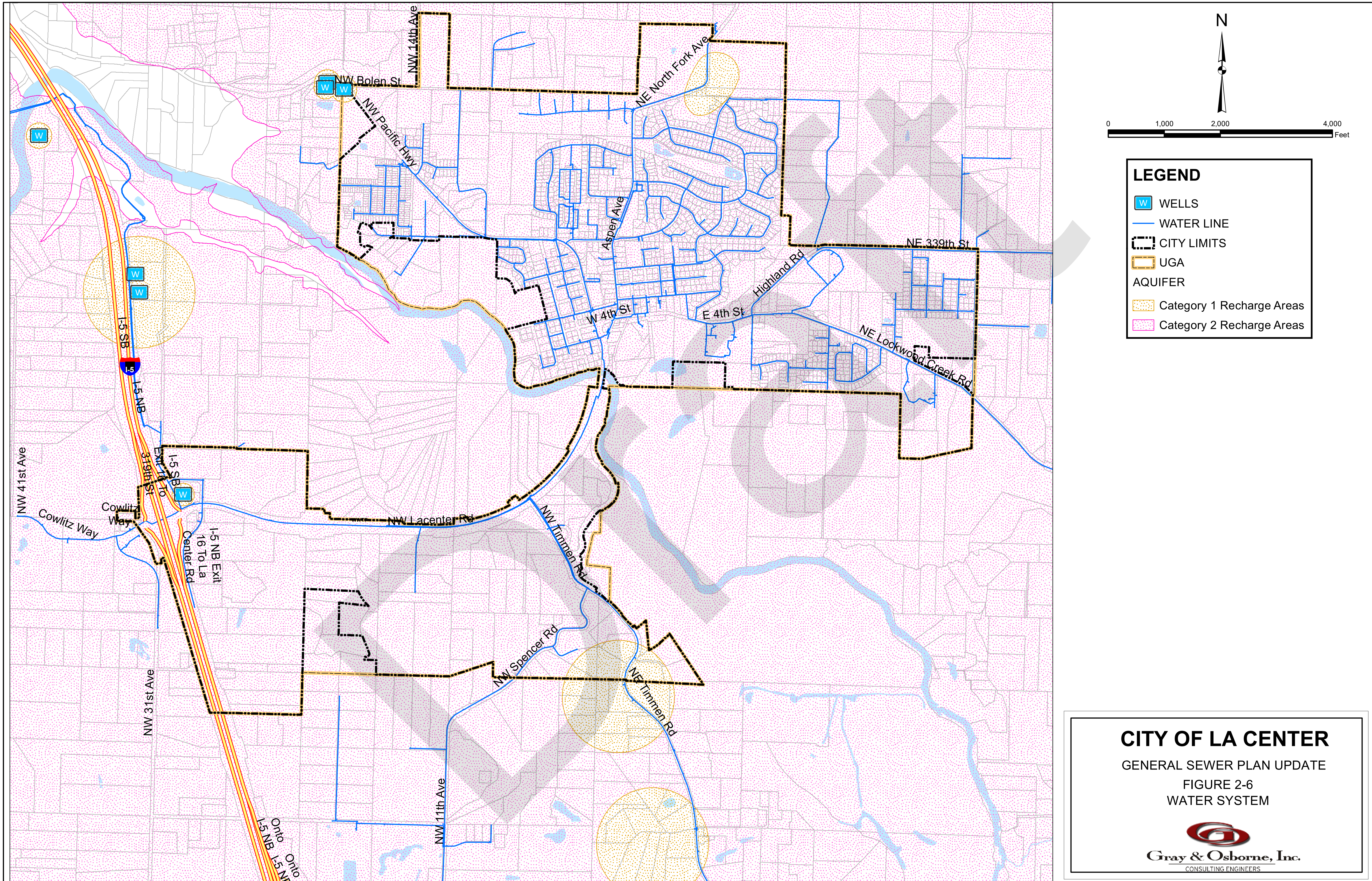
ZONING AND LAND USE

Figure 2-7 provides a map of zoning for the service area. The breakdown of the zoning can be seen in Table 3-2. The majority of the area is residential, with commercial activity concentrated in the downtown core area and industrial development concentrated along I-5.

TABLE 2-2

Existing Zoning in Service Area

Land Use Designation	Land Use Category	Acreage	% of Total Acreage
LDR-7.5	Low Density Residential	994.2	53.9%
MDR-16	Medium Density Residential	81.4	4.4%
R-12	Residential	12.2	0.7%
R1-6	Single-Family Residential	29.8	1.6%
R1-7.5	Single-Family Residential	52.8	2.9%
R1-10	Single-Family Residential	8.1	0.4%
R1-20	Single-Family Residential	32.5	1.8%
JP	Junction Plan	260.6	14.1%
RP	Residential/Professional	73.6	4.0%
MX	Mixed Use	58.4	3.2%
C-1	Downtown Commercial	27.8	1.5%
UP	Urban Public Facilities	101.9	5.5%



NW 14th Ave
NW Bolen St
NW Pacific Hwy
NE North Fork Ave
Aspen Ave
NE 339th St
Highland Rd
E 4th St
W 4th St
NE Lockwood Creek Rd
I-5 SB
I-5 NB
I-5
319th St
I-5 SB Exit 16 To
Cowlitz Way
NW 41st Ave
NW Lacenter Rd
NW Timmen Rd
I-5 NB Exit 16 To La Center Rd
NW Spencer Rd
NE Timmen Rd
NW 31st Ave
NW 11th Ave
I-5 NB Onto
I-5 NB Onto
N
0
1,000
2,000
4,000
Feet
LEGEND
WELLS
WATER LINE
CITY LIMITS
UGA
AQUIFER
Category 1 Recharge Areas
Category 2 Recharge Areas
CITY OF LA CENTER
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FIGURE 2-6
WATER SYSTEM
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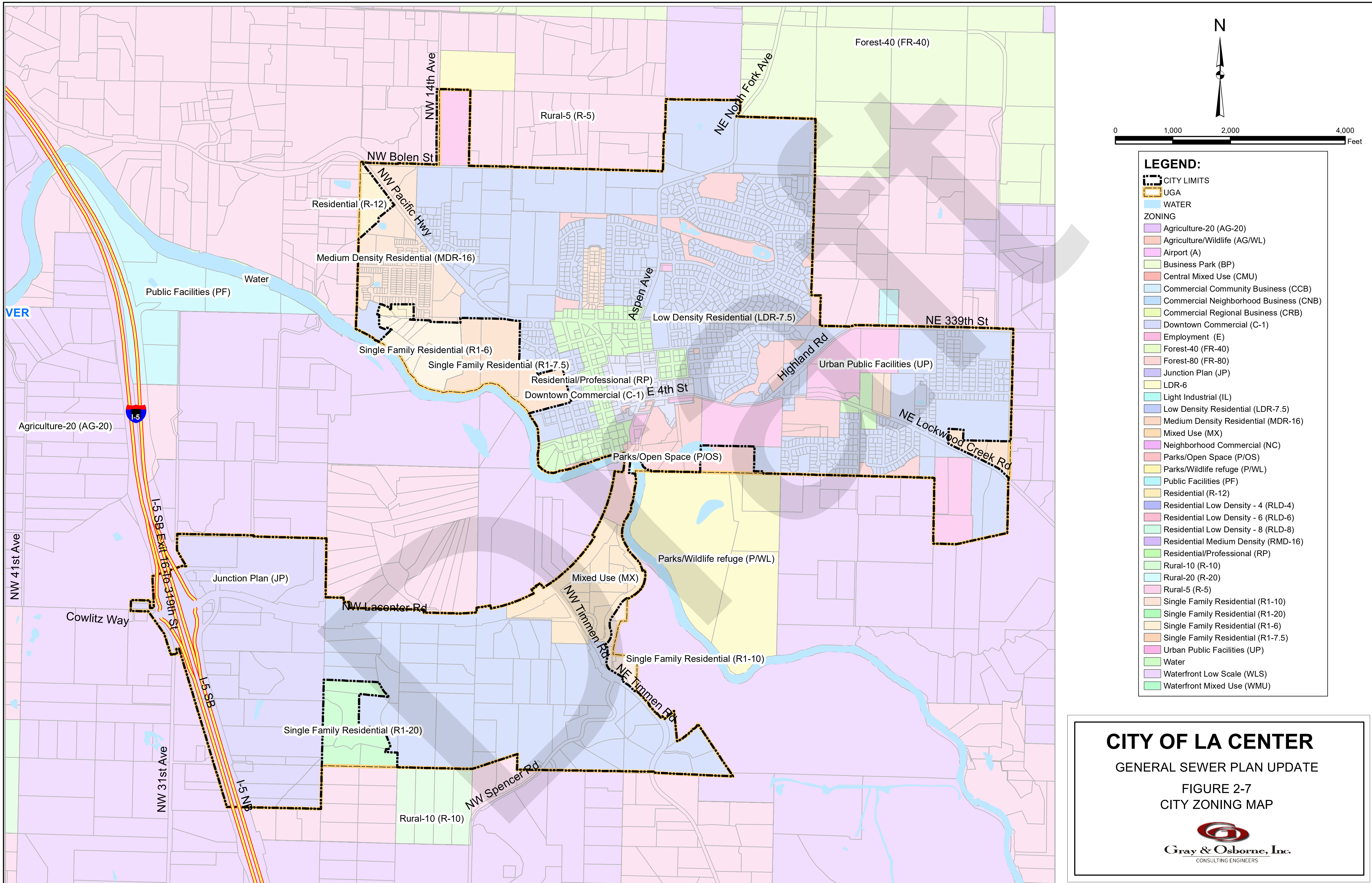
Forest-40 (FR-40)
Rural-5 (R-5)
NW 14th Ave
NE North Fork Ave
NW Bolen St
NW Pacific Hwy
Residential (R-12)
Medium Density Residential (MDR-16)
Water
Public Facilities (PF)
VER
Aspen Ave
Low Density Residential (LDR-7.5)
NE 339th St
Highland Rd
Single Family Residential (R1-6)
Single Family Residential (R1-7.5)
Urban Public Facilities (UP)
Residential/Professional (RP)
Downtown Commercial (C-1)
E 4th St
NE Lockwood Creek Rd
Agriculture-20 (AG-20)
I-5
Parks/Open Space (P/OS)
I-5 SB Exit 16 to 319th St
NW 41st Ave
Junction Plan (JP)
Parks/Wildlife refuge (P/WL)
Mixed Use (MX)
NW Lacenter Rd
Cowlitz Way
NW Timmen Rd
Single Family Residential (R1-10)
NE Timmen Rd
I-5 SB
Single Family Residential (R1-20)
NW 31st Ave
I-5 NB
NW Spencer Rd
Rural-10 (R-10)
N
0
1,000
2,000
4,000
Feet
LEGEND:
CITY LIMITS
UGA
WATER
ZONING
Agriculture-20 (AG-20)
Agriculture/Wildlife (AG/WL)
Airport (A)
Business Park (BP)
Central Mixed Use (CMU)
Commercial Community Business (CCB)
Commercial Neighborhood Business (CNB)
Commercial Regional Business (CRB)
Downtown Commercial (C-1)
Employment (E)
Forest-40 (FR-40)
Forest-80 (FR-80)
Junction Plan (JP)
LDR-6
Light Industrial (IL)
Low Density Residential (LDR-7.5)
Medium Density Residential (MDR-16)
Mixed Use (MX)
Neighborhood Commercial (NC)
Parks/Open Space (P/OS)
Parks/Wildlife refuge (P/WL)
Public Facilities (PF)
Residential (R-12)
Residential Low Density - 4 (RLD-4)
Residential Low Density - 6 (RLD-6)
Residential Low Density - 8 (RLD-8)
Residential Medium Density (RMD-16)
Residential/Professional (RP)
Rural-10 (R-10)
Rural-20 (R-20)
Rural-5 (R-5)
Single Family Residential (R1-10)
Single Family Residential (R1-20)
Single Family Residential (R1-6)
Single Family Residential (R1-7.5)
Urban Public Facilities (UP)
Water
Waterfront Low Scale (WLS)
Waterfront Mixed Use (WMU)
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FIGURE 2-7
CITY ZONING MAP
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TABLE 2-2 – (continued)

Existing Zoning in Service Area

Land Use Designation	Land Use Category	Acreage	% of Total Acreage
PF	Public Facilities	0.4	0.02%
P/OS	Parks/Open Space	108.3	5.9%
Water	Water	1.1	0.1%
AG-20	Agriculture-20	0.03	0.002%
Total		**1,843**	**100%**

POPULATION PROJECTIONS

CURRENT POPULATION

The 2022 census data were released in April 2022 and are accounted for in the subsequent tables and calculations, as summarized in Table 2-3.

TABLE 2-3

City Historical Population 2000 to 2022

Year	Population(1)	Additions/Subtractions	Annual Growth Rate
2000	1,743		
2001	1,834	91	5.2%
2002	1,909	75	4.1%
2003	1,952	43	2.3%
2004	2,076	124	6.4%
2005	2,174	98	4.7%
2006	2,463	289	13.3%
2007	2,504	41	1.7%
2008	2,572	68	2.7%
2009	2,607	35	1.4%
2010	2,955	348	13.3%
2011	2,980	25	0.8%
2012	3,026	46	1.5%
2013	3,052	26	0.9%
2014	3,077	25	0.8%
2015	3,108	31	1.0%
2016	3,144	36	1.2%
2017	3,218	74	2.4%
2018	3,281	63	2.0%
2019	3,404	123	3.7%

TABLE 2-3 – (continued)

City Historical Population 2000 to 2022

Year	Population(1)	Additions/Subtractions	Annual Growth Rate
2020	3,424	20	0.6%
2021	3,605	181	5.3%
2022	3,835	230	6.4%
Average			**3.7%**

The population has increased an average of approximately 3.7 percent per year over the past 22 years and 2.4 percent over the past 10 years. This population increase has been a result of development within the existing city limits as well as some annexations.

PROJECTED FUTURE POPULATION

The 2016 *City Comprehensive Plan* effectively utilized an annual population growth rate of 4.3 percent. After consulting with City staff, a 4.0 percent annual population growth rate will be used in this report to better reflect observed growth pattern. This growth rate is also in line with the County projected annual growth rate of 4.4 percent for La Center UGA.

It is considered highly unlikely that all residentially zoned areas in the City of La Center will be redeveloped over the next 20 years. However, the proposed growth rate will allow for the service to developed areas presently outside the city limits within the UGA, that are presently served by septic tanks or other types of on-site treatment and disposal. These areas may be annexed to the City, or merely receive sewer service.

Table 2-4 shows the projected future population at 5-year increments for the 20-year planning period for the City of La Center based on the 4.0 percent annual growth rate.

TABLE 2-4

City of La Center Projected Population

Year	City Population(1)
2023	3,988
2028	4,852
2033	5,904
2038	7,183
2043	8,739

(1) Includes city limits and areas that could potentially be annexed by the City.

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 the City. 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. Chapters 6 and 7 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 City of La Center

WWTF NPDES Permit WA0023230 and fact sheet are attached as Appendix B. The permit was issued in 2015 and expired January 31, 2021, but is still considered to be in effect. The City's current NPDES permit effluent limitations are summarized in Table 3-1.

TABLE 3-1

Summary of City WWTF NPDES Permit Effluent Limits

Parameter	Average Monthly	Average Weekly
Biochemical Oxygen Demand (5-day) (BOD_5)	30 milligrams/liter (mg/L) 173 lbs/day (phase 1A) 260 lbs/day (phase 1B) 85% removal of influent BOD_5	45 mg/L 260 lbs/day (phase 1A) 390 lbs/day (phase 1B)
Total Suspended Solids (TSS)	30 mg/L 161 lbs/day (phase 1A) 237 lbs/day (phase 1B) 85% removal of influent TSS	45 mg/L 242 lbs/day (phase 1A) 356 lbs/day (phase 1B)
Parameter	**Minimum**	**Maximum**
pH	6.0 standard units	9.0 standard units
Parameter	**Monthly Geometric Mean**	**7-Day Geometric Mean**
Fecal Coliform Bacteria	100/100 milliliter (mL)	200/100 mL
Phase 1A Limits	**Average Monthly**	**Maximum Daily**
Total Ammonia (as NH3-N)	3.6 mg/L (June - October) 13.9 mg/L (November - May)	8.1 mg/L (June - October) 31.3 mg/L (November - May)
Phase 1B Limits	**Average Monthly**	**Maximum Daily**
Total Ammonia (as NH3-N)	3.0 mg/L (June - October) 10.9 mg/L (November - May)	6.8 mg/L (June - October) 24.7 mg/L (November - May)

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

- Maximum month flow – 0.69 mgd (phase 1a), 1.04 mgd (phase1b)
- Peak Day Flow– 1.29 mgd (phase 1a), 1.94 mgd (phase1b)
- Maximum month BOD_5 loading – 1,297 lbs/day (phase 1a), 1,804 lbs/day (phase1b)
- Maximum month TSS loading – 1,070 lbs/day (phase 1a), 1,581 lbs/day (phase1b)

- Maximum month Ammonia (total as Nitrogen) loading – 194 lbs/day (phase 1a), 292 lbs/day (phase1b)

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 are 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 currently no SWDPs issued to facilities in the City's service area.

The NPDES Permit requires the City is to submit one Industrial User Survey per permit cycle. The survey must list all existing, new and proposed significant industrial users (SIUs) and potential significant industrial users (PSIUs) discharging or proposing to discharge to the City's sewer system. The NPDES Permit states that the City must develop a list of SIUs and PSIUs by means of a telephone book search, a water utility billing records search and a physical reconnaissance of the service area.

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.

The East Fork Lewis River and its tributaries are listed on the state's polluted waters list (303d list) for warm water temperatures and fecal coliform bacteria problems. However, the Department of Ecology is still developing the TMDL Implementation Plan.

FEDERAL AND STATE STANDARDS FOR USE OR DISPOSAL OF SLUDGE

The City treats biosolids to Class A standards by mechanical drying of sludge. An evaluation of alternatives for the City's future biosolids treatment and management is provided in Chapter 8.

The generation and use of biosolids, and the disposition of solid waste in general generated from wastewater treatment plants (WWTFs), 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 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 the City's case, this requirement is met by screening the wastewater at the municipality's treatment plant.
- 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.

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 3-2. 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 3-2. 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.

TABLE 3-2

Allowable Biosolids Trace Pollutant Concentrations for Land Application(1)

Element	Symbol	Ceiling Concentration (mg/kg)(1)	EQ Limit (mg/kg)(2)
Arsenic	As	75	41
Cadmium	Cd	85	39
Copper	Cu	4,300	1,500
Lead	Pb	840	300
Mercury	Hg	57	17
Molybdenum	Mo	75	75(4)
Nickel	Ni	420	420
Selenium	Se	100	100
Zinc	Zn	7,500	2,800

(1) WAC-173-308-160 Table 1.
(2) WAC-173-308-160 Table 3.

Cumulative and annual trace pollutant loading rates are designated for nine heavy metals (Table 3-3). 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 3-3, the cumulative loading limits will not be a concern for the City's land application sites.

TABLE 3-3

Biosolids Pollutant Loading Limits for Land Application(1)

Pollutant	Symbol	Cumulative Pollutant Loading Rate (Dry Weight Basis), kg/ha
Arsenic	As	41
Cadmium	Cd	39
Copper	Cu	1500
Lead	Pb	300
Mercury	Hg	17
Nickel	Ni	420
Selenium	Se	100
Zinc	Zn	2800

(1) WAC-173-308 -160 Tables 2.

It is possible that future regulations will be imposed for microconstituents, or trace organic compounds in biosolids. There is some concern regarding leaching from biosolids and into ground, surface, an ultimately drinking waters. Many communities in the U.S., particularly in the Midwest and Northeast where environmental groups and the media are raising concerns, are postponing major capital expenditures associated with biosolids due to the uncertainty associated with this issue. EPA is in the process of evaluating the risks of trace organic compounds in biosolids, in particular PFAS (perfluoroalkyl substances), colloquially knows as "forever chemicals" for their persistence. Many industrial and consumer products are known to contain, and serve as sources of, PFAS, including carpet cleaning and treatment products, stain resistant and porous waterproofing materials, treated paper food packaging, non-stick cookware, treated floor waxes and sealants, cosmetics and firefighting foams.

Although there may be some new regulations associated with biosolids, and impacts to how they are managed, ultimately, implementation of PFAS source control, implemented for commercial dischargers and for consumer products, is expected to be the major impact of the risk analysis. The PFAS issue does present some uncertainty for biosolids planning for the City. None of the Class A or Class B treatment options considered in

Chapter 8 would significantly reduce PFAS concentrations. Only combustion and oxidation processes like incineration and pyrolysis have been shown to remove PFAS.

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.

A higher level of treatment, known as a process to further reduce pathogens (PFRP), will permit biosolids to meet Class A pathogen reduction requirements. 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 in Table 3-2 and the vector attraction reduction requirements as described below in Table 3-6.

The City's WWTP meets the Class B standards through digestion, and follows that process with mechanical drying to meet Class A standards. The pathogen reduction requirements appropriate for these two processes (aerobic digestion for Class B and mechanical drying for Class A) are shown in Tables 3-4 and 3-5, respectively.

TABLE 3-4

Class B Pathogen Reduction Requirements Relevant for Aerobic Digestion

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.
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.

TABLE 3-5

Class A Pathogen Reduction Requirements Relevant for Heat Drying

All Alternatives	Fecal coliform <1,000 MPN per gram total solids, or salmonella <3 MPN per 4 grams total solids.
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.

(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. Table 3-6 summarizes the Vector Attraction Alternatives relevant for the City's biosolids.

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 3-6

Vector Attraction Reduction Alternatives Relevant for Aerobic Digestion and Heat Drying

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.	Treatment by drying. Can include unstabilized primary wastewater solids. Total solids >90 percent before mixing with other materials.

(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.

Land Application Permitting

WAC-173-308-310 lists permitting requirements for municipalities managing biosolids. The primary permit required for biosolids management activities is the Washington 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 environmental 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.

Biosolids 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 3-7. Based on its rate of biosolids production, the City has a minimum monitoring frequency of quarterly.

TABLE 3-7

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, jurisdictions, 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.

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.

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.

<u>Regulatory Criteria for Land Application Siting</u>

The WAC-173-308 and EPA 503 regulations have specific requirements for Class B biosolids application sites, including buffers, prohibited areas. There may also be local land use regulations or policies that apply in specific areas. Criteria are published in the State 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 both Class A and Class B biosolids is prevention of leaching of nitrate to groundwater, a key parameter in determining the agronomic rate for land application is the available nitrogen content. Maintaining a cover crop is absolutely essential for a biosolids 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.

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 Inflow and Infiltration (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 Lewis River Basin support a variety of fish and wildlife species, including several that are currently listed as Threatened or Endangered under the Federal Endangered Species Act (ESA).

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 WWTF 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 WWTF, 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. The City falls in the jurisdiction of the Southwest Clean Air Agency. The Southwest Clean Air Agency (SWCAA) is responsible for enforcing federal, state and local outdoor air quality standards and regulations in Clark, Cowlitz, Lewis, Skamania and Wahkiakum counties of southwest Washington State.

The City's WWTP has a SWCAA Air Permit requiring it to meet certain limitations on emissions from its emergency generator.

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 regulation defines a facility plan as described in federal regulations, 40 CFR Part 35, as an engineering report.

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 WWTF 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 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 3-8 presents Ecology criteria for designation of WWTFs into three reliability classes based on the nature or their receiving water. Per the NPDES Permit and fact sheet, the City’s WWTF has a reliability classification of Class II. Reliability criteria for WWTF in Class II are presented in Table 3-9.

TABLE 3-8

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 3-9

Reliability Requirements for Class II WWTFs

WWTF 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.

TABLE 3-9 – (continued)

Reliability Requirements for Class II WWTFs

WWTF Component	Class II Requirements
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/Mechanical Aerators or Rotors. 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.
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 II plant. This operator must be in responsible charge of the day-to-day operation of the wastewater treatment plant.

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 TMDL.

The State adopted revised water quality standards in March 2022. 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.

Water Quality Classifications for Fresh Waters

The City's outfall discharges to the Lewis River at mile 3.2. The river has established use designations upstream from Mason Creek to the headwaters, but not between Mason Creek and the mouth. The Lewis River at the outfall discharge location is classified in WAC 173-201A-602 as having the following uses:

- Aquatic Life Uses: Core Summer Salmonid Habitat
- Recreation Use: Primary contact recreation
- Water Supply Uses: Domestic water, industrial water, agricultural water, and stock water
- Miscellaneous Uses: Wildlife habitat, harvesting, commerce/navigation, boating, and aesthetics

Per WAC 173-201A-600, "all surface waters of the state not named in Table 602 are to be protected for the designated uses of: Salmonid spawning, rearing and migration; primary contact recreation; domestic, industrial, and agricultural water supply; stock watering; wildlife habitat; harvesting; commerce and navigation; boating; and aesthetic values." Additional protections apply if the waters are within national parks or wilderness areas, or are tributaries to water designated as core summer salmonid habitat. These conditions do not apply to the river at the outfall of the La Center WWTP. The critical use designation for purposes of effluent discharge is, therefore, salmonid spawning, rearing, and migration.

The East Fork Lewis River and its tributaries are on the 2008 303(d) list of impaired water bodies. The parameters of concern are Fecal Coliform and instream temperatures. There are no other nearby point source outfalls. Significant nearby non-point sources of pollutants include livestock and onsite septic systems.

Water quality criteria for the East Fork Lewis River at the City of La Center WWTP Outfall are shown in Table 3-10.

TABLE 3-10

Water Quality Criteria for East Fork Lewis River at the City of La Center WWTP Outfall

Parameter	Surface Water Criteria Value
Temperature	16 degrees C (7-day average of daily maximum temperatures)
Dissolved Oxygen	>9.5 mg/l (lowest 1-day minimum)
Turbidity	<5 NTU over background (background <50 NTU) <10 percent increase over background (background >50 NTU
Dissolved Gas	<110 percent of saturation at any point of sample collection
pH	Not outside the range of 6.5 to 8.5 standard units, with no human-caused variation >0.2 standard unit
Bacteria	**Primary Contact Recreation:** *Fecal coliform organism levels must not exceed a geometric mean value of 100 colonies/100 mL, with not 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 200 colonies/100 mL.*

The water quality standards also have narrative criteria regarding toxic, radioactive, otherwise deleterious materials, or materials that impair aesthetics. These materials are prohibited in concentrations that affect aquatic life, human health, or impair aesthetics.

Numeric criteria for 29 toxic substances are listed in WAC 173-201A-240. Criteria are listed for both an acute and chronic basis and for certain substances (e.g., metals, chlorine, and ammonia), the criteria must be calculated as a function of receiving water pH, hardness, and whether salmonids are present.

Anti-Degradation Policy

The State's anti-degradation policy per WAC 173-201A-200 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.

Discharge Permits

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.

Compliance Schedules

When it is not possible to achieve compliance with the standards in WAC 173-201A on an immediate basis, Ecology may issue an order with a compliance schedule to allow for further water quality studies, implementation of best management practices, or construction of necessary treatment capability. Compliance schedules may only be issued for existing discharges.

Mixing Zone

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, the potential for violating the water quality standards at the edge of the mixing zone and any necessary effluent limits are determined (see Table 3-11). Steady-state models are the most frequently used tools for conducting mixing zone analyses. The mixing zones are defined in the Permit as:

- Acute mixing zone – The width of the acute mixing zone is limited to 1/4 of the river width (21 feet). The length of the authorized acute mixing zone extends 10 feet upstream and 30 feet downstream of the outfall. For Phase 1A, the mixing at the acute boundary based on volumetric flow limits is 1.8:1 in the "summer," and 3.3:1 in the "winter." For Phase 1B, the maximum allowable chronic mixing ratio is 1.5:1 in the "summer", and 2.6:1 in the "winter."

- Chronic mixing zone – The width of the chronic mixing zone is limited to a distance of 1/4 of the width of the river at the outfall location (mixing zone width = 21 feet). The length of the chronic mixing zone extends 100 feet upstream and 300 feet downstream of the outfall. For Phase 1A, the mixing at the chronic boundary based on volumetric flow limits is 15.2:1 in the summer, and 44.6:1 in the winter. For Phase 1B, the maximum allowable chronic mixing ratio is 10.4:1 in the summer, and 29.9:1 in the winter.

TABLE 3-11

Mixing Zone Dilution Factors, La Center WWTP

Criteria	Phase 1A Summer/Winter	Phase 1B Summer/Winter
Acute Aquatic Life	1.8/3.3	1.5/2.6
Chronic Aquatic Life	15.2/44.6	10.4/29.9
Human Health, Carcinogen	15.2/44.6	10.4/29.9
Human Health, Non-carcinogen	15.2/44.6	10.4/29.9

A Reasonable Potential Analysis (RPA) conducted for the 2016 NPDES Permit concluded that there is no reasonable potential to exceed water quality criteria in the Lewis River (and thus no need for effluent permit limits) for the temperature and metals. However, as shown in Table 3-1, the WWTP does have effluent permit limits for ammonia, which were made more stringent in the 2016 Permit.

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.

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 *Reclaimed Water Facilities Manual* defines 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 3-12 and 3-13. 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 3-12

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 3-13

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(1)	23 MPN/100 mL or CFU/100 mL	23 MPN/100 mL or CFU/100 mL(1)	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 (Weekly Average)	Not Applicable	Not Applicable

(1) 7-day median value.
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.

GROWTH MANAGEMENT ACT

The Washington State Growth Management Act (GMA) was enacted in 1990 and requires certain counties and local governments (including Clark County) to plan for the population growth that will occur over the next 20 years within an established Urban Growth Area. The GMA also requires cities and the county to classify critical areas (wetlands, aquifer recharge areas, fish and wildlife habitat areas, geologically hazardous areas, and frequently flooded areas) and to establish development regulations to protect these areas.

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 City WWTF laboratory is currently accredited for testing the following parameters for ammonia, TSS, BOD5, dissolved oxygen, pH 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.

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 WWTF outfall modifications, an HPA will be required. The HPA will include provisions necessary to minimize project-specific and cumulative impacts to fish.

ON-SITE SEPTIC SYSTEMS

In some cases, wastewater may be treated and disposed of on-site either by individual septic systems or community on-site systems. The City indicates there are a few septic systems remaining within the sewer service area. Options for providing sewer service to areas currently unsewered are discussed later in this Plan.

Municipalities, such as cities and counties, are required under the GMA to eventually provide wastewater collection services to all residents of the Urban Growth Area that are currently not connected. On-site septic systems should be designed to meet the DOH design standards. Approval of the systems will be made either by Skagit County Health Department for systems with a capacity of less than 3,500 gpd, by DOH for systems with a capacity between 3,500 gpd and 100,000 gpd, or by Ecology for systems with a capacity greater than 100,000 gpd. The State Board of Health statute that provides the authority for DOH to adopt rules for sewage treatment is RCW 43.20.

SEWER ORDINANCES AND PLANNING POLICIES

The City operates its sewer system as described in the City's Municipal Code Chapter 13.10, Sewer System Rules and Regulations. In addition to the City's municipal code, the siting of any wastewater facilities outside the city limits, such as pump stations, will have to adhere to Clark County's planning and zoning policies at the time of construction.

CHAPTER 4

EXISTING FACILITIES

This chapter summarizes the City's existing collection system and treatment facilities.

EXISTING COLLECTION SYSTEM

The City's collection system includes approximately 12.5 miles of 8-inch gravity main, 2,300 feet of 10-inch gravity main and 2,100 feet of 4-inch force main (per the 2019 draft General Sewer Plan). Figure 4-1 shows the existing collection system. A larger map of the system is included as Appendix C, Existing Basin Map.

Approximately 25 percent of the La Center collection system was built in 1968 and constructed with concrete sewer pipe which is prone to infiltration. In 2009 and 2011, sewer pipes were lined with cured-in-place pipe lining (CIPP) to reduce infiltration. In 2014, the City also completed a project, which consisted of a contractor applying Silica Modified Portland Cement to rehabilitate the inside of 21 manholes.

The collection system utilizes gravity flow as much as possible with the majority of the lines sloping toward the treatment facility located on the north bank of the river. The existing collection system has six lift stations. Lift Station 1, on Aspen Avenue, was previously used to pump flows within the treatment plant. When the plant was upgraded in 2010, the pump station was converted to be used for collecting flows coming from the west of the facility and the La Center Junction area. Lift Station 3 is located on John Storm Avenue and pumps from subdivisions to a gravity main in Lockwood Creek Road. This wastewater then flows by gravity in a sewer line to the north of Stone Creek Drive, where along with other gravity sewers, it discharges to Lift Station 2, located at the intersection of Stone Creek Drive and 4th Street. Lift Station 4 is located on the south side of La Center Road just west of McCormick Creek. This station was built in 2017 to serve the sewer basin west of the Lewis River up to the I-5 interchange. The system includes a gravity sewer from the l-5 junction to the pump station and two force mains that extend east up to and under the La Center Road Bridge and connect to a gravity manhole outside the treatment plant. (Details regarding Lift Station 4 conveyance are described in later sections.)

There are two new pump stations built recently to serve La Center north of the Lewis River. One is Pump Station 5 (Middle School Pump Station) that serves the basin along the north and south side of Lockwood Creek Road west of Highland Road, including the new Middle School. The new force main was connected to the new gravity sewer in Lockwood Creek Road. The gravity sewer in Lockwood Creek Road was extended from Heritage Country Estates, which is north of Lockwood Creek Road, to a manhole just west of John Storm Road. The second is Lift Station 6 (River Side Pump Station) which was constructed on the south side of Pacific Highway just south of Larsen Road at the northwest end of the City boundary. This pump station was built as part of the Riverside Estates Development. A new 6-inch diameter force main conveys wastewater from this pump station east through private property and on Pacific Highway to a manhole in E Avenue.

Chapter 6, Collection System Evaluation, has more information about the delineation of drainage basins. The basins were identified as areas that flow by gravity sewer that drain either towards the main roads, the pump stations or directly to the WWTP.

Table 4-1 summarizes data for the lift stations.

TABLE 4-1

Wastewater Lift Station Data

Lift Station	Location	Pump Description	Number of Pumps	Approximate Capacity (each pump)	Drawdown Test Capacity(1)
1 – Treatment Plant	101 Aspen Avenue (Treatment Plant)	10 hp, Flygt	2, space for 3	950 gpm @ 39' TDH	585 gpm
2 – Stone Creek	4th Street and Stone Creek Drive	5 hp, Flygt	2	200 gpm @ 45' TDH	130 gpm
3 – Johnstorm	NE John Storm Avenue and East 1st Circle	6.5 hp, Flygt	2	450 gpm	
4 – La Center Road	McCormick Creek	20 hp Flygt NP3171-SH3	2	207 gpm @ 148' TDH	
5 – Middle School	At La Center Middle School	5 hp Flygt NP3102.070	2	265 gpm @ 30.6' TDH	
6 – Riverside	1514 NW 339th Street	11 hp Flygt NP3127	2	156 gpm	

(1) Peak design flow with largest pump out of service. Measured with pump drawdown test in 2015.

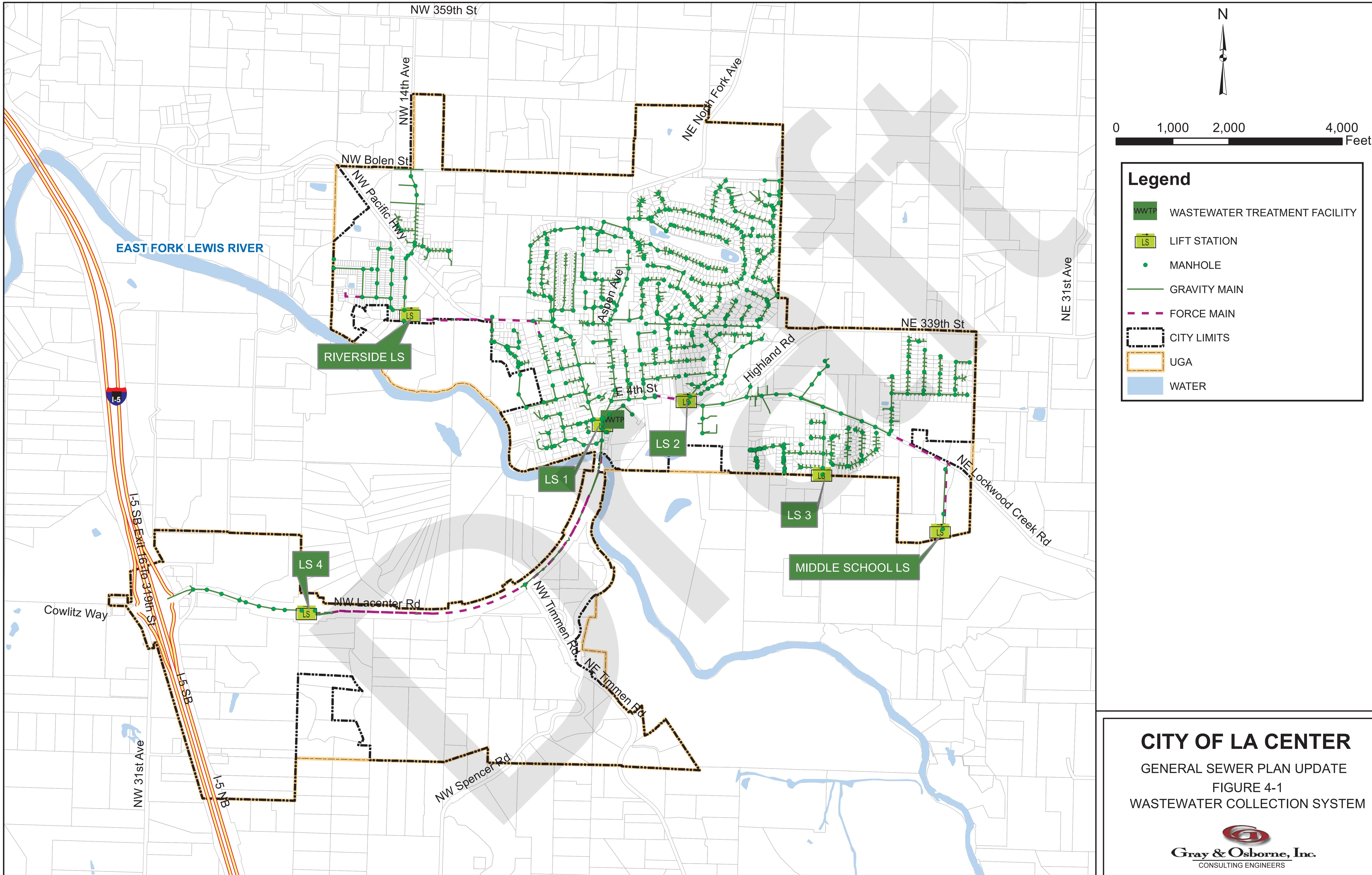
PUMP STATION 1

Pump Station 1 is located at the WWTP and is described in that section.

PUMP STATION 2

Pump Station 2, shown in Figure 4-2, receives flow from Pump Station 3 and the Riverside Pump Station. This pump station was upgraded in 2003 and includes a concrete wet well with two submersible centrifugal pumps, a submersible level sensor and floats.

Operational issues include that the high-level alarm comes on and then the second pump kicks on, but then the wet well level is lowered. There have been no overflows and it is thought that the two upstream pump stations discharging at the same time cause the high


NW 359th St
NW 14th Ave
NE North Fork Ave
NW Bolen St
NW Pacific Hwy
EAST FORK LEWIS RIVER
Aspen Ave
NE 339th St
NE 31st Ave
Highland Rd
E 4th St
RIVERSIDE LS
WWTP
LS
LS 1
LS 2
LS 3
LS 4
MIDDLE SCHOOL LS
NE Lockwood Creek Rd
I-5
I-5 SB Exit 16 to 319th St
Cowlitz Way
NW Lacenter Rd
NW Timmen Rd
NE Timmen Rd
I-5 SB
I-5 NB
NW 31st Ave
NW Spencer Rd
N
0
1,000
2,000
4,000
Feet
Legend
WASTEWATER TREATMENT FACILITY
LIFT STATION
MANHOLE
GRAVITY MAIN
FORCE MAIN
CITY LIMITS
UGA
WATER
CITY OF LA CENTER
GENERAL SEWER PLAN UPDATE
FIGURE 4-1
WASTEWATER COLLECTION SYSTEM
Gray & Osborne, Inc.
CONSULTING ENGINEERS

level in the wet well. This happens on an irregular basis, maybe every couple of days. The second pump coming on is a problem and not consistent with Ecology redundancy requirements.

This pump station uses radio for communication.

This pump station does not have a backup generator and is powered by a portable generator when the power goes out. It is preferable to add a generator to this pump station; however, the existing site is tight and is located in an easement from the neighboring property owner. The wet well does not have a safety grate.

There is considerable corrosion in the wet well.

FIGURE 4-2

Pump Station 2

PUMP STATION 3

Pump Station 3, shown in Figure 4-3, includes a concrete wet well with two submersible centrifugal pumps, a submersible level sensor and floats. The station was upgraded with new Flygt pumps and a new bypass connection about 2 years ago by a developer. The pump discharges were changed from 4 inch to 3 inch.

The pump station cannot be controlled from the WWTP but can be monitored through radio communication. No issues were identified. The pump station is on a tight site with a natural gas-powered generator.

FIGURE 4-3

Pump Station 3

PUMP STATION 4 (LA CENTER ROAD PUMP STATION)

Pump Station 4, also known as the La Center Road Pump Station, was built in 2017 and includes a concrete wet well with two submersible centrifugal pumps, a submersible level sensor and floats.

It does not yet receive flow and is not in operation except for routine exercises. The pump station, shown in Figure 4-4, was constructed to Clark Regional Wastewater District standards. It is operated periodically to discharge infiltration and inflow (I/I) or exercised with water that is conveyed into the wet well with a hose. It has a large Bioxide storage tank that is nearly full. This pump station will start being used as the area by the interchange starts being developed.

This pump station uses fiber optics for communication.

The dual discharge lines pump to a gravity sewer south of the Lewis River Bridge that is controlled by an electronically operated valve that opens when a certain pressure head is reached. When opened it provides flushing action through the sewer that is hung under the bridge and discharges into a manhole at the WWTP, and eventually Pump Station 1. The electronics to the valve were damaged when the valve vault was flooded with groundwater and need to be replaced. Until the electronics are replaced, the valve is left in the open position.

FIGURE 4-4

Pump Station 4

PUMP STATION 5 (MIDDLE SCHOOL PUMP STATION)

Pump Station 5 was built in 2020 and includes a concrete wet well with two submersible centrifugal pumps, a submersible level sensor and floats.

This station was constructed as part of the Middle School project. It was constructed to Clark Regional Wastewater District standards. No issues were identified. This site was not visited due to lack of time available. This pump station uses radio for communication.

PUMP STATION 6 (RIVERSIDE PUMP STATION)

Pump Station 6, shown in Figure 4-5, was built in 2019 and includes a concrete wet well with two submersible centrifugal pumps, a submersible level sensor and floats.

This pump station was constructed as part of the Riverside subdivision development. It was also constructed to Clark Regional Wastewater District standards. No issues were identified in a site visit.

FIGURE 4-5

Riverside Pump Station (Pump Station 6)

EXISTING TREATMENT PLANT

HISTORY

La Center owned and operated the sewer system from around 1967 until 1992, when Clark Public Utilities (CPU) took over ownership and operation of the system. CPU made several improvements to the sequencing batch reactors process wastewater treatment plant (WWTP) in the 1990's, and completely rebuilt the plant in 2004. La Center later purchased the sewer system back from CPU, began operating it in August 2006 and is continuing to own and operate the utility. In late 2010, the facility was converted to a new Membrane Bioreactor process which produces high quality effluent for discharge to the East Fork Lewis River. One of the prior sequencing batch reactor tanks was significantly modified to be part of the membrane bioreactor process. The other was converted to a sludge thickener/digestor. The new MBR plant kept the former sludge digestion basin as well. Improvements to the solids handling train included replacing the belt filter press with a rotary fan press prior to the Fenton sludge dryer and improving the air handling components in the biosolids processing area. The facility creates a low moisture content Class A biosolids product suitable for a variety of applications.

CURRENT TREATMENT PLANT FACILITIES

The treatment and disposal facilities are described in the following paragraphs. Figure 4-6 shows an aerial view of the existing Wastewater Treatment Plant. Figures 4-7 and 4-8 are the site drawing and the process schematic.

FIGURE 4-6

Existing WWTP

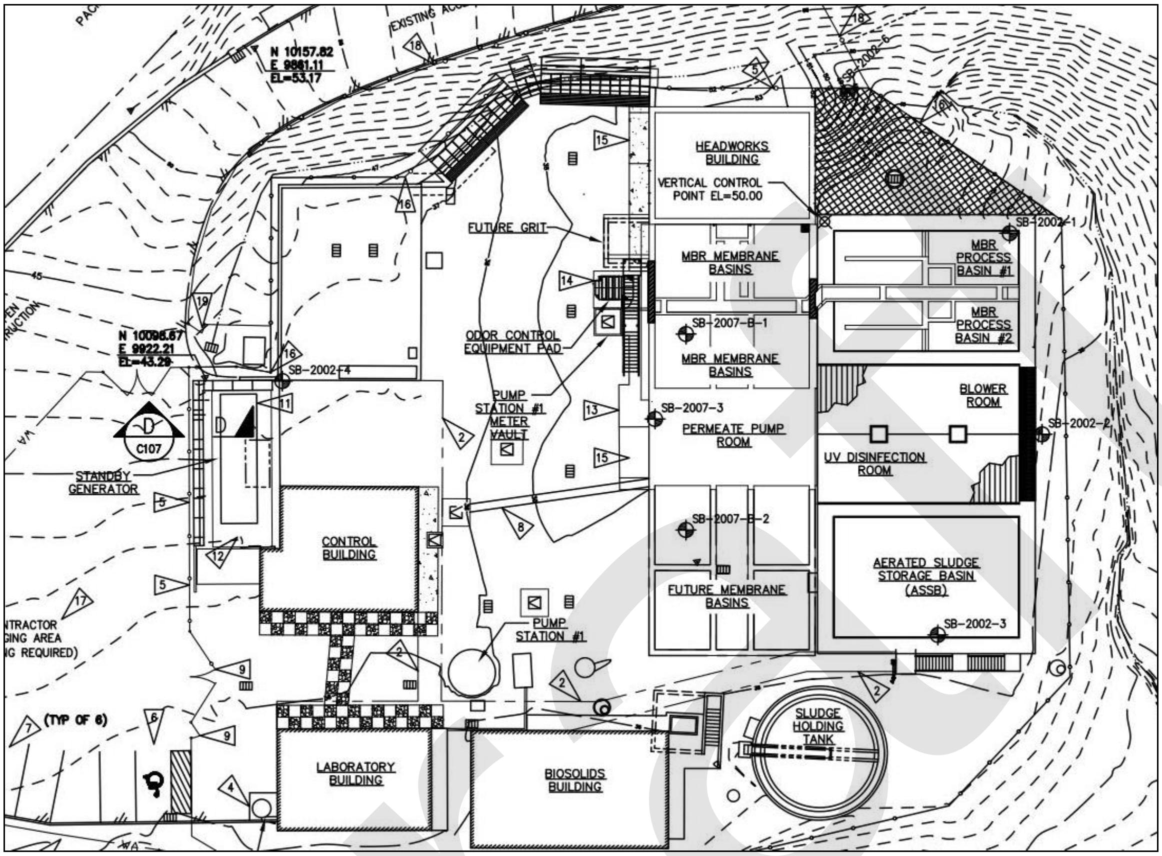


FIGURE 4-7

WWTP Site Plan

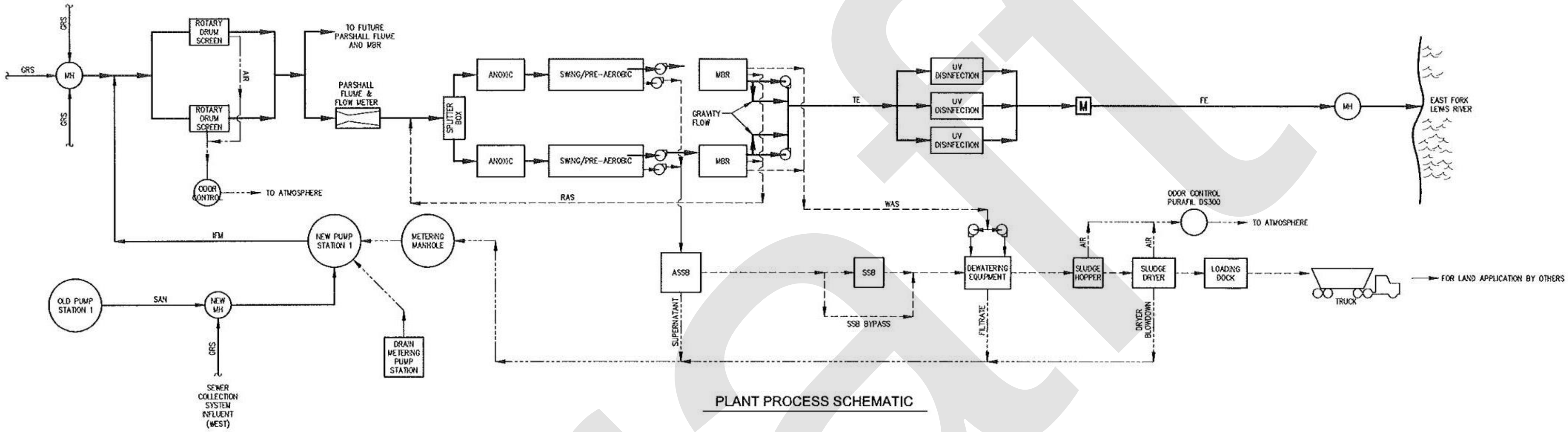


FIGURE 4-8

WWTP Process Schematic

Pump Station 1

The purpose of the in-plant pump station is to receive flows from the service area and convey them to up the headworks:

- 24-inch gravity sewer line from the wastewater collection system
- Plant drain pumps
- Aerated Sludge Storage Basin (ASSB) supernatant
- Dewatering equipment filtrate
- Sludge dryer blowdown

The wastewater flows to a wet well with two variable speed submersible pumps. Each pump has its own discharge pipe, check valve, and isolation valve. The pumps are mounted on guide rails mounted to allow for their removal/replacement without entering the wet well.

The pumped flows combine into a 12-inch pipe, are measured by a magnetic flow meter, and then conveyed to the Headworks for fine screening. The PLC will monitor flow and totalize hourly and daily flow and record the previous day's totalized flows. The totalized flow values are displayed and logged historically in the HMI. The HMI also displays the continuous measurement flow. The influent flow sensor/transmitter produces a 4-20 mA signal proportional to its calibration range that is received by the PLC and scaled from 0 to full scale flow in mgd.

The wet well is accessible through a double-door hatch cover. A safety grate below the hatch covers provides additional safety and a location to rinse pumps when removed.

Table 4-2 summarizes the design data for Pump Station 1.

TABLE 4-2

Pump Station 1 Design Data

Process Unit	Value
Number of pumps	2
Pump type	Submersible, centrifugal
Design flow per pump	950 gpm
Firm capacity(1)	1.4 mgd
Design TDH	38.6 feet

(1) Capacity with redundant pump on is 2.7 mgd.

Headworks

The WWTP Influent enters the Headworks building through an18-inch gravity line (from the majority of the City) and a 12-inch force main from Pump Station 1. (The majority of the flows come into the headworks by gravity sewer. Pump Station 1 contributes ~5 percent to the headworks, which will increase as the interchange area is developed.) The influent enters a channel that allows the wastewater to flow through one or two headworks screens. The screens can be operated independently with slide gates or can be operated in parallel. Each screen is an Envirocare rotary drum fine screen with 3-mm perforated screens with a hydraulic capacity of 6.2 mgd each. Fine screening is accomplished using rotary drum screens to protect the membrane filtration system. Each drum screen has a washer-compactor which cleans the screenings and deposits them into a conveyor which transports them to a garbage can for disposal. The screenings are picked up by the local waste management hauler, Waste Connections, Inc. Slide gates upstream and downstream of each fine screen allow either one of the fine screens to be taken offline.

No dedicated grit removal facilities are provided; however, area was reserved for future grit removal, if deemed necessary. Flow enters a channel on the discharge side of the headworks and a sampler is installed for influent sampling. Piping for splitting the flows for additional treatment trains is installed. The flow then enters a Parshall flume with ultrasonic flow meter for flow measurement prior to entering the recycle channel for secondary treatment.

There is an engineered carbon odor control unit for the headworks which provides 12 air exchanges per hour.

One issue is that flushable wipes get stuck on the screens. Also, one screen seems to collect more solids than the other. Influent (and effluent) sampling is timed, not flow proportional.

Table 4-3 summarizes the design data for Headworks.

TABLE 4-3

Headworks Design Data

Process Unit	Value
General	
Channel width	2 feet
Channel velocity @ AAF	0.81 ft/sec
Channel velocity @ MMWWF	1.03 ft/sec
Channel velocity @ PHF	2.01 ft/sec
Maximum headloss through headworks	1.78 feet
Channel solids resuspension required	YES
Fine Screens	
Type	Rotary drum
Number	2
Opening	3 mm
Capacity (each)	6.2 mgd
Headloss (each) @ PHF	1.00 feet
Screening, washing, and compaction	Integral to screening units
Water demand (NPW) (each)	16.4 gpm
Influent Flow Meter	
Type	Parshall flume
Number	1
Throat width	9 inch
Maximum flume flow capacity	8.0 mgd
Flow depth @ AAF	6.4 inch
Flow depth @ MMWWF	7.9 inch
Odor Control	
Type	Engineered carbon
Number	1
Fan capacity	4,200 cfm
Fan speed	Two speed

Process Basins

The influent and recycle flows from the membrane basins combine and enter the process basins. Recycle flows from solids processing also enters in this channel. The flow is split into two process trains. The WWTP uses the Modified Ludzack-Ettinger (MLE) process for the combined removal of BOD, ammonia, and nitrate/nitrite. The process employs a combination of anoxic and aerobic zones. Each train has an anoxic zone for denitrification, a swing zone that can be either anoxic or aerobic, and an aerobic zone for TSS, BOD, and ammonia removal. Mixers are installed in the anoxic and swing zones and Aerostrip diffusers are installed in the aerobic and swing zones. Air is supplied by blowers located in the blower room. One blower provides air for the Process basins and a standby blower is available for use. The controls are automated through the SCADA system. There is a feed forward pump in each basin which pumps the mixed liquor into the membrane basins. A spare pump is available for installation if needed. There is a submersible pump in each process basin which pumps waste to the Aerated Solids Storage Basin.

Filtrate and pressate from solids handling are returned upstream of the anoxic zones. Typical TSS in the biological treatment system: 7,250 – 7,500 mg/L in the anoxic and pre-air zones, 9,000 mg/L in the MBR tank, and 6,250 mg/L in the post-treatment storage tank.

<u>Anoxic Basins</u>

Each anoxic basin is equipped with a submerged differential pressure level transmitter and two float switches to detect the liquid level in the basin. Low and high analog alarm level set points are operator-adjustable at the HMI. The switches serve as backup instruments for alarm level detection in the event that a level transmitter is no longer functional.

Upon detecting a low liquid level condition an alarm is posted and all rotating equipment installed in the basin will shut down until the condition clears. Upon detecting a high liquid level condition an alarm is posted until the condition clears. Measured liquid level is monitored and recorded at all times.

<u>Swing Basins</u>

The swing basins may be placed into one of four control modes at the operator interface:

1. Anoxic Mode – In Anoxic mode no air flow is initiated to the basin regardless of measured NADH levels or dissolved oxygen concentrations.

2. Dissolved Oxygen Control Mode – In dissolved oxygen control air flow to the basin is regulated to meet the dissolved oxygen demands of the hydraulically linked downstream pre-aeration basin. Enabling dissolved oxygen control in the swing basin also enables dissolved oxygen control for the pre-aeration basin.

3. Symbio® Control Mode – In Symbio® control mode air flow is regulated to meet the demands of conditions that favor simultaneous nitrification and denitrification. Enabling Symbio® in the swing basin also enables Symbio for the hydraulically linked downstream preaeration basin.

4. Air Flow Control Mode – In air flow control mode air flow is regulated to meet the demands of an operator-entered air flow set point.

Pre-Aeration Basins

Dissolved oxygen probes monitor the temperature and oxygen concentration of the mixed liquor in the pre-aeration basins.

The dissolved oxygen transmitters are luminescent style sensors that have been factory calibrated. If the mixed liquor temperature or dissolved oxygen concentration falls below operator adjustable limits, an alarm is activated. Mixed liquor temperature and dissolved oxygen concentration are monitored and recorded at all times. Dissolved oxygen concentration is used to modulate the position of the pre-aeration basin air flow control valves.

Feed Forward Pumps

Each pre-aeration basin is equipped with one submersible pump for mixed liquor feed forward flow. The feed forward pumps will run continuously unless all downstream MBRs are offline or a low level condition is detected in the associated pre-aeration basin.

The feed forward pumps are driven by variable frequency drives (VFDs), Feed forward pump motor speed modulates in order to maintain the calculated RAS flow set point.

WAS/EQ Pumps

Each pre-aeration basin is equipped with one submersible pump for transfer of mixed liquor to the Aerated Sludge Storage Basin (ASSB). Each WAS/EQ pump is driven by a full voltage non-reversing motor (FVNR).

Waste activated sludge, generated as a by-product of the activated sludge process, is wasted directly from the MBR System on a regular basis. The HMI provides the ability to schedule multiple wasting events per day. The user may enable and disable each event schedule and designate a start time and volume to waste for each. When a scheduled WAS event is initiated either automatically or manually, by using the WAS transfer start button, the WAS/EQ pumps will start and remain on until the transferred volume equals the target volume for that event.

WAS/EQ transfer pump operation will be inhibited upon detection of a high-water level condition in the Aerated Sludge Storage Basin.

Table 4-4 summarizes the design data for Process Basins.

TABLE 4-4

Process Basins Design Data

Process Unit	**Value**	
Number of basins	2	
Number of Trains/Basin	1	
Basin volume	205, 340 gal	
Sidewater depth	18.0 ft	
SRT	22 days	
MLSS	8,000 - 12,000 mg/l	
WAS	13,100 gal/day	
Number of anoxic zones	2	
Anoxic zone volume (each)	53,690 gal	
Number of swing zones	2	
Swing zone volume (each)	24,490 gal	
Number of aerobic zones	2	
Aerobic zone volume (each)	24,490 gal	
Aeration Equipment		
Process Blowers	Phase 1A[1]	Phase 1B[1]
Number	2 (1 + 1 common spare)	2(1 + 1 common spare)
Capacity (each)	720 scfm	1,600 scfm
Horsepower (each)	60 hp	100 hp
Process Basin Diffusers		
Type	Fine bubble membrane	
Capacity	1,600 scfm	
Permeate Pumps		
Number	2	
Type	Self-priming centrifugal	
Capacity per pump	1.50 mgd	
Firm capacity	1.50 mgd	
Total capacity	3.00 mgd	
Total dynamic head	31.0 feet	
Variable frequency drive	Yes	
RAS/Feed Forward Pumps		
Number	3 (2 + 1 shelf spare)	
Pump type	Propeller	
Capacity per pump	3.64 mgd (2528 gpm)	
Firm capacity	7.28 mgd (5056 gpm)	
Total capacity	7.28 mgd (5056 gpm)	
Total dynamic head	8.2 feet	
Variable frequency drive	Yes	

TABLE 4-4 – (continued)

Process Basins Design Data

Process Unit	Value
WAS/Equalization Pumps	
Number	3 (2 + 1 shelf spare)
Pump type	Submersible
Capacity per pump	331 gpm
Firm capacity	662 gpm
Total capacity	662 gpm
Total dynamic head	12.2 feet
Variable frequency drive	Yes

(1) Phase 1A was completed in 2014. The next phase (Phase1B) will be constructed when flows and loadings are approaching the Phase 1A design criteria.

MBR Filtration

After biological treatment, Feed Forward Pumps located at the downstream end of the Pre-Aeration Basins transfer mixed liquor from the aeration tank to two membrane tanks where the liquid portion of mixed liquor is separated from the solids by membrane filtration. The membranes are provided in modules that are called Submerged Membrane Units (SMUs). Each SMU contains a diffuser case and two cassettes with 200 membrane plates each. The cassettes are double stacked. There are a total of 2,000 membrane plates in each basin.

All membranes (Kubota) were replaced in Summer 2022. The previous membranes lasted 12 years.

To prevent solids accumulation on the outside of the membrane surface from slowing the filtration process, air scouring (bubbling air across the surface of the filters) is used to keep the surface free of solids. Efficient and equal air scouring is critical to operation. Therefore, air scour flow rates are monitored, recorded and controlled.

Because equal air scouring is required, the diffusers integral to each submerged membrane unit (SMU) must also be kept clean. Diffusers are kept clean by scouring. This process is automatically initiated using an automated Diffuser Cleaning Valve (DCV).

There are currently four membrane tanks each capable of containing 5 double stacked SMUs. During construction the decision was made to not place membranes into two of the tanks since current flows would not require them. The construction was developed to be completed in two phases. All piping and controls were installed and the membranes can be installed and brought into operation relatively easily. The first phase (Phase 1A) was completed in 2014. The next phase (Phase1B) will be constructed when the Phase 1A design criteria are approaching.

The following is a summary of the capacity of the plant with Phase 1A and Phase1B.

<u>Phase 1A</u>

The two basins containing the membranes have 5 double stacked SMU units with Kubota flat plate membranes. Air scour is provided by two blowers and the standby blower is available for backup. According to the facility plan the design capacity is shown in Table 4-5.

TABLE 4-5

Phase 1A MBR Hydraulic Capacity

Parameter	Value
No. MBR Membrane Basins	2
No. SMUs per Basin	5 (double-stacked)
Total SMUs	10
Max Month Flux at 13^o C	10 GFD
Peak Day Flux at 13^o C	18.0 GFD
Peak Hour Flux at 13^o C	26.7 GFD
Average Hydraulic Capacity per Basin	0.37 mgd
Total Average Hydraulic Capacity	0.75 mgd
Total Peak Hydraulic Capacity	1.5 mgd

Note: GFD = gallons per square foot per day.

Permeate piping is shown in Figure 4-9.

FIGURE 4-9

Permeate Piping

According to information from the manufacturer, the installed capacity is slightly higher than shown in Table 4-5. The average daily flux capacity for the RW 400 SMUs is 13.8 gfd at 13^{o}C which is equivalent to 0.43 mgd for each basin and a peak day of 0.86 mgd each. This is a firm hydraulic capacity of 0.86 mgd with a peak day capacity of 1.72 mgd.

Phase 1B

According to the *Facility Plan*, an additional phase of expansion (Phase 1B) was planned when flows reach the maximum month capacity of 0.69 mgd. Phase 1B would consist of installing the additional 10 SMUs in the empty MBR basins and adding the blower capacity required for the air scour of the system.

According to the facility plan the design capacity is shown in Table 4-6.

TABLE 4-6

Phase 1B MBR Hydraulic Capacity

Parameter	Value
No. MBR Membrane Basins	4
No. SMUs per Basin	5 (double-stacked)
Total SMUs	20
Average Flux at 13°C	10 GFD
Peak Flux at 13°C	26.7 GFD
Average Hydraulic Capacity per Basin	0.37 mgd
Total Average Hydraulic Capacity	1.5 mgd
Total Peak Hydraulic Capacity	3.0 mgd

Adding the membranes to the other two tanks and providing additional air scour, the firm hydraulic capacity of the plant would be 1.72 mgd with a peak day capacity of 3.54 mgd.

Chemical Clean

The Clean-In-Place (CIP) system supplies dilute cleaning chemicals for removing accumulated biological growth from the MBR filtration surface.

For cleaning the City uses a 4-hour soak of hypochlorite. (Previously, an overnight soak was provided, but that length of time was not necessary, and could be detrimental to membrane life.) Membranes are cleaned 3-4 times per year, based on the quality of the effluent.

The cleaning is performed after an MBR has been placed offline and the associated basin isolated from the remainder of the system. The cleaning chemicals are transferred to the cassettes to be cleaned in a manner that fills the inside of the cassettes with the cleaning solution, displacing water inside the cassettes back through the membrane into the MBR basin.

Periodic Maintenance Cleans (MCs) are performed only after the biofilm layer has built up beyond the control of the air scour and permeate header relax state cleaning operations designed to maintain optimum biofilm thickness. The entire process is carried out in-situ without draining mixed liquor.

The CIP system consists of an actuated water supply valve, a pressure regulating valve, an eductor, and a flow transmitter.

Blowers

Air supply for the Process Basins and MBR Basins is provided by four Aerzen Positive Displacement blowers installed in the blower room. Each blower has a capacity of 700 scfm. Two 60-hp blowers provide air for preventing the membranes from fouling. One 60-hp blower provides air for the process basins and one 60-hp standby blower is available to back up either the aeration basin or the membrane air scour depending on the need. The blower room has a space available for a fifth blower in the air line. The solids wasted from the process basin are aerated by a separate 40-hp blower.

<u>MBR Blowers</u>

The MBR aeration system consists of two positive displacement duty blowers, with a third to be added in the future, connected to a common plenum. The MBR blowers are driven by variable frequency drives (VFDs). A pressure sensor uses a sealed diaphragm without process isolation to monitor plenum pressure. A standby blower is shared with the Swing and Pre-Aeration basin blower.

Each membrane basin has a dedicated aeration header to supply scour air. Each header consists of a modulating flow control valve and a thermal-dispersion mass flow meter. The meter is factory calibrated and spanned to the expected flow range.

The MBR blowers are shown in Figure 4-10.

FIGURE 4-10

MBR Blowers

Swing and Pre-Aeration Basin Blower

The process basin aeration system consists of one positive displacement duty blower. The blower is driven by variable frequency drives (VFDs).

Shared Standby Blower

When a motor failure is detected in either the MBR or Swing/Pre-Aeration blower groups and no lag motor is available for operation, the shared standby blower will automatically be called into service.

Membrane air scour and aeration requirements are summarized in Table 4-7 and Table 4-8 (based on information in the *Facility Plan*). For Phase 1B additional blower capacity will be provided by moving the Standby Blower to the MBR Air Scour and replacing the Process air blower and the standby blower with at least 1,600 scfm capacity blowers.

TABLE 4-7

MBR Scour and Process Blower Air Ranges

Phase	**Maximum Capacity**	**Maximum Capacity**	**Average Capacity**	**Minimum Flow**
	(PHF)	(EQ PHF)(1)	(MMWWF)	(AAF)
Phase 1 A (originally expected to be 2008 – 2012)				
MBR Scour Flow (scfm)	1,000	1,000	750	375
MBR Process Flow (scfm)	1,875	700	650	460
Phase 1 B (originally expected to be 2012 – 2017)				
MBR Scour Flow (scfm)	2,000	2,000	1,125	750
MBR Process Flow (scfm)	2,600	1,600	985	550

(1) Per Facility Plan Equalized, PHF is 1.1 mgd in Phase 1A and 2.1 mgd in Phase 1B, aeration based upon providing 0.5 mg/l DO residual during peak flow condition.

TABLE 4-8

MBR Scour Aeration Demand

Phase	Year	No. Double-Stacked SMUs	No. Membrane Basins	Min Scour (scfm)	Average Scour (scfm)	Peak Scour (scfm)	Peak Scour w/Safety Factor (scfm)
Phase 1A	2008-2012	10	2	375(1)	750	1,000	1,050
Phase 1B	2012-2017	20	4	1,000	1,500	2,000	2,100

(1) Minimum airflow in Phase 1A assumes only one MBR basin is permeating.

Permeate System

The effluent that is filtered through the membranes is called permeate.

Each MBR is equipped with two permeate headers. Each permeate header includes instruments for measuring permeate header pressure and flow. Flow rate on each header is controlled using a motor-operated valve with a positioning actuator. The pressure sensor uses a sealed diaphragm without process isolation and has a fixed range of -14 to +15 psig.

One turbidimeter analyzes a side-stream sample of all collected permeate flow to indicate how well the system is filtering solids.

<u>Permeate Pump</u>

Typically, each permeate header will operate under gravity flow, where the level differential between the MBR basin and downstream wet-well side-water depths is the primary force used to drive filtration across the membrane surface to meet hydraulic demand. However, the possibility of not being able to permeate required the installation of permeate pumps for the conditions that would limit the ability to permeate by gravity. The pumps are used to aid in maintaining prime within the system and as needed when increased resistances, such as those that occur during airlock or membrane fouling, restrict permeate flow. These increased resistances to flow require additional pressure differential to meet hydraulic demand.

In the permeate pump room two self-priming centrifugal pumps with the capacity of 1,042 gpm @ 31 ft TDH (1.5 mgd) each are installed in the permeate pump room. The pumping line is placed in parallel to the gravity flow line. Piping for additional permeate pumps for the Phase 3 flows are installed.

The permeate pumps are driven by variable frequency drives (VFDs).

Table 4-9 summarizes the design data for MBR Basins.

TABLE 4-9

MBR Basins Design Data

Process Unit	**Value**	
	Phase 1A(1)	Phase 1B(1)
Number of basins	2	4
Submerged membrane units per basin	5 double stacked	5 double stacked
Average flux	10 gfd	10 gfd
Peak flux	27.6 gfd	27.6 gfd
Average flow per basin (13°C)	0.375 mgd	0.375 mgd
Total average flow (13°C)	0.75 mgd	1.5 mgd
Total peak flow (13°C)	1.5 mgd	3.0 mgd
Aeration Equipment		
MBR Scour Blowers		
Number	3 (2+1 common spare)	4(3+1 common spare)
Capacity (each)	720 scfm	720 scfm
Horsepower (each)	60 hp	60 hp
MBR Membrane Basin Diffusers		
Type	Coarse bubble	Coarse bubble
Capacity	1,050 scfm	2,100 scfm
Permeate Pumps		
Number	2	
Type	Self-priming centrifugal	
Capacity per pump	1.5 mgd	
Firm capacity	1.5 mgd	
Total capacity	3.00 mgd	
Total dynamic head	31.0 feet	
Variable frequency drive	Yes	

(1) Phase 1A was completed in 2014. The next phase (Phase1B) will be constructed when influent flows/loadings are approaching the Phase 1A design criteria.

Disinfection

Permeate is disinfected with inline 1250 Aquionics ultraviolet disinfection units. Ultraviolet radiation has proven to be effective at the inactivation of pathogens in treatment plant effluent, without contributing to the formation of toxic disinfection byproducts. Permeate from the four membrane cells in the MBR System is combined into a common 16-inch disinfection feed header, which then distributes flow to each of the three UV units. The two small units each have a rated capacity for membrane permeate of 1.75 mgd and are nearly 20 years old. The third unit, 15 years old, has a capacity of 3.1 mgd, which brings the total firm capacity of the disinfection system up to 3.5 mgd (the capacity of Phase 1B). Piping has been installed to be able to exchange the two older units to the same size as the third bringing the capacity of the disinfection system to 6.0 mgd (the Phase 3 design capacity).

Under low and normal flow conditions MBR permeate flows by gravity through UV disinfection to the river. During higher flow conditions, MBR permeate is pulled through the membranes by four permeate pumps to overcome higher hydraulic head requirements caused either by high flows or a high river water surface elevation (WSE).

Figure 4-11 shows the UV Disinfection System.

FIGURE 4-11

UV Disinfection System

Table 4-10 summarizes the design data for disinfection system.

TABLE 4-10

Disinfection Design Data

Parameter	Value
Number of units	3
Type	In-line, closed channel, high intensity, medium pressure
Design flow per unit	2 @ 1.75 mgd, 1 @ 3.10 mgd
Firm capacity	3.5 mgd
Total capacity	6.60 mgd
Number of lamps per unit	6
Design transmittance	0.7
Fecal coliform standards	100 organisms/100 ml monthly average
	200 organisms/100 ml weekly average

Outfall

After disinfection the effluent flows through a pipe and through the effluent flow meter. The effluent is sampled in the line between the UV and flow meter. A portion of the effluent is put into a basin for use in the utility water system. A tablet chlorinator provides addition disinfection to prevent growth in the utility water system.

The effluent discharges beneath the surface of the East Fork Lewis River at approximately River Mile 3.2. A 10-inch outfall pipe and multiport diffuser extends about 15 feet into the river. The diffuser is a rectangular box with 28 6-inch by 2-inch ports with 14 ports facing upstream and 14 of the ports facing downstream.

Solids Stabilization

<u>Aerated Sludge Storage Basin (ASSB)</u>

Waste solids are removed from the process basins and stored in a 250,000-gallon Aerated Sludge Storage Basin (ASSB) (one of the former sequencing batch reactors) where they are completely mixed and periodically aerated to maintain an aerobic condition. The design average residence time in this basin is approximately 60 days, which would provide adequate mixing and aeration for degradation of volatile solids and stabilization.

Aeration is provided through two diffuser grids in the basin, along with one blower (and one uninstalled shelf spare) and a floating mixer. The aeration blower is used in conjunction with the ASSB DO probe to control DO in the ASSB.

The decanter in the ASSB removes supernatant from the basin. The supernatant is returned to the headworks via the in-plant pump station. Decanting increases the minimum WAS concentration of 0.8 percent to greater than 1.0 percent.

Aerated sludge from the ASSB is discharged via gravity through a 6-inch glass lined ductile iron pipe connected directly to the dewatering equipment feed pumps. A 4-inch gravity ductile iron sludge pipe allows for the WAS to be directly connected to the dewatering feed pumps or to the Sludge Storage Basin.

Sludge Storage Basin (SSB)

The solids are transferred to a 28-foot diameter Sludge Storage Basin (SSB) (former clarifier) with a volume of approximately 45,000 gallons. It is equipped with an Aqua-Jet aerator/mixer. An average of approximately 6 loads of waste sludge per month are received from the Ilani tribal casino operated by the Cowlitz Tribe MBR WWTP (4,500 – 5,000 gal./load at 1.5-2.0 percent solids. Solids are being received from the nearby and are combined in this tank prior to dewatering. The design average residence time in the solids storage basin is about 7 days.

Table 4-11 summarizes the design data for Solids Stabilization.

TABLE 4-11

Solids Stabilization Design Data

Parameter	**Value**
Aerated Sludge Storage Basin	
Volume	267,000 gal
Influent WAS % solids	0.8 - 1.2 %
Influent BOD	3,840 lb/day
Aeration provided	Fine bubble
Diffuser grids	2
Existing diffuser grid capacity (each)	400 scfm
Oxygen airflow demand	600 scfm
Minimum SRT	9.4 days
Number of blowers	1
Blower size	40 hp
Blower capacity	490 scfm
Mixing power provided	7.5 HP
Sludge Storage Basin	
Number of basins	1
Volume	53,000 gal
Aeration/mixing type	Floating aerator/mixer
Mixing power provided	21 lbs O2/hp day
Mixing power provided	15 hp

Solids Processing and Disposal

Solids from the Aerated Sludge Storage Basin (ASSB), the Sludge Storage Basin (SSB), and the MBR Tank Drains are pumped to the Rotary Fan Press by one of two Feed Solids Pumps with a polymer feed system for dewatering. The press, shown in Figure 4-12, produces a cake that is about 13-14 percent solids which is acceptable to be dried in the Fenton Dryer. The rotary fan press and sludge dryer can be operated intermittently. The Dryer uses natural gas to heat a thermal fluid which indirectly dehydrates the solids to greater than 90 percent. The City produces a Class A EQ product which is given away to the public or to Lewis River Reforestation for agricultural use.

Return flows from the rotary fan press, and to a small extent the Fenton Dryer are routed back to the headworks area.

Polymer Addition

A flow meter and check valve are located on the sludge feed pipe. Just before entering the flocculation tank, polymer is injected into the sludge feed pipe. The polymer system consists of a single chemical feed skid and available space for the storage of two 250-gallon totes of polymer. The chemical feed skid mixes the polymer with potable water. The meter located on the sludge pipe can be used to control the polymer dosing.

FIGURE 4-12

Rotary Fan Press

Rotary Fan Press

The flocculation tank overflows into the head box of the rotary fan press. Level sensors in the head box control the pumping of the rotary fan press feed pumps. Once in the head box, the sludge passes through the rotary fan press.

The Rotary Fan Press is an upright disk shaped unit that rotates sludge in a circular channel within the press. Sludge is fed into the inlet, and travels slowly through the interior channel, dewatering through central screens which drain the filtrate to a separate outlet. The cake is extruded as the screens turn slowly and the material behind it builds and advances. The drainage or centrate collects in the drip pan under the screw press and then flows by gravity to the in-plant pump station through the plant drain piping.

The dewatered sludge (typically 11.5-12.0 percent solids) falls out of the rotary fan press onto a belt conveyor and ultimately drops into a trailer to be hauled offsite.

The rotary fan press is approaching the end of its useful life, and will need to be replaced within 10 years; the City is interested in installing a screw press to replace it.

Sludge Dryer

Solids drying processes use heat to evaporate moisture from the solids using either direct or indirect contact with hot gases. The gases are usually produced by burned natural gas or digester gas. The high temperatures and low moisture content inactivate pathogens and significantly reduce the volume of material.

The existing dryer, shown in Figure 4-13, is a Fenton Sludgemaster RK-36, capable of processing 33 cubic feet (1 wet ton) of sludge per batch. The rated capacity of the unit is 6-8 batches per day based on 24-hour/day operation. However, due to safety concerns, the City is not comfortable operating an entire drying batch cycle unattended. This limits the effective dryer capacity to two or three batches per day.

FIGURE 4-13

Sludge Dryer

The end product Exceptional Quality (EQ) biosolids is currently for beneficial use and given away to the public or to Lewis River Reforestation for agricultural use.

The dryer, is reaching the end of its useful life and it is difficult to find replacement parts for it. There has been discussion of expanding the building on the side with the big door.

Table 4-12 summarizes the design data for Solids Processing and Disposal.

TABLE 4-12

Solids Processing Design Data

Parameter	Value
Sludge Dewatering	
Number of units	1
Type of units	Rotary Fan Press
Sludge feed % solids	0.75%-1.2%
Polymer dosage	7 to 10 lb/day
Cake % solids	14%-18%
Sludge Dryer	
Number of units	1
Type of unit	Fenton RK-36
Method of operation	Batch
Capacity	33 cf/batch
Production	1 to 3 batches/day
Input % solids	14%-18%
Dried % solids	>90%

Support Facilities

<u>Utility Water System</u>

The Plant Water System provides pressurized water for all cleaning and in-plant non-potable needs. Treated plant water (permeate) is chlorinated using a chlorine tablet feed system and discharged to the Utility Water Equalization Basin. A packaged booster system pulls chlorinated plant water from the Equalization Basin and pressurizes the water with a hydro-pneumatic tank to ensure adequate pressure for cleaning and equipment needs. The chlorinated and pressurized plant water flows through a distribution loop to service points around the treatment plant.

Table 4-13 summarizes the design data for Utility Water System.

TABLE 4-13

Utility Water System Design Data

Parameter	Value
Utility Water System	
Number of basins	1
Basin Volume	54,000 gals
Chlorination	
Type	Gravity Chlorine Tablet Feeder
Number of units	1
Capacity	260 gpm
Chlorine residual	0.5 mg/L
Packaged Booster System	
Number of pumps	3
Pump design point	1 @ 50 gpm @ 290 ft. TDH 2 @ 130 gpm @ 250 ft. TDH
Firm capacity	180 gpm
Variable frequency drive	Yes
Number of bladder tanks	1
Bladder tank volume	200 gals
Design pressure	150 psi

<u>Chlorination System</u>

The chlorine-tablet feeder installed upstream of the EQ basin is used to maintain a 0.5 mg/l chlorine residual. The chlorine tablet feeder was designed to handle the maximum storage fill rate of 260 gpm, which would equate to a maximum chlorine tablet demand of approximately 1.5 pounds per day.

The tablet feeder uses slow-release 65 percent calcium hypochlorite tablets contained in a rigid PVC vessel. Incoming water from a side stream contacts tablets at the bottom of the vessel, so tablets on the top of the vessel stay dry. Tablets erode at a predictable rate according to the amount of water that enters the chlorinator and the chlorine dose is controlled by the incoming water flow rate. Chlorinator effluent is returned to the unchlorinated main system flow.

A submersible mixer in the EQ basin mixes the contents of the basin, ensuring distribution of the chlorine.

<u>Administrative/Office/Electrical Building</u>

The Disk Filter building was modified during the 2010 upgrade of the facility to contain a training/conference room, supervisory office, operators stations, controls and electrical equipment. Half of the building was modified to contain the electrical system for the plant. Two Motor Control Centers (MCC) for the plant were installed in the air-conditioned electrical room. Incoming power enters the plant in the electrical room. The transformer was upgraded to provide enough capacity for the ultimate buildout of the facility. All incoming power components were upgraded and a 1,000-kilowatt generator, automatic transfer switch and 1,200-gallon diesel fuel tank were installed. The generator and ATS were sized for buildout conditions.

<u>Laboratory Building</u>

The laboratory building is located to the south of the administrative building. This building houses the laboratory, laundry, lunch area, showers, and restroom.

<u>Standby Power Generator</u>

The plant has a single 1000-KW generator set for standby power. This is adequate to maintain majority treatment and disinfection throughout the buildout.

PROVISIONS FOR TREATMENT

The *Facility Plan* also discusses additional phases, Phase 2, which will increase sludge flow capacity, and Phase 3 as shown in Table 4-14, which will increase liquid stream capacity.

TABLE 4-14

Phase 3 MBR Hydraulic Capacity

Parameter	Value
No. MBR Membrane Basins	8
No. SMUs per Basin	5 (double-stacked)
Total SMUs	40
Average Flux at 13°C	10 GFD
Peak Flux at 13°C	26.7 GFD
Average Hydraulic Capacity per Basin	0.37 mgd
Total Average Hydraulic Capacity	3.0 mgd
Total Peak Hydraulic Capacity	6.0 mgd

RELIABILITY CLASSIFICATION

According to the City of La Center Wastewater Treatment Plant NPDES Permit, the plant must maintain a Reliability Class II. Reliability Class II requires a backup power source sufficient to operate all vital components and critical lighting and ventilation during peak wastewater flow conditions. Vital components used to support secondary processes need not be operable to full levels of treatment, but must be sufficient to maintain biota.

The existing electrical service and generator constructed during Phase 1A of the facility upgrades will accommodate the anticipated loads including Phase 2 and Phase 3. As noted above, these electrical upgrades included a 1,000-KVA utility transformer, 1,600 amp rated service entrance equipment and 1,000 KW generator with an automatic transfer switch (ATS).

Reliability Class II standards, as defined in EPA's Technical Bulletin: "Design Criteria for Mechanical, Electrical, and Fluid System Component Reliability," EPA 430-99-74-001. Table 4-15 includes a summary of the reliability criteria and requirements to be considered as part of the Alternatives Evaluation and Recommended Plan.

TABLE 4-15

EPA Class II Reliability Criteria

Treatment Unit Process	Reliability Class II Requirements
Influent Screening	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 (Liquids, Solids and Chemical Feed)	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.
Primary and Secondary Clarification	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% of the total design flow.
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.)
Aeration Blowers and/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.

TABLE 4-15 – (continued)

EPA Class II Reliability Criteria

Treatment Unit Process	Reliability Class II Requirements
Air Diffuser Systems (if applicable)	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.
Chlorine Contact Chamber	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	Two separate and independent power sources, either from two separate utility substations or from a single substation and an on-site generator. The backup power supply shall be sufficient to operate all vital components during peak wastewater flow conditions, including critical lighting and ventilation.

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 five 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 multifamily 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 Shelton 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, and 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.

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

The 2016 City Comprehensive Plan projected that the population would double between 2016 to 2035 to 7,642, equivalent to a 4.3 percent annual growth. An average household size of 2.66 persons was noted in the 2016 Clark County Comprehensive Plan as a basic planning assumption for forecasting new growth for Clark County and all jurisdictions in the county.

Washington State OFM (Office of Financial Management) census data showed a population increase from 2,800 to 3,605 between 2010 and 2021, which is equivalent to an annual growth rates of 2.3 percent. The census data from 2021 showed an average household size of 2.72 persons (3,605/1,327).

For this study, a population growth rate of 4.0 percent is used, per the direction of City staff.

According to the 2016 City Comprehensive Plan, employment is projected to increase from 825 to 2,876 between 2015 and 2035, equivalent to a 6.1 percent annual growth. For this study, an employment growth rate of 6.5 percent is used, per the direction of City staff.

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SUMMARY

The La Center WWTP receives wastewater from the majority of the City. The population for these facilities between 2016 and 2021 has grown steadily, as indicated by the estimates summarized in Table 5-1. The vast majority, but not all, of the population within the sewer service area is connected to the City's sewer system.

TABLE 5-1

Historical Population and Employee Data (2016 to 2021)

	2016	2017	2018	2019	2020	2021
Residents(1)	3,144	3,218	3,281	3,404	3,424	3,605
Employees(2)	876	929	986	1,047	1,111	1,179

(1) Source: Washington State Office of Financial Management.

(2) Source: Based on 6.1 percent growth rate between 2015 to 2036 estimated in City Comprehensive Plan.

EQUIVALENT RESIDENTIAL UNITS

Use of Equivalent Residential Units (ERUs) is a way to express the amount of sewer use by residential customers as well as non-residential customers as an equivalent number of residential customers.

To assist in the determination of the number of residential units with sewer service, the water consumption data between 2020 and 2022 was reviewed. 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.

SERVICE CONNECTIONS

The number of sewer service connections from 20120 to 2022 is provided in Table 5-2. The number of service customers has been stable throughout the years. At the end of 2022, the City had 1,497 connections. The vast majority were residential.

TABLE 5-2

Sewer Service Connections by Customer Class (2020 to 2022)

Customer Classification	Service Connections			
	2020	**2021**	**2022**	**Yearly Average**
Commercial	40	42	72	51
Public	19	19	20	19
District	1	1	1	1
Multiunit	6	6	6	6
Single Residential	1,257	1,360	1,398	1,338
Total	**1,323**	**1,428**	**1,497**	**1,416**

(1) Per the City's available water billing data (from 2020 to 2022).

ERUS

Winter water consumption is typically used to estimate wastewater volumes entering the collection system, since it does not typically include significant amounts of irrigation flows that do not enter the sewer system. The winter water consumption (November through February) for 2022 is provided in Table 5-3.

As described previously, one single-family residential sewer connection is equivalent to one ERU. To simplify the ERU calculations for the other classes, the historical water consumption by class was used to derive the converted ERUs for each class.

This translates to 174 gpd/ERU (= 233,277 gpd/ 1,338 single-family ERUs) being divided by the water usage per class. For multi-family units, a water usage of 12,521 gpd divided by 174 gpd/ERU equates to 72 multi-family ERUs.

The four cardrooms in the commercial category contribute particular high loadings to the WWTP; the ERUs are listed separately for loading projection later in the chapter.

TABLE 5-3

2022 Winter Water Use and Sewer ERUs by Customer Class

Customer Classification	Winter Water Use by Customers (gpd)[(1)]	Sewer ERUs[(2)]
Commercial – Cardrooms	8,240	47
Commercial – Others	43,635	250
Public	35,302	203
Multiunit	12,521	72
Single-Family Residential	233,277	1,338
Total	**332,976**	**1,910**

(1) Per 2022 billing data.
(2) Based upon 174 gpd/ERU (=233,277 gpd water usage / 1,338 residential ERUs).

Assuming 85 percent of the water usage enters the wastewater system (typically it is observed that 85 to 95 percent does), the ERU unit flow is 148 gpd/ERU (= 174 gpd/ERU water usage x 85 percent).

The winter water consumption derived ERU unit flow is at the middle of the range of recent values observed across the State, as shown in Table 5-4.

TABLE 5-4

Sample Wastewater Unit Flow ERUs in Western Washington

City/District	Unit Flow Per ERU	Capita/ERU
City of Burlington	138	3.36
Clark Regional Wastewater District (in Clark County)	200	2.66
Clallam Bay (in Clallam County)	98	2.0
Sekiu (in Clallam County)	158	2.0
Southwest Suburban Sewer District (in Burien, King County)	147	2.45
Alderwood Water and Wastewater District	191	2.9
City of Puyallup	182	2.43
City of Monroe	195	2.9
City of Lynnwood	175	2.5
City of Edmonds	150	2.36
City of Vancouver	243	2.7
City of Shelton	135	2.85

SEWER SERVICE POPULATION EQUIVALENTS

The City's sewer population equivalents, including residential and non-residential, was established by multiplying the number of sewer ERUs by 2.66 residents per ERU, a planning factor indicated in the *2016 City Comprehensive Plan*:

CURRENT SEWER SERVICE POPULATION EQUIVALENTS

As indicated earlier, the City's population has been relatively stable and growing slowly. Table 5-5 summarizes the estimated current sewer population equivalents and ERUs for residential and non-residential classes.

TABLE 5-5

Estimated 2022 Sewer Population Equivalents and ERUs in City

	ERUs	Population Equivalents(1)
Residential	1,410	3,751
Commercial – Cardrooms	47	126
Commercial – Others	250	666
Public	203	539
Total	**1,910**	**5,018**

(1) Determined by multiplying the number of ERUs by 2.66 people/ERU.

FUTURE SEWER SERVICE POPULATION

City of La Center

The City future populations were discussed in Chapter 2. The future population and number of ERUs projected for both residential and non-residential populations from 2023 to 2043 are presented in Table 5-6.

TABLE 5-6

Future Sewered Population and ERUs in City

Service Area and Type	2023	2028	2033	2038	2043
	Population				
City Residential	3,988	4,852	5,904	7,183	8,739
City Residential Unsewered(1)	87	65	44	22	-
City Residential Sewered	3,901	4,787	5,860	7,161	8,739
City Non- Residential(2)	1,403	1,837	2,412	3,177	4,198
Commercial – Cardrooms(3)	134	183	251	344	472
Commercial – Others(3)	709	972	1,331	1,824	2,499
Public(4)	560	682	829	1,009	1,228
	ERUs				
City Residential	1,467	1,800	2,203	2,692	3,285
City Non-Residential	528	691	907	1,195	1,578
City Total	**1,994**	**2,490**	**3,110**	**3,887**	**4,864**

(1) City Unsewered population is City total population less the sewered population, and is assumed to be connected to service at even rate throughout the planning period.
(2) City Non- Residential are all sewered.
(3) Commercial is calculated based on 6.5 percent employment growth rate between 2015 to 2036 estimated in City Comprehensive Plan.
(4) Public is calculated based on the same 4.0 percent growth rate as residential.

EXISTING WASTEWATER FLOWS AND LOADING

WWTP records for the 6-year period from January 2018 through Oct 2022 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 and ERU data to determine projected future wastewater flows and loadings.

WASTEWATER FLOWS

Table 5-7 summarizes reported WWTP flows for the 5-year period of 2018 to 2022. The average dry weather flow increased over that period, reflecting population growth. The peak day flow (PDF) typically occurs between December and March. The comparison of plant influent and rainfall in Figure 5-1 shows that wastewater flow is strongly influenced by rainfall. The peak day flow of 0.808 mgd occurred during a major storm event on December 26, 2022. Peak hour flow (PHF) of 1.054 mgd was recorded during the event, and the derived peaking factor of 1.3 (1.054/0.808) was used to estimate the PHF. Historical peaking factors are presented in Table 5-8.

TABLE 5-7

Historical WWTP Influent Flows (2018 to 2022)

Flow Type	Average Dry Weather Flow(1)	Annual Average Flow	Maximum Monthly Flow	Peak Day Flow	Peak Hour Flow(2)	Annual Rainfall
2018	0.205	0.256	0.381	0.533	0.693	41.6
2019	0.222	0.253	0.353	0.764	0.993	40.6
2020	0.205	0.272	0.430	0.650	0.845	51.8
2021	0.258	0.325	0.458	0.727	0.945	53.5
2022	0.278	0.343	0.424	0.808	1.050	61.2
Average	0.234	0.290	0.409	0.696	0.905	49.7
Maximum	0.278	0.343	0.458	0.808	1.050	61.2

(1) Average of July, August, September.
(2) PHF = 1.3*PDF based on flow recorded during Dec 26, 2022 storm event.

TABLE 5-8

WWTP Influent Flow Historical Peaking Factors (2018 to 2022)

Flow Type(1)	2018	2019	2020	2021	2022
Average Dry Weather Flow	1.0	1.0	1.0	1.0	1.0
Annual Average Flow	1.2	1.1	1.3	1.3	1.2
Maximum Monthly Flow	1.9	1.6	2.1	1.8	1.5
Peak Day Flow	2.6	3.4	3.2	2.8	2.9
Peak Hour Flow(2)	3.4	4.5	4.1	3.7	3.8

(1) Peak Factors are based on average dry weather flow.
(2) PHF assumed to equal 1.3*PDF based on flow recorded during December 26, 2022 storm event.

Monthly discharge monitoring reports (DMR) data for this period are provided in Appendix D and summarized in Table 5-9.

Graphical representations of daily, average monthly and peak day monthly WWTP flows for the period from January 2018 through December 2022 are shown in Figures 5-1 through 5-3. The figures indicate that neither the daily permit limit of 1.29 mgd nor the monthly permit limit of 0.69 mgd has been exceeded over the period of January 2018 through December 2022.

TABLE 5-9

Summary of Discharge Monitoring Reports (DMRs)
WWTP Influent Monthly Averages

Year	Avg. Monthly Flow (mgd)	Max. Daily Flow (mgd)	Avg. Monthly BOD_5 (mg/L)	Avg. Monthly BOD_5 (lb/d)	Avg. Monthly TSS (mg/L)	Avg. Monthly TSS (lb/d)	Avg. Monthly NH3 (mg/L)	Avg. Monthly NH3 (lb/d)
Jan-18	0.381	0.533	203	618	148	448	18	53
Feb-18	0.322	0.431	214	535	188	468	22	54
Mar-18	0.303	0.424	235	560	188	446	24	58
Apr-18	0.286	0.384	236	550	199	467	23	53
May-18	0.208	0.243	292	505	223	384	32	55
Jun-18	0.218	0.246	283	502	219	389	30	53
Jul-18	0.219	0.265	274	483	214	375	31	54
Aug-18	0.200	0.230	298	481	231	372	32	51
Sep-18	0.196	0.250	313	505	253	407	35	57
Oct-18	0.210	0.304	268	450	217	364	31	51
Nov-18	0.227	0.274	283	485	213	368	31	53
Dec-18	0.301	0.466	223	518	168	387	22	49
Jan-19	0.296	0.391	233	561	170	405	23	55
Feb-19	0.353	0.764	198	558	155	431	18	50
Mar-19	0.251	0.320	261	550	202	428	27	56
Apr-19	0.286	0.457	244	526	188	411	26	56
May-19	0.223	0.250	325	596	253	465	32	59
Jun-19	0.213	0.238	288	490	227	385	33	55
Jul-19	0.217	0.239	311	554	237	422	31	55
Aug-19	0.221	0.243	315	580	233	431	32	59
Sep-19	0.227	0.324	298	552	205	383	31	58
Oct-19	0.220	0.309	320	571	215	385	33	59
Nov-19	0.220	0.252	328	596	227	413	34	61
Dec-19	0.308	0.599	256	586	188	434	26	58
Jan-20	0.430	0.536	174	604	118	414	16	56
Feb-20	0.344	0.551	229	581	176	445	22	55
Mar-20	0.271	0.374	241	518	195	416	27	57
Apr-20	0.238	0.310	270	538	231	459	32	63
May-20	0.219	0.251	294	530	226	406	34	62
Jun-20	0.256	0.423	277	599	212	459	29	61
Jul-20	0.209	0.237	348	612	262	459	35	60
Aug-20	0.200	0.213	359	595	260	431	36	60
Sep-20	0.205	0.264	329	559	272	461	36	61
Oct-20	0.225	0.318	297	548	243	452	36	67
Nov-20	0.309	0.419	261	637	221	540	29	69
Dec-20	0.357	0.650	221	620	179	503	25	66
Jan-21	0.417	0.713	209	755	156	572	18	62
Feb-21	0.435	0.557	178	642	143	524	16	59

TABLE 5-9 – (continued)

Summary of Discharge Monitoring Reports (DMRs)
WWTP Influent Monthly Averages

Year	Avg. Monthly Flow (mgd)	Max. Daily Flow (mgd)	Avg. Monthly BOD_5 (mg/L)	Avg. Monthly BOD_5 (lb/d)	Avg. Monthly TSS (mg/L)	Avg. Monthly TSS (lb/d)	Avg. Monthly NH3 (mg/L)	Avg. Monthly NH3 (lb/d)
Mar-21	0.304	0.360	243	606	189	469	25	63
Apr-21	0.260	0.283	273	578	219	463	32	67
May-21	0.251	0.289	291	603	233	483	34	69
Jun-21	0.268	0.339	304	669	235	519	31	67
Jul-21	0.263	0.291	315	684	231	502	31	68
Aug-21	0.249	0.273	321	658	261	533	33	67
Sep-21	0.261	0.413	334	699	236	498	35	75
Oct-21	0.314	0.459	260	649	226	560	34	87
Nov-21	0.416	0.727	230	711	195	601	24	75
Dec-21	0.458	0.638	196	723	148	545	20	74
Jan-22	0.420	0.746	194	665	158	543	21	72
Feb-22	0.338	0.687	283	779	230	630	28	75
Mar-22	0.404	0.624	193	683	162	574	23	77
Apr-22	0.386	0.546	187	619	166	546	23	77
May-22	0.372	0.550	246	713	197	574	27	77
Jun-22	0.345	0.515	296	792	207	572	30	83
Jul-22	0.291	0.307	312	760	244	594	30	72
Aug-22	0.278	0.358	307	705	225	518	32	74
Sep-22	0.266	0.306	345	757	255	560	40	87
Oct-22	0.259	0.328	352	731	242	500	40	82
Nov-22	0.328	0.717	309	810	211	551	30	76
Dec-22	0.424	0.808	283	911	187	590	24	74
Average	***0.290***	***0.414***	***271***	***613***	***209***	***472***	***28***	***64***
Maximum	***0.458***	***0.808***	***359***	***911***	***272***	***630***	***40***	***87***
Minimum	***0.196***	***0.213***	***174***	***450***	***118***	***364***	***16***	***49***

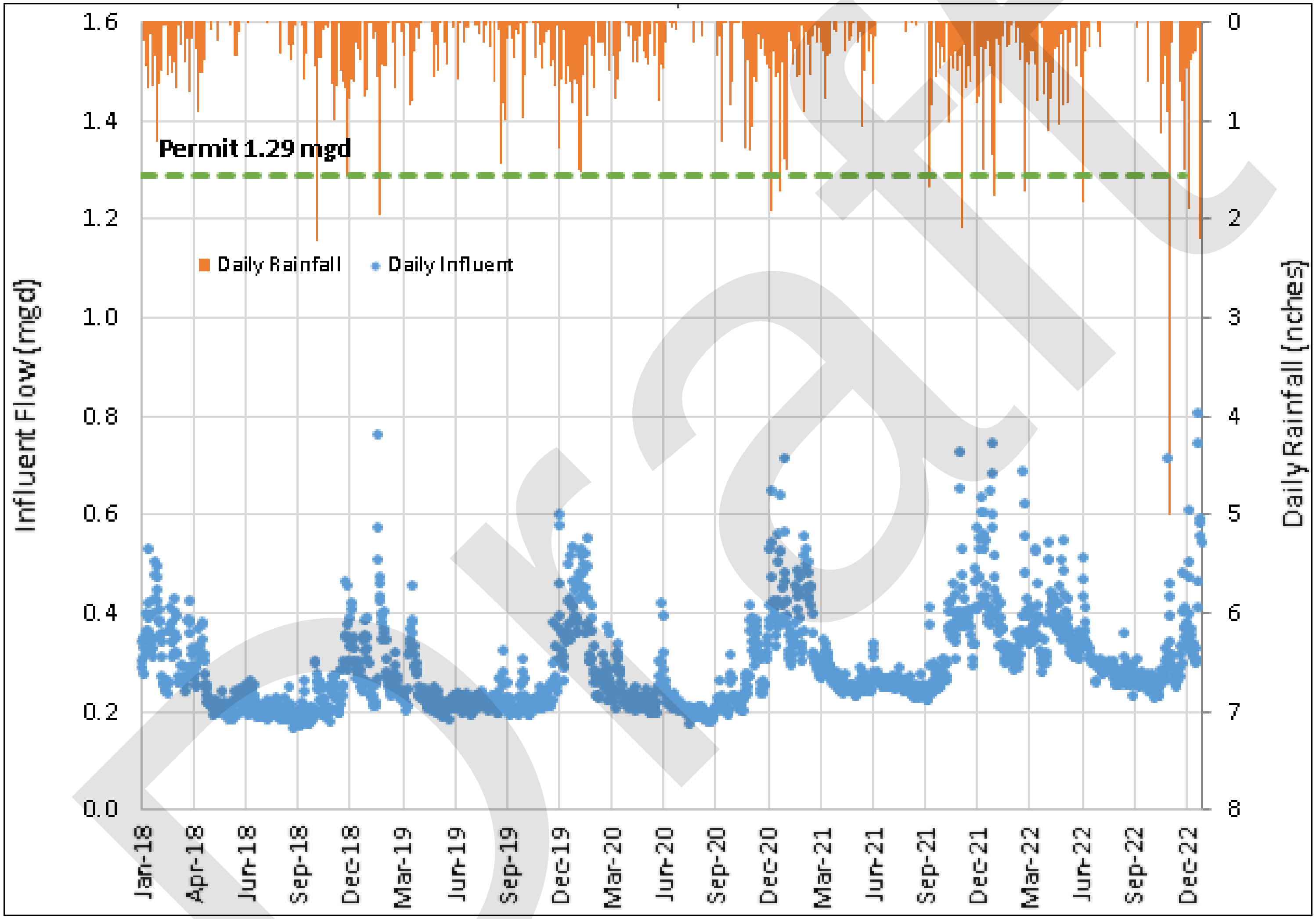


FIGURE 5-1

WWTP Daily Influent Flow

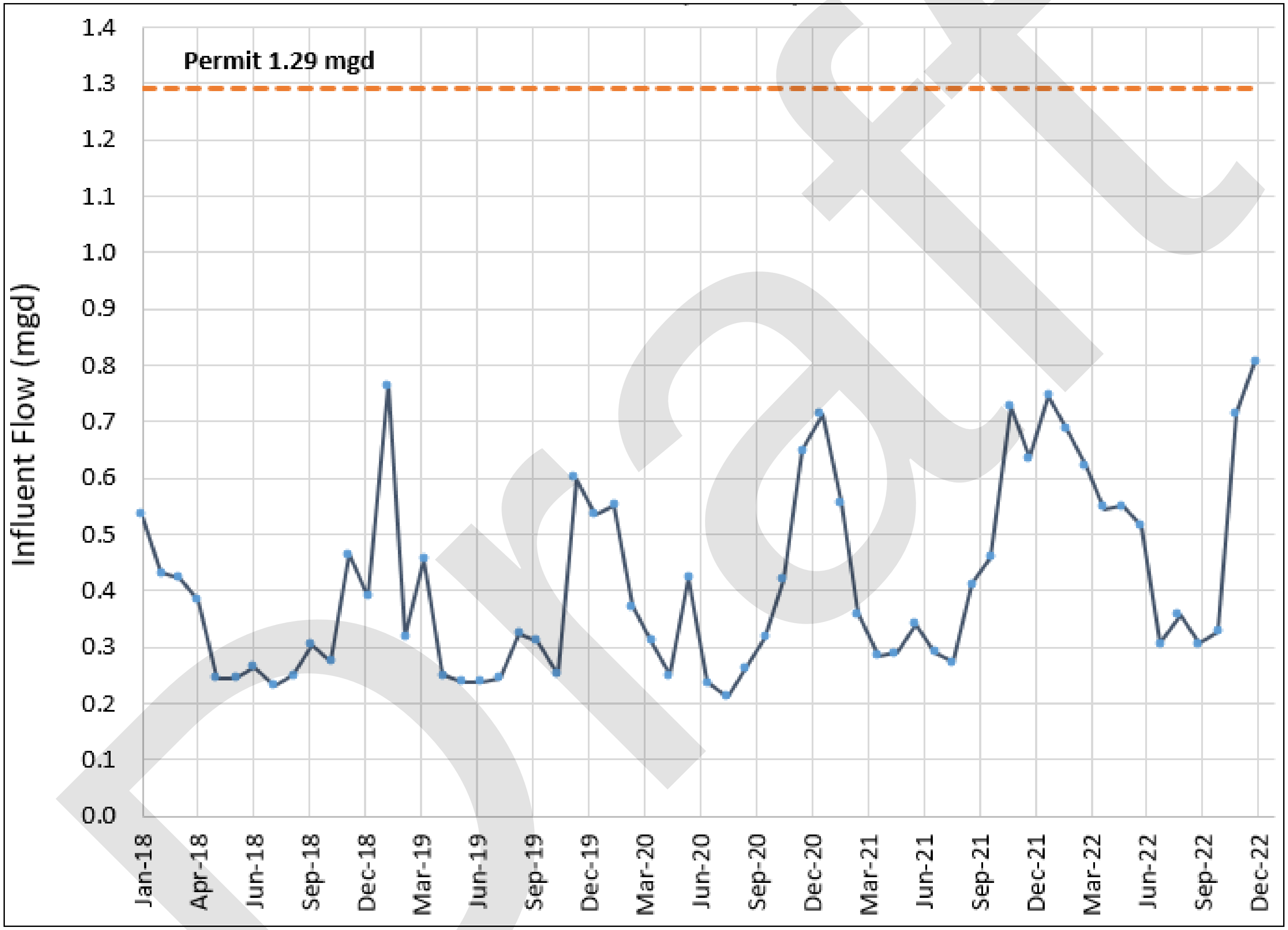


FIGURE 5-2

WWTP Monthly Peak Day Influent Flow

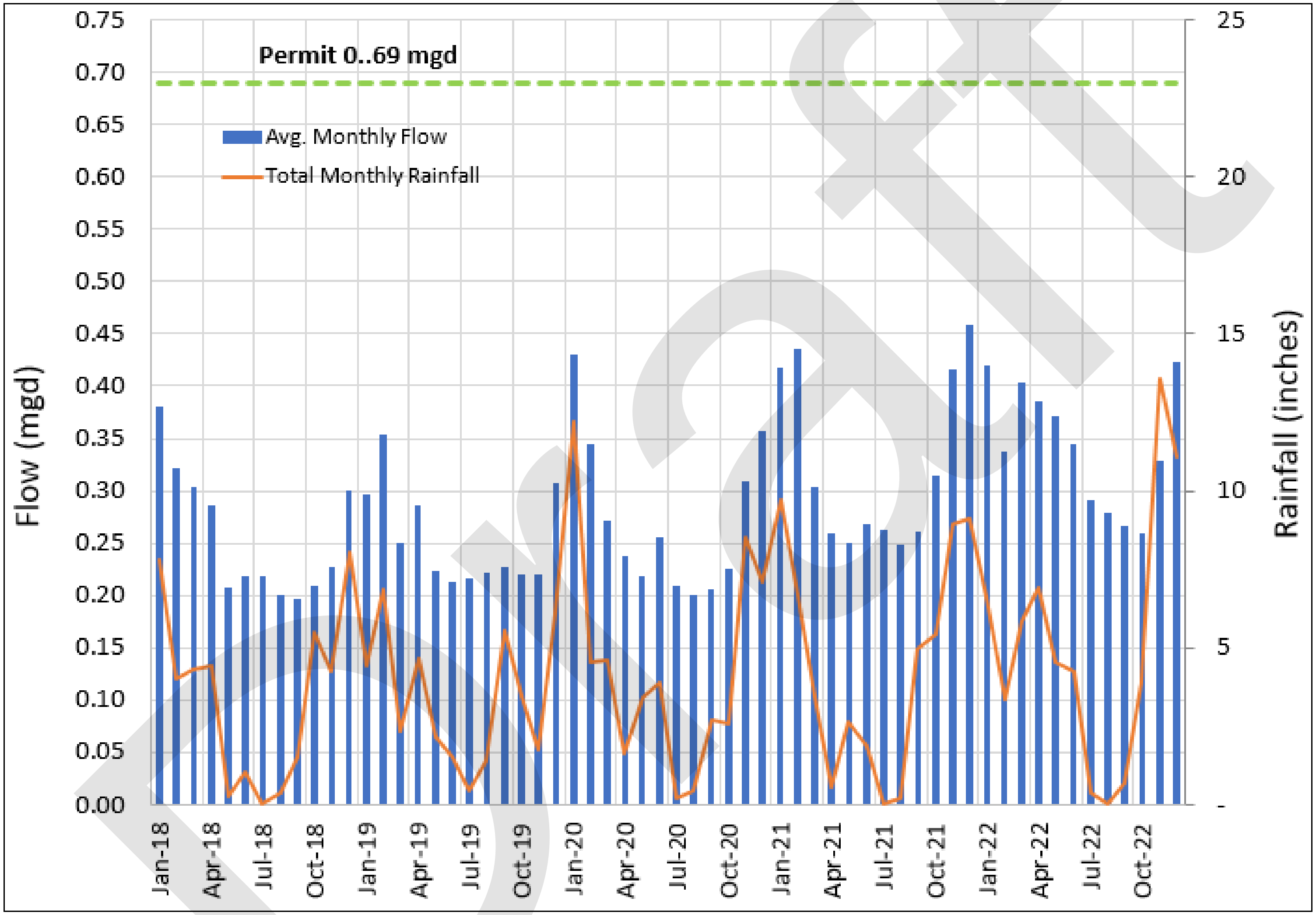


FIGURE 5-3

WWTP Monthly Average Influent Flow

UNIT FLOW FACTOR

A unit flow factor will serve as the basis for projecting future flows within the service area. As previously discussed, the winter water usage for the area suggests a unit flow of 148 gpd/ERU.

WASTEWATER LOADING

Influent BOD_5, TSS and NH_3 loadings as measured for the period from January 2018 through December 2022 are shown in Figure 5-4 through 5-9. The annual average, maximum month, and peak day BOD_5 and TSS loadings for 2018 through 2022 are summarized in Table 5-10. The loadings have been steadily increasing as shown from the reported data. Historical peaking factors are presented in Table 5-11. The peaking factors for year 2022 are used to determine future loadings, since they are considered representative of current loading conditions.

TABLE 5-10

WWTP Influent Annual Average Loadings

Year	Annual Average			Max. Month			Peak Day		
	BOD5 (lb/d)	TSS (lb/d)	NH3 (lb/d)	BOD5 (lb/d)	TSS (lb/d)	NH3 (lb/d)	BOD5 (lb/d)	TSS (lb/d)	NH3 (lb/d)
2018	516	406	53	618	468	58	741	560	63
2019	560	416	57	596	465	61	723	584	72
2020	578	454	61	637	540	69	818	683	92
2021	665	522	69	755	601	87	1,005	951	111
2022	744	563	77	911	630	87	1,111	781	136
Average	613	472	64	703	590	81	978	805	113

TABLE 5-11

WWTP Influent Loading Historical Peaking Factors (2018 to 2022)

Loading Type	2018	2019	2020	2021	2022	Selected PF
BOD5 Loading						
Annual Average	1.0	1.0	1.0	1.0	1.0	1.0
Max. Month	1.2	1.1	1.1	1.1	1.2	1.2
Peak Day	1.4	1.3	1.4	1.5	1.5	1.5
TSS Loading						
Annual Average	1.0	1.0	1.0	1.0	1.0	1.0
Max. Month	1.2	1.1	1.2	1.2	1.1	1.1
Peak Day	1.4	1.4	1.5	1.8	1.4	1.4
NH3 Loading						
Annual Average	1.0	1.0	1.0	1.0	1.0	1.0
Max. Month	1.1	1.1	1.1	1.3	1.1	1.1
Peak Day	1.2	1.3	1.5	1.6	1.8	1.8

BOD_5 Loading

Daily Influent BOD_5 concentrations ranged from 107 mg/L to 493 mg/L. As illustrated in Figure 5-4, the average monthly BOD_5 concentration appears to correlate inversely with rainfall. This provides further evidence of significant inflow and infiltration in the City's wastewater collection system.

As would be expected with a system with significant infiltration and inflow, the historical record indicates that the BOD_5 loading to the wastewater treatment facility has been more consistent than the concentration. Monthly average influent BOD_5 loadings ranged from 450 lb/d to 911 lb/d for the period of analysis, with no apparent correlation with season or rainfall, as shown in Figure 5-5.

The NPDES monthly average influent BOD_5 loading of 1,297 lb/d has not been exceeded frequently during the period of analysis.

The average influent BOD_5 concentration for the 5-year period is 271 mg/L, which would be considered moderate strength domestic wastewater. The average BOD_5 loading between 2018 and 2022, is summarized in Table 5-10 and was 613 lb/d.

Total Suspended Solids Loading

Daily influent TSS concentrations from January 2018 through December 2022 ranged from 68mg/L to 410 mg/L. As shown in Figure 5-6, the average monthly concentration of TSS, like that of BOD_5, appears to correlate inversely with rainfall.

The monthly average TSS loadings ranged from 364 lb/d to 630 lb/d. Similar to BOD_5, the mass loading of TSS appears to be more consistent than concentrations on a monthly basis. There have been no exceedances of the plant loading limit of 1,070 lb/d during the period of analysis.

The average influent TSS concentration is 209 mg/L, which would be considered moderate strength domestic wastewater. As summarized in Table 5-10, the average TSS loading during 2018 to 2022 was 472 lb/d.

Total Ammonia Loading

Daily influent NH_3 concentrations from January 2018 through December 2022 ranged from 8.9 mg/L to 60.6 mg/L. As shown in Figure 5-8, the average monthly concentration of ammonia nitrogen (NH_3-N), like that of BOD_5 and TSS, appears to correlate inversely with rainfall.

The monthly average TSS loadings ranged from 49 lb/d to 87 lb/d. Similar to BOD_5 and TSS, the mass loading of ammonia nitrogen appears to be more consistent than concentrations on a monthly basis. There have been no exceedances of the plant loading limit of 194 lb/d during the period of analysis.

The average influent NH_3-N concentration is 28 mg/L, which would be considered moderate strength domestic wastewater. As summarized in Table 5-10, the average NH_3-N loading during 2018 to 2022 was 64 lb/d.

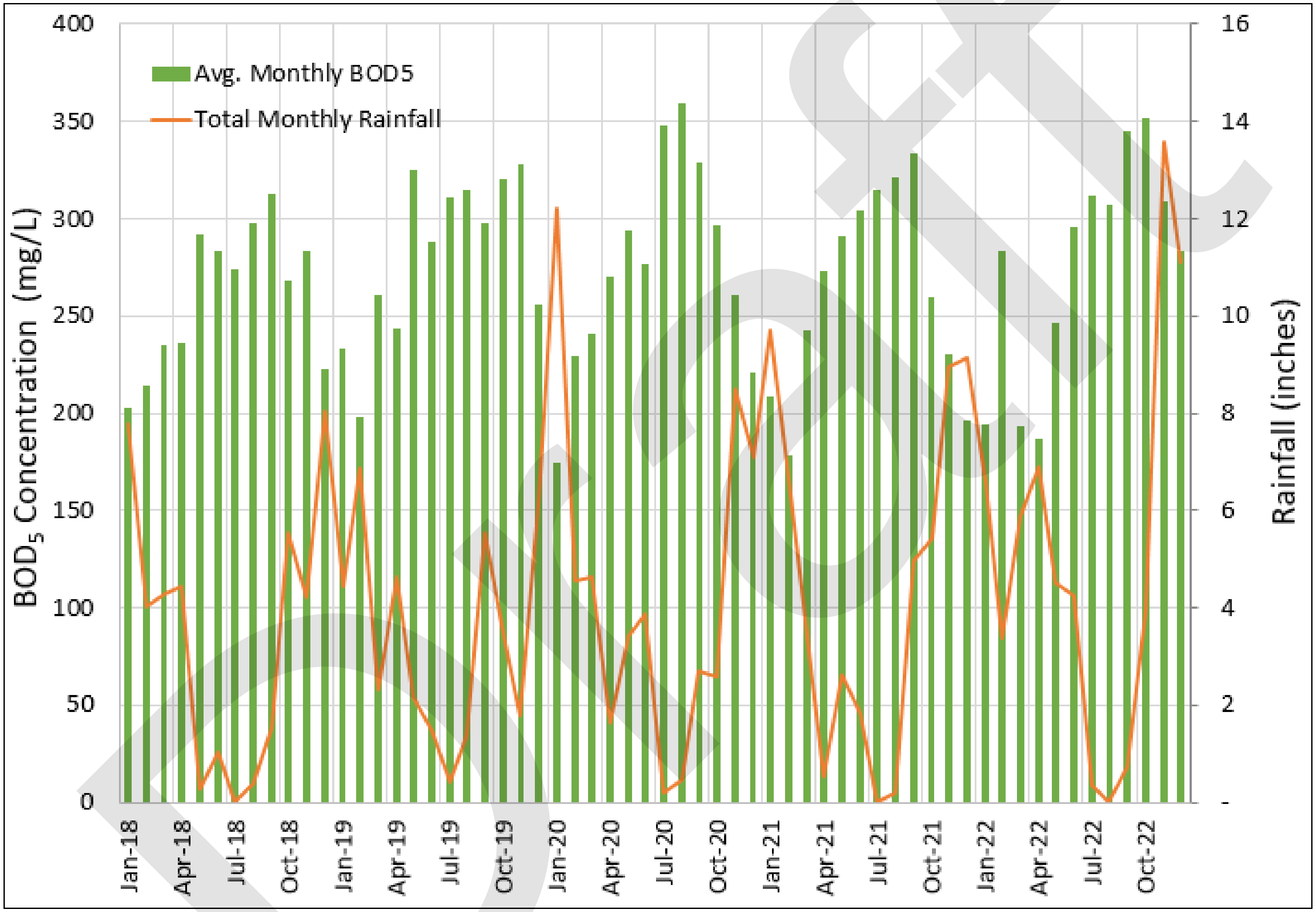


FIGURE 5-4

Monthly Average WWTP Influent BOD_5 Concentrations

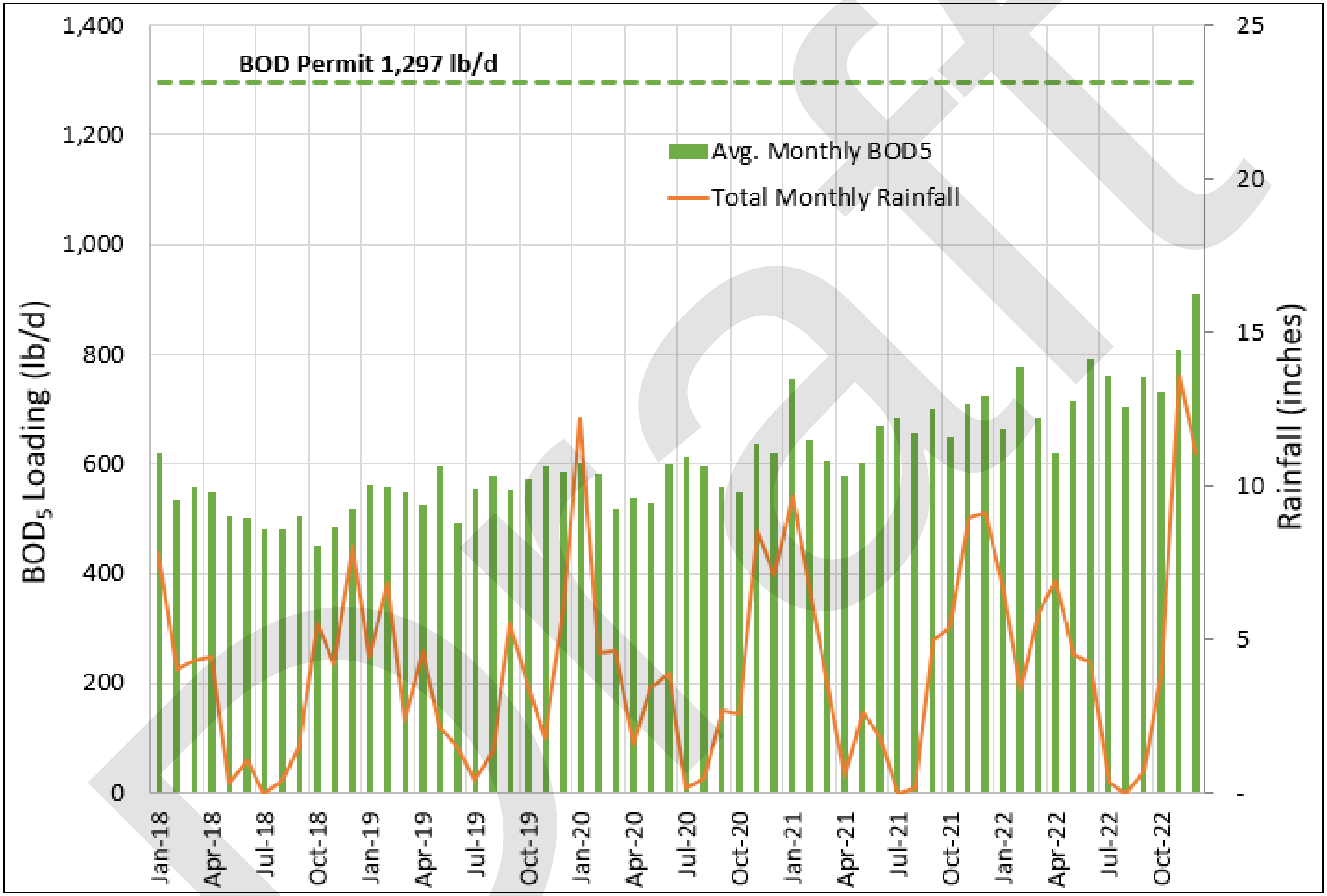


FIGURE 5-5

Monthly Average WWTP Influent BOD_5 Loadings

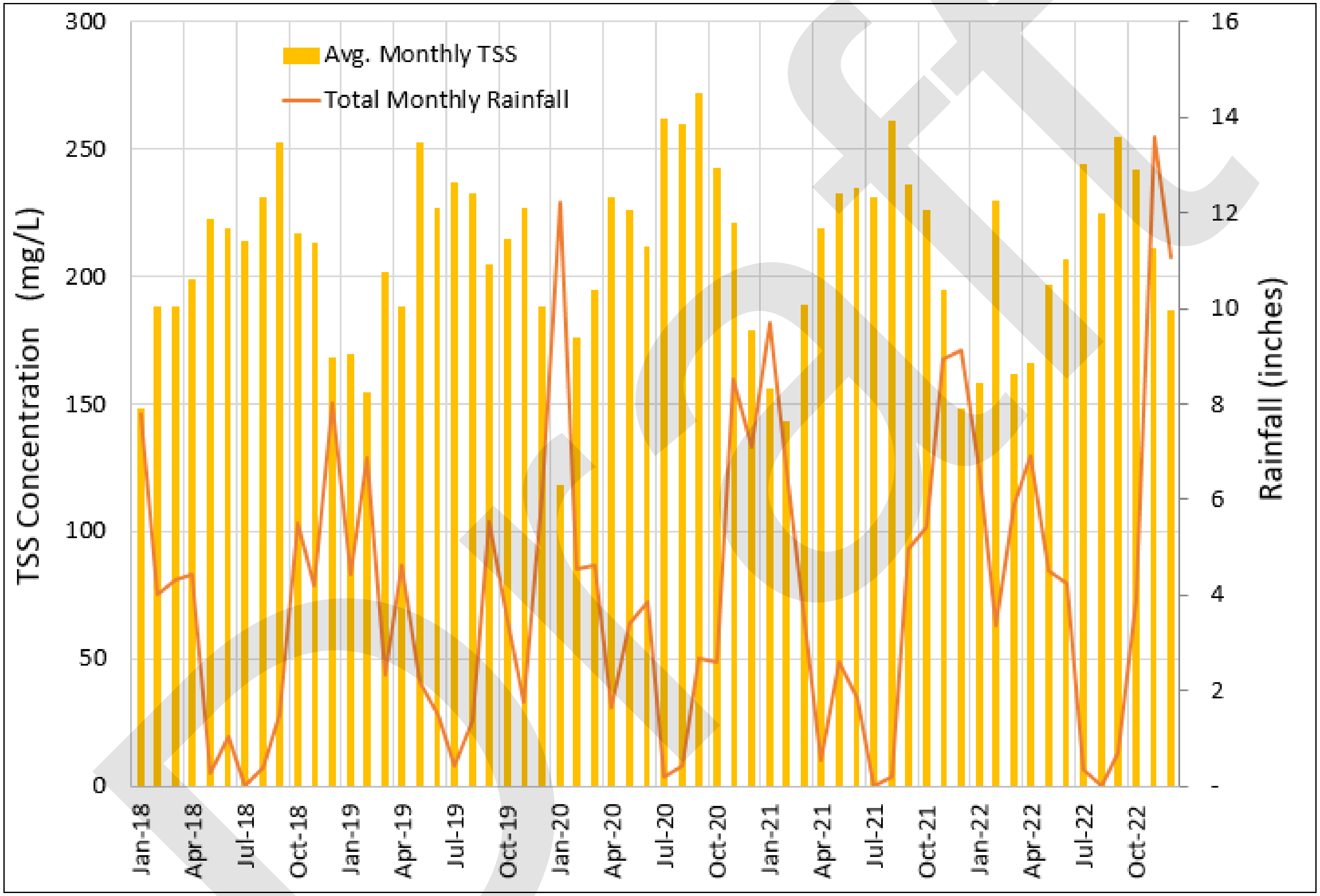


FIGURE 5-6

Monthly Average WWTP Influent TSS Concentrations

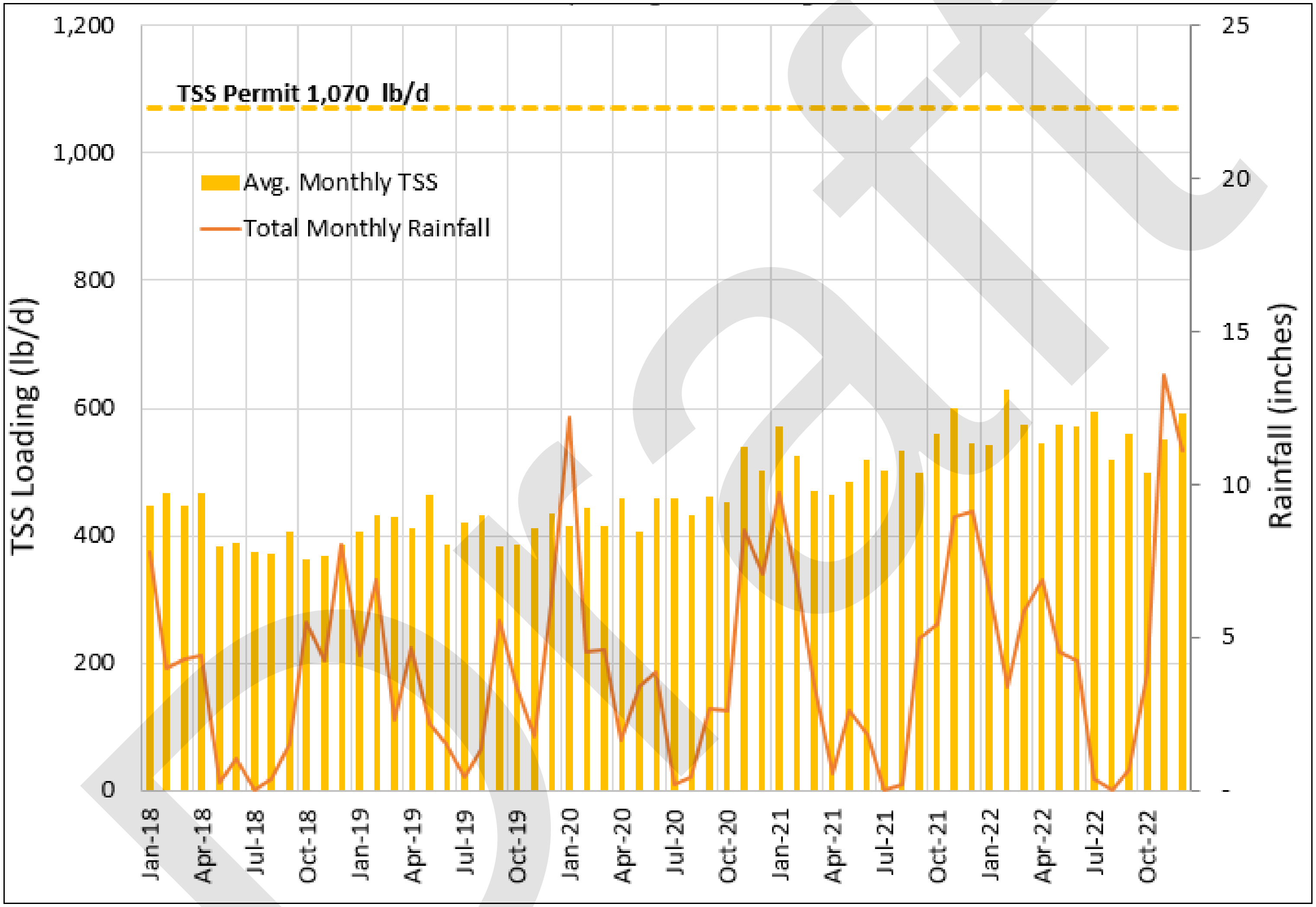


FIGURE 5-7

Monthly Average WWTP Influent TSS Loadings

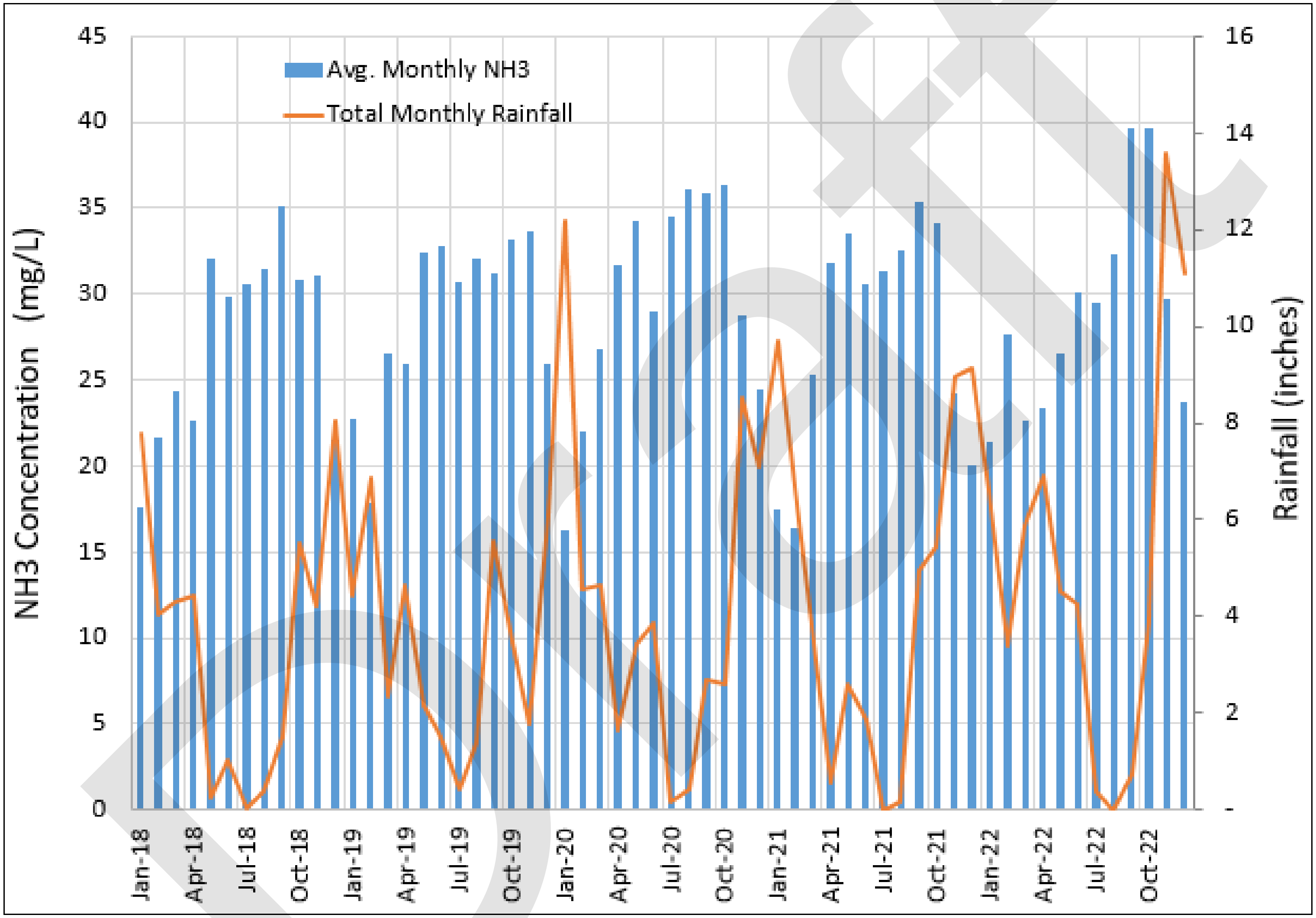


FIGURE 5-8

Monthly Average WWTP Influent NH3 Concentrations

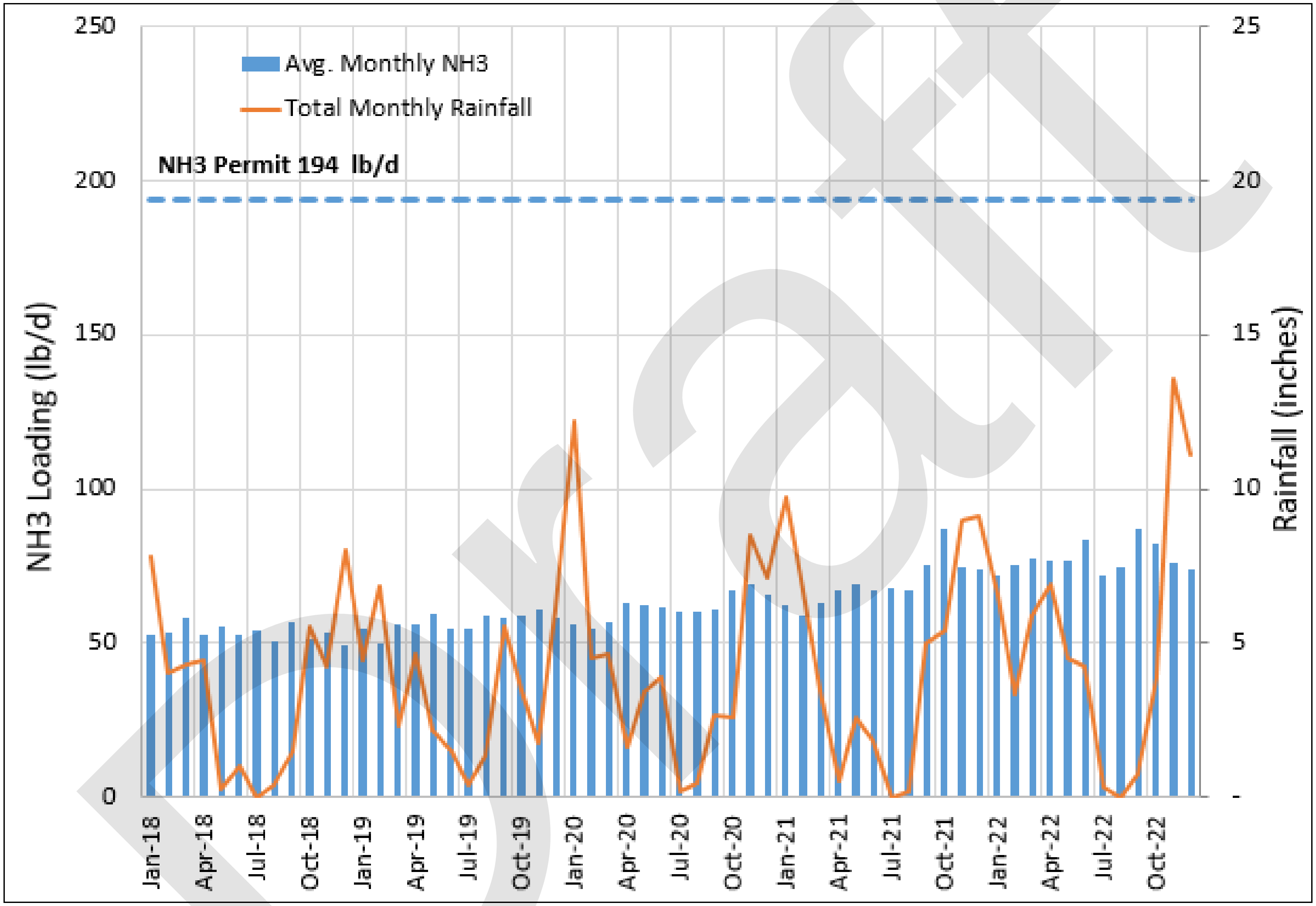


FIGURE 5-9

Monthly Average WWTP Influent NH3 Loadings

UNIT WASTEWATER LOADING FACTORS

Unit loading factors were developed using existing loading and population data for projecting future BOD, TSS and NH_3 loading in the service area. The unit loading factors were established on an ERU basis, and calculated by dividing the annual average loading over the previous years by the service area ERUs.

The wastewater generated by cardrooms is typically high strength. Since recent sampling data were not available, a unit loading factor was derived using Orange Book values for restaurants contributing 50 gpd per seat, and 0.3 lbs/day of BOD and TSS, equivalent to 0.9 lb/d-ERU of BOD and TSS per ERU, based on 148 gpd/ERU. (=148/50*0.3).

The non-cardroom wastewater ERU loading factor was calculated by dividing the total WWTP influent loading minus the cardroom loading by the non-cardroom ERUs. Table 5-12 summarized unit loading factors to be utilized in load projection.

TABLE 5-12

Current Wastewater Loading Factors

	ERUs		
Cardrooms ERUs	47		
Non Cardrooms ERUs	1,863		
	BOD, lb/d	**TSS, lb/d**	**NH3,lb/d(1)**
Cardrooms Annual Average Unit Loading (per ERUs)	0.9	0.9	0.18
Cardrooms Annual Average Loading	42	42	8
WWTP Annual Average Loading	744	563	77
Non Cardrooms Annual Average Loading	702	521	69
Non Cardrooms Annual Average Unit Loading (per ERU)	0.38	0.28	0.04

(1) Based on typical NH3/BOD factor of 0.2.

NPDES PERMIT LOADING LIMITS

Tables 5-13 presents a summary of current flows and loadings compared to the flow and loading limits listed in the current NPDES permits for the WWTP.

The maximum values for the last 5 years were used for comparisons of influent flows and loadings to influent permit limitations. The flow rate was approximately 66 percent of the NPDES monthly limit and 63 percent of the daily limit. BOD, TSS and NH3 loadings were 70, 59 and 45 percent of the NPDES limit, respectively.

TABLE 5-13

WWTP Influent Flow and Loading Limits

Parameter	Units	Current Influent Value	NPDES Permit Limit	Percent of Current NPDES Permit Limit
Max. Month Flow	mgd	0.46	0.69	66%
Max. Day Flow	mgd	0.81	1.29	63%
Max. Month BOD	lb/d	911	1,297	70%
Max. Month TSS	lb/d	630	1,070	59%
Max. Month NH3	lb/d	87	194	45%

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 base 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 La Center is estimated to be approximately 397 acres.

Table 5-14 summarizes the infiltration/inflow analysis for current conditions. The winter of 2022/2023 was evaluated, since it best reflects the current conditions. The peak day flow and peak hour flow were derived from a Dec 26, 2022 storm event, which was determined to be similar to a 20-year storm event for the region.

The data contained in this table is useful as a baseline for evaluating changes in infiltration and inflow in the future.

TABLE 5-14

Estimated WWTP Infiltration and Inflow during 2021/2022 Winter

Flow Type	Influent Flow at WWTP (mgd)	Base Flow (mgd)	I/I (mgd)	Service Area (acre)	I/I (gpad)
Annual Average	0.343	0.278	0.058	397	162
Max. Month	0.458	0.278	0.180	397	453
Peak Day	0.808(1)	0.278	0.468	397	1,334
Peak Hour	1.050(1)	0.390(2)	0.661	397	1,664

(1) Peak day and peak hour flow derived from December 26, 2022 data.
(2) PH/PD for base flow has diurnal peaking factor of 1.4, identified through flow data during dry weather week of 9/15-9/22, 2022

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 5-15.

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 seven to fourteen day non-rainfall period during seasonal high ground water conditions.

2. To determine if excessive *inflow* is present in a collection system, the USEPA 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 5-15

Per Capita WWTP Infiltration and Inflow Based on EPA Criteria

Parameter	EPA Criteria for Excessive I/I (gpcd)	Estimated La Center I/I Value (gpcd)
EPA Excessive Infiltration Criteria	120	89
EPA Excessive Inflow Criteria	275	215

<u>Infiltration</u>

Rainfall records from the WWTP DMR data show a 7-day period (January 22 through 28, 2022) 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 was 0.333 mgd. With a total population of sewer users in 2022 of 3,751, the “EPA I/I Infiltration Value” for La Center is estimated at 89 gpcd which is less than the EPA guideline of 120 gpcd and; therefore, indicates there is not excessive infiltration.

<u>Inflow</u>

The maximum day influent flow at the WWTP over the winter of 2022/2023 was 0.808 mgd (recorded on December 26, 2022). With a total population of sewer users in 2022 of 3,751, the "EPA I/I Inflow Value" for is estimated at 215 gpcd. Because this value is lower than the EPA guideline of 275 gpcd, La Center is not considered to have excessive inflow by EPA criteria.

<u>I/I Reduction</u>

Figure 5-10 shows average monthly influent flows from 2018 through 2022 as a function of total monthly rainfall during the wet season months of November through April.

The increase of the extrapolated y- intercept value, which represents the "no rain" day flow, from 2018 to 2022, indicate there is an increase in base flow and perhaps dry weather infiltration. The similarity of slopes of the linear regression lines throughout the years indicates the I/I has been stable in the collection system.

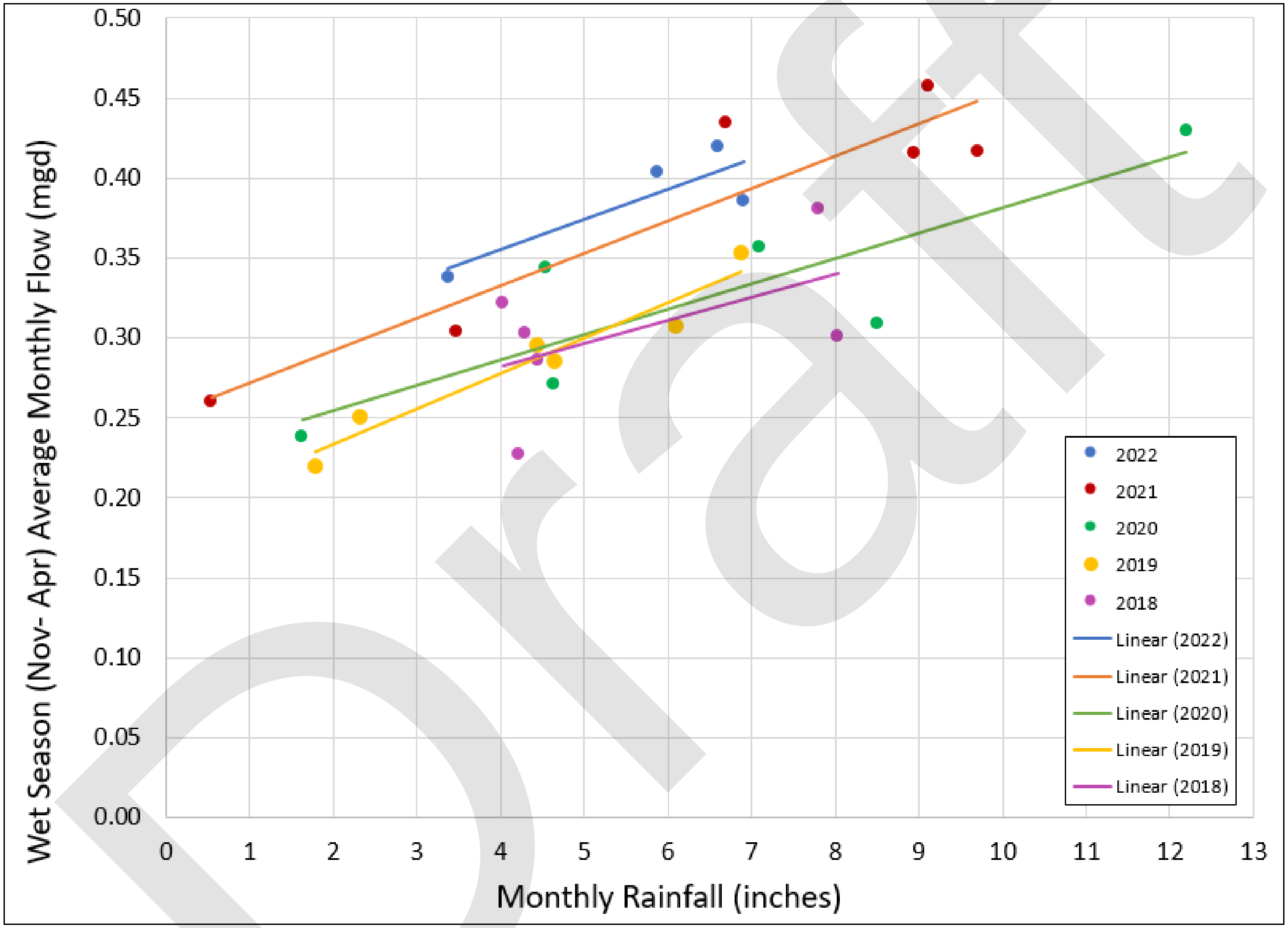


FIGURE 5-10

WWTP Influent Flow as a Function of Rainfall

FLOW AND LOADING PROJECTIONS

PROJECTED ADWF

The projected future ADWF is summarized in Table 5-16. Total residential and non-residential dry weather flows in the City collection system were projected by multiplying the projected number of ERUs (from Table 5-6) by 148 gallons per ERU per day (gal/ERU/day).

TABLE 5-16

Projected Future ERUs and ADWF

	WWTP Total ERUs	WWTP ADWF(1)
2023	1,994	0.30
2028	2,490	0.37
2033	3,110	0.46
2038	3,887	0.58
2043	4,864	0.72

(1) Based on 148 gpd/ERU.

PROJECTED I/I

For this plan, infiltration and inflow for the *existing* service area is assumed to be constant throughout the 20-year planning period. (In other words, ongoing I/I rehabilitation efforts are assumed to compensate for the increase in new I/I due to deterioration of infrastructure). Projected I/I flow is summarized in Table 5-16.

TABLE 5-17

Current and Projected Future I/I

		I/I Flow (mgd)			
Year	**Existing Service Area(1) (acres)**	**Annual Average**	**Max. Month**	**Peak Day**	**Peak Hour**
2022	397	0.06	0.18	0.53	0.66
		I/I Rates for New Service Areas (gpad)			
		150	300	1000	1,500
Year	**New Service Areas(2) (acres)**	**Total I/I Flow (mgd)**			
2028	99	0.08	0.21	0.63	0.81
2033	222	0.10	0.25	0.75	0.99
2038	377	0.12	0.29	0.91	1.23
2043	571	0.15	0.35	1.10	1.52

(1) Existing Service Area reflects sum of currently served parcels.
(2) New Service Area reflects same growth rate as service ERUs.

SUMMARY OF PROJECTED FLOWS

Table 5-18 and Figure 5-11 summarizes projected total flows within the City to the WWTP. To estimate future annual average, maximum month, and peak day flows, the I/I flow rates were added to the ADWF derived from the ERU projections to obtain the respective future WWTP influent flowrates. The projected MMF is expected to exceed the Phase 1A and 1B limits before 2033 and 2043, respectively. The projected PDF is expected to exceed the Phase 1A limit before 2035 and not exceed the Phase 1B limit throughout planning period.

TABLE 5-18

Current and Projected Future WWTP Flow

Flow Type	Current NPDES Permit Limit (Phase 1a)	Future NPDES Permit Limit (Phase 1b)	Projected Flows (mgd)				
			2023	2028	2033	2038	2043
Average Dry Weather	--		0.30	0.37	0.46	0.58	0.72
Average Annual(1)	--		0.34	0.45	0.56	0.70	0.87
Maximum Month(1)	0.69	1.04	0.46	0.58	**0.71**	**0.87**	**1.07**
Peak Day(1)	1.29	1.94	0.81	1.00	1.21	**1.48**	**1.82**
Peak Hour(1)	--		1.05	1.33	1.64	2.03	2.53

(1) AAF, MMF, PDF and PHF were the summation of ADWF in Table 5-16 and I/I flow in Table 5-17. Flows are reflective of the 20-year storm event that occurred in the winter of 2022. BOLD values exceed anticipated NPDES Permit Limits (current phase 1a and future phase 1b design limits).

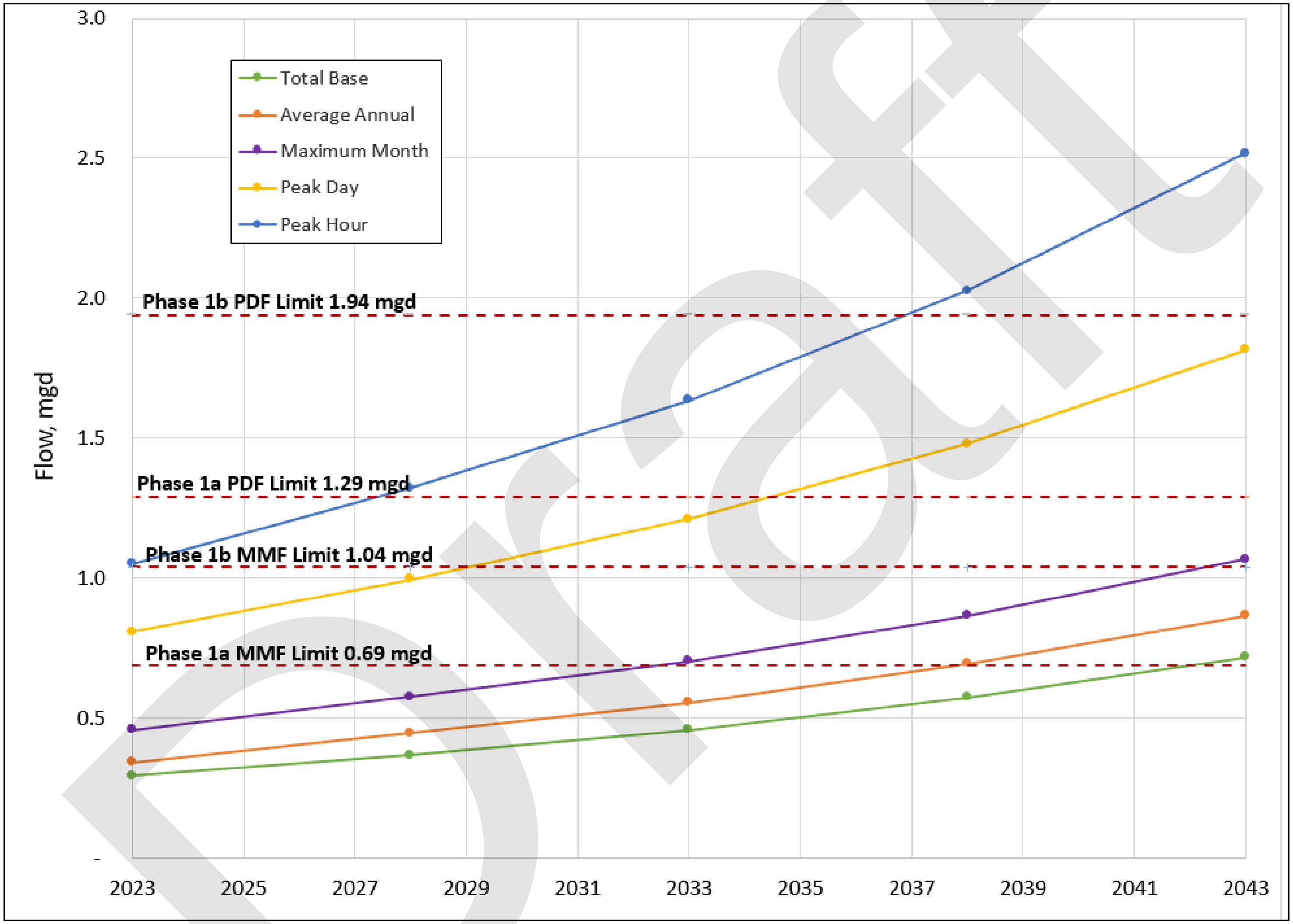


FIGURE 5-11

WWTP Influent Flow Projections

PROJECTED WASTEWATER LOADING

Future BOD_5, TSS and NH3-N annual average WWTP loadings are estimated by multiplying the projected number of ERUs in the City collection system by the respective ERU-based loadings calculated in previous sections. The max month and peak day loading were based on the peaking factors calculated in Tables 5-11.

The strength of the combined industrial/commercial (except cardroom) wastewater is assumed to be the same as that of domestic wastewater for this analysis.

Tables 5-19 and Figure 5-12 through 5-14, provide a summary of projected future influent loadings at the WWTP, including the years that they are projected to reach capacity based on the City's robust growth projections.

TABLE 5-19

Current and Projected WWTP Influent Loadings

ERUs and Loadings (lb/d)	Current NPDES Permit Limit (Phase 1a)	Future NPDES Permit Limit (Phase 1b)	2023	2028	2033	2038	2043
Total ERUs			1,994	2,490	3,110	3,887	4,864
Cardrooms ERUs			50	69	95	129	177
Non-Cardrooms ERUs			1,944	2,421	3,015	3,757	4,686
Annual Average BOD_5			777	973	1,220	1,530	1,923
Max Month BOD_5	1,297	1,804	952	1,192	**1,494**	**1,874**	**2,355**
Peak Day BOD_5			1,161	1,454	1,822	2,286	2,872
Annual Average TSS			588	738	927	1,165	1,467
Max Month TSS	1,070	1,581	658	826	1,038	**1,304**	**1,643**
Peak Day TSS			816	1,024	1,286	1,617	2,037
Annual Average NH_3-N			81	102	128	162	205
Max Month NH_3-N	194	292	91	115	145	183	231
Peak Day NH_3-N			142	179	226	285	360

(1) Values that exceed NPDES Permit limits are shown in **BOLD**.

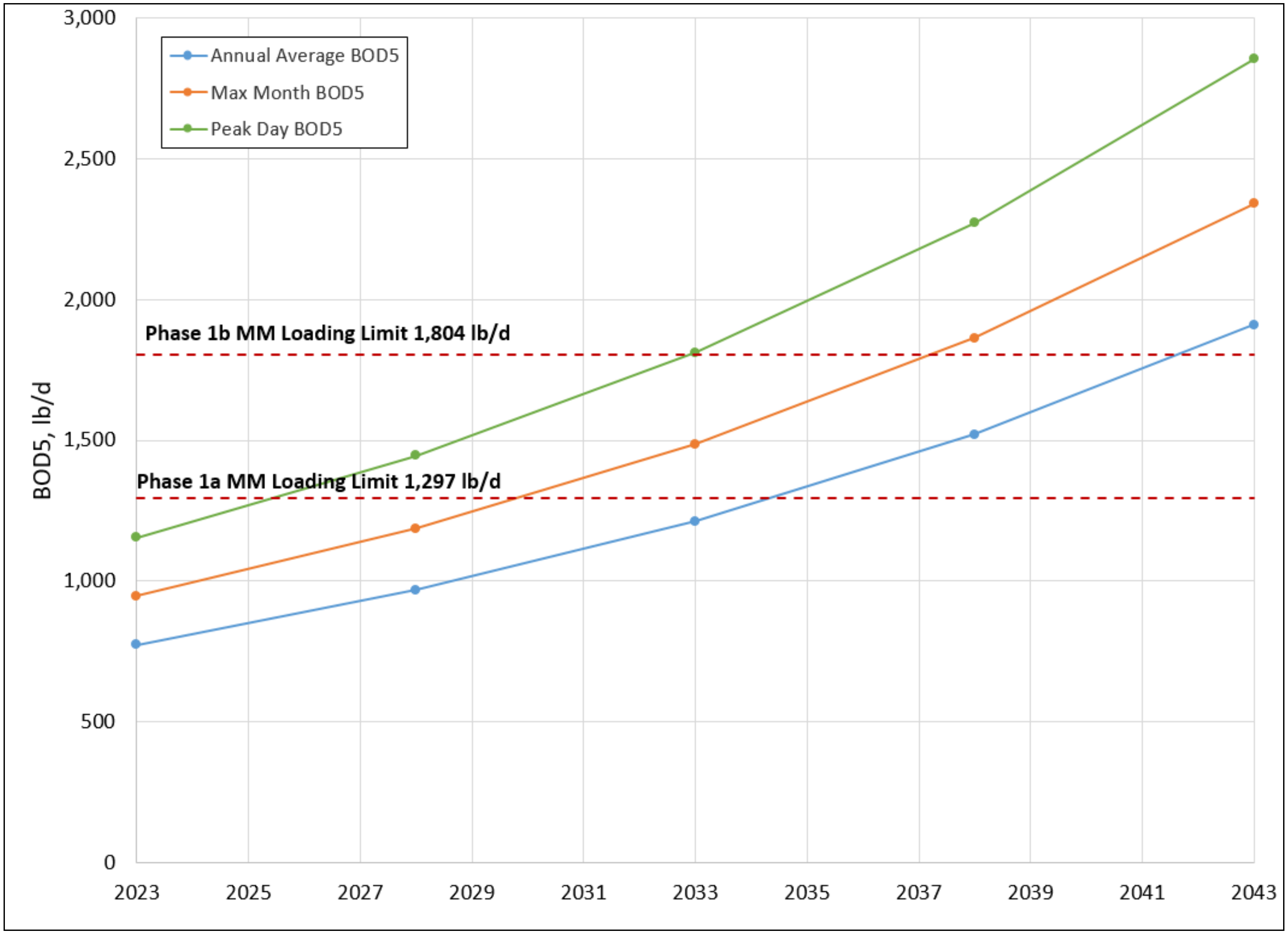


FIGURE 5-12

Projected WWTP BOD_5 Loading

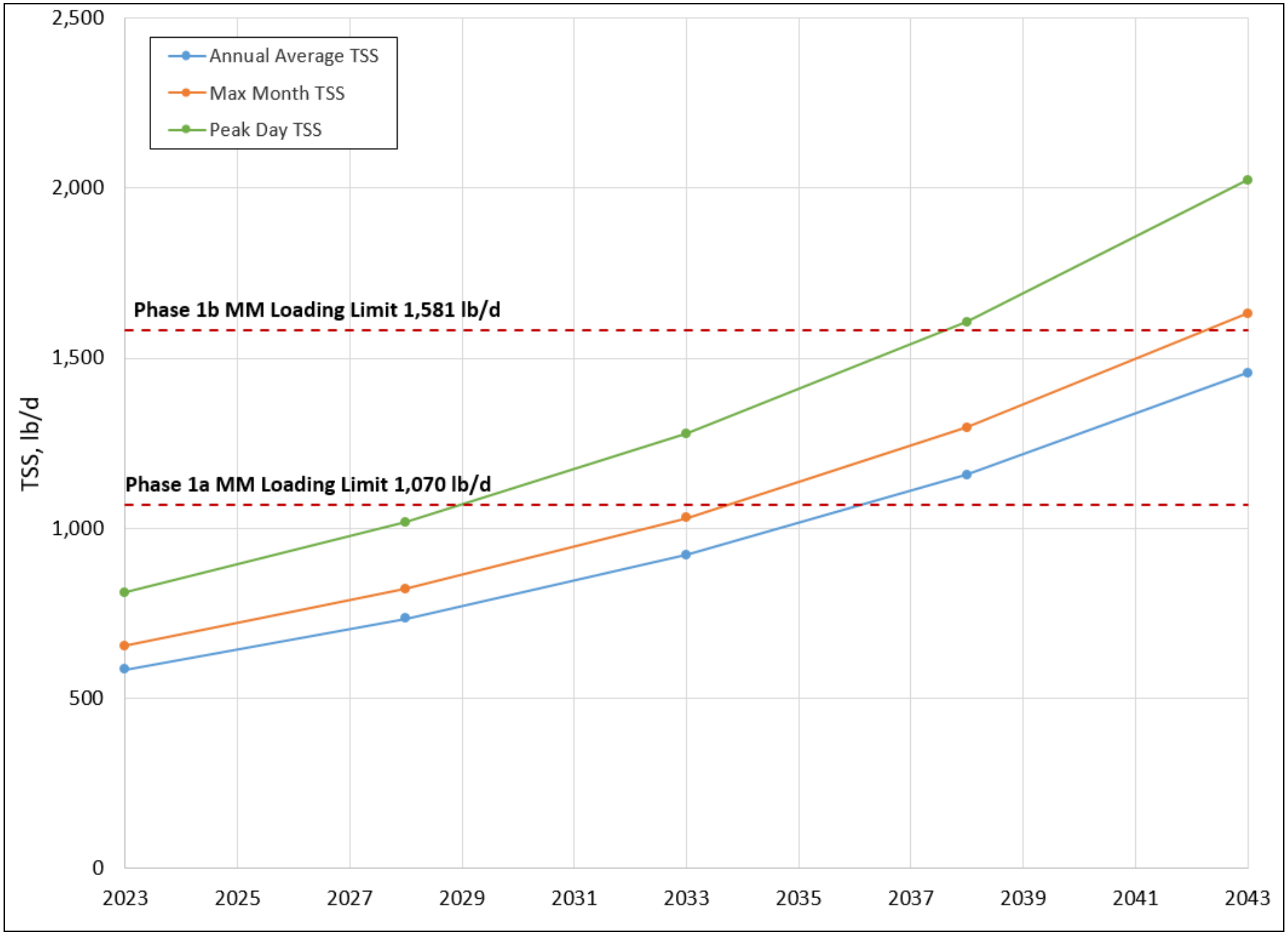


FIGURE 5-13

Projected WWTP TSS Loading

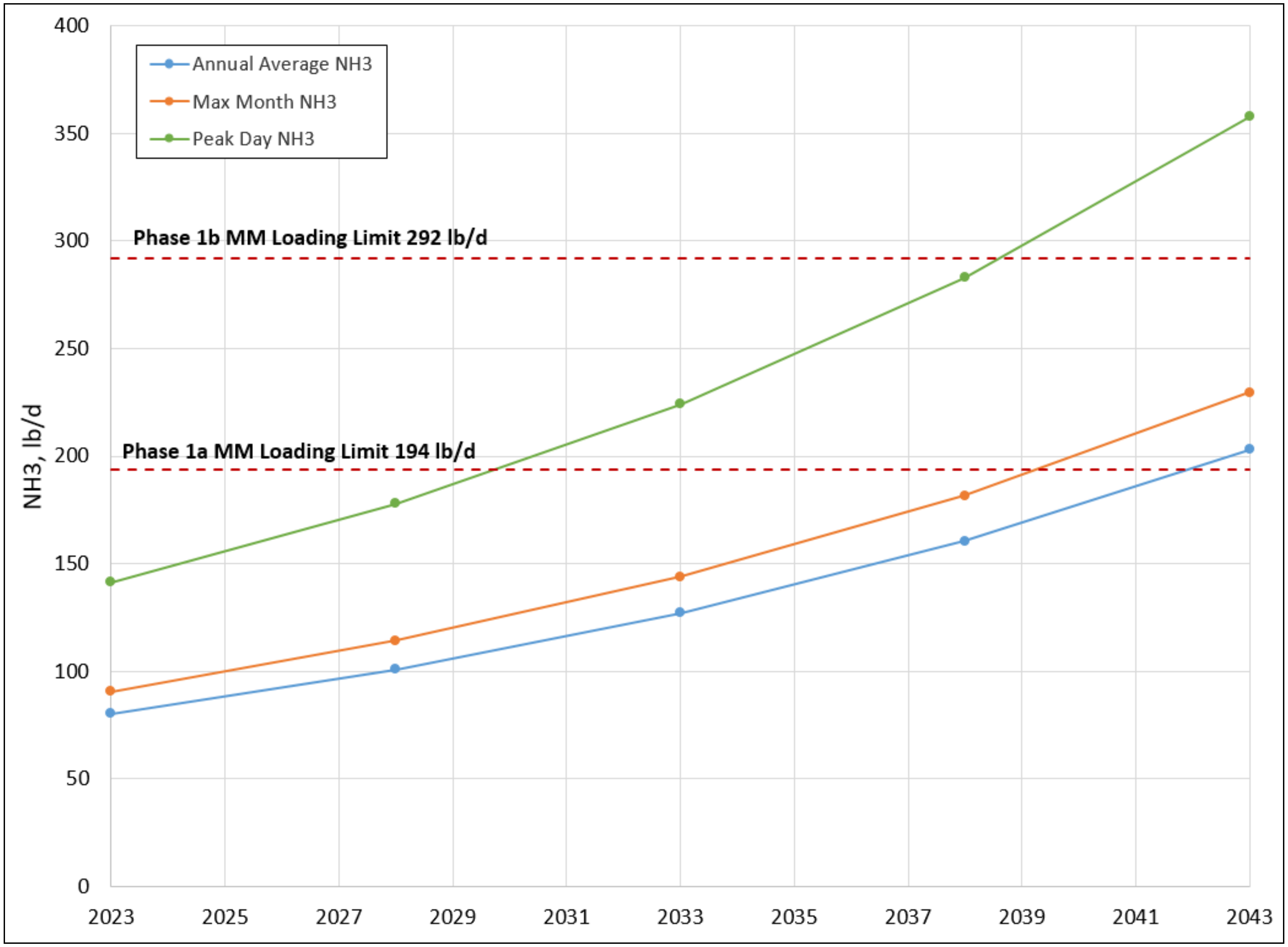


FIGURE 5-14

Projected WWTP NH3 Loading

It should be noted that influent loadings are only part of the loadings at the WWTP that need to be treated; the recycle side streams provide significant loadings back to the aeration basins. These loadings will increase in the future. The impact of these recycle streams is discussed in Chapters 7.

CONCLUSION

Based on the flow and loading analysis, the capacity limits will be reached within the years noted in Table 5-20.

TABLE 5-20

Current and Projected WWTP Influent Loadings

Limit Criteria	Current NPDES Permit Limit (Phase 1a)	Future NPDES Permit Limit (Phase 1b)	Year Reaching Phase 1a Capacity Limit	Year Reaching Phase 1b Capacity Limit
MM Flow	0.69 mgd	1.04 mgd	2033	2043
PD Flow	1.29 mgd	1.94 mgd	2035	Beyond 2043
MM BOD Loading	1,297 ppd	1,804 ppd	2030	2037
MM TSS Loading	1,070 ppd	1,581 ppd	2034	2043
MM NH3 Loading	194 ppd	292 ppd	2039	Beyond 2043

CHAPTER 6

COLLECTION SYSTEM ANALYSIS

INTRODUCTION

In this chapter, the City's wastewater collection system is evaluated for its ability to serve the future population and land use presented in Chapter 2, and the projected wastewater flow rates described in Chapter 5.

The physical condition of the existing wastewater collection system was discussed in Chapter 4, based on review of previous reports, existing City sewer base maps and data, interviews with City staff, and drawdown testing of pump stations. In this chapter, a hydraulic model is discussed that was developed to analyze the capacity of major gravity lines at peak hour wet weather flow rates projected for year 2043. The pump station and force main capacities were analyzed using the projected flow rates developed in Chapter 5 and in this chapter. The results of the capacity analysis and physical condition assessment were used to identify collection system components in need of rehabilitation or replacement.

DRAINAGE BASINS

Figure 6-1 shows the existing sewer service area including the City's existing wastewater drainage basins and the proposed drainage basins within the City's sewer service area. In addition to the existing basins served by the wastewater collection system, as development continues in areas within the City's sewer service area that are currently unsewered, future drainage basins will be formed. Existing drainage basins are shown in blue and proposed drainage basins are shown in pink.

PROJECTED ERUS AND FLOWS FOR THE PLANNING PERIOD

PROJECTED ERUS AND DRY WEATHER FLOW

As discussed in Chapter 5, during the 20-year planning period (2023-2043), an average City-wide annual growth rate of 4 percent is expected for the residential population, while the projected employment growth is 6.5 percent. Table 6-1 shows the current and projected ERUs for the various drainage areas. Current ERUs were derived from the City-provided water billing data. Allocation of the future ERUs was based discussion with City staff.

TABLE 6-1

Estimated Current and Projected Future ERUs per Basin

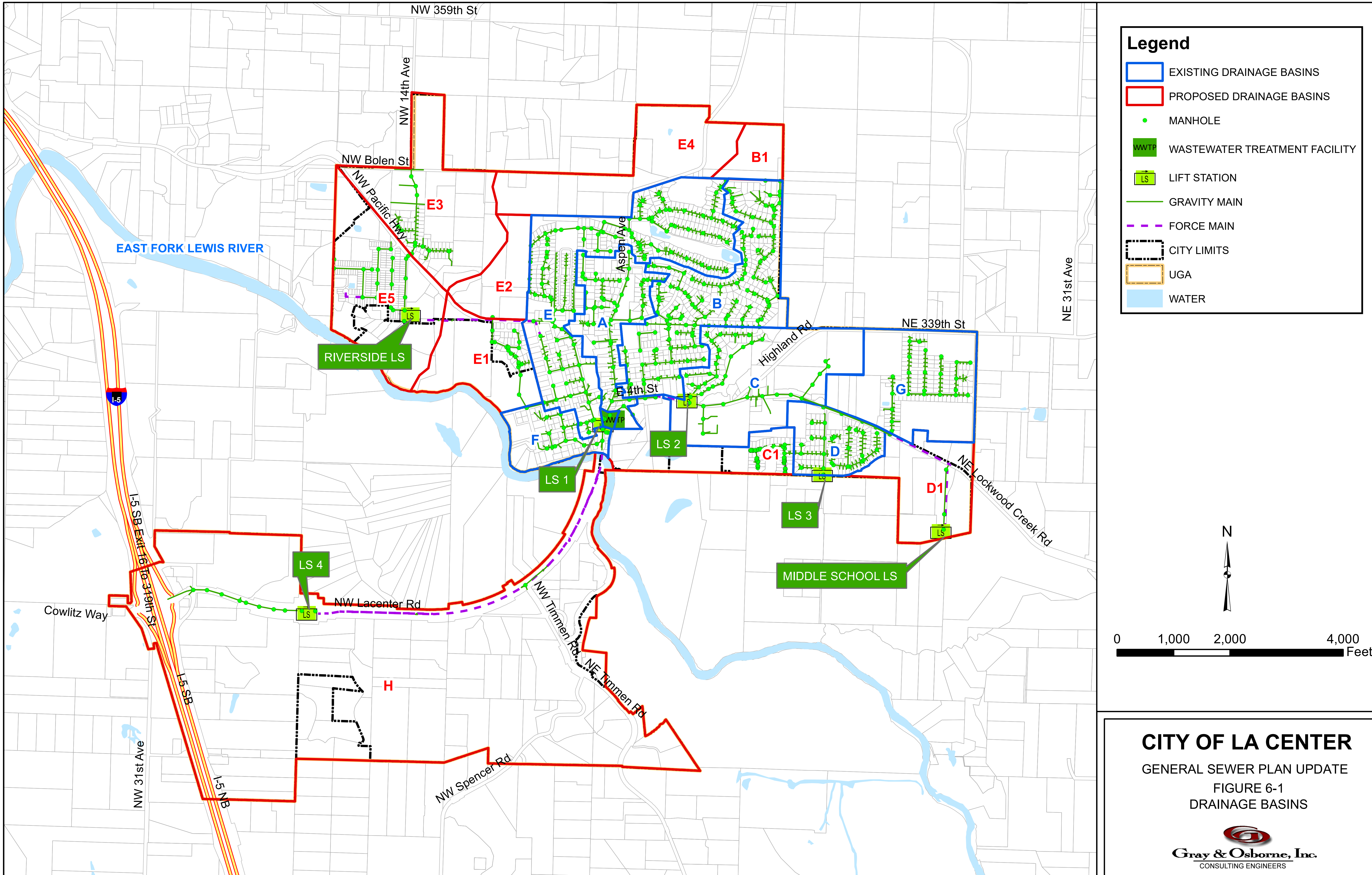
Basin	ERUs		
	2023	2033	2043
A	187	231	231
B	403	403	403
B1	1	98	98
C	153	173	173
C1	117	117	117
D	129	129	129
D1	20	179	179
E	508	508	508
E1	14	14	78
E2	3	23	23
E3	60	256	343
E4	10	182	182
E5	193	295	295
F	71	71	71
G	126	197	197
H	-	234	1,836
Total	**1,994**	**3,109**	**4,863**

Table 6-2 shows the current estimated and projected future average dry weather flows for the basins within the existing city limits using the unit ERU rate of 148 gpd/ERU developed in the previous chapter.

TABLE 6-2

Estimated Current and Projected Future Average Dry Weather Basin Flows

Basin	Average Dry Weather Flows (gpd)		
	2023	2033	2043
A	27,700	34,200	34,200
B	59,700	59,700	59,700
B1	200	14,500	14,500
C	22,600	25,600	25,600
C1	17,300	17,300	17,300
D	19,100	19,100	19,100
D1	2,900	26,500	26,500
E	75,200	75,200	75,200
E1	2,000	2,000	11,500
E2	400	3,400	3,400
E3	8,800	37,900	50,700
E4	1,500	27,000	27,000
E5	28,600	43,700	43,700
F	10,400	10,400	10,400
G	18,700	29,200	29,200
H	-	34,600	271,800
Total	**295,100**	**460,300**	**719,800**



CITY OF LA CENTER

GENERAL SEWER PLAN UPDATE

FIGURE 6-1

DRAINAGE BASINS

PROJECTED PEAK WASTEWATER FLOWS

In order to establish the required capacity of collection system facilities, such as gravity sewer lines and pump stations, peak hour rather than dry weather flow has to be considered. An equation for a default, conservative, ratio of peak hour flow to dry weather flow is provided in *Criteria for Sewage Works Design* (Ecology, 2023). This ratio, termed the "Peaking Factor" (*PF*), is calculated from the following equation:

$$PF = \frac{18 + \sqrt{P}}{4 + \sqrt{P}}$$

where *P* is the population in thousands. The population calculated by multiplying the ERUs by 2.66 cap/ERU, the estimated average size of the household. As stated in *Criteria for Sewage Works Design* (Ecology, 2023), use of the per capita flows and the peaking factor is intended to cover normal I/I for systems built with modern construction techniques. However, an additional allowance should be made for I/I with existing conditions such as high ground water, older systems, or a number of cross connections with storm drains. Thus, in this study, an additional peak hour I/I of 1,664 gpad derived from the I/I analysis in Chapter 5 was included in the current peak hour flow as follows for each of the drainage basins:

Total Wastewater Peak Hour Flow Rate =
*Average Dry Weather Flow (or Sanitary Base Flow) * Peak Factor + Peak Hour I/I Flow Rate*

I/I for the future service area is projected based on a typical I/I rate of 1,000 gpad observed in western Washington.

Table 6-3 provides the projected peak wastewater flow rate. The projected WWTP influent flow presented in Chapter 5 is lower than the sum of the projected basin flows, due to the effect of attenuation in the collection system as well as that individual basin not experiencing the peak flow condition simultaneously.

TABLE 6-3

Basin Current and Future Peak Hour Flows

Basin	2033 Population	2033 Peaking Factor	2033 Peak Hour Flow (gpd)	2033 Peak Hour I/I (gpd)	2033 Peak Hour Flow with I/I (gpd)	2043 Populat ion	2043 Peaking Factor	2043 Peak Hour Flow (gpd)	2043 Peak Hour I/I (gpd)	2043 Peak Hour Flow with I/I (gpd)
A	459	4.0	136,800	80,100	209,700	459	4.0	136,800	80,100	209,700
B	378	4.0	238,800	139,500	375,300	378	4.0	238,800	139,500	375,300
B1	261	4.1	59,500	34,000	80,100	261	4.1	59,500	34,000	80,100
C	65	4.3	110,100	59,800	166,300	65	4.3	110,100	59,800	166,300
C1	142	4.2	72,700	40,400	112,500	142	4.2	72,700	40,400	112,500
D	343	4.1	78,300	44,600	122,300	343	4.1	78,300	44,600	122,300
D1	475	4.0	106,000	61,800	145,800	475	4.0	106,000	61,800	145,800
E	1,260	3.7	278,200	175,800	451,200	1,260	3.7	278,200	175,800	451,200
E1	36	4.3	8,600	4,700	13,200	206	4.1	47,200	26,800	65,100
E2	61	4.3	14,600	7,900	19,700	61	4.3	14,600	7,900	19,700
E3	411	4.0	151,600	88,500	212,700	643	3.9	197,700	118,600	276,900
E4	485	4.0	108,000	63,100	147,300	485	4.0	108,000	63,100	147,300
E5	785	3.9	170,400	102,100	257,400	785	3.9	170,400	102,100	257,400
F	188	4.2	43,700	24,400	67,700	188	4.2	43,700	24,400	67,700
G	525	4.0	116,800	68,300	174,600	525	4.0	116,800	68,300	174,600
H	-	4.0	138,400	80,900	191,700	2,476	3.5	951,300	635,500	1,364,200
Total	**5,873**	**3.2**	**1,463,800**	**1,075,900**	**2,378,800**	**8,751**	**3.0**	**2,166,600**	**1,682,700**	**3,472,600**

PROJECTED ERUS AND FLOWS FOR BUILDOUT

The following assumptions are used for the buildout flow and ERU estimate (consistent with the 2019 Draft GSP):

- Net area was determined by deducting 30% for roads and parking. The parcels are then assessed based on the Land Use:
    - Non-Residential Land Use
        - C zone – Assumes a restaurant with 50 GPD/seat and 100 SF per seat (500 people maximum per day based on Ecology standards)
        - MX- Mixed use assumes a multifamily development with 10 people per acre (according to LCMC)
        - RP Zone – Assumes a shopping Center with 200 gpd per 1,000 sq ft as per Ecology Standards
        - C and RP zones assume 75 percent of the site area is parking.
        - ERUs were calculated by dividing the flow with 148 gpd/ERU.
    - Residential Land Use – ERUs were calculated by multiplying zoning density by the net developable area.

The areas for each land use classification for each basin and the calculated ERUs are summarized in Table 6-4. The flow based on the land use density is not presented, since it is noted that most of the current service areas north of La Center Road are already at buildout, which indicates that reclassification might be necessary to increase the capacity.

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TABLE 6-4

Basin Buildout Zoning Area and ERUs

	Agriculture-20 (AG-20)	Downtown Commercial (C-1)	Forest-40 (FR-40)	Junction Plan (JP)	Low Density Residential (LDR-7.5)	Medium Density Residential (MDR-16)	Mixed Use (MX)	Parks/ Open Space (P/OS)	Parks/ Wildlife Refuge (P/WL)	Public Facilities (PF)	Single Family Residential (R1-10)	Residential (R-12)	Single Family Residential (R1-20)	Single Family Residential (R1-6)	Single Family Residential (R1-7.5)	Rural-5 (R-5)	Residential/ Professional (RP)	Urban Public Facilities (UP)	Water	Total
Acres																				
A	0.0	10.2	0.0	0.0	31.1	0.0	0.0	3.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	14.1	0.8	0.0	59
B	0.0	5.7	0.0	0.0	106.5	0.0	0.0	5.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.7	6.5	0.0	0.0	125
B1	0.0	0.0	0.0	0.0	20.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	20
C	0.0	0.0	0.0	0.0	38.1	0.0	0.0	19.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.6	58.5	0.0	116
C1	0.0	2.9	0.0	0.0	16.8	0.0	0.0	35.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.5	0.1	58
D	0.0	0.0	0.0	0.0	30.2	0.2	0.0	5.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.6	0.0	0.0	40
D1	0.2	0.0	0.0	0.0	25.7	0.0	0.0	2.8	0.0	0.0	0.0	0.0	0.0	0.0	9.0	0.0	0.0	22.6	0.0	60
E	0.0	9.1	0.0	0.0	82.1	9.4	0.0	25.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	28.0	0.1	0.0	154
E1	0.0	0.0	0.0	0.0	16.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.9	42.6	0.0	0.0	0.0	0.0	61
E2	0.0	0.0	0.0	0.0	35.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	35
E3	0.0	0.0	0.0	0.0	76.6	12.3	0.0	0.0	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.0	17.4	0.0	107
E4	0.0	0.0	0.0	0.0	92.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.0	0.0	92
E5	0.0	0.0	0.0	0.0	10.1	54.3	0.0	0.0	0.0	0.0	0.0	12.2	0.0	27.9	1.1	0.2	0.0	0.0	0.3	106
F	0.0	0.0	0.0	0.0	12.0	0.0	0.0	2.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	17.6	1.2	0.0	33
G	0.0	0.0	0.0	0.0	87.4	5.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.0	0.2	0.0	96
H	0.0	0.0	0.0	260.6	312.8	0.0	58.4	9.0	0.0	0.4	8.1	0.0	32.5	0.0	0.0	0.0	0.0	0.0	0.9	683
Total																				**1,845**
ERUs																				
A	0	37	0	0	87	0	0	0	0	0	0	0	0	0	0	0	59	1	0	185
B	0	21	0	0	298	0	0	0	0	0	0	0	0	0	0	2	28	0	0	349
B1	0	0	0	0	57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	57
C	0	0	0	0	107	0	0	0	0	0	0	0	0	0	0	0	3	55	0	165
C1	0	11	0	0	47	0	0	0	0	0	0	0	0	0	0	0	0	2	0	60
D	0	0	0	0	85	3	0	0	0	0	0	0	0	0	0	0	15	0	0	102
D1	0	0	0	0	72	0	0	0	0	0	0	0	0	0	47	0	0	21	0	141
E	0	33	0	0	230	105	0	0	0	0	0	0	0	0	0	0	118	0	0	487
E1	0	0	0	0	45	0	0	0	0	0	0	0	0	8	224	0	0	0	0	277
E2	0	0	0	0	99	0	0	0	0	0	0	0	0	0	0	0	0	0	0	99
E3	0	0	0	0	215	138	0	0	0	0	0	0	0	0	0	1	0	16	0	370
E4	0	0	0	0	259	0	0	0	0	0	0	0	0	0	0	0	0	0	0	259
E5	0	0	0	0	28	608	0	0	0	0	0	102	0	117	6	1	0	0	0	863
F	0	0	0	0	34	0	0	0	0	0	0	0	0	0	0	0	74	1	0	109
G	0	0	0	0	245	57	0	0	0	0	0	0	0	0	0	0	13	0	0	315
H	0	0	0	912	876	0	409	0	0	2	57	0	455	0	0	0	0	0	0	2,710
Total																				**6,548**

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LIFT STATION AND FORCE MAIN CAPACITY ANALYSIS

The capacity evaluation of the pump stations was conducted by comparing the existing capacities to the 2033 and 2043 projected peak hour wastewater flows. The existing pump station capacities are summarized in Chapter 4 – Existing Facilities, and the projected wastewater flows for each pump station are summarized in this chapter. The capacity of each force main is based on a maximum design velocity of 8 feet per second (fps). Table 6-5 presents the results of the pump station and force main capacity evaluation.

TABLE 6-5

Lift Station and Force Main Capacity Evaluation

Pump Station	Contributing Basins	2033 Peak Hour Flow (gpm)	2043 Peak Hour Flow (gpm)	Existing PS Capacity (gpm)	Force Main Diameter (inches)	Existing Force Main Capacity (gpm)(1)
1-Treatment Plant	E, E1-E5, F, H	**945**	**1,840**	585(2)	12	2,074
2-Stone Creek	Basin C, C1, D, D1, G	**501**	**501**	130(2)	6	561
3-Johnstorm	Basin D	85	85	450	4	258
4-La Center Road	Basin H	133	**947**	230	6&8	949 (current) 1,510 (future)
5-Middle School	Basin D1	101	101	265	6	561
6-Riverside	Basin E5	**179**	**179**	156(2)	8	949

(1) Based on pipeline velocity of 8 fps.
(2) Based on drawdown test
(3) **BOLD** values exceed pump station capacity.

HYDRAULIC MODEL

A hydraulic model of the City's wastewater collection system is presented in this section, including a description of model development and the assumptions used in the model. This model has two main functions: (1) to provide information to develop recommended improvements to convey the projected flow rates; and (2) to evaluate the system with the recommended capital improvements to verify capacity. The model can be updated and maintained for use as a tool to aid in future planning and design.

MODEL DEVELOPMENT

Physical Model

The trunk sewer lines of the City's sewer system were modeled using an Excel spreadsheet. Figure 6-2 shows the sewer lines that were included in the hydraulic model.

The hydraulic model was developed based on information provided in the City's 2019 Draft GSP. The accuracy of the hydraulic model results depends on the accuracy of the data input to the model. In some cases, reliable invert elevations of manholes were not known, and invert elevations were linearly interpolated between known invert elevations upstream and downstream. Data used in the hydraulic model is shown in Table 6-6.

TABLE 6-6

Sewer System Information from City Data

Category	Gravity Sewers	Manholes
Number	Pipe ID number based on City's numbering convention	Manhole number based on City's numbering convention
Dimension	Length from City GSP.	Not applicable
Elevation	Upstream and downstream pipe invert elevations from City 2019 Draft GSP	Upstream and downstream pipe invert elevations from City 2019 Draft GSP
Size	Pipe diameters from City 2019 Draft GSP	Assumed 48-inch manhole size
Flow Criteria	Assumed Manning's roughness coefficient of 0.013 which corresponds to average concrete pipe	Not applicable

Wastewater Flow Model

The hydraulic model was used to simulate peak hour flow rates for the projected 2043 flow condition. Sanitary sewer flow projections determined previously in this chapter were applied for each of the sewer drainage basins.

For the loading of the hydraulic model, the projected flows in each drainage basin were distributed evenly to the modeled manhole in the corresponding basin. For proposed drainage basins, flows were input at the anticipated gravity drainage discharge or force main discharge manhole. Only the existing sewer system was modeled, and it was assumed that when future drainage basins are connected to the system, new pipes would be sized to receive projected flows. Table 6-7 summarize the loading to each basin. Appendix E provides the distribution and flow rates input into the sewer model.

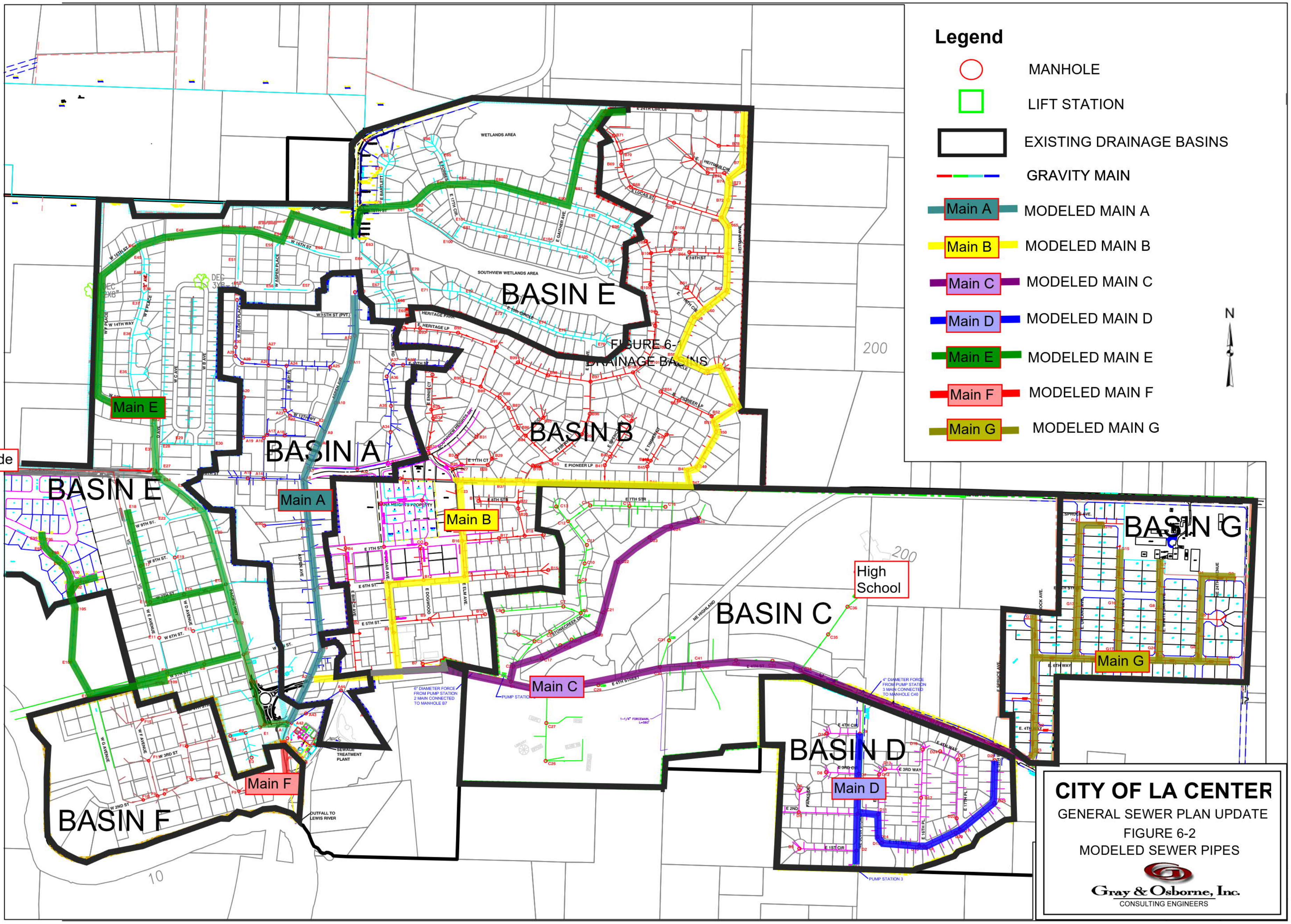

Legend
MANHOLE
LIFT STATION
EXISTING DRAINAGE BASINS
GRAVITY MAIN
Main A MODELED MAIN A
Main B MODELED MAIN B
Main C MODELED MAIN C
Main D MODELED MAIN D
Main E MODELED MAIN E
Main F MODELED MAIN F
Main G MODELED MAIN G
N
BASIN A
BASIN B
BASIN C
BASIN D
BASIN E
BASIN E
BASIN F
BASIN G
Main A
Main B
Main C
Main D
Main E
Main F
Main G
Riverside
High School
WETLANDS AREA
SOUTHVIEW WETLANDS AREA
SEWAGE TREATMENT PLANT
OUTFALL TO LEWIS RIVER
PUMP STATION 3
200
200
10
CITY OF LA CENTER
GENERAL SEWER PLAN UPDATE
FIGURE 6-2
MODELED SEWER PIPES
Gray & Osborne, Inc.
CONSULTING ENGINEERS

TABLE 6-7

Modeled Trunk Sewer Contributing ERUs and Projected Future Flows

Trunk Sewer	Contributing Basins	2033 Contributing ERUs	2033 Peaking Factor	2033 Peak Flow Hour (gpm)	2033 Peak Hour I/I (gpm)	2033 Peak Hour Flow with I/I (gpm)	2043 Contributing ERUs	2043 Peaking Factor	2043 Peak Hour Flow (gpm)	2043 Peak Hour I/I (gpm)	2043 Peak Hour Flow with I/I (gpm)
Main A	A	231	4.0	95	51	146	231	4.0	95	51	146
Main B	B, B1	501	3.9	201	109	310	501	3.9	201	109	310
Main C	C (except High School), C1	141	4.1	60	31	91	141	4.1	60	31	91
Main D	D, D1	308	3.9	123	58	182	308	3.9	123	58	182
Main E	E, E1-E4	982	3.5	353	197	550	1,133	3.5	408	218	626
Main F	F	71	4.2	30	17	47	71	4.2	30	17	47
Main G	G	197	4.0	81	40	121	197	4.0	81	40	121
From Riverside	E5	295	3.9	118	60	179	295	3.9	118	60	179
From High School	High School	148	4.0	61	36	97	148	4.0	61	36	97

Model Evaluation Criteria

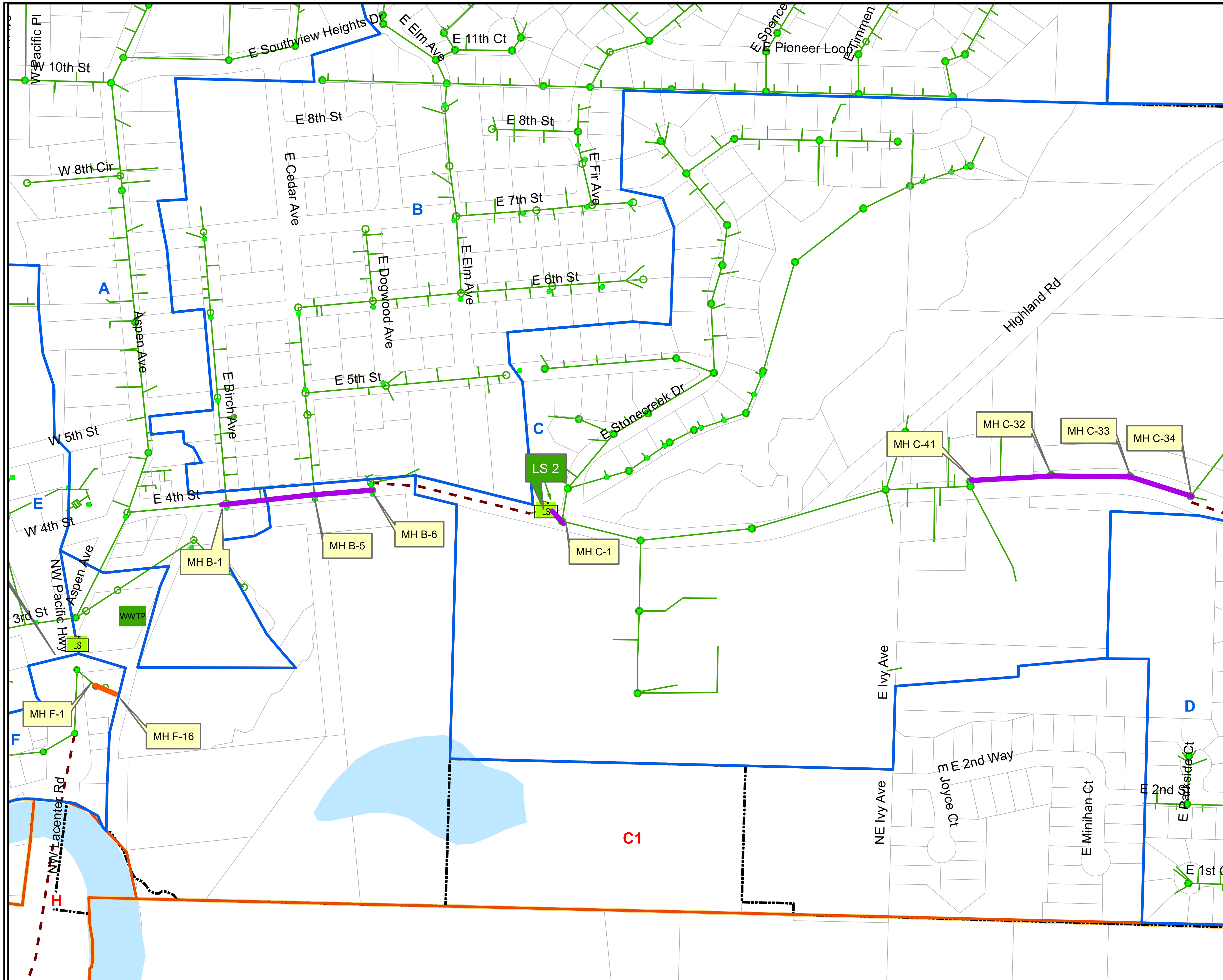
The model runs were used to identify sewers that may be hydraulically deficient during a peak hour flow event. The criterion for listing a gravity sewer pipe as "deficient" is that at peak hour flow, the flow exceeds the capacity of the pipe. The capacity of the pipe is calculated using Manning's equation assuming that the pipe is flowing full. The slope of the pipe is calculated using pipe length and the difference between the pipe invert elevations as recorded in the City's GSP. Pipes with projected flows that marginally exceed their capacity may result in an acceptable surcharge, i.e., a surcharge level in the upstream manhole that does not flood.

RESULTS OF HYDRAULIC MODELING ANALYSIS

The model was run with the projected year 2033 and 2043 flows, and the capacities of the existing sewer pipes were compared to the estimated peak hour flow rates.

The results of the hydraulic model indicated one pipeline capacity deficiency. This deficiency can be attributed to flat or minimally sloped pipes. Since the projected exceedance is minor, the City could wait and monitor growth and flows before deciding if the pipe needs to be upsized.

Table 6-8 provides information on the existing system component that may have insufficient capacity under 2033 and 2043 conditions; Figure 6-3 shows the locations of the deficient pipes.



Legend

- WASTEWATER TREATMENT FACILITY
- LIFT STATION
- EXISTING DRAINAGE BASINS
- PROPOSED DRAINAGE BASINS
- CITY LIMITS
- UGA
- WATER
- MANHOLE
- GRAVITY MAIN
- FORCE MAIN
- OUT OF CAPACITY BY 2033
- OUT OF CAPACITY 2043+

N

0 200 400 800 Feet

CITY OF LA CENTER

GENERAL SEWER PLAN UPDATE

FIGURE 6-3

SEWER MODEL RESULTS

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CONSULTING ENGINEERS

TABLE 6-8

Hydraulic Model Results – Deficiencies at Projected Year 2033 and 2043 Conditions

Pipe Label	Upstream Node	Downstream Node	Existing Pipe Size (inch)	Estimated Capacity (gpm)	Capacity Exceedance at 2033 (gpm)	Pipe Size Required to Accept 2033 Flow (inch)	Capacity Exceedance at 2043 (gpm)	Pipe Size Required to Accept 2043 Flow (inch)
56	B-5	B-1	8	486	306	10	306	10
-	C-34	C-33	8	384	36	10	36	10
la-87	C-33	C-32	8	344	80	10	80	10
la-89	C-32	C-41	8	344	83	10	83	10
-	C-1	LS #2	8	344	139	10	139	10
la-57	B-6	B-5	8	486	4	10	4	10

(1) Information about pipe label, pipe size and slope was obtained from the 2019 GSP.

It was found the pipe section right upstream of the WWTP between MH F-1 and MH F-16 would be out of capacity between 2043 and buildout. Thus, the upsize is recommended to be completed around 2043.

LA CENTER ROAD BASIN COLLECTION SYSTEM

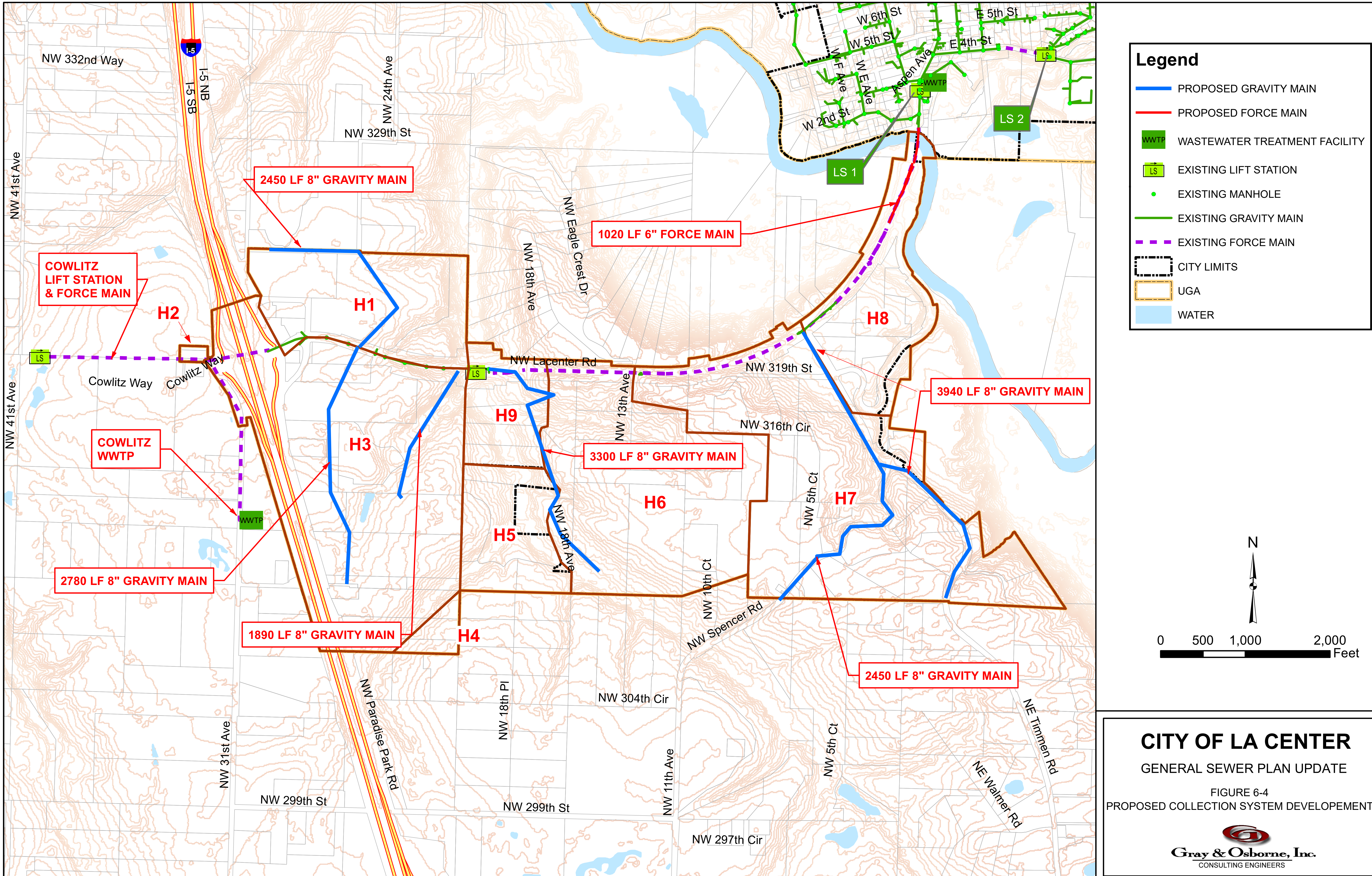
The La Center Road drainage basin (Basin H) extends southwesterly from downtown along La Center Road to the La Center Road interchange with Interstate 5 (I-5). Flows from the basin will be collected in the La Center Road lift station and trunk sewer and conveyed to Manhole F-3 to the west of the WWTP. Basin H is unsewered except for the La Center Road lift station and trunk sewer. It is expected that a significant portion of wastewater from this basin will be collected and conveyed to the WWTP in future. For the following analysis for future collections system improvements, Basin H is further divided into nine subbasins as presented in Figure 6-4.

Cowlitz Tribal Property: The Cowlitz Tribal Properties are served by their own wastewater collection and treatment system. There is a treatment facility located on the east side of NW 31st Avenue approximately 1,800 feet south of Cowlitz Way. A pump station is located near the Cowlitz Way/NW 41st Avenue intersection, approximately a half mile west of I-5, with an 8-inch force main that is connected to the treatment facility. A separate 8-inch force main connects to the Cowlitz Tribe's force main at the Cowlitz Way/NW 31st Avenue roundabout, extends east across the I-5 interchange and connects with the La Center Road trunk sewer at NW Paradise Park Road. This force main is not currently in use and would allow redirecting the flows from the Cowlitz Tribe's pump station to the City of La Center's sewer system in the future. Flow into the separate force main is controlled by valves located in the southwest quadrant of the Cowlitz Way/NW 31st Avenue roundabout.

Basin H1 is located at the northeast quadrant of the I-5 interchange and extends north and east to the city limits. It is assumed that sewers will be constructed as the basin is developed. Future improvements in Basin H1 include 2,450 feet of 8-inch sewer main to be constructed in a future street that will convey wastewater along the north boundary of the basin to the center portion of the basin and to the south where it will connect to the La Center Road trunk sewer.

Basin H2 is located on the west side of I-5 in the northwest quadrant of the freeway interchange. It extends westward and northward to the city limits which is also the boundary of the Cowlitz Tribal properties. The basin area is approximately 1.25 acres and will be served by an 8-inch sewer line that was constructed in Cowlitz Way that flows west to the Cowlitz Tribe's pump station located on the south side of Cowlitz Way. Flows from Basin H2 will be conveyed to the Cowlitz Tribe's sewer system.

Basin H3 is located in the southeast quadrant of the I-5 interchange. It extends south to the city limits and east to the upper slope of McCormick Creek. Sewers will be constructed as the basin is developed and will connect to the La Center Road trunk sewer.



CITY OF LA CENTER

GENERAL SEWER PLAN UPDATE

FIGURE 6-4
PROPOSED COLLECTION SYSTEM DEVELOPEMENT

Gray & Osborne, Inc.
CONSULTING ENGINEERS

Future improvements in Basin H3 include 2,780 feet of 8-inch sewer main in a future street that will extend south from La Center Road in the western portion of the basin, and 1,890 feet of 8-inch sewer main in the eastern portion of the basin.

Basin H4 is located southeast of Basin H3 and is isolated from H3 by topography. It is bounded to the east and south by the city limits. The surrounding property is zoned rural residential, and this small basin will also likely be developed as rural residential. Wastewater service in this area is assumed to be provided by septic systems.

Basin H5 is located along McCormick Creek and is bounded by steep topography on all sides. All lots within the basin have been developed as large lot residential and are served by septic systems. No sewer is needed to serve this area.

Basin H6 is located south of La Center Road and is bounded by McCormick Creek to the west, city limits to the south and Basin H7 to the east. Sewers will be constructed as the basin is developed and will connect to the La Center Road lift station. Future improvements in Basin H6 include 3,300 feet of 8-inch sewer main in a road along the upper edge of the McCormick Creek topography.

Basin H7 is located on the south side of La Center Road between Basin H6 and Timmen Road. It is bordered on the south by city limits. At the southeast corner of this basin there is a small number of large residential lots that lie on the east side of Timmen Road that are served by septic systems. Future improvements include 3,940 feet of 8-inch sewer main in Timmen Road and 2,450 feet of 8-inch sewer main in Spencer Road.

Basin H8 is located southeasterly of La Center Road and is bounded by Timmen Road to the west, Pollock Road to the east and the East Fork Lewis River Greenway to the south and east. Sewers will be constructed as the basin is developed and will connect to the La Center Road trunk sewer.

Basin H9 is located on the south side of La Center Road along McCormick Creek, near the La Center Road lift station. This area is undevelopable due to environmental constraints and does not need to be sewered.

The proposed pipe sizes were determined based on the topography along the pipe route and the projected buildout flows. The capacity of the existing pipes along the La Center Road were evaluated and determined to have sufficient capacity for the buildout flows with the additional 6-inch forcemain between NW Pollock Road and West 1st Street and upsizing of the 8-inch gravity main between West 2nd Street and West 3rd Street.

RECOMMENDED PROJECTS

The following is a list of projects that are recommended for implementation during the planning period based on the evaluation in this chapter:

1. High Priority Projects to be Implemented over the Next 6 Years

   - LS 1 and LS 2 Upgrade: Increase capacity of pumps and force main, also upgrade the high-level alarm, refurbish the wet well, add backup generator, and other improvements.

   - Installation of flow meters at existing Lift Stations (6).

   - Jetter for collection system cleaning.

2. Projects to be Implemented over the Next 10 Years

   - Replace Pipe Sections between MH B-6 and MH B-1, MH C-34 and MH C-41, MH C-1 and LS#2, with a higher capacity (10-inch) 1,300 lf sewer.

   - LS 6 Upgrade

   - Develop collection system south of the La Center Road and connect to the existing collection system. The proposed improvements including gravity main, forcemain and lift station are presented in Figure 6-4.

3. Projects to be Implemented over the Next 20 Years

   - LS 4 and Downstream Force main Upgrade of additional 6-inch force main between NW Pollock Road and West 1st Street.

   - Replace Pipe Section between MH F-1 and MH F-16 with a higher capacity (12-inch) 40 lf sewer.

ESTIMATED COST

The estimated project costs for the recommended collection system projects in the next 20 years are summarized in Table 6-9.

TABLE 6-9

Project Cost of Recommended Upgrades to Collection System – 2024-2043

Pipe Section No.	Basin	Pipe Route	Type	Pipe Size (in)	Pipe Length (ft)	Estimated Project Cost	Constructed By Year
Piping Upgrades							
General			Collection System Cleaning	-	-	$50,000	2026
1	Basin A and C1	MH B-6 to MH B-1 on East 4th Street, between East Edgewood Avenue and East Birth Avenue	Existing Sewer Main (8-inch) Upsize	10	520	$521,000	2033
2	Basin C	MH C-1 to LS 2 near intersection of East 4th Street and East Stonecreek Drive	Existing Sewer Main (8-inch) Upsize	10	60	$97,000	2033
3	Basin C	MH C-34 to MH C-41 on East 4th Street, west of NE Highland Avenue	Existing Sewer Main (8-inch) Upsize	10	700	$662,000	2033
4	Basin H1	Near northeast quadrant of the I-5 interchange, between NW La Center Road and northern city limits, on future street	New Sewer Main	8	2,450	$1,979,000	2033
5	Basin H3	Near southeast quadrant of the I-5 interchange, between NW La Center Road and southern city limits, on future street	New Sewer Main	8	2,780	$2,233,000	2033
6	Basin H3	Near southeast quadrant of the I-5 interchange, extend from NW La Center Road to the south of the city limits	New Sewer Main	8	1,890	$1,584,000	2033
7	Basin H6 and H9	Along McCormick Creek, between NW La Center Rd and southern city limits	New Sewer Main	8	3,300	$2,593,000	2033
8	Basin H7	Along Spencer Road, between NW Timmen Road and southern city limits	New Sewer Main	8	2,450	$1,979,000	2033
9	Basin H7	Along NW Timmen Road, between NW La Center Rd and southern city limits	New Sewer Main	8	3,940	$3,038,000	2033
10	Basin H1-H9	LS 4 FM along NW La Center Road, between NW Pollock Road and West 1st Street La Center	New Force Main	6	1,020	$935,000	2043

TABLE 6-9 – (continued)

Project Cost of Recommended Upgrades to Collection System – 2024-2043

Pipe Section No.	Basin	Pipe Route	Type	Pipe Size (in)	Pipe Length (ft)	Estimated Project Cost	Constructed By Year
11	Basin H1-H9	Upstream of WWTP between West 2nd Street and West 3rd Street	Existing Sewer Main (8-inch) Upsize	12	40	$90,000	2043
Piping Upgrades Total			**$15,761,000**				

Lift Station	Type of Upgrade	LS Capacity (gpm)	Estimated Project Cost	Constructed By Year
Lift Station Upgrades				
General	Flow Meter Installation at Lift Stations (6)	-	$535,000	2026
LS 1	Lift Station Upgrade	2,100	$2,301,000	2028
LS 2	Lift Station Upgrade	550	$1,287,000	2028
LS 6	Lift Station Upgrade	200	$1,075,000	2033
LS 4	Lift Station Upgrade	1,100	$1,544,000	2043
Lift Station Upgrades Total	**$6,207,000**			
Collection System Upgrades Total	**$21,968,000**			

CHAPTER 7

WASTEWATER TREATMENT PLANT EVALUATION

INTRODUCTION

The purpose of this chapter is to evaluate the City of La Center Wastewater Treatment Plant (WWTP) for its ability to meet its treatment objectives at the projected future flow and loading rates, and to identify improvements necessary to meet treatment objectives. The projected flow and loading rates for the 20-year planning period (2023 to 2043) were established in Chapter 5. The treatment plant effluent quality must meet the projected limits in the National Pollution Discharge Elimination System (NPDES) permit. Recommended modifications to increase the operational efficiency or performance of the WWTP and to replace aging infrastructure are also provided.

PROJECTED FLOW AND LOADING RATES

Table 7-1 presents a comparison of the NPDES-permitted capacity for influent flow and loading with the projected flow and loading rates that were developed in Chapter 5. Based on these projections, the permitted parameters for Phase 1B (still to be constructed) will be exceeded as early as 2038.

The City's NPDES permit (Appendix A) mandates that when the monthly average flow or loading reaches 85 percent of the capacity listed in the permit for 3 consecutive months or it is projected that the facility would reach design capacity within 5 years, the City must submit a plan to maintain adequate capacity (PMAC) to the Washington State Department of Ecology (Ecology). These requirements are typical in NPDES permits because sufficient time is needed to plan, design, and construct additional capacity. The maximum month influent flow, peak hour influent flow, BOD_5, TSS or NH3 have not exceeded 85 percent of the NPDES-permitted design criteria during the past 3 years.

TABLE 7-1

Comparison of NPDES-Permitted Capacity to Current and Projected Flow and Loading Rates

	NPDES Permit Capacity Phase 1A	NPDES Permit Capacity Phase 1B	Existing (2023)	Projections			
				2028	2033	2038	2043
Average Annual Flow (mgd)	-	-	0.34	0.45	0.56	0.70	0.87
Maximum Month Flow (mgd)	0.69 mgd	1.04 mgd	0.46	0.58	0.71	0.87	**1.07**
Peak Hour Flow (mgd)	1.29 mgd	1.94 mgd	1.05	1.33	1.64	**2.03**	**2.53**
Annual Average BOD_5 Loading (lb/d)	-	-	777	973	1,220	1,530	1,923
Maximum Month BOD_5 Loading (lb/d)	1,297 ppd	1,804 ppd	952	1,192	1,494	**1,874**	**2,355**
Annual Average TSS Loading (lb/d)	-	-	588	738	927	1,165	1,467
Maximum Month TSS Loading (lb/d)	1,070 ppd	1,581 ppd	658	826	1,038	1,304	**1,643**
Annual Average NH_3-N Loading (lb/d)	-	-	81	102	128	162	205
Maximum Month NH_3-N Loading (lb/d)	194 ppd	292 ppd	91	115	145	183	231

(1) Condition S4.A of City's NPDES permit (see Appendix A).
(2) Values in **BOLD** exceed Phase 1 B criteria.

OVERALL PLANT PERFORMANCE

The Discharge Monitoring Reports (DMRs) for the last 5 years have been reviewed to evaluate compliance with the permit.

EFFLUENT BOD AND TSS

As shown in Figure 7-1 through 7-4, loadings and concentrations of effluent BOD and TSS have been compliant with and well under the permit limits over the 5 years of record.

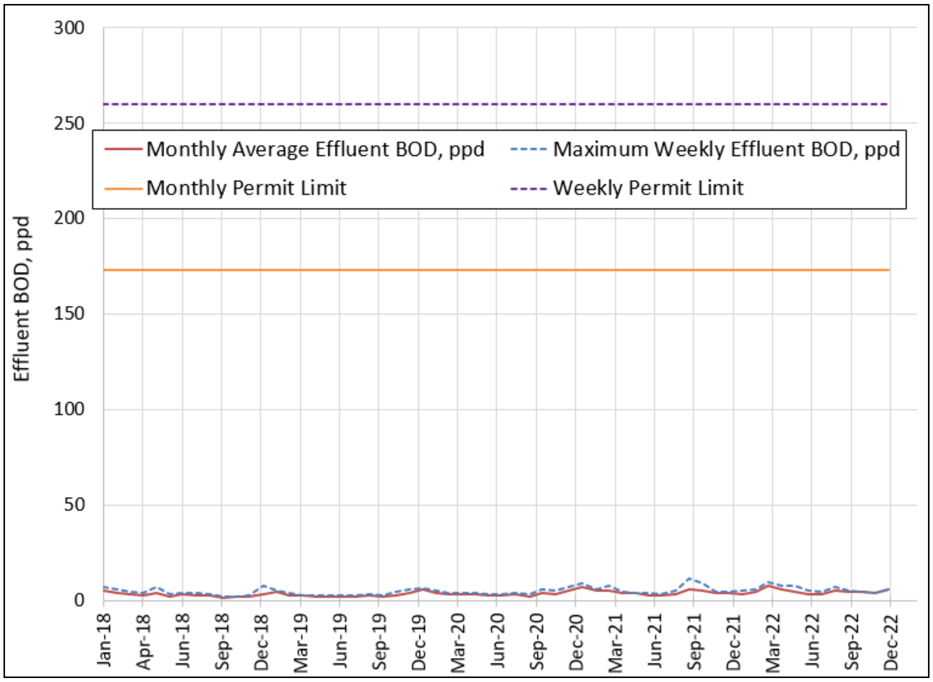


FIGURE 7-1

Effluent BOD Loadings

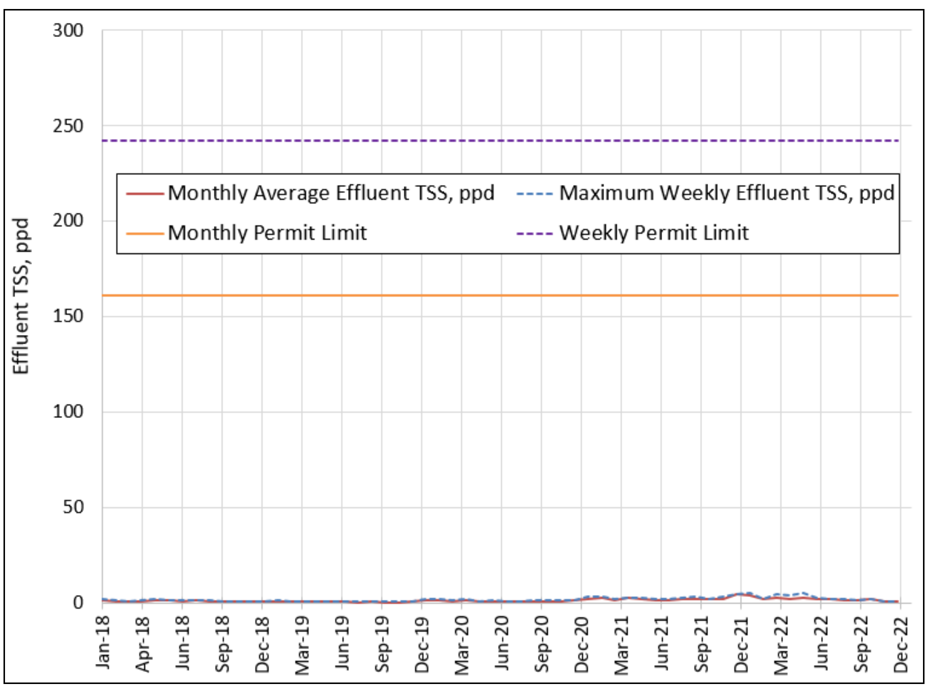


FIGURE 7-2

Effluent TSS Loadings

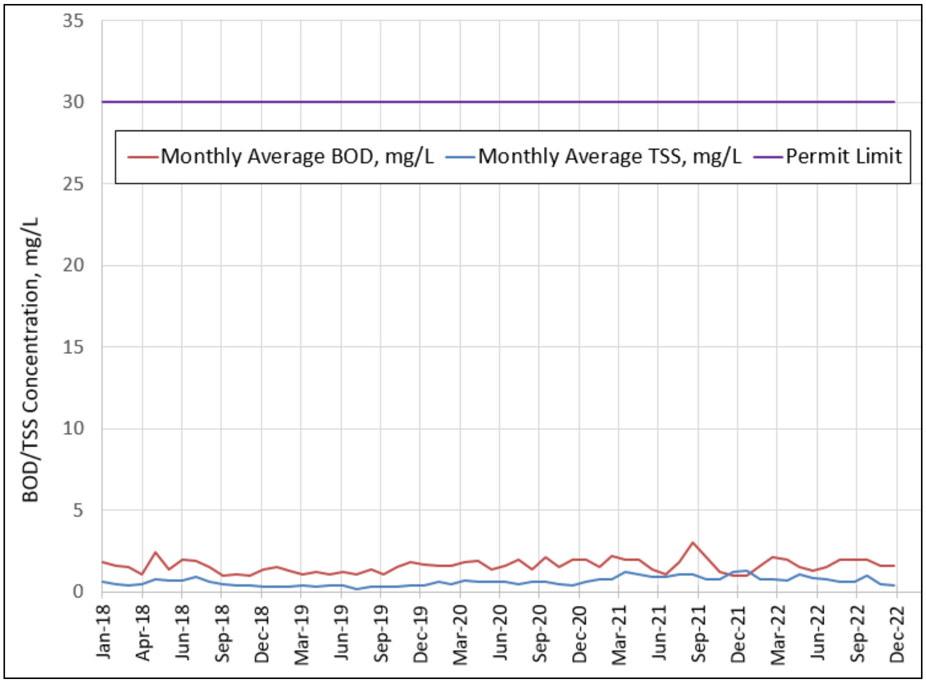


FIGURE 7-3

Monthly Average Effluent Concentrations

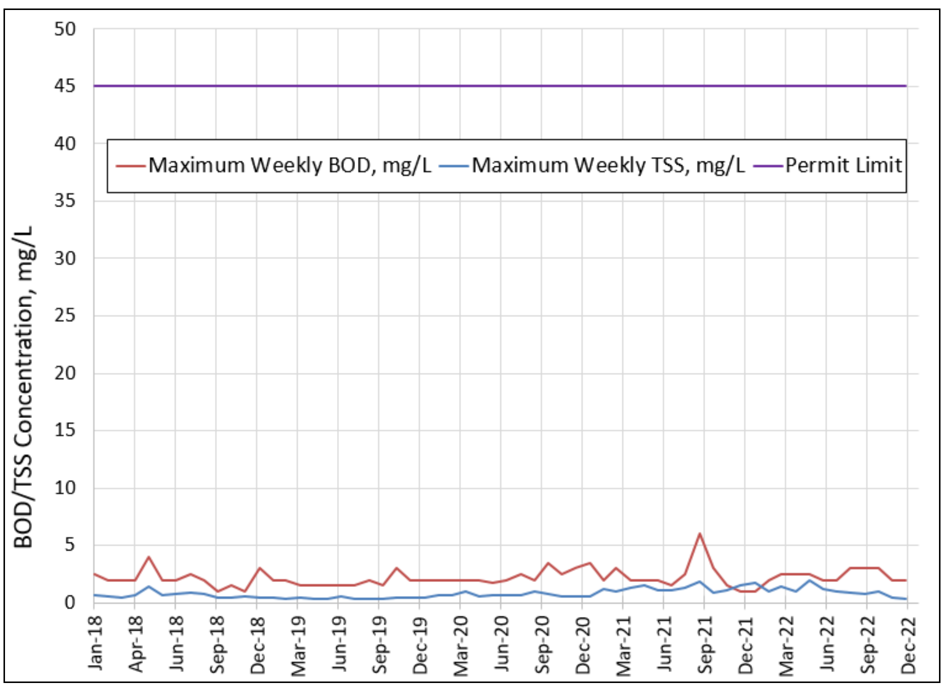


FIGURE 7-4

Monthly Average Effluent Concentrations

These effluent records indicate excellent performance by the activated sludge system, including good solids removal by the membrane bioreactors.

Figure 7-5 shows that the plant has been in compliance with the monthly average percent removal throughout the period of record for both BOD and TSS.

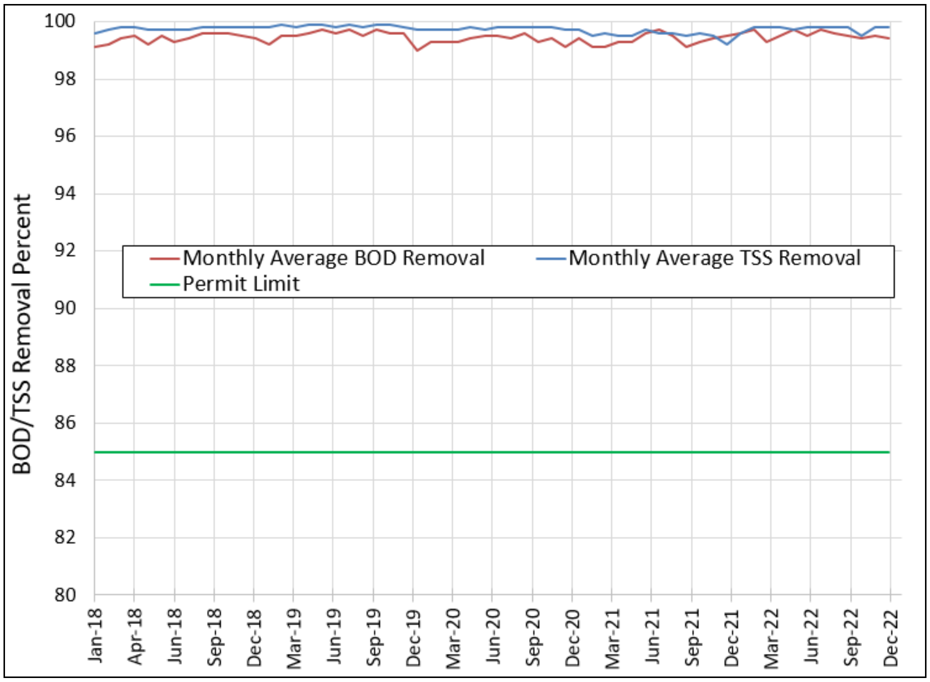


FIGURE 7-5

Monthly Average Removal Percentages, BOD and TSS

EFFLUENT TOTAL AMMONIA

Figures 7-6 and 7-7 present effluent ammonia nitrogen concentrations for the period from 2018 to 2020. Ammonia removal is accomplished by the biologically mediated process of nitrification in the aeration basins, which yields nitrate. Nitrate is then removed by the process of denitrification, which is primarily accomplished in the anoxic (low oxygen) basins after recycling the mixed liquor back from the aeration basins.

Over the 5 years of record, ammonia removal has been excellent and since March 2018, the plant has consistently produced a final effluent with ≤0.2 mg/L ammonia nitrogen, which is significantly lower than the average monthly and maximum daily permit limits for both dry season (June-October) and wet season (November-May).

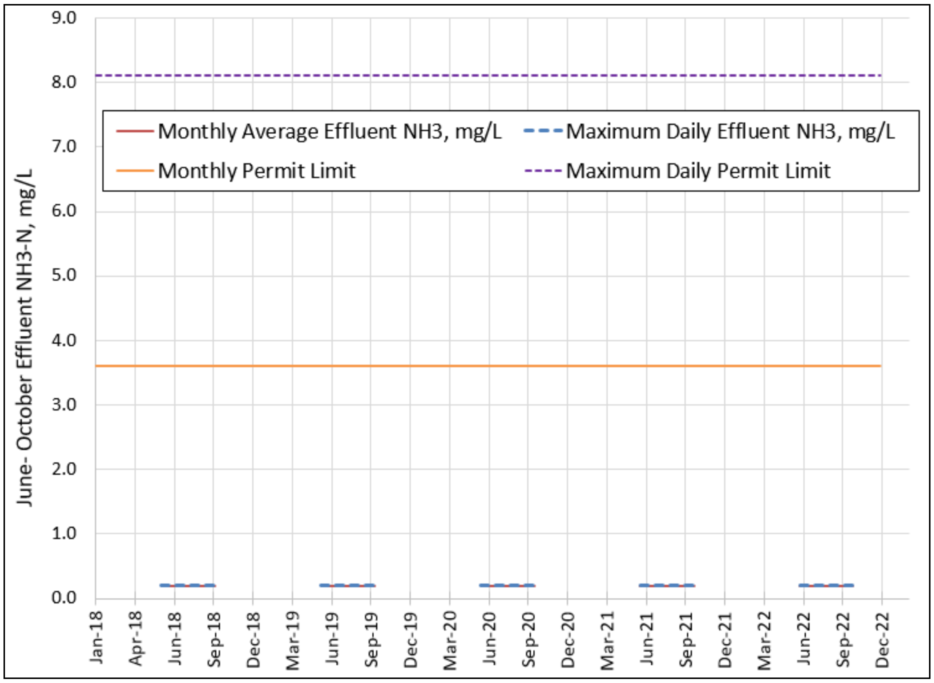


FIGURE 7-6

Effluent Ammonia (NH3-N) Concentrations, June-October

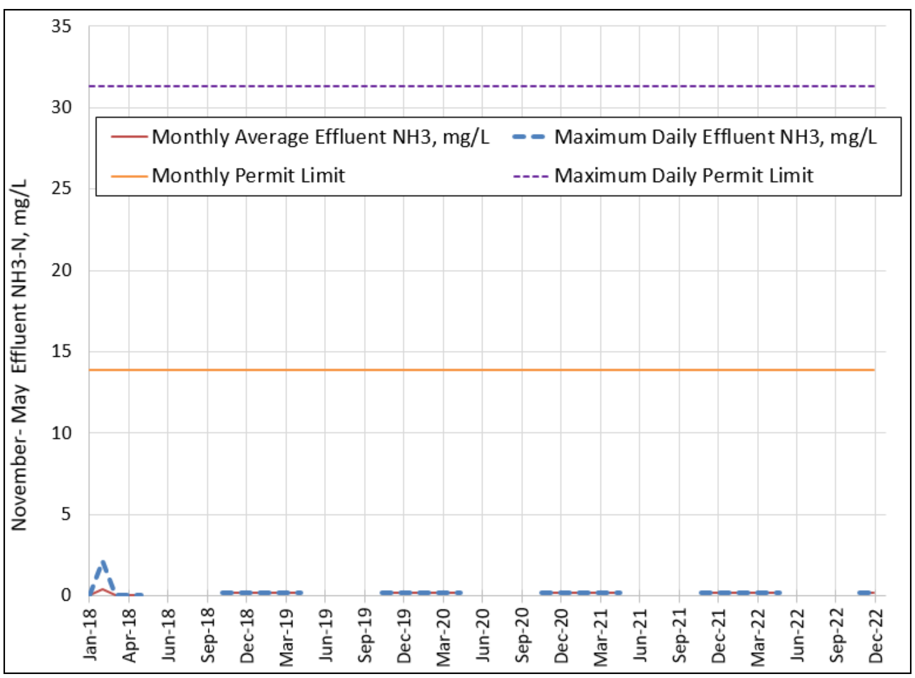


FIGURE 7-7

Effluent Ammonia (NH3-N) Concentrations, November-May

FECAL COLIFORM

The NPDES permit limits for fecal coliform bacteria are 100 per 100 ml on a monthly average basis and 200 per 100 ml on a maximum week. Effluent records for 2018 through 2022 are shown in Figure 7-8. The plant has been in compliance with the monthly and weekly permit limits throughout the period of record.

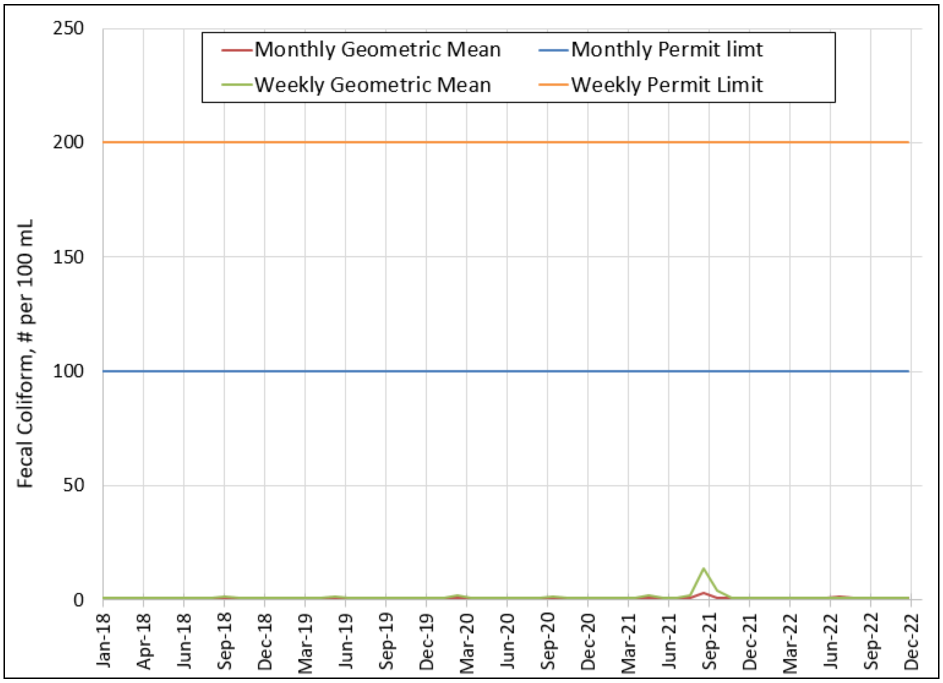


FIGURE 7-8

Effluent Fecal Coliform History – Monthly and Weekly Average

pH

Figure 7-9 shows that effluent pH has been in compliance with the permit range of 6 to 9 throughout the record period.

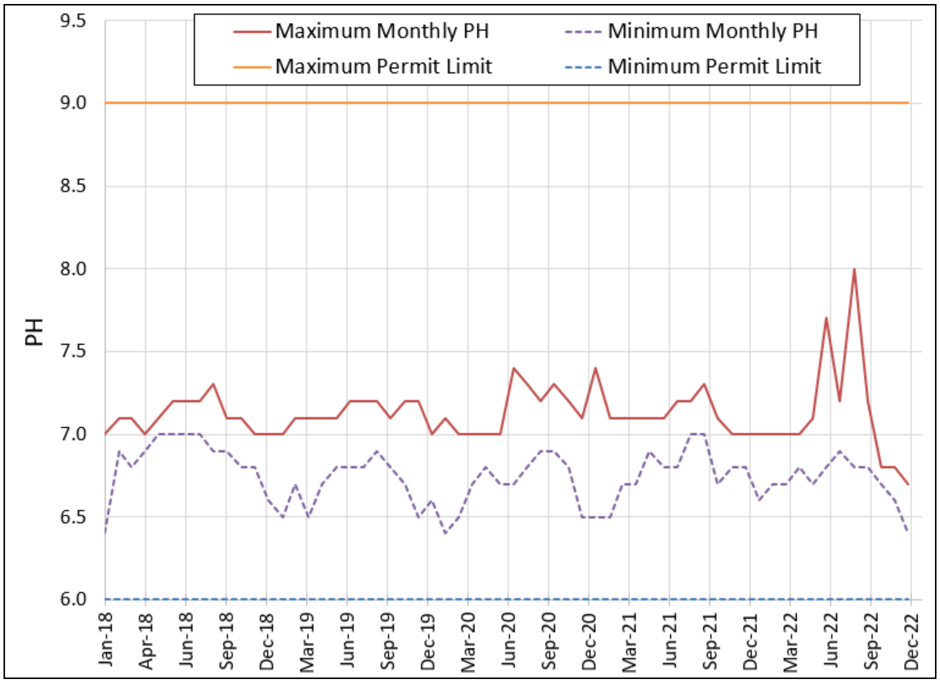


FIGURE 7-9

Effluent pH

OVERALL TREATMENT PLANT PERFORMANCE

Table 7-2 and 7-3 summarizes effluent data for five main performance parameters for the WWTP: BOD, TSS, NH3-N, Fecal Coliform, and pH. The plant has exhibited excellent performance over the record period: not only has there been no exceedance of any of the permit limits, but the effluent quality has been consistently high.

TABLE 7-2

Monthly Effluent Concentration Data

Description	BOD (mg/L)	TSS (mg/L)	BOD (lb/d)	TSS (lb/d)	BOD Removal %	TSS Removal %	NH3-N (mg/L) June-Oct	NH3-N (mg/L) Nov-May	Fecal Coliform (#/100 ml)
Permit Requirement	**30**	**30**	**173**	**161**	**85**	**85**	**3.6**	**13.9**	**100**
Average	1.6	0.6	3.8	1.5	99.4	99.7	0.2	0.1	1.0
Minimum	1.0	0.2	1.7	0.4	99.0	99.2	0.2	0.03	1.0
Maximum	3.0	1.3	7.6	4.5	99.7	99.9	0.2	0.4	3.0
2018 Average	1.6	0.6	3.1	1.1	99.4	99.7	0.2	0.1	1.0
2019 Average	1.3	0.3	2.7	0.7	99.6	99.9	0.2	0.2	1.0
2020 Average	1.7	0.5	3.8	1.1	99.3	99.8	0.2	0.2	1.0
2021 Average	1.8	0.9	4.6	2.4	99.4	99.5	0.2	0.2	1.2
2022 Average	1.7	0.8	4.8	2.2	99.5	99.8	0.2	0.2	1.0

TABLE 7-3

Weekly and Daily Effluent Concentration Data

Description	Weekly BOD (mg/L)	Weekly TSS (mg/L)	Weekly BOD (lb/d)	Weekly TSS (lb/d)	Daily NH3-N (mg/L) June-Oct	Daily NH3-N (mg/L) Nov-May	Weekly Fecal Coliform (#/100 ml)	Daily pH Min	Daily pH Max
Permit Requirement	**45**	**45**	**260**	**242**	**8.1**	**31.3**	**200**	**6.0**	**9.0**
Average	2.2	0.8	5.1	2.0	0.2	0.2	1.3	6.7	7.1
Minimum	1.0	0.3	2.2	0.6	0.2	0.04	1.0	6.4	6.7
Maximum	6.0	2.0	11.5	5.6	0.2	2.2	14.0	7.0	8.0
2018 Average	2.0	0.7	4.3	1.5	0.2	0.5	1.0	6.9	7.1
2019 Average	1.9	0.5	4.0	0.9	0.2	0.2	1.0	6.7	7.1
2020 Average	2.3	0.7	4.7	1.6	0.2	0.2	1.1	6.7	7.1
2021 Average	2.5	1.2	6.3	3.0	0.2	0.2	2.5	6.8	7.1
2022 Average	2.3	1.1	6.2	3.0	0.2	0.2	1.0	6.7	7.1

TREATMENT EVALUATION AT PROJECTED FLOW AND LOADING RATES

This section provides an evaluation of the capacity, and ability to accommodate projected 2033 and 2043 flows and loadings, of major WWTP process components based on 2018-2022 plant process data, design criteria, discussions with staff and site visits. Where applicable, current and projected flows and loadings to process components are compared to manufacturers' and published design criteria, such as those published in the Ecology's *Criteria for Sewage Works Design* (Orange Book, 2018), WEF *Design of Municipal Wastewater Treatment Plants, Manual of Practice No. 8* (2018) and *Wastewater Engineering, Disposal and Reuse* (Metcalf and Eddy, 2014) to determine if capacity is sufficient for the projected loading rates. In addition to the capacity of the current facilities, the capacity after implementation of Phase 1B improvements, which are expected to be implemented in the near future, are compared to the future conditions.

The process unit evaluation is provided in Table 7-4.

The NPDES permit requires the La Center WWTP to maintain a Reliability Class II in accordance with *Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability* (EPA Publication 430-99-74-001, 1973). Table 7-4 also shows reliability requirements for specific unit processes based on this publication.

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TABLE 7-4

Comparison of Component Design Capacity/Criteria and Projected Condition

Component (Parameter)	Recommended Criteria/Capacity	Reliability Requirement	Reference	Current Operating Condition (Criteria Met?)	2033 Operating Condition (Criteria Met?)	2043 Operating Condition (Criteria Met?)
Max Month Flow	current 0.69 mgd/ 1.04 mgd by Phase 1B		Plant Design Data	0.5 mgd (yes)	0.7 mgd (no/yes)	1.1 mgd (no)
Peak Hour Flow	current 1.29 mgd/ 1.94 mgd by Phase 1B		Plant Design Data	1.1 mgd (yes)	1.6 mgd (no/yes)	2.5 mgd (no)
Maximum Month BOD	current 1,297 lb/d/ 1,804 lb/d by Phase 1B		Plant Design Data	952 lb/d (yes)	1,494 lb/d (no/yes)	2,355 lb/d (no)
Maximum Month TSS	current 1,070 lb/d/ 1,581 lb/d by Phase 1B		Plant Design Data	658 lb/d (yes)	1,038 lb/d (yes)	1,643 lb/d (no)
Maximum Month NH3	current 194 lb/d/ 292 lb/d by Phase 1B		Plant Design Data	91 lb/d (yes)	145 lb/d (yes)	231 lb/d (no/yes)
Headworks Screen	6.2 mgd firm capacity, PHF	Equipment sized to provide design capacity with the largest unit out of service.	Manufacturer	1.1 mgd (yes)	1.6 mgd (yes)	2.5 mgd (yes)
Flow Meter	8.0 mgd, PFH	All units in service for maximum flow conditions.	Manufacturer	1.1 mgd (yes)	1.6 mgd (yes)	2.5 mgd (yes)
Headworks Building Ventilation System	12 air exchanges per hour			need to verify		
Anoxic Zone Detention Time	10-45 min, MMF	A backup basin is not required. But at least two equal volume basins must be provided. All units in service for maximum flow and loading conditions.	Ecology, 2019	330 min	213 min	140 min
	5-25 min, PHF		M&E, 2014	(yes)	(yes)	(yes)
	20-60 min, MMF					
Aerobic Solids Retention Time (SRT) for Nitrification	≥12 days MMF		Design calculations per M&E, 2014	22.0 days (yes)	16.0 days (yes)	8.8 days (no)
Aeration Tank Detention Time	≥6 hrs, AAF		Ecology, 2019	12.5 hr (yes)	11.2 hr (yes)	7.2 hr (no/yes)
	≥0.5 hr, PHF		M&E, 2014	4.1 hr (yes)	3.8 hr (yes)	2.5 hr (yes)
Aeration Tank MLSS	8000-12000 mg/l, MMF		Ecology, 2019	10,000 mg/l (yes)	10,000 mg/l (yes)	10,000 mg/l (yes)
BOD_5 Mass Loading Rate	≤60 ppd/1,000 ft3 MMF		Ecology, 2019	43 ppd/1,000 ft3 (yes)	46 ppd/1,000 ft3 (no/yes)	73 ppd/1,000 ft3 (no)
	≤120 ppd/1,000 ft3 PDF		M&E, 2014	36 ppd/1,000 ft3 (yes)	57 ppd/1,000 ft3 (yes)	91 ppd/1,000 ft3 (no/yes)
Fine Bubble Diffuser (Redundancy)		Each aeration basin shall be designed such that the largest section of diffusers can be isolated and repaired without measurably impairing the oxygen transfer capability of the system.	Ecology, 2019	(yes)		

TABLE 7-4 – (continued)

Comparison of Component Design Capacity/Criteria and Projected Condition

Component (Parameter)	Recommended Criteria/Capacity	Reliability Requirement	Reference	Current Operating Condition (Criteria Met?)	2033 Operating Condition (Criteria Met?)	2043 Operating Condition (Criteria Met?)
MBR Hydraulic Capacity	3.0 mgd, PHF	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.	Manufacturer	1.1 mgd (yes)	1.6 mgd (yes)	2.5 mgd (yes)
MBR Process Aeration Capacity	720 cfm (1600 cfm by Phase 1B) firm capacity	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.	Manufacturer	703 cfm (yes)	1,020 cfm (no/yes)	1,467 cfm (no/yes)
MBR Scouring Aeration Capacity	720 cfm firm capacity	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.	Manufacturer	333 cfm (yes)	666 cfm (yes)	666 cfm (yes)
RAS Pump	5,056 gpm, PHF	Equipment sized to provide design capacity with the largest unit out of service.	Manufacturer	2,279 gpm (yes)	3,523 gpm (yes)	5,355 gpm (no)
WAS Pump	331 gpm, 8 hours/day, PDF	Equipment sized to provide design capacity with the largest unit out of service.	Manufacturer	0.7 hrs/day (yes)	1.2 hrs/day (yes)	2.1 hrs/day (yes)
UV Disinfection System	3.5 mgd, PHF		Manufacturer	1.1 mgd (yes)	1.6 mgd (yes)	2.5 mgd (yes)
UV Disinfection System (Redundancy)		Equipment sized to provide maximum daily dose with single largest unit out of service.	Ecology, 2019	(yes)		
Sludge Storage Basins	12-18 days (Note: This is a recommendation, not a requirement.)	A backup basin or tank is not required.	M&E, 2014	28.2 days (yes)	17.6 days (no/yes)	10.1 days (no)
	490 cfm firm capacity	At least two blowers or mechanical aerators shall be provided. It is permissible for less than design oxygen transfer capability to be provided with one unit out of service.	Manufacturer	346 cfm (yes)	534 cfm (no)	909 cfm (no)
Rotary Fan Press	Hydraulic capacity 25 gpm, MMF, 60 hr/week		Manufacturer	53 hrs (yes)	85 hrs (no)	148 hrs (no)
	Loading capacity 133 pph, MMF, 60 hr/week		Manufacturer	48 hrs (yes)	74 hrs (no)	126 hrs (no)
Sludge Dryer	Rated capacity 2-3 batches/day, MMF, 40 hr/week		Manufacturer	3.1 batches/day (no)	4.8 batches/day (no)	8.1 batches/day (no)
Sludge Treatment (Redundancy)		Alternative methods of sludge disposal and/or treatment shall be provided for each sludge treatment unit operation without installed back up capability		(no)		

(1) Yellow shade indicates the criteria is not met under current condition, but will be met with Phase 1B upgrade; Pink shade indicate the criteria will not be met even after Phase 1B upgrade. Values were calculated including Phase 1B upgrade.

OVERVIEW OF CAPACITY AND CONDITION OF TREATMENT PROCESS UNITS

This section provides an evaluation of the capacity of the liquid and solids treatment processes to treat the projected flow and loading rates. All the treatment units must meet Class II Reliability Requirements. The resulting process loading rates are compared to accepted design criteria for each treatment process as presented in Table 7-4. In general, some of the liquid treatment units and most of the solids treatment units will require capacity upgrades during the planning period (through 2043) to accommodate the projected increase in influent flows and loadings. All treatment units will have to be expanded to accommodate buildout conditions.

A spreadsheet model was used to evaluate the projected 2043 flows and loadings to the major treatment system process units. The evaluation assumed that the Phase 1B upgrades would be implemented within the next 10 years; the Phase 1B improvements were originally planned for 2017, but were postponed due to slower than projected growth. The Phase 1B capacities are included in the current NPDES permit.

This section also provides a brief analysis of each component and the applicable criteria, summarizes the condition of the facilities, and develops recommended improvements. It should be noted that since most of the mechanical components were installed during the 2010 improvements project, they would approach the useful life span, typically 20 years, in the next 10 years. The recommended equipment replacement due to aging is listed at the end of the section. The following analysis of are mainly based on capacity of the treatment component.

INFLUENT PUMP STATION

The majority of the flows the headworks by gravity sewer. The capacity analysis of the Influent Pump Station (Pump Station 1) was addressed in the Chapter 6, Collection System Evaluation.

HEADWORKS

The existing headworks consist of a Parshall flume flow meter and two rotary drum fine screens. The Parshall flume influent flow meter has a capacity of 8.0 mgd, which is more than adequate for 20-year planning period conditions. The existing influent screens each have a rated peak hour flow capacity of 6.2 mgd, for a total capacity of 12.4 mgd. This capacity is more than the 20-year planning period projected peak hour flow of 2.5 mgd, including factoring in the reliability requirement for providing design capacity with the largest unit out of service. Therefore, no additional fine screen *capacity* is required for the planning period (in fact, they could potentially be down-sized). However, as noted below, the screens will likely need to be replaced within the 20-year planning period.

The headworks facilities are in good condition; however, as noted in Chapter 4, flushable wipes get stuck on the screens, causing a significant maintenance burden. Also, one screen appears to collect more solids than the other. It is anticipated that the screens will need to be replaced within the 20-year planning period, as their age will exceed the projected useful life.

Influent is sampled using a timed sampler instead of a flow-proportional sampler. Timed sampling is not as accurate as flow-proportional sampling and can result in underestimation of influent loadings. It is recommended that flow-proportional sampling be implemented.

SECONDARY TREATMENT WITH MBR SOLIDS SEPARATION

Recycle Pump

The recycle pump in an MBR serves two purposes: (1) circulating the mixed liquor at a high enough rate to keep it from becoming too concentrated near the membranes; and (2) returning NOx (nitrate and nitrite) to the anoxic zone where it can be denitrified. Recycle rates in MBR WWTFs are typically at a minimum of four times the AAF. However, for the La Center WWTP, the original design was based on the more conservative ratio of 6 times the rate of the influent MMF. As shown in Table 7-4, to satisfy Ecology's reliability and redundancy criteria with one unit out of service, the projected 20-year flow will slightly exceed the existing recycle pump capacity. Due to the conservative recycle flow ratio, the recycle pump capacity is likely sufficient. (Performance and the potential for upsizing could be re-evaluated after 10 years, as flows increase toward the projected 20-year planning levels.)

Aeration Basins

Common design criteria for aeration basin design are aerobic detention time, BOD loading and solids retention time (SRT).

<u>Aerobic Zone Detention Time</u>

According to Water Environmental Foundation (WEF) *Manual of Practice No. 8 (Design of Municipal Wastewater Treatment Plants*, 2010) and Orange Book guidelines, the aerobic zones in a secondary treatment system designed for nitrogen removal should have a minimum detention time of approximately 6 hours. However, this criteria applies to conventional activated sludge systems that are typically designed to operate at mixed liquor suspended solids (MLSS) concentrations of around 3,500 mg/L and is not directly applicable to membrane bioreactor treatment systems such as the La Center WWTF. The membrane bioreactor system here is designed for an MLSS concentration of 8,000 mg/L and is ultimately capable of operating at MLSS concentrations of 12,000 mg/L and above. It is reported that current MLSS concentrations are 7,250-7,500 mg/l MLSS in the pre-aeration basins and 9,000 mg/L in the MBR tanks, which are considered adequate for the treatment process.

In treatment systems such as this, there is a higher concentration of biomass including nitrifying and denitrifying microorganisms for ammonia and nitrogen removal than in a conventional activated sludge system, which reduces the detention time required to achieve the same level of treatment. The projected 2043 flows will result in an aerobic detention time of 7.2 hours in the combined aerated tank volumes of the aeration basins and membrane basins after Phase 1B completion, which meets the 6 hours criterion. (This is, in fact, more than sufficient given the higher operating mixed liquor concentration reducing the needs for detention.)

<u>BOD Loading</u>

Activated sludge systems can be operated over a wide range of loadings. The Orange Book provides a desirable range of design values for aeration tank organic loading of 20 to 60 pounds of BOD per day per 1,000 cubic feet of tank volume (ppd/kcf). M&E recommends a maximum BOD loading rate of 120 ppd/kcf. As shown in Table 7-4, the projected 2043 BOD loading of 73 ppd/kcf exceeds the aeration basin capacity based on the standard sizing criterion of 60 pounds of BOD per day per 1,000 cubic feet. However, as stated above, membrane bioreactor systems typically operate at higher concentration of biomass. Thus, higher BOD loading capacity for membrane bioreactors can be allowed for than for conventional activated sludge system on which the loading criteria are based. For MBR systems, hydraulic capacity needs predominantly dictate tank volumes.

<u>SRT</u>

SRT (Solids Retention Time) is the average time the activated sludge solids are in the system. (Aerobic SRT is the average time the activated sludge solids are under aeration or in oxygenated portions of the system, so it is somewhat less than true SRT.) SRT is an important factor affecting the performance of nutrient removal and sludge characteristics. At the WWTF, nitrification is required to meet effluent NBOD limits; a typical SRT value range for complete nitrification is between 6 and 18 days, depending on mixed-liquor temperature. Including residence time in the membrane basin, the total SRT at the WWTF has historically averaged 22 days.

The membrane manufacturer prefers an aerobic SRT in the aeration basins of at least 12 days including safety factor of 2, with a typical recommended range of 15 to 25 days. A longer SRT allows slower growing, diverse, microbial populations, such as nitrifiers, the opportunity to establish viable populations, allowing increased resistance to toxic upsets and better degradation of complex organics. High SRTs also create an enabling environment for low sludge production. However, extremely high SRTs are not desirable as they increase membrane fouling due to biofouling, increasing sludge viscosity and reduced aeration efficiency. Also, when the SRT is too low (< 15 days), the bacteria can foul the membranes with muco-polysaccharides and foul the membranes, causing the TMP to rise rapidly.

SRTs were calculated at projected 2033 and 2043 loadings based on a design MLSS of 10,000 mg/L and compared to the minimum required aerobic SRT; the calculated SRT values are shown in Table 7-4. With the Phase 1B aerobic tank volume, for 2043 conditions, the SRT of 8.8 days is below the required minimum aerobic SRT. However, the SRT and the MLSS are directly correlated. A higher SRT will be achieved by increasing the MLSS with the aeration basins volume unchanged. Lower internal recycle rates will increase the membrane basin MLSS. The effective 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. It is recommended that as loadings approach the BOD_5 removal capacity of the aeration basins, the operational parameters should be reviewed and analyzed to establish appropriate expansion schemes.

<u>Aeration Capacity</u>

Aeration demand was calculated using measured quantities based on the configuration of the aeration basin and typical correction factors as described by Metcalf and Eddy (2014). For the purpose of this assessment, the values listed in Table 7-4 reflect the aeration capacity of the Aeration Basin alone, and do not include the oxygen transferred by the coarse bubble diffusers in the MBR Basin. As indicated in Table 7-4, the aeration system capacity should be sufficient through the planning period with the Phase 1B improvements. The blowers will need to be replaced within the 20-year planning period.

Anoxic Tank

Anoxic basins with a minimum capacity of 25 percent of the aeration basin is typically sufficient for denitrification; therefore, the existing anoxic basin is also adequately sized for the projected 20-year flows and loadings. The hydraulic detention times are in compliance with the Ecology and M&E recommended 10 to 30 minutes throughout the 20-year period.

Membrane Basins

The membrane system was evaluated based on equipment capacity. Ecology's reliability criteria require the membrane basins to be sized with the capacity to process 50 percent of peak flow with one train out of service.

Two basins currently containing the membranes with a capacity of 1.5 mgd are adequate for projected flow through year around 2028. Installing membrane in the other two basins in the Phase 1B expansion project will double the capacity to 3.0 mgd, which is sufficient throughout the 20-year planning period.

The membrane filtration system is supported by the scouring air blowers. The existing blowers will provide sufficient capacity for year 2043 conditions.

In general, the anoxic basins, aeration basins, membranes and associated equipment are in good condition. The City's last set of submerged membranes lasted for 12 years and was replaced in 2022. (Flat plate membrane often will last longer than 10 years if they are properly maintained and cleaned regularly.) For the purposes of this plan, the membranes are expected to have 12 years of useful life, similar to the last set.

ULTRAVIOLET DISINFECTION SYSTEM

The existing ultraviolet disinfection system has three units, two small units each with a rated capacity of 1.75 mgd and one large unit rated at 3.1 mgd, bringing the peak hour capacity to 3.5 mgd, considering the reliability requirements (largest unit out of service). With this capacity, the existing ultraviolet disinfection system should be adequate throughout the 20-year planning period. The larger unit, installed in 2009, is in good condition. The two small units are in fair condition but are nearly 20 years old and equipped with the older (less safe) control panels; these units will likely need to be replaced within 10 years.

OUTFALL

The 10-inch outfall piping system has a capacity of about 2.8 mgd at peak velocity of 8 feet per second, which is sufficient to accommodate the projected 20-year peak hour flow of 2.5 mgd.

WAS PUMPS

Sludge production rates for 2033 and 2043 were projected from the activated sludge process spreadsheet evaluation summarized in Table 7-4, which was calibrated for actual sludge production during 2022. As shown in Table 7-4, each 331-gpm WAS pump can pump maximum daily sludge volume in 2.1 hours per day in 2043. Thus, the WAS pumps should have sufficient capacity for the projected flows, through the 20-year planning period, including the reliability requirement with one pump standby.

AERATED SLUDGE STORAGE BASIN (ASSB) AND SLUDGE STORAGE BASIN (SSB)

The objective of the aerated sludge storage basin (ASSB) and sludge storage basin (SSB) is to provide additional aerobic detention time to increase sludge stabilization and improve dewatering characteristics, optimizing operation of the downstream dewatering and drying process. Textbook values suggest that WAS from facilities without primary clarifiers can be aerobically conditioned in 12-18 days (Metcalf and Eddy 2014). These detention times fall short of the 40 to 60 days of detention time required to meet Class B Biosolid requirements, so further processing with the sludge dryer will continue to be required, which will be evaluated in Chapter 8. The fact that the MLSS concentration (6,250 mg/L) in the ASSB is low compared to the feed WAS concentration (9,000 mg/L), suggests removal of volatile solids through digestion activities in the basins.

The ASSB also allows WWTF staff flexibility with maintenance and runtimes of the dewatering system by providing additional storage volume for WAS.

As shown in Table 7-4, providing the City with enough flexibility in dewatering operations, the projected storage detention time of 17.6 days in 2033 and 10.1 days in 2043 at maximum month loadings should be sufficient based on a reasonable seven-day detention time. Providing at least 12 days of aerobic detention time, the existing sludge holding tank is not sufficient for projected year 2043 conditions. (12 days of aerobic detention is not a requirement for partial sludge stabilization in the basins, but a guideline.) Thus, either additional storage, or WAS thickening, as suggested in the 2008 Facility Plan should be considered to increase the detention time, as necessary, after 2033.

Aeration within the holding tank is not used to meet the digestion requirements, but rather provide partial aerobic stabilization and solids mixing. As shown in Table 7-4, the aeration requirement will exceed the current aeration capacity around 2033. It is recommended that additional aeration capacity be added by then.

SLUDGE DEWATERING

The capacity of the existing dewatering rotary fan press was evaluated at the projected WAS flows. The unit currently produces dewatered sludge of 11-13 percent solids, but the system was designed for 18 percent solids. The hydraulic capacity of 25 gpm and solids loading capacity of 113 lb/hr were used to determine the required runtimes for maximum WAS sludge production. Notably, this system was originally designed based on 12 hours for 5 days each week, and, alternatively, 8.6 hours for 7 days each week. As shown in Table 7-4, these schedules would limit operational capacity to 12,860 gpd and 1,140 lb/day, which are inadequate for current and future loading. However, even extending the hours of operation to 85 hours per week (12.1 hours for 7 days each week) will only meet the maximum monthly capacity requirement until 2033. The dewatering capacity needs to be increased within the next 10 years by either adding extra unit(s) or replacing the existing with larger size unit(s).

In addition to capacity issues, the fan press is in moderate condition and would need to be replaced within the 20-year planning period. Plant staff have expressed that a Huber screw press is a preferred option. (G&O would recommend consideration of other presses as well, including those manufactured by FKC in Port Angeles, if a screw press is ultimately selected by the City.)

SLUDGE DRYING

The existing sludge dryer is capable of processing 33 cubic feet of sludge per batch. The rated capacity of the unit is 6-8 batches per day based on 24 hour/day operation; however, due to mechanical problem with the dryer, La Center is not comfortable operating an entire drying batch cycle unattended. This limits the effective dryer capacity to two to three batches per day. As shown in Table 7-4, it is expected that dryer feed will be 4.8 batches per day in 2033 and 8.1 batches per day in 2043, based on design feed dewatered sludge of 18 percent solids.

Theoretically, the 2033 maximum monthly loading requirements can be met by extending the runtime to 64 hours each week (12.8 hours for 5 days or alternatively 9.1 hours for 7 days). The sludge drying unit operates most efficiently when running continuously. However, continuous operation, continuous oversight, and additional shifts are not realistic. The dryer is approaching 20 years of service and is reaching the end of its useful life; in addition, it is difficult to find replacement parts for it.

The fan press and dryer constitute single points of failure, putting the City at risk of having to haul liquid sludge if they are shut down.

Additional information regarding current and future biosolids treatment and management is presented in Chapter 8.

SUMMARY OF EXISTING FACILITIES EVALUATION

The evaluation presented above has concluded that the following facilities do not have the treatment capacity for the projected flows and loadings for the 20-year planning period. Improvements required to meet future capacity needs, incorporated with the phased development from 2008 Facility Plan are summarized below and will be further discussed in Chapter 6.

- In the next 5 years (2024-2028), it is recommended that the Phase 2 WWTP Expansion be completed, including upgrading sludge dewatering, improving sludge stabilization by adding sludge thickening or increasing aeration sludge storage.

- In the following 5years (2029-2033), it is recommended that the Phase 1B WWTP Expansion be completed, including installing membranes in the two empty MBR basins and adding one additional blower for the aeration system.

The rerated capacity with each phase of upgrades was obtained from 2008 Facility Plan and is summarized in Table 7-5 with the original estimated year of completion. Rerated capacity for Phase 2 upgrade was not specified since solids handling improvements is not directly related to the influent flows. However, the original estimated timing for Phase 2 is 2013, right after the completion of Phase 1A in 2012. Phase 3 upgrade is not required for the 20-year planning period based on current operation performance and effluent discharge requirement. However, it is recommended that the City re-evaluate the potential for upsizing after 10 years.

Figure 7-10 shows the overall site map from the 2019 Draft General Sewer Plan of the upgrades to the treatment plant by phase.

TABLE 7-5

Rerated Capacity after Phased Upgrades from 2008 Facility Plan

Parameters	Phase 1A	Phase 1B	Phase 3
Est.Year of Completion in 2008 Facility Plan	2012	2017	2027
Influent Flows			
Annual Average Flow, mgd	0.51	0.76	1.65
Annual Dry Weather Flow, mgd	0.42	0.62	1.35
Annual Wet Weather Flow, mgd	0.55	0.83	1.60
Max Month Wet Weather Flow, mgd	0.69	1.04	2.25
Peak Daily Flow, mgd	1.29	1.94	4.20
Peak Hourly Flow, mgd	1.91	2.88	6.22
Influent Loadings			
BOD			
Annual Average, lb/day	1,013	1,409	3,001
Max Month Dry, lb/day	1,276	1,775	3,781
Max Month Wet, lb/day	1,297	1,804	3,841
TSS			
Annual Average, lb/day	817	1,207	2,638
Max Month Dry, lb/day	964	1,424	3,113
Max Month Wet, lb/day	1,070	1,581	3,456
Ammonia			
Annual Average, lb/day	148	223	482
Max Month Dry, lb/day	175	263	569
Max Month Wet, lb/day	194	292	631

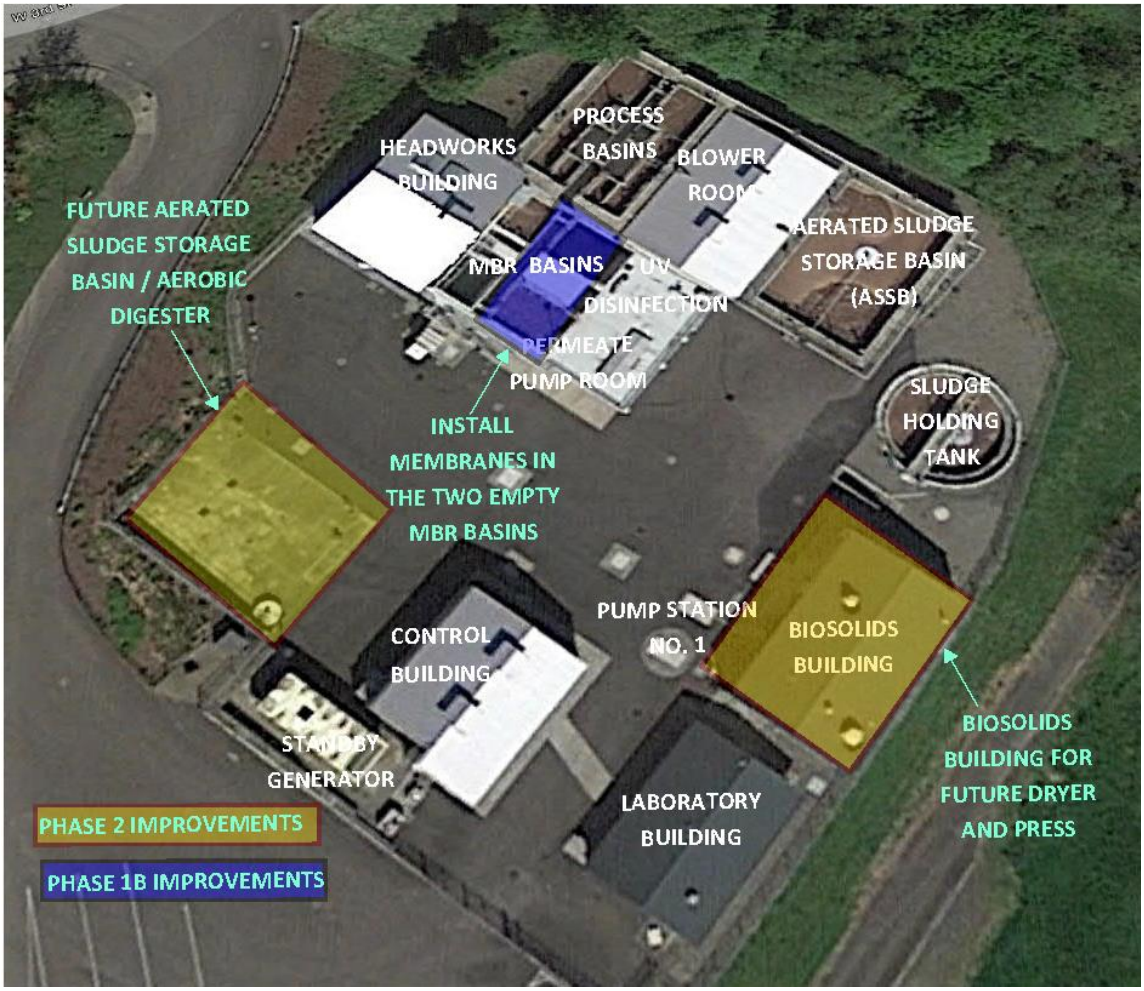


FIGURE 7-10

WWTP Proposed Improvements by Phase

Cost Estimates for Liquid Stream Improvements

Table 7-6 presents the capital costs, additional operations costs, and 20-year life cycle costs for the WWTP *liquid stream* improvements recommended for the next 10 years -the Phase 1B membrane system expansion improvements. Capital costs are total project costs, in January 2024 dollars, inclusive of construction, contingency, administration, engineering and sales tax. The cost estimate for the Phase 2 *solids treatment* improvements is presented in Chapter 8 - Biosolids Treatment and Management.

TABLE 7-6

Cost Estimates for WWTP Liquid Stream Improvements – 2024-2043

Improvements	Capital Cost	Relative Annual Operations Cost(2)	20-Yr Life Cycle Cost(1)
Phase 1B	$4,331,000	$154,000	$6,616,000

(1) Life cycle cost assumes a 3 percent interest rate.
(2) Includes labor, chemical, material, electricity, structural maintenance and equipment replacement costs.

As noted earlier, the age of much of the process equipment will be beyond its useful life in the next 10 years. The replacement cost of the aging equipment is summarized in Table 7-7.

TABLE 7-7

Cost Estimates for Equipment Replacement

Equipment Description	Estimated Replacement – Project Cost	
	10 Years	20 Years
Headworks Upgrade	$2,371,000	
UV System Replacement	$1,465,000	
MBR Basin 1 and 2 Membranes	$800,000	
Preaeration and MBR Basin 1 and 2 Blowers	$873,000	
SCADA and Controls		$500,000
Misc. Mechanicals (pumps, mixers, odor control, etc.)	$2,318,000	
Total	**$7,827,000**	**$500,000**

CHAPTER 8

BIOSOLIDS TREATMENT AND MANAGEMENT

This chapter provides an evaluation of the treatment and management of biosolids from the City's WWTP and provides recommendations to ensure reliable service through the 20-year planning period and beyond. Alternatives are evaluated in terms of capital cost, annual operation and maintenance cost, ease of operations and maintenance, sustainability, and reliability.

As discussed in Chapter 7, Wastewater Treatment Plant Analysis, all the solids treatment process units will require capacity increases or replacement due to aging equipment throughout the planning period. Alternatives are evaluated and recommendations are provided for upgrading and increasing the capacity of the biosolids treatment and management system.

A discussion of the applicable regulatory biosolids treatment and management requirements is provided in Chapter 3, Regulatory Requirements. A description of the existing WWTP and current aerobic digestion and Class A biosolids treatment system is included in Chapter 4, Existing Facilities.

Background information regarding the existing system and general issues regarding biosolids treatment and management is provided immediately below, followed by a discussion of alternatives and recommendations.

EXISTING SYSTEM

The liquid stream treatment processes at the City's WWTP produce waste activated sludge (WAS) which is pumped to the aerobic sludge storage basin (ASSB), which serves as an aerobic digestion system. Here, the volatile fraction of waste sludge is reduced to decrease the overall mass of biosolids leaving the facility. Aerobic digestion also helps the solids handling system meet vector attraction reduction requirements in accordance with WAC (Washington Administrative Code) 173-308-180. Digested sludge is either transferred to the sludge storage basin (SSB) or pumped to the rotary fan press dewatering system to reduce the volume and water content of biosolids sent to the existing dryer system.

The SSB is another aerated tank where digested sludge may be stored prior to being pumped to the rotary fan press dewatering system. The SSB also receives waste sludge trucked in from the Ilani Tribal Casino, Kalama, and Ridgefield wastewater treatment plants. The contents of the SSB are pumped to the rotary fan press dewatering system.

The WWTP uses an indirect thermal fluid dryer to produce Class A biosolids from its waste solids. Pathogen reduction requirements (WAC 173-308-170) for Class A biosolids are accomplished through the dryer, which uses heat to drive evaporation of dewatered sludge until the solids percentage of the sludge exceeds 90 percent dry solids by weight. The existing dryer is located in a building and discharges the Class A biosolids to a surge bin. From there, a series of water-jacketed conveyors cool the solids as they are transported to the storage area. The City successfully distributes Class A biosolids to the public, which is used as soil conditioner, fertilizer, or fertilizer supplement, or to Lewis River Reforestation for agricultural use.

CONDITION

The existing dryer, Fenton Model RK36, was installed in 2002 is fast approaching the end of its useful life; replacement parts have not been available. Given the age of the system as well as ongoing issues obtaining replacement parts and the expected difficulty of repairing and maintaining the system in the future, the City would like to replace the dryer system with another dryer or another technology.

BIOSOLIDS PRODUCED

Table 8-1 summarizes biosolids production by the existing dryer. The data shown in Table 8-1 is compiled from the annual biosolids reports for the La Center WWTP.

TABLE 8-1

Biosolids Processed by the Existing Fenton Dryer

Year	Biosolids Production, Dry Tons
2017	71.4
2018	106.5
2019	99.3
2020	96.2
2021	125.0
2022	89.0

BIOSOLIDS QUALITY

Existing Standards

The City's existing treatment system easily meets the existing biosolids quality standards promulgated in WAC-173-308 and discussed in Chapter 2. As shown in Table 8-2, the City easily meets the Table 3 (Exceptional Quality) standards for trace pollutants. For pathogen reduction and vector attraction, the WWTP meets Class A utilizing thermal drying. Because thermal drying is utilized to produce Class A biosolids, the aerobically digested biosolids have never been tested for Class B biosolids standards. The pathogen reduction requirements appropriate for aerobic digestion for Class B and thermal drying for Class A are shown in Tables 8-3 and 8-4, respectively. Table 8-5 summarizes the vector attraction reduction requirements.

TABLE 8-2

Allowable Biosolids Trace Pollutant Concentrations for Land Application[1]

Element	Symbol	Ceiling Concentration (mg/kg)[1]	EQ Limit (mg/kg)[2]	City of La Center 2019-2021 (mg/kg)	
				Maximum	Average[4]
Arsenic	As	75	41	6.0	4.5
Cadmium	Cd	85	39	0.9	0.7
Copper	Cu	4,300	1,500	218	179
Lead	Pb	840	300	10.3	7.4
Mercury	Hg	57	17	0.5	0.2
Molybdenum	Mo	75	75 [3]	11.8	9.0
Nickel	Ni	420	420	16.5	13.7
Selenium	Se	100	100	4.7	4.1
Zinc	Zn	7,500	2,800	616	512

(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.
(4) Average of Samples with Detectable Concentration of Element.

TABLE 8-3

Class B Pathogen Reduction Requirements Relevant for Aerobic Digestion

Alternative 1	Fecal coliform 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.
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.

TABLE 8-4

Class A Pathogen Reduction Requirements Relevant for Heat Drying

All Alternatives	Fecal coliform <1,000 MPN per gram total solids, or salmonella <3 MPN per 4 grams total solids.
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.

TABLE 8-5

Vector Attraction Reduction Alternatives Relevant for Aerobic Digestion and Heat Drying

No.	Description
1.	Biosolids digestion process with >38 percent volatile solids reduction.
2.	Test end product of an aerobic digestion process: 40-day aerobic 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.	Treatment by drying. Can include unstabilized primary wastewater solids. Total solids >90 percent before mixing with other materials.

Potential Future Standards – Microconstituents, Including PFAS

Microconstituents are natural or manmade compounds that are detected in the environment with a potential effect on organism development and human health. Pharmaceuticals and personal care products (PPCPs) are significant sources of microconstituents in the environment; however, most of the concern about microconstituents currently is regarding PFAS (per- and polyfluoroalkyl substances).

PFAS are a class of several thousand different compounds that have been manufactured and used in a large number of products and applications since the 1940s. Their extreme chemical and biological stability, which provides their product usefulness, makes them very persistent and mobile in the environment. PFAS were incorporated into components of inks, varnishes, waxes, firefighting foams, metal plating, cleaning solutions, coating formulations due to their unique chemical properties as lubricants, water and oil repellents, paper, and textiles. They are frequently detected around the world in water, air, soil, wildlife, household dust and products, and humans. A growing number of studies have linked exposure to PFAS, primarily through ingestion, to a range of negative health effects. Perfluorooctane sulfonate (PFOS) and Perfluorooctanoic Acid (PFOA) are the best-studied perfluorinated chemicals, and the members of the family that carry the greatest health concerns. The compounds have been shown to affect liver function, alter reproductive hormones and increase infant mortality. Concern over the health effects led to a voluntary phase-out of the manufacture of several PFAS in the United States between 2000 and 2015. However, many forms of PFAS are still widely used, and even the phased-out forms are still produced in outside the US and found in products imported to the US.

Long chain PFAS compounds have been banned but the PFAS replacement compounds have the capability of degrading into long chain PFAS precursors. The carbon fluorine bond in PFAS is one of the strongest bonds and does not breakdown easily. Thus, conventional wastewater treatment technologies are not capable of removing PFAS. This results in PFAS leaching into agricultural soils and entering crops after land application.

PFAS can be found in high concentrations in industrial discharges. Several studies of industrial dischargers have shown that landfill leachate had the highest PFAS concentrations among the industries tested, although significant concentrations have been found in other industries including electronics manufacturing, the aviation industry and any producer or user of fire-fighting foams. In addition, many consumer products are known to contain, and serve as sources of, PFAS, including:

- Carpet and carpet cleaning and treatment products
- Textiles for furniture and clothing and stain resistant and porous waterproofing materials
- Treated paper food packaging – for water, oil, and grease resistance and non-stick performance
- Non-stick cookware
- Treated floor waxes and stone and wood sealants
- Cosmetics
- Many other consumer products

Per the Washington State Department of Ecology, fire fighting foams are the leading cause of PFAS contamination in Washington State. PFOS and PFOA were discovered in drinking water in Airway Heights, leading the town near Spokane to advise residents to drink bottled water for several weeks while the city switched over to a different water supply. Private wells on Whidbey Island were also found to contain PFOS and PFOA. In past years, the chemicals have been found in wells serving DuPont and Issaquah as well.

Regulations

EPA has established health advisory levels (70 ppt, parts per trillion, for PFOA plus PFOS) in drinking water; however, there have not been any federal or statewide regulations for treated wastewater or biosolids. Maine has imposed some of the most stringent regulations for PFAS, including for three PFAS compounds in biosolids, and other states have regulated PFAS or are looking to regulate.

Wastewater secondary sludge and biosolids have shown to be routes for PFAS emission to the environment. Treatment techniques for removal of PFAS in biosolids have been evolving but the results are not publicly available and have not been compared to each other.

Washington State

PFAS Chemical Action Plan (CAP) was issued by Washington State in November 2021. The State's objectives were to:

1. Ensure drinking water is safe.
2. Manage environmental PFAS contamination – contaminated sites and identified industries.
3. Reduce PFAS in products.
4. Understand and manage PFAS in wastewater (municipal and industrial), landfills, and biosolids.

PROPOSED SYSTEM

The City of La Center has expressed interest in maintaining and upgrading the existing biosolids treatment process and management system consisting of the following unit processes:

- Aerobic Digestion to meet Class B Biosolids and Vector Attraction regulations.

- Dewatering to prepare the digested solids for sludge drying or possibly another process producing Class A biosolids, as well as hauling of Class B biosolids to a permitted utilization site.

- Sludge drying or another process producing Class A biosolids.

PROJECTED BIOSOLIDS LOADS

The loads to the biosolids treatment and management facilities at the La Center wastewater treatment facilities originate from several sources. Besides the waste activated sludge generated at the treatment plant, waste sludges are hauled in by tanker trucks from three external sources:

- Cowlitz Tribe Casino Wastewater Treatment Plant
- Ridgefield Wastewater Treatment Plant
- Kalama Wastewater Treatment Plant

The existing and projected future total biosolids loads to be treated at the La Center biosolids treatment facilities and summarized in Table 8-6.

TABLE 8-6

Untreated Biosolids Loads

Year	**2023**	**2033**	**2043**
Annual Average Suspended Solids (lbs/day)	869	1,324	2,279
Maximum Month Suspended Solids (lbs/day)	1,054	1,747	2,956
Peak Day Suspended Solids (lbs/day)	1,425	2,563	4,106
Annual Average Volatile Suspended Solids (lbs/day)	732	1,114	1,916
Maximum Month Volatile Suspended Solids (lbs/day)	888	1,470	2,485
Peak Day Volatile Suspended Solids (lbs/day)	1,200	2,156	3,452
Annual Average flow (gpd)	11,780	18,600	32,920
Maximum Month Flow (gpd)	13,940	24,330	42,440
Peak Day Flow (gpd)	18,880	35,930	59,060

AEROBIC DIGESTION

The existing aerobic digestion system at the La Center treatment plant has satisfactorily served the City for many years and can be readily expanded to accommodate future conditions. The aerobic digestion system presently consists of the aerated sludge storage tank (ASST), with a volume of 267,000 gallons, and the sludge storage tank (SST) with a volume of 53,000 gallons.

Class B biosolids pathogen reduction requirements can be met by providing an aerobic digester volume that will provide 60 days solids detention time at 15^{o}C. This solids detention time can be shortened to 40 days if the temperature of the digester contents can be maintained at 20^{o}C. However, this is unlikely to take place during the winter months, when the peak month solids production is expected to occur.

Furthermore, the solids detention time can be shortened by 30 percent if the aerobic digestion system is designed as a two-stage system consisting of two equal-size digesters operating in series. This reduction will result in a total solids detention time of 42 days at 15°C for a two-stage aerobic detention system.

Another common method for achieving Class B biosolids pathogen reduction is anaerobic digestion. This process, however, is more appropriate for treatment facilities that incorporate primary sedimentation. Primary sludge contains mostly anaerobic microorganisms and will not require significant acclimation when introduced to the anaerobic digestion process. The treatment process at the La Center WWTP does not include primary sedimentation and all the sludge to be digested contain aerobic organisms exclusively. It is therefore recommended that the aerobic digestion process be continued at the La Center WWTP.

It is recommended that the SST be abandoned and that a new 267,000-gallon aerobic digester be constructed to operate in series with the ASST, resulting in a two-stage aerobic digestion system with a total volume of 534,000 gallons. This volume will provide 42 days of detention time if the solids in the digesters are thickened to a solids concentration of approximately 1.93 percent by decanting the supernatant. This is based on a 38 percent removal of volatile suspended solids resulting in a 2043 peak month digested sludge suspended solids load of 2,042 pounds per day.

The new aerobic digester will be aerated using fine bubble diffused air, similar to the ASST, providing complete mixing in the digester tank. The air supply will be 600 cfm.

The new aerated digester will be equipped with decant facilities to allow the operators to occasionally skim off the supernatant so that the solids concentration in the digesters may be increased to 1.9 percent.

DEWATERING

Table 8-7 shows the biosolids loads to the dewatering unit after 38 percent removal of volatile suspended solids and thickening to 1.9 percent solids in the aerobic digesters.

TABLE 8-7

Biosolids Loads after Aerobic Digestion

Year	2023	2033	2043
Annual Average Suspended Solids (lbs/day)	591	901	1,551
Maximum Month Suspended Solids (lbs/day)	717	1,188	2,042
Peak Day Suspended Solids (lbs/day)	969	1,744	2,794
Annual Average Volatile Suspended Solids (lbs/day)	454	691	1,188
Maximum Month Volatile Suspended Solids (lbs/day)	551	911	1,541
Peak Day Volatile Suspended Solids (lbs/day)	744	1,337	2,140
Annual Average flow (gpd)	3,730	5,690	9,800
Maximum Month Flow (gpd)	4,530	7,510	12,710
Peak Day Flow (gpd)	6,120	11,020	17,660

It is assumed that the dewatering unit will operate half-time under 2043 peak month conditions. This will result in a required capacity of approximately 170 pounds per hour. This will correspond to a dewatering system feed flow of about 18 gpm at a solids concentration of 1.9 percent, which will be the capacity of the dewatering system feed pump.

The dewatering system feed pump would be a progressive cavity pump with a capacity of 20 gpm at 30 psi discharge head. An additional equal capacity pump will also be provided for standby capacity to meet reliability criteria.

The City has expressed interest in changing the dewatering process from a rotary fan press to a screw press system. Other common dewatering processes include centrifuges and belt filter presses. Centrifuges are capable of producing a high solids sludge cake (20-25 percent TSS), but they require a considerable amount of electrical power. Belt filter presses are compatible to screw press in dewatering performance, cost, and power consumption. For the purpose of developing costs for biosolids treatment improvements, it is assumed that a screw press will be installed.

In addition to the screw press itself, this system is anticipated to include a polymer feed system, a mixed flocculation tank, an electrical control panel, field instrumentation, final product conveyors, a support structure, and an access platform. The polymer system will be sized to deliver up to 35 pounds of polymer per ton of solids to be dewatered. This will result in a capacity of approximately 3.0 pounds of neat polymer per hour.

FKC, Ltd., a manufacturer of screw press dewatering systems located in Port Angeles, Washington, was contacted to provide a proposal for a dewatering system meeting the criteria outlined in this Plan. The system proposed by FKC, Ltd., will form a basis for further discussions of dewatering systems presented in this Plan, such as preliminary layouts and costs. This system would be installed where the existing dewatering system is located. It is recommended that other dewatering systems be evaluated during preliminary designs for the biosolids treatment and management facilities.

It is estimated that a screw press will produce a dewatered sludge cake with a solids concentration of 16 percent. This solids concentration is based on tests conducted onsite by FKC Ltd. in 2008. Table 8-8 shows the biosolids loads after dewatering assuming a 95 percent solids capture rate through the dewatering unit.

TABLE 8-8

Biosolids Loads after Dewatering

Year	2023	2033	2043
Annual Average Suspended Solids (lbs/day)	561	856	1,473
Maximum Month Suspended Solids (lbs/day)	681	1,129	1,940
Peak Day Suspended Solids (lbs/day)	921	1,657	2,654
Annual Average Volatile Suspended Solids (lbs/day)	431	656	1,129
Maximum Month Volatile Suspended Solids (lbs/day)	523	865	1,492
Peak Day Volatile Suspended Solids (lbs/day)	707	1,270	2,033
Annual Average flow (gpd)	421	641	1,104
Maximum Month Flow (gpd)	510	846	1,432
Peak Day Flow (gpd)	690	1,241	1,989

These loads will be used to establish the capacities of facilities producing Class A Biosolids. The City would also like to explore the potential alternative of Class B biosolids hauling of these loads by a third party for treatment/disposal.

COST OF AEROBIC DIGESTION AND DEWATERING

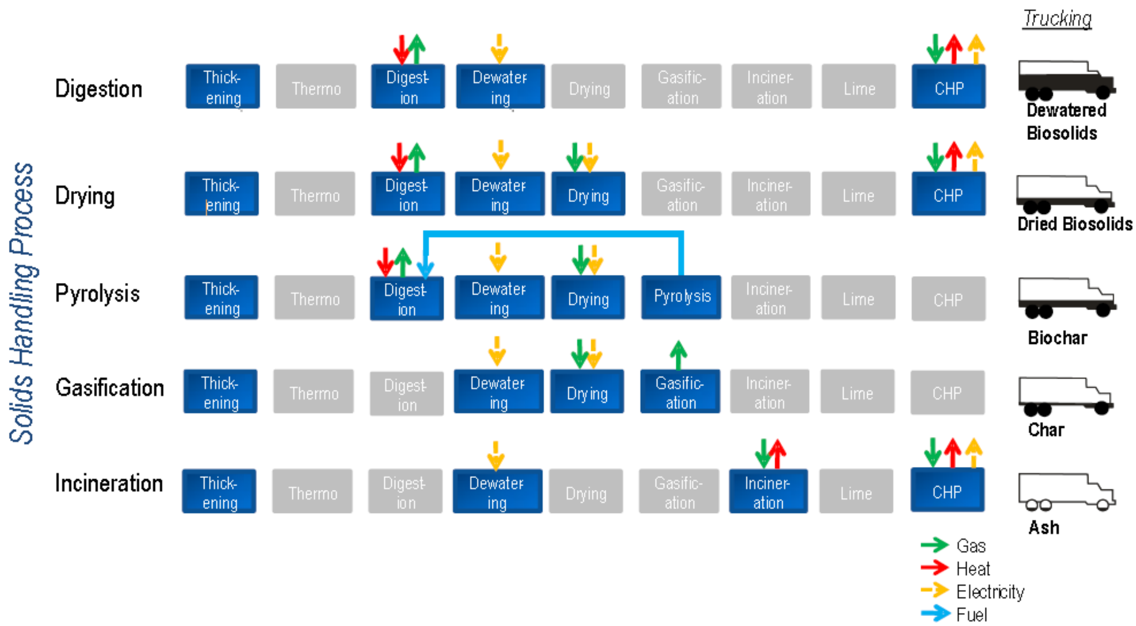
The total project cost of an aerobic digester expansion and a dewatering unit to accommodate year 2043 biosolids loads have been estimated to be $5,293,000. This cost includes construction of the new facilities, sales tax, engineering, and administrative costs.

CLASS A BIOSOLIDS TREATMENT TECHNOLOGIES

This section provides a summary of Class A biosolids treatment technologies. The technologies considered include:

1. Biosolids Drying
2. Alkaline Stabilization
3. Composting
4. Gasification
5. Pyrolysis
6. Incineration

These technologies (except composting and alkaline stabilization) are shown schematically in Figure 8-1.



Parry, 2014.

FIGURE 8-1

Schematics of Biosolids Treatment Technologies

TECHNOLOGY ALTERNATIVE 1 – BIOSOLIDS DRYING

Drying of sludge has been successfully implemented at WWTPs since the 1920s and produces a marketable dry solids product that can be used as a fertilizer or biofuel. As of 2012, there were more than 60 drying systems operating in the U.S., and more than 100 in Europe. Drying is based on the removal of water from dewatered solids, which accomplishes both volume and mass reduction. At municipal WWTPs, dewatered biosolids are conveyed to the drying system where the temperature of the wet solids mass is raised and most of the water is removed via evaporation, resulting in a product with approximately 90 percent or higher total solids. This drying process retains the majority of the nutrient content of the biosolids.

For most systems, the high temperatures used in drying assure that the US EPA (40 CFR Part 503) and Ecology (WAC 173 308) time and temperature requirements for pathogen inactivation are met. Drying also meets the EPA vector attraction reduction standards by desiccating the wastewater solids to greater than 90 percent solids (or to greater than 75 percent solids if the solids have been previously stabilized). Although high temperatures are used in many drying systems, the temperatures are generally low enough to prevent oxidation (burning) of the organic matter. Thus, most of the organic matter is preserved in the dried material.

Drying systems may produce a variety of forms of dry material, including fine dust, flakes, small pellets, or larger fragments, depending on the type of drying system used, the characteristics of the solids processed, and the intended use of the final product.

Process Description

In the most general terms, drying is the use of heat to evaporate water from wastewater residual solids. The drying system, in addition to the dryer itself, generally consists of materials handling and storage equipment, heat generation and transfer equipment, air movement and distribution equipment, emissions control equipment, and ancillary systems. These equipment systems can take many forms, using different methods for heat transfer, including convection, conduction, and radiation heating. To some extent, multiple methods of heat transfer are used by individual systems, but they are generally categorized by their primary method of heat transfer – direct or indirect.

Direct Dryers

Systems that primarily use convection for heat transfer are often referred to as “direct” dryers. In direct heat dryers, hot air/gas flows through a process vessel and comes into direct contact with particles of wet solids. The contact between the hot air and cold wet cake allows the transfer of thermal energy, which causes an increase in wet cake temperature and evaporation of water. The hot air/gas can be produced by almost any source of heat, but most often is produced by a gas or oil-fired furnace. Examples of direct drying equipment are rotary drum dryers and belt dryers. A schematic diagram and photograph of a typical rotary drum drying system are included in Figures 8-2 and 8-3, respectively.

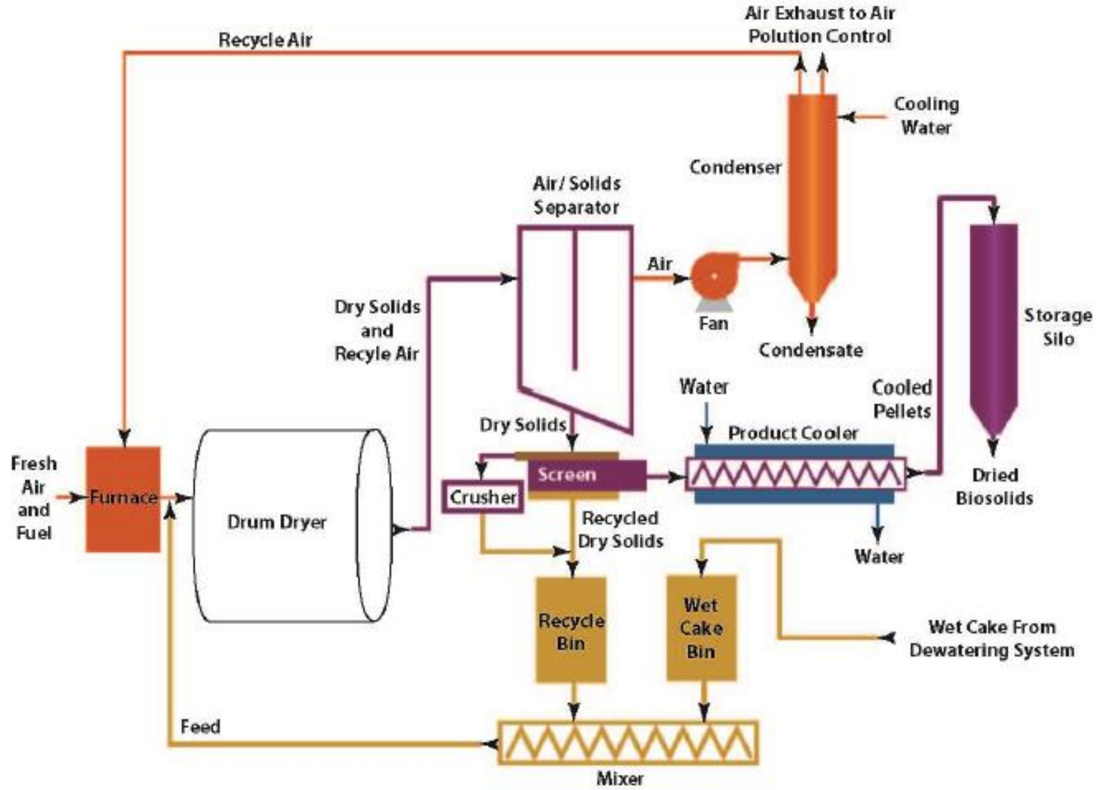


FIGURE 8-2

Typical Schematic of Direct Drying System (Drum)

In this type of system, the heat supply is via a fuel-burning furnace that exhausts directly to the dryer drum. The dried material is separated from the warm exhaust gas and is then screened and processed for either recycling back to the dryer or routed to storage silos. The exhaust air/gas is cooled and part of it is recycled back to the dryer. The remainder of the air/gas is treated in air pollution control equipment and then vented to the atmosphere. Recirculation of the dryer exhaust accomplishes three important functions. First, it increases the overall thermal efficiency of the dryer system, second, it minimizes the volume of exhaust gas requiring air pollution control (APC), and third, it provides a safety feature by limiting the oxygen concentration in the system, which reduces the risk for explosions. APC systems for drum dryers typically consist of additional particulate removal followed by regenerative thermal oxidation to destroy odors and volatile organic compounds (VOCs). Other methods of APC, such as biofilters, are often used with different drying systems. Present day direct drying systems typically recirculate 70 to 90 percent of the dryer exhaust, thereby greatly reducing the size of the APC equipment. Direct drying systems vary considerably depending upon the type of equipment used to process the wet and dried biosolids. Even rotary drum systems as shown in this figure vary considerably in general layout and the equipment used.

FIGURE 8-3

Drum Dryer System Diagram (by Andritz)

Another type of direct dryer that is seeing increased use in the U.S. and Europe is the belt dryer. This is typically a lower temperature system compared to a rotary drum system. The heat supply is usually a fuel-burning furnace, but in contrast to the rotary drum system, the system exchanges its heat to a thermal fluid, hot water or flue gas to air heat exchanger instead of the furnace exhausting directly into the dryer cabinet. The belt drying system distributes dewatered cake onto a slow-moving belt, allowing for high surface area exposure to the hot air. Belt drying systems can utilize multiple belts to help minimize the size of the dryer cabinet. High dryer air recirculation (>90 percent) and low vent rates are common. Due to the gentle handling on the slow-moving belts, dust generation within the dryer cabinet is low and the quantity of fines in the dried product should be low. Some belt drying technologies require dried product recycling to elevate the inlet solids composition to above the sticking point, while others inject dewatered sludge cake without additional recirculation equipment. The lower temperature belt drying system can more adequately utilize lower grade waste heat (in addition to high temperature waste heat). A photograph of a typical belt drying system is included in Figure 8-4.

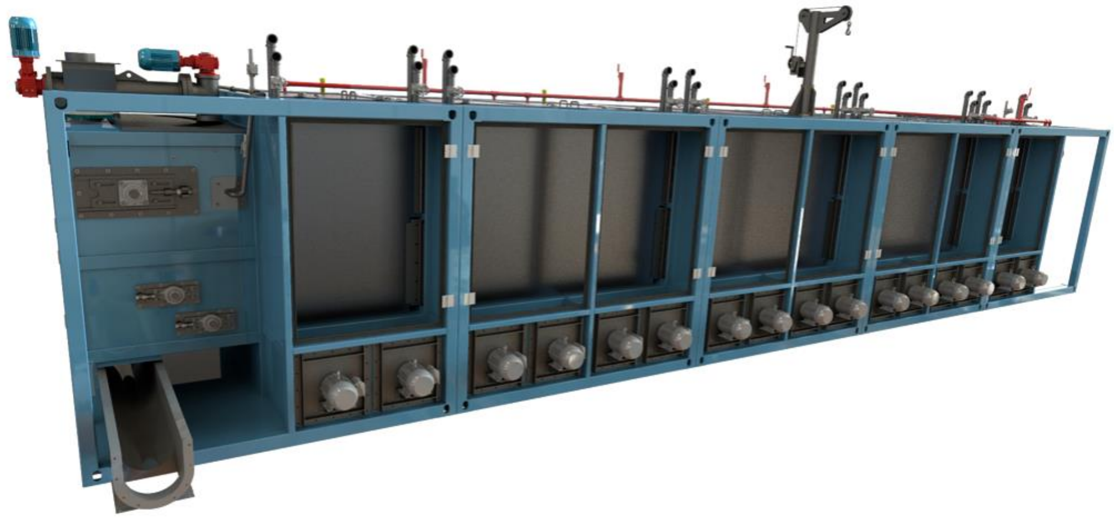


FIGURE 8-4

Belt Dryer System by Centrisys

Due to the relative simplicity of generating heat and heat recovery systems in these dryers, direct dryers generally have short start-up times. The short start-up times are advantageous for batch operation. Some direct dryers are also appropriate for continuous and nearly automated operation. This is particularly true for low-temperature belt dryers. However, staff at many WWTPs still prefer to have an operator present to ensure safe and reliable operation despite manufacturer claims of unattended operation capability.

Examples of belt drying systems in Washington State include the Andritz systems in Camas and Shelton.

Indirect Dryers

Systems that primarily use conduction for heat transfer are referred to as "indirect" dryers. The City's existing Fenton system is an indirect dryer. With indirect dryers, solid metal walls separate the wet cake from the heat transfer medium (such as steam, hot water, or oil). Thermal energy is transferred from the heat transfer medium into the metal wall and then from the metal wall into the cold cake. The solids temperature is elevated by contact with hot metal surfaces and the solids never come in direct contact with the primary heating medium. Some types of indirect dryers do not require recycle of dried material, simplifying the system. Indirect thermal drying equipment includes paddle dryers with varying configurations, vertical tray dryers, and an indirect-type of fluidized bed dryer. A schematic diagram of an indirect drying system is shown in Figure 8-5, and a photograph of a paddle dryer is shown in Figure 8-6.

An important benefit of indirect dryer technologies for this application is the typically smaller and more flexible footprint requirements compared to direct dryers. This allows smaller ancillary equipment (e.g., thermal fluid heater, nitrogen purge system) to be installed away from the dryer unit and in unclassified areas.

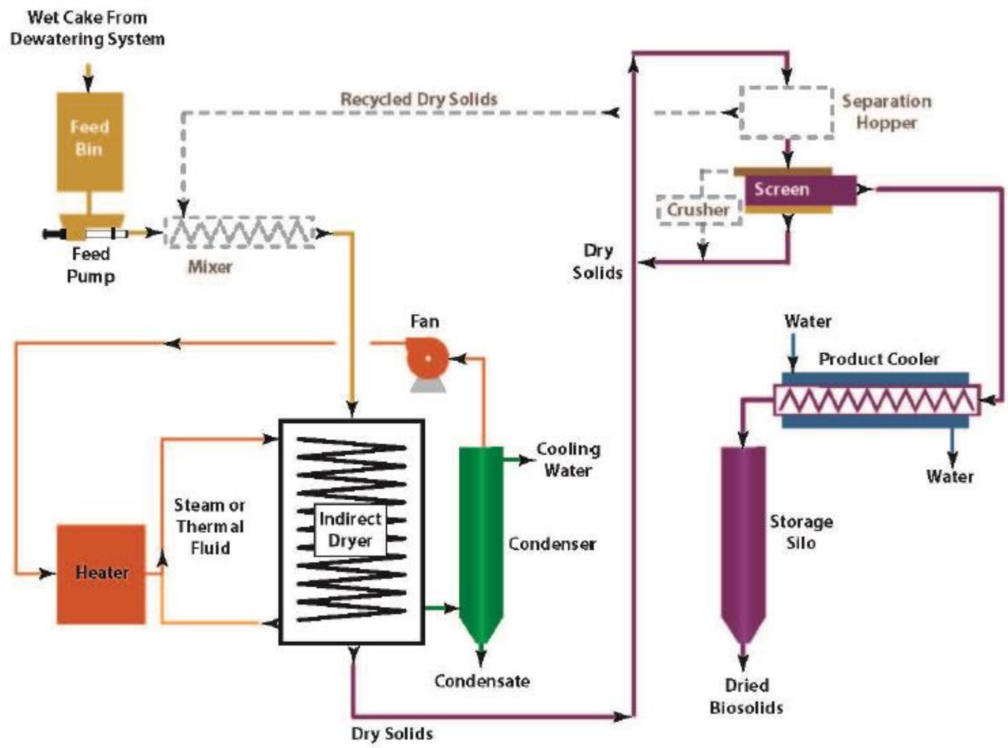


FIGURE 8-5

Typical Schematic of Direct Drying System (Drum)

In this type of system, the heat supply is via a fuel-burning furnace that exhausts to a heat exchanger to heat oil, which is recirculated through the dryer. Steam, air, water, or other heat transfer fluids are other media that can be used. The solids are mechanically moved through the dryer and pick up heat from direct contact with the hot surfaces. Following the dryer, the material handling equipment is similar to that used in the direct system. In this system, the dryer exhaust primarily consists of water vapor and a small quantity of air which inadvertently enters the dryer with the wet feed. The exhaust from the dryer is sent to a condenser where the water vapor is condensed and sent back to the WWTP and the small air flow (containing some non-condensable organics) is then treated using various air pollution control methods, depending on the system and supplier. For example, some systems send the exhaust to the furnace for use as combustion air (WEF MOP No. 8).

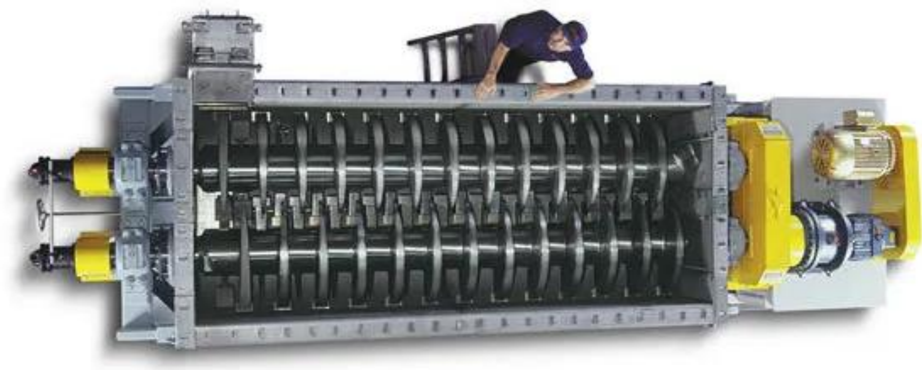


FIGURE 8-6

Paddle Dryer System (by Komline-Sanderson)

BIODRYERS

Biodrying is a convective drying process that relies on heat generated by metabolic reactions by the microbes in the biosolids. Unlike the conventional drying technologies described above, biodrying uses aeration to drive biological heat generation and evaporation. In this way, biodrying uses a pathogen-reduction strategy similar to composting. For this scale, biodrying is a new technology. One regional installation by Bioforcetech is operational at the Yakima Legends Casino while another is planned for the City of Yelm's wastewater treatment facility. Typical biodrying equipment is shown in Figure 8-7.

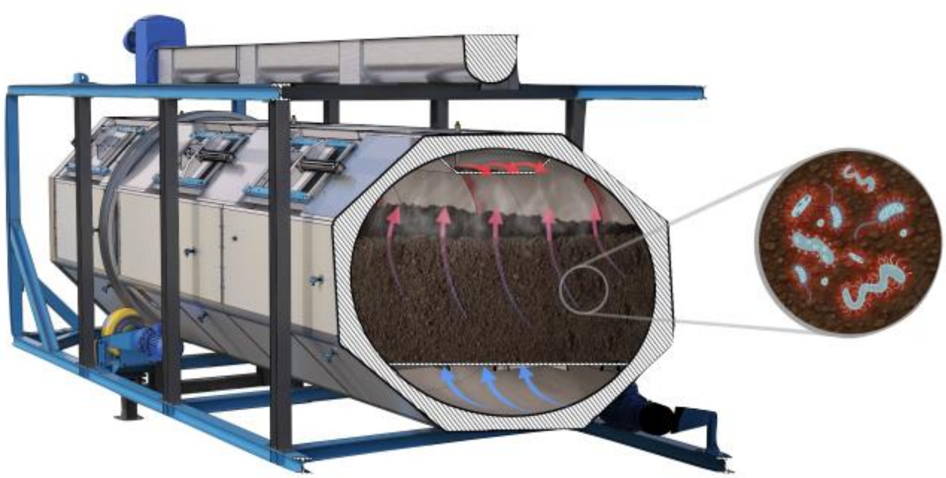


FIGURE 8-7

Biodryer Diagram (by Bioforcetech)

Biodrying is expected to have low energy requirements; the predominant energy-consuming component is the blower for each unit whereas conventional dryers require higher levels of energy to generate heat. In addition, biodrying is a low-temperature drying technology, which reduces the air handling requirements.

Notably, this biodryer manufacturer (Bioforcetech) offers pairing their biodryer with additional equipment to conduct pyrolysis of dried solids. Pyrolysis breaks down materials at relatively high temperatures in the absence of oxygen. Pyrolysis, gasification, and incineration are three technologies able to reduce per- and polyfluoroalkyl substances (PFAS) in waste solids. As some states are beginning to tests biosolids for PFAS, pyrolysis could be an attractive treatment option for PFAS reduction when compared to incineration, which has environmental and cost concerns. However, at this moment, there is uncertainty over any potential regulations on PFAS in municipal biosolids in Washington State. Additionally, both pyrolysis and gasification are emerging technologies in the field and have a limited history of installations in the country. Therefore, there is no clear guidance on what magnitude of PFAS reduction will be necessary to meet potential limits and what technologies would be most suitable. While it is necessary to monitor potential PFAs regulations, no additional treatment of dried biosolids is necessary at this time.

This equipment may only be operated as a batch process and is only offered in one unit size. As such, at least two units would be required to handle the quantity and frequency of dewatered solid loads.

Solar Drying

Solar drying systems rely on radiant energy from the sun. Dewatered solids are distributed into the greenhouse uniformly, either by automated mechanical means or by a manually operated tractor or truck. The sun's radiant energy passes through the greenhouse enclosure (walls and roof) to heat and evaporate moisture from the sludge. The greenhouse enclosure prevents rain from adding water to the sludge and allows for a semi-controlled greenhouse environment, including air convection to help accelerate evaporation and enclosure of odors that can be processed through an odor control system. Solar drying systems are sensitive to local weather conditions, including solar radiation (considering typical cloud cover), relative humidity, and temperature. There are some successful solar drying installations in Eastern Washington, including at the City of Wenatchee. However, western Washington is generally considered to not have adequate sunshine or temperatures for solar drying (WEF, 2014).

Dryer Project Implementation Considerations

Considerations for implementing biosolids dryer projects include sustainability, relative costs, and operational safety considerations.

Sustainability

Sustainability is the ability of a process to endure and remain an economically and environmentally sound means of wastewater solids management. The demands and pressures on drying can come from state and federal regulatory agencies, the general public, or from economic conditions. Regulatory agencies are continually scrutinizing pollutant and odorous air emissions from drying plants and imposing tighter emissions criteria on new facilities. Recent drying plants have shown that they can meet the strictest odor and pollutant emissions criteria. Federal and state statutes also regulate the quality of the product. Specifically, for a heat-dried product to be applied to land as an Exceptional Quality (EQ) product, it must meet stringent quality parameters including pathogen density reduction (Class A), vector attraction reduction, and low metals concentrations. Operating experience at drying facilities (including at La Center, as discussed above) has shown that these criteria can be confidently and consistently met.

In the past odors were one of the more problematic aspects of drying systems. However, present day design of drying plants has incorporated recirculation of dryer exhaust gas and the use of regenerative thermal oxidizers (and other techniques) to deodorize the final exhaust gas such that odorous emissions are no longer a significant impact. Dryers with high air recirculation rates or indirect dryers with low off-gas volumes can tie their low vent flow directly to the plant's existing odor control system. In general, the public now perceives drying as an environmentally acceptable technology for solids processing.

Presently, one of the major pressures on drying systems is the energy demand of the process, particularly the high fuel usage. Drying does use a considerable amount of fuel in comparison with other beneficial reuse technologies; however, the value and acceptability of the final product is much higher for a heat-dried product than a product from most alternate technologies. Thus, the energy demands and associated costs of drying have been acceptable because the municipality is assured that the final product can be safely used and, in many cases, will generate income. Furthermore, innovations have been developed in the last decade to improve the ability of some systems to use waste heat to reduce energy consumption. Drying should continue to be a highly sustainable solids processing technology in the future (WEF, 2014).

Relative Costs

Because of the large variety in types of drying systems, levels of processing, procurement methods, and general equipment differences, the capital cost of thermal drying systems varies a great deal. Factors that affect capital cost include the type of system selected, existing infrastructure, such as buildings and utilities, conveyance needs for moving dewatered solids to the process, and finished product storage requirements. Due to the implementation of additional processing, safety and monitoring improvements, capital costs are significantly higher for new systems compared to the cost of the City's existing Fenton system.

The operating and maintenance (O&M) costs of thermal drying systems are also dependent on the type of system selected and the energy source. Energy recovery can make O&M costs very competitive with O&M costs for other methods of solids processing, especially other systems that create Class A stabilized products. The level of mechanization and automation used in the system will also have a significant effect on labor and maintenance costs.

Although drying systems may have higher capital costs than other processes, the substantial reduction in volume of material to be transported offsite, the flexibility of outlets available, and the value of the product can help make these systems cost-competitive with other solids processing systems (WEF, 2014).

Operational Safety Considerations

In the past there were serious concerns with the safety aspects of wastewater solids thermal drying plants. These safety concerns included the following:

- Potential for fires in the dryer
- Potential for dust explosions in the process components containing dried material
- Potential for fires in the product storage silos from auto-oxidation of the dried material

As the design of drying plants has evolved, engineers and system suppliers have learned how to design safer drying facilities. The potential for fires in direct dryers has been greatly reduced by maintaining an oxygen-deficient atmosphere in the dryer. This is done by recirculating the dryer exhaust gas and limiting the amount of infiltration air such that the oxygen level in the dryer is held less 9 percent. In addition, dryers are equipped with quench sprays to extinguish a fire or burning embers in the dryer. Quench sprays are usually automatically activated based on a rise in the dryer exhaust gas temperature indicating that combustion is occurring in the dryer.

The potential for a dust explosion in many of the system components (dryer, solids separator vessel, recirculation duct) has been eliminated by maintaining an oxygen deficient atmosphere in these components. In some plants select equipment is provided with nitrogen blanketing to prevent explosions.

Similarly, the potential for fires in the product storage silo has been addressed by using inert gas (nitrogen) blanketing systems to maintain an oxygen deficient atmosphere in the product silo. In addition, cooling of the product prior to storage has proven to be an effective means of retarding auto-oxidation of the material and preventing fires. Storage silos are typically monitored by thermal sensors hung within the silos to detect any rise in temperature. Another monitoring technique is to use carbon monoxide monitors, which can detect the initiation of any combustion reactions. Thus, through experience and careful engineering the potential safety concerns with thermal drying systems have been satisfactorily addressed.

Advantages and Disadvantages of Drying

Drying technologies offer relative advantages and disadvantages as compared to other solids processing technologies. Some of these are listed below:

Advantages

Dried material typically meets the requirements of the US EPA Part 503 and WAC 173-308 standards for vector and pathogen control and the product is typically classified as Exceptional Quality and Class A with respect to pathogen density levels, but this is dependent on the type of system.

- Drying is a well-proven existing technology.
- Odors arising from the process can be contained and controlled.
- There is a wide range of outlet options for the dried material. Dried material can be used as a fertilizer, fertilizer supplement, soil conditioner, or biofuel.
- The heat-dried product is easily handled, conveyed and stored. The material can be delivered to consumers in bulk, bags, or another container.
- Drying reduces the volume and mass of wet cake produced at the plant. This results in reduced transportation costs for beneficial use.
- The product can be sold or given away which can partially offset the high costs associated with operation of the drying facility.
- Drying has a higher potential for public acceptance than many other processes.
- Drying has reduced regulatory record keeping and reporting requirements, if application to land is desired.

Disadvantages

- Safety concerns of drying include the explosive potential of the dust and the potential for product overheating and fires. Current design measures significantly reduce these safety hazards.

- The complexity of some drying systems requires a qualified operating staff. Maintenance requirements are typically high for direct and indirect systems.

- Air emissions are produced at any drying facility. Air permitting and air pollution control may be required.

- Capital and O&M costs of a drying facility are relatively high, typically higher than other solids processing alternatives (land application of digested biosolids, alkaline stabilization, etc.).

Drying of wastewater solids has proven to be a safe, reliable, environmentally acceptable, cost effective, and sustainable processing technology that can produce a high quality biosolids product suitable for use as a fertilizer or biofuel. Furthermore, in comparison with other solids processing alternatives, drying is one of the most environmentally and socially acceptable means of achieving beneficial reuse of wastewater solids.

TECHNOLOGY ALTERNATIVE 2 – ALKALINE STABILIZATION

Alkaline stabilization is the process of adding alkaline chemicals to biomass to inactivate harmful pathogens. The added lime or alkaline chemicals raise the pH levels of the biomass, creating unfavorable conditions for the growth of organisms. The types of chemicals traditionally used are hydrated lime and quicklime. Class A biosolids can be achieved through alkaline stabilization by maintaining the pH at greater than 12 for at least 72 hours, maintaining the temperature above 125˚F for at least 12 hours within that period, and drying the solids to greater than 50 percent total solids.

This process does not result in any reduction in the mass of the solids. In fact, the addition of a significant amount of lime increases the mass of the resulting biosolids. The stabilized product can be used for a variety of end uses such as landscaping, agriculture, mine reclamation, and landfill cover. Depending on the soil to which the product is applied, the final product can be more favorable for some vegetation (particularly grasses which prefer a higher pH) as it can increase soil pH due to the added alkaline chemicals. However, extensive odor control may be required to treat ammonia and other off-gases.

Leading systems include the RDP Process and the “Class A FKC Process” Lime Stabilization followed by Simultaneous Pasteurization and Dewatering (FKC Co., Port Angeles, WA). The Cities of Sequim and Forks, and the Willapa Regional WWTP in Raymond, WA, use the FKC process, while the City of Centralia uses the RDP process. Staff from these facilities indicate that odors can be a significant issue for operations, as well as for the finished product.

TECHNOLOGY ALTERNATIVE 3 – COMPOSTING

Raw solids composting is a biological process to decompose organic material to produce humus. Active composting accelerates the natural process by controlling the carbon-to-nitrogen ratio, temperature, moisture content, and oxygen supply. After active composting, the material is cured and stored until ready for transport. The resulting product is rich in nutrients and suitable for promoting soil tilth and plant growth. Composting is typically applied to dewatered solids that have already been digested, but can be used on unstabilized (i.e., raw) solids as well. Solids are typically dewatered to between 14 and 30 percent total solids before mixing with a bulking agent such as wood chips, sawdust, or yard waste. Retention times of 10 to 30 days are required to sufficiently stabilize the material; additional time is often needed to air-dry the solids to allow for beneficial use.

There are three different compost methods typically available for wastewater solids: (1) aerated static pile; (2) windrow; and (3) in-vessel. All methods can produce Class A biosolids; the decomposition of organics matter is exothermic and if controlled properly can lead to the appropriate level of pathogen reduction. The major differences between the different methods is how the solids are aerated (e.g., either through forced aeration or by mixing).

Biosolids can be composted with waste or debris material to make excellent mulches and topsoils for horticultural and landscaping purposes. Some waste or debris material includes sawdust, wood chips, yard clippings, storm debris, food waste, manure or crop residues, or food processing wastes. While these materials have traditionally been viewed as waste, they can play a valuable role as soil amendments in urban and agricultural settings. Many professional landscapers and master gardeners use composted biosolids for landscaping new homes and businesses. Home gardeners also find composted biosolids to be an excellent addition to planting beds and gardens.

Similar to the process requirements described for the composting of unstabilized solids, composted biosolids must meet certain criteria, which include meeting pathogen reduction limits, complying with required sampling and analysis protocols, maintaining compost temperature and retention time records, and product labeling requirements. Compost products provide nutrients and organic matter and sequester carbon, thereby conserving resources, restoring soils, and combating climate change. Composting is a common method used to produce Class A biosolids, and many large and small communities in the Pacific Northwest (including the Cities of Westport, Port Townsend, LaConner, and Granite Falls in Washington State) have active composting operations.

Demand for compost often exceeds the available supply. Private composting companies in some areas receive biosolids from multiple communities and market their products to landscapers and home gardeners. Local delivery programs return a portion of the composted material to the communities from which they originated. However, private biosolids composters are very limited in the state of Washington.

Soil Blending

While Class B biosolids may be composted to produce Class A material with certain technologies, producing manufactured soils is a specialized class of biosolids product development. To be suitable for use by the public, the feedstock for this production is Class A biosolids cake. Class A biosolids are blended with a mixture of sawdust/bark and sand to produce a product that can be publicly distributed in bag or bulk form. The TAGRO product produced by the City of Tacoma is a manufactured compost soil comprising two parts Class A dewatered cake, two parts sawdust, and one part sand. Manufactured soils could potentially be custom-tailored to meet certain landscaping needs. As examples, TAGRO products include potting soil and a green roof blend in addition to the classic topsoil blend.

TECHNOLOGY ALTERNATIVE 4 – GASIFICATION

Gasification is a thermal oxidation process that oxidizes dried solids under high-temperature sub-stochiometric conditions. The resulting products are an inert ash (i.e., biochar) and a mixture of carbon monoxide (CO), methane, hydrogen, and other volatile components (i.e., syngas). To sustain the process, a portion of the syngas is used to dry the feed solids (required); the remaining syngas requires significant treatment before being used as a renewable fuel source. Gasification has a long history of being used with fossil fuels to convert coal to a gaseous fuel, but its use to stabilize wastewater solids is relatively new. Whereas several gasification projects have been developed in North America, most have been mothballed due to poor economics and operational issues. Only one small operating facility has been identified as currently operating in a wastewater treatment plant in North America. (However, additional facilities are in the planning or design stage of development, since it is a technology that can potentially remove PFAS through combustion, although higher temperatures may be necessary to remove all the compounds.) Gasification on biosolids requires an additional feedstock to properly process solids, such as wood waste or used tires.

TECHNOLOGY ALTERNATIVE 5 – PYROLYSIS

Pyrolysis is a thermal conversion process similar to gasification, but it is accomplished at a lower temperature and does not require the presence of oxygen. Similar to gasification, drying is required upstream of the pyrolysis system to sustain the process and produce a usable final product. There are varying by-products of pyrolysis depending on different heating rates. The products of the pyrolysis process are biochar and bio-oil (a condensed liquid fuel). The liquid nature of the bio-oil makes it more useful as a fuel, but this end product requires further processing to be used beneficially. Of the bio-oil produced, typically 40 percent can be recovered in this manner. In addition, a significant amount of energy is required to dry the feed solids prior to the pyrolysis process, reducing the overall net energy produced. Pyrolysis is a process widely used in the chemical industry that historically has not been successfully applied to manage wastewater solids.

However, since it is a technology that can remove PFAS through combustion (although potentially requiring higher temperatures), there is increasing interest in this technology with several facilities in planning and design around the country. The City of Edmonds is planning to replace its incinerator with an integrated drying, pyrolysis and gasification process; the preliminary capital project cost estimate is $26 million.

TECHNOLOGY ALTERNATIVE 6 – INCINERATION

Sewage sludge incineration (full combustion of sludge with excess air) is a well-established technology that yields ash for disposal. The technology has largely fallen out of favor due the lack of resource recovery, permitting/siting challenges, and recent stringent regulations for pollutant emissions. No new sludge incinerators have been constructed in Washington State in the last 20 years; it is more popular in the northeast and parts of Europe due to population density, lack of biosolids land application sites, and more recently, concerns over microconstituents in non-incinerated sludge. Sewage sludge incineration is currently employed by the Cities of Lynnwood, Edmonds, Vancouver and Bellingham in Washington State, but all of those municipalities are looking to replace their incinerators with other technologies in the near future (Bellingham with temperature phased anaerobic digestion and soil blending, at an estimated total capital cost exceeding $200 million, and Edmonds with the aforementioned integrated drying, pyrolysis and gasification process.

EVALUATION

Table 8-9 summarizes an evaluation of the available technologies. (10 is the highest rating, while 1 is the lowest rating.) The evaluation includes the following criteria:

1. **Regulatory Approval –** The expected ease of permitting and regulatory approval from Ecology, EPA, and local authorities including biosolids and air permitting.

2. **Proven as Reliable Treatment Technology –** “Track record” of technology including years of successful, reliable operation treating municipal biosolids.

3. **Reliable End Use Options –** Demonstrated level of success utilizing the product for beneficial purposes.

4. **Safe Working Environment –** Demonstrated level of safety for plant staff and neighbors, including protection from explosions, fires, and odors. Availability of safety measures including system automation, alarms, monitoring, shut down and fire suppression systems.

5. **Environmental Benefits –** Sustainability of technology reflecting the impact of biosolids pollutants, greenhouse gas emissions, carbon sequestration, fertilizer benefits, and gas emissions.

6. **Footprint –** Relative area needed at the WWTP site for the technology and all associated technologies. Ability of equipment to fit within the footprint of existing structures.

7. **Ease of Operation –** Operation and maintenance needs for the technology, factoring both anticipated labor hours and operational complexity.

8. **Flexibility –** Ability of the technology to be expanded or modified for changing regulations or end uses.

9. **Capital Costs –** Anticipated relative initial total project capital cost.

10. **Life Cycle Costs –** Anticipated relative 20-year life cycle costs, including capital costs and net present value of operation and maintenance costs.

TABLE 8-9

Comparison of Class A Treatment Technologies

Technology Alternative	Biosolids Drying	Alkaline Stabilization	Composting	Gasification	Pyrolysis	Incineration
Regulatory Approval	10	10	10	4	4	1
Proven as Successful Technology	10	8	8	2	2	8
Reliable End Use Options	10	3	10	5	5	1
Safe Working Environment	7	5	10	3	3	5
Environmental Benefits	6	5	7	5	5	1
Footprint	8	8	1	7	7	6
Ease of Operation	6	7	5	2	2	1
Flexibility	8	4	6	6	6	1
Capital Costs	6	7	7	3	3	1
Life Cycle Costs	6	8	5	4	4	5
Total	**77**	**65**	**69**	**41**	**41**	**30**

(1) 10 is the highest rating, while 1 is the lowest rating.

For Class A treatment technologies, thermal drying is recommended. The other technologies have the following major issues, reflected in Table 8-9:

- Alkaline stabilization: Requires handling of additional chemicals (lime, an alkaline, toxic chemical, and would result in significant additional odors for the treatment process and the final product.

- Composting: Requires significant additional land footprint and labor, import of additional organic matter, and has odor emissions

- Gasification: Is not well-established technology in the US with sufficient similar installations, requires additional feedstock to maintain consistent operation, and does not recover the nutrients in the biosolids.

- Pyrolysis: Is not well-established technology in the US with sufficient similar installations, requires additional feedstock to maintain consistent operation, and does not recover the nutrients in the biosolids. Does have the capability of removing PFAS.

- Incineration: Does not yield a useful production. Would be very difficult to permit, both for Ecology approval and air permitting. Is not consistent with Washington State resource recovery policies, so would not receive funding or support from the State.

Besides the alternative of continuing to produce the Class A biosolids through drying technology, the City would also like to explore the potential alternative of Class B biosolids hauling by a third party for treatment/disposal.

PREFERRED DRYER ALTERNATIVE

The indirect, continuous flow, paddle dryers are generally used for small to medium-sized wastewater treatment plants and typically have lower capital costs and a smaller footprint relative to direct dryers.

An indirect dryer applies heat to an oil, water, or steam medium that is passed in a closed loop through discs or paddles in a sludge heater. Heating the paddles requires a boiler to support the closed loop for the heated medium. At the scale of biosolids production at the La Center WWTP, it would be reasonable to run the system continuously while it is periodically attended by an operator. Therefore, this analysis will assume the use of an indirect paddle or drum dryer system. As for the screw press dewatering unit, the dryer will be assumed to operate approximately half-time. This will result in a dryer with an evaporative capacity of approximately 775 lbs per hour of water.

The dryer (assumed, for purposes of this discussion, to be a Komline-Sanderson Model 6W-600 paddle dryer) would provide a 380°F heated environment, drying the dewatered solids to 90+ percent to meet Class A requirements.

The dryer would be installed where the existing dryer is located. The heating system, however, will have to be located in a separate room, which may require a small building addition.

A paddle or drum dryer system typically includes the following components:

- Wet cake hopper;
- Drum/Paddle drying unit;
- Condenser;
- Final product cooler;
- Control panel;
- Class A Biosolids screw conveyor.

The total project capital cost for producing Class A Solids utilizing a Thermal Drying System is $10.67 M, including construction, sales tax, engineering, and administration.

Operation and maintenance costs for the Class A biosolids treatment alternative include power, natural gas, repair/maintenance, and labor. The estimated power requirements for 2043 conditions consist of an annual electricity consumption of about 278,300 kWh and annual natural gas consumption of about 4,081,000 cf. The estimated capital cost, annual operational and maintenance cost, as well as the resultant 20-year net present worth have been calculated based on the system described above. A Komline-Sanderson dryer was included based on a longer track record and performance history. These costs are presented in Table 8-10. The estimated life span of a new dryer is 20 years.

TABLE 8-10

Alternative 1 – Class A Solids Thermal Drying System – Cost Estimates

Item	Estimated Value
Thermal Drying System Preliminary Capital Cost	$10,673,000
Thermal Drying System Preliminary Annual O&M Cost(1)	$255,200
Thermal Drying System 20-Year Net Present Worth(2)	$13,722,000

(1) All costs in 2023 dollars applied to 2043 conditions.
(2) Assumed 5.5 percent discount rate.

Table 8-11 shows the anticipated annual average amounts of dried solids produced by the sludge dryer based on a 95 percent solids content in the dried sludge.

TABLE 8-11

Estimated Annual Average Amounts of Dried Solids Produced by the Sludge Dryer

Year	Annual Average Dried Solids Produced (lbs/day)
2023	591
2033	901
2043	1,551

CLASS B BIOSOLIDS HAULING

Class B biosolids production would consist of hauling of dewatered aerobically digested sludge. This alternative would require the installation of the aerobic digester expansion and the dewatering unit. These items are not included in the cost comparison with Class A drying, as they would be required to be installed for both alternatives. The installation of an elevated dewatered sludge storage hopper would have to be installed, however, to facilitate biosolids loading into trucks. This is estimated to have a project cost of about $2,913,000, including construction, sales tax, engineering, and administration costs. The on-site facilities at the WWTP would include an elevated storage hopper, from which biosolids can be loaded into trucks, and a series of screw conveyors transporting the dewatered biosolids from the dewatering unit to the hopper. Regarding operation and maintenance costs, the production of Class B biosolids would require the costs of contracted biosolids hauling, which a Class A alternative would not. The cost of hauling Class B biosolids is estimated to be $72 per wet ton if contracted with Fire Mountain Farm, Inc., a company specialized in biosolids transportation and land application, located about 75 miles away from the City of La Center. It is estimated that the annual biosolids quantity to be hauled is 1,681 wet tons in 2043.

Operation and maintenance costs for the Class B biosolids hauling alternative include power, repair/maintenance, and labor. The estimated power requirements for 2043 conditions consist of an annual electricity consumption of about 32,700 kWh for operating screw conveyors. The estimated capital cost, annual operational and maintenance cost, as well as the resultant 20-year net present worth have been calculated based on the system described above. The estimated capital cost, annual operational and maintenance cost, as well as the resultant 20-year net present worth have been calculated based on the above. These costs are presented in Table 8-12 below.

TABLE 8-12

Alternative 2 – Class B Contracted Hauling – Cost Estimates

Item	Estimated Value
Capital Cost	$2,913,000
Contracted Hauling Annual O&M Cost[1]	$142,200
Contracted Hauling 20-Year Net Present Worth[2]	$4,612,000

(1) All costs in 2023 dollars applied to 2043 conditions.
(2) Assumed 5.5 percent discount rate.

RECOMMENDED ALTERNATIVE

Estimates of the costs associated with the alternatives show that production of Class B biosolids would result in lower costs by a significant margin. For the Class B biosolids alternative, there are some additional capital costs associated with sludge loading improvements. As a result, the 20-year net present worth ($4,612,000) is significantly lower than the preferred Class A alternative ($13,722,000), due to the much higher capital cost for the replacement dryer, which is reflected in the 20-year net present worth comparison. However, there are additional non-monetary factors that might affect a decision on whether to continue with Class A biosolids or pursue Class B biosolids production.

The production of Class A biosolids would provide a benefit to the community. Thermally dried biosolids are considered desirable and welcomed by the residential, development, and agricultural communities. In addition, the reduction in hauling would provide some benefit to the community directly neighboring the WWTP due to the lower traffic and noise from trucks. However, the costs associated with Class A biosolids production and higher sewer rates may diminish the net benefit to the community.

The production of Class A biosolids requires significant labor costs from WWTP staff, including operator attention, maintenance and periodic repair.

Another qualitative factor in this analysis is the long-term reliability of Class B biosolids application. Currently, a limited number of sites in this region will accept Class B biosolids for permitted land application. If fewer sites are able accept Class B biosolids for land application or further treatment, contracted hauling costs would likely increase, particularly if permitted sites are only located in Eastern Washington. This may increase the hauling distance from 75 miles to 325 miles, assuming Boulder Park is the destination. This could increase the hauling costs by $40 per wet ton, or more. Also, the reliability of the trucking may be compromised during winter driving conditions, as the hauling route will be through at least one mountain pass. Thus, significant benefits would result from the continued use of Class A biosolids production, which would be relatively independent from the uncertainties of contracted hauling costs and reliability.

The above discussion is quantified through a decision matrix in Table 8-13, which shows that the combined benefits of continued Class A biosolids treatment slightly outweigh the benefits of Class B biosolids hauling. (10 is the highest rating, while 1 is the lowest rating.) However, because of the significant cost advantage of Class B biosolids hauling based on present-day conditions, it would be difficult to justify the implementation of improvements required for future production of Class A biosolids at the present time. Thus, it is recommended to continue Class A biosolids treatment with the existing dryer equipment until the time the capacity of this equipment is exceeded or until this equipment becomes inoperable. In the meantime, it is recommended that aerobic digestion and sludge dewatering facilities be constructed. Upon the discontinuation of the dryer operation, it is recommended that the City of La Center initiate contract trucking of Class B biosolids to a beneficial utilization site.

TABLE 8-13

Biosolids Classification Decision Matrix

	Class A Thermal Drying	Class B Hauling
Regulatory Approval	10	8
Proven as Successful Technology	10	6
Reliable End Use Options	10	1
Safe Working Environment	5	7
Environmental Benefits	9	3
Footprint	2	8
Ease of Operation	6	9
Flexibility	8	2
Capital Costs	3	10
Life Cycle Costs	3	10
Total	**66**	**64**

If the Class B hauling distances change in the future, it is recommended that the feasibility of Class A biosolids drying is re-evaluated.

The PFAS issue has upended biosolids planning in many parts of the country. Research, regulation, public perception, and legal issues regarding PFAS are fast-moving. If the EPA or the State are going to promulgate PFAS standards for biosolids, it will likely be within the next 2 years, given all the attention this issue has received. The City should incorporate actual costs for any new relevant PFAS regulations, into biosolids management decision-making at that time.

PROPOSED LAYOUT

A proposed layout of the proposed biosolids treatment improvements is shown on Figure 8-8. Figure 8-9 shous a layout of the new screw press dewatering unit and dryer in the existing Biosolids Handling Building. It appears that the proposed dewatering screw press and paddle dryer can be located within the footprint of the existing Biosolids Handling Building without any major building modifications.

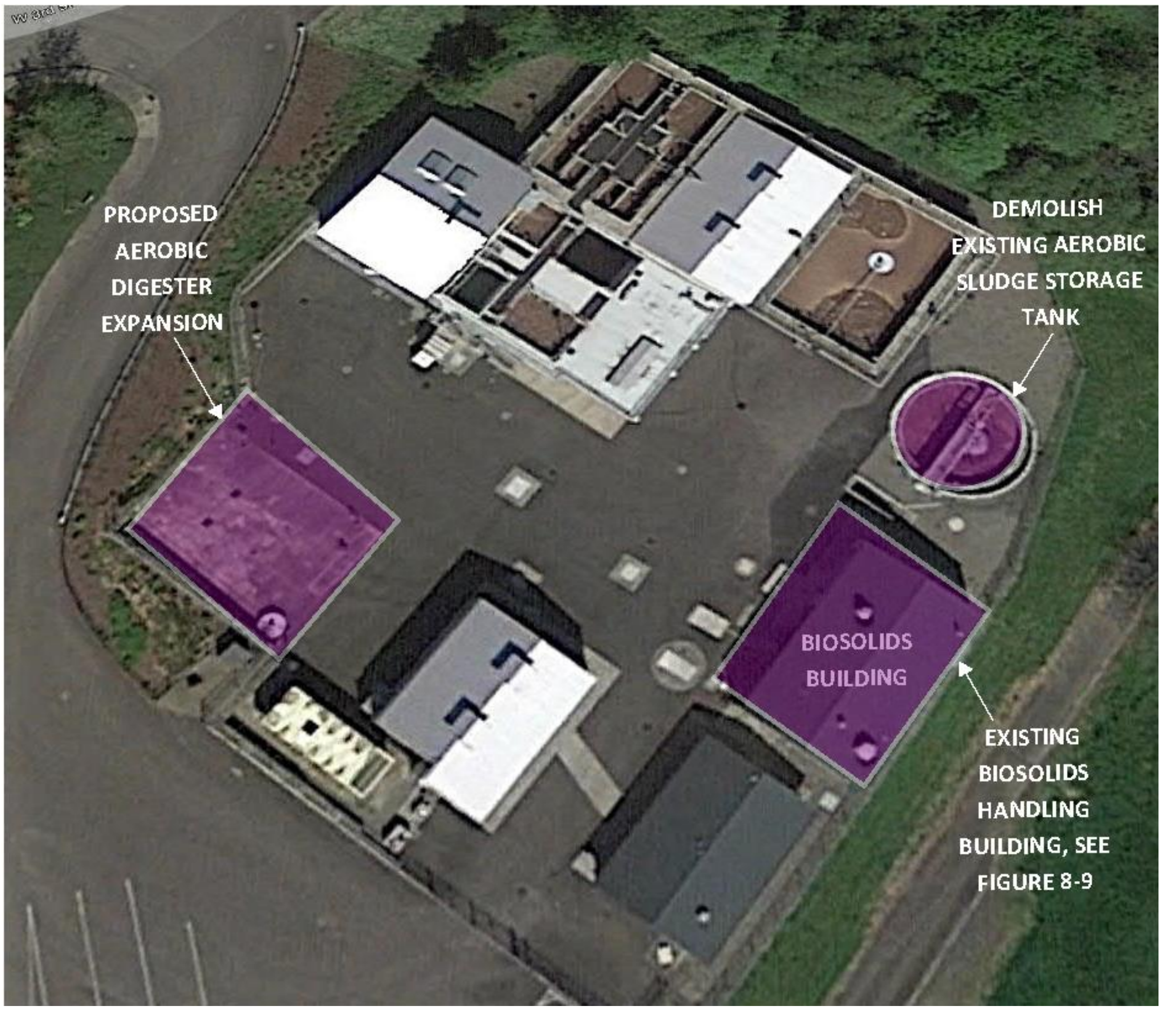
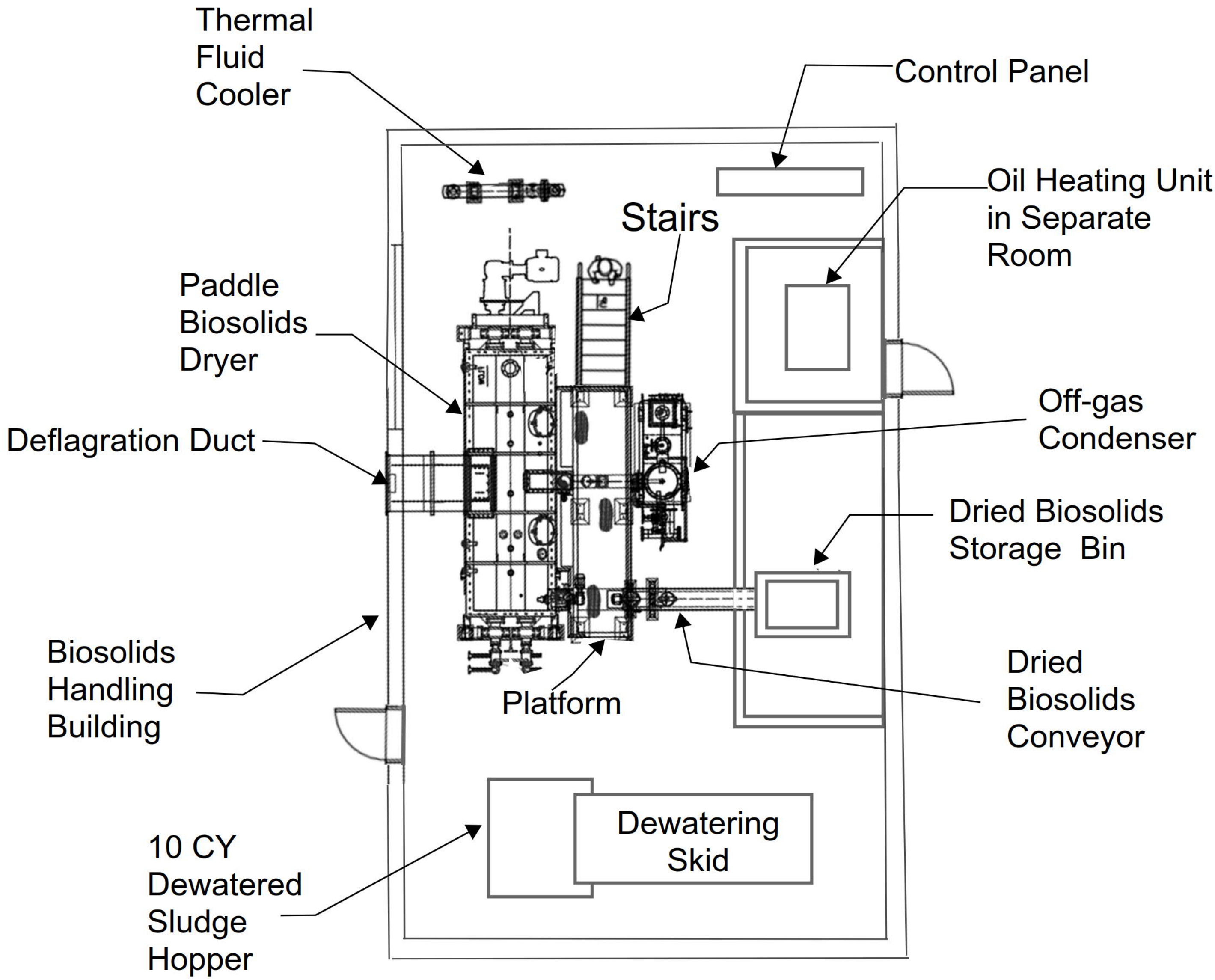


FIGURE 8-8

Proposed Biosolids Treatment Improvements



CITY OF LA CENTER

GENERAL SEWER PLAN UPDATE

FIGURE 8-9
NEW DEWATERING AND DRYER FACILITIES IN EXISTING BIOSOLIDS HANDLING BUILDING

Gray & Osborne, Inc.
CONSULTING ENGINEERS

REFERENCES

WEF, 2014.Water Environment Federation, Drying of Wastewater Solids Fact Sheet, 2014

Ecology, 2010. Control of Toxic Chemicals in Puget Sound Summary Technical Report for Phase 3: Loadings from POTW Discharge of Treated Wastewater, Washington State Department of Ecology

CHAPTER 9

CAPITAL IMPROVEMENT PLAN

INTRODUCTION

This chapter summarizes the City of La Center Wastewater System Capital Improvement Plan (CIP) including recommended capital improvements for the wastewater collection system and treatment plant outlined in the previous chapters.

CAPITAL IMPROVEMENT PLAN

Wastewater capital improvements have been identified and prioritized based on the collection and plant capacity evaluations, regulatory requirements, condition assessments, operation and maintenance considerations, system benefits, and costs. For all proposed projects identified in this chapter, detailed preliminary project cost estimates are presented in Appendix K. Detailed discussion of the proposed improvement projects is included in Chapters 6 (for the collection system), 7 and 8 (for the WWTP).

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 those that are evaluated 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 re-evaluated to consider the most recent planning efforts as the design of 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. These estimated capital costs are total project costs (in January 2024 dollars) inclusive of contingency (30 percent), sales tax (8.5 percent), engineering (13 percent), construction administration (12 percent), and legal, City administration, and permitting (5 percent). Full cost estimates are included in Appendix K. It should be noted that this analysis and costs presented throughout the Plan are based on planning level (Class 4 AACE) cost estimates. Per the AACE, for Class 4 estimates, “variability of -15 percent to -30 percent on the low side, and +20 percent to +50 percent on the high side is considered normal.”

Figures 9-1 and 9-2 show the locations of the projects in the CIP.

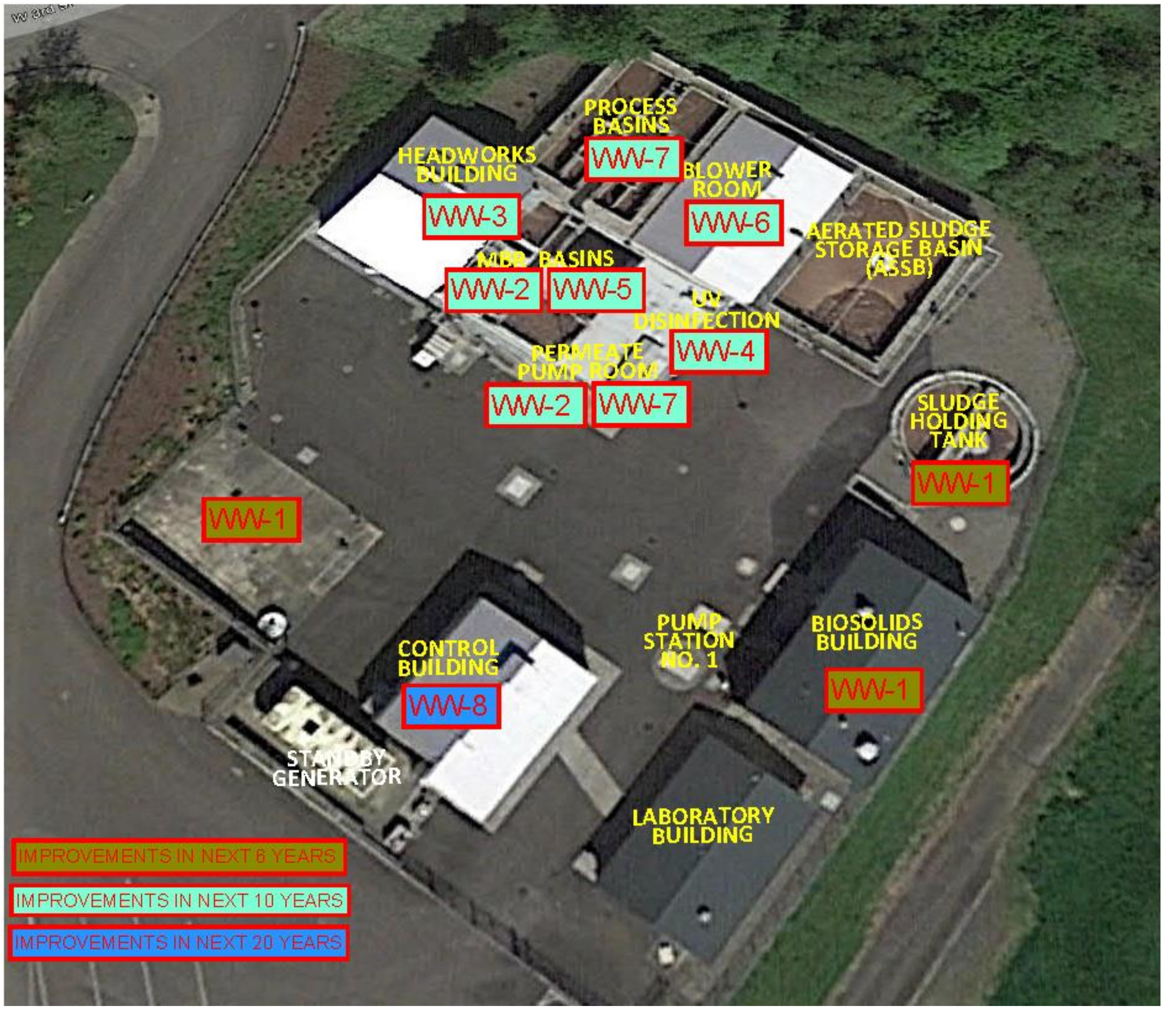
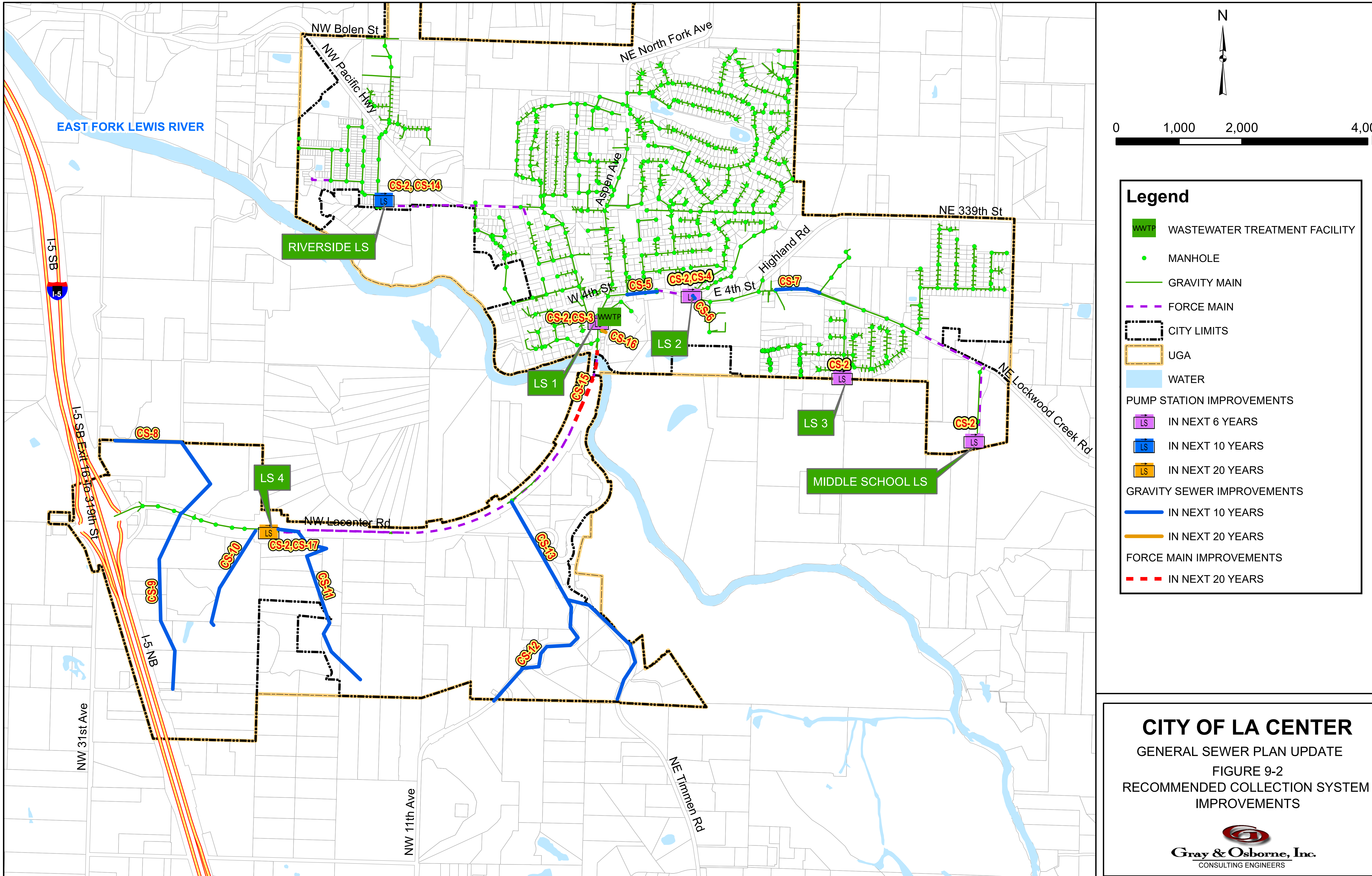


FIGURE 9-1

Wastewater Treatment Plant CIP



CITY OF LA CENTER

GENERAL SEWER PLAN UPDATE

FIGURE 9-2

RECOMMENDED COLLECTION SYSTEM IMPROVEMENTS

Gray & Osborne, Inc.
CONSULTING ENGINEERS

TABLE 9-1

Collection System – 6-Year Capital Improvement Plan

Project No.	Project Name	Components	Estimated Project Cost	Estimated Year of Completion
6-Year CIP				
CS-1	Collection System Cleaning	Jetter for collection system cleaning	$50,000	2026
CS-2	Flow Meter Installation	Installation of flow meters at existing Lift Stations (6)	$535,000	2026
CS-3	Lift Station#1 Upgrade	Upsize pumps to 2,100 gpm to meet 2043 capacity requirement. Replace mechanical, electrical and I&C; Rehabilitate wet well concrete surface; Add bypass piping connection	$2,301,000	2028
CS-4	Lift Station#2 Upgrade	Upsize pumps to 550 gpm to meet 2043 capacity requirement. Replace mechanical, electrical and I&C; Rehabilitate wet well concrete surface; Add bypass piping connection	$1,287,000	2028
Total 6-Year CIP		**$4,174,000**		
CIP for Years 7 to 20				
CS-5	Existing Sewer Main (8-inch) Upsize	Upsize 520 lf sewer main to 10-Inch from MH B-6 to MH B-1 on East 4th Street, between East Edgewood Avenue and East Birth Avenue	$521,000	2033
CS-6	Existing Sewer Main (8-inch) Upsize	Upsize 60 lf sewer main to 10-Inch from MH C-1 to LS 2 near intersection of East 4th Street and East Stonecreek Drive	$97,000	2033
CS-7	Existing Sewer Main (8-inch) Upsize	Upsize 700 lf sewer main to 10-Inch from MH C-34 to MH C-41 on East 4th Street, west of NE Highland Avenue	$662,000	2033
CS-8	New Sewer Main	Construct new 2,450 lf of 8-Inch sewer main near northeast quadrant of the I-5 interchange, between NW La Center Road and northern city limits, on future street	$1,979,000	2033
CS-9	New Sewer Main	Construct new 2,780 lf of 8-Inch sewer main near southeast quadrant of the I-5 interchange, between NW La Center Road and southern city limits, on future street	$2,233,000	2033

TABLE 9-1 – (continued)

Collection System – 6-Year Capital Improvement Plan

Project No.	Project Name	Components	Estimated Project Cost	Estimated Year of Completion
CS-10	New Sewer Main	Construct new 1,890 lf of 8-Inch sewer main near southeast quadrant of the I-5 interchange, extend from NW La Center Road to the south of the city limits	$1,584,000	2033
CS-11	New Sewer Main	Construct new 3,300 lf of 8-Inch sewer main along McCormick Creek, between NW La Center Road and southern city limits	$2,593,000	2033
CS-12	New Sewer Main	Construct new 2,450 lf of 8-Inch sewer main along Spencer Road, between NW Timmen Road and southern city limits	$1,979,000	2033
CS-13	New Sewer Main	Construct new 3,940 lf of 8-Inch sewer main along NW Timmen Road, between NW La Center Rd and southern City limits	$3,038,000	2033
CS-14	Lift Station #6 Upgrade	Upsize pumps to 200 gpm to meet 2043 capacity requirement. Replace mechanical, electrical and I&C; Rehabilitate wetwell concrete surface; Add bypass piping connection	$1,075,000	2033
CS-15	New Forcemain	Construct new 1,020 lf of 6-Inch LS#4 FM along NW La Center Road, between NW Pollock Road and West 1st Street	$935,000	2043
CS-16	Existing Sewer Main (8-inch) Upsize	Upsize 40 lf sewer main to 12-Inch upstream of WWTP between West 2nd Street and West 3rd Street	$90,000	2043
CS-17	Lift Station 4 Upgrade	Upsize pumps to 1,100 gpm to meet 2043 capacity requirement. Replace mechanical, electrical and I&C; Rehabilitate wetwell concrete surface; Add bypass piping connection	$1,544,000	2043
Total CIP – Years 7 to 20		**$18,329,000**		

TABLE 9-2

Wastewater Treatment Plant – Capital Improvement Plan

Project No.	Project Name	Total Project Cost (2024 $)	Estimated Year of Completion	Description
6-Year CIP				
WW-1	Biosolids Treatment Improvements	$15,966,000	2028	Biosolid treatment improvements including expanded aerobic digester, new dewatering unit, and new dryer. *(This assumes the City selects the Class A Biosolids alternative.)*
Total 6-Year CIP		**$15,966,000**		
CIP for Years 7 to 20				
WW-2	MBR Expansion	$4,331,000	2033	MBR expansion including installing in the two empty MBR basins the modules, blowers, level switches, automated backwash system, automated dosing equipment and PLC control System
WW-3	Headworks Upgrade	$2,371,000	2033	Upgrade mechanical screening system
WW-4	UV System Replacement	$1,465,000	2033	Replace UV disinfection units
WW-5	Membrane Units Replacement	$800,000	2033	Replace membrane unit in MBR Basin 1 and 2
WW-6	Blower Replacement	$873,000	2033	Replace blowers for preaeration basin, MBR Basin 1 and 2
WW-8	Misc. Mechanical Equipment Replacement	$2,318,000	2033	Replace mechanical units including pumps, mixers, odor control, etc.
WW-7	SCADA / Controls Upgrade	$500,000	2043	Upgrade plan SCADA control system
Total CIP – Years 7 to 20		**$12,658,000**		