Philomath
OREGON

WASTEWATER SYSTEM FACILITIES PLAN

October 2017

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WASTEWATER SYSTEM FACILITIES PLAN

CITY OF PHILOMATH, OREGON

October 2017

Prepared for
City of Philomath
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Kate Brown, Governor

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October 23, 2017

Mr. Kevin Fear, Public Works Director
City of Philomath
Post Office Box 400
Philomath, Oregon 97370

Re: Philomath Wastewater Facilities Plan Approval
File No. 103463
Benton County

Dear Mr. Fear:

We have reviewed the final Wastewater System Facilities Plan prepared by Westech Engineering, Inc. and received in our office October 10, 2017. Additional revisions were received October 20, 2017. The revised document addresses all our comments as per our letter of September 23, 2017. Therefore, we are approving the document.

The next step is to start pre-design work. To avoid extra work and cost overruns, the City should not authorize final design until a pre-design report is reviewed and agreed on by City staff and DEQ.

Should the implementation of the proposed alternative in the facilities plan lapse over five years, we strongly recommend that you consult with DEQ staff to ensure that the proposed plan and issues are still relevant. It is sometimes possible that preparation of a new document may be warranted after five years.

Please feel free to call me at [541] 687-7341 should you have any questions or comments.

Sincerely,

Jaime Isaza

Jaime Isaza, Project Officer

ec: Christopher Brugato - Westech Engineering
David Cole, Mike Kucinski, Tim McFetridge - DEQ

ji: Philomath FP appr.docx

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FOREWORD

Using this Report

This report will be used by many people whose needs for information will differ widely. Accordingly, an Executive Summary appears at the beginning of this report. The summary provides an overview of the report and presents the main conclusions. Readers may gain a good general understanding of the report and its contents by reading the summary. Additional detailed information is presented in the body of the report.

LIST OF ABBREVIATIONS

AAF	average annual flow
AC	asbestos cement
ADWF	average dry weather flow
ATS	automatic transfer switch
AWWA	American Water Works Association
AWWF	average wet weather flow
BGS	below ground surface
BOD	biochemical oxygen demand
BSF	base sewage flow
CFS	cubic feet per second
CIP	capital improvement plan
CMU	concrete masonry units
DAF	dissolved air flotation
DEQ	Oregon Department of Environmental Quality
DHS	Oregon Department of Human Services
DO	dissolved oxygen
EPA	US Environmental Protection Agency
FEMA	Federal Emergency Management Agency
FM	force main
FPS	feet per second
FRP	fiber reinforced plastic
GPD	gallons per day
HDPE	high density polyethylene
HP	horsepower
IGA	intergovernmental agreement
KW	kilowatt
MAO	mutual agreement and order
MBR	membrane bioreactor
MBBR	moving bed bioreactor
MH	manhole
MMDWF	maximum month dry weather flow
MMWWF	maximum month wet weather flow
MG	million gallons
MGD	million gallons per day
NPDES	National Pollutant Discharge Elimination System
OAR	Oregon Administrative Rule
ODOT	Oregon Department of Transportation

OPSC	Oregon Plumbing Specialty Code
ORS	Oregon Revised Statutes
PDF	peak day flow
PHF	peak hour flow
PIF	peak instantaneous flow
PSI	pounds per square inch
PVC	Polyvinyl chloride
RPM	revolutions per minute
SBR	sequencing batch reactor
SCADA	Refers to a Supervisory Control and Data Acquisition (telemetry) system
SDC	system development charge
SF	square feet
SRT	solids retention time
TDH	total dynamic head
TSS	total suspended solids
TV	television
UGB	urban growth boundary
USGS	United States Geological Survey
UV	ultraviolet light
VFD	variable frequency drive
WEF	Water Environment Federation
WWTP	wastewater treatment plant

EXECUTIVE SUMMARY

Summary Outline

EXECUTIVE SUMMARY

INTRODUCTION

The purpose of this study is to provide a comprehensive evaluation of the City's wastewater system with respect to its existing and future needs, identify improvements and associated costs necessary to meet those needs, and provide the City with a framework for the provision of sanitary sewer service through the year 2037.

This executive summary has been prepared to provide a concise overview of the evaluations and analyses performed in each chapter of the study. A summary of the recommended capital improvement program costs appears at the end of this summary.

PROJECT OBJECTIVES

This Wastewater Facilities Plan was completed to achieve the following objectives;

- *Evaluate Current and Future Needs*

Evaluate the City's sanitary sewerage facilities with respect to existing and future needs, identify improvements and associated costs necessary to meet those needs, and provide the City with a guide for future development of the City's sanitary sewerage system.

- *Satisfy Funding Agency Requirements*

As with most small cities, Philomath may have some difficulty accumulating sufficient resources to construct the required improvements. Therefore, outside funding may be desired. The federal and state funding agencies that distribute funds for public wastewater projects have published guidelines for the preparation of Facilities Plans. This plan is intended to conform to those guidelines.

BACKGROUND INFORMATION AND NEED FOR PLAN

The City of Philomath is located west of Corvallis along Highway 20/34 in Benton County. The urban growth boundary encompasses approximately 2,540 acres. Of this area, approximately 1,320 acres are located within the City Limits. The current population of Philomath is approximately 4,650.

The City currently operates the City's wastewater utility under an NPDES permit issued by the Oregon Department of Environmental Quality (ODEQ). The City's wastewater utility consists of a conventional gravity collection system that drains to one of three wastewater pump stations. The pump stations pump wastewater through a common forcemain to a facultative lagoon treatment plant. During the winter months treated effluent from the City's system is discharged to the Marys River. During the summer months effluent from the City's system is used for irrigation at City-owned land application sites near the lagoons. The City's treatment plant is located south of the City and west of Bellfountain Road.

The existing Sewerage System Facilities Plan was prepared by Westech Engineering and was adopted by the City in July of 2004. This document included a list of recommended improvements for the wastewater utility. The City has completed many of the improvements identified in the 2004 plan. As such, the existing plan is somewhat outdated and a new plan is desired at this time. The capital improvement priorities identified in this document will be used to update the City's user fees and SDC fees.

Additional background and introductory information is presented in Chapter 1 of the plan.

STUDY AREA AND PLANNING CONSIDERATIONS

The City's Comprehensive Plan established an urban growth boundary (UGB) encompassing 2,540 acres, approximately 1,210 acres of which are outside the present City limits. Eventually all areas inside the UGB will be part of the City and will be served by the City's utility systems. The Oregon Department of Land Conservation and Development (DLCD) mandates that the planning area for facilities planning be limited to the land within the present UGB of the City. Therefore, the improvements recommended in this plan are based on development of land within the UGB in its present location, as well as the existing land use zoning for these areas. It is assumed that no significant development will occur within the study area that will require major changes to the existing zoning, and that there will be no significant expansions of the UGB within the study period. Changes in any of these assumptions could change the recommendations contained in this facilities plan. Should significant changes in any of the above occur, the facilities plan should be updated accordingly. Additional information regarding the study area and planning considerations is presented in Chapter 2.

The DEQ recommends a minimum 20-year planning period for wastewater facilities planning. In order to assess the City's needs over this time, population growth projections must be made to determine future wastewater flows and loads. The DEQ mandates the use of County coordinated growth rates and population projections. Therefore, the growth rates and population projections used in the Facilities Plan are based on figures developed by the Portland State Population Research Center which will presumably be adopted by Benton County. Using these projected growth rates, the projected municipal population of Philomath in the year 2037 is expected to be approximately 7,294 (see Section 5). Wastewater flow and load projections are detailed in Chapter 5. Some of the projects will not be constructed until several years after this document is adopted. As such, the designers for these projects will need to make new flow and loading projections that utilize current flow data and are based on 20 year projections from the date that construction is completed for each project.

BASIS FOR FACILITIES PLANNING

During the coming years, improvements to the City's existing wastewater collection and treatment facilities will be required to ensure reliable operation and compliance with regulatory standards. Haphazard improvements that do not adequately consider all of the issues that impact the system may end up costing the City more in the long run than well thought-out, carefully applied solutions. For example, if a particular sewer pipe cannot convey the volume of wastewater that flows into it, a logical solution is to replace the pipe with a larger pipe. However, if the larger pipe is sized only to accommodate the existing flow volumes and future growth upstream of the pipe occurs, the pipe size may need to be increased a second time to accommodate the flow increases. Instead of replacing the pipe twice, a more cost-effective solution is to replace the pipe once with a pipe sized to accommodate the existing flows plus the anticipated future growth. As this simple example illustrates, most wastewater facilities are not well suited for incremental expansion to accommodate growth. More often than not, the most cost effective solution is to initially size the facilities to accommodate anticipated growth within the planning period. Therefore, this Facilities Plan not only considers the existing deficiencies, but also considers what improvements are likely to be required during the planning period as the City grows and develops. The intent of the recommend capital improvement plan is to provide the City with reliable wastewater facilities that not only meet current demands, but will also adequately serve the City well into the future.

The City currently operates the wastewater facility under an NPDES permit issued by DEQ. All future facilities must be developed and maintained to ensure that the City can remain in compliance with the NPDES permit. Detailed descriptions of the regulatory requirements relevant to the City's wastewater utility are presented in Chapter 3.

OVERVIEW OF EXISTING FACILITIES

Chapter 4 provides a detailed description of existing wastewater collection and treatment facilities serving the City. The City currently serves approximately 1,569 user accounts. The City's existing wastewater facilities consist of a conventional gravity collection system that conveys wastewater to one of three pump stations. The City's gravity collection piping includes approximately 107,000 feet of mainline piping, 460 manholes, and 1,700 service laterals. In general, flow from the western portion of the City flows to Pump Station A and flow from the eastern portion of the City flows to the Newton Creek Pump Station. The third pump station is a relatively small station that serves the Timber Estates Subdivision near 19th Street and Chapel Drive. All three of these pump stations discharge into a common forcemain pipe that conveys water to the treatment plant located south of the Marys River. The City's original collection system was constructed in 1952. This piping utilized the materials that were availed at the time. Unfortunately, these older materials are not as well sealed as modern piping systems. As a result, the City's collection system admits large amounts of groundwater infiltration during the winter months. The original 1952 system is also more than 60 years old and will be more than 80 years old by the end of the planning period. Due to the large amounts of groundwater infiltration and the age of the original 1952 piping, it is appropriate for the City to continue implementing aggressive I/I corrective measures during the planning period.

The City's treatment facility is located on the south side of the Marys River near Bellfountain Road. The treatment plant consists of a headworks, three facultative lagoon cells, two chlorine contact chambers, and an irrigation pump station and land application facilities. A gas chlorine feed system is used to disinfect the effluent prior to discharge. During the winter months, the chlorine in the effluent is neutralized by the addition of sulfur dioxide solution. During the winter months, treated effluent is discharged to Marys River. During the summer months, treated effluent is used to irrigate the farmed land adjacent to the lagoons. The land currently under irrigation is owned by the City. There are periods each year when the City does not discharge any effluent. Each spring and fall there is a period of several weeks where the City is not allowed to discharge to the Marys River and the land application sites are too wet to receive irrigation water. The City currently leases the farming rights to a local grower. No irrigation occurs during the harvest season. Therefore, the City also stores water in the lagoons during the harvest season.

The plant also includes a small building that houses the chemical feed equipment as well as a small office and laboratory building. An overall schematic representation of the existing wastewater treatment system is presented in Figure 4-8. Detailed maps of the collection system are included in Appendix B. More detailed descriptions of the existing facilities are included in Chapter 4.

WASTEWATER FLOWS AND LOADS

Chapter 5 of the plan includes an analysis of the existing wastewater flow rates, organic loading rates, and solids loading rates to the treatment plant. Population projections are used to estimate future flows and loads. The design flows and loads are used to analyze the existing systems. The design flows and loads consist of the existing flows and loads, plus the flows and loads due to population growth. The reader is referred to Chapter 5 for a description of the flow projection methodology and the results.

COLLECTION SYSTEM DEFICIENCIES AND RECOMMENDED IMPROVEMENTS

Chapter 6 presents an analysis of the wastewater collection system. Current operation and maintenance practices are first reviewed. Since the adoption of the 2004 Facilities Plan, the City has been actively rehabilitating the original 1952 collection system and has made significant progress. This plan recommends continuing this work until the entire 1952 collection system has been rehabilitated. This work effort is formally described in this plan as the Sewer Rehabilitation and Replacement Program (annual Program #1) with a recommended annual budget of $200,000 per year. Background information for this recommendation is presented in Section 6.2.5.

In addition to operation and maintenance practices, the ability of the existing collection system to convey the anticipated wastewater flows is analyzed in Chapter 6. This analysis shows areas of the existing collection system that lack the capacity to adequately convey existing and projected wastewater flows. A hydraulic model was used to simulate flow through the collection system. At design flows, the model predicts widespread surcharging and raw sewage overflows. In order to correct these problems improvements to the collection system are identified. These improvements largely consist of replacing undersized gravity sewer pipes with larger diameter pipes. A listing of the recommended gravity collection system projects is included in Chapter 6. These improvements are later prioritized in Chapter 8 to develop the recommended Capital Improvement Plan (see discussion below).

Chapter 6 also includes an analysis of the pump stations and the common forcemain. Improvements are needed to ensure that these facilities have adequate capacity for the projected flows at the end of the planning period. The recommend plan for the Timber Estates Pump Station is to construct a new gravity sewer from the Timber Estates Pump Station wet well to the Newton Creek Trunk Sewer and abandoning the Timber Estates Pump Station. A new forcemain pipe from the Newton Creek Pump Station to the Wastewater Treatment Plans is also recommended to increase the capacity of both the Newton Creek Pump Station and Pump Station A. Finally, a major upgrade to the Newton Creek Pump Station may be needed if growth projections materialize. These pump station and forcemain improvements are described in greater detail in Chapter 6.

TREATMENT SYSTEM DEFICIENCIES AND RECOMMENDED IMPROVEMENTS

Chapter 7 includes an analysis of the City's treatment system. The City completed a major treatment plant improvement project in 2011. As such, the treatment facilities are in relatively good condition and should continue to serve the City for most of the planning period. Two relatively small projects are identified for inclusion on the Capital Improvement Plan. These include expansion of the land application system to increase the area of land used for dry weather effluent disposal and the installation of an outfall diffuser to improve effluent mixing in the Marys River.

Based on the estimated population growth rates, the organic treatment capacity of the plant may be stressed toward the end of the planning period. Should these projections materialize, the City may need to install aeration equipment in the lagoons to increase the organic treatment capacity of the plant. This project also includes the installation of a screening facility to screen materials from the influent stream and improve the treatment process. New blowers would be installed in a blower building to feed air to the diffused aeration equipment.

All three of these projects are discussed in greater detail in Chapter 7.

RECOMMENDED CAPITAL IMPROVEMENT PLAN

The Facilities Plan identifies a number of deficiencies and includes several recommended improvement projects. Some of these projects are more critical than others. Some projects should be constructed early in the planning period. Other projects are not needed immediately, but may be needed as the City grows and the existing system continues to age.

A prioritizing process was developed to rank the improvement projects. Factors utilized in the prioritizing process included several measures of criticality, as well as the cost/benefit ratio of each project. This process identified essential, high benefit to cost projects for early implementation, and the deferral of less critical, lower value projects. Each of the projects identified in the plan were examined and assigned a priority for implementation and appear in Table ES-1 below.

Priority 1 projects are considered to be needed immediately. They have been developed to resolve existing or near term system deficiencies. It is recommended that Priority 1 improvements are undertaken as soon as practical. Priority 2 projects will be needed beyond the near term of the Priority 1 projects to improve the quality of service throughout town. Although not critical at this time, they will likely be required at the some point during the planning period. Priority 3 projects are long-term improvements designed to provide sanitary sewer service to areas that develop in response to population growth. While important, they are not considered to be critical at the present time and should not be included in the City's list of proposed improvements for the next 20 year planning period.

At a minimum, all of the Priority 1 and Priority 2 improvements should be included in the CIP. The Priority 3 improvements are largely growth driven. It is envisioned that the Priority 3 improvements will be constructed as part of future development and that individual developers will construct and pay for the Priority 3 improvements on an incremental basis.

Several potential funding programs are available to assist communities with the funding of major infrastructure improvements. A number of these programs are identified and discussed in Chapter 8. Even with funding assistance, increases in user rates and SDC fees may be required to fund the needed improvements.

Table ES-1 | Recommended Capital Improvement Priorities

Project Code [1]	Project	Priority	Total Estimated Project Cost [2]
G-1	9th Street to 7th Street Sewer Lines – Manhole #35 to Manhole #184	1	$398,000
G-2	10th Street Sewer Lines – Manhole #34 to Manhole #45	1	$126,000
G-3	Main Street Sewer Lines – Manhole #45 to Manhole #52	1	$230,000
G-4	8th & College Street Sewer Lines – Manhole #52 to Manhole #56	1	$189,000
G-8	Applegate Street and 20th Street Trunk Sewer - Manhole #1 to Manhole #6	1	$344,000
G-16	Timber Estates Trunk Sewer	1	$370,000
F-1	Newton Creek Forcemain	1	$1,441,000
T-3	Land Application System Expansion	1	$394,000
		Subtotal Priority 1….	**$ 3,492,000**
G-5	Pioneer and 11th Street Sewer Lines – Manhole #71 to Manhole #74	2	$237,000
G-6	15th Street Trunk Sewer (South) – Manhole #27 to Manhole #288	2	$510,000
G-7	15th Street Trunk Sewer (North) – Manhole #288 to Manhole #94	2	$116,000
P-3	Newton Creek Pump Station Improvements	2	$1,479,000
T-1	Marys River Outfall Diffuser	2	$173,000
T-2	Lagoon Aeration and Headworks Screening	2	$2,500,000
T-4	Facilities Plan Update	2	$65,0000
		Subtotal Priority 2….	**$ 5,080,000**
G-9	Newton Creek Trunk Sewer – Newton Creek Pump Station to Manhole 476	3	$764,000
G-10	19th Street Trunk Sewer South	3	$917,000
G-11	Railroad Trunk Sewer	3	$1,014,000
G-12	19th Street/Green Road Trunk Sewer	3	$1,271,000
G-13	Industrial Way Trunk Sewer	3	$866,000
G-14	Sewer Basin N5 Trunk Sewer	3	$622,000
G-15	Chapel Drive Trunk Sewer	3	$1,056,000
P-1	Basin P1 Pump Station and Forcemain	3	$530,000
P-2	Basin P2 Pump Station and Forcemain	3	$500,000
		Subtotal Priority 3….	**$ 7,054,000**
		TOTAL….	**$ 15,626,000**
Recurring Annual Programs			
Pgm-1	Sewer Collection System Rehabilitation Program (Program – 1)		$200,000
		Subtotal Recurring Annual Programs….	**$ 200,000**

[1] Project Code Legend:
G = Gravity Sewer T = Treatment Pgm = Annual Program P = Pump Station F = Forcemain

[2] See Section 8.3 for basis of project cost estimates

CHAPTER 1

INTRODUCTION

Chapter Outline

INTRODUCTION

CHAPTER 1

1.1 INTRODUCTION

The City of Philomath is located on Highway 20 approximately five miles west of Corvallis in Benton County, Oregon. The current population of Philomath is approximately 4,650. The City was founded in 1882. The past economic activity in Philomath has centered around the forest products industries. With the decline of the forest products industries in western Oregon future prosperity of Philomath appears to be tied to diversified light industries together with a growing residential community. Many of the residents of Philomath work in Corvallis and other nearby communities.

The City is bisected east to west by the Corvallis-Newport Highway 22/34. The Marys River is located south of the City. Philomath's original sewerage facilities were constructed in 1952 and served most of the area within the present City limits west of Newton Creek. The existing wastewater treatment lagoons are located south of the Marys River outside of the City's Urban Growth Boundary. The collection system is a conventional gravity collection system with three pump stations. The treatment plant consists of a three-cell facultative lagoon system with chlorine disinfection. Treated effluent is discharged to the Marys River during the winter months and used for irrigation during the summer months.

The City's current development standards require findings that adequate capacity is available in the utility systems prior to development occurring. The implementation of this standard is difficult without a current sanitary sewer system master plan that identifies the required basin-wide improvements. An understanding of how the collection system works and how development within the basin impacts its performance allows one to determine what improvements to the sanitary sewer system are required by new development.

The existing Sewerage System Facilities Plan was prepared by Westech Engineering and was adopted by the City in July of 2004. This document included a list of recommended improvements for the wastewater utility. The City has completed many of the improvements identified in the 2004 plan (Table 1-1). As such, the existing plan is somewhat outdated and a new plan is desired at this time. The capital improvement priorities identified in this document will be used to update the City's user fees and SDC fees.

1.2 AUTHORIZATION

The City authorized Westech Engineering to proceed with the preparation of this Wastewater Facilities Plan in February of 2016. The plan has been prepared to meet the current requirements of the regulatory and funding agencies.

Table 1-1 | Status of 2004 Facilities Plan Capital Improvement Projects

Project	Priority Ranking	Project Status
I/I Reduction Plan (Original 1952 Collection System)	1A	Ongoing
Pump Station A (16th & Cedar)	1A	Completed
Overflow Structure (15th & College)	1A	Completed
Buried Fuel Tank at Newton Creek Pump Station	1A	Completed
WWTP Phase I Improvements	1B	Completed
Cedar Street Sewer (MH 200 to MH 29)	1A	Completed
13th Street Sewer (MH 29 to MH 31)	1A	Completed
Applegate Street Sewer (MH 31 to MH 32)	1A	Completed
Applegate Street Sewer (MH 1 to MH 2)	1A	Not Completed
20th Street Sewer (MH 2 to MH 6)	1A	Not Completed
College Street Sewer (MH 6 to MH 9)	1A	Completed
12th Street Sewer (MH 32 to MH 71)	1A	Completed
Applegate Street Sewer (MH 32 to MH 34)	1A	Completed
Applegate Street Sewer (MH 34 to MH 35)	1A	Completed
10th Street Sewer (MH 34 to MH 45)	1A	Under Construction
Main Street Sewer (MH 45 to MH 46)	1A	Under Construction
WWTP Phase II Improvements	2	Completed
Applegate Street Sewer (MH 203 to MH 205)	2	Completed
Applegate Street Sewer (MH 205 to MH 208)	2	Completed
9th Street Sewer (MH 35 to MH 36)	2	Under Construction
Alley Sewer (MH 36 to MH 38)	2	Under Construction
Main Street Sewer (MH 46 to MH 52)	2	Under Construction
8th Street Sewer (MH 52 to MH 53)	2	Not Completed
Timber Estates Pump Station Improvements	2	Not Completed
New Force main from Newton Creek PS to WWTP	2	Not Completed
Newton Creek Pump Station Improvements	2	Not Completed

1.3 PURPOSE

The purpose of this plan is to provide a comprehensive evaluation of the City's wastewater system with respect to its existing and future needs, identify improvements and associated costs necessary to meet those needs, and provide the City with a framework for the provision of wastewater service through the year 2037.

This plan will assist the City in the planning and implementation of capital improvements and will assist the development community as the wastewater system is expanded for future growth. The plan will benefit the current and future residents of the City by enhancing the quality of life through improved water quality, planned growth, scheduled improvements, and an equitable distribution of improvement costs.

1.4 SCOPE OF STUDY

The scope of the Wastewater Facilities Plan is intended to comply with the applicable requirements of DEQ and the City. Study area characteristics were identified and included both physical and socioeconomic conditions. Existing population and land use were examined and projected into the future.

The existing wastewater system was investigated. Data was collected on the existing wastewater collection and treatment systems from operating records, conversations with City staff, on-site investigations, maps, as-built records, and other pertinent documentation. Existing facilities were evaluated in terms of location, sizing, capacity, condition, limitations, and performance. Consideration was given to the manner in which existing and proposed facilities could be used in the future as the study area develops to City zone densities.

Typical wastewater characteristics were identified in terms of loads, flows, strength and I/I allowances throughout the year. Future characteristics were projected to establish capacity requirements. Flows were addressed for both dry period and wet period conditions, and unit design values were established. Future wastewater characteristics were projected.

The basis for planning was established. Applicable regulatory requirements were identified and addressed, including current and future treatment criteria and discharge standards. The design capacity of the City's collection piping, pump stations, and treatment facilities was examined to determine impacts to present and future operation of wastewater facilities. Alternatives were identified for collection, treatment, and effluent disposal/reuse. Alternatives for system administration were identified and evaluated.

Nonviable options were screened out, and a limited number of selected alternatives were established and evaluated in detail. Finally, a recommended plan was identified that will enable the City to provide wastewater collection and treatment within the study area. This plan includes preliminary design data, capital improvement and operational costs, and a description of potential financing options. This report does not include a wetland inventory or delineation(s), topographic or aerial surveys, on-site environmental investigations or geotechnical investigations.

1.5 PREVIOUS STUDIES AND REPORTS

The following reports and studies were referenced in the preparation of this study:

- *Construction Drawings, City of Philomath Wastewater Treatment System Improvements,* Philomath, Oregon, Westech Engineering, Inc., April 2011.
- *Construction Drawings, City of Philomath Sanitary Sewer System*, City of Philomath, Oregon, Cornell, Howland, Hayes, & Merryfield, April 1951
- *Construction Drawings, Philomath Sanitary Sewer Pump Station and Trunk Sewer Improvements*, City of Philomath, Oregon, Westech Engineering, Inc., February 2009
- *Recycled Water Use Plan*, City of Philomath, Oregon, Cascade Earth Sciences, February 2011
- *Wastewater System Facilities Plan*, City of Philomath, Oregon, Westech Engineering, Inc., July 2004.

1.6 WASTEWATER TERMS AND DEFINITIONS

An understanding of key wastewater terms and definitions is necessary for an understanding of the discussions in this and subsequent sections. The following does not include all terms used in this report, but will provide a useful glossary for those readers not familiar with wastewater terminology. The different sewage flow classifications are defined in Chapter 5.

- Aerobic - Microorganisms living in the presence of free oxygen, or biological treatment processes that occur in the presence of oxygen.
- Anaerobic - Microorganisms capable of living without the presence of free oxygen, or biological treatment processes that occur in the absence of oxygen.
- Anoxic Denitrification - The process by which nitrate nitrogen is converted biologically to nitrogen gas in the absence of oxygen. This process is also known as anaerobic denitrification.
- Attached Growth Process - A biological treatment process in which the microorganisms responsible for the conversion of the organic matter or other constituents in the wastewater to gases and cell tissue are attached to some inert medium such as rocks, slag, ceramic or plastic materials. Attached growth treatment processes are also known as fixed film processes.
- Biological Treatment Processes - Treatment processes by which the stabilization and decomposition of organic material in sewage is accomplished by living microorganisms. The organic matter is used as a food source for microorganisms, and converted to forms which can either be removed from the waste stream (soluble organics) or are sufficiently stabilized to allow disposal without negatively affecting the environment (insoluble organics).
- Biological Nutrient Removal - The removal of nitrogen and/or phosphorus with biological treatment processes.
- Biosolids – Treated sludge that is removed from a treatment facility for beneficial reuse or disposal.
- BOD (Biochemical Oxygen Demand) - The amount of oxygen required to biologically stabilize the organic material in sewage by aerobic treatment processes. All references to BOD in this report are to 5-day BOD at 20°C (BOD5).
- Chlorine Residual - The measured residual of chlorine used in disinfecting wastewater. Chlorine residual can exist in two forms; combined or free. The specific form is dependent on the rate of formation, which is controlled by the pH and temperature. A free chlorine residual is the most effective in achieving disinfection.
- Denitrification - The biological process by which nitrate is converted to nitrogen and other gaseous end products.
- DEQ – Oregon Department of Environmental Quality
- Facultative Processes - Biological treatment processes in which the organisms can function in the presence or absence of molecular oxygen.
- Fecal Coliform - Bacteria which are used as an indicator of fecal pollution.
- Industrial Wastes - Wastes produced as a result of manufacturing or processing operations.
- Infiltration/Inflow (I/I) - Groundwater and stormwater which enters the sanitary sewer system.

- Excessive I/I - Portion of infiltration or inflow which can be removed from the sewerage system through rehabilitation at less cost than continuing to transport or treat that portion of I/I.
- Infiltration - Water that enters the sewage system from the surrounding soil. Common points of entry include broken pipe and defective joints in pipe and manhole walls. Although generally limited to sewers laid below the normal groundwater level, infiltration also occurs as a result of rain or irrigation water soaking into the ground and entering mains, manholes, or shallow house sewer laterals with defective joints or other faults.
- Base Infiltration - Water that enters the sanitary sewer system from the surrounding soil during periods of low groundwater levels.
- Rainfall Induced Infiltration - Additional infiltration which enters the sewerage system during and for several days after a period of rainfall. Rainfall often percolates into sewer ditches, especially ditches with granular backfill, and establishes a perched water table. This water then infiltrates into faulty sewers and manholes.
- Sludge - Solid and semisolid residuals resulting from wastewater treatment operations.
- Inflow - Stormwater runoff which enters the sewerage system only during or immediately after rainfall. Points of entry may include connections with roof and area drains, storm drain connections, holes in manhole covers in flooded streets, and manhole cones located in ditch lines and that do not have watertight joints.
- Lagoon (Stabilization Pond) - A shallow basin constructed by excavating the ground and diking, for the purpose of treating raw sewage by storage under conditions that favor natural biological treatment and accompanying bacterial reduction.
- MAO – Mutual Agreement and Order
- Nitrification - The biological process by which ammonia nitrogen is converted first to nitrite, then to nitrate.
- NPDES - National Pollutant Discharge Elimination System.
- pH - The degree of acidity or alkalinity of waste water, 7.0 being neutral, a lower number being acidic, and a higher number being basic.
- Sanitary Sewage - Waterborne wastes principally derived from the sanitary conveniences of residences, business establishments, and institutions.
- Suspended Growth Process - A biological treatment process in which the microorganisms responsible for the conversion of the organic matter or other constituents in the wastewater to gases and cell tissue are maintained in suspension within the liquid.
- TSS (Total Suspended Solids) - All of the solids in sewage that can be removed by settling or filtration. The quantity of TSS removed during treatment impacts the sizing of sludge handling and disposal processes, as well as the effectiveness of disinfection.
- Wastewater - The total fluid flow in a sewerage system. Wastewater may include sanitary sewage, industrial wastes, and infiltration and inflow (I&I).

CHAPTER 2

STUDY AREA AND PLANNING CONSIDERATIONS

Chapter Outline

STUDY AREA AND PLANNING CONSIDERATIONS CHAPTER 2

2.1 INTRODUCTION

Philomath is situated north of the Marys River near the center of Benton County. The City is located on Highway 20/34 approximately five miles west of Corvallis. The Corvallis-Newport Highway 20/34 bisects Philomath east to west and provides the major road transportation into and through the City. Other major roads include Green Road and West Hills Road entering the City from the north, and Fern Road and Bellfountain Road entering the City from the south. The Union Pacific Railroad (formerly Southern Pacific Railroad Co.) also has a rail line passing through the City.

The City's Comprehensive Plan was developed in 1983 and updated in 2003. The Comprehensive plan established a large urban growth boundary (UGB) which encompasses approximately 2,540 acres. Of this area, approximately 1,210 acres are outside the present City Limits. Eventually the entire area will be part of Philomath and will be served by the City's utility systems. Figure 2-2, presented at the end of this chapter for formatting reasons, is a vicinity map depicting these features.

2.2 STUDY AREA

The study area of this report is the entire area within the UGB. The improvements recommended in this plan are based on the development of land within the UGB in its present location, as well as the existing comprehensive plan designations and land use zoning for these areas. It is assumed that no significant development will occur within the study area that will require major changes to the existing comprehensive plan designations and zoning, and that there will be no significant expansions of the UGB within the study period. Changes in any of these assumptions could change the recommendations contained in this plan. Should significant changes in any of the above occur, this plan should be updated accordingly.

2.3 STUDY PERIOD

Choosing a "reasonable" design period for which a utility system should be designed is a somewhat arbitrary decision. If the design period is too short the public faces the prospect of continual upgrades and replacements as demands exceed capacity. On the other hand, choosing a design period that is too long can lead to facilities with excess capacity that may never be needed if population growth does not occur at the projected rates. Such facilities can place an economic burden on the present population and may become obsolete before being fully utilized.

The Oregon Department of Environmental Quality (ODEQ) has established 20 years as a proper planning period for sanitary sewer system improvements. This report will evaluate the anticipated sewage collection, pumping, treatment, and disposal needs for the 20 year planning period. The collection system piping will be planned for the ultimate development of land within the UGB based on current land use designations. Although this may result in capacities greater than those needed during the 20-year planning period, sewage collection lines are, by their very nature, unsuited for incremental expansion without extensive capital outlays. The planning period used in this report is 20 years and ends in the year 2037. Some of the projects will not be constructed until several years after this document is adopted. As such, the designers for these projects will need to make new flow and loading projections that utilize current flow data and are based on 20 year projections from the date that construction is completed for each project.

It should be recognized that projections into the future are subject to many variables and assumptions, some of which may prove inaccurate. Accordingly, it is recommended that the City review its wastewater system at five-year intervals and update this report as appropriate.

2.4 PHYSICAL ENVIRONMENT

2.4.1 Climate and Rainfall Patterns

The study area is located in the Willamette Valley along the eastern foothill of the coast range. The climate in Philomath is relatively mild throughout the year, characterized by cool, wet winters and warm, dry summers. Growing seasons in the Willamette Valley are long, and moisture is abundant during most of the year (although summer irrigation is common).

The study area has a predominant winter rainfall climate. Typical distribution of precipitation includes about 50 percent of the annual total from December through February, lesser amounts in the spring and fall, and very little during summer. Rainfall tends to vary inversely with temperatures -- the cooler months are the wettest, the warm summer months the driest.

Extreme temperatures in the study area are rare. Days with maximum temperature above 90°F occur only 5-15 times per year on average, and below 0°F temperatures occur only about once every 25 years. Mean high temperatures range from the low 80s in the summer to about 40°F in the coldest months, while average lows are generally in the low 50s in summer and low 30s in winter.

Although snow falls nearly every year, amounts are generally quite low. Willamette Valley floor locations average 5-10 inches per year, mostly during December through February. High winds occur several times per year in association with major weather systems.

Relative humidity is highest during early morning hours, and is generally 80-100 percent throughout the year. During the afternoon, humidity is generally lowest, ranging from 70-80 percent during January to 30-50 percent during summer. Annual evaporation is about 35 inches.

Winters are likely to be cloudy. Average cloud cover during the coldest months exceeds 80 percent, with an average of about 26 cloudy days in January (in addition to 3 partly cloudy and 2 clear days). During summer, however, sunshine is much more abundant, with average cloud cover less than 40 percent; more than half of the days in July are clear.

There are extensive weather records for Hyslop Field between Corvallis and Albany. While the data from this weather station is not specifically for the City of Philomath, these values are generally believed to be representative for the immediate area around Philomath. Although there may be daily and weekly variations, the annual average climate is approximately the same. The climate data from Hyslop Field is used throughout the remainder of this document.

The study area receives an average of approximately 43 inches of precipitation annually, with the majority of the rainfall occurring during the winter months. The wettest year (since 1910) was 1996 when approximately 73 inches of rainfall was measured. The second wettest year was 1998, with approximately 60 inches of rainfall. Approximately 78% percent of the annual precipitation occurs between November 1 and April 30.

2.4.2 Topography

Philomath is located on the western edge of the Willamette Valley, near the point where the Marys River leaves the Coast Range. The City center is located on the second bench north of the Marys River. The natural surface drainage across the study area flows to the south, and the existing storm drainage system intercepts and routes flow into the Marys River.

The topography within the study area ranges from relatively flat south of Main Street and along Newton Creek, to steeper slopes and hills to the north, east and west of the City. Generally, the topography is gently sloping and undulating. Slopes over most of the area are between 0 and 3 percent. The northwest part of Philomath has steeper slopes ranging to 14 percent. The elevation within the study area ranges from approximately 260 feet along the Marys River to a high point of 450 feet at the northwestern corner of the UGB.

2.4.3 Soils

Several different soil types have been identified and mapped within the study area and appear on Figure 2-4, presented at the end of this chapter. Most of the local features are formed from water-deposited sediments. This alluvium is mainly derived from sandstones and siltstones with a small amount of tuffaceous deposits from volcanic ash. Seven major soil types are present in the Philomath area. Five of the soil types are predominately loams with good drainage, the McAlpin-Abiqua, Malabon-Coburg, Woodburn-Willamette, Dixonville-Philomath, and Jory-Bellpine associations. The remaining two soil types are more poorly drained loams, the Waldo-Bashaw and Dayton-Amity associations. Some of these soils are poorly suited for septic systems. However, none of the soil types outright preclude the construction of typical wastewater facilities from a foundation stability point of view. A detailed geotechnical report will be required prior to final design of the recommended improvements.

This discussion of soil types is based on the information included in the Soil Survey of Benton County, Oregon (July 1975) prepared by the Soil Conservation Service (now the Natural Resource Conservation Service). This document shows the approximate location of the soil types in the study area. The reader is referred to the Benton County Soil Survey for more detailed definitions and descriptions of the individual soil designations.

2.4.4 Geologic Hazards

Known geologic hazards within the study area include steep slopes, high seasonal groundwater, seismic concerns, and flooding.

2.4.4.1 Steep/Unstable Slopes

The only areas of potential slope stability concerns within the study area are on Neabeack Hill in the southeast corner of town and in the hills on the northwest corner of town. Steep slopes can have the potential for either mass movement or slope erosion. Mass movement results from shifting of rock or soil material in response to gravity, such as landslides and rock slides. These mass movements are often precipitated or aggravated by excessive groundwater. Slope erosion is the removal of soils or rock that occurs as a result of sheet flow, resulting in surface erosion or gully erosion. This is primarily caused by private land use practices (mainly land clearing and road construction) that can exacerbate slope erosion.

The 1979 "Engineering Hazard Map of the Corvallis Quadrangle" identifies no steep slope or mass movement hazards within the study area. However, the geologic hazard maps generally do not identify these types of hazards for areas less than 5 to 10 acres. Therefore, although this area shows no signs of recent movement, it is considered a geologically sensitive area for siting critical facilities, such as pump stations or treatment plants.

2.4.4.2 High Groundwater.

Seasonal high groundwater is a common occurrence within the study area, and is a primary cause for the observed high levels of infiltration and inflow. The high groundwater problems are caused primarily by perched water tables due to soil saturation and lack of local drainage.

2.4.4.3 Seismic

The 2008 U.S. Geological Survey (USGS) National Seismic Hazard Maps display earthquake ground motions for various probability levels across the United States. These factors are applied in the seismic provisions of building codes, insurance rate structures, risk assessments, and other public policy. A review of these maps identifies Oregon as having a relatively high seismic risk. The Oregon Structural Specialty Code shares this assessment and has adopted similar ground motion data as the USGS. Seismic risk factors for structures are typically influenced by a combination of factors including the geographical location, specific building and structural configurations, and local soil types. The construction and rehabilitation of significant structures recommended by this report (buildings and hydraulic structures) will require detailed geotechnical reports and site specific seismic evaluations.

2.4.4.4 Flooding

The Marys River is the primary stream within the study area, with Newton Creek being the only major tributary within the study area. The Marys River extends approximately 40 miles from its confluence with the Willamette River to its headwaters northwest of Philomath. Newton Creek enters the Marys River at river mile 10.0. The Marys River has a streamflow pattern similar to other Willamette Valley streams. It is typified by high flows during the winter and low flows during the summer months.

The Federal Emergency Management Agency (FEMA) has established a 100-year floodplain designation and insurance ratings for the study area. While sometimes referred to as the "100 year flood", it is more accurate to consider it the flood having a 1 percent chance of occurrence in any year, or a 10 percent chance of occurrence during any 10 year period.

During a 100-year flood (as defined by the Federal Emergency Management Association, FEMA), the Marys River and Newton Creek rise out of their normal channels creating a large floodplain. The limits of the 100 and 500 year floodplains are shown on Figure 2-5 at the end of this chapter. Flood profiles and maps for those portions of the Marys River adjacent to the study area are included in the Flood Insurance Study prepared for the City of Philomath as follows.

- Inside City Limits
  Floodway panel 410011-0001, June 15, 1982
  FIRM panel 410011-0001 B, June 15, 1982
- Outside City Limits
  Floodway panel 410008-0067 (panel 67 of 250), August 5, 1986
  Floodway panel 410008-0090 (panel 90 of 250), August 5, 1986
  FIRM panel 410008-0067C (panel 67 of 250), August 5, 1986
  FIRM panel 410008-0086C (panel 86 of 250), August 5, 1986
  FIRM panel 410008-0090C (panel 90 of 250), August 5, 1986

It should be noted that the Floodplain and Floodway boundaries shown on the FEMA flood maps and the maps enclosed in this report are based on flood elevations, and as such the actual boundaries may vary slightly from the location shown. Final determinations of whether property is within the floodway or floodplain must be determined based on a topographic survey of the property in question.

2.4.5 Public Health Hazards

There are no known public health hazards with the City of Philomath.

2.4.6 Energy Production and Consumption

Electricity is provided to the community by either Pacific Power or the Consumers Power. Natural gas service is provided by Northwest Natural Gas. There are no known power generation facilities with the City. With regards to energy consumption, the major energy consumers in a wastewater collection and treatment system are the electric motors required to drive pumps, and other equipment. It is recommended that these components be specified as having high or premium efficiency motors, which will reduce the operating costs over the life of the project. Depending on the current programs in place with the electric utility providing service, there may be rebates available if high/premium efficiency electrical motors are specified that will tend to offset the slightly higher capital construction cost.

2.4.7 Water Resources

The City of Philomath has two water sources: surface water from the Marys River, and groundwater from a well located on 11th Street. Surface water quality protection is subject to extensive regulation by the State of Oregon. Water quality regulations related to the treatment and disposal of wastewater are summarized in Chapter 3. The primary groundwater concern is the potential for contamination of drinking water supplies from sewage or treated effluent. Oregon Health Division regulations specify minimum separation distances between wastewater facilities and groundwater wells. The City also monitors the groundwater quality in the vicinity of the wastewater lagoons by annual sampling and testing at three monitoring wells.

2.4.8 Flora and Fauna

The study area encompasses upland areas as well as riparian areas associated with the Marys River and its tributaries. The natural vegetation within the study area has been largely replaced by urban development, rural residential, or agricultural (pasture or seed grass) uses. The area is capable of supporting lowland meadows or forests but to a large extent these have been replaced.

Typical native vegetation along lowland foothill areas include such tree species as Douglas fir, Western Red Cedar, Big Leaf maple, Vine Maple, California black Cottonwood, Pacific Yew, Ash, Oregon oak, and Hawthorn. Shrubs that can be found are Salal, Snowberry, Indian Plum and Western Hazel. Willows and various grasses are also found in this habitat. Common wildlife species include Muskrat, Beaver, Opossum, Raccoon, Skunk, Coyote, and Deer. The Marys River provides habitat for rainbow trout, coastal cutthroat trout, dace, sculpin, salmon, and steelhead.

2.4.9 Environmentally Sensitive Areas

The Marys River, Newton Creek, and the riparian areas and wetlands adjacent to these natural waterways are considered to be environmentally sensitive areas. Figure 2-6 included at the end of this chapter shows the locations of designated wetlands within the study area. These wetland areas were identified as part of a Local Wetlands Inventory performed by the City of Philomath in 1996. Any projects that impact jurisdictional wetlands will require permitting through the Oregon Department of State Lands and the US Army Corps of Engineers.

2.4.10 Cultural Resources

Incorporated in 1882, Philomath has a rich history as one of the early settlements in the Willamette Valley. Several buildings and structures throughout town are included on the National Register of Historic Places. The mid Willamette valley was inhabited with the Calapooia people when the first western settlers arrived in the mid 1840's. It is also likely that prehistoric people inhabited the study area at one time. Remains of these cultures will likely be located adjacent to the Marys River. Cultural resources are protected by State Laws. Therefore, cultural resource investigations should be prepared in advance of any project that has the potential to impact cultural resources.

2.5 SOCIOECONOMIC ENVIRONMENT

Growth within the study area will depend on socioeconomic conditions. The following section contains a general discussion of economic conditions, trends, population, land use, and public facilities relating to the both the study area and the City.

2.5.1 Economic Conditions and Trends

Population growth and the resultant wastewater flows within the study area are linked to the economic conditions and trends of the City of Philomath and the greater Corvallis-Philomath metropolitan area. Growth in the City of Corvallis has to some extent met resistance from local residents. This has displaced some of the growth that may have occurred in Corvallis to Philomath. Philomath is an attractive town with a rural atmosphere that offers more affordable housing options than Corvallis. Philomath is to some extent evolving into a bedroom community for persons employed in Corvallis. With little significant industrial or commercial growth expected in the near future, this characterization is likely to remain valid throughout the planning period.

Philomath has experienced average levels of development during the past decade. This pace is expected to continue over the planning period. However, shorter periods of high growth rates are likely to be intermixed with shorter periods of low growth rates. For example, residential growth during the last five years has been relatively slow. However, a developer is currently working on a large housing development in the southeastern portion of the City that will add several hundred homes to the City. The exact timing of this project is unknown and the area must be annexed into the City by public vote. However, this one development has the potential to generate a rapid short-term spike in population growth.

2.5.2 Population and Growth Projections

Philomath's population in 2016 was approximately 4,650[1]. In June of 2017, population projections for Benton County were prepared by the Portland State University Population Research Center[2]. These projections estimate the 2035 population of Philomath to be 7,222. This value is known as the "county coordinated population projection" and will be used for planning purposes in order to conform to state-wide planning goals. As noted elsewhere in this document, the study period ends in 2037. Therefore, the 2035 population was extrapolated for two additional years for the preparation of this document. The coordinated population projections are based on an average annual growth rate of 0.5% from 2035 to 2067. This growth rate was applied to the 2035 population of 7,222 to estimate the 2037 population of 7,294.

A more detailed discussion of future population growth is presented in Chapter 5 -Wastewater Flows and Loads.

2.5.3 Land Use

The City's Comprehensive Plan includes a large urban growth boundary (UGB) that encompasses approximately 2,540 acres with approximately 1,320 acres within the current City Limits.

Eventually the entire area within the UGB will be part of Philomath and will be served by the City's utility systems. The planning area is made up of land in two general categories, namely land inside of City limits and land outside of the City limits, all of which is inside the Urban Growth Boundary. Land use zoning in Philomath is comprised primarily of residential uses, although the Comprehensive Plan sets aside large areas for industrial and commercial development. Total areas under each zoning designation are listed in Table 2-1 and ranked in Figure 2-1. A map showing the UGB, City limits and land use zoning areas appears on Figure 2-3 at the end of this chapter.

The majority of the land within the City limits is currently developed or partially developed. The majority of the land inside the UGB, but outside the City limits, is undeveloped or underdeveloped. Of the undeveloped land inside the planning area and outside the City limits, the majority (approximately 68%) is zoned for residential use and the remainder for a mix of commercial, industrial, and parks/open space.

[1] Portland State University, Population Research Center
[2] Portland State University, Population Research Center, Coordinated Population Forecast Benton County Oregon 2017-2067

Table 2-1 | Approximate Areas by Land Use Zone Within Current City Limits

Land Use	Total (Acres)	Total (%)
Downtown Commercial (C-1)	5.1	0.4%
General Commercial (C-2)	37.5	3.3%
Heavy Industrial (HI)	145.5	12.8%
Industrial Park (IP)	226.9	20.0%
Light Industrial (LI)	80.6	7.1%
Office Residential (O/R)	32.0	2.8%
Public (P)	160.8	14.2%
Low-Density Residential (R-1)	283.2	24.9%
Medium-Density Residential (R-2)	114.2	10.1%
High-Density Residential (R-3)	50.2	4.4%
Total	1,136	100%

Table 2-2 | Approximate Areas by Comp. Plan Designation (Inside UGB and Outside Current City Limits)

Land Use	Total (Acres)	Total (%)
Commercial	0.6	0.1%
Industrial	360.4	31.4%
Public Area	9.7	0.8%
Low-Density Residential	605.2	52.7%
Medium-Density Residential	172.5	15.0%
Total	1,148	100%

Figure 2-1 | Ranked Land Uses for All Land Inside the UGB

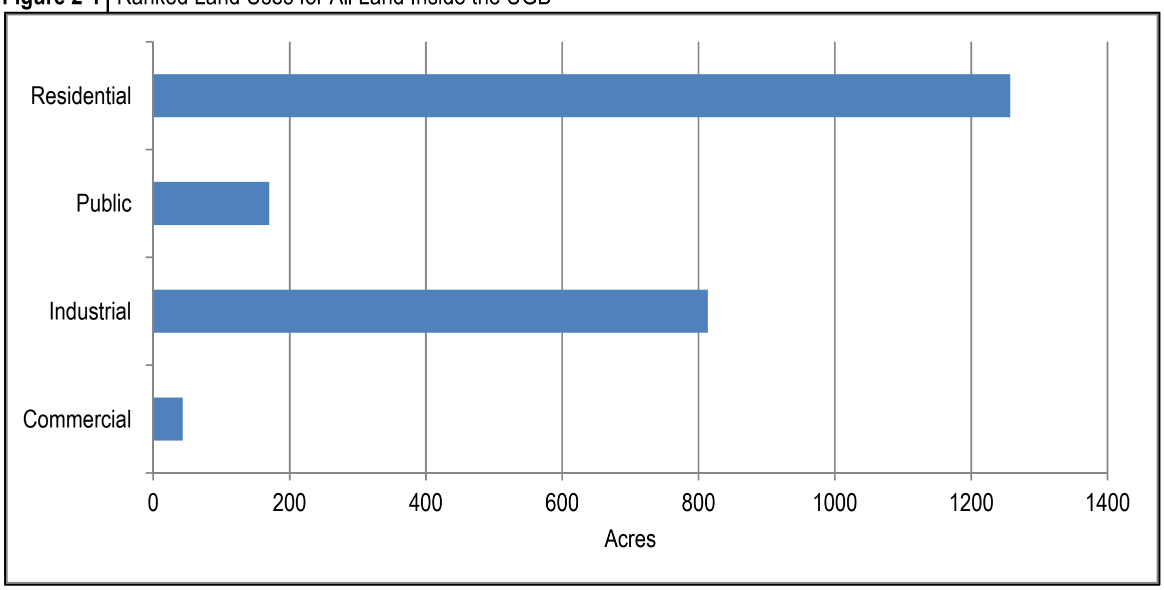


Figure 2-2 | Study Area and Vicinity Map

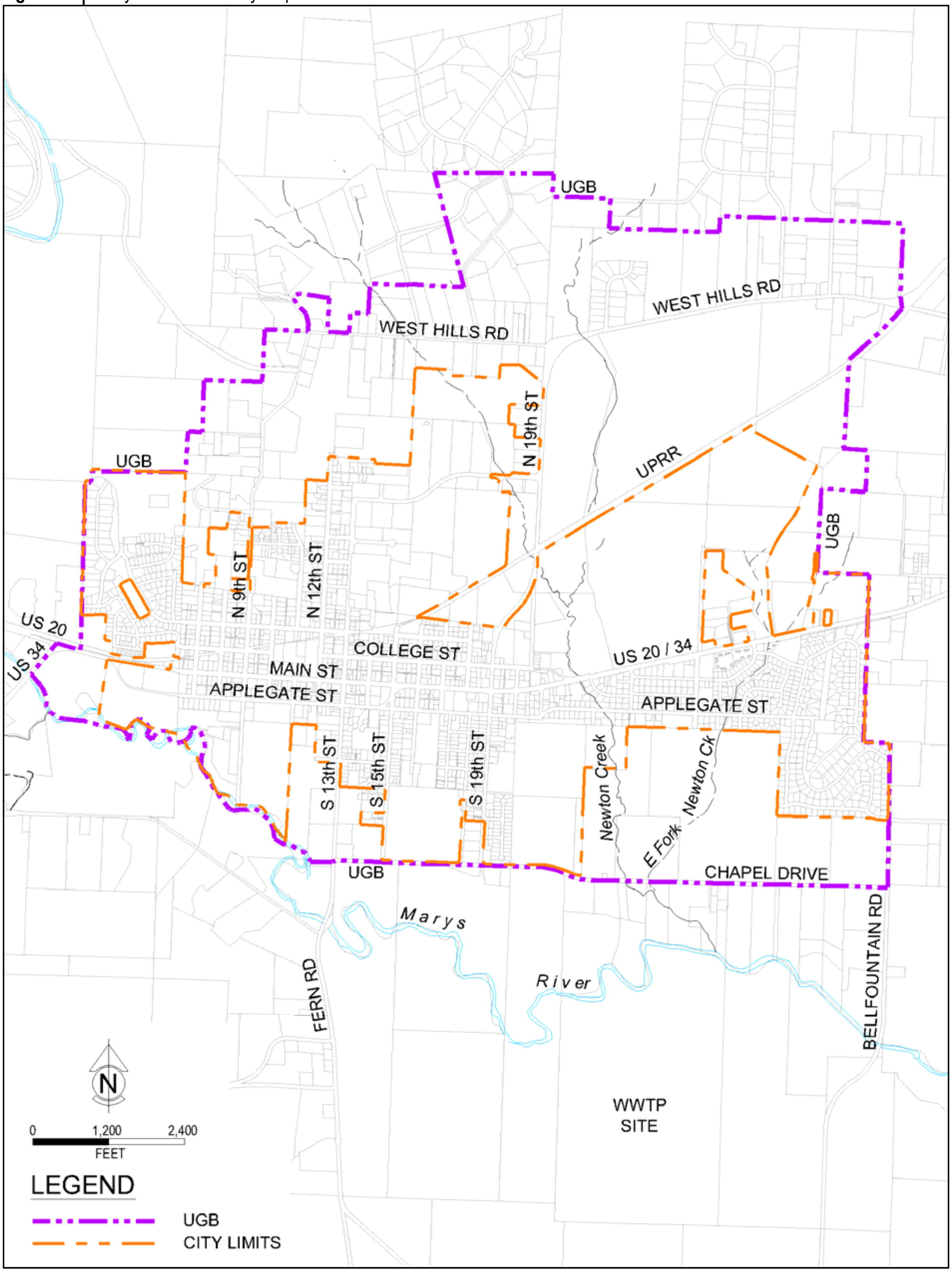


Figure 2-3 | Comprehensive Plan Designations

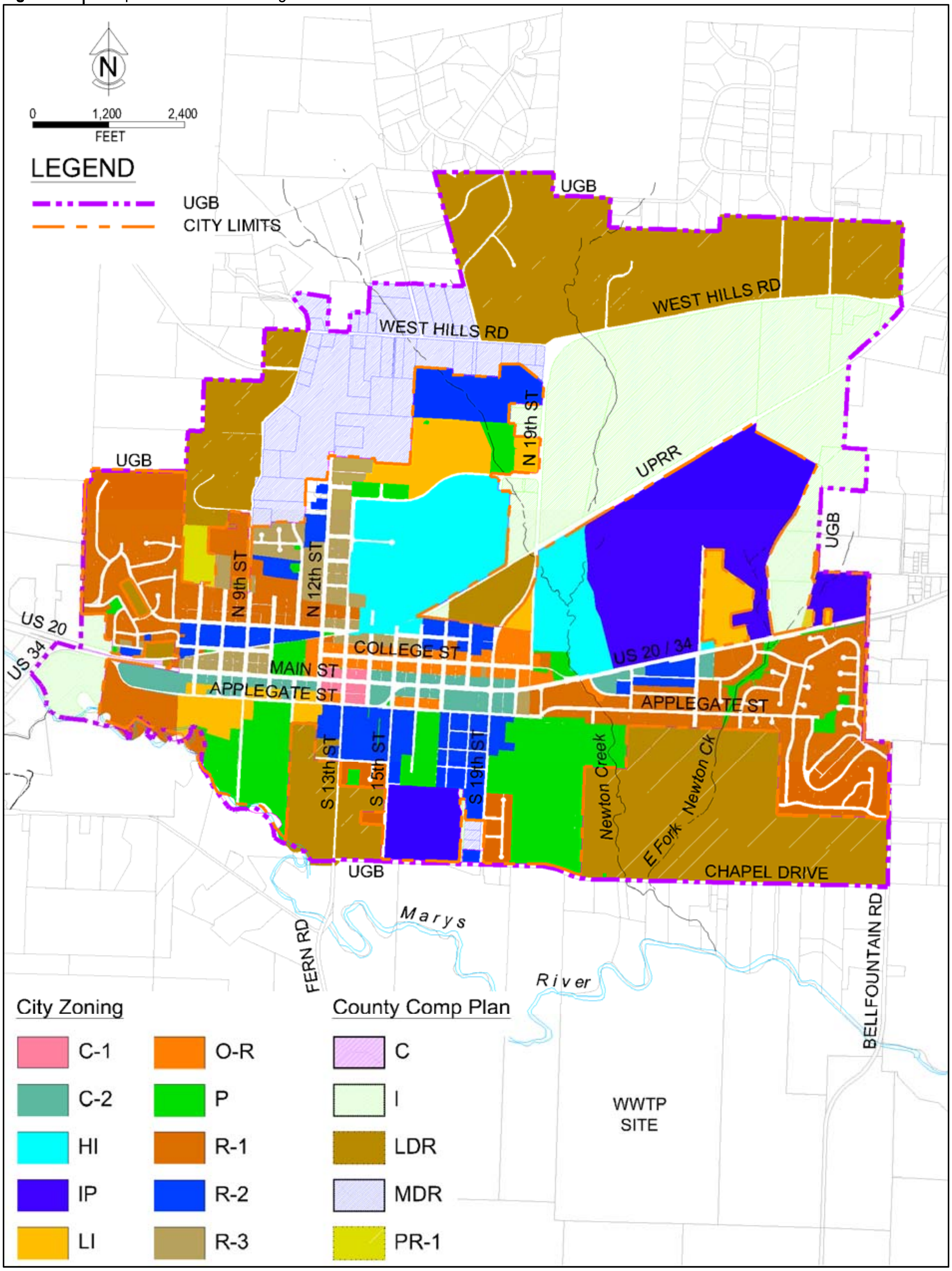


Figure 2-4 | Soils Map

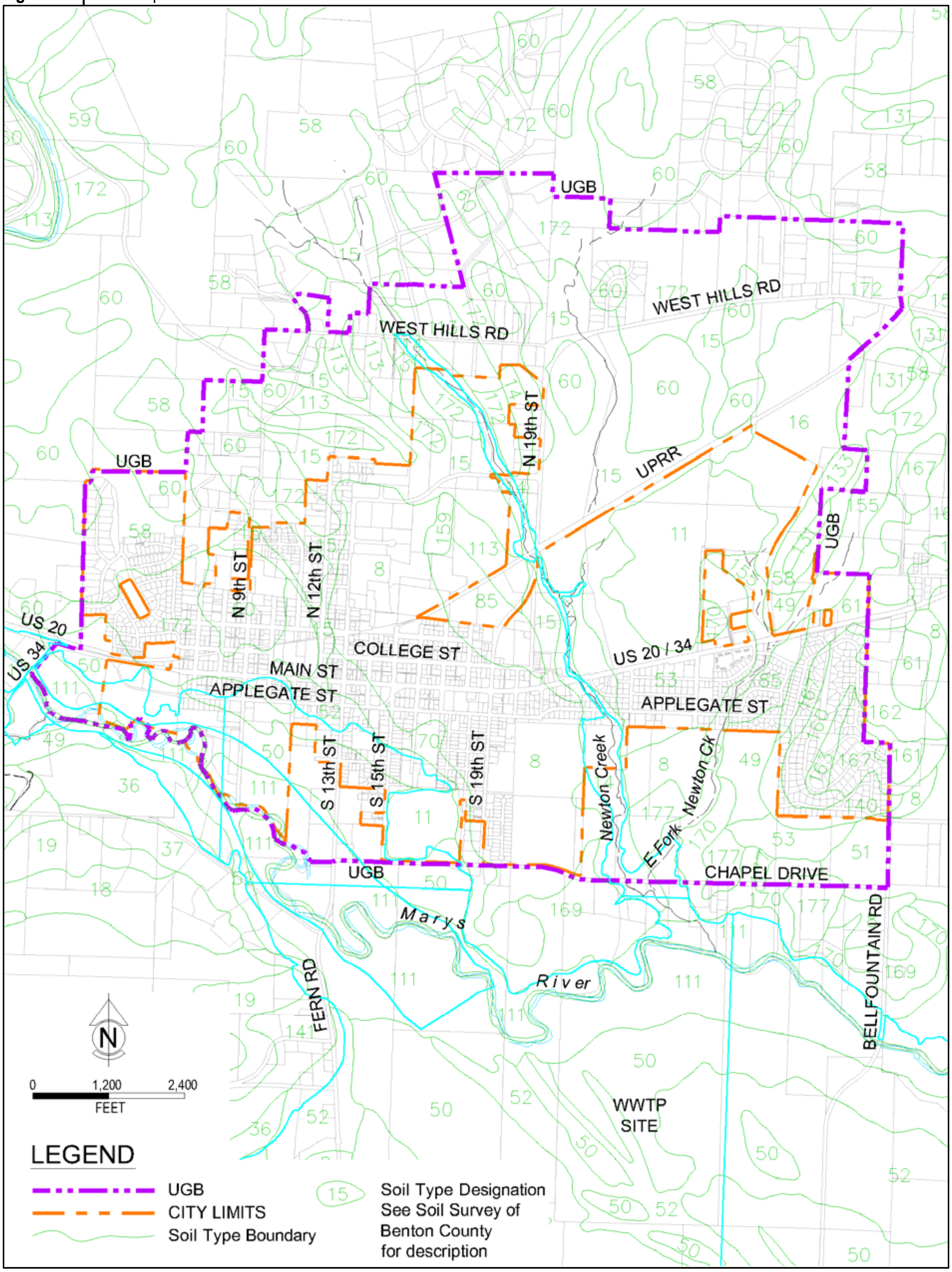


Figure 2-5 | 100 Year Flood Plain Map

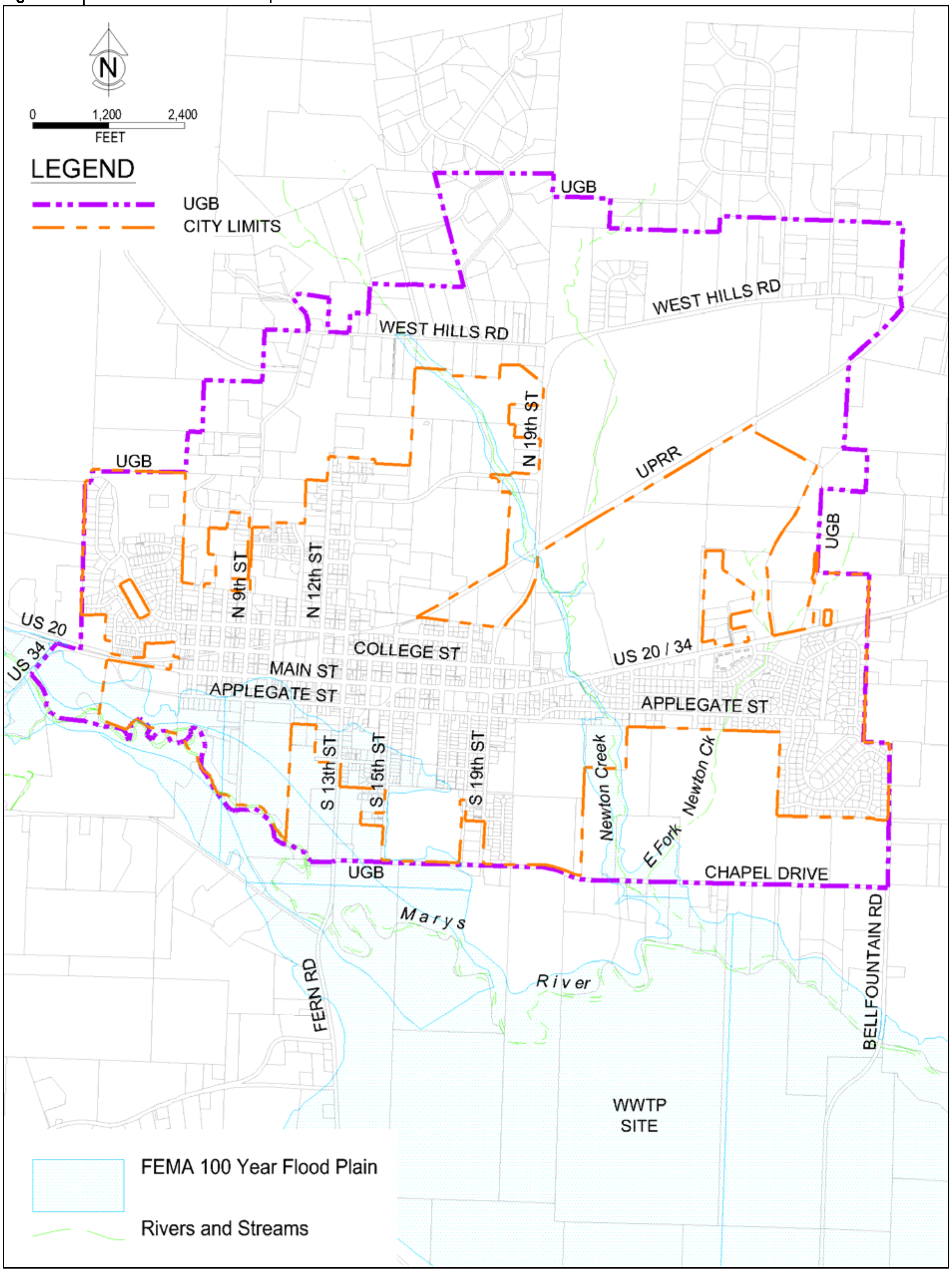
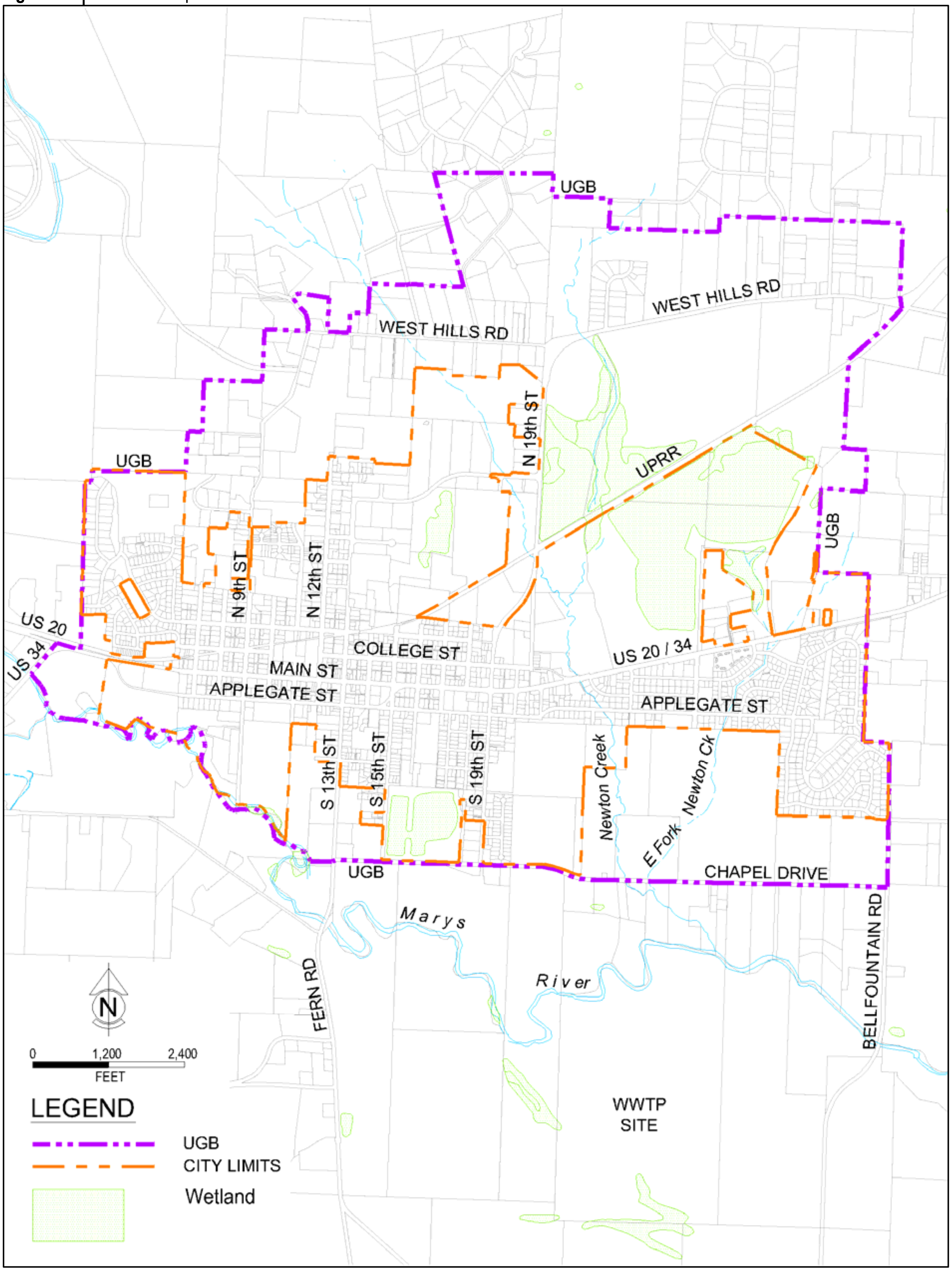


Figure 2-6 | Wetlands Map



CHAPTER 3

BASIS OF PLANNING

Chapter Outline

BASIS OF PLANNING

3.1 INTRODUCTION

The purpose of this chapter is to present an overview of the regulatory requirements as well as the basic design criteria used to develop and evaluate the various alternatives. This chapter presents the common baseline used to evaluate each of the recommended improvements. All of the recommended improvements must meet all applicable regulatory requirements and provide reliable service for a reasonable cost.

3.2 REGULATING AGENCIES

The U.S. Environmental Protection Agency (EPA) regulates disposal and/or reuse of sewage sludge and septage, as well as the discharge of wastewater effluent to surface waters. Subsurface disposal of treated effluent is regulated by the Oregon Department of Environmental Quality (DEQ). The basis of the regulations imposed or overseen by the EPA is the Federal Water Pollution Control Act of 1972 (Public Law 92-500) often referred to as the Clean Water Act (CWA). The scope of the Clean Water Act has been revised and expanded over the subsequent years. The EPA promulgates regulations to implement the requirements of the CWA and subsequent legislation, and is required to coordinate its requirements with other federal agencies such as the National Oceanic and Atmospheric Administration, the U.S. Army Corps of Engineers, the U.S. Fish and Wildlife Service, and with state agencies such as the Department of Environmental Quality (DEQ), the Oregon Department of Fish and Wildlife, and the Department of Health.

In Oregon, the Oregon Department of Environmental Quality (DEQ) is the EPA's delegated agency to implement the Clean Water Act.

3.3 EXISTING PERMIT REQUIREMENTS

The City's existing treatment plant is regulated under a National Pollutant Discharge Elimination System (NPDES) permit issued by DEQ (Appendix A). The existing permit was issued on January 1, 2013 and expired on June 30, 2017. The City is currently permitted to discharge treated effluent to the Marys River from November 1 through April 30 of each year. No discharge to surface waters is allowed from May 1 through October 31. In addition to seasonal limitations, the NPDES permit includes several other limitations with respect to effluent quality and quantity (Table 3-1). The rate at which water can be discharged is also limited by the flowrate in the Marys River (Table 3-2). Operators must check the stream flow in the Marys River on a daily basis and adjust the discharge rate to ensure compliance with Table 3-2. The City utilizes the USGS gauge (14171000) at the Bellfountain Road Bridge to monitor stream flows.

Table 3-1 | Current NPDES Permit Discharge Limitations

NPDES Permit Schedule A, Treated Effluent, Outfall 001(Marys River)
Discharge Permitted November 1 – April 30

Constituent	Max. Concentration (mg/L)		Max. Mass Load (lb/day)		
	Avg. Monthly	Avg. Weekly	Avg. Monthly	Avg. Weekly	Daily
BOD5	30	45	460	690	920
TSS	50	80	760	1100	1500
pH		Range		6.0 – 9.0	
E. coli Bacteria		Monthly Geometric Mean		126 cts/100 ml	
		Maximum Single Sample		406 cts/100 ml	
BOD5 Removal Efficiency		Min. Monthly Average Removal		65%	
TSS Removal Efficiency		Min. Monthly Average Removal		55%	
Total Chlorine Residual		Maximum Monthly Average		0.01 mg/L	

Table 3-2 | Receiving Stream Flow Discharge Rate Limitations

Marys River Flow (cfs)	Maximum Effluent Flow (mgd)
< 25	No Discharge Allowed
25-50	0.25
50-80	0.53
80-100	0.95
100-150	1.10
150-200	1.78
200-250	2.38
250-300	2.98
300-350	3.51
> 350	4.00

In addition to the surface water discharge, the City is also permitted to use recycled water for crop irrigation during the dry weather months. Under the NPDES permit for outfall 002, no discharge to the waters of the state is allowed from May 1 to October 31. All discharge must be land applied in accordance with a recycled water use plan approved by the DEQ subject to the following additional requirements.

- The water must be used and applied at a rate that does not have the potential to adversely impact groundwater quality.
- The water must be applied at a rate in accordance with site management practices that ensure continued agricultural, horticultural, or silvicultural production and does not reduce the productivity of the site.
- The water must be irrigated using sound irrigation practices to prevent:
    - Offsite surface runoff or subsurface drainage through drainage tile
    - Creation of odors, fly and mosquito breeding or other nuisance conditions
    - Overloading of land with nutrients, organics, or other pollutant parameters

The NPDES allows for the production of several classes of recycled water. At the present time, the City's recycled water use plan calls for the production of Class C or Class D recycled water. Class C recycled water must be disinfected to reduce total coliform to 240 organisms per 100 mL in two consecutive samples, and a 7-day median of 23 organisms per 100 mL. Class D recycled water must be disinfected to reduce total *E.coli* counts below a 30-day log mean of 126 per 100 mL without exceeding a count of 406 per 100 mL in any single sample. Class C and Class D also have different property line setback requirements and high-wind shutdown requirements. The City currently distributes recycled water for irrigation at fields located immediately North and West of the existing lagoons.

3.3.1 Mixing Zone

The City discharges effluent to the Marys River through a single port outfall located on the south bank of the River. Federal Regulations and Oregon Administrative Rules allow DEQ to suspend all or part of the water quality standards in small, designated areas around a discharge point. These areas are known as "regulatory mixing zones." The NPDES permit establishes a mixing zone for Philomath's discharge. The mixing zone is defined as that portion of the Marys River where effluent mixes with 25% of the stream flow but in no case may it extend farther than twenty (20) feet towards midstream and extending from a point ten (10) feet upstream of the outfall to a point one hundred (100) feet downstream from the outfall. The Zone of Immediate Dilution (ZID) is defined as that portion of the mixing zone that is within ten feet of the point of discharge. The ZID is a small area where acute water quality criteria can be exceeded as long as it does not cause acute toxicity to organisms drifting through it. The larger mixing zone is an area where acute criteria must be met but chronic criteria can be exceeded.

In 2010, the City completed a mixing zone study for the outfall. This study was required by the DEQ and the results are used by the DEQ to evaluate the City's effluent as part of the NPDES permit renewal process. The mixing zone study showed that the lowest centerline dilution ratio at the edge of the ZID is 2.3 and lowest average dilution ratio at the edge of the mixing zone is 7.5. The mixing zone study included an environmental mapping summary indicating that there are no nearby public recreation areas, no drinking water intakes within 0.5 miles, no cold water refugia, and no other nearby dischargers with NPDES permits.

3.4 Receiving Stream Water Quality

During the months of November through April, the City discharges treated effluent to the Marys River which is a tributary of the Willamette River. The drainage area is 146 square miles at the City's outfall location. Approximately 68.5% of the land use within the basin is forestry. The Marys River exceeds water quality standards for several parameters (Table 3-3) and is, therefore, deemed to be water quality limited for those parameters. For these parameters, the DEQ is required to establish Total Maximum Daily Loads (TMDLs) of pollutants that are believed to affect the particular water quality impairment. The TMDL may assign waste load allocations (WLA) to pollution sources such as Philomath's effluent discharge.

It is unclear how the listings in Table 3-3 will affect the City of Philomath. The DEQ prepared a "Permit Evaluation Report" to support the renewal of the City's existing NPDES permit. This report includes a discussion of these water quality limitations for the Marys River. In the Permit Evaluation Report, the DEQ concluded that the City's discharge does not have the potential for temperature impacts to the receiving stream. Therefore, the temperature limitations should not affect the City as long as the DEQ's position on the matter does not change. The Permit Evaluation Report also states that the water quality standards for Iron and Manganese have changed and that the listings for these parameters are no longer accurate. Again, if the DEQ's position on this holds, it is unlikely these listings will impact the City. The listing for dissolved oxygen has the largest potential to impact the City. As a result of the listing, the DEQ cannot approve a mass load allocation increase for BOD. As such, the City must be able to comply with the existing BOD mass load limits in the NPDES permit. Eventually, the DEQ will prepare a TMDL for dissolved oxygen in the Marys River. At that time, a waste load allocation may be issued for the City that will affect treatment plant operations. It is difficult to speculate on these impacts at this time. The City currently discharges at river mile 10.6. The bulk of the oxygen sag caused by the City's discharge may occur downstream of the Willamette River confluence. Therefore, the City's discharge may have a minimal effect on the dissolved oxygen in the Marys River. If the City's discharge is found to have a significant effect on Marys River dissolved oxygen levels, the DEQ may add additional limits to the City's permit. Again, it is difficult to speculate on these limits at this time. They could be relatively minor and have little effect on the City's day to day operation or they could require the City to make major improvements to the wastewater treatment plant. At this time, it is unknown when the DEQ will issue a TMDL for dissolved oxygen in the Marys River. This may not occur for many years. As such, this planning document is based on the assumption that the City's existing discharge limits will remain essentially unchanged during the planning period.

Table 3-3 | Marys River Water Quality Limitations

Waterbody Name	Listed River Mile	Parameter	Season	TMDL Completed
Marys River	0 to 41.1	Dissolved Oxygen	January 1 – May 15	No
Marys River	0 to 41.1	Iron	Year Around	No
Marys River	0 to 41.1	Manganese	Year Around	No
Marys River	0 to 41.1	Temperature	Summer Rearing	Yes

3.5 GROUNDWATER PROTECTION

Groundwater is a critical natural resource providing domestic, industrial, and agricultural water supply as well as other beneficial uses. Groundwater also provides base flow for rivers, lakes, streams, and wetlands. All groundwater in the state is protected from pollution. Oregon's groundwater protection rules are described in OAR 340-040. With respect to the City's wastewater utility, the facultative lagoons and the land application facilities have the highest potential to impact groundwater quality. The lagoons were constructed with clay liners to minimize seepage loss. The two lagoons that were constructed in the mid 1980s (i.e., cells 2 & 3) were tested for seepage in 2002. The most recently constructed lagoon (cell 1) was tested for seepage in 2012. These test showed that leakage from the lagoons was less than 1/8 – inch per day. This indicates that the lagoons are not leaking in excess of DEQ's guidelines. In addition, the City's NPDES permit does not require any further evaluation of groundwater impacts. There is no evidence to suggest the City's existing lagoons impact ground water quality. Therefore, improvements to the lagoon liner system are not likely to be needed during the planning period.

Land application of recycled water is performed in accordance with the DEQ approved recycled water use plan. In accordance with the recycled water use plan, the application rate of recycled water is matched to agronomic uptake rates. As such, the potential for recycled water to impact groundwater quality is low.

3.6 WASTEWATER RECYCLING

An alternative to direct discharge to surface water is to recycle the treated effluent for other uses such as irrigation or industrial process water. The City of Philomath currently uses recycled water for irrigation of the agricultural fields north and west of the existing lagoons.

Reuse of effluent by land application is governed by OAR 340-055, Recycled Water Use, and groundwater quality is governed by OAR 340-040, Groundwater Quality Protection. Per OAR 340-055 recycled wastewater is characterized in five classes including Class A through D and Non-disinfected water. These classes range in quality from Class A being the most treated to Non-disinfected water being the least treated. Each wastewater class has different treatment and testing requirements and beneficial purposes. The treatment requirements and possible beneficial uses described in the rules are summarized in Table 3-4 and Table 3-5.

Table 3-4 | Treatment & Monitoring Requirements for use of Recycled Water

Reuse Class	A	B	C	D	Non-Disinfected
Minimum Treatment Required	Oxidation, filtration & disinfection	Oxidation & disinfection	Oxidation & disinfection	Oxidation and disinfection	Oxidized
Parameter - Total Coliform (number/100 mL)					
7 day median	2.2	2.2	23	No Limit	No limit
Maximum single sample	23	23	240	No limit	No limit
Parameter – E. coli (number /100 mL)					
30 day LOG mean	Not Required	Not Required	Not Required	126/100ML	No limit
Maximum Single Sample	Not Required	Not Required	Not Required	406/100ML	No limit
Parameter – Turbidity Prior to Disinfection (NTU)					
24 hour mean	2	No limit	No limit	No limit	No limit
5% of the time during any 24 hour period	5	No limit	No limit	No limit	No limit
Maximum any sample	10	No limit	No limit	No limit	No limit
Minimum Monitoring Requirements					
Total Coliform	Daily	3/week	1/week	Not Required	As in NPDES or WPCF Permit
Turbidity	Hourly	Not Required	Not Required	Not Required	Not Required
E. Coli	Not Required	Not Required	Not Required	1/week	Not Required
Public Access					
	Controlled: Same as Class D for some uses and unrestricted for others	Controlled: Same as Class D	Controlled: Same as Class D plus direct contact restrictions for some uses	Controlled: Notification of staff and signs posted around the perimeter of use area	Prevented: fences, gates, locks
Set-Back Requirements					
From property line where irrigation is applied directly to the soil	None	10 feet	10 feet	10 feet	Site specific
From property line where sprinkler irrigation is used	None	50 feet	70 feet	100 feet	Site specific
From food preparation or serving area or drinking fountain to edge of sprinkler irrigation	Cannot be sprayed directly on to use area	10 feet	70 feet	70 feet	Site specific
From edge of irrigation to water supply source for human consumption	None	None	100 feet	100 feet	150 feet

Table 3-5 | Allowable Uses for Recycled Water

Beneficial Purpose	Class A	Class B	Class C	Class D	Non-disinfected
Irrigation					
Fodder, fiber, seed crops not intended for human ingestion, commercial timber	Yes	Yes	Yes	Yes	Yes
Firewood	Yes	Yes	Yes	Yes	No
Sod	Yes	Yes	Yes	Yes	No
Pasture for animals	Yes	Yes	Yes	Yes	No
Processed food crops	Yes	Yes	Yes	No	No
Orchards or vineyards if an irrigation method is used to apply recycled water directly to the soil	Yes	Yes	Yes	No	No
Golf Courses, cemeteries, highway medians, industrial or business campuses	Yes	Yes	Yes	No	No
Any agricultural or horticultural use	Yes	No	No	No	No
Parks, playgrounds, school yards, residential landscapes, other landscapes accessible to the public	Yes	No	No	No	No
Industrial, Commercial, or Construction					
Industrial cooling	Yes	Yes	Yes	No	No
Rock crushing, aggregate washing, mixing concrete	Yes	Yes	Yes	No	No
Dust control	Yes	Yes	Yes	No	No
Nonstructural fire fighting using aircraft	Yes	Yes	Yes	No	No
Street sweeping or sanitary sewer flushing	Yes	Yes	Yes	No	No
Stand alone fire suppression systems in commercial and residential buildings	Yes	Yes	No	No	No
Non-residential toilet or urinal flushing, floor drain trap priming	Yes	Yes	No	No	No
Commercial car washing	Yes	No	No	No	No
Fountains when the water is not intended for human consumption	Yes	No	No	No	No
Impoundments or Artificial Groundwater Recharge					
Water supply for landscape impoundments including, but not limited to, golf course water ponds and non-residential landscape ponds	Yes	Yes	Yes	No	No
Restricted recreational impoundment	Yes	Yes	No	No	No
Nonrestricted recreational impoundments including, but not limited to, recreational lakes, water features accessible to the public, and public fishing ponds	Yes	No	No	No	No
Artificial groundwater recharge	Yes	No	No	No	No

3.7 SLUDGE STABILIZATION REQUIREMENTS

Sludge accumulates in the lagoons and must be removed periodically. The regulations regarding sludge stabilization and disposal are summarized in this subsection. Sludge that is not stabilized may be disposed at a landfill. However, most other disposal methods require that the sludge be stabilized in accordance with the requirements described in this subsection.

The term "sludge" refers to the solids that settle and are removed when a liquid with suspended solids passes through a settling basin or tank. Sludge may originate from several sources in a wastewater treatment plant, but can typically be classified as either raw or primary sludge (primary settling of untreated sewage) or secondary sludge (excess biological sludge from secondary treatment processes). All sludge must be stabilized prior to reuse or disposal. Stabilized sludge is a mixture of solids and liquids that is one of the end products of the wastewater treatment process. Adequately processed sludge is classified in regulations as "biosolids." It is commonly disposed of by applying it to agricultural or forest land after adequate processing.

3.7.1 Biosolids Quality

Wastewater biosolids are subject to differing regulations and restrictions based on quality. The Code of Federal Regulations (40 CFR 503) defines standards for three measures of biosolids quality:

- Pathogens
- Vector attraction (the tendency of the sludge to attract rodents, insects and other organisms that can spread disease)
- Trace elements

Biosolids that meet the higher of two standards for all three of these measures are designated exceptional quality (EQ) biosolids. EQ biosolids have fewer reporting and monitoring requirements and virtually no restrictions on use. Use is restricted for biosolids that do not meet the higher standard by any of these three measures. The following is a short discussion of each of these measurements of biosolids quality.

3.7.2 Pathogen Requirements

Pathogen requirements define two classes of biosolids - Class A and Class B. Class A is the higher standard and requires complete destruction of pathogens before disposal. Class B requirements call for reducing pathogens before disposal and applying the biosolids to land in such a way that pathogens are further reduced.

To be classified as Class A, biosolids must be treated using one of the EPA's Processes to Further Reduce Pathogens (PFRP), or an equivalent process. These processes include composting, heat drying, heat treatment, thermophilic aerobic digestion, beta ray irradiation, gamma ray irradiation, and pasteurization. Regardless of the process used, Class A biosolids must not exceed maximum allowable fecal coliform density or Salmonella bacteria density.

Class B biosolids must be treated using one of the EPA's Processes to Significantly Reduce Pathogens (PSRP), or an equivalent process. These processes include aerobic digestion, air drying, anaerobic digestion, composting, and lime stabilization.

3.7.3 Vector Attraction Requirements

Biosolids must meet one of the following requirements for reducing vector attraction if they are to be applied to land without restrictions:

- Volatile solids in the sludge shall be reduced by a minimum of 38 percent.
- The specific oxygen uptake rate for sludge treated by aerobic digestion shall be less than or equal to 1.5 mg oxygen per hour per gram of total solids at a temperature of 20°C.
- Aerobic processes shall treat the sludge for a minimum of 14 days with an average temperature of at least 45°C and a minimum temperature of 40°C.
- Alkali addition shall raise the pH of the sludge to a minimum of 12 for two hours and maintain the pH at a minimum of 11.5 for an additional 22 hours without additional alkali.

The use of the land where the biosolids is applied is restricted if vector attraction reduction is achieved by measures, such as injecting the biosolids below the surface of the land or disposing of them on the surface and incorporating them into the soil within six hours.

3.7.4 Trace Elements

Ten elements typically found in biosolids have been identified as critical. Two limits have been set for each of these trace elements: Exceptional Quality (EQ) and a ceiling limit. If all the trace elements are below the EQ limit, then no restrictions are placed on loading rates. If any of the trace elements are over the ceiling limit, then the biosolids are not suitable for land application. If the trace elements fall between these two limits, restrictions are placed on loading rates.

3.7.5 Biosolids Use

Table 3-6 outlines some of the general restrictions on the use of biosolids depending on the quality of the biosolids.

Table 3-6 | Biosolids Use Restrictions Based on Quality Rating

Biosolids Quality Rating by Category			
Pathogens	Vector Attraction	Trace Elements	Use Restrictions
EQ	EQ	EQ	No restrictions are imposed on application or use with regard to pathogens, vector attraction, or trace elements.
Class B	EQ	EQ	Application is subject to EPA defined waiting periods for crops, grazing, and public access. Biosolids cannot be distributed for home use, in bags, or in containers.
EQ	-	EQ	Biosolids must be injected or tilled into the soil. Biosolids cannot be distributed for home use, in bags, or in containers.
EQ	EQ	-	Bulk application must not exceed EPA defined cumulative loading rates. Biosolids distributed in bags or containers are subject to annual loading rate restrictions.

All Other Biosolids Qualities	Application is subject to trace loading requirements and pathogen waiting periods. Biosolids must be injected or tilled into the soil and cannot be distributed for home use, in bags, or in containers.

EQ – Exceptional Quality Biosolids

3.7.6 Biosolids Land Application Site Criteria

Site criteria for land applying biosolids includes geological formation, flood plain proximity, groundwater and surface water proximity, topography, and soils, as well as method of application. Table 3-7 contains an overview of some of the general criteria contained in OAR 340-050.

Land application of biosolids at sites used for agricultural purposes requires special management considerations. These relate to access to the site, types of crops grown, plant nutrient-uptake rates, timing and duration of biosolids application (i.e., site life and seasonal constraints), and grazing restrictions. A brief discussion of each of these issues follows.

- Access. Controlled access must be provided for municipal biosolids application sites for 12 months following surface application of biosolids. Controlled access is defined as public entry or traffic being unlikely. Privately owned rural land is typically assumed to have controlled access, while public lands such as parks may require fencing to ensure access control.
- Crops. Biosolids or biosolids derived products are not to be used directly on fruits or vegetables which may be eaten raw. As a general rule, crops grown for human consumption should not be planted within 18 months of application of municipal biosolids. If the edible parts will not be in contact with the biosolid amended soil, or if the crop will be processed or treated prior to marketing in such a manner to ensure that pathogen contamination is not a concern, this requirement may be waived by DEQ. There are no restrictions on planting times for crops not grown for direct human consumption.
- Nutrient Loading. Biosolids application to agricultural land should not exceed the annual nitrogen loading required for maximum crop yield. Biosolids are, therefore, typically managed according to their fertilizer value. Biosolids may be applied above agronomic rates on a onetime basis or less than once per year so long as runoff, nuisance conditions, and groundwater concerns are adequately addressed. In cases of higher than agronomic application rates, the acceptable loading rate and application frequency is typically based on nitrogen accumulation and annual nitrogen use.
- Site Life. Sites generally have a limited application life, which may be determined by the chemistry of the soil and the metals loading from the biosolids. Site life is determined by dividing lifetime biosolids loading limits (based on the most limiting constituent) by the annual application rate.
- Seasonal Constraints. The main consideration in land applying on sloping ground is to avoid surface runoff and soil erosion. Additionally, biosolids application should be restricted to the

dry season to prevent soil damage that may occur from equipment traffic during the wet season.

- Grazing Restrictions. Grazing animals should not be allowed on pasture or forage for 30 days after application of stabilized biosolids, 180 days after application of non-stabilized biosolids, and 7 days after application of air-dried biosolids.
- Site Monitoring and Reporting. As previously noted, site monitoring is typically not required where "EQ" biosolids are applied at or below agronomic rates based on crop nitrogen requirements. However, if the biosolids contain high concentrations of heavy metals or other toxic elements, or if crop nitrogen requirements are exceeded on a regular basis, soil monitoring and special management practices may be required. At the discretion of DEQ, monitoring wells and groundwater background characterization and/or monitoring may be required on any site on a case by case basis.

Table 3-7 | Site Criteria for Biosolids Application

Parameter	Criteria
Geology	Must have a stable formation
Within Flood Plain	Restricted period of application and incorporation of biosolids
Groundwater	At time of application; 4-foot minimum depth to permanent groundwater; 1-foot minimum depth to temporary groundwater
Topography	Must have appropriate management to eliminate surface runoff
Slope less than or equal to 12%	• Surface application of liquid dewatered or dried biosolids
Slope greater than 12% but less than 20%	• Direct incorporation of liquid biosolids into the soil, surface application of dewatered or dried biosolids
Soils	• Minimum rooting depth of 24 inches • No rapid leaching • Avoid saline or alkali soil • pH of 6.5 to 8.2 for heavy metal accumulator crops, or pH can be raised by adding lime to the soil.
Method of Application & Proximity to Water Bodies	Buffer strips may be required to protect water bodies. Size depends on method of application and proximity to sensitive area (determined at discretion of DEQ), generally as follows: • Direct injection: no limit required • Truck spreading: less than 50 foot buffer strip • Spray irrigation: 300 to 500 foot buffer strip • Near ditch, pond, channel, or waterway: greater than 50 foot buffer strip • Near domestic water source or well; greater than 200 foot buffer strip

3.8 RELIABILITY AND REDUNDANCY REQUIREMENTS

The EPA has established minimum standards for mechanical, electrical, fluid systems, and component reliability for all new or expanding sewerage facilities, including treatment plants. These reliability standards establish minimum levels of reliability for three classes of sewerage facilities. Pump stations associated with, but physically removed from the actual treatment works may have a different classification than the treatment works itself.

The purpose of these reliability standards is to ensure that the treatment facilities will operate effectively on a day-to-day basis and that provisions are made for operation during power failures, flooding, peak loads, equipment failures, and maintenance shutdowns. These reliability and redundancy standards are designed to ensure that unacceptable degradation of the receiving water will not occur due to the interrupted operation of specific treatment process or unit operation.

The reliability classification will be based on the water quality and public health consequences of a component or system failure. Specific requirements pertaining to treatment plant unit processes for each reliability class are described in EPA's technical bulletin, Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability, EPA 430-99-74-001. EPA and DEQ guidelines for classifying sewerage works are summarized as follows:

- Reliability Class 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.
- Reliability Class 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.
- Reliability Class III. These are works not otherwise classified as Reliability Class I or Class II.

For this Facilities Plan, it is assumed that all treatment plant and pump station improvements will be designed to EPA Reliability Class I standards. Table 3-8 contains the typical redundancy requirements for treatment plant and pump station components that are designed in accordance with the EPA Reliability Class I standards. In addition to the standards listed in the table, unit operations must be designed to pass the peak hydraulic flow with one unit out of service. Mechanical components in the facility must also be designed to enable repair or replacement without violating the effluent limitations or causing diversion of untreated sewage. The information in this table is not specific to the proposed alternative, and some of the plant components shown are not necessarily included in the existing or future facilities. Some of the items listed below apply regardless of the reliability classification of the treatment facility.

Table 3-8 | EPA Reliability Class I Requirements

System Component	Capacity/Redundancy Requirements
Raw Sewage Pumps	Handle peak flow with largest unit out of service. As a minimum, the Peak flow is defined as the flow associated with a 5-year, 24-hour storm.
Mechanical Bar Screens	Provide one backup with either manual or mechanical cleaning (manual cleaning acceptable if only two screens)
Grit Removal	Provide a minimum of two units.
Primary Sedimentation	Handle 50% of design flow capacity with largest unit out of service. Design flow is defined as the flow used as the design basis of the component.
Activated Sludge Process	A minimum of two equal size basins. No backup basin required.
Aeration Blowers	Supply the design air capacity with the largest unit out of service. Provide a minimum of two units.
Air Diffusers	Allow for the isolation of largest section of diffusers (within a basin) without measurably impairing oxygen transfer.
Secondary Sedimentation	Handle 75% of design flow capacity with largest unit out of service. Design flow is defined as the flow used as the design basis of the component.
Disinfection Contact Basin	Handle 50% of the design flow with largest unit out of service. Design flow is defined as the flow used as the design basis of the component.
Effluent Pumps	Handle peak flow with largest unit out of service. Peak flow is defined as the maximum wastewater flow expected during the design period of the treatment works.
Electrical Power	Two separate and independent sources of electrical power shall be provided, either from two separate utility substations or from a single substation and a plant based generator. Designated backup source shall have sufficient capacity to operate all vital components, critical lighting, and ventilation during peak flow conditions, except that components used to support the secondary processes need not be included as long as treatment equivalent to sedimentation and disinfection is provided.

3.9 COLLECTION SYSTEM DESIGN CRITERIA

The requirements and regulations covering the design and sizing of the collection piping portion of the wastewater conveyance system include both City design standards and DEQ guidelines. The City has Public Works Design Standards that apply to all public sewer improvements within existing and proposed public right-of-way and public utility easements, as well as to all improvements to be maintained by the City. This includes both gravity collection piping and pump stations.

The City design criteria dictates that the collection system piping must be designed to convey all flows projected at the ultimate development of land within the tributary area based on current land use designations. Although this may result in capacities greater than those needed during the 20-year planning period, sewage collection lines are, by their very nature, unsuited for incremental expansion without extensive capital outlays. Under DEQ guidelines, there is one allowable exception to this requirement as it relates to large diameter trunk sewers serving tributary areas that are not expected to develop for 30 or more years. However, none of the proposed new gravity sewers within the study area fall under this category.

The City Public Works Design Standards and associated details implement and clarify current DEQ standards as contained in OAR 340-052, Appendix A and DEQ design guidelines. Table 3-9 includes a list of the minimum allowable slope based on mainline pipe sizes.

Table 3-9 | Minimum Mainline Pipe Slopes

Inside Pipe Diameter (inches)	% Slope (ft/100 ft)
8	0.40
10	0.28
12	0.22
15	0.15
18	0.12
21	0.10
24	0.09
27	0.08

3.10 PUMP STATION AND FORCEMAIN DESIGN CRITERIA

DEQ has extensive design guidelines for public pump stations. Under the authority granted by OAR 340-052, DEQ has established requirements and guidelines for the design of public sanitary sewer pump stations. These design guidelines include OAR 340-052 Appendix B and various design memoranda issued by DEQ. DEQ has established 20-years as being the proper planning period for pump stations. Table 3-10 below summarizes design criteria assumed for new pump stations or the upgrades of the existing pump stations.

Table 3-10 | City Pump Station Minimum Design Criteria

Category	Minimum Design Criteria
Design Flows	• 20-year peak instantaneous flow (5 yr, 24 hour storm)
Pump Station Structure	
• Wetwell Type	• Precast concrete, hatches with integral hatches/fall protection
• Operational Storage	• Based on pump starts or overflow storage as appropriate
• Valve Vault	• Precast concrete vault adjacent to wetwell
• Overflow	• Provide bypass in accordance with DEQ historical design requests.
Pumps	
• Pump Station Capacity	• Convey design flow with largest single unit out of service
• Type	• Submersible pumps
• Number	• 2 minimum
• Motor Size	• HP as required, 480 volt, 3 phase power preferred
• Min. Pump Cycle Time	• 6 minutes (10 starts per hour total)
• Pump Retrieval	• Jib or davit crane installed on or adjacent to wetwell
Force Mains	
• Minimum Size & Material	• 4-inch, C-900 PVC, Class 52 Ductile Iron or fused HDPE
• Min Velocity / Max Velocity	• 3.5 fps / ±8 fps
Instrumentation & Control System	
• Location	• Building adjacent to pump station
• Control Building	• CMU block
• Pump Speed Control	• Soft starters or VFDs if required by City or utility company
• Flow Measurement	• Mag meter in vault downstream of valve.
Auxiliary Power	
• Type	• Permanent diesel generator w/ATS
• Location	• Control building adjacent to P.S.
• Fuel Supply	• Sub-base tank, 24 hour minimum or as required by City
• Silencer	• Critical grade, insulated
Telemetry	
• Type	• Match City system, programmed per City direction
• Alarms	• Remote alarms as required by City

CHAPTER 4

EXISTING WASTEWATER FACILITIES

Chapter Outline

EXISTING WASTEWATER FACILITIES

4.1 INTRODUCTION

This chapter provides an inventory of the existing wastewater system components that serve the study area. This inventory includes a description of funding mechanisms and operation and maintenance budgets. The evaluation of these specific systems and the development of improvement alternatives are performed in other chapters of this study.

The City of Philomath operates and maintains the wastewater system that provides sanitary sewer service to customers within the City Limits. The City's system currently serves approximately 1,569 user accounts. The City's municipal wastewater system consists of a conventional gravity collection system with three wastewater pump stations. Discharge from the pump stations is conveyed to the treatment plant in a common forcemain pipe. The treatment plant includes a headworks, three facultative lagoon cells, and a gas chlorine disinfection system. During the winter months, treated effluent is discharged to the Marys River through a single port outfall pipe. During the summer months, treated effluent is used for irrigation of fields adjacent to the lagoons.

4.2 GENERAL OVERVIEW OF EXISTING WASTEWATER FACILITIES

Philomath's wastewater facilities consist of a conventional gravity collection system that conveys wastewater from the users to one of three pump stations. In general, the collection system serving the western portion of town conveys flows to Pump Station A. Pump Station A is located at the City shops complex near the south end of 16th street. The collection system serving the eastern portion of town conveys flows to the Newton Creek Pump Station. The Newton Creek Pump Station is located on Chapel Drive west of the Newton Creek Bridge. The Timber Estates Pump Station is a minor pump station located on Chapel Drive west of the Newton Creek Pump Station. This pump station serves a moderately sized residential subdivision known as Timber Estates. The three stations pump wastewater through a common force main to the treatment plant located south of the Marys River on the west side of Bellfountain Road. The treatment plant consists of a headworks for flow measurement and influent sampling, three facultative lagoon cells, a chlorination building with chlorine disinfection equipment, a laboratory building, a chlorine contact chamber, an outfall to the Marys River, an irrigation pump station, and land application sites. An overall schematic representation of the existing wastewater pumping and treatment system including pump stations and force mains is presented in Figure 4-1. A detailed collection system map is included in Appendix B. The reader is encouraged to refer to these figures throughout the following discussion.

4.3 HISTORY AND DEVELOPMENT OF WASTEWATER SYSTEM

Philomath's original sewer system was built in 1952. It served most of the area within the present City limits west of Newton Creek. Concrete, mortar joint pipe was used for the sewer construction. Two sewage lift stations were built at that time. One was located at the old sewage treatment plant and was intended to lift all sewage into the treatment plant. This was known as pump station A. At that time the sewage treatment plant was located at the City shops facility near the south end of 16th Street. A second lift station was constructed near the High School on Applegate Street. This lift station was known as pump station B and originally served the portion of the City east of 16th Street on the north side of Main Street and all of the area east of 19th Street on the south side of Main Street. Pump station B discharged through a force main into Manhole #117 on Applegate Street where sewage flowed by gravity to pump Station A. Pump stations A and B essentially divided the City into two distinct drainage basins. This configuration remained until the mid 1980's, when large-scale sewer improvements were constructed.

In 1985, the capacity of pump station A was increased. The station was converted to a submersible pump station with four new submersible pumps, new pump controls, new discharge piping, and a new top slab on the wet well. In 1986 the City constructed large-scale sewerage system improvements that changed the overall configuration of the collection and treatment system. As part of this project, pump station B was abandoned. This reduced flows to pump station A, since pump station B discharged into the collection system that drained to pump station A. As part of the 1986 project, flows from the collection system that originally drained to pump station B were rerouted to a new pump station on Chapel drive. This new pump station is known as the Newton Creek Pump Station, and is currently in service. In order to reroute flows, the sewer between Manhole #1 and Manhole #202A was reconstructed. The slope of this segment was reversed so that flow occurred from Manhole #1 to Manhole #202A. A new 21-inch gravity trunk sewer was constructed to convey flows south of Manhole #202A to the Newton Creek Pump Station.

The 1986 project also included the construction of a new treatment plant south of the Marys River. New forcemain piping was constructed to convey flows from Pump Station A and the Newton Creek Pump Station to the new treatment plant, From pump station A, a new 14-inch ductile iron force main was constructed south to Chapel Drive and east along Chapel Drive where it joined with a common 18-inch ductile iron force main at the Newton Creek Pump Station. Flows from both pump stations are conveyed through a common force main east of the Newton Creek Pump Station across Newton Creek and south to the new treatment plant.

The 1986 project included a new treatment plant that was designed as a summer-holding winter-discharge facility with a headworks, two facultative lagoons, and disinfection facilities. A new single-port discharge into the Marys River was also constructed. Once the new plant was placed into service, the existing plant near the City shops was decommissioned.

The Timber Estates Pump Station was constructed in the 1990's to convey wastewater from the Timber Estates residential development located northwest of the intersection of Chapel Drive and 19th Street. The Timber Estates Pump Station discharges into the 14-inch ductile iron force main between Pump Station A and the Newton Creek Pump Station.

Between 2005 and 2011 the City made significant improvements to the wastewater utility. In 2009, the City completed the construction of an entirely new pump station to replace Pump Station A. The new Pump Station A is located behind the current Public Works Office. The City also installed a new control system and a new crane for pump removal at the Newton Creek Pump Station. In 2011, the City completed a major upgrade to the wastewater treatment plant. This project included a new headworks, a new facultative lagoon cell, a new irrigation pump station, and an irrigation distribution and sprinkler system for land application of recycled water during the dry weather months. Since 2005, the City has also rehabilitated large sections of the original 1952 collection system. As of the summer of 2016, the City has rehabilitated, replaced, or abandoned approximately 13,000 feet, or 38% of the original 1952 collection system. The rehabilitation work has generally included the rehabilitation or replacement of the mainlines, manholes, public service laterals and the private service laterals. In addition to this work, the City is currently planning to complete the rehabilitation of an additional 5,000 feet of the 1952 system in the summer of 2017.

4.4 WASTEWATER COLLECTION SYSTEM

The City's existing sanitary sewage collection system collects wastewater from residences, businesses, industries, and public facilities and conveys the water to one of three pump stations where the water is pumped to the treatment plant. This chapter provides an overview of the existing wastewater collection system within the study area with an emphasis on flow routing and known and reported problems.

Although all public sewers within the study area are owned by the City, three entities have jurisdiction over the right-of-ways within which the sewer mainlines are located. In addition to the City, the Oregon Department of Transportation (ODOT) has jurisdictional oversight for facilities constructed within the Highway 20/34 right-of-way. Benton County has jurisdictional oversight for sewer facilities constructed within County right-of-ways such as 13th Street, 19th Street, 20th Street, Chapel Drive, and West Hills Road.

4.4.1 Service Area and User Connections

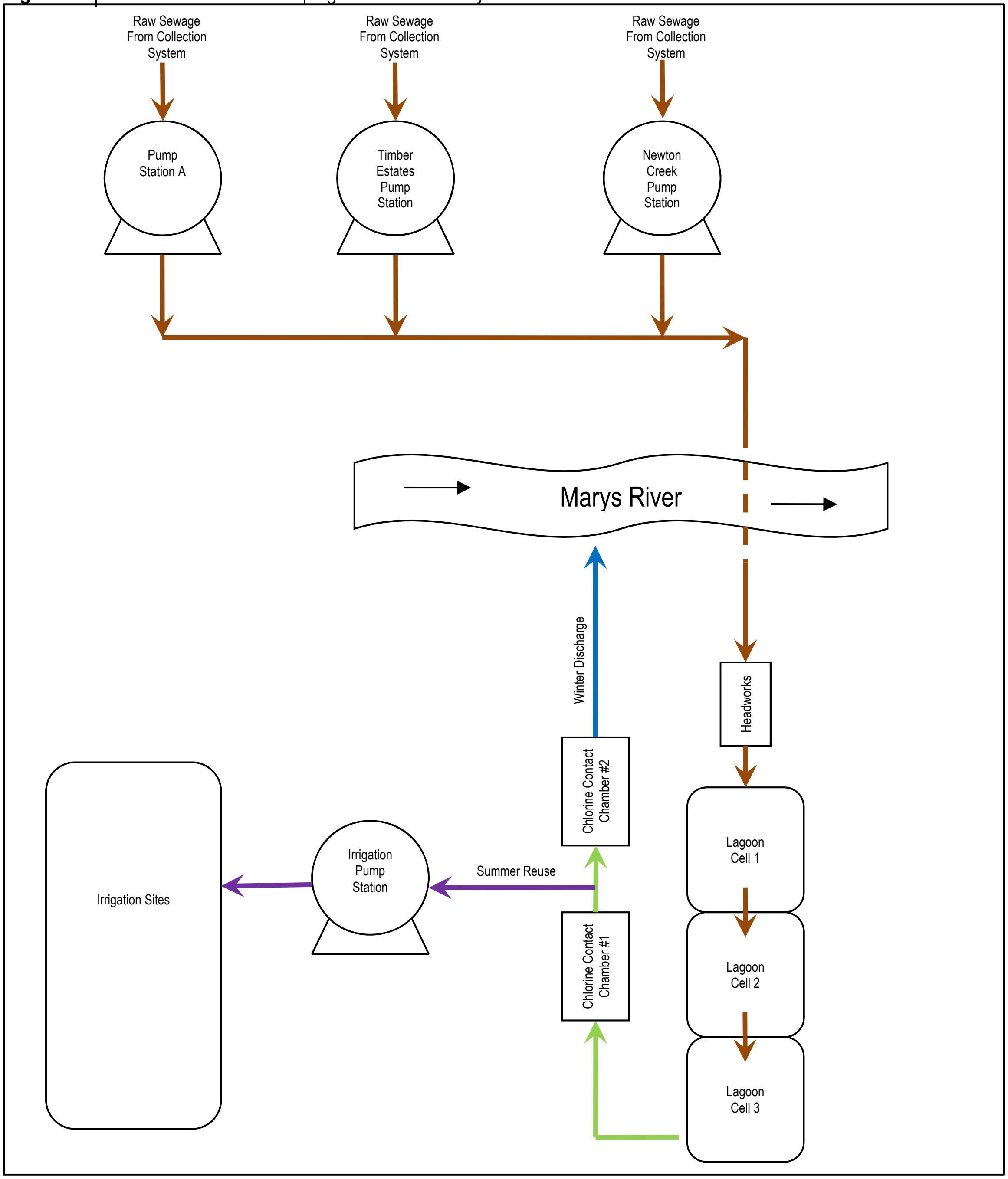
The City's system currently serves approximately 1,569 user connections. These connections consist of dwellings, commercial services, and industrial services (Table 4-1). There are no large industrial or commercial users that currently discharge to the sewer system.

Table 4-1 | Sewer User Summary

User Classification	City System
Residential	1,342
Duplex	48
Multifamily (1)	45
Commercial	128
Industrial	6
Total	1,569

(1) The number of living units (i.e., apartments) per connection varies from three to 75. The total number of living units served is 363

Figure 4-1 | Overall Wastewater Pumping and Treatment System Schematic



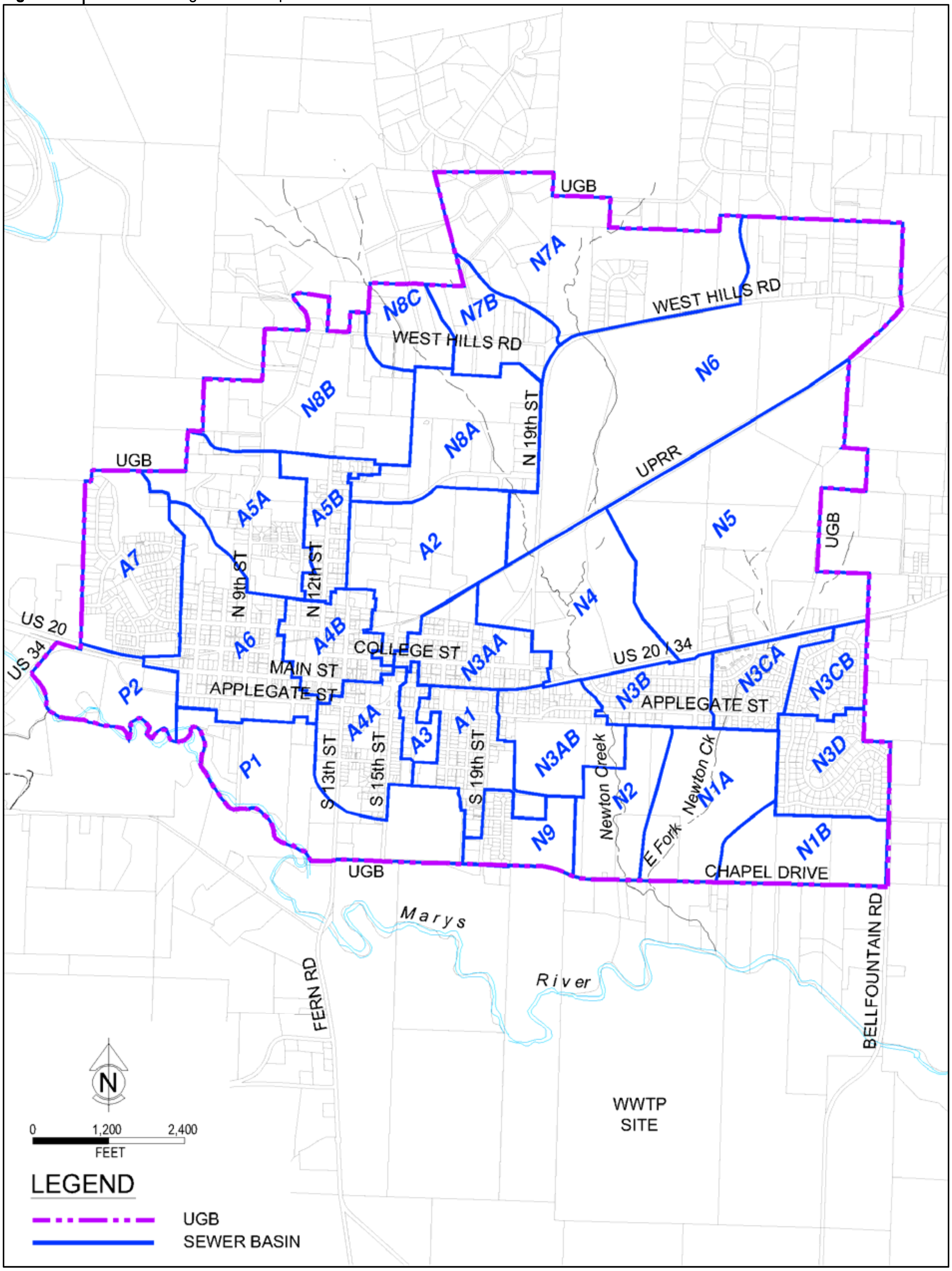
4.4.2 Drainage Basins

To aid in the analysis of the collection system, it is convenient to divide the collection system into separate drainage basins. The basin boundaries are based on a combination of factors including topography, urban growth boundaries, as well as the existing drainage patterns and trunk sewer locations. The collection system is divided into 29 distinct basins as shown in Figure 4-2. The approximate area within each of the major sewer drainage basins is listed in Table 4-2. The letters "A" and "N" in the basin designation correspond to basins that drain to Pump Station A and the Newton Creek Pump Station respectively. There is one exception to this convention. Manhole #288 at the College Street/15th Street intersection has two outlets. One outlet leaves the manhole to the east directing water to the Newton Creek Pump Station. The other outlet leave the manhole to the south and directs water to Pump Station A. Since Manhole #288 receives all flow from Basin A2, a portion of the flow from Basin A2 is routed to the Newton Creek Pump Station. A weir in the manhole is used to split the flow between the two basins. During low and moderate flow conditions, all flow into the manhole is directed to the Newton Creek Pump Station. As flow increase, the water overflows the weir and the flow is split between the two pump stations. The purpose of this weir is to limit peak flows to the collection system that feeds into the Newton Creek Pump Station. The gravity sewer lines east of Manhole #288 do not have the capacity to convey all of the flow that enters the manhole. As such, flow that exceeds the capacity of these lines is routed to Pump Station A. In addition to the basins that drain to Pump Station A and the Newton Creek Pump Station, there are two basins (Basin P1 and Basin P2) in the southwest portion of the City that are generally located at elevations below what can be served by the sewer lines that extend from Pump Station A. Development in these basins will ultimately require a new pump station.

Table 4-2 | Sewer Drainage Basin Areas

Basin	Total Area (Acres)	Sewered Area (Acres)	Non-Sewered Area (Acres)
A1	56.3	52.2	4.1
A2	103.5	53.4	50.1
A3	14.7	14.7	0.0
A4A	61.4	45.5	15.9
A4B	49.6	44.5	5.1
A5A	84.5	14.1	70.4
A5B	41.0	30.3	10.7
A6	106.2	89.3	16.9
A7	96.6	60.2	36.4
N1A	88.5	0.0	88.5
N1B	63.5	0.0	63.5
N2	56.8	0.0	56.8
N3AA	54.4	42.3	12.1
N3AB	61.9	54.0	7.9
N3B	40.9	39.9	1.0
N3CA	40.9	40.9	0.0
N3CB	33.5	28.7	4.8
N3D	62.1	62.1	0.0
N4	110.9	8.9	102.0
N5	263.7	0.0	263.7
N6	341.6	0.0	341.6
N7A	185.0	0.0	185.0
N7B	50.1	0.0	50.1
N8A	99.2	39.5	59.7
N8B	131.0	0.0	131.0
N8C	36.2	0.0	36.2
N9	37.2	12.8	24.4
P1	123.2	0.0	123.2
P2	54.0	0.0	54.0
Totals	2548.4	733.3	1815.1

Figure 4-2 | Sewer Drainage Basin Map



4.4.3 Gravity Collection System

The collection system serving Philomath includes approximately 107,000 feet of mainline pipe, 460 manholes, and 1700 service laterals. Pipe sizes range from 6-inch to 24-inch diameter (Figure 4-3). Most of the piping is 8-inch diameter. The collection system includes three public pump stations. The original collection system was built in 1952. The original collection system utilized primarily concrete pipe with mortar joints. Approximately 60% of the original 1952 piping remains in service. The original collection system has been extended over the years. Early extensions used concrete and asbestos cement pipe. Since the 1970s most extensions have been made using PVC pipe or concrete pipe with rubber joints. As a result of this history, the City has a variety of pipe materials (Figure 4-4).

Most pipelines installed after the original sewer system use more modern (e.g., AC, PVC, etc.) pipe materials and generally leak much less than 1950s era concrete pipe. Most new construction has utilized PVC pipe with rubber gaskets. Public Works design standards were adopted in recent years that allow only rubber gasketed PVC, polypropylene, and ductile iron pipe for the construction of gravity sewers. The City has also used HDPE pipe installed by pipe bursting for rehabilitating existing concrete.

Figure 4-3 | Pipe Inventory by Diameter

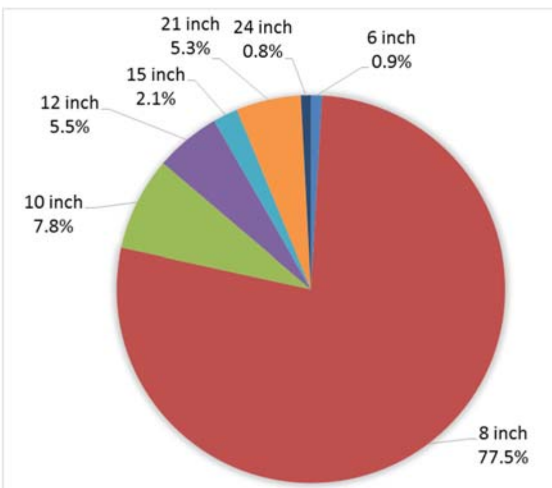
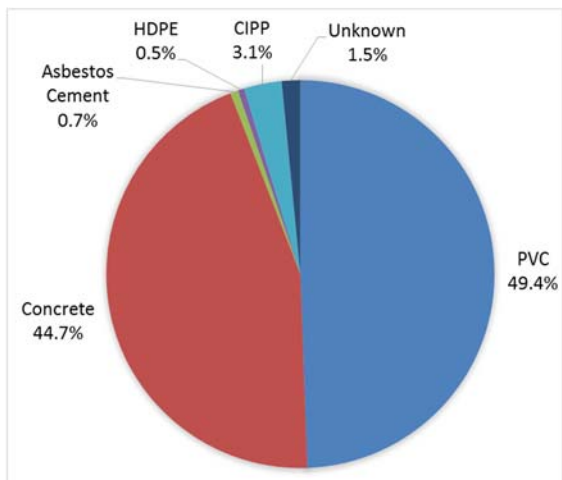


Figure 4-4 | Pipe Inventory by Material



4.4.4 Pump Stations

Wastewater is conveyed by the gravity collection system to one of three pump stations as discussed above. In general, the collection system serving the western portion of town conveys flows to Pump Station A. Pump Station A is located at the City shops complex near the south end of 16th street. The collection system serving the eastern portion of town conveys flows to the Newton Creek Pump Station. The Newton Creek Pump Station is located on Chapel Drive west of the Newton Creek Bridge. The Timber Estates Pump Station is a minor pump station located on Chapel Drive west of the Newton Creek Pump Station. This pump station serves the Timber Estates subdivision northwest of the 19th Street and Chapel Drive intersection. The three stations pump wastewater through a common force main to the treatment plant located south of the Marys River on the west side of Bellfountain Road.

A fourth minor pump station is located at the City Park south of Applegate Street on 23rd Street. This pump station serves the public restroom facility located at Pioneer Park. This pump station is relatively small and similar in nature to a single-unit residential pump station. As such, it is not included in the facilities planning effort. Table 4-3 contains a summary of some of the characteristics of each of the three pump stations. A more detailed description of each of the stations is presented in the following sections.

Table 4-3 | Summary of Existing Pump Stations

Category	Pump Station A (PSA)	Newton Creek Pump Station (NCPS)	Timber Estates Pump Station (TEPS)
General			
▪ Basins served	A1, A2, A3, A4A, A4B, A5A, A5B, A6, A7, N8A	N3AA, N3AB, N3B, N3CA, N3CB, N3D, NA8, A2	N9
▪ Construction date(s)	2009	1986	1995
▪ Type	Submersible	Submersible	Submersible
Firm Capacity (1)	4.75 mgd @ 90 ft TDH	2.0 mgd @43 ft TDH	0 mgd @ 37 ft TDH
Wet Well			
▪ Type	Concrete	Concrete	Concrete
▪ Size	10 ft. by 21 ft.	16 ft diameter	6 ft diameter
▪ Rim Elevation	265.00 ft.	252.00 ft.	272.50 ft.
▪ Influent Invert Elev.	245.20 ft.	241.25 ft.	262.67 ft.
▪ Bottom Elev.	239.00 ft.	228.25 ft.	256.50 ft.
▪ Depth (Rim to Bottom)	26.00 ft.	23.75 ft.	16.00 ft.
Pumps			
▪ Type	Submersible	Submersible	Submersible
▪ Number	2 Jockey, 2 Primary	2 Jockey, 2 Primary	2
▪ Manufacturer & Model	Flygt NP 3171.091/433 CP 3231/615/630/450	Flygt CP 3152/432 CP 3127/461	ABS AFP 1040-1 Series AFP1
▪ Motor Size & Speed	34 HP 1750 RPM 90 HP 1185 RPM	20 HP 1750 RPM 10 HP 1750 RPM	3.7 HP 1780 RPM
▪ Power Supply	460-Volt 3-Phase	460-Volt 3-Phase	240-Volt 1-Phase
Pump Speed Control	VFD's	VFD's	none
Force Main			
▪ Size & Type	14" DI (PSA to NCPS) 18" DI (NCPS to WWTP)	18" DI (NCPS to WWTP)	4" DI (to Common FM) 14" DI (TEPS to NCPS) 18" DI (NCPS to WWTP)
▪ Length	4500 ft (PSA to NCPS) 3700 ft (NCPS to WWTP)	3700 ft (NCPS to WWTP)	30 ft (to Common FM) 1500 ft. (TEPS to NCPS) 3700 ft (NCPS to WWTP)
▪ FM Discharge	WWTP Headworks	WWTP Headworks	WWTP Headworks
▪ FM Discharge Elev.	± 259.00 ft	± 259.00 ft	± 259.00 ft
Level Control	Submerged Pressure Transducer	Submerged Pressure Transducer	Float Switches
Hydrogen Sulfide Control	None	none	none
Auxiliary Power			
▪ Type & Location	300 KW Fixed Gen	30 KW Fixed Gen	Portable Generator
▪ Fuel Supply	Diesel	Diesel	Gas

Table 4-3 | Summary of Existing Pump Stations

Category	Pump Station A (PSA)	Newton Creek Pump Station (NCPS)	Timber Estates Pump Station (TEPS)
▪ Transfer Switch	Automatic	Automatic	Manual
Telemetry	City SCADA System	City SCADA System	Telephone Dialer System
Overflow	Storm Drain MH Adjacent to Station	Chapel Drive Roadside Ditch	Chapel Drive Roadside Ditch
Overflow Elevation	260.45	250.00	268.25
Overflow Discharge Point	Marys River	Newton Creek	Newton Creek
Overflow Storage	26,700 gallons	18,000 gallons	2,300 gallons

(1) Firm capacities based on the largest single out of service at each station.

4.4.4.1 Pump Station A

Pump Station A is located behind the City's Public Works office at the east end of Willow Lane. In general, Pump Station A receives wastewater from the western portion of the City. The station as it now exists was constructed in 2009. As such, it is relatively new and in good condition.

Figure 4-5 | Pump Station A

Pump Station A consists of a rectangular wet well, a valve vault, a masonry block building, four submersible sewage pumps, and associated controls and piping. Pump Station A is a quadraplex pump station with two 90 HP primary pumps and two 34 HP jockey pumps. The pumps are individually controlled in response to the water level in the wet well. Pump speeds increases as the level in the wet well increases in an attempt to match pump discharge rates to the flowrate into the station. The primary pumps only run during large winter storms. As such, the jockey pumps handle most of the flow to the station. The pumps pump from the concrete wet well through valved discharge piping and into the 14-inch force main. The discharge pipes are fitted with check valves, pressure gauges, and isolation valve located in a valve vault adjacent to the wet well. The level in the wet well is monitored by a submerged pressure transducer. The pump control panel and the variable frequency drives are located in a masonry utility building adjacent to the wet well. The control building also houses an auxiliary power generator. A jib crane with an electric hoist is mounted over the wet well for pump removal.

The pump station is integrated into the City's SCADA system. The SCADA system includes an autodialer for alarm callouts. Various alarm conditions are monitored including high level, overflow, pump fail, etc..

Pump Station A discharges into a ductile iron forcemain that conveys water to the Wastewater Treatment Plant. The first 4,600 feet of the forcemain between Pump Station A and the Newton Creek Pump Station is 14-inch diameter. The forcemain size increases to 18-inch diameter downstream of the Newton Creek Pump Station.

Overall, Pump Station A is relatively new and is adequately sized to convey projected flows during the planning period. As such, no significant improvements to this station are anticipated during the planning period. Work at this station will largely consist of maintenance activities such as pump rebuilds, the occasional pump replacement, generator maintenance, control system updated, etc. This type of work is envisioned to be funding from normal operations and maintenance budgets.

4.4.4.2 Newton Creek Pump Station

The Newton Creek Pump Station is located on Chapel Drive near the Newton Creek Bridge. In general, the station receives wastewater from the eastern portion of the City.

Figure 4-6 | Newton Creek Pump Station

The Newton Creek Pump Station consists of a circular wet well, a small masonry control building, four submersible sewage pumps, an auxiliary power unit, and associated controls and piping. The station is a quadraplex station with two 20 HP primary pumps and two 10 HP jockey pumps. The pump station was originally constructed in 1986. In 2009, the station controls were upgraded and variable frequency drives were installed to control the speed of the pumps. A jib crane with an electric hoist was also installed in 2009 to facilitate pump removal.

The wet well is a cast in place concrete structure approximately 25 feet deep and 16 feet in diameter. While the existing pumps provide a capacity of 3.04 MGD, the wet well is sized for an ultimate capacity of 7.05 MGD. In addition to the main 21-inch inlet, a 10-inch diameter ductile iron stub enters the wet well at an invert elevation of 237.90 feet. This stub is intended to serve the areas north and east of the existing station. The wet well is accessed through a hatch on the northeastern quarter of the top slab. A ladder descends to within one foot of the wet well floor. The wet well is equipped with ventilation equipment.

Most of the pump discharge piping and valves are located in the wet well. Sewage is pumped from the wet well into vertical standpipes. Each standpipe is equipped with a check valve. An 8-inch pipe collects sewage from the small pumps and conveys it outside of the wet well. Likewise, 10-inch pipe conveys flow from the large pumps to the outside of the wet well. Immediately outside the wet well, the 8-inch and 10-inch pipes flow into a welded steel manifold. The steel manifold is connected to a short section of 14-inch ductile iron pipe. The 14-inch pipe section extends a short distance to Chapel Drive where it intersects the 14” force main from Pump Station A. The two force mains are connected with a ductile iron tee. On the downstream side of the tee, the force main increases to 18-inches in diameter. From this location approximately 4,000 feet of 18-inch pipeline conveys wastewater to the treatment plant.

The pumps are individually controlled in response to the water level in the wet well. Pump speeds increases as the level in the wet well increases in an attempt to match pump discharge rates to the flowrate into the station. The level in the wet well is monitored by a submerged pressure transducer.

The control building is located adjacent to the pump station with approximate dimensions of 11 feet by 19 feet. The building houses the electrical control panels, the variable frequency drives, the auxiliary power unit, and the blower and ductwork for the wet well ventilation system. The station is equipped with a 30 KW auxiliary power unit. The auxiliary power unit is sized to operate the two 10 HP pumps, the control system and building lighting.

The pump station is integrated into the City's SCADA system. The SCADA system includes an autodialer for alarm callouts. Various alarm conditions are monitored including high level, overflow, pump fail, etc..

4.4.4.3 Timber Estates Pump Station

Figure 4-7 | Timber Estates Pump Station

Timber Estates Pump Station is located on Chapel Drive east of 19th Street. The station receives water from the Timber Estates Residential Subdivision and discharges into the common forcemain in Chapel Drive. The Station consists of a circular wet well, two submersible sewage pumps, and associated controls and piping. Timber Estates Pump Station is a duplex pump station with two 3.7 HP submersible pumps that are individually controlled in response to the water level in the wet well. The pumps pump from the concrete wet well through valved discharge piping and into the 14-inch force main. The pump station controls are located in a control cabinet adjacent to the wet well.

The wet well is a cast in place concrete structure approximately 16 feet deep and 6 feet in diameter. An 8-inch gravity sewer enters at invert elevation 262.67. There is an 8-inch overflow located at invert elevation 268.50 feet. The overflow discharges in the roadside ditch on the north side of Chapel Drive. This ditch ultimately discharges to Newton Creek. The wet well is accessed through a hatch on the top slab.

The existing pumps provide a capacity of 0.155 MGD. Each pump is mounted on a guide rail system to permit raising and lowering of the pumps. A lifting chain is attached to each pump. The pumps are controlled according to the water level in the wet well. There are four float switches in the wet well that open and close the control circuits, thereby starting and stopping the individual pumps.

The 4-inch discharge pipe from each pump passes through a 4-foot diameter precast circular valve vault. The vault is constructed using manhole barrel sections and contains no drainage provisions. Consequently, the valve vault remains full of water most of the time. The valve vault contains two 4-inch ball type check valves and two 4-inch gate valves. Shortly outside the valve vault the two discharge pipes joint into a single 4-inch pipe that connects to the 14" common force main in Chapel Drive.

The City owns a portable gas generator that is capable of running one of the pumps at the station. A manual transfer switch is provided for switching to auxiliary power. The pump station is equipped with a telephone dialer system for alarm telemetry.

In general, the station has operated as designed since 1995 with no modifications. The existing pumps lack the discharge head to pump into the forcemain when forcemain pressure are elevated. Occasionally during large storm events, the 90 HP primary pumps at Pump Station A will start. The resulting high pressures in the forcemain tend to reduce discharge rate from the Timber Estates Station. If the 90 HP primary pumps run for extended periods of time, the water levels in the Timber Estates Pump Station sometimes rise to high levels. Under these conditions, the City must manually turn down the pumps at Pump Station A to allow the Timber Estates Pumps to dewater the wet well. This is a problem that should be addressed during the planning period. The Timber Estates Pump Station is also more than 20 years old and major upgrades to the mechanical and electrical equipment are also appropriate during the planning period.

4.4.5 Inflow and Infiltration

The collection system is typical of many western Oregon sewer systems in that it experiences higher flows during the winter months because of infiltration and inflow (I/I). The average dry weather flow measured at the WWTP during the months of May through and October is approximately 0.52 MGD. The average flow during the wet weather months (November through May) is approximately 1.18 MGD. The highest daily flows measured most years are over 4.5 MGD. The ratio between average dry weather flow and the peak day flow is approximately 8.7. This ratio is common for similar municipal collection systems in Western Oregon. Despite the fact that no known raw sewage overflows from the collection system have ever been documented, significant portions of the collection system surcharge during large winter storms. This surcharging indicates that high I/I flows cause capacity issues in the system. High I/I flows are problematic for a number of reasons. I/I utilizes reserve capacity and ultimately decreases the useful life of the gravity collection system. I/I is also a burden to the treatment facilities since it must be treated and discharged as though it was wastewater. This increases operations and maintenance costs.

The high amount of I/I collected by the City's gravity collection network is common for similar systems. The original collection system that was constructed in 1952 utilized concrete pipe laid in short sections. The joints between each section were sealed using concrete mortar. Over time, the concrete mortar cracks and breaks, creating a pathway for groundwater infiltration at every joint. As a result, groundwater infiltration rates in systems with old concrete pipe are high.

The City has been aggressive about implementing I/I corrective measures. Since 2005, the City has rehabilitated approximately 11,200 feet of the collection system including mainlines, manholes, and service laterals. Most of this work was within the original 1952 collection system. The City has not traditionally performed I/I corrective work on an annual basis, but has instead chosen to implement larger projects every few years. In addition to this rehabilitation work, the City also inspects every manhole on an annual basis and repairs manholes with high leakage rates. The City recently purchased television inspection equipment and has started implementing a comprehensive television inspection program. The City has not performed smoke testing recently and plans to do so in the coming years.

As the City's collection system continues to age and deteriorate, groundwater infiltration rates are likely to increase. As such, the City must continue to implement I/I corrective improvements in order to keep infiltration rates at their current levels. Alternatives for I/I correction are considered in Chapter 6.

4.4.6 Known Collection System Non-Compliance Issues

The City has not received any warning letters from DEQ over the past few years regarding problems in the collection system.

There are a number of areas in the collection system that will likely experience compliance problems unless significant upgrades are completed within the planning period. These include the replacement or reconstruction of over-capacity and faulty sewers that contribute significant I/I. Continued I/I control efforts are needed in the collection system regardless if growth within the collection system occurs. The specific projects are discussed in more detail in Chapter 6.

4.4.7 Collection System Deficiencies

Problems with the Collection System were identified from meetings and discussions with City staff and from field investigations. During major winter storms, portions of the collection system surcharge due to inadequate trunk sewer capacity and large amounts of infiltration and inflow. Infiltration and inflow problems are largely limited to original 1952 collection system. These portions of the sewer collection system will generally reach the end of their useful life during the planning period.

The Timber Estates Pump Station lacks the capacity to convey peak flows during certain conditions. The mechanical and electrical equipment are also likely to reach the end of their useful life during the planning period. Therefore, improvements to this station will be required.

Large amounts of infiltration and inflow is far and away the most significant problem in the City's collection system. It is the underlying cause of the capacity problems in the trunk sewers. Alternatives for I/I correction are considered in Chapter 6. Table 4-4 outlines the major known problem areas, as well as the category that the problem falls under.

Table 4-4 | Known Collection System Deficiencies

Location	Problem Category
Original 1952 Collection System	High I/I, End of Useful Life
Timber Estates Pump Station	Lack of Capacity, End of Useful Life (Mech. Equip)

4.5 EXISTING WASTEWATER TREATMENT AND DISPOSAL SYSTEM

This section includes a discussion of the treatment and disposal system owned by the City. The City of Philomath owns, operates and maintains the wastewater treatment plant (WWTP) serving the City. The plant is located south of the City on the south side of the Marys River west of Bellfountain Road. The WWTP has three stabilization lagoons that normally operate in series. From November through April, treated effluent is disinfected and discharged into the Marys River. Between May and October treated effluent is applied to fields adjacent to the lagoons. In addition to the lagoons, the treatment plant includes a headworks for flow measurement and influent sampling, a chemical building, a laboratory building, two chlorine contact chambers, and an irrigation pump station. The chemical building houses a chlorine gas feed system used for chlorination as well as a sulfur dioxide gas feed system used to dechlorinate the effluent prior to discharge to the Marys River. The wastewater facilities are schematically presented below in Figure 4-8. The layout of the existing treatment plant plan is presented below in Figure 4-9 and Figure 4-10. A summary of the design data for the existing treatment facilities is presented in Table 4-5 followed by a short discussion of the individual unit processes.

Table 4-5 | Existing Treatment Plant Design Data

Design Flows (see section 5.3 for definition of flowrate acronyms)			
• ADWF	• 0.83 MGD		
• AWWF	• 1.99 MGD		
• MMDWF	• 1.61 MGD		
• MMWWF	• 2.77 MGD		
• PDF	• 6.68 MGD		
• PHF	• 11.6 MGD		
Design Loadings			
Average Annual BOD	• 1,515 PPD		
Average Annual TSS	• 1,670 PPD		
Headworks			
• Influent Screening	• None		
Parshall Flume			
• Throat Width	• 18-inches		
• Flume Invert	• 258 feet		
• Water Depth at PHF	• 2.08 ft @ 12 MGD		
Flow Measurement	• Ultrasonic level transducer		
Influent Sampler	• Refrigerated automatic composite sampler		
Lagoon/Features	Cell 1	Cell 2	Cell 3
• Type	• Facultative	• Facultative	• Facultative
• Surface Area	• 31 Ac	• 17.5 Ac	• 20 Ac
• Top of Dike Elevation	• 257.5 ft	• 255.5 ft	• 255.5 ft
• Min. Water Depth/Elevation	• 2 ft / 248.5 ft	• 2 ft / 246.5 ft	• 2 ft / 246.5 ft
• Max. Water Deth/Elevation	• 8 ft / 254.5 ft	• 8 ft / 252.5 ft	• 8 ft / 252.5 ft
• Minimum Freeboard	• 3 ft	• 3 ft	• 3 ft
• Usable Storage Volume	• 186 Ac –ft	• 105 Ac -ft	• 120 Ac -ft
• Total Plant Storage Volume	• 411 Ac –ft		
• Cell 1 Surface Loading Rate	• 49 lb BOD /day		
• Plant Surface Loading Rate	• 22 lb BOD /day		

Table 4-5 | Existing Treatment Plant Design Data

Disinfection Facilities	
• Type	• Gas Chlorine
• Chemical Delivery	• Ton Cylinders
• Typical Feed Concentration	• 1.5 mg/L
• Avg Winter Chlorine Use	• 20 ppd
• Avg. Summer Chlorine Use	• 25 ppd
• Chlorinator Rotameter Capacity	• 100 ppd
• Total Storage Capacity	• three 1-ton cylinders
• Typical Chlorine Delivery	• Twice per year
• Control System	• Manual On/Off with flow proportional dosing
• Injection Point	• Cell 3 outlet structure or upstream end of contact chamber
• Chemical Mixing	• Natural turbulence
• Contact Chamber Number	• 3 (all used during winter, only pipe & first tank used during summer)
• Contact Chamber Type	• Buried pipe and Baffled concrete tanks
• Contact Chamber Geometry	• 90:1 Minimum Length to Width Ratio
• Winter Contact Volume	• 105,000 gallons
• Min. Summer Contact Volume	• 86,000 gallons at low lagoon levels
• Max. Summer Contact Volume	• 135,000 gallons at high lagoon levels
• Winter Contact Time	• 75 minutes @ 2 MGD typical discharge rate • 38 minutes @ 4 MGD peak discharge rate
• Summer Contact Time	• 55 minutes @ 3.5 MGD with lagoon water depth = 8 feet • 35 minutes @ 3.5 MGD with lagoon water depth = 2 feet
Dechlorination Facilities	
• Type	• Sulfur Dioxide Gas
• Chemical Delivery	• 150 pound cylinders
• Typical Feed Concentration	• 0.75 mg/L
• Typical Usage Rate	• 12.5 ppd
• Rotameter Capacity	• 30 ppd
• Total Storage Capacity	• 10 150 pound cylinders
• Typical SO2 Gas Delivery	• Once every 2 to 3 months
• Control System	• Manual On/Off with manual dosing
• Injection Point	• Downstream end of chlorine contact chamber
• Chemical Mixing	• Natural turbulence from contact chamber outlet weir
Winter Effluent Flow Measurement & Sampling	
• Primary Device	• 4 foot rectangular weir without end contractions
• Device Location	• Downstream end of second contact chamber
• Measurement Range	• 0.5 – 10 MGD
• Flow Meter	• Ultrasonic level transducer
• Effluent Sampler	• Refrigerated automatic composite sampler
• Sample Location	• From compliance manhole downstream of second contact chamber
Marys River Outfall (winter discharge)	
• Type	• Single port diffuser to Marys River
• Material	• Ductile Iron
• Size	• 16-inch

Table 4-5 | Existing Treatment Plant Design Data

Irrigation Pumps Station			
• Purpose	• Dry season discharge to irrigation sites		
Irrigation Pumps			
• Pump Type & Number	• 2 Vertical Turbine, Variable Speed		
• Pump Size	• 60 HP each		
• Pump Capacity	• 1200 gpm @ 70 psi each		
• Pump Control	• Pump speed adjusted to maintain current discharge pressure		
Bypass Pump			
• Purpose	• To maintain flow through contact chamber during periods of no irrigation		
• Location	• Irrigation pump station wet well		
• Discharge Location	• Lagoon cell 1B or contact chamber		
• Pump Size	• 5 HP		
• Capacity	• 800 gpm @ 12 ft TDH		
Wet Well			
• Type	• Rectangular cast in place concrete		
• Level Control	• None, water level matches lagoon level		
Irrigation Flow Meter			
• Type	• Magnetic		
• Location	• Vault outside pump station		
• Size	• 10 inch		
Irrigation Pressure Transducer			
• Type	• Digital sensor/transducer mounted on an isolation ring		
• Location	• Vault outside pump station		
• Size	• 10 inch isolation ring		
Irrigation Sites	Site 1	Site 2	Site 3 (future)
• Location	• West of lagoons	• North of lagoons	• Brooks Farms
• Irrigated Area	• 100 acres	• 15 acres	• 500 acres
• Sprinkle Type	• Linear	• Big Gun	• TBD
• Application Rate	• 650 gpm	• Varies	• TBD

Figure 4-8 | Existing Wastewater Treatment Plant Schematic

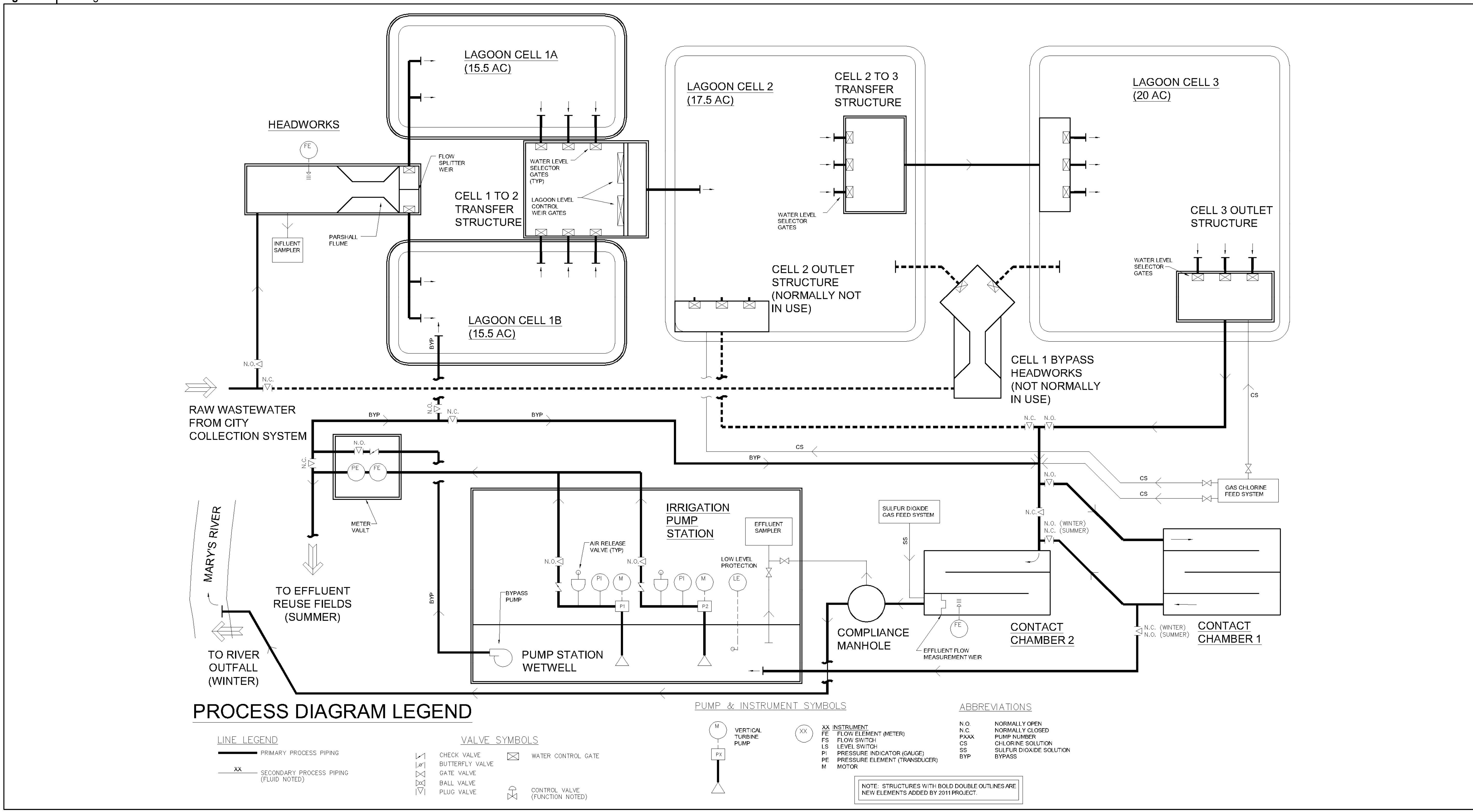


FIGURE 4-8

Figure 4-9 | Overall Layout of Existing Wastewater Treatment Plant

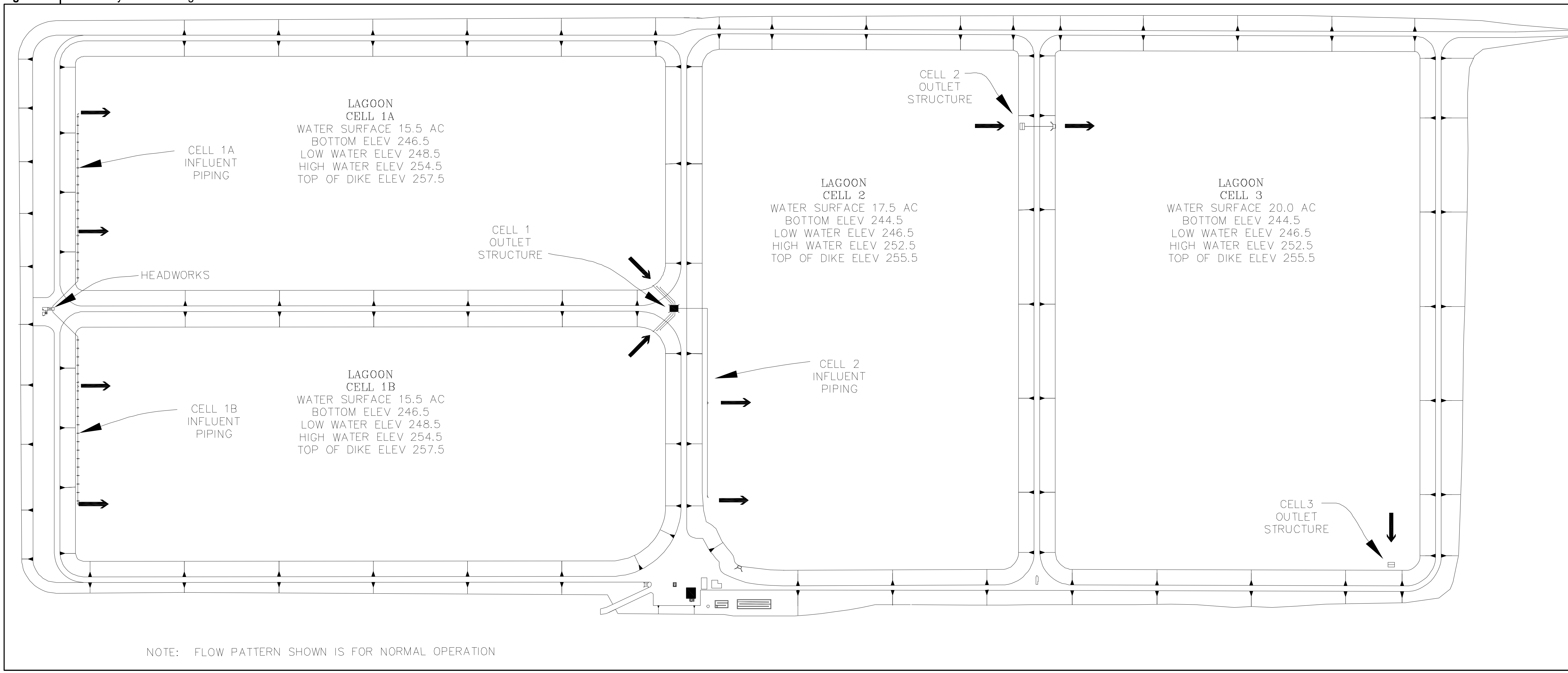
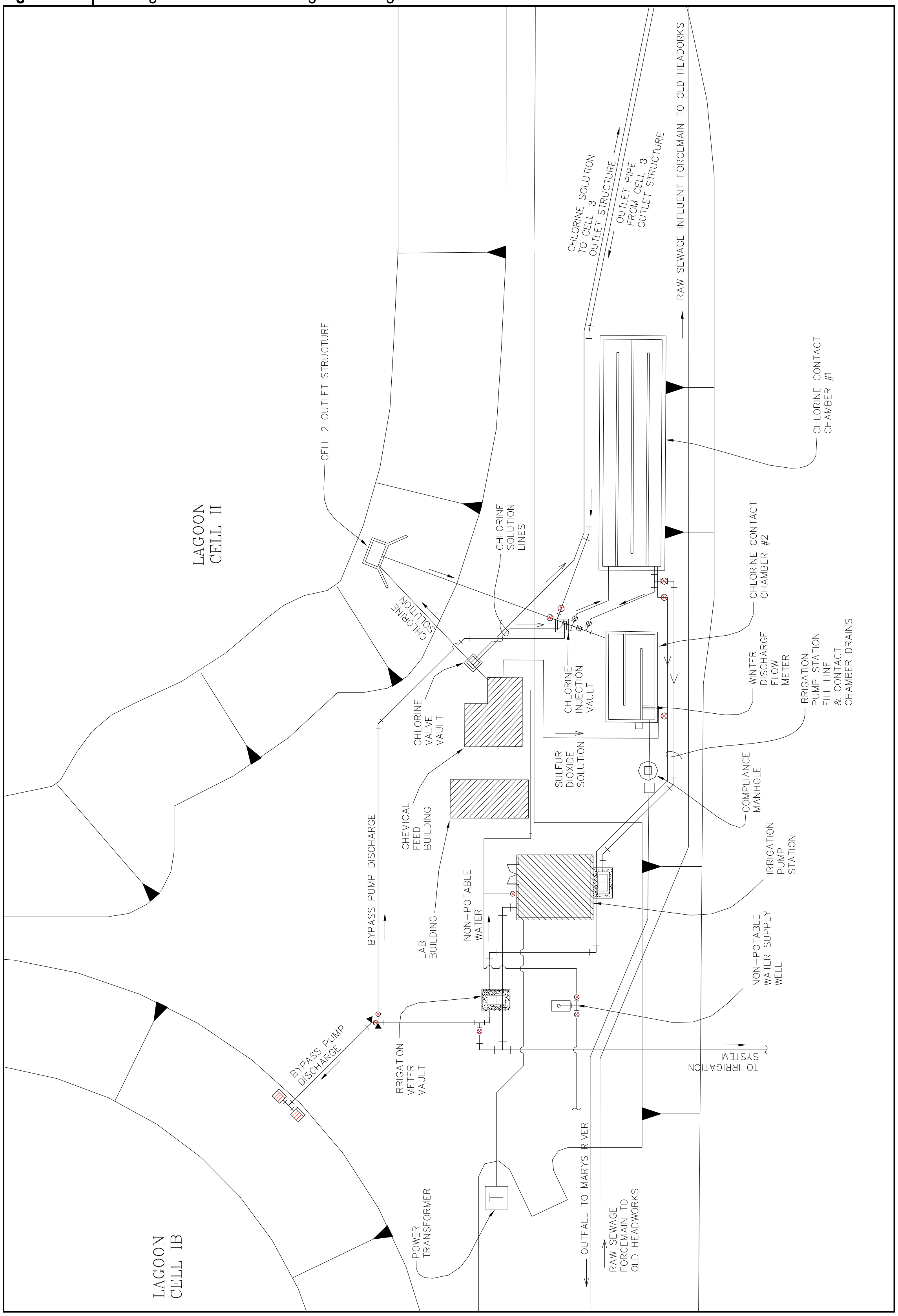


FIGURE 4-9

Figure 4-10 | Existing Wastewater Discharge Handling Facilities



4.5.1 Plant Performance

The City's existing effluent permit requires the production of effluent BOD and TSS concentrations below 30 mg/L and 50 mg/L during the winter discharge season. Average monthly effluent BOD and TSS concentrations are listed in Table 4-6 for the last three discharge seasons. As demonstrated in Table 4-6, the existing plant is capable of reliably meeting the effluent BOD and TSS concentration limits under existing loading conditions.

Table 4-6 | Existing Treatment Plant Average Monthly Effluent BOD and TSS Concentrations (mg/L)

Discharge Season	2012-2013		2013-2014		2014-2015		Average	
Month	BOD	TSS	BOD	TSS	BOD	TSS	BOD	TSS
November	No discharge		No discharge		No discharge		No Discharge	
December	5.0	21.0	15.0	11.0	2.3	5.7	7.4	12.6
January	3.0	18.3	13.0	11.0	10.5	13.0	8.8	14.1
February	10.5	18.0	6.0	10.0	11.5	13.0	9.3	13.4
March	4.0	6.0	4.5	5.0	14.0	17.0	7.5	9.3
April	5.0	8.0	5.0	4.0	11.5	12.5	7.2	8.2
Average	5.5	14.3	8.7	8.2	10.0	12.2	8.0	11.5

Note: Effluent BOD and TSS limits in the NPDES permit are 30 mg/L and 50 mg/L respectively.

In addition to the effluent concentration limits, the City's discharge permit also limits the total amount of pollutant that may be discharged by setting mass load limits. Mass load limits are determined by multiplying the effluent concentration of a pollutant by the effluent flow rate. Mass load limits are usually expressed in pounds of pollutant per day. Since flow and concentration are multiplied, increases in the flow rate must be offset by decreases in the pollutant concentration in order to maintain a constant effluent mass load. The existing permit allows for the discharge of 460 pounds per day of BOD and 760 pounds per day of TSS on a monthly average basis during the winter discharge season. The existing permit does not allow for any discharge to surface waters during the summer months. Average monthly effluent BOD and TSS mass loads are listed in Table 4-7 for the last three discharge seasons. It is clear from an examination of Table 4-7, that the existing plant is able to consistently produce an effluent quality that is sufficient to comply with the effluent mass load limits for BOD and TSS.

Table 4-7 | Existing Treatment Plant Average Monthly Effluent BOD and TSS mass loads (pounds per day)

Discharge Season	2012-2013		2013-2014		2014-2015		Average	
Month	BOD	TSS	BOD	TSS	BOD	TSS	BOD	TSS
November	No discharge		No discharge		No discharge		No discharge	
December	119	502	120	88	43	105	94	232
January	44	267	90	76	191	236	108	193
February	103	177	152	254	149	169	135	200
March	26	40	61	68	52	64	46	43
April	35	57	25	20	84	92	48	56
Average	65	209	90	101	104	133	86	145

Note: Effluent BOD and TSS mass load limits are 460 and 760 pounds per day respectively.

4.5.2 Headworks

Figure 4-11 | Wastewater Treatment Plant Headworks

Raw wastewater enters the plant through an 18-inch diameter forcemain that discharges into the treatment plant headworks. The headworks includes a Parshall flume for flow measurement and an automatic wastewater sampler for collecting influent samples. Flow measurement is accomplished in an 18-inch Parshall flume outfitted with an ultrasonic flow meter. The Influent flow meter is connected to the City's SCADA system for remote monitoring and data recording. The flow meter transmitter and the sampler are located inside a fiberglass enclosure adjacent to the headworks structure.

On the downstream end of the headworks, a weir is used to split the flow between lagoon cells 1A and 1B. Flow from the headworks is routed to cells 1A and 1B through 24-inch diameter ductile iron piping. The existing headworks does not include any screening or grit removal equipment. All solids that pass through the headworks remain in the waste stream and pass into the lagoons. The headworks was constructed in 2011. As such, the headworks is in good condition and should serve the City for many years. Routine maintenance will be required to ensure that the flow meter and sampler operate reliably. The City's NPDES permit also required annual flow meter calibrations. The City should plan to complete these calibrations on an annual basis.

4.5.3 Facultative Lagoons

The three facultative lagoons provide sedimentation, biological treatment, and sludge digestion. The lagoons also provide storage for non-discharging periods. The lagoons were constructed by compacting native clay materials into a natural clay liner system. The interior dike slopes are covered with riprap to protect the dikes from wave action. Water flows through the lagoon cells sequentially from cell 1 to cell 2 to cell 3. Cell 1 is subdivided into cells 1A and 1B. Water from the headworks is split equally between cells 1A and 1B. The discharge from cells 1A and 1B are combined in a flow control structure and routed to cell 2. Similarly, a flow control structure is used to control the flow of water from cell 2 to cell 3. Under normal operating conditions plant discharge is routed to the chlorine contact chamber from the cell 3 outlet structure. The plant piping allows for the temporary isolation of each of the lagoon cells. Piping is also in place to allow discharge from cell 2 as well as cell 3.

The City is not permitted to discharge effluent to the Marys River after April 30. During the first several weeks after the end of the winter discharge season, the land application sites are sometimes too wet to receive treated effluent. During these times all incoming waste water must be stored in the lagoons. Upon the onset of fall rains, the land application sites can no longer receive treated effluent. All wastewater flowing into the plant must be stored prior to the start of the winter discharge season in November. In order to meet the storage needs, the lagoons are designed with water level controls that allow the operator to lower water levels in anticipation of the storage periods. All lagoon cells can be drawn down to a minimum level of 2 feet if needed.

Each lagoon cell is fitted with an outlet control structure. The structures are designed with multiple ports set at different elevations. This gives operators the ability to draw water from various elevations in the water column. In general, most algae growth occurs in the upper layers of the lagoons. Therefore it can be useful to draw water from below the algae layer in order to optimize the quality of effluent discharged from the lagoons.

Figure 4-12 | Typical Lagoon Outlet Control Structure

Cells 2 and 3 were constructed in the mid 1980s. At that time, cell 3 was used as the first lagoon cell. Therefore, from the mid 1980s to 2011 sludge accumulated in cell 3 (i.e., formerly cell 1). In 2002, the City measured the sludge depth in cell 3 and found a cone of sludge approximately 2 feet deep near the inlet pipe that decreased to less than 6 inches within approximately 75 feet from the inlet pipe. At the present time, the average sludge depth in cell 3 is likely to be between 6 and 12 inches. In 2011, the new lagoon cell 1 was constructed. Since 2011 all raw wastewater has been routed to the new cell 1. Sludge will continue to accumulate in cell 1 and will eventually need to be removed. However, it is unlikely that sludge removal will be required during the planning period. The existing sludge in cell 3 will continue to stabilize and may eventually need to be removed. However, it is unlikely that this sludge will adversely impact the operation of the plant during the planning period.

4.5.4 Disinfection System

Figure 4-13 | Chlorine Contact Chambers

Chlorine gas is used to disinfect the treated effluent prior to discharge. The chemical feed equipment is located in the chemical building. Chlorine gas is delivered to the plant in one-ton cylinders. The gas feed equipment consists of cylinder valves that automatically switch to a full cylinder when one cylinder is emptied. The cylinder valves will also close automatically if a chlorine leak is detected. The gas feed system is a vacuum system. Gas is removed from the cylinders under a vacuum and routed to a gas chlorinator. The chlorinator is used to mix the gas with carrier water to create a chlorine solution. This solution is injected into the effluent stream. Water is supplied to the injector from an onsite well and pressure tank system similar to a residential well water system. Chlorine solution can be added to the effluent stream at the cell 3 outlet structure or immediately before the chlorine contact chamber. Chlorine contact time is provided in two contact chambers. The contact chambers are baffled concrete structures with parallel flow channels. Water flows through the two contact chambers sequentially. During the winter discharge season water flows through both contact chambers. The discharge rate from the plant is controlled using the sluice gates at the cell 3 outlet structure. An effluent flow measurement weir with an ultrasonic flow meter is mounted on the downstream end of the second contact chamber. A dechlorination chemical is added to the effluent immediately upstream of the flow measurement weir. Plant discharge flows by gravity from the effluent flow measurement weir to the Marys River through a 24 inch concrete pipe. The effluent compliance sample is collected from a manhole on the 24 inch pipe approximately 15 feet downstream of the second contact chamber. An automatic wastewater sampler is used to collect effluent samples.

During the summer irrigation season, only the first contact chamber is used. The walls of the first contact chamber match the elevation of the lagoon dikes. Therefore, the water level in the chamber can be filled to match the water level in cell 3. Effluent from the first contact chamber is routed to the irrigation pump station wet well. During the irrigation season, the discharge from the plant is controlled by the irrigation pump discharge rate.

The chlorination equipment allows for flow paced dosing. During the winter discharge season, the flow signal from the effluent flow measurement weir is used to pace the chlorine dosage. During the summer irrigation season the flow signal from a magnetic flow meter on the irrigation pump discharge piping is used to pace chlorine dosage.

During the winter discharge season, the residual chlorine is removed from the effluent stream on the downstream end of the contact chamber. A gas sulfur dioxide solution is added to the effluent stream near the effluent flow measurement weir. The sulfur dioxide feed equipment is similar to the chlorine gas feed equipment. Sulfur dioxide is delivered to the plant in 150 pound bottles. Cylinder valves are used to control the flow of gas from the bottles to the sulfur dioxide gas injector. The sulfur dioxide gas injector mixes the gas with a carrier water stream in a similar fashion as the chlorinator. Mixing energy for the dechlorination reaction is provided in the turbulence below the effluent flow measurement weir. The sulfur dioxide dosing rate is set manually by the operator.

4.5.1 Irrigation Pump Station

The purpose of the irrigation pump station is to distribute water at a suitable pressure to the irrigation sites. The irrigation pump station is also used in a drainage mode to drain the chlorine contact chambers.

Figure 4-14 | Irrigation Pump Station

During the summer irrigation season, effluent from the chlorine contact chambers is routed to the irrigation pump station wet well. Two vertical turbine pumps are mounted in a building above the wet well. These pumps are automatically controlled to adjust the pumping speed as needed to maintain a constant discharge pressure in the irrigation system piping. A magnetic flow meter and pressure transducer are located in a vault outside of station. As irrigation demand changes, the pump speed is adjusted to maintain a constant pressure. The pump discharge piping is fitted with isolation valves, check valves, and air release valves. This equipment is located in the irrigation pump station building. The pump control panel and variable frequency drives are also located in the pump station building. The building and wet well are sized to accommodate the installation of a third pump in the future.

The station was designed with a small submersible pump in the wet well. This pump is used to bypass the irrigation pump station and also to drain the chlorine contact chambers. During short periods (i.e., less than 48 hours) when the irrigation system is not in use, the bypass pump can be used to maintain flow through the contact chamber. The intent of this system is to keep the water in the contact chamber from stagnating thereby ensuring that the water is ready for irrigation use. The bypass pump discharges either into lagoon cell 1 or into the upstream end of the contact chamber. The bypass pump is also used to drain the station and the contact chambers during extended periods of non-use.

The irrigation pump station is monitored using the City’s SCADA system. An autodialer located in the City’s water treatment plant is used to call out alarm conditions.

The irrigation pump station was constructed in 2011. Therefore, the station is relatively new, in good condition, and with normal maintenance should serve the City for the remainder of the planning period.

4.5.2 Lab and Chemical Buildings

In addition to the irrigation pump station the plant includes two other buildings. These are the chemical building discussed above and the lab/office building. These buildings are all located in the same general vicinity. The chemical building is a masonry building that houses the chlorination and dechlorination chemical feed equipment. The gas cylinders are stored in the larger room and the chemical injection and electrical equipment is located in the smaller room. The chemical storage room is equipped with gas leak detection equipment. Alarm conditions are monitored using the City's SCADA system. Alarm lights are also located on the building exterior to indicate a chlorine or sulfur dioxide leak alarm. The chemical building was constructed in the mid 1980s. Certain components of the building such as the roofing, paint, windows, etc. are showing signs of age and may need to be replaced during the planning period. However, this work is generally considered to be routine maintenance rather than a capital improvement. With such maintenance, the building should serve the City for the remainder of the planning period.

Figure 4-15 | WWTP Chemical Building

The lab/office building includes a lab room and a restroom. The structure is a modular building that was constructed in the 1980s. The pressure tank and pressure switch for the onsite well are also located in this building. The lab is relatively simple, but suitable for the nature of the plant. Again, this building should serve the City for many years with normal routine maintenance.

4.5.1 Surface Water Outfall

During the wet weather months (November – May), treated effluent flows by gravity through a 24-inch pipeline to the Marys River. The 24-inch pipeline discharges into a small concrete structure near the river. The outfall pipeline between this structure and the river is a 16-inch single port outfall. There is no diffuser on the end of the 16 inch pipe.

The current NPDES permit provides for a mixing zone in the Marys River where federal regulations and Oregon Administrative Rules allow the DEQ to suspend all or part of the water quality standards in small designated areas. For Philomath, this area is defined as the area of the river where the effluent mixes with no more than 25 percent of the stream flow but in no case may the mixing zone extend more than 20 feet toward the midstream. The mixing zone also may not extend more than 10 feet upstream or 100 feet downstream from the outfall pipeline. The permit also defines a zone of immediate dilution (ZID) as that portion of the stream within 10 feet of the point of discharge.

Figure 4-16 | Marys River Near Outfall Pipeline

The outfall provides marginal mixing. The City prepared a mixing zone study in 2010 that showed the lowest centerline dilution ratio at the ZID was 2.3 and the dilution ratio at the edge of the mixing zone was 7.5. These values are relatively low and are not unexpected since there is no diffuser on the outfall pipeline. To date, the DEQ has been able to show that the City's discharge does not create a reasonable potential to violate water quality standards as long as the conditions in the permit are satisfied. However, with future changes in water quality regulations, the City may eventually be required to install a multi-port diffuser to improve mixing at the discharge point. Outfall diffusers have also become the industry standard for new treatment facilities. It would be unusual to construct a new surface water discharge that did not include an outfall diffuser. As such, it would be a good idea for the City to plan to install an outfall diffuser at some point during the planning period. A new diffuser will significantly improve the mixing of treated effluent with the receiving stream and provides some degree of protection to the City from future regulatory changes.

4.5.2 Recycled Water Disposal System

Figure 4-17 | Existing Linear Irrigation Sprinkler

During the dry weather months (May – October), treated effluent is pumped from the irrigation pump station to irrigation sites located west and north of the existing lagoons. These sites are owned by the City. The total area irrigated is approximately 115 acres. Effluent is distributed on the 100 acres west of the existing lagoons using a linear irrigation sprinkler. Big guns and hand lines are used to distribute effluent on the 15 acres north of the existing lagoons. The City has a lease agreement with a local grower who manages the agricultural activities at these sites. The sites are currently used for growing grass seed crops. Underground piping with vertical risers are in place to distribute recycled water to the irrigation sites.

The City prepared a recycled water use plan in 2011. The plan includes the eventual irrigation of up to 500 additional acres on private land located east of Bellfountain Road. At the present time, the pipelines are not in place to distribute recycled water to the east side of Bellfountain Road. It is envisioned that this pipeline will be installed as the population in the City grows and the amount of irrigation water increases.

4.5.3 SCADA System

Much of the equipment at the treatment plant is monitored remotely by the City's SCADA system. The primary SCADA terminal is located at the City's Water Treatment Plant on South 9th Street which serves as the primary office for operations staff. The SCADA terminal includes custom designed screens for some of the unit processes at the plant. The SCADA system is used primarily for monitoring purposes with limited control capabilities. Alarms are dialed out to the operations staff using an autodialer located at the water treatment plant.

4.5.4 Water Supply System

Wash down water and carrier water for the chlorination and dechlorination systems are provided to the wastewater treatment plant by a small onsite well with a pressure tank located inside the laboratory building. The system is relatively simple and similar to a typical residential system.

4.5.5 Access Roads

Vehicular access to the treatment plant is along a gravel driveway that extends from Bellfountain Road west approximately 1600 feet to the treatment plant site. Vehicular access around the plant is by gravel roadways constructed on the top of the lagoon dikes. For the most part, vehicular access is good. The lagoon dike roads are in relatively good condition. Potholes and isolated rutting sometimes occur on the main entrance road. The City periodically repairs the potholes and rutting as part of normal routine maintenance practices.

4.5.6 Wastewater Treatment Plant Operational Problems

Since the treatment plant was upgraded in 2011, most of the operational issues were addressed at that time. As such, there are no major operational problems with the existing treatment facilities.

4.5.7 Summary of Treatment and Disposal System Deficiencies

The treatment and disposal system is in relatively good condition with no major deficiencies. This is expected since a major improvement project was completed in 2011. The only deficiency that is known at the present time is that the Marys River outfall lacks an effluent diffuser as discussed above. As discussed on Chapter 7, the construction of a new outfall diffuser is recommended.

4.6 WASTEWATER SYSTEM OPERATOR LICENSING

The City's wastewater collection system currently requires a level 2 certification for operation. The City's existing treatment system requires a level 1 certification.

4.7 WASTEWATER SYSTEM FUNDING MECHANISMS

Funding for the City's existing wastewater system comes from two major sources, user fees and system development charges (SDCs).

4.7.1 User Fees

User fees are monthly charges to all residences, businesses, and other users that are connected to the wastewater system. User fees are established by the City Council and are typically the sole source of revenue to finance wastewater system operation and maintenance. The City's user fee system is established by Ordinance Number 624. The user fees and charges were most recently revised by Resolution Number 16-01. Together these documents provide the basis for assessing sewer user fees.

The City's sewer fund must provide sufficient revenues to properly operate and maintain the wastewater system and provide reserves for normally anticipated replacement of key system components such as pumps, motors, pump station control equipment, chemical feed equipment, manholes and sewer collection piping. Although the City relies exclusively on sewer fees for operation and maintenance costs, the sewer fund is typically not adequate to finance major capital improvements without outside funding sources.

During the winter months (November – April), residential and duplex users are charged a monthly base charge of $25.00 plus a consumption charge of $5.00 for each 100 cubic feet of water usage. Multi-residential users (three or more dwellings) are charged a monthly base charge of $12.50 per dwelling plus a consumption charge of $5.00 for each 100 cubic feet of water. The base charge for commercial and industrial users is $25.00 and the consumption charge is also $5.00 for every 100 cubic feet of water consumed. During the summer months (May – October), users can request the City charge a monthly rate equal to the average of the previous six winter months. The City automatically calculates the summer-time rates in this manner for residential users.

For a typical residential user that uses 5,000 gallons of water per month, the monthly user rate would be about $58 per month. The revenue from sewer billings for the fiscal year 2014/15 was approximately $910,000. Including other various charges and interest earnings, the total sewer fund revenues for the 2014/15 fiscal year were approximately $1,090,000.

4.7.2 System Development Charges

A system development charge (SDC) is a fee collected by the City as each piece of property is developed. SDCs are used to finance necessary capital improvements and municipal services required by the development. SDCs can be used to recover the capital costs of infrastructure required as a result of the development, but cannot be used to finance either operation and maintenance, or replacement costs. The City currently charges SDC fees based on water meter size (Table 4-8). The City updates SDC fees annually based on the ENR construction cost index.

Table 4-8 | Philomath SDC Fees (as of January 2016)

Water Meter Size	Sewer SDC Fee
3/4 inch	$7,275
1 inch	$10,331
1.5 inch	$17,462
2 inch	$27,938
3 inch	$56,432

4.7.3 Annual Sewer System Costs & Existing Debt Service

Annual operations and maintenance costs are recurring costs typically funded through user rates. The various expenditures from the sewer fund are listed below (Table 4-9) for the fiscal year 2014/15. The total expenditures for the 2014/15 fiscal year are approximately $944,000. These expenditures included a debt service payment of $332,000. The City issued $6,125,000 in bond debt in 2009 to fund major improvements to the wastewater system. As of May 2016, the outstanding principal balance is $5,656,000. The annual payment for this debt steadily increases to approximately $600,000 in 2033 which is when the debt is scheduled for retirement.

Table 4-9 | Sewer Utility Fund Expenditures 2014/2015 Fiscal Year

Item	Expenditure
Personnel Services	$ 265,000
Materials and Services	$ 292,000
Debt Service	$ 332,000
Transfers	$55,000
TOTAL EXPENDITURES	**$ 944,000**

4.7.4 Sewer SDC and Improvement Funds

The City currently has two funds that are used to save money for capital improvements. These include the System Development Fund with a current balance of approximately $65,000, and a Land, Building, & Equipment Fund with a balance of approximately $88,000. These are the anticipated balances at the end of the 2016-2017 fiscal year. In the last three years, the City has received an average of approximately $50,000 in sewer SDC revenue.

CHAPTER 5

WASTEWATER FLOWS AND LOADS

Chapter Outline

WASTEWATER FLOWS AND LOADS

5.1 INTRODUCTION

In order to select and size both collection and treatment facilities for the planning period, projected wastewater flows and organic loadings must be determined. The projected flows and organic loadings were determined based on a number of variables including the following:

- Rate of projected population increase
- Land use zoning within the UGB
- Projected per capita and per acre flowrates and organic loadings.

This chapter develops wastewater flow and loading projections which are used for sizing the collection system components as well as the treatment plant components. The projected design flowrates were determined based on a number of variables including zoning of land within the service area, anticipated development density at buildout and within a 20-year planning period, and projected per capita and per acre flowrates. It should be noted that some of the projects will not be constructed until several years after this document is adopted. As such, the designers for these projects will need to make new flow and loading projections that utilize current flow data and are based on 20 year projections from the date that construction is completed for each project.

5.2 POPULATION

Population projections serve as the basis for future wastewater flow projections. Much of the challenge in projecting the growth of the wastewater system relates to the difficulty in accurately tracking or projecting actual populations.

This section evaluates anticipated growth from a review of several data sources; including historical population data (census information & PSU estimates), County coordinated population projections, and anticipated development.

5.2.1 Historic Population

Population histories provide a tool for determining the future growth rate of the municipal wastewater system. The population in Philomath has steadily increased from approximately 840 people in 1940 to an estimated population of 4,650 in the year 2016. The City has experienced a slowdown in new residential development and building due to the economic downturn of the past several years, and the population has held steady at approximately 4,650 since the year 2010. Residential growth was strong between 1990 and 2010 due to the development of residential areas in the Neabeack Hill Area and in the hills on the west end of the City.

5.2.2 Anticipated Future Development

Philomath is likely to experience continued growth in the future as a suburb of the greater Corvallis area. During the planning period, the City anticipates future residential development to continue as both new subdivisions and as infill development (i.e., partitions & redevelopment). Any major commercial or industrial development that would dramatically increase the employment opportunities in Philomath are not anticipated during the planning period.

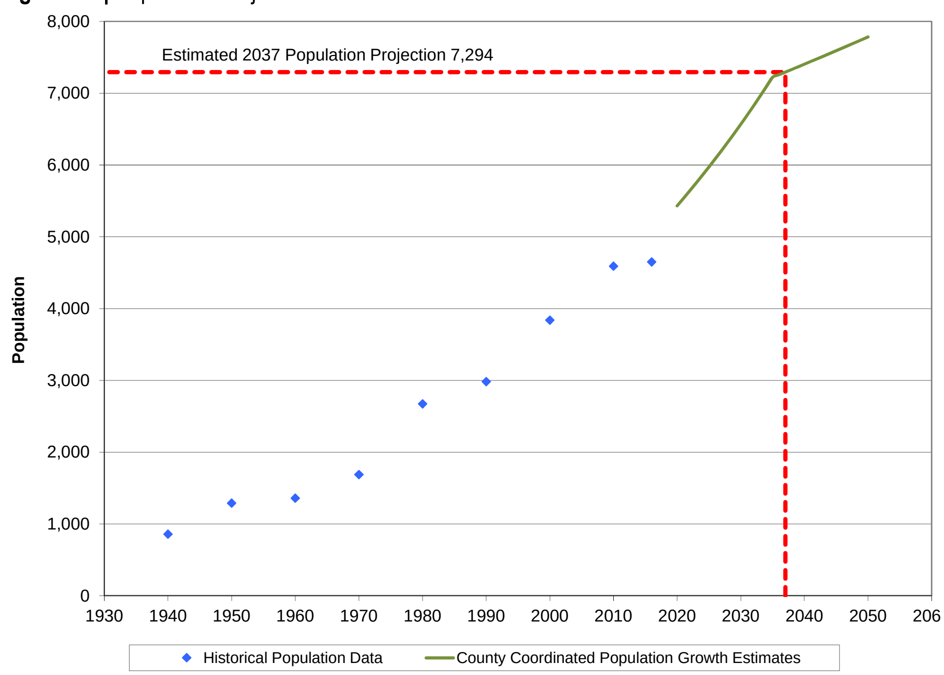
5.2.3 Future Population Projections

As previously noted, the planning period used in this plan is 20 years. The 20-year planning period is assumed to extend to 2037. In order to be eligible for many public funding sources, population projections (and associated flow projections) must be shown to be compatible with local and statewide planning goals, including adopted statewide and County population allocations (which are used as the 'coordinated number' for evaluating population projections). In 2017, the Portland State Population Research Center projected a population for Philomath of 7,222 in 2035. Beyond, 2035 the Portland State Population Research Center estimated and annual average growth rate of 0.5%. This average annual growth rate was used to project beyond 2035 to the design year of 2037. A tabulation of population data for select years during the planning period is presented in Table 5-1 and shown graphically in Figure 5-1.

Table 5-1 | Population Projection Summary

Year	Projected Municipal Population
2020	5431
2025	5972
2030	6567
2035	7222
2037	7294

Figure 5-1 | Population Projections



5.3 WASTEWATER FLOWS

Wastewater facility evaluation and design typically account for the following standard flow rates:

- Average dry-weather flow (ADWF) - Average daily wastewater flow during the dry-weather months of May through October
- Average wet-weather flow (AWWF) - Average daily wastewater flow during the wet weather months of November through April
- Average annual flow (AAF) - Daily wastewater flow averaged over the entire year
- Maximum-month dry-weather flow (MMDWF) - Maximum monthly flow during the dry weather months
- Maximum-month wet-weather flow (MMWWF) - Maximum monthly flow during the wet weather months
- Peak-day flow (PDF) - Maximum one-day flow during the weather months
- Peak-hour flow (PHF) - Maximum flow over a short duration (peak hour).

5.3.1 City Wastewater Treatment Plant Flow Records

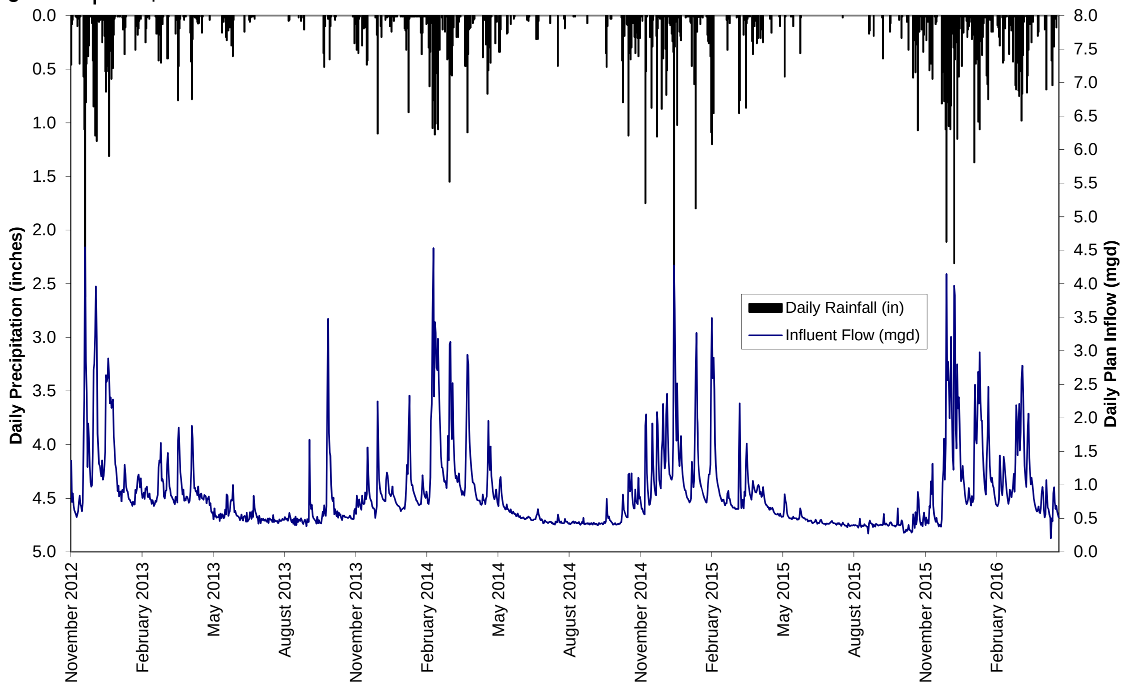
The City's treatment plant Discharge Monitoring Reports (DMRs) filed with the DEQ for the period from November 2013 through April 2016 were evaluated to identify flow patterns and evaluate current flows to the plant.

Plant inflows in Philomath are strongly influenced by precipitation (Figure 5-2). This is common for wastewater collection systems in the Willamette Valley. Winter rains cause groundwater levels to rise. The groundwater enters the collection system through faults and cracks in the collection piping and manholes (infiltration) and through direct connections to storm drainage collection facilities (inflow). Infiltration and inflow (I/I) results in increased flows measured at the treatment plant. As shown in Figure 5-2, plant inflows during the winter months are significantly higher than flows during the dry summer months. This can also be seen in Table 5-2 where the various flow components are tabulated for the last three years in millions of gallons per day (mgd).

Table 5-2 | Summary of City Wastewater Treatment Plant Flow Data 2013 through 2015.

Year	ADWF (mgd)	AAF (mgd)	AWWF (mgd)	MMDWF (mgd)	MMWWF (mgd)	PDF (mgd)
2013	0.580	0.868	1.162	0.758	1.937	4.545
2014	0.530	0.839	1.154	0.710	1.833	4.529
2015	0.443	0.787	1.136	0.550	1.671	4.273
Average	0.518	0.831	1.151	0.673	1.814	4.419

Figure 5-2 | Precipitation Effects on Plant Influent Flow



5.3.2 Wastewater System Existing Flow Estimates

The DEQ has published guidelines for the estimation of wet weather flows in Western Oregon. The purpose of these guidelines is to identify a methodology that can be used to estimate wastewater flows if all bottlenecks in the system were removed. In most systems such as Philomath's where large amounts of I/I enter the collection piping and manholes, the flows can increase to the point that surcharging occurs in the system. Surcharging tends to decrease the amount of I/I that could occur if the surcharging were not present. In theory, the wet weather flow components listed in Table 5-2 are influenced by this phenomenon and the wet weather flows to the wastewater treatment plant would actually be higher if all the bottlenecks could be removed. It is important to consider the flowrates in the absence of throttling because as the improvements described in this plan are implemented, the bottlenecks will be removed and the wet weather flows to the treatment plant are likely to increase beyond the flows listed in Table 5-2.

In order to estimate the wet weather flow components that would occur in the absence of bottlenecks, the DEQ has published guidelines that describe a methodology to correlate wastewater flows to rainfall during moderate rainfall events when surcharging is believed to be absent. This mathematical correlation is then used to extrapolate flows at higher rainfall events associated with peak wet weather flow conditions.

To establish a relationship between monthly rainfall and average monthly flow, the average monthly wastewater flowrates for the wet weather months are plotted against their corresponding monthly rainfall values. The monthly average flow and corresponding rainfall totals for the 2013 through 2015 winter months are plotted in Figure 5-3. A linear regression is performed to establish the relationship between monthly rainfall and average monthly flow. This relationship can be used to predict plant inflows as a function of monthly rainfall depth.

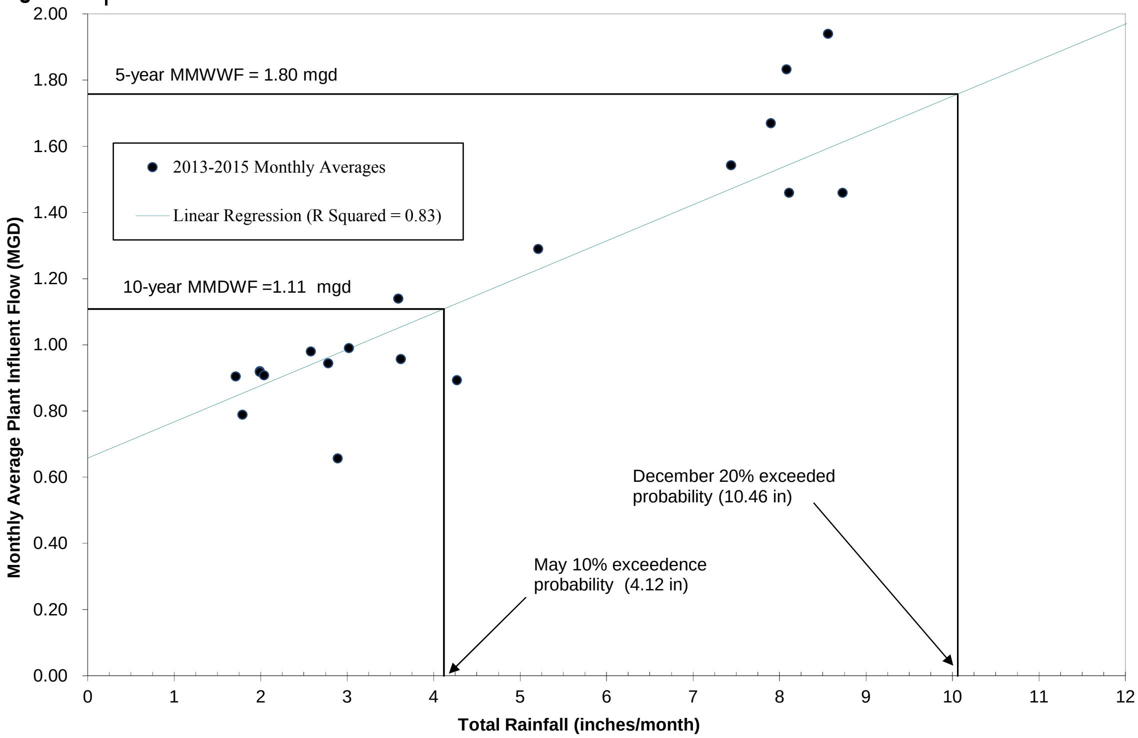
The Maximum Month Dry Weather Flow (MMDWF) is the monthly average flow for the rainiest summer month of high ground water. During the dry weather months of May through October, the MMDWF will typically occur in May following a wetter than normal spring. For the purposes of this report, the MMDWF is defined by the 10-year recurrence interval. Therefore, the MMDWF may be estimated by the monthly flowrate for the month of May with a 10-year recurrence interval. The linear regression established in Figure 5-3 may be used if the rainfall depth for the month of May that is associated with a 10-year recurrence interval is known. Rainfall depths corresponding to various exceedance probabilities have been calculated for the Hyslop Field weather station in Corvallis[3]. This data set is assumed to be generally representative of rainfall patterns in Philomath. For the month of May, the rainfall depth associated with the 10% exceedance probability (i.e., 10-year recurrence interval) is 4.12 inches. Using this rainfall depth and the relationship established in Figure 5-3 the MMDWF can be estimated. As shown in Figure 5-3, the MMDWF is approximately 1.11 MGD.

The Maximum Month Wet Weather Flow (MMWWF) represents the highest monthly average attained during the winter period of high groundwater. The DEQ methodology is based on the assumption that high groundwater levels are not consistently maintained until the month of January. Therefore, heavy storms do not begin to cause a reliable or consistent infiltration and inflow response until January. This leads to the assumption that the MMWWF occurs in January However, a review of the rainfall and flow data in Philomath shows that the maximum month wet weather flow often occurs in December. This is the case in 2012, 2014, & 2015. Also, the average precipitation for December is higher than the average for January. Therefore, for this report, it is assumed that the MMWWF occurs in December rather than January. This results is a higher estimate of MMWWF that better corresponds to the measured flows at the wastewater treatment plant.

In the same manner used to determine the MMDWF, the rainfall depth associated with a 20% probability of exceedance (i.e., 5-year recurrence interval) for the month of December is used in the correlation between plant flows and rainfall to determine the MMWWF. Again, using the rainfall data from the Hyslop Field weather station, the December rainfall total associated with the 20% exceedance probability is 10.46 inches. Using this rainfall depth and the relationship established in Figure 5-3 the MMWWF can be estimated. As shown in Figure 5-3, the MMWWF using this methodology is approximately 1.80 MGD.

[3] U.S. Dept. of Commerce, NOAA, Climatography of the United States No. 20, Corvallis State Univ, OR

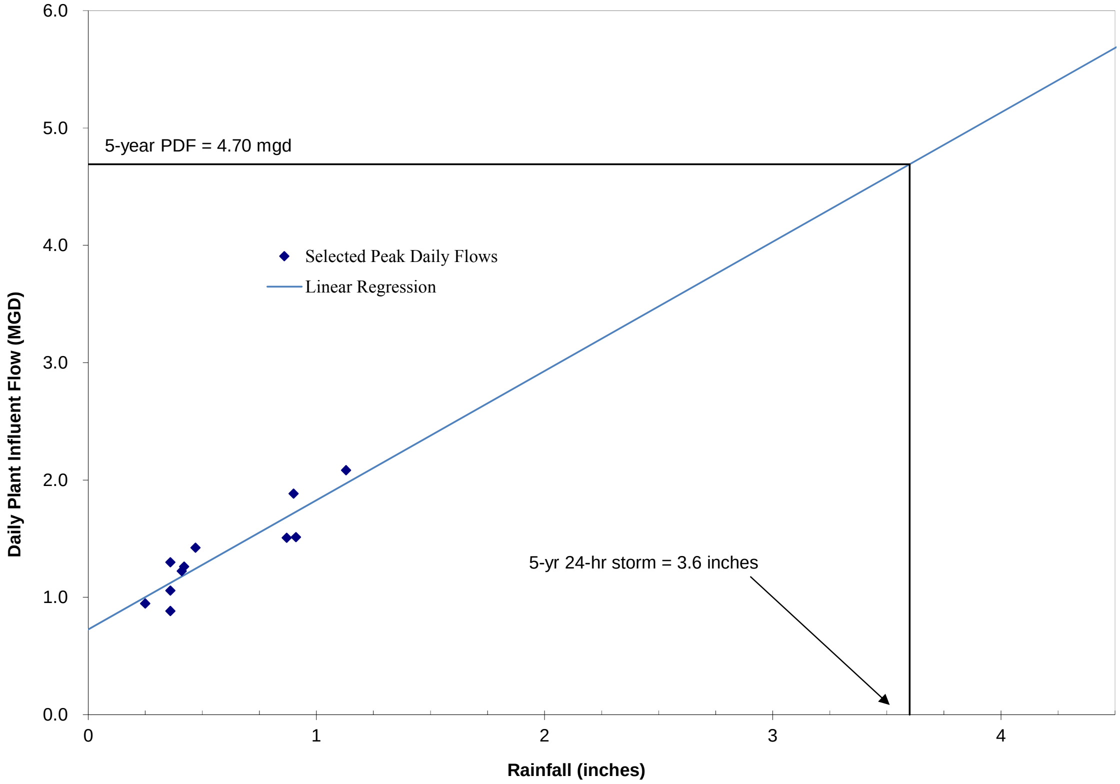
Figure 5-3 | MMWWF and MMDWF Determination



The Peak Day Flow (PDF) that would occur in the absence of bottlenecks may be estimated by determining the peak daily flow associated with a 5-year storm. This PDF will occur under saturated subsurface conditions when the influence of rainfall on infiltration and inflow is the strongest. The PDF is determined by plotting observed peak average daily flow against the corresponding daily rainfall depths. The 5-year 24-hour rainfall depth is used in a linear regression of the data to determine the PDF. The data used to determine the PDF is plotted in Figure 5-4. These data points were carefully selected to ensure that groundwater levels were saturated for the period over which flow data was collected. The data were also screened to ensure that the flow measurements were not collected under significantly surcharged conditions as this would tend to decrease the flow measurements and result in erroneously low estimates. The 5-year 24-hour rainfall depth for Philomath is approximately 3.6 inches[4]. Using this rainfall depth and the relationship established in Figure 5-4 the PDF associated with a 5-year 24-hour storm can be estimated. As shown in Figure 5-4, the PDF is approximately 4.7 MGD.

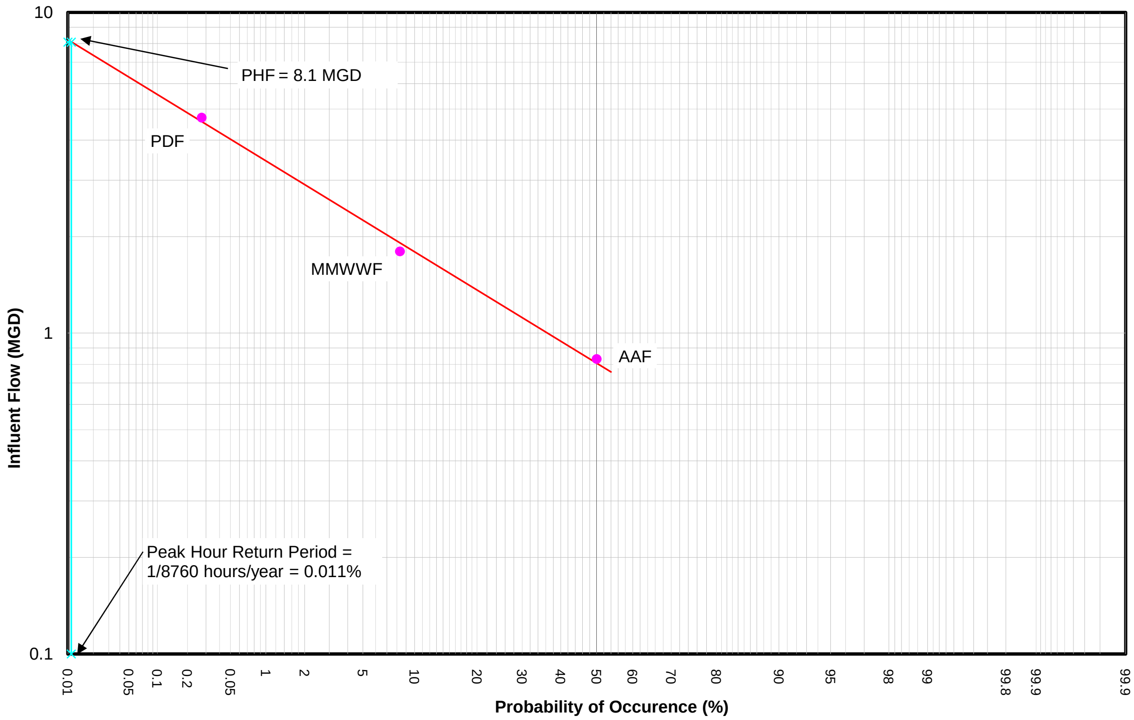
[4] U.S Dept. of Commerce, NOAA, Atlas 2, Volume X (Oregon), figure 26

Figure 5-4 | PDF Determination



A statistical approach is used to determine the Peak Hour Flow (PHF) that would occur in the absence of bottlenecks. This approach involves assuming that a particular year includes a 5-year storm with high groundwater conditions producing the MMWWF and the PDF. During this 5-year storm the PHF occurs within the peak day. These assumptions enable one to determine the portion of the year over which each flow component occurred. For example, the MMWWF occurs 1/12 of the time or with an 8.33% probability, the AAF occurs half of the time or with a 50% probability, and so on. The rainfall depth is assumed to be a random variable with a log-normal probability distribution. If this assumption is accurate, the AAF, MMWWF, and PDAF should plot as a straight line on log-probability paper. These flow components are plotted on Figure 5-5. Since the PHF occurs 1 hour out of this hypothetical year (i.e., 1/8760 or 0.011% probability), by extrapolating a linear regression to a probability of 0.011%, the PHF may be determined. Using this approach the PHF is approximately 8.1 mgd as shown in Figure 5-5.

Figure 5-5 | PHF Determination



5.3.3 Summary of Existing Wastewater Flows

Table 5-3 includes a summary of existing flow estimates presented in the previous sections. These flow estimates will be used throughout the remainder of this document. The wet weather flow components (i.e., MMDWF, MMWWF, PDF, PHF) for the City's system are intended to be the theoretical maximum values that would occur if all bottlenecks in the system were to be removed.

Table 5-3 | Summary of Existing Flow Estimates

Flow Component	Value (mgd)
Average Dry Weather Flow (ADWF)	0.518
Average Annual Flow (AAF)	0.831
Average Wet Weather Flow (AWWF)	1.15
Maximum Month Dry Weather Flow (MMDWF)	1.11
Maximum Month Wet Weather Flow (MMWWF)	1.80
Peak Day Flow (PDF)	4.70
Peak Hour Flow (PHF)	8.10

5.3.4 Wastewater Flow Projections

This section builds on the discussions of population projections in Section 5.2 and the existing flow estimates listed in Table 5-3. Projections of future wastewater flows through the planning period were based on the existing flows combined with flow from the anticipated population growth. Peaking factors were used to estimate the increases in flows during wet weather periods.

The projected wastewater flowrates were based on the following assumptions.

- Population growth will occur in accordance with the projections in Section 5.2.
- Flow rates will increase in proportion to population increase.
- The per capita average dry weather flow rate associated with the population increase will remain constant during the planning period at a value of 100 gallons per capita per day.
- There will be no contribution from “wet” industries during the planning period. Commercial and industrial development will be “dry” with flows comparable to residential developments.
- The ratio of industrial and commercial development to municipal population will remain constant over the planning period.
- The City will continue to implement infiltration and inflow reduction measures that will prevent any increase in infiltration and inflow into the existing collection system. In other words, existing I/I contributions will remain constant.
- All growth will occur in conformance with current land use policies as outlined in the City’s Comprehensive Plan.
- The increase in the AWWF over the planning period is equal to twice the increase in the ADWF.
- The increase in the MMDWF over the planning period is equal to twice the increase in the ADWF.
- The increase in the MMWWF over the planning period is equal to three times the increase in the ADWF.
- The increase in the PDF over the planning period is equal to four times the increase in the ADWF.
- The increase in the PHF over the planning period is equal to five times the increase in the ADWF.

Based on these assumptions, the future estimates of wastewater flow listed in Table 5-4 were prepared.

Table 5-4 | Future Wastewater Flow Projections

		Projected Wastewater Flows (mgd)						
Year	Population	ADWF	AAF	AWWF	MMDWF	MMWWF	PDF	PHF
2020	5431	0.60	0.95	1.31	1.26	2.03	5.42	9.30
2025	5972	0.65	1.03	1.41	1.36	2.20	5.64	9.57
2030	6567	0.71	1.12	1.53	1.48	2.38	5.88	9.87
2035	7222	0.78	1.22	1.66	1.61	2.57	6.14	10.20
2037	7294	0.78	1.23	1.68	1.63	2.59	6.17	10.23

5.3.5 Drainage Basin Service Area Flows

The peak discharge from each basin was estimated to evaluate the capacity of the trunk sewers and pump stations. Estimates of existing peak flows as well as projected peak flows associated with buildout were developed. In Chapter 6, the existing peak flows are used to determine existing deficiencies and the projected peak flows associated with buildout are used for sizing the recommended improvements. Flows associated with buildout conditions are used for sizing purposes because trunk sewers are not suited for incremental expansion. In small Cities like Philomath it is generally more cost effective to install a sewer line sized for complete development of the upstream service area. This is due to the fact that the pipe sizes are relatively small (i.e., less than 24 inches in diameter). Over the life of a particular pipeline, it is generally not cost effective to install a smaller diameter pipe (e.g., a 12-inch diameter pipe), then later replace this pipe with a larger pipe (e.g., 18-inch diameter pipe) before the smaller diameter pipe has reached the end of its useful life. Due to the relatively long life cycle of modern pipeline materials (i.e., 70+ years), it is usually more cost effective to install a larger pipe sized for buildout of the upstream basin. For this reason, peak flows associated with complete buildout of the UGB are used in this plan to size the trunk sewers in the City.

The peak flow from each basin at buildout conditions was determined by summing the following quantities.

- Existing average dry weather flow multiplied by a peaking factor of 3
- Existing I/I contribution
- Additional base sewage flow from growth multiplied by a peaking factor of 3
- Additional I/I from future development

The existing ADWF as measured at the treatment plant was allocated to each sewer basin by the ratio of the sewered area within each basin to total sewered area of the City. The existing I/I contribution from each basin was estimated based on the age and total length of piping within each basin.

The additional ADWF associated with growth in the basin was determined by multiplying estimates of sewage flow per acre (Table 5-5) by the area of undeveloped land for each land use within each basin. A peaking factor of three was applied to these values to estimate PHF from new development. The additional I/I from future development was determined by multiplying 1,600 gallons per acre per day by the total undeveloped area within each basin. This allowance for I/I in currently undeveloped areas is used only to size the collection system piping serving those areas. The overall I/I collected in the City is anticipated to remain relatively constant due to the recommended rehabilitation and replacement program described later in this document (section 6.2.5).

Table 5-5 | Flow Rates Per Acre Used for Estimates of Flow from Undeveloped Areas

Land use	Flow (gallons/acre/day)
Low Density Residential	1,300
Medium Density Residential	2,000
High Density Residential	3,000
Commercial	1,500
Industrial	1,500
Public	500

Using the approach described above, the existing peak flows and the projected peak flows at buildout were determined (Table 5-6).

Table 5-6 | Projected Drainage Basin Service Area Flows at Buildout of the System

Basin	Total Area (Acres)	Sewered Area (Acres)	Existing ADWF[1] (mgd)	Existing I/I (mgd)	**Existing PHF (mgd)**	Future ADWF[1] (mgd)	Future I/I (mgd)	**Buildout PHF (mgd)**
A1	56.3	52.2	0.037	0.297	**0.408**	0.005	0.007	**0.429**
A2	103.5	53.4	0.038	0.403	**0.516**	0.075	0.080	**0.821**
A3	14.7	14.7	0.010	0.175	**0.207**	-	-	**0.207**
A4A	61.4	45.5	0.032	0.829	**0.925**	0.024	0.025	**1.024**
A4B	49.6	44.5	0.031	0.519	**0.613**	0.011	0.008	**0.655**
A5A	84.5	14.1	0.010	0.142	**0.172**	0.096	0.113	**0.572**
A5B	41.0	30.3	0.021	0.196	**0.260**	0.025	0.017	**0.353**
A6	106.2	89.3	0.063	1.061	**1.250**	0.024	0.027	**1.348**
A7	96.6	60.2	0.043	0.131	**0.258**	0.048	0.058	**0.462**
N1A	88.5	0	-	-	**-**	0.115	0.142	**0.487**
N1B	63.5	0	-	-	**-**	0.083	0.102	**0.349**
N2	56.8	0	-	-	**-**	0.074	0.091	**0.312**
N3AA	54.4	42.3	0.030	1.002	**1.091**	0.018	0.019	**1.165**
N3AB	61.9	54	0.038	0.485	**0.599**	0.004	0.013	**0.624**
N3B	40.9	39.9	0.028	0.495	**0.579**	0.001	0.002	**0.585**
N3CA	40.9	40.9	0.029	0.620	**0.707**	-	-	**0.707**
N3CB	33.5	28.7	0.020	0.033	**0.094**	0.006	0.008	**0.121**
N3D	62.1	62.1	0.044	0.217	**0.349**	-	-	**0.349**
N4	110.9	8.9	0.006	0.014	**0.033**	0.151	0.163	**0.648**
N5	263.7	0	-	-	**-**	0.396	0.422	**1.609**
N6	341.6	0	-	-	**-**	0.498	0.547	**2.040**
N7A	185.0	0	-	-	**-**	0.242	0.296	**1.022**
N7B	50.1	0	-	-	**-**	0.084	0.080	**0.332**
N8A	99.2	39.5	-	0.063	**0.147**	0.098	0.096	**0.535**
N8B	131.0	0	-	-	**-**	0.235	0.210	**0.913**
N8C	36.2	0	-	-	**-**	0.072	0.058	**0.275**
N9	37.2	12.8	0.009	0.020	**0.047**	0.016	0.039	**0.134**
P1	123.2	0	-	-	**-**	0.141	0.197	**0.620**
P2	54	0	-	-	**-**	0.110	0.086	**0.418**
Totals	2548	733	0.518	6.682	**8.256**	2.652	2.904	**19.115**

Notes
1) ADWF multiplied by a peaking factor of 3 to estimate peak hour flow

5.4 WASTEWATER LOADS

In addition to the expected wastewater flow, evaluation and design of wastewater facilities requires estimates of the expected loads of various pollutants in the wastewater. Treatment facilities must be designed with operating capacity to treat the highest expected loads of pollutants over the planning period. Pollutants used as design parameters for this study were biochemical oxygen demand (BOD; sometimes referred to as a five-day oxygen demand expressed as BOD_5) and total suspended solids (TSS). The following classifications of wastewater pollutant loads were used.

- Average Load – Average daily wastewater load.
- Maximum Month Load – Daily wastewater load during the maximum month.

5.4.1 City Wastewater Treatment Plant Load Records

The City's treatment plant Discharge Monitoring Reports (DMRs) filed with the DEQ for the period from 2013 through 2015 were evaluated to identify loading patterns and evaluate current loads to the plant. This data set includes BOD and TSS measurements from 24 hour composite samples taken from the wastewater treatment plant influent flow stream two times per month.

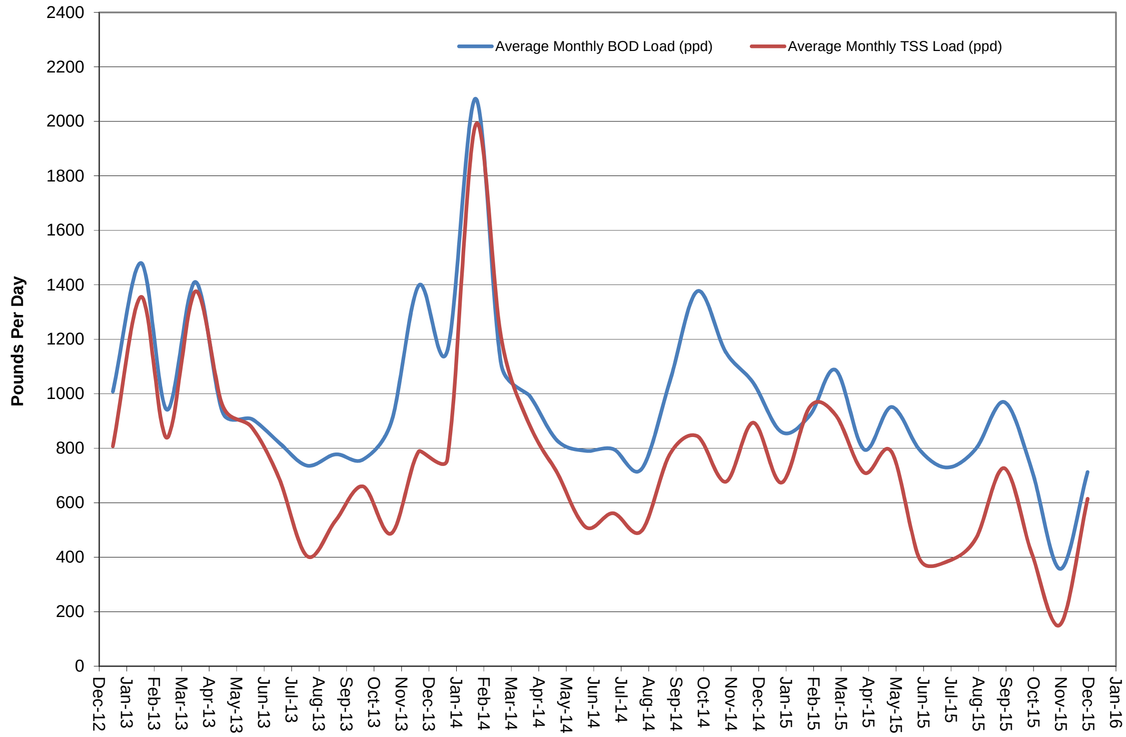
Pollutant loads in pounds per day were calculated for BOD and TSS using the data set described above. Pollutant load calculations were based on the concentration for each pollutant multiplied by the influent flow on the day the sample was collected.

The average monthly influent BOD and TSS loads measured at the treatment plant are plotted in Figure 5-6. The annual average influent loading and the maximum month loading are listed in Table 5-7 for BOD and TSS.

Table 5-7 | Summary of Plant BOD and TSS Loading Data 2013 through 2015.

	BOD Load (pounds per day)		TSS Load (pounds per day)	
Year	Average Annual	Maximum Month	Average Annual	Maximum Month
2013	1010	1980	810	1380
2014	1080	2080	850	1990
2015	800	1090	600	950
Average	960	1720	750	1440

Figure 5-6 | Plant BOD and TSS Loading History



The average population of Philomath from 2013 to 2015 was approximately 4,635 [5]. Dividing the average loading rates listed in Table 5-7 by this population results in average per capita BOD and TSS loading rates of 0.21 and 0.16 respectively. Based on the engineering literature[6], typical BOD values in domestic wastewater fall in the range of 0.11 – 0.26 pounds per capita per day. TSS values are typically in the range of 0.13-0.33 pounds per capita per day. The BOD and TSS loading rates in Philomath are within these ranges. Therefore, the loading data collected at the treatment plant is generally considered to be reliable.

5.4.2 Load Projections

This section builds on the discussions of population projections in Section 5.2 and the existing load data listed in Table 5-7. Projections of future wastewater loads through the planning period were based on the existing loads combined with loads from the anticipated population growth. Peaking factors were used to estimate the increases in loading rates for the peak month.

The projected wastewater loading rates were based on the following assumptions.

- Population growth will occur in accordance with the projections in Section 5.2.
- BOD and TSS loading rates will increase in proportion to population increase.

[5] Certified Population Estimates, Population Research Center, Portland State University
[6] Metcalf & Eddy. 2003

- All growth will occur in conformance with current land use policies as outlined in the City's Comprehensive Plan.
- The per capita BOD loading rate for new population growth will be 0.21 pounds per person per day.
- The per capita TSS loading rate for new population growth will be 0.18 pounds per person per day.
- The ratio of peak monthly BOD and TSS loads to average loads is 1.8.

Based on these assumptions, the future estimates of influent wastewater loads listed in Table 5-8 were prepared.

Table 5-8 | Future Wastewater Load Projections

Year		BOD (ppd)		TSS (ppd)	
	Population	Average Annual	Peak Month	Average Annual	Peak Month
2020	5431	960	1728	750	1350
2025	5972	1124	2023	891	1603
2030	6567	1238	2228	988	1778
2035	7222	1363	2453	1095	1971
2037	7294	1500	2700	1213	2183

CHAPTER 6

COLLECTION SYSTEM EVALUATION

Chapter Outline

COLLECTION SYSTEM EVALUATION

6.1 INTRODUCTION

This chapter includes an analysis of the collection system. The first subsection focuses on operation, maintenance, and rehabilitation of the collection system. This is followed by the development of alternatives for potential improvements to the wastewater collection system.

This chapter addresses the following key questions:

- What are the current collection system operation and maintenance practices and how can they be improved?
- What are the existing collection system deficiencies?
- What collection system components are likely to become deficient during the planning period or prior to complete buildout of the system?
- What are the alternatives for correcting existing and projected deficiencies?

The existing and projected collection system deficiencies are presented. Where appropriate different alternatives for addressing each of the deficiencies are presented and discussed. The alternatives are evaluated against each of the collection system deficiencies to generate complete collection system recommendation. In Chapter 7, the treatment system is evaluated and alternatives for correcting treatment system deficiencies are identified and evaluated.

6.2 COLLECTION SYSTEM OPERATION, MAINTENANCE, & REHABILITATION

This section discusses the need for maintenance of the gravity sewer collection piping and provides recommendations for the basic elements necessary for a maintenance program. The need for system-wide preventive maintenance is addressed first, and then the general recommended approaches to collection system maintenance are outlined.

6.2.1 Need for System-Wide Preventative Maintenance

Maintenance of sewerage systems is necessary to insure the proper operation of the facilities and to obtain the full useful life of those facilities. Sanitary sewer systems represent significant investment of public capital. If a sewer system is allowed to fall into disrepair because of the lack of maintenance, it will not operate efficiently or as designed. Health problems and property damage may result from sanitary sewer backups, surcharging and/or overflows. Without proper maintenance, a system's capacity can be reduced by debris clogging, root intrusion growth, structural damage, infiltration and inflow (I/I), and other factors that eventually lead to failures throughout the system. Repair of failed sections of a sanitary sewer system are costly, quite often exceeding the original cost of construction. In spite of this, many jurisdictions do not adequately fund the level of maintenance necessary to protect their investment in the sewerage system. Collection system maintenance can be separated into two types: preventive and corrective.

Preventive maintenance involves scheduled inspection of the system and data gathering to identify problem areas and analysis of this data so that scheduled maintenance can be targeted at specific problems. As a general rule, as preventative maintenance increases, the amount of corrective maintenance required decreases.

Corrective maintenance, often referred to as emergency maintenance, is typically performed when the sewer system fails to convey sewage. Causes for initiating corrective maintenance may include blockages, solids buildup, excessive I/I, flooding and sewer breaks. Corrective maintenance requires immediate action, and the jurisdiction will typically pay a premium to have this work performed.

6.2.2 Present Maintenance Practices

At the present time, the City has a good collection system maintenance program. The City typically cleans all sanitary sewer mainlines and inspects all manholes on an annual basis. Spot repairs are completed on an as-needed basis. Those portions of the collection system that are more susceptible to plugs or similar problems are inspected and cleaned three times per year. The City also cleans all pump station wet wells three times per year. Before the end of the 2016 calendar year, the City intends to procure a sewer television camera inspection truck. With this equipment, the City will likely begin a regular television inspection program. The exact goals for the amount of the system that will be inspected on an annual basis will be refined in the coming years based on the City's experience operating the equipment.

In addition to maintenance of the system, the City has aggressively implemented a sanitary sewer rehabilitation program. In 2004, the City began a long-range program to rehabilitate the original collection system that was constructed in 1952. Since 2004, the City has rehabilitated approximately 9,000 feet of the 1952 system. The original 1952 system included approximately 35,000 feet of mainline. Therefore, the City has rehabilitated approximately 26% of the system.

6.2.3 Preventative Maintenance Program Recommendations

The following paragraphs outline some recommendations for implementing preventive and corrective maintenance throughout the City's sanitary sewer collection system. These include the following:

- Continue the existing collection system maintenance program.
- Continue the sewer rehabilitation and replacement program.

6.2.4 Collection System Maintenance Program

As described above, the City's existing collection system maintenance program is good and it is important that the program be continued indefinitely. Regular cleaning is necessary to prevent blockages, grease accumulation and sediment buildup in sewer lines. Normally, sanitary sewers laid at steep grades require less frequent cleaning than those laid at flat grades. Sewers at flat grades can experience sedimentation and grease buildup problems and will require more frequent cleaning and maintenance.

As part of the cleaning program, it is important that the City continue to keep records, including conditions encountered such as pipe failures, grease and solids buildup, and other problems. These records are useful in scheduling corrective work and to establish a long term cleaning frequency schedule for different sewers. As the database is established, a schedule for subsequent cleaning can be tailored to the physical character of each line, the area served, and its performance history. Specific problem areas requiring more frequent cleaning can be incorporated into this program.

The inspection component of the program should include both above ground and internal inspection of the sewer system. Above ground inspection is performed by inspecting right-of-ways and easements and noting evidence of structural failure, flooding, manhole covers above or below the present level of streets, or other problems.

The two common methods of internal inspection are TV inspection performed in conjunction with the cleaning activities, and smoke testing. TV inspection of a sewer system utilizes a specially designed television camera and equipment to view the interior of the piping system. A videotape and written record of the inspection is generated and retained by the City. Leaking sewer service connections, debris or root buildup, structural failures, leaking joints and other problems can be easily identified and documented. TV inspection of sewers requires that the sewers be cleaned immediately prior to the inspection. TV inspection of sewers is typically performed during the winter months so that sources of I/I can more easily be noted and identified.

Smoke testing is conducted by blowing harmless nontoxic smoke into the sewer system and observing the points at which it escapes. Smoke testing is typically performed during the summer months so that groundwater does not interfere with the smoke. Smoke testing can be used to identify potential leaks into the system caused by broken pipes, bad joints, manhole failures, and similar deficiencies. Smoke testing is also very effective for locating storm sewer cross connections and illegal connections such as roof and foundation drains.

Overall, the City existing maintenance practices are good and generally conform to the above recommendations. The only recommended change to current practices is to perform smoke testing of the collection system early in the coming years. Smoke testing has not been completed in the recent past and is a good way to identify storm water inflow sources for later repair.

6.2.5 Sewer Rehabilitation & Replacement Program (Program – 1)

As described in Chapter 4 (subsection 4.4.5), the City's original collection system was constructed in the early 1950's and collects large amounts of groundwater infiltration. The original collection system will be more than 80 years old by the end of the planning period. As such, it will likely reach the end of its useful life during planning period due to the age of the piping. Rehabilitation of the 1952 collection system is a well-established policy in the City. In 2004, the City adopted a facilities plan that recommended replacing all of the 1952 collection piping over a 30 year period. Since that time, the City has aggressively implemented rehabilitation measures. In 2004, approximately 35,000 feet of the 1952 system was in place. Of this amount approximately 9,000 feet has been rehabilitated leaving approximately 26,000 feet of the 1952 system in place. The City plans to rehabilitate approximately 5,600 feet of the 1952 system in 2017. The 2017 project may be nearing completion as this plan is being adopted by the City. Therefore, the 2017 project is considered to be complete for the purposes of this document. With completion of the 2017 project, approximately 20,400 feet of the original 1952 system remains in service and the City should continue to rehabilitate the remaining system during the planning period.

To determine the appropriate funding rate for the proposed sewer rehabilitation program, one simply needs to sum all the mainlines, manholes, and service laterals that are to be included in the rehabilitation scheme (i.e., determine the scope of the work effort), estimate the total cost of rehabilitating these facilities, and determine the number of years over which the rehabilitation should occur. The recommended scope of the rehabilitation effort includes the remaining 1952 collection system. This includes approximately 20,400 feet of mainline pipe, 65 manholes, and 15,000 feet of service lateral piping. Assuming that approximately 75% of the service laterals will need to be replaced, the total cost for this recommended rehabilitation work is listed in Table 6-1.

Table 6-1 | Sewer Rehabilitation Program Total Costs

Item	Quantity	Unit Cost	Total Cost
Sewer Mainline	20,400 ft (1)	$110 / ft (2)	$2,244,000
Sewer Manholes	65 (3)	$4,500 / each (3)	$292,500
Service Laterals	12,000 ft	$60 / ft	$720,000
Total Rehabilitation Construction Cost			**$3,256,500**

Notes:
(1) Total length of 1952 mainline piping in place upon the completion of the 2017 rehabilitation project.
(2) Average unit cost based on a typical mix of CIPP, pipe bursting, and open cut reconstruction.
(3) Average unit cost base on a typical mix of replacement and rehabiliation.

As shown (Table 6-1), the total construction costs for the recommended rehabilitation project is approximately $3,257,000 in 2016 dollars. To account for soft costs, engineering is assumed to be 15% of the construction cost. Legal, permitting, and administration costs are assumed to be 5% of the construction cost. A construction contingency of 5% is also added. Therefore, the total soft costs are assumed to be 25% of the construction costs. Including soft costs, the total project costs for the recommended sewer rehabilitation plan is approximately $4,070,000.

With the 2004 facilities plan, the City adopted the goal of rehabilitating the 1952 collection system by 2035. This roughly corresponds to the end of the current planning period. Assuming that the recommended rehabilitation work is completed over the 20 year period, the annual funding rate should be approximately $200,000 per year. As with all the cost estimates presented in this plan, this budget amount is in 2016 dollars and will need to be adjusted over the years to account for increases in construction costs. At the end of the planning period other portions of the collection system will be another 20 years older and will have deteriorated further. Therefore, once the rehabilitation efforts are completed for the original 1952 collection system, the City may want to consider establishing a permanent rehabilitation program. Future rehabilitation efforts should focus on other problem areas in the City. Upon the complete rehabilitation of the 1952 collection system, the City should re-evaluate the scope and funding needs of the program. Soon after this plan is adopted, the City should consider smoke testing the entire collection system to identify inflow sources that can be disconnected relatively inexpensively.

6.3 COLLECTION SYSTEM DEFICIENCIES

The purpose of this section is to determine the components of the existing collection system that are or will become deficient. This includes components that lack capacity to convey existing peak flows or will lack capacity as flows increase due to growth. The intent of this section is to present an overall list of deficiencies that must be addressed by the City.

6.3.1 Gravity Main Capacity Analysis

The peak design flows developed in Chapter 5 were used as the basis for an evaluation of the existing sanitary sewer trunk lines. Pipe sizes, lengths, slopes, and locations were determined from City records. The evaluation was limited to the main trunk lines conveying sewage through the basins. This approach was taken since most of the pipes within a basin will actually encounter only a fraction of the capacity of the pipe. Typical practice is to construct sewer lines with pipe no smaller than 8-inches in diameter. This facilitates solids conveyance, cleaning, and maintenance. In the upper ends of the drainage basins, flows do not approach the capacity of the 8-inch diameter pipes. Therefore, it is not necessary to model all of the smaller diameter pipes in the collection system.

A model of the main trunk lines was developed using the SWMM5 hydraulic model. The hydraulic model simulates the routing of flow through the collection system. SWMM5 is a fully dynamic model that can simulate backwater, surcharging, split flows, and looped connections that occur in sewer systems. The peak drainage basin service area flows (Table 5-6) were used as inputs to the model. Both the existing peak flows and the projected peak flows associated with buildout were used in the modeling effort. The existing peak flows were used to determine existing deficiencies, and the projected peak flows associated with buildout were used for sizing the recommended improvements. The choice to use flow projections associated with buildout of the collection system for trunk sewer sizing is based on the fact that buried sewer collection pipes are not well suited for incremental expansion. Cases rarely exist where it is appropriate to size trunk sewers for 20 year flow projections. The design life of buried sewer collection pipes is 50-70 years. Therefore, it is not cost effective to upsize these sewer pipelines at 20-year intervals. It is more cost effective to size these facilities to convey projected peak flows associated with buildout of the entire upstream basin.

The existing and projected flow estimates were added to the main trunk lines where their respective basins discharge into the main trunk lines. The model was run until steady-state flow conditions were achieved. These steady state conditions were used to locate the collection system deficiencies. This approach is somewhat conservative since, in reality, the peak drainage basin service area flows only persist for a short period of time (e.g., a few hours). After these peaks, the flows will begin to decrease and steady state conditions are not likely to actually occur.

Though somewhat conservative, this steady-state approach is reasonable for smaller systems like Philomath.

The model was used to identify capacity deficiencies. Capacity deficiencies are defined as locations where overflows occur and flow does not reach the treatment plant, or where a pipe is surcharged and the hydraulic grade line (HGL) is within a specified distance from the ground surface. For the purposes of this analysis, pipe surcharge is allowed. When the modeled water surface reached a level less than 6 feet from the ground surface (freeboard less than 6 feet) a deficiency was identified. The 6-foot freeboard deficiency criterion was determined to be appropriate for short-term peak flows and adequate to protect from overflows. Basement flooding was not considered to be a significant concern given the relatively limited number of basements in the City and the lack of historical basement flooding complaints. For shallow pipes (pipes with less than 8 feet of available freeboard measured from ground to top of pipe) a capacity deficiency criterion that allows no more than 2 feet of surcharge was used instead of 6 feet minimum freeboard allowed for deeper pipes. The capacity deficiencies identified by the hydraulic analysis indicate where improvements may be needed to ensure that overflows do not occur and that adequate capacity is provided.

The hydraulic model was used to identify capacity deficiencies in the existing trunk sewer system as shown in Figure 6-1. As noted above, the flows used for this analysis are the existing peak drainage basin service area flows (Table 5-6). The hydraulic model predicts widespread surcharging throughout the City. However, the depths of the surcharging are relatively minor and generally below the deficiency criteria identified in the previous paragraph. The model does predict significant surcharging the areas identified in Figure 6-1. These areas are considered to be deficient, and improvements to address these deficiencies are identified later in this chapter.

Figure 6-1 | Existing System Capacity Analysis (Existing Flow Conditions)

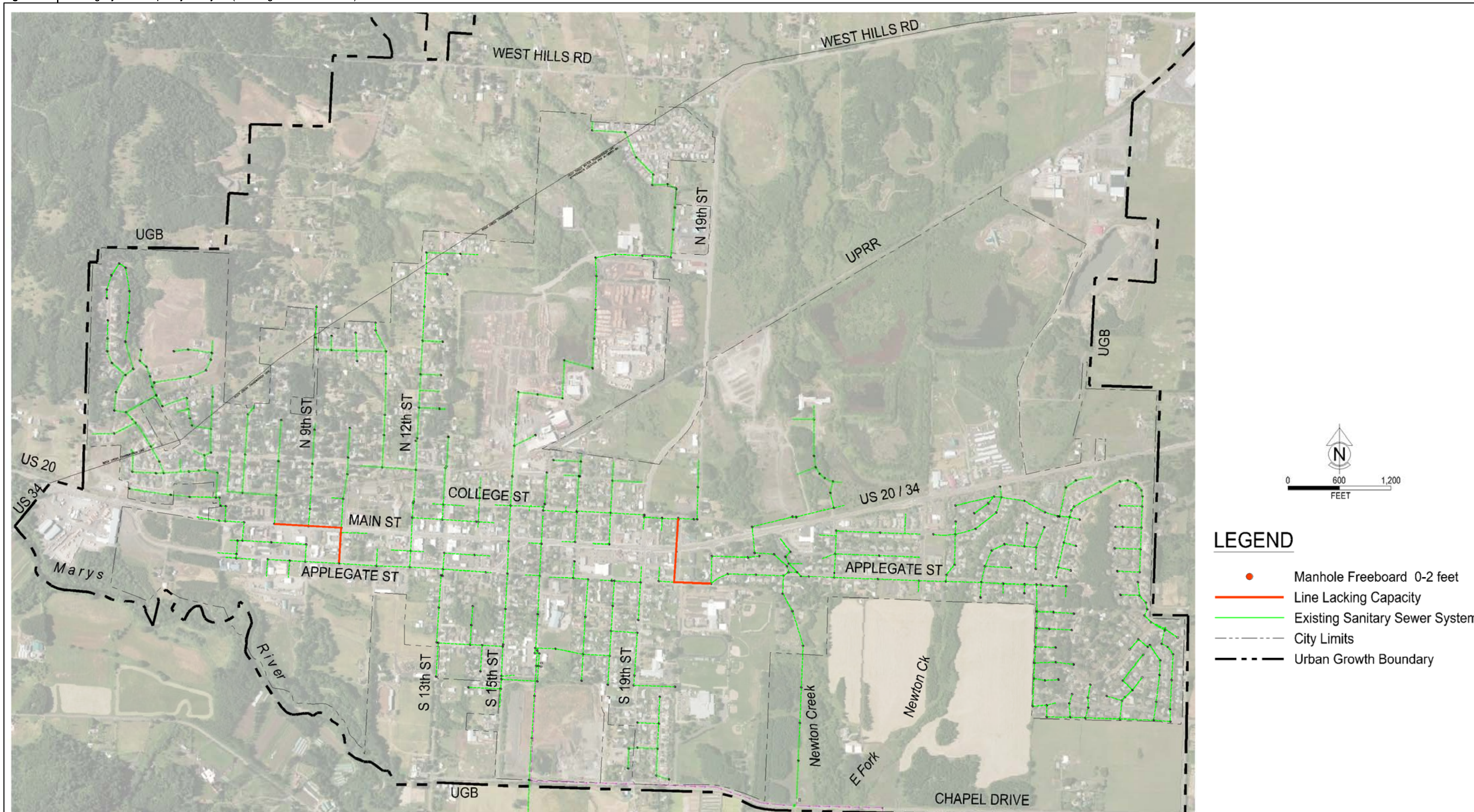


FIGURE 6-1

6.3.1 Collection System Improvements to Serve Currently Undeveloped Areas

In addition to the sewers lacking capacity, there are a number of areas within the City that are currently undeveloped and/or areas that lack gravity sewer service. New gravity mainlines will need to be installed to serve underdeveloped or underserved areas as they develop. In some cases pump stations may be needed to convey wastewater to the existing system. Current City ordinances require that mainlines serving these areas are to be installed at the expense of the developer. These lines should be sized as required to serve all upstream areas. Also some lines downstream of the undeveloped areas will need to be upsized to accommodate the additional flow from the newly developed areas. The recommended improvements to serve the undeveloped areas are discussed below in section 6.5.

6.3.2 Pump Station Capacity Analysis

To evaluate the pumping capacity of the existing stations, a hydraulic model of the three pump stations feeding into the common forcemain was developed using the WaterCAD software package. Physical data such as pipe size and station elevations were taken from as-built drawings. The capacities of each pump were taken from manufacturer's pump curves. For the sake of estimating the firm capacity of each station, the model was run with the largest single unit at each station out of service. The estimated firm capacities were then compared to the estimated peak hour flows to each station (Table 6-2). The peak hour flows to each station were determined from the peak flows from each basin routed through the collection system using the SWMM5 software package as described above (Section 6.3.1).

Table 6-2 | Summary Pump Station Capacity Analysis

Pump Station	Estimated Firm Capacity(1) (MGD)	Existing Estimated Peak Flows (MGD)	Estimated Peak Flows at the End of the Planning Period (MGD)	Required Buildout Capacity (MGD)
Pump Station A	4.75	4.60	5.1	5.9
Newton Creek Pump Station	2.00	3.45	4.0	12.2
Timber Estates pump Station	0.0	0.05	0.05	0.05

(1) Firm capacities based on all but the largest single unit running at each station.

This analysis shows that Pump Station A lacks the capacity to convey projected peak flows at the end of the planning period, but has adequate capacity for existing peak flows. The modelling shows that the Newton Creek Pump Station lacks the capacity to convey existing peak flows. The model also shows that the Timber Estates Pump Station lacks the ability to pump any water due to the high pressures in the forcemain during peak pumping events. This "blinding off" of the Timber Estates Pump Station during peak pumping events has been observed in the field and should be corrected during the planning period.

It is important to note that the flow projections listed in Table 6-2 are intended to be flows associated with the 5-year 24-hour storm event in the absence of bottlenecks in the system (see section 5.3.2). To some extent, these are theoretical peak flow rates that are unlikely to materialize from a practical point of view. Under 5-year 24-hour storm conditions, large portions of the collection system will be surcharged. This will tend to reduce the actual flowrate to the stations under very large storm events.

As another approach for evaluating the capacity of the pump stations, the pump run times were reviewed for 2012 through 2015. The pump run time data shows there was never a time from 2012 through 2015 when the Newton Creek Pump Station and Pump Station A were operating above capacity. This means, that the two smaller pumps and one of the larger pumps at each station were able to successfully convey flows entering the station. This suggests that improvements are not likely needed early in the planning period, but will be required as the City grows. It is also important to note that there has never been a documented raw sewage overflow from the collection system. This observation again supports the idea that the stations do not have an immediate capacity problem. As one looks out over the planning periods, it does make sense for the City to have a plan in place to increase the capacity of Pump Station A and the Newton Creek Pump Station. However, the actual implementation of this plan may not be needed for many years. Recommendations for improvements to the pump stations are described below.

6.3.3 Summary of Collection System Deficiencies

The known deficiencies described in Chapter 4 have been combined with the deficiencies described above to develop a complete list of collection system deficiencies. These are listed below (Table 6-3).

Table 6-3 | Summary of Collection System Deficiencies

Location	Problem Category
Original 1952 Collection System	High I/I, End of Useful Life
10th Street Sewer from Applegate Street to Main Street	Lack of Capacity
Main Street Sewer from 10th Street to 8th Street	Lack of Capacity
20th Street Sewer from Applegate Street to College Street	Lack of Capacity
Applegate Street Sewer from 21st Street to 20th Street	Lack of Capacity
Timber Estates Pump Station	Lack of Capacity, End of Useful Life (Mech. Equip)
Newton Creek Pump Station	May Lack Capacity to Convey Existing Peak 5-year 24 hour flow event
Pump Station A	May Lack Capacity to Convey Peak 5-year 24 hour flow event at the end of the planning period
Undeveloped Areas	No Sewer Service

6.4 COLLECTION SYSTEM ALTERNATIVES

The shortcomings identified in Table 6-3, will need to be addressed by implementing a comprehensive I/I correction program, increasing the size of trunk sewers, extending gravity sewer service to currently undeveloped areas, and constructing pump station or forcemain improvements.

Facilities planning requires the examination of a broad range of alternatives for each portion of the wastewater system. This section examines the alternatives for collecting wastewater within the study area and conveying it to the point of treatment. This section develops and screens wastewater collection alternatives using criteria such as land requirements, topographic constraints, reliability, operational flexibility, construction and long-term O&M costs, and regulatory restrictions. The alternatives listed in this section represent the tools used in the facilities planning effort to address the previously listed deficiencies in order to provide a comprehensive long-term solution for the City's collection system.

6.4.1 No Action

The no action approach implies that no improvements will be made to the existing collection system (excluding maintenance or repairs). Obviously, this approach is recommended for those areas of the system which have sufficient capacity to convey the design flows and are in acceptable condition. Although this approach may be justified in isolated areas within the system on a case-by-case basis where there is insufficient capacity to convey peak design flows (i.e., minor surcharging for short periods of time), this approach is effectively eliminated by DEQ guidelines and regulations.

Although it is always an option to not improve the system, the result will be health risks, damages, and inconveniences where sewage collection and facilities are inadequate. Furthermore, delaying required improvements almost inevitably leads to a greater future problem. However, to ensure that system improvements are justified, it is necessary to consider the costs and advantages of proposed improvements against the risks entailed by the no action alternative. It should be noted that since resources are limited and the sewer system cannot be upgraded all at one time, the phasing plan adopted by the City for the improvements will in effect require that the no action alternative be adopted on a temporary basis for all but the first phase improvements.

6.4.2 Reroute Sewage

Under this scenario, sewage would be diverted or rerouted from one sewer basin or system to another. This approach is practical in cases where an existing sewer has capacity in excess of that needed to convey design flows from that basin, and where flow diversion is practical from a construction and topographic standpoint.

6.4.3 Upgrade Existing Facilities

This approach involves constructing replacement pipes or pump stations to provide adequate capacity for the design flows. This is the most obvious alternative since it provides assurance that the sewage collection system can convey the design flows through the City and that overflows will be kept to a minimum, which in turn limits the City's liability and health risks to residents.

6.4.4 Infiltration/Inflow Reduction

As stated previously, the collection system collects large amounts of I/I during the winter months. While reduction of the existing I/I flows and minimization of future I/I flows is important, experience in Western Oregon has shown that the goal of complete elimination of I/I is unreasonable and largely unattainable. For the purposes of this study, it was assumed that I/I reduction efforts would keep I/I amounts at their current level. In other words, no reduction in flows is assumed as a result of the recommended sewer rehabilitation and replacement program (i.e., Program-1). This assumption is based on the idea that I/I reduction should be an ongoing work effort included in the City's maintenance budget indefinitely. This approach is recommended because as the I/I corrective work is performed, other areas in the collection system will continue to age and deteriorate and new I/I sources will appear over time. These new I/I sources will replace the I/I sources that were removed as a result of the corrective work. This assumption may turn out to be somewhat conservative. If so, future flow projections during the next planning cycle can be adjusted accordingly.

6.4.5 Construct New Facilities

The construction of new collection system components including trunk sewers, lift stations, and force mains is the only method considered herein for providing service to undeveloped areas. This method basically involves extending the conventional gravity collection system into the undeveloped areas and installing new pump stations where topographical limitations require. Septic tank effluent pumping (STEP) or Septic tank effluent gravity (STEG) collection systems were not considered practical given the City's reliance on a conventional gravity system and the potential deterioration of concrete components in the existing system from hydrogen sulfide present in STEP and STEG effluents.

6.5 RECOMMENDED GRAVITY COLLECTION SYSTEM IMPROVEMENTS

To address the I/I problems in the original 1952 collection system, the I/I reduction plan (i.e., Program-1) is recommended. This program is discussed in greater detail above (Section 6.2.5).

To address the capacity problems listed in Table 6-3, it is recommended that several sewer segments be replaced with larger diameter lines. In conjunction with the replacement of all sewer lines, it is also recommended that the manholes and service laterals either be replaced or rehabilitated to help control I/I.

To provide service to areas that are currently undeveloped, future pump station locations, trunk sewer sizes, and conceptual alignments are also recommended. It is important to note that the actual alignment of these sewers will likely change from those shown when the undeveloped areas are platted and the public right of ways are established.

As noted previously, the recommended pipe sizes are based on complete buildout of the UGB in its current configuration. The decision to size the trunk sewers to convey peak flows associated with buildout conditions is based on the fact that buried trunk sewer pipelines are not well suited for incremental expansion. In other words, it is more cost effective in the long-run to install trunk sewers sized for complete buildout of the upstream basin rather than for 20-year flow projections.

The recommended sewer pipeline improvements are described in the following subsections. Maps graphically showing these improvements are included at the end of this section (Figure 6-2 through Figure 6-5). Cost estimates along with a ranked prioritization of these projects into a comprehensive capital improvement plan is presented in Chapter 8.

6.5.1 Recommended Improvements to the Existing Collection System

- *9th Street to 7th Street Sewer Lines – Manhole #35 to Manhole #184 (Project G-1)*

The existing 8-inch diameter pipes lack the capacity to convey projected peak flows. These pipes also are also part of the original 1952 collection system. The recommended improvements include replacing the existing pipes with approximately 1,450 feet of new 10-inch diameter pipe (Figure 6-4). The City is currently planning on replacing these lines in the summer of 2017. As such, this project may be complete by the time this document is formally adopted by the City.

- *10th Street Sewer Line –Manhole #34 to Manhole #45 (Project G-2)*

The existing 8-inch diameter pipes lack the capacity to convey existing peak flows. These pipes also are also part of the original 1952 collection system. The recommended improvements include replacing the existing pipes with approximately 400 feet of new 12-inch diameter pipe (Figure 6-4). The City is currently planning on replacing this segment in the summer of 2017. As such, this project may be complete by the time this document is formally adopted by the City.

- *Main Street Sewer Line – Manhole #45 to Manhole #52 (Project G-3)*

The existing 8-inch diameter pipes lack the capacity to convey existing peak flows. These pipes also are also part of the original 1962 collection system. The recommended improvements include replacing the existing pipes with approximately 800 feet of new piping. A 12-inch dimeter pipe is recommended from 10th to 9th Streets and a 10 inch diameter pipe is recommended from 9th to 8th Streets (Figure 6-4). The City is currently planning on replacing this segment in the summer of 2017. As such, this project may be complete by the time this document is formally adopted by the City.

- *8th & College Street Sewer Lines – Manhole #52 to Manhole #56 (Project G-4)*

The existing 8-inch diameter pipes lack the capacity to convey projected peak flows. These pipes also are also part of the original 1952 collection system. The recommended improvements include replacing the existing pipes with approximately 750 feet of new 10-inch diameter pipe (Figure 6-4).

- *Pioneer and 11th Street Sewer Lines – Manhole #71 to Manhole #74 (Project G-5)*

The existing 8-inch diameter pipes lack the capacity to convey projected peak flows. These pipes also are also part of the original 1952 collection system. The recommended improvements include replacing the existing pipes with approximately 800 feet of new 10-inch diameter pipe (Figure 6-4).

- *15th Street Trunk Sewer (South) – Manhole #27 to Manhole #288 (Project G-6)*

The existing 8-inch diameter pipes lack the capacity to convey projected peak flows. These pipes also are also part of the original 1952 collection system. The recommended improvements include replacing the existing pipes with approximately 1,650 feet of new 12-inch diameter pipe (Figure 6-4).

- *15th Street Trunk Sewer (North) – Manhole #288 to Manhole #94 (Project G-7)*

The existing 8-inch diameter pipe lacks the capacity to convey projected peak flows. This pipe is are also part of the original 1952 collection system. The recommended improvements include replacing the existing pipe with approximately 350 feet of new 12-inch diameter pipe (Figure 6-4). The City is currently planning on replacing this segment in the summer of 2017. As such, this project may be complete by the time this document is formally adopted by the City.

- *Applegate Street and 20th Street Trunk Sewer - Manhole #1 to Manhole #6 (Project G-8)*

The existing 8-inch diameter pipes lack the capacity to convey existing peak flows. These pipes are also part of the original 1952 collection system. The recommended improvements include replacing the existing pipes with approximately 1,200 feet of new 12-inch diameter pipe (Figure 6-5).

- *Newton Creek Trunk Sewer – Newton Creek Pump Station to Manhole #476 (Project G-9)*

The existing 21-inch diameter pipe has adequate capacity to convey existing peak flows, but lacks the capacity to convey projected peak flows at buildout of the upstream collection system. This trunk sewer must be upsized to 24-inch diameter in order to convey the projected peak flows at buildout of the collection system. The total length of the project is approximately 2,650 feet (Figure 6-5). This trunk sewer was constructed in the mid 1980s using PVC piping materials. Therefore, this pipeline has a significant amount of useful life remaining. Also it is difficult to accurately estimate future development patterns. There are several large wetland areas in the upstream basins that flow to this trunk sewer. These areas are not likely to develop to the projected densities. As such, future peak flows to this trunk sewer may be lower than the projections prepared herein. For these reasons, it is unlikely that this project will be needed during the planning period. As such, it will be assigned a low priority in Chapter 8.

6.5.2 Recommended Improvements to Serve Undeveloped Areas

Several large areas of undeveloped land exist inside the UGB. Some of these parcels will be served by relatively short extensions of the existing system from the lower portions of the sewer basin. These extensions are relatively routine and are not discussed in this section since the needed line extensions are relatively obvious. This section does identify several sewer extension projects that are needed to serve the larger parcels of undeveloped land within the UGB. These are shown in Figure 6-2 through Figure 6-5 at the end of this section. It should be noted that the alignments shown in these figures are somewhat conceptual and the final alignments, overall project lengths, and costs will depend upon the locations of future right of ways and similar development issues. It is envisioned that these improvements will largely be built by developers as these larger portions of undeveloped land are annexed and developed.

- *19th Street Trunk Sewer South (Project G-10)*

This project is a major trunk sewer extension that is needed to extend sewer service to basins N6, N7, and N8 on the northern edge of the UGB. This line is an extension of the existing 24-inch trunk sewer that crosses Highway 20 near Newton Creek. This project includes connecting to the end of the existing 24-inch line north of the Highway and extending a new 24 inch line west to 19th Street. From 19th Street, a new 24-inch line will extend north and cross the railroad track. The total length of the project is approximately 2,700 feet (Figure 6-5).

- *Railroad Trunk Sewer (Project G-11)*

This project is a major trunk sewer extension that is needed to extend sewer service to basin N6A. This project includes connecting to the end of the 24-inch line installed as part of Project G-10 and extending a new 18 inch line northwest along the northern end of the railroad right of way. The total length of the project is approximately 3,200 feet (Figure 6-3).

- *19th Street/Green Road Trunk Sewer (Project G-12)*

This project is a major trunk sewer extension that is needed to extend sewer service to basin N7 and N6B. This project includes connecting to the end of the 24-inch line installed as part of Project G-10 and extending a new 21-inch line north along 19th Street to the intersection of Industrial Way. From Industrial Way, a 15-inch line will extend to the Green Road/West Hills Road intersection. From this intersection, a 12-inch line will extend west along West Hills Road. The total length of the project is approximately 5,000 feet (Figure 6-3).

- *Industrial Way Trunk Sewer (Project G-13)*

This project is a major trunk sewer extension that is needed to extend sewer service to basins N8A and N8B. A portion of Basin N8A is currently served by a line that flows to the south. The existing sewer infrastructure to the south is not design to handle the flow from the northern portion of the UGB. Therefore, a new trunk sewer must be extended to serve these areas. The new trunk sewer will direct all flow from basins N8A and N8B to the Newton Creek Pump Station. This project includes connecting to the end of the 21-inch line installed as part of Project G-11 at the Green Road/Industrial Way Intersection and extending a new 12-inch line west along Industrial Way to the outlet of basin N8B. The new line will need to be constructed at a depth sufficient to intercept and redirect flow from the existing sanitary sewer line serving basin N8A. The total length of the project is approximately 2,800 feet (Figure 6-3).

- *Sewer Basin N5 Trunk Sewer (Project G-14)*

This project is a major trunk sewer extension that is needed to extend sewer service to sewer basin N5. This project includes connecting to the end of the existing 15-inch line that terminates at Manhole #440. From this location the new line will extend east along Highway 20/34 to serve basin N5. The total length of the project is approximately 2,400 feet (Figure 6-5).

- *Chapel Drive Trunk Sewer (Project G-15)*

This project is needed to extend sewer service to sewer basins N1A and N1B in the southeast corner of the UGB. This project includes extending a new 10-inch gravity sewer from the Newton Creek Pump Station east along Chapel Drive. This alignment crosses two high points and two low points. The design for the lower sections of this sewer line must be sufficiently deep to ensure that gravity service can be provided to the eastern edge of the UGB. This will require a relatively deep installation across the two high points. The total length of the project is approximately 4,200 feet (Figure 6-5).

- *Sewer Basin P1 Pump Station and Forcemain (Project P-1)*

This project is needed to extend service to Sewer Basin P1 in the southwest portion of the UGB. This area is a natural low area that cannot be served by gravity sewers extensions from the City's existing system. The recommended improvements include the construction of a pump station near the 13th Street/Chapel Drive Intersection. The pump station will discharge into a 6-inch forcemain that will convey water to the 15th Street/Chapel Drive Intersection where it will connect to the existing common forcemain pipe. New gravity trunk sewers will generally extend northwest to the boundary of Basin P1. The pump station must be designed at a sufficient depth to allow extension of gravity sewers at appropriate pipe slopes to the edge of the sewer basin. This project includes a new wastewater pump station and approximately 1,300 feet of 6-inch forcemain (Figure 6-4).

- *Sewer Basin P2 Pump Station and Forcemain (Project P-2)*

This project is needed to extend service to Sewer Basin P2 in the western portion of the UGB. This area is a natural low area that cannot be served by gravity sewers extensions from the City's existing system. The recommended improvements include the construction of a pump station near the low point in the basin. The pump station will discharge into a 6-inch forcemain that will convey water to the existing gravity sewer system in Applegate Street. From this location, pump station discharge will flow by gravity to Pump Station A. New gravity trunk sewers will extend from the pump station to the boundaries of sewer basin P2. The pump station must be designed at a sufficient depth to allow extension of gravity sewers at appropriate pipe slopes to the edge of the sewer basin. This project includes a new wastewater pump station and approximately 1,000 feet of 6-inch forcemain (Figure 6-4).

6.6 EXISTING PUMP STATIONS AND FORCEMAIN IMPROVEMENTS

This subsection includes a description of the recommended improvements to the City's existing pump stations and common forcemain. Where appropriate the various improvement alternatives that were considered are discussed along with the reasons for the selection of the preferred alternative.

6.6.1 Timber Estates Sewer Extension (Project G-16)

As described above (6.3.2), the Timber Estates Pump Station lacks the capacity to overcome pressures in the common forcemain during peak pumping events. There are two approaches for correcting this problem. One is to install larger pumping equipment. The other is to eliminate the station by constructing a new gravity sewer from the Timber Estates Pump Station to the Newton Creek Trunk Sewer. Both alternatives were developed and evaluated as part of this planning effort.

In addition to the lack of pumping capacity, the station has several other shortcomings that should be addressed during the planning period. The electrical control system is antiquated, will reach the end of its useful life during the planning period, and is in need of upgrades. The existing pump discharge valves are located in an open-bottom manhole with poor access and drainage. As such, the manhole and valves should be replaced with modern equipment. The station also lacks emergency backup power in the event of a power failure. If the station is to remain in service, the City will need to rehabilitate the station by replacing the pumping equipment, replacing the discharge valves and manhole, upgrading the control system and installing a backup power generator.

As an alternative to rehabilitating the station, it could be removed from service by installing a new gravity sewer line from the pump station wet well to the Newton Creek Trunk Sewer. This would require the installation of approximately 2,000 feet of 8 inch diameter sewer line (Figure 6-5).

The construction and long-term operation and maintenance costs for these two alternatives were compared. Over the life cycle of the facilities, the second option of installing a gravity sewer line was determined to be the more cost-effective solution for the City. As such, the recommended improvements include installing a new 8-inch gravity sewer line from the pump station wet well to the Newton Creek Trunk Sewer and removing the station from service (i.e., Project G-16).

6.6.2 Newton Creek Pump Station Forcemain (Project F-1)

Based on the capacity analysis presented above (Section 6.3.2), the Newton Creek Pump Station lacks the capacity to convey existing peak flows and Pump Station A lacks the capacity to convey peak flows at the end of the planning period. The most cost effective approach to increase the capacity of both stations is to construct a new forcemain pipe from the Newton Creek Pump Station to the wastewater treatment plant. This pipeline will operate in parallel with the existing 18-inch pipeline. The piping should include valving that allows both Newton Creek Pump Station and Pump Station A to discharge through one or both pipelines and for each station to discharge through a dedicated pipeline. The new parallel pipeline will result in lower forcemain pressures which will allow the various pumps to discharge at higher rates. With a new 18-inch pipeline from the Newton Creek Pump Station to the wastewater treatment plant, the firm capacities of Pump Station A and Newton Creek Pump station will increase as shown in Table 6-4. The total length of the new forcemain pipe is approximately 4,100 feet. The alignment includes a new crossing of Newton Creek and the Marys River. It is envisioned that these crossings will be installed by horizontal directional drilling or by the installation of an auger bored casing.

Table 6-4 | Summary Pump Station Firm Capacities After Completion of Project F-1

Pump Station	Estimated Firm Capacity After Completion of Project F-1(1) (MGD)	Existing Estimated Peak Flows (MGD)	Estimated Peak Flows at the End of the Planning Period (MGD)	Required Buildout Capacity (MGD)
Pump Station A	5.1	4.60	5.1	5.9
Newton Creek Pump Station	3.1	3.45	4.0	12.2

(1) Firm capacities based on all but the largest single unit running at each station.

6.6.3 Newton Creek Pump Station Improvements (Project P-3)

Upon the completion of the new Newton Creek Forcemain (Project F-1), modelling shows that the Newton Creek Pump Station will still lack the capacity to convey existing and projected peak flows (Table 6-4). As such, the City should plan to upgrade the mechanical components of the station during the planning period. This will be a major pump station improvement project. It is envisioned that the existing wet well will remain in service. A bypass pumping operation will be set up during construction and new pumps and discharge piping will be installed in the wet well. New wet well hatches will also be installed to facilitate the larger pumps. The existing check and isolation valves will be relocated from inside the wet well to newly constructed valve vaults adjacent to the wet well. The station should include a flow meter located between the valve vaults and the forcemain connection points. This will allow the City to track flows from the Newton Creek Pump Station and Pump Station A on a daily basis. A new diesel powered generator will be installed at the site that will provide backup power for the station. It is envisioned that a new control panel and variable frequency drives will be installed in the existing pump station building. As such, the existing building will be salvaged.

The new pumping equipment should be sized based on a minimum of 20-year flow and growth projections. Since the timing of this improvement is not known at the present time, the sizing of the station will be deferred to the preliminary design phase of the project. That said, the new station will likely have a firm capacity in the 5 to 6 MGD range. Another approach to sizing the station is to install pumping equipment that maximizes the capacity of the 18-inch Newton Creek Forcemain (i.e., project F-1). This would result in a station with a firm capacity of approximately 7 MGD (i.e., 18-inch pipe velocity of 6.5 feet per second). If this approach is pursued, the first phase of the project could include the installation of three new pumps with the future installation of a fourth pump. The pumps would be sized such that the installation of the fourth pump increases the firm capacity of the station to approximately 7 MGD. With this approach the firm capacity with three installed pumps would be approximately 5.5 MGD. The total recommended budget for this project is $1,480,000. A detailed cost estimate is included in Appendix C.

6.7 SUMMARY OF RECOMMENDATIONS

The recommended improvements described above are summarized in Table 6-5 and are shown in the figures at the end of this chapter. These improvements will result in a sewage collection system with the capacity needed to convey flows from within the planning area assuming development to current zoning densities.

The recommended improvements are based on the complete development of the land within the UGB. Therefore, some of the improvements may not be required during the planning period. The improvements address existing deficiencies, as well as potential deficiencies at the end of the planning period and at buildout. Only the improvements that address the existing deficiencies are required at this time. The remaining deficiencies are growth dependent. Of these, some may be required before the end of the planning period and some may not. The improvements are prioritized in Chapter 8.

The alignment of future lines through the undeveloped portions of town has not yet been definitively determined. The final alignment of sewer lines in these areas should be determined as property develops. Sewer lines should be placed within right-of-ways whenever possible. If the City Limits or UGB are to be expanded in the future, the sewer system should be re-examined to determine where additions are needed and if alternate alignments are justified.

Table 6-5 | Recommended Collection System Improvements

Project Code	Project Description	Recommended Diameter/Capacity	Length
Gravity Collection System Improvements			
G-1 [1]	9th Street to 7th Street Sewer Lines – Manhole #35 to Manhole #184	10	1,450
G-2 [1]	10th Street Sewer Lines – Manhole #34 to Manhole #45	12	400
G-3 [1]	Main Street Sewer Lines – Manhole #45 to Manhole #52	10 & 12	800
G-4	8th & College Street Sewer Lines – Manhole #52 to Manhole #56	10	750
G-5	Pioneer and 11th Street Sewer Lines – Manhole #71 to Manhole #74	10	800
G-6	15th Street Trunk Sewer (South) – Manhole #27 to Manhole #288	12	1,650
G-7	15th Street Trunk Sewer (North) – Manhole #288 to Manhole #94	12	350
G-8	Applegate Street and 20th Street Trunk Sewer - Manhole #1 to Manhole #6	12	1,200
G-9	Newton Creek Trunk Sewer – Newton Creek Pump Station to Manhole 476	24	2,650
G-10	19th Street Trunk Sewer South	24	2,700
G-11	Railroad Trunk Sewer	18	3,200
G-12	19th Street/Green Road Trunk Sewer	21,15,12	5,000
G-13	Industrial Way Trunk Sewer	12	2,800
G-14	Sewer Basin N5 Trunk Sewer	15	2,400
G-15	Chapel Drive Trunk Sewer	10	4,200
G-16	Timber Estates Trunk Sewer	8	2,000
Pump Station and Forcemain Improvements			
P-1	Basin P1 Pump Station and Forcemain	0.62 MGD 6-inch	1,300
P-2	Basin P2 Pump Station and Forcemain	0.42 MGD 6-inch	1,000
P-3	Newton Creek Pump Station Improvements	TBD	-
F-1	Newton Creek Forcemain	18-inch	4,100
General Collection System			
Pgm-1	Sewer Collection System Rehabilitation Program (Program – 1)	Rehabilitate remaining 1952 collection system	

[1] Project scheduled for completion in 2017

Figure 6-2 | Recommended Collection System Improvements (NW Quadrant)

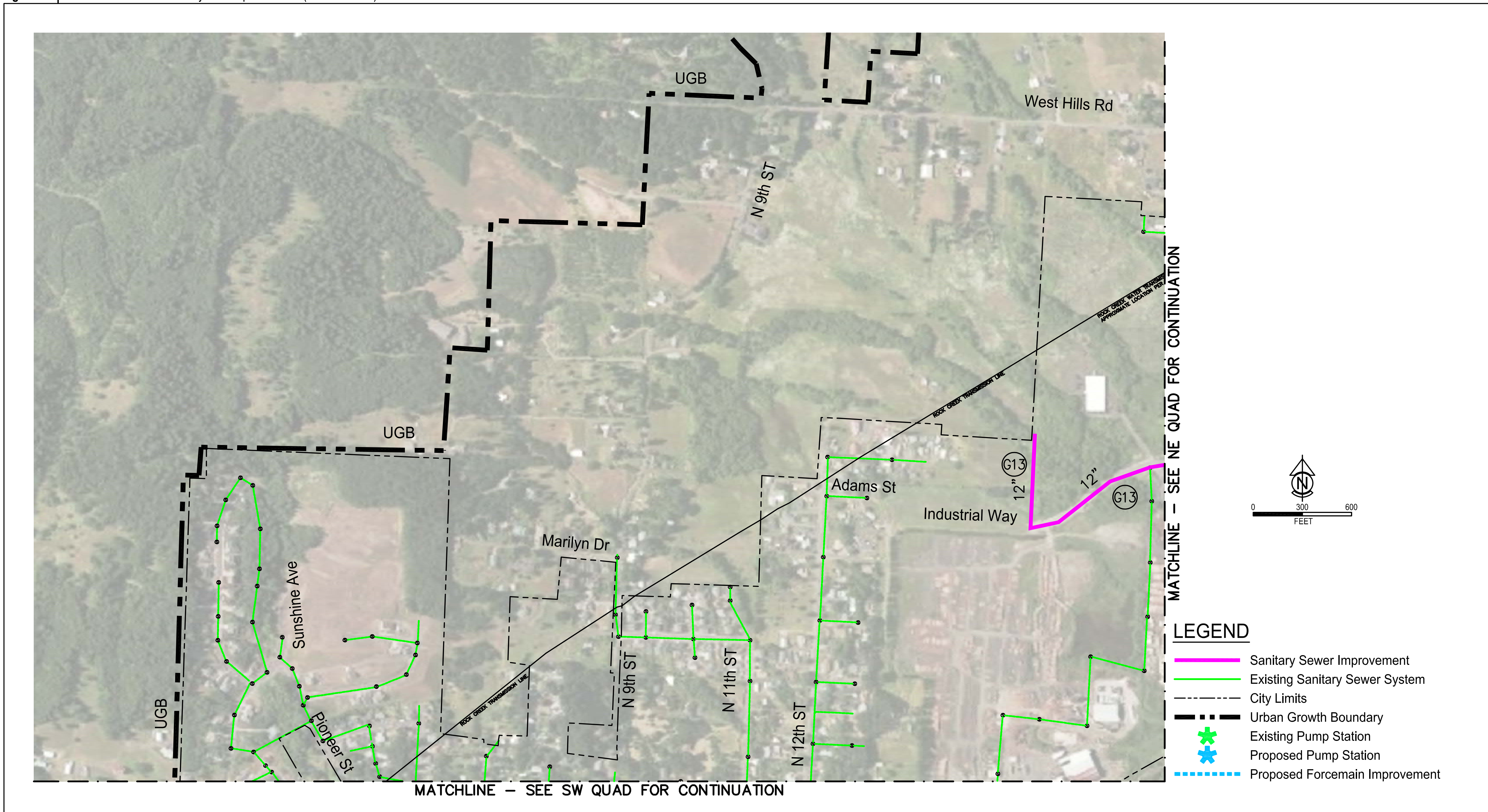


FIGURE 6-2

Figure 6-3 | Recommended Collection System Improvements (NE Quadrant)

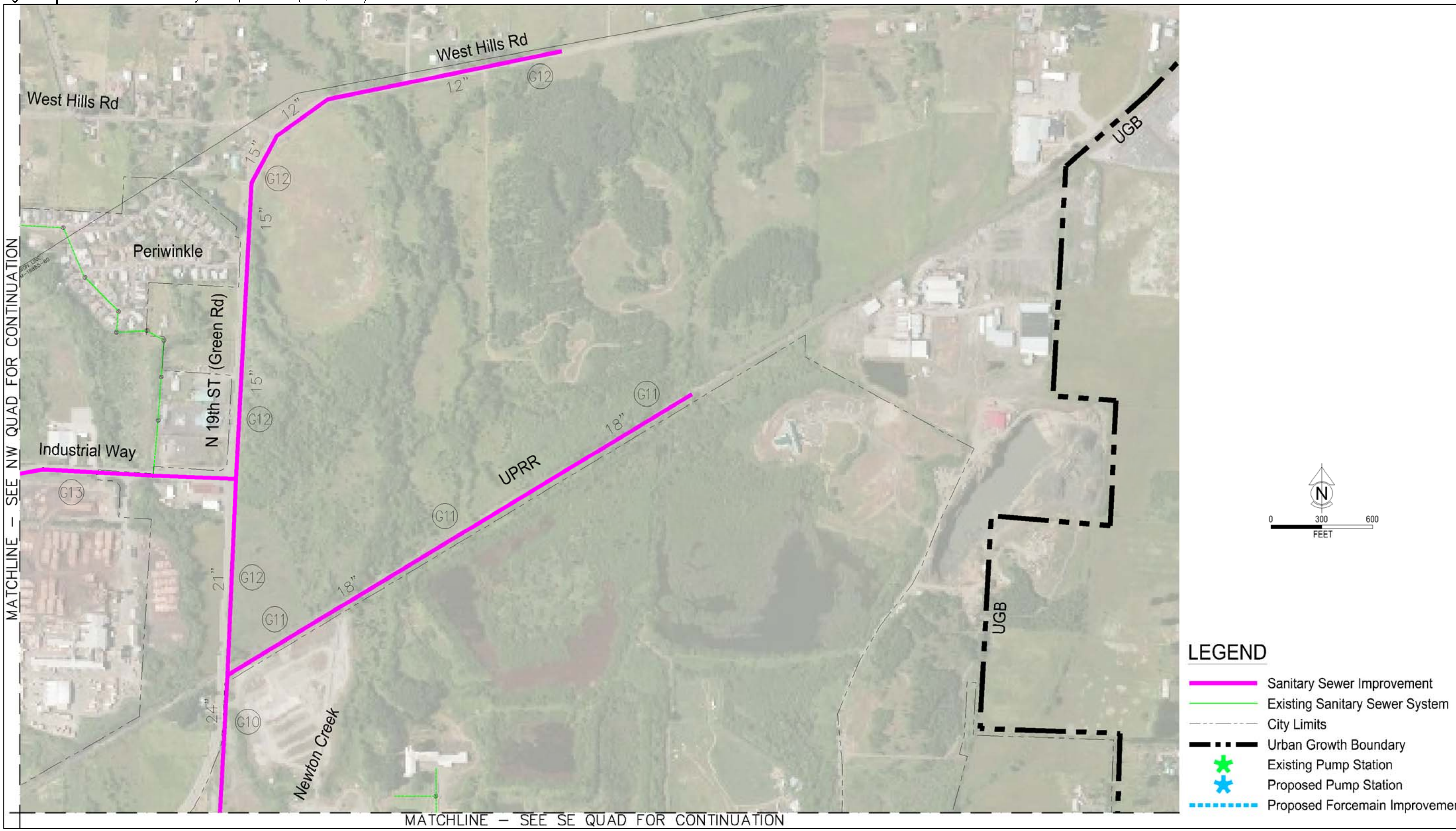


FIGURE 6-3

Figure 6-4 | Recommended Collection System Improvements (SW Quadrant)

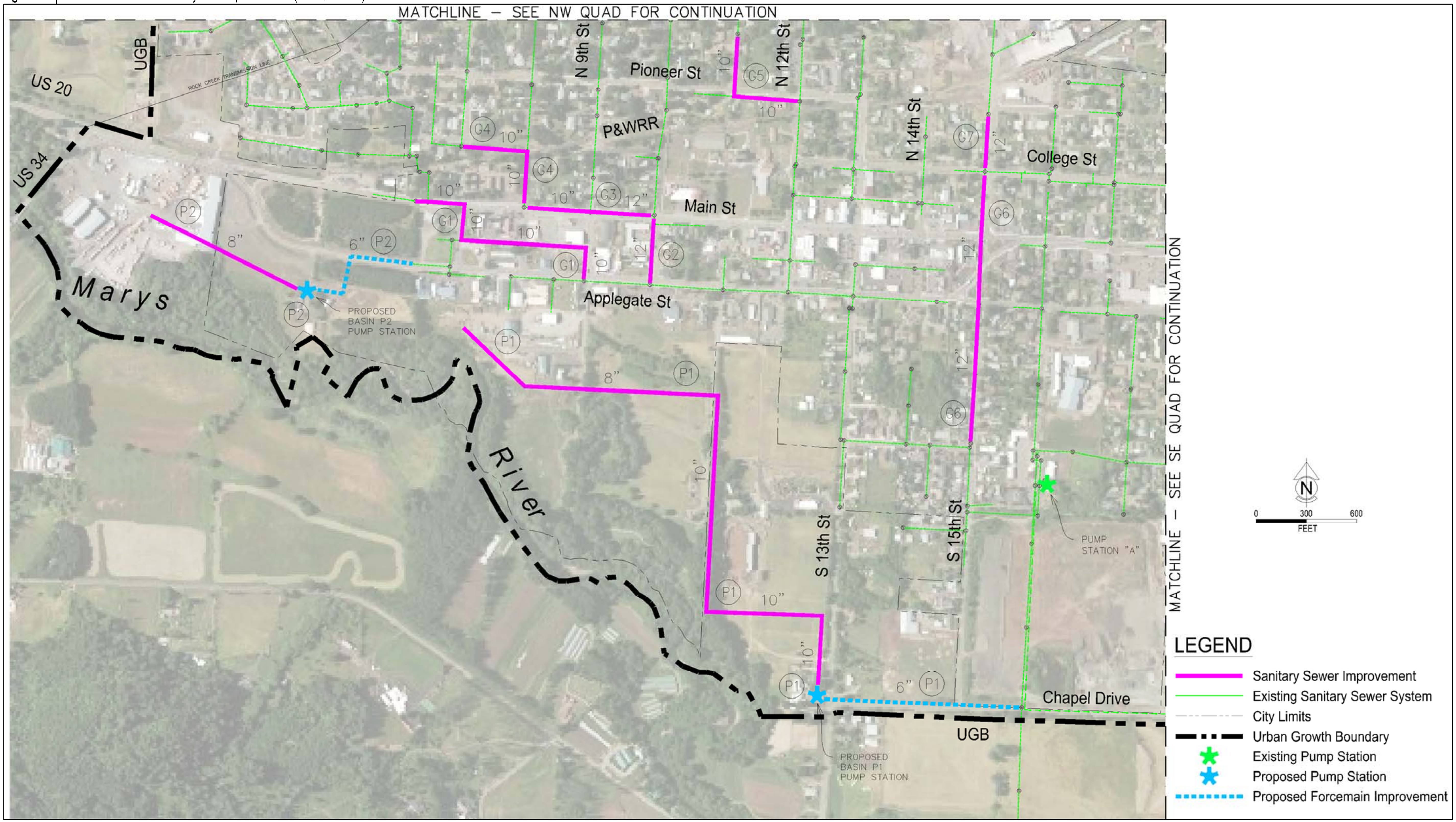


FIGURE 6-4

Figure 6-5 | Recommended Collection System Improvements (SE Quadrant)

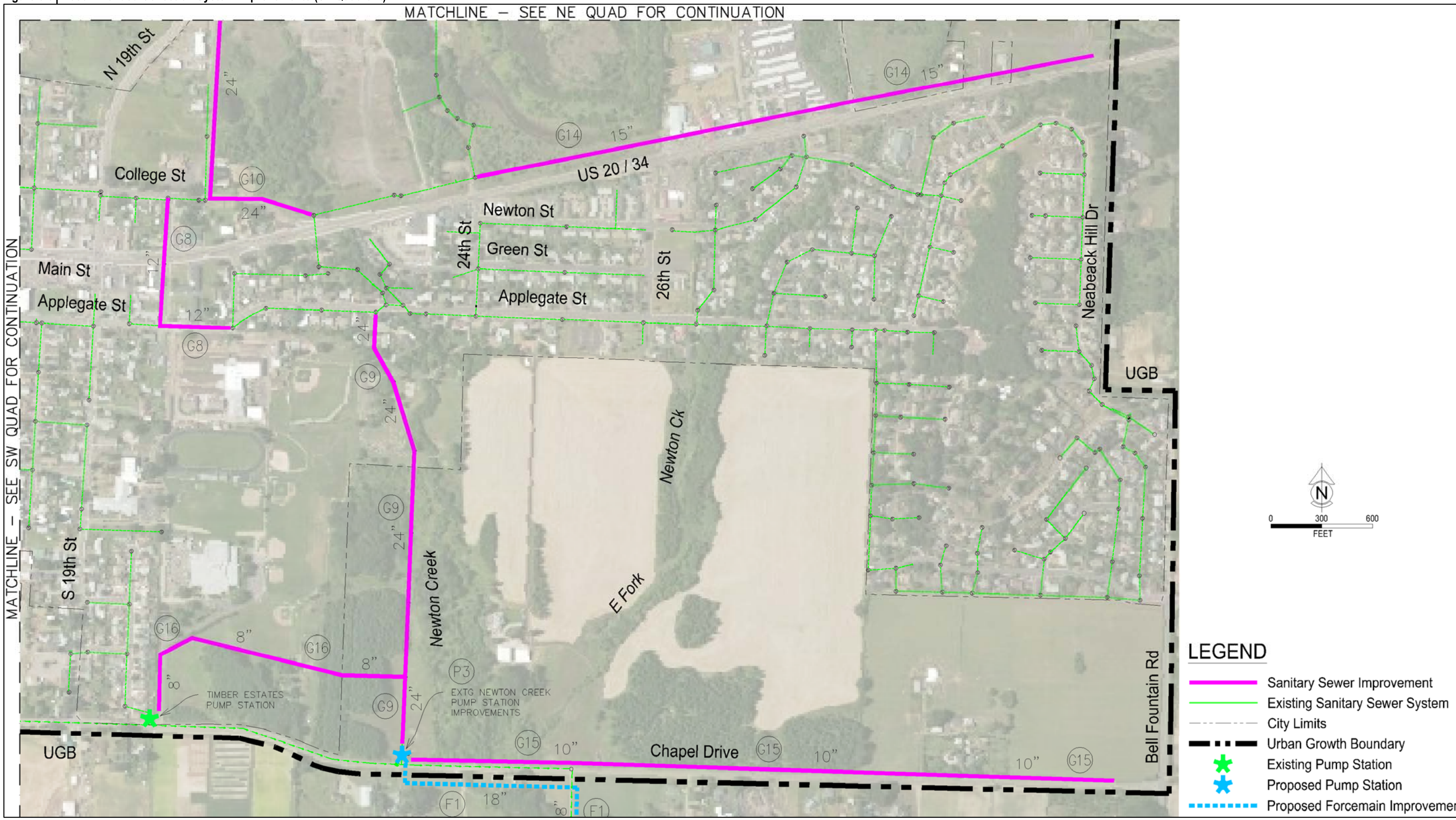


FIGURE 6-5

CITY OF PHILOMATH
Wastewater System Facilities Plan
Philomath, Oregon

CHAPTER 7

TREATMENT SYSTEM EVALUATION

Chapter Outline

TREATMENT SYSTEM EVALUATION

CHAPTER 7

7.1 INTRODUCTION

Chapter 4 includes a listing of some minor existing treatment system deficiencies (Section 4.5.7). This chapter builds on the information from Chapter 4 by evaluating the existing treatment system with respect to future flows and loads. The deficiencies identified in Chapter 4 are first summarized. This is followed by a detailed analysis of the existing treatment and disposal system with respect to future flows and loads. The purpose of this analysis is to identify treatment system components that are likely to become deficient during the planning period as a result of increased flows and loads due to growth. A comprehensive list of existing and projected shortcomings is then presented.

The second portion of this chapter includes a listing of the recommended improvements to address each deficiency. In some cases, the recommended improvement is relatively straightforward and a detailed alternatives analysis is not included. In cases where the recommended improvement is not obvious, a more detailed alternatives analysis is presented. This chapter concludes with a listing of the recommended improvements for the treatment system.

7.2 EXISTING TREATMENT SYSTEM DEFICIENCIES

For completeness, the treatment system shortcomings identified in Chapter 4 are listed in this subsection. These relatively minor shortcomings include the following items.

- The Marys River outfall lacks an effluent diffuser.
- Rutting and potholes in the plant entrance roadway can be a problem.

7.3 TREATMENT SYSTEM EVALUATION

This section includes a quantitative evaluation of the treatment plant with respect to the projected wastewater flows and loadings. The purpose of this analysis is to identify treatment system components that are likely to become deficient during the planning period as a result of increased flows and loads due to population growth.

7.3.1 Headworks

The existing influent headworks is relatively new and in good condition. From a hydraulic perspective, the headworks can accommodate a minimum of 12 MGD. At the end of the planning period, the peak flow rate to the headworks from the City's pumping stations is expected to be less than 10 MGD (see section 5.3.4). This is the flowrate when the improvements to the Newton Creek Pump Station and the new forcemain pipe from the Newton Creek Pump Station to the treatment plant are completed (see Chapter 6). Since the headworks is adequately sized for the foreseeable future and the facility is relatively new, no improvements are needed to increase the hydraulic capacity of the structure. Improvements may be needed to provide screening of the raw wastewater. This is discussed below.

7.3.2 Hydraulic Storage Capacity

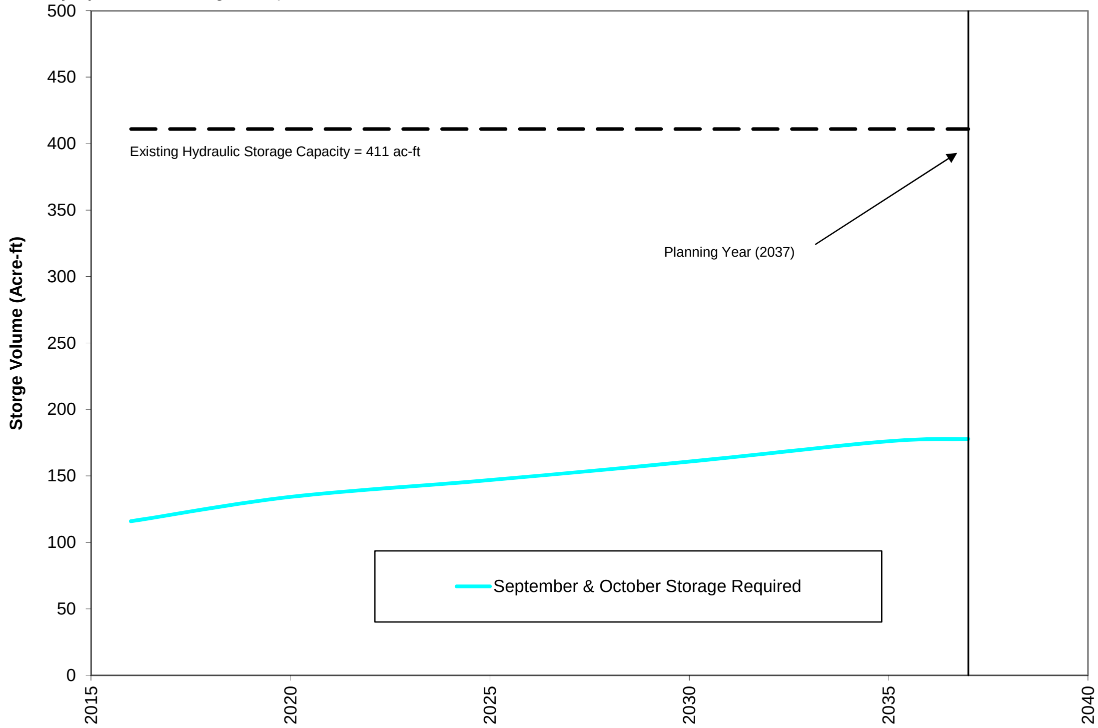
Throughout the year, there are two periods of time when the City is unable to discharge treated effluent from the lagoons and all wastewater that flows into the plant must be stored in the lagoons. The City's current discharge permit does not allow discharge to the receiving stream between April 30 and October 31. In May and early June, the irrigation sites are often too wet to be irrigated and all wastewater must be stored in the Lagoons. As fall rains start in October, the irrigation sites also become too wet to receive irrigation water and the City is not permitted to discharge to the receiving stream until November 1. Therefore, during the month of October, all wastewater must also be stored in the lagoons.

The existing storage capacity provided by the lagoons is approximately 411 acre-feet (Table 4-5). To evaluate the adequacy of this volume, a water balance can be performed for the spring and fall storage periods. The water balance includes summing all the water inputs and outputs from the lagoons to estimate the total storage requirements. Water balances were performed at 5-year intervals during the planning period using the projected flows listed in Chapter 5. The resulting storage requirements are plotted with the storage capacity of the treatment system in Figure 7-1. The calculations show that the storage requirements for the September and October storage period are greater than the Spring storage season. Therefore, September & October storage requirements control the sizing of the lagoons with respect to hydraulic storage. For the sake of clarity, the spring storage requirement line is not shown in Figure 7-1 since it is less than the storage requirement for October. The water balance calculations are based on the following assumptions.

- In an effort to be conservative with respect to the storage needs, the water balance calculations will be based on the assumption that no irrigation will occur during the months of May and June in the Spring and the months of September and October in the Fall. This is a fairly conservative assumption since irrigation during these months is fairly common.
- Based on a review of recent data, the average influent flow to the lagoons during the May and June storage period and during the September and October storage period can be approximately 25% higher than the average dry weather flow for the entire dry weather period. Therefore a peaking factor of 1.25 will be applied to future estimates of average dry weather flow to estimate the average flow during the Spring and Fall storage periods.
- Zero wastewater outflow during the Spring and Fall storage periods.
- The monthly average pan evaporation is 4.59 inches, 5.88 inches, 5.09 inches, and 2.37 inches for May, June, September, and October respectively [7]. Pan evaporation is multiplied by a pan coefficient of 0.745 to estimate the free surface evaporation from the lagoons.
- The monthly average rainfall is 1.97 inches, 1.23 inches, 1.45 inches, and 3.17 inches for May, June, September and October respectively[7].

[7] Corvallis State University Weather Station. Period of Record, 1890 through 2005

Figure 7-1 | Hydraulic Storage Requirements



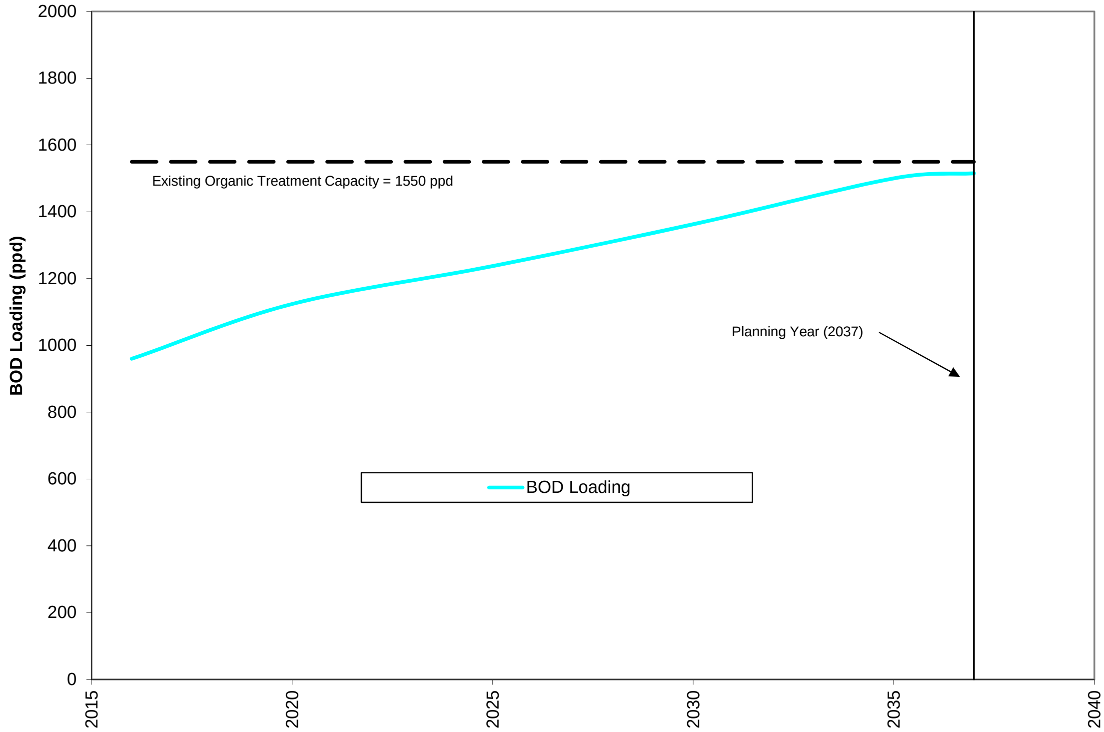
The water balance calculations show the existing facilities provide more than enough storage to accommodate flows from the City for the entire planning period. Therefore, additional lagoon storage should not be needed during the planning period. The storage capacity of the existing plant (411 ac-feet) is based on complete drawdown of the lagoons to minimum water depths of two feet. The above analysis demonstrates that the existing lagoons do not need to all be drawn down to minimum levels to accommodate the spring and fall storage seasons.

7.3.3 Organic Treatment Capacity

The facultative lagoons provide primary and secondary treatment of the waste stream. The organic treatment capacity of the lagoons is finite. If this capacity is exceeded, compliance problems will result. The lagoon cells are designed to operate in series with wastewater flowing sequentially from Cell 1 to Cell 2 to Cell 3. In Western Oregon, a typical design approach is to size lagoons for an overall organic loading rate of 35 pounds of BOD per acre per day, with a maximum of 50 pounds of BOD per acre per day to the first cell on an average annual basis. When operated in series mode, the organic treatment capacity of the plant is controlled by size of Cell 1. Cell 1 is 31 acres in size. At 50 pounds per acre per day, the organic treatment capacity of the plant is approximately 1,550 pounds of BOD per day. The projected BOD loads to the plant are plotted in Figure 7-2 together with the treatment capacity of the plant with the lagoons operating in series.

As shown (Figure 7-2), the existing plant has sufficient organic treatment capacity to serve the City for the remainder of the planning period. Therefore, no improvements are required to increase the organic treatment capacity of the plant.

Figure 7-2 | Organic Treatment Evaluation



7.3.4 Discharge Facilities Capacity Evaluation

Once water enters the first lagoon cell, the flowrate through the plant is controlled by the discharge rate selected by the operator. During the winter months, the discharge rate is adjusted by opening and closing one of the sluice gates on the lagoon outlet structure. Water flows through the sluice gate to the chlorine contact chambers and through the contact chambers to the outfall pipeline and ultimately to the Marys River. Winter discharge occurs entirely by gravity. During the summer season two vertical turbine pumps are used to convey water to the irrigation sites. The speed of the irrigation pumps is controlled to maintain a constant pressure in the irrigation distribution system. As more water is used for irrigation, the pressure in the irrigation piping drops and the speed of the irrigation pumps is increased to increase the pressure to the desired set point. The opposite sequence occurs as the amount of irrigation water being used decreases.

An analysis was completed of the various hydraulic facilities used to convey water from Cell 1 to the Marys River as required during the winter discharge season. This analysis showed that all of the various transfer pipes and hydraulic structures used to convey water from the first lagoon cell to the Mary's River Outfall are adequately sized to convey at least 3 MGD. For the purposes of this study, the firm capacity of the winter discharge facilities will be taken as 3 MGD. During the summer discharge season (May – October) the irrigation pumps are used to discharge effluent at a maximum firm capacity of 1200 GPM or 1.7 MGD.

To determine if these capacities are adequate, water balances were performed on a seasonal basis. The water balances include summing all the inputs and outputs from the lagoons to determine the minimum discharge rate that is needed to convey the treated water through the plant and dispose of water that accumulated during the previous non-discharging period. Water balances were performed for various years during the planning period to estimate the required minimum discharge rate for each year.

As the City grows, flows to the plant will steadily increase and the amount of water that must be discharged will also increase. For winter discharge operation, the minimum required discharge rates are plotted with the discharge capacity of the treatment plant in Figure 7-3. For summer irrigation operation, the minimum required irrigation rates are plotted along with the summer irrigation capacity of the treatment plant in Figure 7-4. The water balance calculations are based on the following assumptions.

- While the total winter discharge season is 165 days. The actual November 1 through April 30 discharge window is 181 days long. The shorter timeframe is used to be conservative and to account for equipment malfunctions and other similar events that may impact discharge operations.
- Summer discharge (i.e., irrigation) occurs over 56 days. This is equivalent to 8 weeks of irrigation. Based on grass seed crops currently grown at the irrigation sites. A realistic irrigation scheme is approximately 2 weeks of irrigation prior to harvest and approximately 6 weeks after harvest.
- The average November – April rainfall depth is 32.22 inches[8].
- The average May – October rainfall depth is 8.73 inches[8].
- There is no evaporation during the winter discharge season.
- The average May – October pan evaporation is 32.76 inches.
- Pan evaporation is multiplied by a pan coefficient of 0.745 to estimate the free surface evaporation from the lagoons, which equals 24.4 inches.
- Zero lagoon seepage. This is conservative since some seepage from the lagoons will occur.
- 200 acre-feet of water stored in the lagoons must be discharged during the winter and summer discharge seasons.

[8] Western Regional Climate Center data for Corvallis State University

Figure 7-3 | Required Plant Winter Discharge Rate

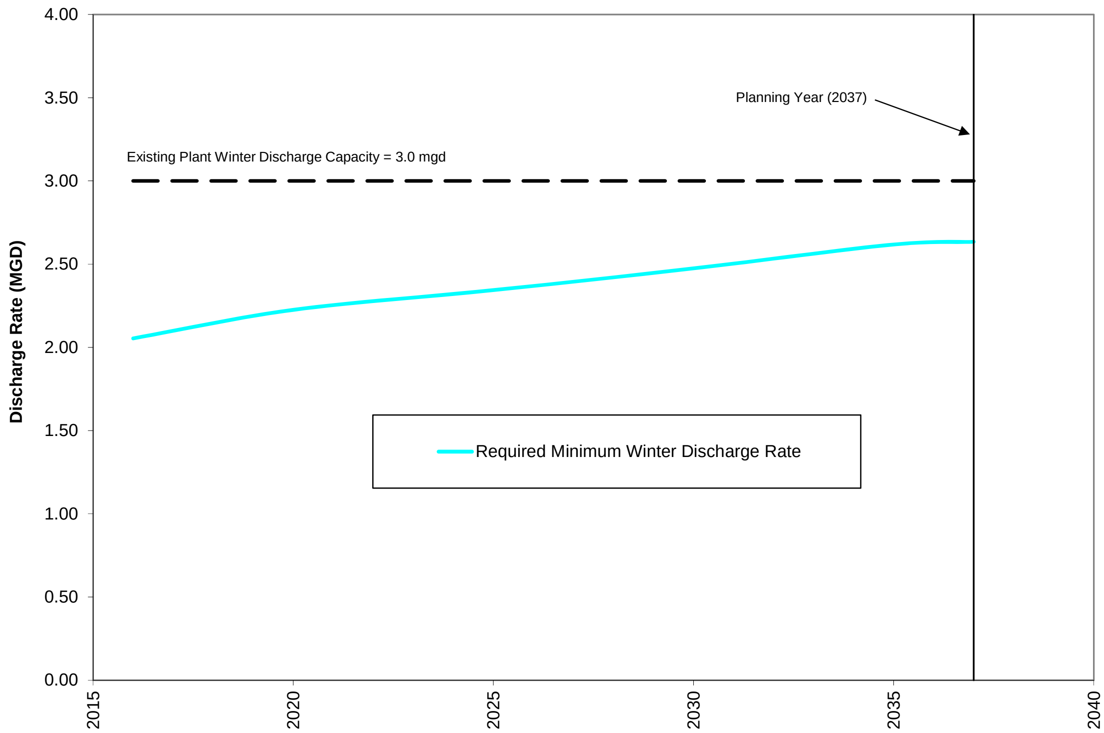
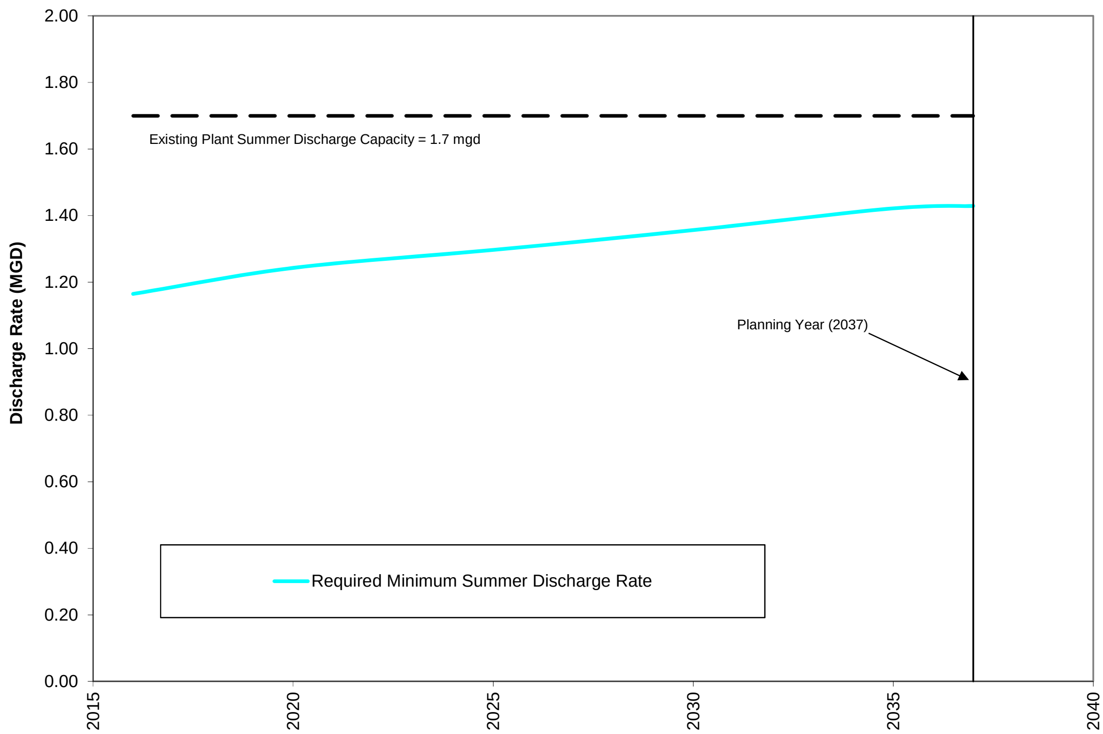


Figure 7-4 | Required Plant Summer Irrigation Rate



As shown in Figure 7-3 and Figure 7-4, the discharge rates required to dispose of the increased wastewater flows that are anticipated to occur during the planning period are less than the capacities of the existing winter and summer discharge facilities. As such, improvements to increase the plant discharge rate are not likely to be needed during the planning period.

7.3.5 Disinfection System Capacity

Chlorine is added to disinfect the effluent prior to disposal. Disinfection by Chlorine requires contact time with the effluent. Chlorine contact time is provided in three locations. These include the effluent pipe downstream of cell 3 and in the two chlorine contact chambers. During the winter discharge season, the buried pipe and both contact chambers are used. The water level in the contact chambers is controlled by the effluent weir on the downstream end of the second contact chamber. At a maximum discharge rate of 3.0 MGD, the chambers provide approximately 50 minutes of contact time in the winter configuration. During the summer irrigation season, only the buried pipe and the first contact chamber are used. The water level in the contact chamber is equal to water level in cell 3. In this configuration, the contact volume varies between 105,000 gallons (cell 3 depth of four feet) and 135,000 gallons (cell 3 depth of 8 feet). Therefore, at an irrigation pumping rate of 1200 gpm, the facilities provide a minimum of about 85 minutes of contact time. The contact time provided by the facilities during both the winter and summer discharge seasons should be sufficient for the remainder of the planning period and no major improvements that increase chlorine contact times are required.

The existing chlorine feed system is capable of feeding up to 100 pounds per day. This is sufficient to disinfect the anticipated effluent flows for the remainder of the planning period. Similarly, the sulfur dioxide feed system is also adequately sized for the remainder of the planning period with a maximum feed rate of 30 pounds per day. Therefore, with normal maintenance and the occasional equipment replacement the existing chemical feed systems should be sufficient for the remainder of the planning period.

7.3.6 Receiving Stream Capacity

Treated effluent is discharged to the Marys River during the wet weather discharge season (November – April). Discharge to the receiving stream is regulated by the City’s existing NPDES permit (Section 3.3). The NPDES permit requires effluent BOD and TSS concentrations below 30 mg/L and 50 mg/L respectively. Total BOD and TSS effluent mass loads are also limited to 460 and 760 pounds per day on an average monthly basis respectively. At effluent BOD and TSS concentrations of 30 mg/L and 50 mg/L respectively, the discharge rate cannot exceed 1.83 mgd (460 ppd ÷ 30 mg/L ÷ 8.34 = 1.83 mgd).

As growth in the community continues, the amount of water that will need to be discharged will increase. Water balance calculations (Figure 7-3) show that the City will need to discharge at average rate of approximately 2.6 mgd at the end of the planning period. In order to discharge at 2.6 mgd in compliance with the permitted mass loads, effluent BOD and TSS concentrations must be below 21 mg/L and 35 mg/L respectively. During the previous 10 years, effluent BOD and TSS data have not exceeded these values. This includes several years prior to the 2011 lagoon expansion project. This suggests that the plant should be capable of producing effluent BOD and

TSS concentrations below 21 mg/L and 35 mg/L respectively for most of the discharge season. However, as flows increase, the detention time in the lagoons will decrease and the treatment efficiency may also decrease. As such, it is recommended that the capital improvement plan include a project to improve the treatment efficiency of the lagoons in order to ensure compliance with the effluent mass loads in the NPDES permit. The need to implement this particular project will depend on the performance of the treatment plant. The project will only be needed if the treatment plant is consistently unable to produce effluent BOD and TSS concentrations below 21 mg/L and 35 mg/L respectively. If and when this occurs, the City can implement the required improvements. It is likely that the need for this project will not occur during the current planning period. However, a prudent approach is to plan to implement the project toward the end of the planning period. Alternatives for improving the treatment efficiency of the plant are evaluated below.

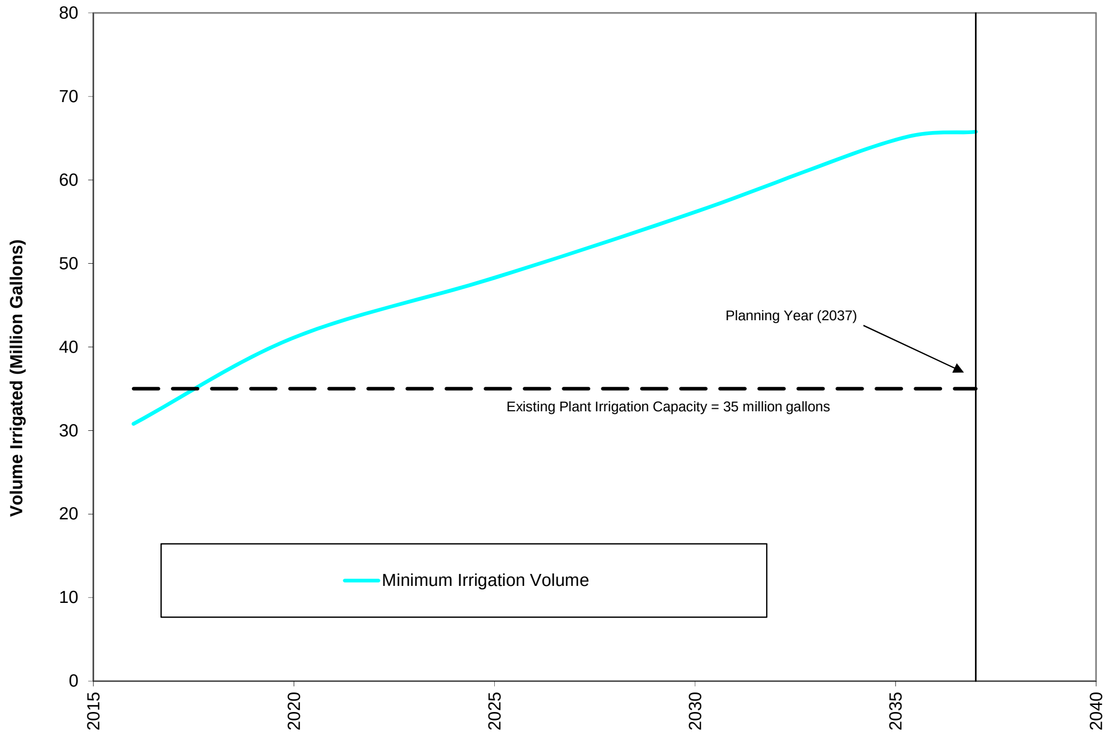
7.3.7 Capacity of Land Application Facilities

During the dry weather irrigation season (May – October), treated effluent is disposed by irrigating grass seed crops located west and north of the existing lagoons. These sites are owned by the City. The total area irrigated is approximately 115 acres. Effluent is distributed on the 100 acres west of the existing lagoons using a linear irrigation sprinkler. Big guns and hand lines are used to distribute effluent on the 15 acres north of the existing lagoons. During the irrigation season (May-October), grass seed crops require approximately 17 inches of net irrigation[9] on average. This is in addition to precipitation that naturally falls on the fields. In practice, grass seed growers do not generally irrigate the crops when pollination is occurring and during harvest. As such, the practical application rate is less than 17 inches. Philomath's current recycled water use plan lists the average gross irrigation rate as 11.2 inches. This value will be used for the remainder of the calculations in this section. Multiplying the gross irrigation rate (11.2 inches) by the total area available for irrigation (115 acres) and converting units results in a total irrigation capacity of 35 million gallons. In other words, the exiting land disposal system can accept approximately 35 million gallons per year on average during the May – October irrigation season.

In order to determine if 35 million gallons per year is sufficient to dispose of effluent during the summer irrigation season, water balance calculations were performed for the May through October irrigation season. The assumptions used for the water balance calculations are generally the same as used above (section 7.3.4). The increase in the minimum volume of water that must be irrigated over the planning period is shown in Figure 7-5. As shown in Figure 7-5, the minimum amount of water that must be irrigated will exceed the capacity of the City's irrigation sites prior to the end of the planning period. Therefore, the City's existing land disposal system lacks the capacity to serve the City for the remainder of the planning period. Alternatives to increase the City's dry weather disposal capabilities are evaluated below.

[9] Oregon Crop Water Use and Irrigation Requirements, Table 5, OSU Extension Service

Figure 7-5 | Minimum Irrigation Volume Requirements



7.4 SUMMARY OF TREATMENT SYSTEM DEFICIENCIES

The previous subsection (section 7.3) includes an analysis of the plant with respect to its ability to treat and dispose of the future flows and loadings anticipated during the planning period. This analysis revealed two shortcomings that will likely need to be addressed during the planning period. In addition to these projected shortcomings, a number of existing shortcomings were also identified in Chapter 4 (see section 7.2). For the sake of completeness, all of the existing and projected deficiencies are summarized in Table 7-1.

Table 7-1 | Summary of Treatment System Deficiencies

Deficiency Number	Description
D-1	The Marys River outfall lacks an effluent diffuser.
D-2	Near the end of the planning period, the existing treatment plant may lack the ability to produce effluent of the quality needed to discharge to the Marys River in accordance with the City's NPDES permit.
D-3	The volume of water that can be irrigated during the summer months at the existing irrigation sites is not sufficient for the entire planning period. Additional summer discharge capacity is needed.

7.5 RECOMMENDED IMPROVEMENTS

The following sections include the recommended improvements to address the deficiencies identified in Table 7-1.

- *Marys River Outfall Diffuser (Project T-1)*

As described in Chapter 4 (see subsection 4.5.1), the existing outfall pipeline to the Marys River lacks a multiport diffuser. As such, the effluent is not well mixed with the receiving stream. This could become a problem for the City if future regulations become more stringent. Therefore, the recommended capital improvement plan includes the construction of a new outfall diffuser.

The existing outfall pipe extends into the stream from a cast in place concrete structure located at the top of the stream bank. This structure was originally designed with stop logs that maintained full pipe flow in the upstream pipeline. The piping upstream of the structure was originally designed to provide chlorine contact time for disinfection. When the second chlorine contact chamber was constructed, the contact time provided in this pipe was no longer required and the stop logs were removed. It is envisioned that the new outfall diffuser pipe will be connected to this concrete structure on the upstream side of the stop logs and extended into the stream. New stop logs will be installed to force flows into the new diffuser pipe. During high water events in the Marys River, the effluent will overflow the stop logs into the existing outfall pipe which will serve as a high water overflow pipe. During the majority of time, effluent will flow exclusively through the diffuser pipe. Water will only bypass the diffuser pipe during short-duration, high-water events. This approach enables the diffuser to be designed with higher head losses and higher jet velocities than could be achieved if effluent had to be routed through the diffuser pipe during high water events. From the outfall structure, the new diffuser pipe will be extended down the stream bank and a new multiport diffuser manifold will be installed below the stream bed. The individual diffuser pipes will extend from the manifold vertically into the stream channel. The ends of the diffuser pipes will be fitted with duck-bill style check valves to improve discharge velocities.

The proposed work requires excavation below the ordinary high water levels. This triggers an extensive permitting process. Wetland fill permits will be required. Consultation with NMFS for impacts to threatened and endangered fish species will be required. Cultural resource investigations will also be required as part of the permitting process. The permitting for the work will be substantial and the recommended budget for the project includes funds to pay for this effort. Once the new diffuser is installed, a new mixing zone study should be prepared and submitted to DEQ for their records. The DEQ will then use information in the new mixing zone study to analyze the effects of the outfall as part of future NPDES permit renewals. The total recommended budget for this project is $173,000. A detailed cost estimate is included in Appendix C.

- *Lagoon Aeration and Headworks Screening (Project T-2)*

As described in subsection 7.3.6, the treatment plant must be capable of reducing effluent BOD and TSS levels below 21 mg/L and 35 mg/L respectively on a continuous basis for the duration of the planning period. This effluent quality is required to comply with the monthly average effluent mass load limits in the NPDES permit (i.e., 460 pounds per day of BOD and 760 pounds per day of TSS). Past plant performance suggests that the plant should be able to meet this requirement for the entire planning period. However, due to a number of factors, the possibility exists that the City may have difficulty reliably achieving this level of treatment as flows and organic loading increase due to population growth. As such, the recommended capital improvement plan includes a lagoon aeration project to improve the treatment efficiency of the lagoons. It is likely that this project will not be required during the planning period. As such, this project will be assigned a relatively low priority and may be delayed until the next planning period. It is envisioned that this project will only be implemented if effluent BOD and TSS values rise to the point where the City is experiencing difficulty complying with effluent mass load limits in the NPDES permit.

The goal of this project is to improve the organic treatment capacity of the plant. This will decrease the nutrients available for algae growth which is the primary cause of high BOD and TSS values. As an alternative to adding aeration to the lagoons, other options include floating lagoon covers to limit algae growth, or the construction of algae removal treatment processes on the downstream end of the lagoons. These alternatives were considered and ultimately rejected due to higher costs. Improving the organic treatment capacity of the lagoons will be needed at some point in the future. Figure 7-2 above, shows the anticipated increase in organic loading with respect to the existing organic treatment capacity of the plant. The organic loading is not expected to rise above the treatment capacity of the plant during the planning period, but will during the next planning period if projected population growth rates hold. Aeration is the only practical alternative for increasing the organic treatment capacity of the lagoons. As such, lagoon aeration will eventually be required whether driven by the need to improve effluent quality or not. Therefore, lagoon aeration is a logical choice as a first step to improving plant performance beyond the capabilities of traditional facultative lagoons.

The recommended improvements include the installation of a diffused aeration system in the first lagoon cell (both cells 1A & 1B). A diffused aeration system utilizes blowers and air distribution piping to distribute air to diffusers mounted in the lagoons. As an alternative to a diffused aeration system, floating mechanical aerators can also be installed in lagoons. The drawback of floating mechanical aerators is that lagoon water levels must be sufficiently high to prevent scouring of the lagoon bottom. The advantage of floating mechanical aerators is lower overall cost. A diffused aeration system was selected as the preferred aeration alternative despite the higher cost. The reason for this is that a diffused aeration system will allow operators the flexibility to draw lagoon water levels down and thereby maximize the amount of water that can be disposed of during the irrigation season. With floating mechanical aerators, the minimum water level that must be maintained is approximately six feet. With a diffused aeration system, the water level can be drawn down below six feet.

At the present time, the raw influent to the plant is not screened. All large material including rags and paper enter the lagoons and eventually settle and are incorporated in the sludge layer. These large solid materials can interfere with aeration equipment and decrease the overall treatment efficiency of the plan. For this reason, most aerated lagoon systems include fine screening systems at the headworks that remove large solid material from the raw wastewater prior to the lagoons. Therefore, the proposed improvements include the installation of a fine screen (i.e., 6 millimeter screen size) to remove large materials from the raw influent. It is envisioned that the existing headworks structure will be modified to accommodate the installation of the screening equipment. This will include extending the concrete headworks channel north from the existing structure. The influent pipes will be modified to connect to the upstream end of the new channel extension. The northern wall of the existing structure will be cut out or cored to allow flow from the new portion of the channel to the old portion of the channel. A new fine screen will be installed in the new portion of the structure. The fine screen will include solids washing and dewatering. Dewatered solids will fall into a dumpster mounted adjacent to the channel. The dumpster will be emptied by the local solid waste collection company. A second channel parallel to the screen channel will also be constructed for a manually-cleaned bar screen that will serve as a backup screening system. Upstream of the screen a side overflow weir will be constructed with an overflow pipe routed directly to lagoon cell 1. This will control the flow of water during extreme high flow events or upon a failure of the screening equipment.

The total recommended budget for this project is $2,500,000. A detailed cost estimate is included in Appendix E and design criteria are listed in Table 7-2.

Table 7-2 | Recommended Lagoon Aeration and Headworks Screening Design Criteria

Screen Type	Fine screen with shaft-less spiral auger
Screen Opening Size	6 mm
Screenings Disposal	Dumpster collected by local solid waste company on weekly basis
Redundant Screening	Manual Bar Screen
Lagoon Aeration Equipment	Diffused Aeration Grid
Total Aeration Power	120 Hp

- *Land Application System Expansion (Project T-3)*

As described above (subsection 7.3.7), the land application facilities will need to be expanded during the planning period. The need for this project was anticipated as part of the 2011 treatment plant improvement project. The City's current recycled water use plan identified approximately 500 acres on the east side of Bellfountain Road that can be irrigated. This land is owned by Brooks Farms LLC. All discussions with Brooks Farms LLC to date, have indicated that they are expecting to use the water when it becomes available. The current agreement with Brooks Farms calls for the City to extend an irrigation distribution pipeline from the existing system to the east across Bellfountain Road. Brooks Farms will be responsible for the installation of all piping and sprinkler equipment on the east side of Bellfountain Road. No major modifications to the irrigation pump station are needed to expand the land application system.

The existing irrigation distribution piping terminates near the southwest corner of lagoon cell 3. From this location, a new 16-inch diameter pipeline will be constructed along the access road on the south side of lagoon cell 3 to Bellfountain Road. The crossing of Bellfountain Road will be by auger boring. It is envisioned that the need for this project will be triggered by increases in dry weather flows to the plant caused by population growth. In addition to the construction work, a minor revision of the recycled water use plan will be required prior to applying recycled water on the fields east of Bellfountain Road. The recommended project budget includes funds for this purpose. The total recommended budget for this project is $394,000. A detailed cost estimate is included in Appendix E.

- *Facilities Plan Update(Project T-4)*

The planning assumptions used as the basis for this study are subject to change over the years. As such, the City should update this document at approximately 10 year intervals. To facilitate this, a facilities plan update project is included in the recommended capital improvement plan. The recommended budget for this work is $65,000.

7.6 SUMMARY OF RECOMMENDATIONS

The recommended treatment system improvements described above are summarized below (Table 7-3). These improvements should result in a treatment system that will serve the City for the remainder of the planning period if population growth does not exceed the projections presented in this plan.

Table 7-3 | Recommended Treatment System Improvements

Project Code	Project Description	Recommended Budget
Treatment System Improvements		
T-1	Marys River Outfall Diffuser	$173,000
T-2	Lagoon Aeration and Headworks Screening	$2,500,000
T-3	Land Application System Expansion	$394,000
General Treatment System		
T-4	Facilities Plan Update	$65,000

CHAPTER 8

CAPITAL IMPROVEMENT PLAN

Chapter Outline

CAPITAL IMPROVEMENT PLAN

8.1 INTRODUCTION

As documented in the previous sections, there is a need for wastewater system improvements within the study area to correct existing and projected deficiencies. Some of these deficiencies are more critical than others. Some deficiencies exist under current conditions, while other deficiencies will manifest as the City grows and/or the existing systems continue to age.

Recommended improvements for specific components of the City's wastewater system have been described in previous chapters. This chapter builds on that work by assigning a priority to each of the improvement recommendations. The cost estimates have been developed to a conceptual level, for planning and budgeting purposes. More detailed cost estimates will be necessary as the projects are implemented.

8.2 PRIORITIZED IMPROVEMENTS

A prioritizing process is required since the scope of the proposed improvements is large. Projects that resolve immediate deficiencies should naturally have a higher priority than long term growth related improvements. The following approach is designed to provide a basis for evaluating and ranking the improvement projects.

8.2.1 Prioritization Criteria

The assignment of a particular project or capital improvement program to a priority level was made after an evaluation using the following criteria:

- Public Health Concerns—Projects targeted to resolve existing or near term regulatory compliance issues were assigned the highest priority.
- Capacity or Size Deficiencies—The severity of the deficiency was considered and compared with the service improvements provided by the replacement components. The projected 'yield' or cost-benefit ratio of a project was used to assign a priority of high, medium or low.
- Consumed Infrastructure—Projects to replace damaged or deteriorated infrastructure, particularly those facilities that have reached the end of their useful life and no longer function as designed were assigned a higher priority.
- City Priority—Projects identified by City operations and maintenance personnel to be high priority due to operational or maintenance problems.
- Development Demand —The anticipated timeframe for the development of land within the service area of proposed improvements was considered. Projects to serve approved or near term developments were given higher priority, while improvements targeted to long term developments were deferred.

8.2.2 Prioritized Groups

In order to assist the City with their planning, scheduling and construction efforts each improvement project was assigned to one of three priority levels. The priority levels are:

- Priority 1—Near Term Improvements

  These projects are targeted to problem areas needing immediate attention. They have been developed to resolve existing or near term system deficiencies, resolve regulatory compliance issues or to serve known near-term anticipated developments. It is recommended that Priority 1 improvements are undertaken as soon as practical.

- Priority 2—Intermediate Improvements

  These projects will be needed beyond the near term of the Priority 1 projects to provide service to anticipated future developments or to address problems with existing infrastructure that is likely to become deficient during the planning period. Although not critical at this time, Priority 2 improvements should be considered as improvement projects that will be upgraded to Priority 1 at some point during the planning period.

- Priority 3—Long Term Improvements/Possible Future Need

  These projects are needed to improve system reliability or to supply future demands if land develops to the zoned densities. While important, they are not considered to be critical at the present time. If possible, improvements in this category should be incorporated into ongoing citywide development and improvement projects to capture the savings associated with concurrent construction. Projects that will need to be constructed by developers in conjunction with future developments were assigned to this group.

8.2.3 Prioritized Capital Improvement Projects

To aid in the development of a wastewater system capital improvement program (CIP), each improvement project was examined and assigned to one of the priority classes described above. Table 8-1 is a comprehensive listing of these projects. An overall map is included in Appendix D that shows the improvement priorities in a graphical format. The reader is referred to previous chapters of this report for more detailed descriptions of the individual projects.

At a minimum, all of the Priority 1 and Priority 2 improvements should be included in the CIP. The Priority 3 improvements are largely growth driven. In general, it is envisioned that the Priority 3 improvements will be constructed as part of future development and that the developer will pay for the improvements. Should the City desire to promote development in certain areas, selected Priority 3 improvements may also be included in the CIP.

Following adoption of this plan and the CIP listed below, the City should consider the preparation of a financial analysis of the wastewater utility that includes recommendations for changes to utility rates and SDC fees.

Table 8-1 | Recommended Capital Improvement Priorities

Project Code [1]	Project	Priority	Total Estimated Project Cost [2]
G-1 [3]	9th Street to 7th Street Sewer Lines – Manhole #35 to Manhole #184	1	$398,000
G-2 [3]	10th Street Sewer Lines – Manhole #34 to Manhole #45	1	$126,000
G-3 [3]	Main Street Sewer Lines – Manhole #45 to Manhole #52	1	$230,000
G-4	8th & College Street Sewer Lines – Manhole #52 to Manhole #56	1	$189,000
G-8	Applegate Street and 20th Street Trunk Sewer - Manhole #1 to Manhole #6	1	$344,000
G-16	Timber Estates Trunk Sewer	1	$370,000
F-1	Newton Creek Forcemain	1	$1,441,000
T-3	Land Application System Expansion	1	$394,000
		Subtotal Priority 1….	**$ 3,492,000**
G-5	Pioneer and 11th Street Sewer Lines – Manhole #71 to Manhole #74	2	$237,000
G-6	15th Street Trunk Sewer (South) – Manhole #27 to Manhole #288	2	$510,000
G-7	15th Street Trunk Sewer (North) – Manhole #288 to Manhole #94	2	$116,000
P-3	Newton Creek Pump Station Improvements	2	$1,479,000
T-1	Marys River Outfall Diffuser	2	$173,000
T-2	Lagoon Aeration and Headworks Screening	2	$2,500,000
T-4	Facilities Plan Update	2	$65,0000
		Subtotal Priority 2….	**$ 5,080,000**
G-9	Newton Creek Trunk Sewer – Newton Creek Pump Station to Manhole 476	3	$764,000
G-10	19th Street Trunk Sewer South	3	$917,000
G-11	Railroad Trunk Sewer	3	$1,014,000
G-12	19th Street/Green Road Trunk Sewer	3	$1,271,000
G-13	Industrial Way Trunk Sewer	3	$866,000
G-14	Sewer Basin N5 Trunk Sewer	3	$622,000
G-15	Chapel Drive Trunk Sewer	3	$1,056,000
P-1	Basin P1 Pump Station and Forcemain	3	$530,000
P-2	Basin P2 Pump Station and Forcemain	3	$500,000
		Subtotal Priority 3….	**$ 7,054,000**
		TOTAL….	**$ 15,626,000**
Recurring Annual Programs			
Pgm-1	Sewer Collection System Rehabilitation Program (Program – 1)		$200,000
		Subtotal Recurring Annual Programs….	**$ 200,000**

[1] Project Code Legend:
G = Gravity Sewer T = Treatment Pgm = Improvement Program
P = Pump Station F = Forcemain

[2] See Section 8.3 for basis of project cost estimates

[3] Project scheduled for completion in 2017

8.2.4 Environmental Impact

It should be noted that while the improvements recommended in this report are not anticipated to have significant adverse impacts on the environment, each CIP project will need to undergo project-specific environmental review (as applicable) as part of the preliminary and final design process.

8.3 BASIS OF COSTS

In order to forecast municipal capital expenditures, cost estimates have been prepared for each improvement alternative. The preparation methodology and intended use of these cost estimates are summarized below.

8.3.1 Accuracy of Cost Estimates

The accuracy and precision of cost estimates is a function of the level to which improvement alternatives are developed (i.e., detail and design) and the techniques used in preparing the actual estimate. Estimates are typically divided into three basic categories as follows:

- Planning Level Estimate. These are order-of-magnitude estimates made without detailed engineering design data. They are often performed at the zero to 2 percent stage of project completion and typically range from 35 percent over, to 25 percent below the final project cost. A relatively large contingency is typically included to reduce the risk of underestimating. This is particularly important since many times the project financing must be secured before the detailed design can proceed.
- Budgetary Estimates. This level of estimate is prepared during the preliminary design phase using process flow sheets, preliminary layouts and equipment details. This type of estimate is typically accurate to +30 and –15 percent of the final project cost.
- Engineer's Estimate. This estimate is prepared on the basis of well-defined engineering data, typically when the construction plans and specifications are completed. The estimating process at this level relies on piping and instrument diagrams, electrical diagrams, equipment data sheets, structural drawings, geotechnical data and a complete set of specifications. This estimate is sometimes called a definite estimate. The engineer's estimate is expected to be accurate within +15 percent to –5 percent of the pricing secured during the bidding process.

The project costs prepared as part of this study are planning level estimates. Actual project costs will depend on the final project scope, labor and material costs, market conditions, construction schedule, and other variables at the time the project is built. These variables are typically uncertain at the time planning level estimates are performed.

8.3.2 Adjustment of Cost Estimates over Time

A commonly used indicator to evaluate the change of construction costs over time is the Engineering News-Record (ENR) construction cost index. The index is computed from the prices for structural steel, Portland cement, lumber, and common labor, and is based on a value of 100 in the year 1913. The construction costs developed in this analysis are based on October 2016 ENR 20 City Construction Cost Index of 10,440. As the planning period elapses, the costs presented in this study can be updated to the present, by applying the ratio of the current cost index to the index used during the preparation of the estimate.

8.3.3 Engineering and Administrative Costs and Contingencies

The cost of engineering services for major projects typically covers special investigations, pre-design reports, topographic surveying, geotechnical investigations, contract drawings and specifications, construction administration, inspection, project start-up, the preparation of O&M manuals, and performance certifications. Depending on the size and type of the project, engineering costs may range from 16 to 25 percent of the contract cost when all of the above services are provided. The lower percentage applies to large projects without complex mechanical systems. The higher percentage applies to smaller, more complex projects that require the integration of a complex design into an existing facility and where full time inspection is required by the funding agencies or desired by the Owner.

The City will have administrative costs associated with any construction project. These include internal planning and budgeting costs, administration of engineering and construction contracts, legal services, and coordination with regulatory and funding agencies.

8.4 CONSTRUCTION COST ESTIMATES

The planning level estimates for the improvements recommended in this study are based on a number of assumptions as follows. The cost estimates reflect projects bid in late winter or early spring for summer construction. The estimates are based on construction costs of similar historical projects and on current estimates solicited from material and equipment vendors. The estimates are expected to have accuracies of +35 percent and –25 percent of the actual project cost. The following sections describe the cost estimating process for the various categories of projects.

8.4.1 Gravity Collection System Improvement Costs

The cost estimates for the proposed gravity pipeline improvements were based on the following assumptions.

- Normal depth sewer pipeline construction
- 8 inch gravity pipeline construction cost (materials, installation & surface restoration, etc.) - $130 per foot
- 10 inch gravity pipeline construction cost (materials, installation & surface restoration, etc.) - $140 per foot
- 12 inch gravity pipeline construction cost (materials, installation & surface restoration, etc.) - $150 per foot
- 15 inch gravity pipeline construction cost (materials, installation & surface restoration, etc.) - $170 per foot
- 18 inch gravity pipeline construction cost (materials, installation & surface restoration, etc.) - $180 per foot

- 21 inch and 24 inch gravity pipeline construction cost (materials, installation & surface restoration, etc.) - $190 per foot
- New Manholes (materials, installation, and surface restoration) - $6,000 each
- Service Laterals (materials, installation, and surface restoration) - $3,000 each
- Railroad, Highway, and Stream Bores - $1000 per foot
- Construction Contingencies - 10% of estimated construction cost
- Engineering Costs (surveying, engineering design, and construction administration) - 20% of estimated construction cost
- Legal, Permits & Administrative Costs (permitting, administration, legal, easement acquisition and financing) - 10% of estimated construction cost

The recommended project budgets for each project are listed in Table 8-1. A detailed breakdown of the construction costs, contingency, design, and administration costs are included in Appendix C. The cost estimates for the improvements to the existing collection system are generally based on open cut construction techniques. The cost estimates also include manhole replacement and replacement of the public and private portions of the service laterals. The cost estimates for the sewer line extensions needed to serve undeveloped areas only include the pipeline and manhole costs.

8.4.2 Pump Station and Forcemain Improvement Costs

Construction costs for new pump stations include site preparation, foundation, wetwell construction, building, pumps, mechanical piping, emergency power generation, and electrical and instrumentation. Project costs have been based on historical construction cost information for similarly sized projects, discussions with manufacturers, and the assumption that the pump stations will be constructed in accordance with the pump station design criteria listed in Chapter 3. A construction contingency of 10%, an engineering design cost of 20% and an administrative, legal and permitting cost of 10% was assumed for these projects.

8.4.3 Wastewater Treatment Improvement Costs

Construction costs for the wastewater treatment plant improvements include site preparation and foundations, buildings, tankage, treatment equipment for each unit process, associated mechanical piping and pumping, chemical feed equipment, yard piping, outfall piping, and electrical and instrumentation.

A construction contingency of 10% of the estimated construction cost was used for the treatment plant estimates. Engineering, Legal, and administration costs were assumed to be 20% of the estimated construction cost. Permitting costs were assumed to be 2% of the estimated construction cost.

8.5 FUNDING SOURCES

As a general rule, small communities are not able to finance major wastewater system improvements without some form of government funding such as low interest loans or grants. It is anticipated that the funding for the recommended capital improvement plan outlined in this report will be secured from multiple sources, including system development charges (SDCs), monthly user fees, as well as state and federal grant and loan programs. The following section outlines the major local and State/Federal funding programs that may be available for these projects.

8.5.1 Local Funding Sources

To a large degree, the type and amount of local funding used for the improvements will depend on the amount of grant funding obtained and the requirements of any loan funding. Local revenue sources for capital improvements include ad valorem taxes (property taxes), various types of bonds, user fees, connection fees and SDCs. Local revenue sources for operating costs include ad valorem taxes and user fees. The following sections discuss local funding sources and financing mechanisms that are most commonly used for the type of capital improvements presented in this study.

8.5.1.1 User Fees

User fees are monthly charges to all residences, businesses, and other users that are connected to the system. User fees are established by the City Council and are typically the sole source of revenue to finance operation and maintenance. These fees are periodically modified to account for changes in operation and maintenance costs, and the need for new improvements. Although user fees are not always sufficient to finance major capital construction projects, they can be used to repay long term financing. The reader is referred to Section 4.7.1 for a description of the City's current user fee structure.

8.5.1.2 System Development Charge Revenues

A system development charge (SDC) is a fee collected by the City as each piece of property is developed. SDCs are used to finance necessary capital improvements and municipal services required by the development. SDCs can be used to recover the capital costs of infrastructure required as a result of the development, but cannot be used to finance either operation and maintenance, or replacement costs. The reader is referred to Section 4.7.2 for information on the City's current SDC charges.

As established in ORS 223, a SDC can have two principal elements, the reimbursement fee and the improvement fee. Fees are collected at issuance of building permits. The reimbursement portion of the SDC is the fee for buying into either existing capital facilities or those that are under construction. The reimbursement fee represents a charge for utilizing excess capacity in an existing facility that was paid for by other parties. The revenue from this fee is typically used to repay existing improvement loans. The improvement portion of the SDC is the fee designed to cover the costs of capital improvements that must be constructed to provide an increase in capacity.

8.5.1.3 Connection Fees

Many cities charge connection fees to cover the cost of connecting a new development to the municipal sewer system. There are two types of connection fees. The first is for newly constructed connections and is designed to cover the cost of City inspections at the time of connection to the collection system. The second type of fee is designed to defray the City's administrative cost of setting up a new account and is charged against newly constructed connections, as well as transfers of an existing service to a new owner.

8.5.1.4 Capital Construction Fund

Capital construction funds, or sinking funds, are often established as a budget line item to set aside money for a particular construction purpose. A set amount from each annual budget is deposited in a sinking fund until sufficient reserves are available to complete the project. Such funds can also be developed from user fee revenues or from SDCs. The status of the City's capital improvement funds is discussed in Chapter 4 (see section 4.7.4).

8.5.1.5 General Obligation Bonds

The sale of municipal general obligation bonds is a traditional method of funding municipal improvement projects. General obligation bonds utilize the City's basic taxing authority and are retired with property taxes based on an equitable distribution of the bonded obligation across the City's assessed valuation. General obligation bonds are normally associated with the financing of facilities that benefit an entire community and must be approved by a majority vote of the City's voters.

General obligation bonds are backed by the City's full faith and credit, as the City must pledge to assess property taxes sufficient to pay the annual debt service. This portion of the property tax is outside the State constitutional limits that restrict property taxes to a fixed percentage of the assessed value. The City may use other sources of revenue, including user fee revenues, to repay the bonds. If it uses other funding sources to repay the bonds, the amount collected as taxes is reduced commensurately.

The general procedure followed when financing improvements with general obligation bonds is typically as follows:

- Determination of the capital costs required for the improvement
- An election by the voters to authorize the sale of bonds
- The bonds are offered for sale
- The revenue from the bond sale is used to pay the capital cost of the project(s)

General obligation bonds can be "revenue supported", wherein a portion of the user fee is pledged toward repayment of the bond debt. The advantage of this method is that the need to collect additional property taxes to retire the bonds is reduced or eliminated. Such revenue supported general obligation bonds have most of the advantages of revenue bonds in addition to a lower interest rate and ready marketabilty.

The primary disadvantage with the use of general obligation bonds is that the debt incurred by this method is often added to the debt ratios of the City. This has the potential to limit flexibility of the municipality to issue debt for other purposes.

8.5.1.6 Revenue Bonds

Revenue bonds are similar to general obligation bonds, except they rely on revenue from the sales of the utility (i.e., user fees) to retire the bonded indebtedness. The primary security for the bonds is the City's pledge to charge user fees sufficient to pay all operating costs and debts service.

Because the reliability of the source of revenue is relatively more speculative than for general obligation bonds, revenue bonds typically have slightly higher interest rates.

The general shift away from ad valorem property taxes makes revenue bonds a frequently used option for payment of long term debt. Many communities prefer revenue bonding, because it ensures that no additional taxes are levied. In addition, repayment of the debt obligation is limited to system users since repayment is based on user fees.

One advantage with revenue bonds is that they do not count against a City's direct debt. This feature can be a crucial advantage for a municipality near its debt limit. Rating agencies closely evaluate the amount of direct debt when assigning credit ratings. There are normally no legal limitations on the amount of revenue bonds that can be issued; however, excessive issue amounts are generally unattractive to bond buyers because they represent high investment risks.

Under ORS 288.805-288.945, Cities may elect to issue revenue bonds for revenue producing facilities without a vote of the electorate. Certain notice and posting requirements must be met and a sixty (60) day waiting period is mandatory.

The bond lender typically requires the City to provide two additional securities for revenue bonds that are not required for general obligation bonds. First, the City must set user fees such that the net projected cash flow from user fees plus interest will be at least 125% of the annual debt service (a 1.25 debt coverage ratio). Secondly, the City must establish a bond reserve fund equal to maximum annual debt service or 10% of the bond amount, whichever is less.

8.5.1.7 Improvement Bonds

Improvement (Bancroft) bonds are an intermediate form of financing that are less than full-fledged general obligation or revenue bonds. This form of bonding is typically used for Local Improvement Districts.

Improvement bonds are payable from the proceeds of special benefit assessments, not from general tax revenues or user fees. Such bonds are issued only where certain properties are recipients of special benefits not occurring to other properties. For a specific improvement, all property within the designated improvement district is assessed on the same basis, regardless of whether the property is developed or undeveloped. The assessment is designed to divide the cost of the improvements among the benefited property owners. The manner in which it is divided is in proportion to the direct or indirect benefits to each property. The assessment becomes a direct lien against the property, and owners have the option of either paying the assessment in cash, or applying for improvement bonds. If the improvement bond option is taken, the City sells Bancroft Improvement Bonds to finance the construction, and the assessment is paid over 20 years in 40 semiannual installments plus interest.

The assessments against the properties are usually not levied until the actual cost of the project is determined. Since the determination of actual costs cannot normally be determined until the project is completed, funds are not available from assessments for the purpose of paying costs at the time of construction. Therefore, some method of interim financing must be arranged.

The primary disadvantage to this source of revenue is that the development of an assessment district is very cumbersome and expensive when facilities for an entire City are contemplated.

Therefore, this method of financing should only be considered for discrete improvements to the collection system where the benefits are localized and easily quantified.

8.5.1.8 Certificates of Participation

Certificates of Participation are a form of bond financing that is distinct from revenue bonds. While it is more complex, and typically has a higher interest rate than revenue bonds, it is a process controlled by the City Council, and it does not have to be referred to the voters. This can result in significant time savings.

8.5.1.9 Ad Valorem Property Taxes

Ad valorem property taxes were often used in the past as a revenue source for public utility improvements. These taxes were the traditional means of obtaining revenue to support all local governmental functions. Ad valorem taxation is a financing method that applies to all property owners that benefit, or could potentially benefit from an improvement, whether the property is developed or not. The construction costs for the improvement project are shared proportionally among all property owners based on the assessed value of each property. Ad valorem taxation, however, is less likely to result in individual users paying their proportionate share of the costs as compared to their benefits.

8.5.2 State and Federal Grant and Loan Programs

Several state and federal grant and loan programs are available to provide financial assistance for municipal wastewater system improvements. The primary sources of funding available for wastewater system financing are Rural Utilities Service (RUS), Special Public Works Fund (SPWF), the Water/Wastewater (W/W) Financing Program, the Community Development Block Grant (CDBG) program, and the Clean Water State Revolving Fund (CWSRF).

8.5.2.1 USDA Rural Development

USDA Rural Development (RD) provides federal loans and grants to rural municipalities, counties, special districts, Indian tribes, and not-for-profit organizations to construct, enlarge, or modify water treatment and distribution systems and wastewater collection and treatment systems (https://www.rd.usda.gov/programs-services/water-waste-disposal-loan-grant-program/or). Preference is given to projects in low-income communities with populations below 10,000.

Borrowers of RD loans must be able to demonstrate the following:

- Monthly user rates must be at or above the local area-wide average.
- They have the legal authority to borrow and repay loans, to pledge security for loans, and to operate and maintain the facilities and services.
- They are financially sound and able to manage the facility effectively.
- They have a financially sound facility based on taxes, assessments, revenues, fees, or other satisfactory sources of income to pay for all facility costs including O&M and to retire indebtedness and maintain a reserve.

The maximum RD loan term is 40 years, but the finance term may not exceed statutory limitations on the agency borrowing the money or the expected useful life of the improvements. The reserve can typically be funded at 10 percent per year over a ten-year period. Interest rates for RD loans vary based on median household income, but tend to be lower than those obtained in the open market. RD funding programs typically provide funding for project once construction is completed. As such, these programs require a recipient to arrange for interim financing to fund the design and construction of the project before RD funds are made available.

8.5.2.2 Oregon Infrastructure Finance Authority

The Oregon Infrastructure Finance Authority (IFA) manages a number of grant and low interest loan programs as described in the following sections.

Special Public Works Fund (http://www.orinfrastructure.org/Infrastructure-Programs/SPWF/)

The IFA administers the Special Public Works Fund (SPWF) program. The SPWF is a lottery-funded loan and grant program that provides funding to municipalities, counties, special districts, and public ports for infrastructure improvements to support industrial/manufacturing and eligible commercial economic development. Eligible commercial economic development is defined as commercial activity that is marketed nationally, or internationally, and attracts business from outside Oregon. Funded projects are usually linked to a specific private sector development and the resulting direct job creation (i.e., firm business commitment), of which 30% of the created jobs must be "family wage" jobs. The program also funds projects that build infrastructure capacity to support industrial/manufacturing development where recent interest by eligible business(s) can be documented.

The SPWF is primarily a loan program, although grant funds are available based on economic need of the community. Although the maximum loan term is 25 years, loans are generally made for 20-year terms. The maximum loan amount for projects funded with direct SPWF money is $1 million, while the maximum for projects financed with bond funds is $10 million.

Water/Wastewater Financing Program (http://www.orinfrastructure.org/Infrastructure-Programs/WW/)

The IFA also administers the W/W Financing Program, which gives priority to projects that provide system-wide benefits and helps communities meet the Clean Water Act or the Safe Drinking Water Act standards. It is intended to assist local governments that have been hard hit with state and federal mandates for public drinking water systems and wastewater systems. In order to be eligible for this program, the system must be out of compliance with federal or state rules, regulations or permits, as evidenced by issuance of Notice of Non-Compliance by the appropriate regulatory agency. The funded project must be needed to meet state or federal regulations. Priority is given to communities under economic distress.

Similar to the SPWF, the W/W Financing Program is primarily a loan program, although grant funds are available in certain cases, based on economic need of the community. Although the maximum loan term is 25 years, loans are generally made for 20-year terms. The maximum loan amount for projects funded with direct W/W money is $500,000, while the maximum for projects financed with bond funds is $10 million.

Economic and Community Development Block Grant (http://www.orinfrastructure.org/Infrastructure-Programs/CDBG/)

The IFA administers the CDBG, but the funds are from the U.S. Department of Housing and Urban Development (HUD), so all federal grant management rules apply to the program. The federal eligibility standards are strict. There are two subcategories of Public Works projects eligible for funding, "Public Water and Wastewater," and "Public Works for New Housing." Only the former is considered in this discussion.

Grants are available for critically needed construction, improvement, or expansion of publicly owned water and wastewater systems for the benefit of current residents. Generally, projects must be necessary to resolve regulatory compliance problems identified by state and/or federal agencies and the project must serve a community that is comprised of more than 51% of low and moderate income persons.

The program separates projects into three parts. Grants are available for:

- Preliminary Engineering and Planning Projects

  Generally, these grants fund preparation or update of Water System Master Plans and Wastewater Facility Plans, as required by the Oregon Department of Environmental Quality or Oregon Health Division. In addition, funds for grant administration and preparation of a final design funding application can be included in the project budget. All plans produced with grant funds must be approved by the appropriate regulatory agency. Grants of up to $10,000 can also be made for problem identification studies to delineate problems and corrective measures, as required by a regulatory agency.

- Final Design and Engineering Projects

  Final design and engineering, bid specifications, environmental review, financial feasibility, rate analysis, grant administration, and preparing a construction funding application are all eligible project activities. The final design, plans and specifications must be approved by the appropriate regulatory agency before a grant will be awarded.

- Construction Projects

  These grants fund construction and related activities, grant administration, and land/permanent easement acquisition.

  IFA has established an evaluation system that gives priority to projects that provide system-wide benefits. The overall maximum grant amount per water or wastewater project is $2,000,000 (including all planning, final engineering, and construction). The project cannot be divided locally into phases with the expectation of receiving more than one $2,000,000 grant. In order to qualify for grant funding under this program, the water user rates must be at or above statewide averages.

Based on the 2016 guidelines for the Community Development Block Grant Program, approximately 37% of the families in Philomath are classified as having low or moderate incomes. This is below the 51% threshold to be eligible for a Community Development Block Grant. As such, it does not appear that the City qualifies for a Block Grant. However, the requirements for these funding programs do change periodically, so it is worth verifying with the IFA.

8.5.2.3 Clean Water State Revolving Fund

The Clean Water State Revolving Fund (CWSRF) is administered by Oregon DEQ and provides loans to cities, counties, special districts, and Indian tribes to construct, expand, or rehabilitate water pollution control, estuary management projects, and non-point source control plants (http://www.oregon.gov/deq/wq/cwsrf/Pages/default.aspx).

Interest rates on loans are about 80% of the general obligation bond rate; however, there are additional financing costs and annual service fees that increase the effective rate. The maximum loan amount per project is 15% of the total available money in a particular year. The maximum loan term is 20 years, but there is an option for longer-term financing for treatment works for terms up to 30 years. This is accomplished by the community selling DEQ a revenue bond with repayment terms up to 30 years or the operational life of the treatment works, whichever is less.

8.5.3 Funding Recommendations

Based on the infrastructure improvements and cost projections presented in this plan, the existing user fee and SDC fee structures may not be sufficient to meet the planning period goals. This plan accordingly recommends that the City complete a full review of its user fee and SDC rate structure and update these fees accordingly. Should the City choose to pursue funding assistance from one of the state and federal agencies an important early step is to schedule a "one stop meeting" with Oregon Infrastructure Finance Authority (IFA). These meetings are designed to gather staff from the various federal and state funding agencies to evaluate the applicability of the various funding sources to a particular municipal project.

APPENDIX A

NPDES PERMIT

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM
WASTE DISCHARGE PERMIT
Department of Environmental Quality
Western Region – Salem Office
750 Front Street NE, Suite 120, Salem, OR 97301-1039
Telephone: (503) 378-8240

Issued pursuant to ORS 468B.050 and The Federal Clean Water Act

ISSUED TO:
City of Philomath
P.O. Box 400
Philomath, OR 97370

SOURCES COVERED BY THIS PERMIT:

Type of Waste	Outfall Number	Outfall Location
Treated Wastewater	001	R.M. 10.6
Reclaimed Water Reuse	002A	Class C
	002B	Class D

FACILITY TYPE AND LOCATION:
Stabilization Lagoons without Aeration
Philomath WWTP
4702 Bell Fountain Rd
Corvallis, Oregon
Treatment System Class: Level I
Collection System Class: Level II

RECEIVING STREAM INFORMATION:
Basin: Willamette
Sub-Basin: Upper Willamette
Receiving Stream: Marys River
LLID: 1232609445565-10.6-D
County: Benton

EPA REFERENCE NO: OR0032441
Issued in response to Application No. 968014 received December 21, 2010. This permit is issued based on the land use findings in the permit record.

Steve Schnurbusch, Acting Water Quality Manager
Western Region North

12/27/12
Date

PERMITTED ACTIVITIES

Until this permit expires or is modified or revoked, the permittee is authorized to construct, install, modify, or operate a wastewater collection, treatment, control and disposal system and discharge to public waters adequately treated wastewaters only from the authorized discharge point or points established in Schedule A and only in conformance with all the requirements, limitations, and conditions set forth in the attached schedules as follows:

Unless specifically authorized by this permit, by another NPDES or WPCF permit, or by Oregon Administrative Rule, any other direct or indirect discharge of waste is prohibited, including discharge to waters of the state or an underground injection control system.

SCHEDULE A

1. Waste Discharge Limitations not to be exceeded after permit issuance.

a. Treated Effluent Outfall 001 (see Note A1.)

(1) May 1 - October 31: No discharge to waters of the State

(2) November 1 - April 30:

Stream Flow (cfs) (see Note A2)	Maximum Effluent Flow (MGD)
<25 cfs	No Discharge Allowed
25-50 cfs	0.25 MGD
50-80 cfs	0.53 MGD
80-100 cfs	0.95 MGD
100-150 cfs	1.10 MGD
150-200 cfs	1.78 MGD
200-250 cfs	2.38 MGD
250-300 cfs	2.98 MGD
300-350 cfs	3.51 MGD
>350 cfs	4.00 MGD

The discharge period may be extended into May if:

(i) It is projected that the lagoon level would be higher than the minimum operating level on April 30, assuming average April river flows in the Mary's River, and effluent is discharged in accordance with the above table;
(ii) Documentation is submitted in writing to DEQ by April 1; and,
(iii) Written authorization is obtained from DEQ.

Parameter	Average Effluent Concentrations		Monthly* Average lb/day	Weekly* Average lb/day	Daily Maximum lbs
	Monthly	Weekly			
BOD_5	30 mg/L	45 mg/L	460	690	920
TSS	50 mg/L	80 mg/L	760	1100	1500

* Average dry weather design flow to the facility equals 0.475 MGD. Mass load limits based upon the winter discharge rate of 1.82 MGD to allow for disposal of summer accumulations of treated wastewater as well as winter stormwater impacting lagoon surface.

(3) Other Parameters

Year-round (except as noted)	Limitations
E. coli Bacteria	Shall not exceed 126 organisms per 100 mL monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL. (See Note A3)
pH	Shall be within the range of 6.3 - 9.0
BOD_5 and TSS Removal Efficiency	Shall not be less than 65% monthly average for BOD_5 and 55% monthly for TSS.
Total Residual Chlorine	Must not exceed 0.03 mg/L as a daily maximum and 0.01 mg/L as a monthly average (see Note A4)

b. Regulatory Mixing Zone

No wastes may be discharged or activities conducted that cause or contribute to a violation of water quality standards in OAR Chapter 340, Division 41 applicable to the Willamette Basin except within the following regulatory mixing zone:

The regulatory mixing zone is that portion of the Mays River where the effluent mixes with 25 percent of the stream flow but in no case may it extend farther than twenty (20) feet towards midstream and extending from a point ten (10) feet upstream of the outfall to a point one hundred (100) feet downstream from the outfall. The Zone of Immediate Dilution (ZID) is defined as that portion of the regulatory mixing zone that is within ten (10) feet of the point of discharge.

c. Recycled Water Outfall 002A and 002B

The permittee is authorized to distribute recycled water if it is:

a. Treated and used according to the criteria listed in the Table below.
b. Managed as described in its DEQ-approved Recycled Water Use Plan unless exempt as provided in Schedule D, Condition 2.
c. Used in a manner and applied at a rate that does not impact groundwater quality.
d. Applied at a rate and in accordance with site management practices that ensure continued agricultural, horticultural, or silvicultural production and does not reduce the productivity of the site.
e. Irrigated using sound irrigation practices to prevent:
   i. Offsite surface runoff or subsurface drainage through drainage tile;
   ii. Creation of odors, fly and mosquito breeding, or other nuisance conditions; and
   iii. Overloading of land with nutrients, organics, or other pollutants.

Recycled Water Limits

Class	Level of Treatment (after disinfection unless otherwise specified)	Beneficial Uses
C	Oxidized and disinfected. Total coliform may not exceed: • A median of 23 total coliform organisms per 100 mL, based on results of the last 7 days that analyses have been completed. • 240 total coliform organisms per 100 mL in any two consecutive samples.	• Irrigation of processed food crops; Irrigation of orchards or vineyards if an irrigation method is used to apply recycled water directly to the soil. • Landscape irrigation of golf courses, cemeteries, highway medians, or industrial or business campuses. • Industrial, commercial, or construction uses limited to: industrial cooling, rock crushing, aggregate washing, mixing concrete, dust control, nonstructural fire fighting using aircraft, street sweeping, or sanitary sewer flushing. • Irrigation of firewood, ornamental nursery stock, Christmas trees, sod, or pasture for animals. • Irrigation for growing fodder, fiber, seed crops not intended for human ingestion, or commercial timber.
D	Oxidized and disinfected. *E. coli* may not exceed: • A 30-day log mean of 126 organisms per 100 mL. • 406 organisms per 100 mL in any single sample.	• Irrigation of firewood, ornamental nursery stock, Christmas trees, sod, or pasture for animals. • Irrigation for growing fodder, fiber, seed crops not intended for human ingestion, or commercial timber.

d. **Septage Requirements**

Septage may not be accepted at this facility for treatment or processing without written approval from DEQ. Waste from recreational vehicles may be accepted.

e. **Groundwater Protection**

The permittee may not conduct any activities that could cause an adverse impact on existing or potential beneficial uses of groundwater. All wastewater and process related residuals must be managed and disposed of in a manner that will prevent a violation of the Groundwater Quality Protection Rules (OAR Chapter 340, Division 40).

NOTES:

A1. The permit may be re-opened, and new limits assigned upon approval of a Total Maximum Daily Load for dissolved oxygen basin.

A2. Stream flow must be based on USGS gage 14171000 unless otherwise approved in writing by DEQ.

A3. If a single sample exceeds 406 organisms per 100 mL, then five consecutive re-samples may be taken at four-hour intervals beginning within 48 hours after the original sample was taken. If the log mean of the five re-samples is less than or equal to 126 organisms per 100 mL, a violation shall not be triggered.

A4. When the total residual chlorine limitation is lower than 0.05 mg/L, DEQ will use 0.05 mg/L as the compliance evaluation level; that is, daily maximum or monthly average concentrations at or below 0.05 mg/L will be considered in compliance with the limit.

SCHEDULE B
Minimum Monitoring and Reporting Requirements

1. **Monitoring and Reporting Protocols**
   a. Quality Assurance and Quality Control (QA/QC)
   See Schedule F, Section C, Monitoring and Records for further instruction on proper sampling techniques and test methods and the use of laboratories with QA/QC procedures.

   b. Re-analysis, Re-sampling and Reporting of Data if QA/QC Requirements Not Met
   If QA/QC requirements are not met for any analysis, the permittee must have the sample re-analyzed. If the sample cannot be re-analyzed, the permittee must re-sample at the earliest available opportunity. Permittee must include the results of samples not meeting QA/QC in the report but must not use the data in the calculations required by the permit.

   c. Reporting Procedures
   i. *Quantitation Limits (QL)*
   The QL must be reported along with any result reported as "nondetect" or "ND". The QL is the Method Reporting Limit (MRL) or Limit of Quantitation (LOQ). It is the lowest level at which the entire analytical system must give a recognizable signal and acceptable calibration for the analyte. It is equivalent to the concentration of the lowest calibration standard assuming that all method-specified sample weights, volumes, and cleanup procedures have been employed

   ii. *Calculating Mass Loads*
   The permittee must calculate mass loads as follows:
   Flow (in MGD) X Concentration (in mg/L) X 8.34 = Pounds per day

Influent Monitoring Requirements
The permittee must monitor the influent at the Parshall Flume:

Table B1: Influent Monitoring

Item or Parameter	Time Period	Minimum Frequency	Sample Type
Total Flow (MGD)	Year-round	Daily	Measurement
Flow Meter Calibration		Annually	Verification
BOD_5 (mg/L)	Year-round	1 per 2 Weeks	24-hour composite
TSS (mg/L)	Year-round	1 per 2 Weeks	24-hour composite
pH (S.U.)	Year-round	2 per Week	Grab

Effluent Monitoring Requirements
The permittee must monitor the effluent from Outfall 001 for all grab and composite samples and measurements from the compliance manhole:

Table B2: Effluent Monitoring

Item or Parameter	Time Period	Minimum Frequency	Sample Type
Total Flow (MGD)	When discharging	Daily	Measurement
Flow Meter Calibration		Annually	Verification
BOD_5	When discharging	1 per 2 Weeks	24-hour composite

Item or Parameter	Time Period	Minimum Frequency	Sample Type
TSS (mg/L)	When discharging	1 per 2 Weeks	24-hour composite
pH (S.U.)	When discharging	2 per Week	Grab
E. coli (MPN/100mL)	When discharging	Weekly	Grab
Ammonia (NH3-N)	When discharging	Monthly	24-hour composite
Pounds discharged TSS & BOD_5	When discharging	1 per 2 Weeks	Calculation
% removed TSS & BOD_5	When discharging	Monthly	Calculation
Temperature	When discharging	2 per Week	Grab
Quantity Chlorine Used (lbs)	When discharging	Daily	Measurement
Total Residual Chlorine, Pre De-chlorination	When discharging	Daily	Grab; taken before dechlorination
Total Residual Chlorine	When discharging	Daily	Grab; taken after dechlorination and before discharge
Marys River flow	When discharging	Daily	Record (see Note B1)

Recycled Water Monitoring Requirements: Outfall 002
The permittee must monitor recycled water for all grab and composite samples and measurements from the sample taps on the irrigation pump station discharge piping or directly from the irrigation pump station wetwell.

Table B3: Recycled Water Monitoring

Item or Parameter	Minimum Frequency	Sample Type
Total Flow (MGD) or Quantity Irrigated (inches/acre)	Daily when irrigating	Measurement
Flow Meter Calibration	Annually	Verification
Quantity Chlorine Used (lbs)	Daily when irrigating	Measurement
Total Residual Chlorine	Daily when discharging	Grab
pH	2 per Week when discharging	Grab
Total Coliform (Class C)	Weekly when discharging	Grab
E. coli (Class D)	Weekly when discharging	Grab
Nutrients (TKN, NO2+NO3-N, NH3, Total Phosphorus)	Quarterly when discharging	Grab

Additional Information
The permittee must report the information listed below.

Table B4: Additional Information

Information	Minimum Frequency	Action
Name, certificate classification, and grade level of each responsible principal operator designated by the permittee and identification of each system classification.	Monthly	Record
Quantity and method of use or disposal of all wastewater solids removed from the treatment facility.	Monthly	Record
Equipment breakdowns and bypassing.	Monthly	Record

Minimum Reporting Requirements
The permittee must report monitoring results as listed below.

Table B5: Reporting Requirements and Due Dates
(This is a summary of previously-stated requirements. In the event that it contains information that differs from prior language in the permit, the prior language takes precedence.)

Reporting Requirement	Frequency	Due Date	Report Form	Submit To:
Table B1: Influent Monitoring Table B2: Effluent Monitoring	Monthly	15th day of the month following data collection	DEQ-approved discharge monitoring report (DMR) form.	DEQ Regional Office
Table B2: Effluent Monitoring Table B3: Recycled Water Monitoring	Monthly when discharging or irrigating	15th day of the month following data collection	DEQ-approved discharge monitoring report (DMR) form.	DEQ Regional Office
1. Recycled water annual report describing effectiveness of recycled water system in complying with the DEQ-approved recycled water use plan, OAR 340-055, and this permit. See Schedule D for more detail.	Annually	January 15	2 hard copies	One each to: • DEQ Regional Office • DEQ Water Reuse Program Coordinator
Inflow and infiltration report	Annually	February 1	1 hard copy	DEQ Regional Office

NOTES:

1. Stream flow must be based on USGS gage 14171000 unless otherwise approved in writing by DEQ.

SCHEDULE D

Special Conditions

1. **Recycled Water**
   a. Recycled Water Use Plan
   The permittee must maintain a Recycled Water Use Plan meeting the requirements in OAR 340-055-0025. The permittee must submit substantial modifications to an existing plan to DEQ for approval at least 60 days prior to making the proposed changes. Conditions in the plan are enforceable requirements under this permit.
   b. Exempt Activities
   The permittee is exempt from the requirement to prepare a Recycled Water Use Plan and the limits in Schedule A, Condition 1.c. when recycled water is used at the wastewater treatment system for landscape irrigation or for in-plant processes at a wastewater treatment system and all of the following conditions are met:
   i. The recycled water is an oxidized and disinfected wastewater.
   ii. The recycled water is used at the wastewater treatment system site where it is generated or at an auxiliary wastewater or sludge treatment facility that is subject to the same NPDES or WPCF permit as the wastewater treatment system. Contiguous property to the parcel of land upon which the treatment system is located is considered the wastewater treatment system site if under the same ownership.
   iii. Spray or drift or both from the use does not occur off the site.
   iv. Public access to the site is restricted.

2. **Inflow and Infiltration**
   An annual inflow and infiltration report must be submitted to DEQ as directed in Schedule B. The report must include the following:
   i. Details of activities performed in the previous year to identify and reduce inflow and infiltration.
   ii. Details of activities planned for the following year to identify and reduce inflow and infiltration.
   iii. A summary of sanitary sewer overflows that occurred during the previous year.

3. **Operator Certification**
   a. Definitions
   i. "Supervise" means to have full and active responsibility for the daily on site technical operation of a wastewater treatment system or wastewater collection system.
   ii. "Supervisor" or "designated operator", means the operator delegated authority by the permittee for establishing and executing the specific practice and procedures for operating the wastewater treatment system or wastewater collection system in accordance with the policies of the owner of the system and any permit requirements.
   iii. "Shift Supervisor" means the operator delegated authority by the permittee for executing the specific practice and procedures for operating the wastewater treatment system or wastewater collection system when the system is operated on more than one daily shift.
   iv. "System" includes both the collection system and the treatment systems.

   b. The permittee must comply with OAR Chapter 340, Division 49, "Regulations Pertaining to Certification of Wastewater System Operator Personnel" and designate a supervisor whose certification corresponds with the classification of the collection and/or treatment system as specified on p. 1 of this permit.

   c. The permittee must have its system supervised full-time by one or more operators who hold a valid certificate for the type of wastewater treatment or wastewater collection system, and at a grade equal to or greater than the wastewater system's classification as specified on p. 1 one of this permit.

d. The permittee's wastewater system may not be without the designated supervisor for more than 30 days. During this period, there must be another person available to supervise who is certified at no more than one grade lower than the classification of the wastewater system. The permittee must delegate authority to this operator to supervise the operation of the system.

e. If the wastewater system has more than one daily shift, the permittee must have another properly certified operator available to supervise operation of the system. Each shift supervisor, if any, must be certified at no more than one grade lower than the system classification.

f. The permittee is not required to have a supervisor on site at all times; however, the supervisor must be available to the permittee and operator at all times.

g. The permittee must notify DEQ in writing of the name of the system supervisor. The permittee may replace or re-designate the system supervisor with another properly certified operator at any time and must notify DEQ in writing within 30 days of replacement or re-designation of operator in charge. As of this writing, the notice of replacement or re-designation must be sent to Water Quality Division, Operator Certification Program, 2020 SW 4th Avenue, Suite 400, Portland, OR 97201. This address may be updated in writing by DEQ during the term of this permit.

h. Upon written request, DEQ may grant the permittee reasonable time, not to exceed 120 days, to obtain the services of a qualified person to supervise the wastewater system. The written request must include a justification for the time needed, schedule for recruiting and hiring, date the system supervisor availability ceased, and name of the alternate system supervisor as required by above.

4. **Emergency Response and Public Notification Plan**

The permittee must develop and maintain an Emergency Response and Public Notification Plan (the Plan) per Schedule F, Section B, and Conditions 7 & 8. The permit holder must develop the plan within six months of permit issuance and update the Plan annually to ensure that telephone and email contact information for applicable public agencies are current and accurate. An updated copy of the plan must be kept on file at the wastewater treatment facility for Department review. The latest plan revision date must be listed on the Plan cover along with the reviewer's initials or signature

5. **Wastewater Solids**

a. Transfers

i. *Within state.* The permittee may transfer wastewater solids to another facility permitted to process or dispose of wastewater solids, including but not limited to: another wastewater treatment facility, landfill, or incinerator. The permittee must monitor, report, and dispose of solids as required under the permit of the receiving facility.

ii. *Out of state.* If wastewater solids, including Class A and Class B biosolids, are transferred out of state for use or disposal, the permittee must obtain written authorization from DEQ, meet Oregon requirements for the use or disposal of wastewater solids, notify in writing the receiving state of the proposed use or disposal of wastewater solids, and satisfy the requirements of the receiving state.

b. Acceptance

i. *Within state.* The permittee may accept wastewater solids from another wastewater treatment facility. The permittee must monitor, report, and dispose of solids as required by this permit.

ii. *Out of state.* The permittee may accept wastewater solids from out-of-state facilities for treatment and land application when authorized in writing by DEQ provided the pollutant concentrations in the out-of-state solids do not exceed the ceiling concentration limits in Table 1 of 40 CFR Part 503.13.

6. **Lagoon Solids**

At least six months prior to the removal of accumulated solids from the lagoon, the permittee must submit to DEQ a biosolids management plan and land application plan developed in accordance with OAR 340-050. DEQ will provide an opportunity for comment on the biosolids management plan and land application plan as directed by OAR 340-050-0015(8).The permittee must follow the conditions in the approved plan.

SCHEDULE F
NPDES GENERAL CONDITIONS – DOMESTIC FACILITIES

SECTION A. STANDARD CONDITIONS

A1. Duty to Comply with Permit

The permittee must comply with all conditions of this permit. Failure to comply with any permit condition is a violation of Oregon Revised Statutes (ORS) 468B.025 and the federal Clean Water Act and is grounds for an enforcement action. Failure to comply is also grounds for DEQ to terminate, modify and reissue, revoke, or deny renewal of a permit.

A2. Penalties for Water Pollution and Permit Condition Violations

The permit is enforceable by DEQ or EPA, and in some circumstances also by third-parties under the citizen suit provisions 33 USC § 1365. DEQ enforcement is generally based on provisions of state statutes and Environmental Quality Commission (EQC) rules, and EPA enforcement is generally based on provisions of federal statutes and EPA regulations.

ORS 468.140 allows DEQ to impose civil penalties up to $10,000 per day for violation of a term, condition, or requirement of a permit. The federal Clean Water Act provides for civil penalties not to exceed $32,500 and administrative penalties not to exceed $11,000 per day for each violation of any condition or limitation of this permit.

Under ORS 468.943, unlawful water pollution, if committed by a person with criminal negligence, is punishable by a fine of up to $25,000, imprisonment for not more than one year, or both. Each day on which a violation occurs or continues is a separately punishable offense. The federal Clean Water Act provides for criminal penalties of not more than $50,000 per day of violation, or imprisonment of not more than 2 years, or both for second or subsequent negligent violations of this permit.

Under ORS 468.946, a person who knowingly discharges, places, or causes to be placed any waste into the waters of the state or in a location where the waste is likely to escape into the waters of the state is subject to a Class B felony punishable by a fine not to exceed $250,000 and up to 10 years in prison per ORS chapter 161. The federal Clean Water Act provides for criminal penalties of $5,000 to $50,000 per day of violation, or imprisonment of not more than 3 years, or both for knowing violations of the permit. In the case of a second or subsequent conviction for knowing violation, a person is subject to criminal penalties of not more than $100,000 per day of violation, or imprisonment of not more than 6 years, or both.

A3. Duty to Mitigate

The permittee must take all reasonable steps to minimize or prevent any discharge or sludge use or disposal in violation of this permit that has a reasonable likelihood of adversely affecting human health or the environment. In addition, upon request of DEQ, the permittee must correct any adverse impact on the environment or human health resulting from noncompliance with this permit, including such accelerated or additional monitoring as necessary to determine the nature and impact of the noncomplying discharge.

A4. Duty to Reapply

If the permittee wishes to continue an activity regulated by this permit after the expiration date of this permit, the permittee must apply for and have the permit renewed. The application must be submitted at least 180 days before the expiration date of this permit.

DEQ may grant permission to submit an application less than 180 days in advance but no later than the permit expiration date.

A5. Permit Actions

This permit may be modified, revoked and reissued, or terminated for cause including, but not limited to, the following:

a. Violation of any term, condition, or requirement of this permit, a rule, or a statute.
b. Obtaining this permit by misrepresentation or failure to disclose fully all material facts.
c. A change in any condition that requires either a temporary or permanent reduction or elimination of the authorized discharge.
d. The permittee is identified as a Designated Management Agency or allocated a wasteload under a total maximum daily load (TMDL).
e. New information or regulations.
f. Modification of compliance schedules.
g. Requirements of permit reopener conditions
h. Correction of technical mistakes made in determining permit conditions.
i. Determination that the permitted activity endangers human health or the environment.
j. Other causes as specified in 40 CFR §§ 122.62, 122.64, and 124.5.
k. For communities with combined sewer overflows (CSOs):
   (1) To comply with any state or federal law regulation for CSOs that is adopted or promulgated subsequent to the effective date of this permit.
   (2) If new information that was not available at the time of permit issuance indicates that CSO controls imposed under this permit have failed to ensure attainment of water quality standards, including protection of designated uses.
   (3) Resulting from implementation of the permittee's long-term control plan and/or permit conditions related to CSOs.

The filing of a request by the permittee for a permit modification, revocation or reissuance, termination, or a notification of planned changes or anticipated noncompliance does not stay any permit condition.

A6. Toxic Pollutants

The permittee must comply with any applicable effluent standards or prohibitions established under Oregon Administrative Rule (OAR) 340-041-0033 and section 307(a) of the federal Clean Water Act for toxic pollutants, and with standards for sewage sludge use or disposal established under section 405(d) of the federal Clean Water Act, within the time provided in the regulations that establish those standards or prohibitions, even if the permit has not yet been modified to incorporate the requirement.

A7. Property Rights and Other Legal Requirements

The issuance of this permit does not convey any property rights of any sort, or any exclusive privilege, or authorize any injury to persons or property or invasion of any other private rights, or any infringement of federal, tribal, state, or local laws or regulations.

A8. Permit References

Except for effluent standards or prohibitions established under section 307(a) of the federal Clean Water Act and OAR 340-041-0033 for toxic pollutants, and standards for sewage sludge use or disposal established under section 405(d) of the federal Clean Water Act, all rules and statutes referred to in this permit are those in effect on the date this permit is issued.

A9. Permit Fees

The permittee must pay the fees required by OAR.

SECTION B. OPERATION AND MAINTENANCE OF POLLUTION CONTROLS

B1. Proper Operation and Maintenance

The permittee must at all times properly operate and maintain all facilities and systems of treatment and control (and related appurtenances) that are installed or used by the permittee to achieve compliance with the conditions of this permit. Proper operation and maintenance also includes adequate laboratory controls and appropriate quality assurance procedures. This provision requires the operation of back-up or auxiliary facilities or similar systems that are installed by a permittee only when the operation is necessary to achieve compliance with the conditions of the permit.

B2. Need to Halt or Reduce Activity Not a Defense

For industrial or commercial facilities, upon reduction, loss, or failure of the treatment facility, the permittee must, to the extent necessary to maintain compliance with its permit, control production or all discharges or both until the facility is restored or an alternative method of treatment is provided. This requirement applies, for example, when the primary source of power of the treatment facility fails or is reduced or lost. It is not a defense for a permittee in an enforcement action that it would have been necessary to halt or reduce the permitted activity in order to maintain compliance with the conditions of this permit.

B3. Bypass of Treatment Facilities

a. Definitions
   (1) "Bypass" means intentional diversion of waste streams from any portion of the treatment facility. The permittee may allow any bypass to occur which does not cause effluent limitations to be exceeded, provided the diversion is to allow essential maintenance to assure efficient operation. These bypasses are not subject to the provisions of paragraphs b and c of this section.
   (2) "Severe property damage" means substantial physical damage to property, damage to the treatment facilities which causes them to become inoperable, or substantial and permanent loss of natural resources that can reasonably be expected to occur in the absence of a bypass. Severe property damage does not mean economic loss caused by delays in production.
b. Prohibition of bypass.
   (1) Bypass is prohibited and DEQ may take enforcement action against a permittee for bypass unless:
      i. Bypass was unavoidable to prevent loss of life, personal injury, or severe property damage;
      ii. There were no feasible alternatives to the bypass, such as the use of auxiliary treatment facilities, retention of untreated wastes, or maintenance during normal periods of equipment downtime. This condition is not satisfied if adequate backup equipment should have been installed in the exercise of reasonable engineering judgment to prevent a bypass that occurred during normal periods of equipment downtime or preventative maintenance; and
      iii. The permittee submitted notices and requests as required under General Condition B3.c.
   (2) DEQ may approve an anticipated bypass, after considering its adverse effects and any alternatives to bypassing, if DEQ determines that it will meet the three conditions listed above in General Condition B3.b.(1).
c. Notice and request for bypass.
   (1) Anticipated bypass. If the permittee knows in advance of the need for a bypass, a written notice must be submitted to DEQ at least ten days before the date of the bypass.
   (2) Unanticipated bypass. The permittee must submit notice of an unanticipated bypass as required in General Condition D5.

B4. Upset

a. Definition. "Upset" means an exceptional incident in which there is unintentional and temporary noncompliance with technology based permit effluent limitations because of factors beyond the reasonable control of the permittee. An upset does not include noncompliance to the extent caused by operation error, improperly designed treatment facilities, inadequate treatment facilities, lack of preventative maintenance, or careless or improper operation.

b. Effect of an upset. An upset constitutes an affirmative defense to an action brought for noncompliance with such technology-based permit effluent limitations if the requirements of General Condition B4.c are met. No determination made during administrative review of claims that noncompliance was caused by upset, and before an action for noncompliance, is final administrative action subject to judicial review.

c. Conditions necessary for a demonstration of upset. A permittee who wishes to establish the affirmative defense of upset must demonstrate, through properly signed, contemporaneous operating logs, or other relevant evidence that:
   (1) An upset occurred and that the permittee can identify the causes(s) of the upset;
   (2) The permitted facility was at the time being properly operated;
   (3) The permittee submitted notice of the upset as required in General Condition D5, hereof (24-hour notice); and
   (4) The permittee complied with any remedial measures required under General Condition A3 hereof.

d. Burden of proof. In any enforcement proceeding the permittee seeking to establish the occurrence of an upset has the burden of proof.

B5. Treatment of Single Operational Upset

For purposes of this permit, a single operational upset that leads to simultaneous violations of more than one pollutant parameter will be treated as a single violation. A single operational upset is an exceptional incident that causes simultaneous, unintentional, unknowing (not the result of a knowing act or omission), temporary noncompliance with more than one federal Clean Water Act effluent discharge pollutant parameter. A single operational upset does not include federal Clean Water Act violations involving discharge without a NPDES permit or noncompliance to the extent caused by improperly designed or inadequate treatment facilities. Each day of a single operational upset is a violation.

B6. Overflows from Wastewater Conveyance Systems and Associated Pump Stations

a. Definition. "Overflow" means any spill, release or diversion of sewage including:
   (1) An overflow that results in a discharge to waters of the United States; and
   (2) An overflow of wastewater, including a wastewater backup into a building (other than a backup caused solely by a blockage or other malfunction in a privately owned sewer or building lateral), even if that overflow does not reach waters of the United States.

b. Prohibition of overflows. Overflows are prohibited. DEQ may exercise enforcement discretion regarding overflow events. In exercising its enforcement discretion, DEQ may consider various factors, including the adequacy of the conveyance system's capacity and the magnitude, duration and return frequency of storm events.

c. Reporting required. All overflows must be reported orally to DEQ within 24 hours from the time the permittee becomes aware of the overflow. Reporting procedures are described in more detail in General Condition D5.

B7. Public Notification of Effluent Violation or Overflow

If effluent limitations specified in this permit are exceeded or an overflow occurs that threatens public health, the permittee must take such steps as are necessary to alert the public, health agencies and other affected entities (for example, public water systems) about the extent and nature of the discharge in accordance with the notification procedures developed under General Condition B8. Such steps may include, but are not limited to, posting of the river at access points and other places, news releases, and paid announcements on radio and television.

B8. Emergency Response and Public Notification Plan

The permittee must develop and implement an emergency response and public notification plan that identifies measures to protect public health from overflows, bypasses, or upsets that may endanger public health. At a minimum the plan must include mechanisms to:

a. Ensure that the permittee is aware (to the greatest extent possible) of such events;

b. Ensure notification of appropriate personnel and ensure that they are immediately dispatched for investigation and response;
c. Ensure immediate notification to the public, health agencies, and other affected public entities (including public water systems). The overflow response plan must identify the public health and other officials who will receive immediate notification;
d. Ensure that appropriate personnel are aware of and follow the plan and are appropriately trained;
e. Provide emergency operations; and
f. Ensure that DEQ is notified of the public notification steps taken.

B9. Removed Substances
Solids, sludges, filter backwash, or other pollutants removed in the course of treatment or control of wastewaters must be disposed of in such a manner as to prevent any pollutant from such materials from entering waters of the state, causing nuisance conditions, or creating a public health hazard.

SECTION C. MONITORING AND RECORDS

C1. Representative Sampling
Sampling and measurements taken as required herein must be representative of the volume and nature of the monitored discharge. All samples must be taken at the monitoring points specified in this permit, and must be taken, unless otherwise specified, before the effluent joins or is diluted by any other waste stream, body of water, or substance. Monitoring points must not be changed without notification to and the approval of DEQ.

C2. Flow Measurements
Appropriate flow measurement devices and methods consistent with accepted scientific practices must be selected and used to ensure the accuracy and reliability of measurements of the volume of monitored discharges. The devices must be installed, calibrated and maintained to insure that the accuracy of the measurements is consistent with the accepted capability of that type of device. Devices selected must be capable of measuring flows with a maximum deviation of less than ± 10 percent from true discharge rates throughout the range of expected discharge volumes.

C3. Monitoring Procedures
Monitoring must be conducted according to test procedures approved under 40 CFR part 136 or, in the case of sludge use and disposal, approved under 40 CFR part 503 unless other test procedures have been specified in this permit.

C4. Penalties of Tampering
The federal Clean Water Act provides that any person who falsifies, tampers with, or knowingly renders inaccurate any monitoring device or method required to be maintained under this permit may, upon conviction, be punished by a fine of not more than $10,000 per violation, imprisonment for not more than two years, or both. If a conviction of a person is for a violation committed after a first conviction of such person, punishment is a fine not more than $20,000 per day of violation, or by imprisonment of not more than four years, or both.

C5. Reporting of Monitoring Results
Monitoring results must be summarized each month on a discharge monitoring report form approved by DEQ. The reports must be submitted monthly and are to be mailed, delivered or otherwise transmitted by the 15th day of the following month unless specifically approved otherwise in Schedule B of this permit.

C6. Additional Monitoring by the Permittee
If the permittee monitors any pollutant more frequently than required by this permit, using test procedures approved under 40 CFR part 136 or, in the case of sludge use and disposal, approved under 40 CFR part 503, or as specified in this permit, the results of this monitoring must be included in the calculation and reporting of the data submitted in the discharge monitoring report. Such increased frequency must also be indicated. For a

pollutant parameter that may be sampled more than once per day (for example, total residual chlorine), only the average daily value must be recorded unless otherwise specified in this permit.

C7. Averaging of Measurements
Calculations for all limitations that require averaging of measurements must utilize an arithmetic mean, except for bacteria which must be averaged as specified in this permit.

C8. Retention of Records
Records of monitoring information required by this permit related to the permittee's sewage sludge use and disposal activities must be retained for a period of at least 5 years (or longer as required by 40 CFR part 503). Records of all monitoring information including all calibration and maintenance records, all original strip chart recordings for continuous monitoring instrumentation, copies of all reports required by this permit and records of all data used to complete the application for this permit must be retained for a period of at least 3 years from the date of the sample, measurement, report, or application. This period may be extended by request of DEQ at any time.

C9. Records Contents
Records of monitoring information must include:
a. The date, exact place, time, and methods of sampling or measurements;
b. The individual(s) who performed the sampling or measurements;
c. The date(s) analyses were performed;
d. The individual(s) who performed the analyses;
e. The analytical techniques or methods used; and
f. The results of such analyses.

C10. Inspection and Entry
The permittee must allow DEQ or EPA upon the presentation of credentials to:
a. Enter upon the permittee's premises where a regulated facility or activity is located or conducted, or where records must be kept under the conditions of this permit;
b. Have access to and copy, at reasonable times, any records that must be kept under the conditions of this permit;
c. Inspect at reasonable times any facilities, equipment (including monitoring and control equipment), practices, or operations regulated or required under this permit; and
d. Sample or monitor at reasonable times, for the purpose of assuring permit compliance or as otherwise authorized by state law, any substances or parameters at any location.

C11. Confidentiality of Information
Any information relating to this permit that is submitted to or obtained by DEQ is available to the public unless classified as confidential by the Director of DEQ under ORS 468.095. The permittee may request that information be classified as confidential if it is a trade secret as defined by that statute. The name and address of the permittee, permit applications, permits, effluent data, and information required by NPDES application forms under 40 CFR § 122.21 are not classified as confidential [40 CFR § 122.7(b)].

SECTION D. REPORTING REQUIREMENTS

D1. Planned Changes
The permittee must comply with OAR 340-052, "Review of Plans and Specifications" and 40 CFR § 122.41(l)(1). Except where exempted under OAR 340-052, no construction, installation, or modification involving disposal systems, treatment works, sewerage systems, or common sewers may be commenced until the plans and specifications are submitted to and approved by DEQ. The permittee must give notice to DEQ as soon as possible of any planned physical alternations or additions to the permitted facility.

D2. Anticipated Noncompliance
The permittee must give advance notice to DEQ of any planned changes in the permitted facility or activity that may result in noncompliance with permit requirements.

D3. Transfers
This permit may be transferred to a new permittee provided the transferee acquires a property interest in the permitted activity and agrees in writing to fully comply with all the terms and conditions of the permit and EQC rules. No permit may be transferred to a third party without prior written approval from DEQ. DEQ may require modification, revocation, and reissuance of the permit to change the name of the permittee and incorporate such other requirements as may be necessary under 40 CFR § 122.61. The permittee must notify DEQ when a transfer of property interest takes place.

D4. Compliance Schedule
Reports of compliance or noncompliance with, or any progress reports on interim and final requirements contained in any compliance schedule of this permit must be submitted no later than 14 days following each schedule date. Any reports of noncompliance must include the cause of noncompliance, any remedial actions taken, and the probability of meeting the next scheduled requirements.

D5. Twenty-Four Hour Reporting
The permittee must report any noncompliance that may endanger health or the environment. Any information must be provided orally (by telephone) to the DEQ regional office or Oregon Emergency Response System (1-800-452-0311) as specified below within 24 hours from the time the permittee becomes aware of the circumstances.

a. Overflows.
   (1) Oral Reporting within 24 hours.
      i. For overflows other than basement backups, the following information must be reported to the Oregon Emergency Response System (OERS) at 1-800-452-0311. For basement backups, this information should be reported directly to the DEQ regional office.
         (a) The location of the overflow;
         (b) The receiving water (if there is one);
         (c) An estimate of the volume of the overflow;
         (d) A description of the sewer system component from which the release occurred (for example, manhole, constructed overflow pipe, crack in pipe); and
         (e) The estimated date and time when the overflow began and stopped or will be stopped.
      ii. The following information must be reported to the DEQ regional office within 24 hours, or during normal business hours, whichever is earlier:
         (a) The OERS incident number (if applicable); and
         (b) A brief description of the event.
   (2) Written reporting within 5 days.
      i. The following information must be provided in writing to the DEQ regional office within 5 days of the time the permittee becomes aware of the overflow:
         (a) The OERS incident number (if applicable);
         (b) The cause or suspected cause of the overflow;
         (c) Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the overflow and a schedule of major milestones for those steps;
         (d) Steps taken or planned to mitigate the impact(s) of the overflow and a schedule of major milestones for those steps; and
         (e) For storm-related overflows, the rainfall intensity (inches/hour) and duration of the storm associated with the overflow.

      DEQ may waive the written report on a case-by-case basis if the oral report has been received within 24 hours.

b. Other instances of noncompliance.
   (1) The following instances of noncompliance must be reported:
      i. Any unanticipated bypass that exceeds any effluent limitation in this permit;
      ii. Any upset that exceeds any effluent limitation in this permit;
      iii. Violation of maximum daily discharge limitation for any of the pollutants listed by DEQ in this permit; and
      iv. Any noncompliance that may endanger human health or the environment.
   (2) During normal business hours, the DEQ regional office must be called. Outside of normal business hours, DEQ must be contacted at 1-800-452-0311 (Oregon Emergency Response System).
   (3) A written submission must be provided within 5 days of the time the permittee becomes aware of the circumstances. The written submission must contain:
      i. A description of the noncompliance and its cause;
      ii. The period of noncompliance, including exact dates and times;
      iii. The estimated time noncompliance is expected to continue if it has not been corrected;
      iv. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance; and
      v. Public notification steps taken, pursuant to General Condition B7.
   (4) DEQ may waive the written report on a case-by-case basis if the oral report has been received within 24 hours.

D6. Other Noncompliance
The permittee must report all instances of noncompliance not reported under General Condition D4 or D5 at the time monitoring reports are submitted. The reports must contain:
a. A description of the noncompliance and its cause;
b. The period of noncompliance, including exact dates and times;
c. The estimated time noncompliance is expected to continue if it has not been corrected; and
d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance.

D7. Duty to Provide Information
The permittee must furnish to DEQ within a reasonable time any information that DEQ may request to determine compliance with the permit or to determine whether cause exists for modifying, revoking and reissuing, or terminating this permit. The permittee must also furnish to DEQ, upon request, copies of records required to be kept by this permit.

Other Information: When the permittee becomes aware that it has failed to submit any relevant facts or has submitted incorrect information in a permit application or any report to DEQ, it must promptly submit such facts or information.

D8. Signatory Requirements
All applications, reports or information submitted to DEQ must be signed and certified in accordance with 40 CFR § 122.22.

D9. Falsification of Information
Under ORS 468.953, any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit, including monitoring reports or reports of compliance or noncompliance, is subject to a Class C felony punishable by a fine not to exceed $125,000 per violation and up to 5 years in prison per ORS chapter 161. Additionally, according to 40 CFR § 122.41(k)(2), any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit including monitoring reports or reports of compliance or non-compliance will, upon conviction, be punished by a federal civil penalty not to exceed $10,000 per violation, or by imprisonment for not more than 6 months per violation, or by both.

D10. Changes to Indirect Dischargers

The permittee must provide adequate notice to DEQ of the following:

a. Any new introduction of pollutants into the POTW from an indirect discharger which would be subject to section 301 or 306 of the federal Clean Water Act if it were directly discharging those pollutants and;
b. Any substantial change in the volume or character of pollutants being introduced into the POTW by a source introducing pollutants into the POTW at the time of issuance of the permit.
c. For the purposes of this paragraph, adequate notice must include information on (i) the quality and quantity of effluent introduced into the POTW, and (ii) any anticipated impact of the change on the quantity or quality of effluent to be discharged from the POTW.

SECTION E. DEFINITIONS

E1. *BOD* or *BOD_5* means five-day biochemical oxygen demand.
E2. *CBOD* or *$CBOD_5$* means five-day carbonaceous biochemical oxygen demand.
E3. *TSS* means total suspended solids.
E4. *Bacteria* means but is not limited to fecal coliform bacteria, total coliform bacteria, *Escherichia coli* (*E. coli*) bacteria, and *Enterococcus* bacteria.
E5. *FC* means fecal coliform bacteria.
E6. *Total residual chlorine* means combined chlorine forms plus free residual chlorine
E7. *Technology based permit effluent limitations* means technology-based treatment requirements as defined in 40 CFR § 125.3, and concentration and mass load effluent limitations that are based on minimum design criteria specified in OAR 340-041.
E8. *mg/l* means milligrams per liter.
E9. *µg/l* means microgram per liter.
E10. *kg* means kilograms.
E11. *m^3/d* means cubic meters per day.
E12. *MGD* means million gallons per day.
E13. *Average monthly effluent limitation* as defined at 40 CFR § 122.2 means the highest allowable average of daily discharges over a calendar month, calculated as the sum of all daily discharges measured during a calendar month divided by the number of daily discharges measured during that month.
E14. *Average weekly effluent limitation* as defined at 40 CFR § 122.2 means the highest allowable average of daily discharges over a calendar week, calculated as the sum of all daily discharges measured during a calendar week divided by the number of daily discharges measured during that week.
E15. *Daily discharge* as defined at 40 CFR § 122.2 means the discharge of a pollutant measured during a calendar day or any 24-hour period that reasonably represents the calendar day for purposes of sampling. For pollutants with limitations expressed in units of mass, the daily discharge must be calculated as the total mass of the pollutant discharged over the day. For pollutants with limitations expressed in other units of measurement, the daily discharge must be calculated as the average measurement of the pollutant over the day.
E16. *24-hour composite sample* means a sample formed by collecting and mixing discrete samples taken periodically and based on time or flow. The sample must be collected and stored in accordance with 40 CFR part 136.
E17. *Grab sample* means an individual discrete sample collected over a period of time not to exceed 15 minutes.
E18. *Quarter* means January through March, April through June, July through September, or October through December.
E19. *Month* means calendar month.
E20. *Week* means a calendar week of Sunday through Saturday.
E21. *POTW* means a publicly-owned treatment works.

APPENDIX B

COLLECTION SYSTEM MAP

APPENDIX C

COST ESTIMATES

Collection System Improvements Cost Estimates

Newton Creek Pump Station Improvements Cost Estimate

Marys River Outfall Diffuser Cost Estimates

Lagoon Aeration and Headworks Screening Improvements Cost Estimate

Land Application System Expansion Cost Estimate

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SEPTEMBER, 2016

City of Philomath, Oregon

Sanitary Sewer System Collection Map

10/24/2017 9:52:07 AM
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Recommened Collection System Improvements Cost Estimates

Philomath Wastewater System Facilities Plan
10/24/2017

Priority Ranking
1= priority 1
2= priority 2
3= priority 3

Project Code	Priority	Project & Location(s)	Size/capacity	Length (ft)	Pipe Cost ($/ft)	Manholes #	Manhole Cost[1] ($)	Service Laterals #	Service Lateral Cost [2] ($)	Other [3,4,5,6,7] Costs	Construction Cost	10% Construction Contingency	20% Engineering	10% Legal, Permits, Easement, Admin	Total Project	Rounded Total	Total project costs rounded to nearest $1000 Prior 1	Prior 2	Prior 3
Gravity Collection System Improvements																			
G-1	1	9th Street to 7th Street Sewer Lines – Manhole #35 to Manhole #184	10 inch	1450	$ 140.00	5	$ 30,000.00	17	$ 51,000.00		$ 284,000.00	$ 28,400.00	$ 56,800.00	$ 28,400.00	$ 397,600.00	$ 398,000	$398,000	$0	$0
G-2	1	10th Street Sewer Lines – Manhole #34 to Manhole #45	12 inch	400	$ 150.00	2	$ 12,000.00	6	$ 18,000.00		$ 90,000.00	$ 9,000.00	$ 18,000.00	$ 9,000.00	$ 126,000.00	$ 126,000	$126,000	$0	$0
G-3	1	Main Street Sewer Lines – Manhole #45 to Manhole #52	12 inch	400	$ 150.00	2	$ 12,000.00	12	$ 36,000.00		$ 164,000.00	$ 16,400.00	$ 32,800.00	$ 16,400.00	$ 229,600.00	$ 230,000	$230,000	$0	$0
			10 inch	400	$ 140.00														
G-4	1	8th & College Street Sewer Lines – Manhole #52 to Manhole #56	10 inch	750	$ 140.00	2	$ 12,000.00	6	$ 18,000.00		$ 135,000.00	$ 13,500.00	$ 27,000.00	$ 13,500.00	$ 189,000.00	$ 189,000	$189,000	$0	$0
G-5	2	Pioneer and 11th Street Sewer Lines – Manhole #71 to Manhole #74	10 inch	800	$ 140.00	3	$ 18,000.00	13	$ 39,000.00		$ 169,000.00	$ 16,900.00	$ 33,800.00	$ 16,900.00	$ 236,600.00	$ 237,000	$0	$237,000	$0
G-6	2	15th Street Trunk Sewer (South) – Manhole #27 to Manhole #288	12 inch	1650	$ 150.00	3	$ 18,000.00	33	$ 99,000.00		$ 364,500.00	$ 36,450.00	$ 72,900.00	$ 36,450.00	$ 510,300.00	$ 510,000	$0	$510,000	$0
G-7	2	15th Street Trunk Sewer (North) – Manhole #288 to Manhole #94	12 inch	350	$ 150.00	1	$ 6,000.00	8	$ 24,000.00		$ 82,500.00	$ 8,250.00	$ 16,500.00	$ 8,250.00	$ 115,500.00	$ 116,000	$0	$116,000	$0
G-8	1	Applegate Street and 20th Street Trunk Sewer - Manhole #1 to Manhole #6	12 inch	1200	$ 150.00	4	$ 24,000.00	14	$ 42,000.00		$ 246,000.00	$ 24,600.00	$ 49,200.00	$ 24,600.00	$ 344,400.00	$ 344,000	$344,000	$0	$0
G-9	3	Newton Creek Trunk Sewer – Newton Creek Pump Station to Manhole 476	24 inch	2650	$ 190.00	7	$ 42,000.00		$ -		$ 545,500.00	$ 54,550.00	$ 109,100.00	$ 54,550.00	$ 763,700.00	$ 764,000	$0	$0	$764,000
G-10	3	19th Street Trunk Sewer South	24 inch	2700	$ 190.00	7	$ 42,000.00		$ -	$ 100,000.00	$ 655,000.00	$ 65,500.00	$ 131,000.00	$ 65,500.00	$ 917,000.00	$ 917,000	$0	$0	$917,000
G-11	3	Railroad Trunk Sewer	18 inch	3200	$ 180.00	8	$ 48,000.00		$ -	$ 100,000.00	$ 724,000.00	$ 72,400.00	$ 144,800.00	$ 72,400.00	$ 1,013,600.00	$ 1,014,000	$0	$0	$1,014,000
G-12	3	19th Street/Green Road Trunk Sewer	21 inch	1000	$ 190.00	13	$ 78,000.00		$ -		$ 908,000.00	$ 90,800.00	$ 181,600.00	$ 90,800.00	$ 1,271,200.00	$ 1,271,000	$0	$0	$1,271,000
			15 inch	2000	$ 170.00														
			12 inch	2000	$ 150.00														
G-13	3	Industrial Way Trunk Sewer	12 inch	2800	$ 150.00	7	$ 42,000.00				$ 462,000.00	$ 46,200.00	$ 92,400.00	$ 46,200.00	$ 866,000.00	$ 866,000	$0	$0	$866,000
G-14	3	Sewer Basin N5 Trunk Sewer	15 inch	2400	$ 170.00	6	$ 36,000.00				$ 444,000.00	$ 44,400.00	$ 88,800.00	$ 44,400.00	$ 621,600.00	$ 622,000	$0	$0	$622,000
G-15	3	Chapel Drive Trunk Sewer	10 inch	4200	$ 140.00	11	$ 66,000.00			$ 100,000.00	$ 754,000.00	$ 75,400.00	$ 150,800.00	$ 75,400.00	$ 1,055,600.00	$ 1,056,000	$0	$0	$1,056,000
G-16	1	Timber Estates Trunk Sewer	8 inch	1800	$ 130.00	5	$ 30,000.00				$ 264,000.00	$ 26,400.00	$ 52,800.00	$ 26,400.00	$ 369,600.00	$ 370,000	$370,000	$0	$0
Pump Station and Forcemain Improvements																			
P-1	3	Basin P1 Pump Station and Forcemain	0.62 MGD/6-inch	1300	$ 100.00					$ 400,000.00	$ 530,000.00				$ 530,000.00	$ 530,000	$0	$0	$530,000
P-2	3	Basin P2 Pump Station and Forcemain	0.42 MGD/6-inch	1000	$ 100.00					$ 400,000.00	$ 500,000.00				$ 500,000.00	$ 500,000	$0	$0	$500,000
P-3	2	Newton Creek Pump Station Improvements					See detailed estimate for Newton Creek Pump Station Improvements								$ 1,479,000.00	$ 1,479,000	$0	$1,479,000	$0
F-1	1	Newton Creek Forcemain	18 inch	4100	$ 190.00		$ -		$ -	$ 250,000.00	$ 1,029,000.00	$ 102,900.00	$ 205,800.00	$ 102,900.00	$ 1,440,600.00	$ 1,441,000	$1,441,000	$0	$0
															Totals	$ 12,980,000	$ 3,098,000	$ 2,342,000	$ 7,540,000

Notes
1) Unit cost for manholes = $6,000 each
2) Unit cost for service laterals = $3,000 each
3) Other costs include 100 feet of bored railroad crossing for project G-10.
4) Other costs include 100 feet of bored stream crossing for project G-11.
5) Other costs include 100 feet of bored stream crossing for project G-15.
6) Other costs include a pump station a for projects P-1 & P-2.
7) Other costs include 250 feet of bored stream crossing for project F-1.

Philomath Wastewater System Facilities Plan

Newton Creek Pump Station Improvements (Project P-3)

Item	Qty	Unit	Unit Cost	Total Cost
Mobilization (percentage of total)	8.0%	LS	$71,000	$71,000
Bypass Pumping	1	LS	$80,000	$80,000
Site Work				
Baserock & AC Paving	1	LS	$7,500	$7,500
Bollards	4	EA	$750	$3,000
Influent Piping Modificaitons	70	LF	$250	$17,500
Manholes	2	EA	$8,000	$16,000
Erosion Control	1	LS	$2,500	$2,500
New Entrance Gate	1	LS	$10,000	$10,000
New Site Fencing	200	LF	$12	$2,400
Forcemain Piping	200	LF	$90	$18,000
Forcemain Isolation Valves	2	EA	$5,000	$10,000
Connections to Existing Forcemain Pipes	2	EA	$5,000	$10,000
Washdown water piping	50	LF	$40	$2,000
Washdown water stations	1	EA	$1,000	$1,000
Compaction & Materials Testing	1	LS	$2,500	$2,500
Wet Well & Valve Vault Improvements				
New Hatches	100	SF	$110	$11,000
New Top Slab	10	CY	$1,000	$10,000
Wet Well Baffling	2	CY	$1,000	$2,000
Miscellaneous Mechanical Equipment	1	LS	$10,000	$10,000
Submersible Sewage Pumps & Appurtenances	1	LS	$235,000	$235,000
Wetwell & Valve Vault Piping and Appurtenances	1	LS	$150,000	$150,000
Wet Well coring for New Pipe Penetrations	1	LS	$10,000	$10,000
Sluice Gate	1	LS	$6,500	$6,500
Valve Vault Structure	2	EA	$15,000	$30,000
Flow Meter Vault	1	EA	$15,000	$15,000
Flow Meter & Vault Piping	1	LS	$20,000	$20,000
Electrical & Controls				
Power Service, Complete	1	LS	$40,000	$40,000
Pump Control Panel	1	LS	$35,000	$35,000
Variable Frequency Drives	3	EA	$20,000	$60,000
Generator, Slab, Tank, Sound Attenuations	1	LS	$150,000	$150,000
Misc Electrical & Controls (8% of Total Cost)	1	LS	$83,000	$83,000
Construction Total				**$1,121,000**
Soft Costs				
Construction Contingencies	10%	LS	$112,000	$ 112,000
Engineering, Legal, & Admin	20%	LS	$224,000	$ 224,000
Permitting	2%	LS	$22,000	$ 22,000
Total Project Budget				**$1,479,000**

Philomath Wastewater System Facilities Plan

New Marys River Outfall Diffuser Planning Level Cost Estimate (Project T-1)

Construction Costs				
Item	**Qty**	**Unit**	**Unit Cost**	**Total Cost**
Mobilization (percentage of total)	10.0%	LS	$8,800	$8,800
Connection to existing outfall structure	1	LS	$6,000	$6,000
New Stop Logs for outfall structure	1	LS	$2,000	$2,000
Oufall piping installed	1	LS	$20,000	$20,000
Instream Work Isolation	1	LS	$20,000	$20,000
Diffuser installed	1	LS	$25,000	$25,000
Rip rap and surface restoration	1	LS	$15,000	$15,000
Construction Total				$97,000
Soft Costs				
Construction Contingencies	10%	LS	$10,000	$ 10,000
Engineering, Legal, & Admin	20%	LS	$19,000	$ 19,000
Environmental Permitting	1	LS	$35,000	$ 35,000
Mixing Zone Study	1	LS	$12,000	$ 12,000
Total Project Budget				**$173,000**

Philomath Wastewater System Facilities Plan

Lagoon Aeration and Headworks Screening Improvements (Project T-2)

Item	Qty	Unit	Unit Cost	Total Cost
Mobilization (percentage of total)	8.0%	LS	$159,400	$159,400
Lagoon Aeration System				
Blower Building, Complete	480	SF	$220	$105,600
Aeration Header Piping	3000	LF	$70	$210,000
Power Service Modifications	1	LS	$20,000	$20,000
Diffused Aeration Equipment	1	LS	$1,060,000	$1,060,000
Aeration Equipment Installation (@ 20%)	1	LS	$212,000	$212,000
Headworks Screening Improvements				
Core Existing Headworks Wall	1	LS	$2,000	$2,000
Clearing & Grubbing	1	LS	$2,500	$2,500
Baserock	30	CY	$40	$1,200
Gravel Surfacing	175	SY	$15	$2,600
Bollards	2	EA	$1,000	$2,000
Piping				
New Forceain Discharge Piping	15	LF	$150	$2,300
Permanent bypass pipe	90	LF	$180	$16,200
Connections to existing pipes	2	LS	$3,000	$6,000
Washdown water piping	50	LF	$40	$2,000
Washdown water stations	1	EA	$1,000	$1,000
Concrete				
Foundations & Footings	7	CY	$1,000	$7,000
Walls	15	CY	$1,000	$15,000
Slabs on Grade	5	CY	$750	$3,800
Slide Gates	3	EA	$3,000	$9,000
Sluice Gates	1	EA	$5,000	$5,000
Manual Bar Screen	1	EA	$3,000	$3,000
Misc. Mechanical	1	LS	$25,000	$25,000
Handrails	60	LF	$75	$4,500
Grating & Frame	120	SF	$60	$7,200
Fine Screening Equipment	1	LS	$100,000	$100,000
Fine Screen Installation (20% of Equip. Cost)	1	LS	$20,000	$20,000
Misc Electrical & Controls (8% of Total Cost)	1	LS	$148,000	$148,000
Construction Total				**$2,152,000**
Soft Costs				
Construction Contingencies	10%	LS	$215,000	$ 215,000
Engineering, Legal, & Admin	20%	LS	$430,000	$ 430,000
Permitting	2%	LS	$43,000	$ 43,000
Total Project Budget				**$2,840,000**

Philomath Wastewater System Facilities Plan

Land Application System Expansion (Project T-3)

Construction Costs				
Item	**Qty**	**Unit**	**Unit Cost**	**Total Cost**
Mobilization (percentage of total)	10.0%	LS	$26,300	$26,300
Connection to existing system	1	LS	$1,500	$1,500
New 16-inch irrigation piping, installed	3020	LF	$70	$211,400
Bellfountain Road Auger Bore	70	LF	$500	$35,000
Miscellanous Valve and Irrigation Risers	1	LS	$15,000	$15,000
Construction Total				$289,000
Soft Costs				
Construction Contingencies	10%	LS	$29,000	$ 29,000
Engineering, Legal, & Admin	20%	LS	$58,000	$ 58,000
Permitting	2%	LS	$6,000	$ 6,000
Recyled Water Use Plan Revisions	1	LS	$12,000	$ 12,000
Total Project Budget				**$394,000**

APPENDIX D

CAPITAL IMPROVEMENT PRIORITIES MAP

City of
Philomath, Oregon
Sanitary Sewer System
CAPITAL IMPROVEMENT PRIORITIES
LEGEND
Urban Growth Boundary
City Limits
Existing Sanitary Sewer Line
Existing SS Forcemain & Pump Station
Existing Manhole (manhole number)
Existing Cleanout (cleanout number)
IMPROVEMENT PROJECTS
12" G13 PIPE SIZE, PROJECT CODE
PRIORITY 1 PROJECTS
PRIORITY 2 PROJECTS
PRIORITY 3 PROJECTS
NOTE:
THESE MAPS ARE SCHEMATIC UTILITY MAPS ONLY & DO NOT SHOW EXACT LOCATIONS OF UTILITIES. FIELD VERIFY ALL LOCATIONS PRIOR TO DESIGN OR CONSTUCTION.
0 200 400 800
FEET
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OCTOBER, 2017
Marys
River
Newton Creek
Newton Ck
E Fork
WWTP
LAGOONS
PROPOSED
BASIN P1
PUMP STATION
PROPOSED
BASIN P2
PUMP STATION
PUMP STATION "A"
TIMBER ESTATES
PUMP STATION
NEWTON CREEK
PUMP STATION
P1
P2
P3
G1
G2
G3
G4
G5
G6
G7
G8
G9
G10
G11
G12
G13
G14
G15
G16
F1
T1
T2
T3