2015 GENERAL SEWER PLAN UPDATE

April 2016

Prepared by

J-U-B ENGINEERS, Inc.
2810 W. Clearwater Ave. Ste. 201
Kennewick, WA 99336

2015 GENERAL SEWER PLAN UPDATE

April 2016

Certification

This 2015 General Sewer Plan Update (with exception of Chapter's 5 and 8) for the City of Richland has been prepared under the direction of the following Registered Professional Engineers. In compliance with the Washington Department of Ecology Requirements for General Sewer Plans, WAC 173-240-050.

Alex J. Fazzari, P.E.
J-U-B ENGINEERS, INC.
2810 W. Clearwater Ave., Ste 201
Kennewick, WA 99336
(509) 783-2144

2015 GENERAL SEWER PLAN UPDATE

April 2016

Certification

Chapter 5 of this 2015 General Sewer Plan Update for the City of Richland has been prepared under the direction of the following Registered Professional Engineer. In compliance with the Washington Department of Ecology Requirements for General Sewer Plans, WAC 173-240-050.

Alan J. Straub, P.E.
Carollo Engineers, Inc.
1218 Third Ave., Ste 100
Seattle, WA 98901
(206) 684-6532

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List of Commonly Used Abbreviations

AC	Acre
BFHD	Benton Franklin Health District
CFS	Cubic Foot per Second
CIP	Capital Improvement Plan
d/D	Depth Over Diameter
DU	Dwelling Unit
ERU	Equivalent Residential Unit
FEMA	Flood & Emergency Management Agency
FOG	Fats, Oils, & Grease
FT	Feet
GIS	Geographical Information System
GMA	Growth Management Act
GPAD	Gallons Per Acre per Day
GPDU	Gallons Per Dwelling Unit
GPM	Gallons Per Minute
HP	Horsepower
I&I	Infiltration and/or Inflow
IN	Inches
J-U-B	J-U-B ENGINEERS, Inc.
RMC	Richland Municipal Code
LS	Lift Station
MH	Manhole
MHID	Manhole Identification Number
MGD	Million Gallons per Day
OFM	Washington State Office of Financial Management
ROW	Right-of-Way
SHPO	State of Washington Historical Preservation Office
UGA	Urban Growth Area
USGS	United States Geologic Survey
WAC	Washington Administrative Code
WDOH	Washington State Department of Health
WDOE	Washington State Department of Ecology
WWTP	Wastewater Treatment Plant

EXECUTIVE SUMMARY

ES-1 Purpose

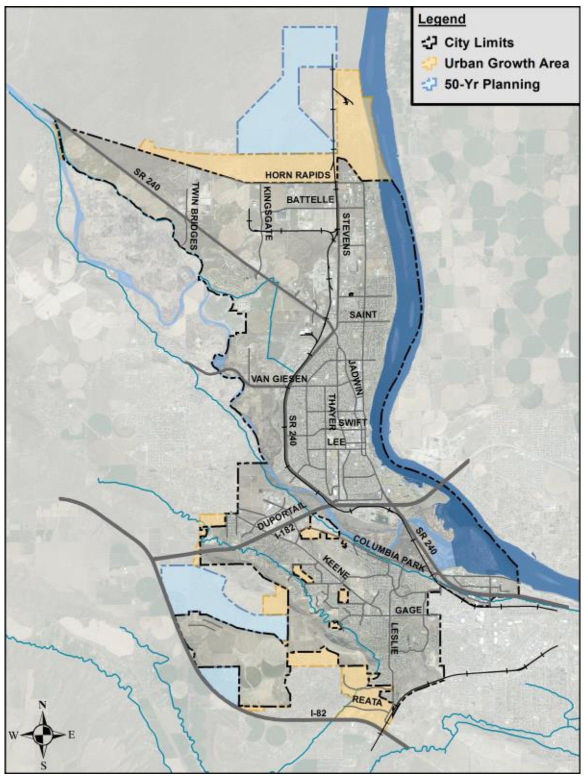
In accordance with WAC 173-240-020(7), the City of Richland (City) maintains a General Sewer Plan which has been reviewed and approved by the Washington State Department of Ecology (WDOE). Long term planning should be reviewed and periodically updated to incorporate changes in population, land use, and regulations. It is recommended that updates occur at 5-10 year intervals. The last comprehensive General Sewer Plan for the City was completed in 2004. The City has experienced significant growth since then and much of the 2004 plan needs updating. The City authorized J-U-B ENGINEERS, Inc. to undertake a General Sewer Plan Update in 2014/2015. The major goals of the 2015 General Sewer Plan Update are as follows:

- Provide a general evaluation of the Wastewater Treatment Plant (WWTP)
- Update the hydraulic model of the sewer collection system to assess the existing conditions (current flows), near-term conditions (areas the City has committed to serve that may be developed soon), and long-term conditions (areas beyond the current City limits to the expected 50-year boundary)
- Identify limitations in the existing collection system and necessary improvements to maintain an appropriate level of service
- Incorporate recent analysis from the South Sewer Study and summarize the history and current plan for providing sewer service to the Badger Mountain Sub-Area
- Update the collection system master plan to serve the expected 50-year boundary
- Develop "Risk of Failure" ratings that incorporate sewer pipe condition data in order to prioritize improvement projects.
- Develop "Consequence of Failure" ratings for sewer pipes in order to further prioritize improvement projects.
- Develop overall scoring criteria for sewer pipes utilizing hydraulics, "Risk of Failure," and "Consequence of Failure" criteria such that City Staff can combine this data with separate scoring of water pipes and roadways in order to identify and prioritize infrastructure projects.
- Establish a comprehensive Capital Improvement Plan (CIP) with particular emphasis on the next 5 to 10 years
- Document the sewer utility's financial condition and assess its ability to support the recommendations of the CIP.
- Summarize the City's current Operations & Maintenance Program and suggest potential changes.
- Summarize the City's current Pre-Treatment Program and develop a framework for a Fats, Oils, & Grease (FOG) program.
- Satisfy WDOE and WAC requirements for a General Sewer Plan.

ES-2 Planning Boundaries

This General Sewer Plan evaluates the hydraulic capacity of all of the existing sewer pipes that are 10-inches and larger in diameter. The pipes are evaluated not only on existing flow conditions, but the expected flow conditions when the entire Urban Growth Boundary is completely developed. Any existing pipes that were identified as needing to be upsized upon buildout of the UGA, were further evaluated to serve a 50-year boundary – with the goal in mind that any pipes constructed today will have the capacity to function properly through the end of their design life. Similarly, any new pipe extensions were also sized to serve the 50-year boundary. The planning boundaries are depicted in **Figure ES-1**.

Figure ES-1 – Planning Boundaries



ES-3 Collection System Summary

The City's public collection system has expanded from an initial series of pipelines serving the old downtown Richland area to a system containing over 262 miles of gravity pipelines and 14 pumping stations providing public sewer service to a residential population of 53,054. The total area that can be provided with public sewer service totals over 25,000 acres or approximately 40 square miles. The total linear feet of sewer pipelines within the City's public collection system has more than tripled over the past 30 years. The existing wastewater collection system consists of gravity pipelines ranging in size from 6 inches in diameter up to 54 inches in diameter.

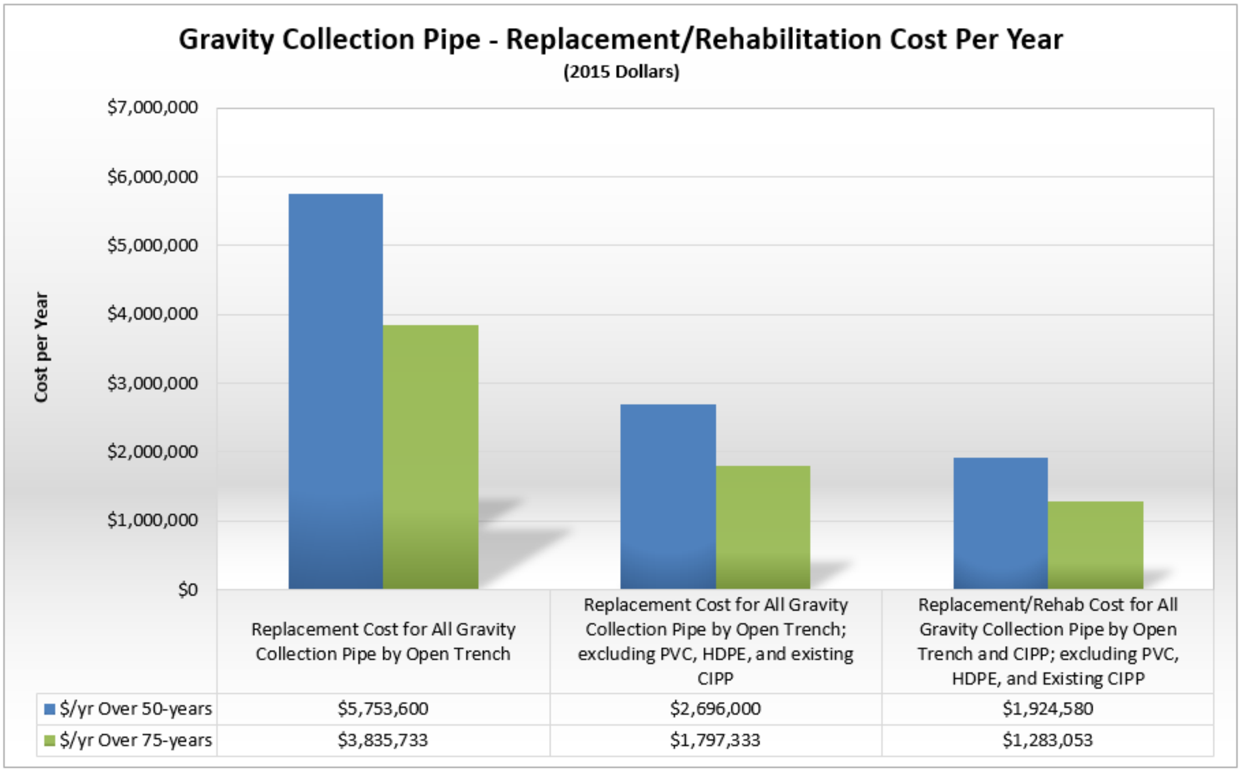
Overall, the existing collection system has adequate capacity to convey current flows through master plan flows as the CIP is implemented. This is evidenced by the relatively few capacity issues within the existing system compared to necessary upgrades to accommodate growth beyond the City's current service limits.

The hydraulic model used in this analysis was created based on land use and zoning conditions at the time of the study, both of which will change over time. Since the models are based on these parameters, it is critical to keep them updated over time to reflect up-to-date conditions. The General Sewer Plan will therefore require periodic updates to remain a current, accurate, and applicable tool in future evaluations. As part of this ongoing maintenance, the Wastewater Utility currently plans to update the Master Plan Model every five to ten years with the assistance of a consultant. Updates may be implemented more frequently if there are significant changes to land use, impact area, collection system, or the rate of development.

Although the hydraulic analysis indicated relatively few capacity issues, the collection system is showing its age and a proactive renewal and replacement program has been developed to address this. A significant effort of this plan was spent prioritizing pipes for replacement and developing a CIP.

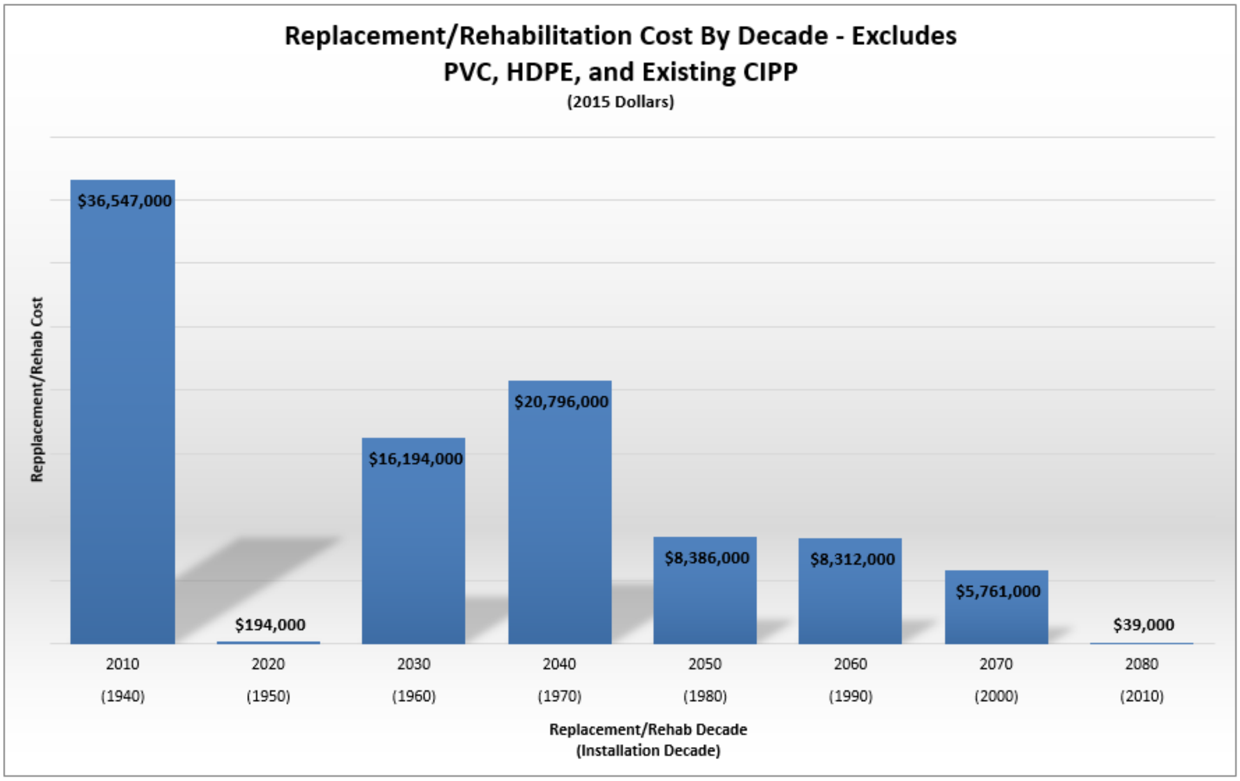
Prioritizing pipes for replacement involves determining which are more likely to fail. For this analysis, the prioritization focused on the City's non-PVC pipe inventory and its useful life. This recommendation assumes that the non-PVC pipe that has not yet been rehabilitated can be rehabilitated/replaced every 75 years with a mixture of trenched replacement and trenchless rehabilitation. This analysis assumes that PVC pipes and any pipes that have recently been rehabilitated will not have to be rehabilitated in the next 75 years. Based on this approach, the City should be budgeting approximately $1.5 million dollars per year (2015 dollars) for collection system rehabilitation/replacement. A summary of the cost of the various replacement scenarios is depicted in **Figure ES-2**.

Figure ES-2 – Collection System Replacement Cost Analysis



It is worth noting that the above analysis does not take into account the age of the existing pipes. The City has limited data on pipe age; however, an estimate of pipe installation by the decade was developed in order to identify the potential timing of replacement. **Figure ES-3** depicts potential cost of replacement per decade for the next several decades. This assumes a 75-year lifespan for the non-PVC pipe that has not yet been rehabilitated. Because a significant portion of the City was constructed in the 1940s, replacement of a large portion of the City is likely required soon. The City has been aggressively rehabilitating approximately 130,000 LF pipe since 1997 – nonetheless, there is still a significant portion of the aged system remaining. This emphasizes the need for immediate CCTV inspection and condition rating of the system in order to verify if the pipes are in fact near the end of their service life.

Figure ES 3 – Potential Timing of System Replacement Costs



A pipe replacement program was developed to prioritize sewer pipes with the greatest need for replacement each budget year. The prioritization method is composed of two main categories: likelihood of failure (pipe condition) and consequence of failure (risk). The City maintains only a limited amount of data regarding the existing pipes in the system; therefore, several assumptions were made using the existing data as best as possible. Through workshops with City staff, each category and associated criteria were assigned a weighting value to reflect relative importance. These weights are easily modified and will likely be adjusted and fine-tuned over time as the City implements the replacement and rehabilitation program.

Through the development of the pipe scoring criteria, it became evident that the lack of condition rating for the existing pipes was a key piece of information that was missing. Therefore, the CIP includes an intensive survey of the existing pipes in order to determine condition ratings over the course of approximately three years and at a cost of approximately $0.5 million per year. Once this data is acquired, the City will then be able to update the scoring criteria and re-prioritize replacement projects to determine which projects to focus on for annual renewals/replacements.

ES-4 Capital Improvement Plan Summary

The CIP identifies and describes the improvements necessary to provide service to the future wastewater service area at a suitable level of service and reserve capacity. It also provides an approximate timeline for implementation of these projects. **Table ES-1** lists the CIP projects with recommended action. **Figure A14** shows the location and type of each project in the CIP. **Appendix I** contains a project summary and associated capital cost for each CIP project. Projects are categorized as follows:

- Capacity Projects: Required to relieve insufficient hydraulic capacity of existing pipes in the near future; funded by connection fees
- System Expansion: Required to serve new areas within the UGA; funded by connection fees
- Collection System Improvements: Required to upgrade existing pipes and lift stations; funded by a mix of connection fees and rates
- Rehabilitation/Replacement: Required to maintain the integrity of the existing system; funded by rates
- WWTP Improvements: Required to improve capacity maintain the integrity of the existing system; funded by a mix of connection fees and rates
- WWTP Rehabilitation and Replacement: Required to maintain the integrity of the existing system; funded by rates
- Developer Driven Projects: Required to expand the collection system within the UGA but timing is unknown; driven by development.

Table ES-1 – CIP Projects

ID	Description/System Name	Recommend Action	Timeframe and Capital Cost										
			2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	With Growth[1]
Capacity Projects – Funded by Connection Fees													
CP.1	Leslie Rd Trunk Replacement	Replace 18-inch bottleneck section											$329,000
CP.2	Keene Rd Collector Replacement	Replace 10-inch bottleneck section							$329,000				
CP.3	Upper North Interceptor Improvements	New lift station and piping to address neighborhood surcharging										$2,238,000	
CP.4	Bellerive LS Pump Upgrade & Downstream Improvements	New lift station pumps and downstream pipe replacement to address surcharging										$1,785,000	
System Expansion – Funded by Connection Fees													
SE.1	Leslie Interceptor Extension	Collection system expansion to extend utility service	$800,000										
Collection System Improvements – Funded by a split of Connection Fees and Rates													
CS.1	Montana Lift Station Standby Generator	Generator installation to operate lift station during power outages	$40,000										
CS.2	Columbia Lift Station Standby Generator	Generator installation to operate lift station during power outages	$25,000										
CS.3	Waterfront Lift Station Replacement	Replace deficient lift station			$608,000								
Rehabilitation and Replacement Projects – Funded by Rates													
RR.1	Renewals and Replacement	10-yr rehabilitation and replacement program based on Condition Assessment	$250,000	$258,000	$1,599,000[2]	$1,652,000[2]	$1,705,000[2]	$1,761,000	$1,818,000	$1,878,000	$1,939,000	$2,002,000	
RR.2	Annual Street Overlay Areas	Annual repair and replacement of sewer deficiencies in areas scheduled for re-paving	$100,000	$103,000	$107,000	$110,000	$114,000	$117,000	$121,000	$125,000	$129,000	$133,000	
RR.3	Infiltration and Inflow Study								$200,000				

ID	Description/System Name	Recommend Action	Timeframe and Capital Cost										
			2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	With Growth[1]
WWTP Improvements – Funded by Rates/Connection Fees													
WWTP. 1	Influent Upgrades	Influent Upgrades			$2,133,000								
WWTP. 2	Engineering Report	Re-Rating Study for Design Criteria						$411,000					
WWTP Rehabilitation and Replacement – Funded by Rates													
WWTP. RR.1	WWTP Renewals and Replacements	General rehabilitation and replacement				$551,000	$568,000	$587,000	$606,000	$626,000	$646,000	$667,000	
WWTP. RR.2	Plant Wide HVAC Improvements	System improvements to current HVAC equipment	$290,000										
WWTP. RR.3	Digester Building MCC	Replace obsolete and failing motor control center hardware	$80,000										
WWTP. RR.4	Primary Clarifier #2 Coating	Recoat primary clarifier #2 to protect from corrosion		$165,000									
WWTP. RR.5	Digester #1 Tank Coating	Recoat digester #1 tank		$330,000									
WWTP. RR.6	Secondary Clarifier #2 Coating	Recoat secondary clarifier #2 to protect from corrosion		$227,000									
WWTP. RR.7	Clarifier Gear Drive Replacements	Replace obsolete and failing gear drive on the clarifier			$325,000								
WWTP. RR.8	Plant Pump and Piping Replacement	Annual pump and piping maintenance			$80,000								
Annual Capital Improvement Plan Total													
Yearly Totals			$1,585,000	$1,083,000	$4,852,000	$2,313,000	$2,387,000	$2,876,000	$3,074,000	$2,629,000	$2,714,000	$6,825,000	

[1] *All capital costs are in 2015 dollars.*

[2] *$500,000 will be allocated to CCTV and Pipe Condition Rating*

Table ES-2 – Developer Driven Growth Projects

ID	Description/System Name	Recommend Action	Timeframe and Capital Cost										
			2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	With Growth[(1)]
Developer Driven Growth Projects – Projects to serve growth both inside and outside the UGA													
DD.1	Country Ridge Downstream Improvements	Upgrade downstream pipe to provide for future lift station upgrades and additional pumping capacity											$4,070,000
DD.2	East Badger South Lift Station	Construction required for development within the East Badger South Basin – SRSR CIP #1 (AHBL est.)											$5,500,000
DD.3	West Badger South Lift Station	Construction required for build-out of West Badger South and East Badger South											$3,180,000
DD.4	Horn Rapids Interceptor Extension	From Kingsgate Sports Complex to Village Pkwy/Construction as required with growth											$450,000
DD.5	SR 240 Interceptor	From Village Pkwy to Horn Rapids Rd/Construction as required with growth											$3,214,000
DD.6	600 Area (South) Interceptor	From Battelle Blvd to Horn Rapids Rd & North/Construction as required with growth											$3,467,000
Developer Driven Growth Project Total													
													$19,881,000

[(1)] All capital costs are in 2015 dollars.

ES-5 Budgeting CIP Projects

The CIP recommends a total of approximately $30.3 million be spent in capital improvements to the Wastewater Utility over the next 10 years. Improvements proposed include those necessary for the renewal and replacement of existing collection system and WWTP infrastructure to continue providing a safe, reliable, and cost-effective public sewer system. Those expansion improvements which are directly related to growth have been identified in the Master Plan but are not included in the CIP budget because they will generally be financed by developers. The extent of the City's participation, if any, would depend on the implementation of capital projects that may coincide with development.

The financial plan discussed in **Chapter 8** was prepared by FCS GROUP to provide a financial program that allows the wastewater utility to remain financially viable during the planning period.

The objective of the financial plan is to identify the total cost of providing sewer service and to present a financial program that allows the sewer utility to remain financially viable during the study period. The analysis considers the historical financial condition of the utility, the financial impact of executing the capital improvement plan (CIP), the sufficiency of utility revenues to meet future financial and policy obligations, and rate affordability.

The financial plan optimizes the capital funding resources as described in this plan. Local resources may include Facilities Fees, Local Facilities Charges, and utility cash reserves. External resources may include Department of Ecology grants and loans, Community Economic Revitalization Board grants and loans, Public Works Board loans, general obligation bonds and revenue bonds.

The results of the analysis indicate that rate increases are necessary to fund ongoing operating needs and the identified capital program. The City is in the process of completing a rate study to determine the annual rate increase strategy to meet the utility's financial obligations. The findings of the forecast for this GSP indicate that a cumulative increase of 21.5 percent meets the sewer utility's requirements through 2020, while remaining well within the affordability threshold.

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CHAPTER 1

Introduction

Chapter 1 – Introduction

1.1 Background

The City of Richland (City) has experienced several accelerated growth rates over the past 60 years in the form of residential housing developments as well as commercial development. Due to this growth, the General Sewer Plan has been updated at a frequency of about every 10 years – with updates in 1992 and 2004. The General Sewer Plan provides a tool that the City can use to maintain, operate, and expand the sewer system to meet the needs of the existing customers as well as planned growth.

The 2004 Plan identified a number of improvements to the existing system. Several of the CIP projects from the 2004 General Sewer Plan have been constructed by the City including: Leslie-Badger Sewer Trunk, Logston Boulevard Sewer Trunk, East Amon Wasteway Sewer Trunk, Duportail Sewer Relocation, SR 240 West Sewer Trunk Extension, Battelle Sewer Trunk Extension, Broadmoor Lift Station Replacement, Bellerive Lift Station and Force Main, and Conversion to Anoxic Selector at WWTP.

In 2013, the South Sewer Study was performed in order to provide more detailed sewer master planning for the portion of the City that is generally south of the Yakima River. As part of this Study, the existing collection system model in the south half of the City was updated with improvements made since the 2004 Plan and also included planned extensions for future growth. The land use and flow generation layers of the previous hydraulic model were also reevaluated for the study area through additional flow monitoring and calibration. The South Sewer Study was developed in such way as to be easily incorporated into this 2015 General Sewer Plan Update for the entire City.

1.2 Related Plans

1.2.1 City of Richland Comprehensive Land Use Plan

The City of Richland's Comprehensive Land Use Plan (2012) provides the land use planning that is assumed for future development of currently undeveloped areas within the sewer service area. The Land Use Plan also discusses guidelines for the provision of utility service in new and re-developed areas in order to provide orderly expansion of the utility services. Financing for capital projects is also discussed in the Land Use Plan. The financial plan section of the Land Use Plan will be updated by the City based upon the capital projects recommended in this General Sewer Plan Update.

1.2.2 Benton County Comprehensive Plan

The Benton County Comprehensive Plan (2013) has a number of components that are applicable to the City's sewer planning. The County is responsible for administering certain aspects of Washington's Growth Management Act (GMA) – the most important of which is the determination and management of the Urban Growth Areas (UGA) within the County. The County Comprehensive Plan also includes population projections that are directly applicable to the City's sewer planning. County-wide population projections are developed by the Washington State Office of Financial Management (OFM) for GMA planning purposes. The OFM projections forecast when growth will occur within the counties. In 2013, Benton County and jurisdictions within Benton County determined the percent allocation of the OFM population projections to each City and rural area in the County. The results of this allocation are used for population projections and planning purposes in this document. See **Section 2.11** for more details regarding population.

1.2.3 City of Kennewick

The City of Kennewick operates a sewer system adjacent to the City of Richland. Kennewick's system does not discharge to or otherwise interconnect with Richland's system. The City of Kennewick's General Sewer Plan reveals no service area boundary conflicts.

1.2.4 City of West Richland

The City of West Richland also operates a sewer system adjacent to the City of Richland. West Richland's system likewise does not discharge to or otherwise interconnect with Richland's system. The City of West Richland's most recent sewer plan was published in 1997. There are no apparent service area boundary conflicts between West Richland and Richland.

1.3 Study Scope

Since the last General Sewer Plan was completed nearly ten years ago, the City authorized J-U-B to undertake a General Sewer Plan Update in 2014. This plan identifies the sewer capital improvement projects that will be needed for rehabilitation and replacement to meet the needs of the planning period through 2035.

The items specifically addressed in this General Sewer Plan are as follows:

- Update the hydraulic model to incorporate infrastructure that has been constructed since 2004
- Update current and planned land uses during the study period
- Analyze available water meter usage and evaluate flow generation assumptions used in the previous modeling efforts
- Re-calibrate the updated collection system model with new flow monitoring information
- Evaluate the existing collection system trunk pipes based on existing dry weather flows and wet weather flows to determine recommended improvements under current conditions
- Evaluate the existing collection system trunk pipes to provide service to all lands within the current UGA
- Estimate probable build-out extents, densities, and total population in conjunction with City Planning and available population projection data to aid in developing the Committed and Master Plan hydraulic model scenarios
- Review existing gravity sewer alignments and lift stations to determine if future pipes could be constructed to eliminate the lift stations
- Conceptually route future trunk sewers ten inches and larger to the ultimate service boundary
- Determine preferred flow routing through the existing system and impacts to the existing system
- Establish long-term improvements for the collection system with a specific 5-year Capital Improvement Plan (CIP) based on established prioritization criteria
- Provide a general overall evaluation of the condition and capacity of the Wastewater Treatment Plant
- Document the sewer utility's financial condition and assess its ability to support the recommendations of CIP

- Summarize the City's current Operations & Maintenance Program
- Summarize the City's current Pre-Treatment Program

Subsequent chapters in this report are summarized as follows:

Chapter 2 – Planning Information

The planning area characteristics, land use, and population projections are presented in this chapter. In addition, service area agreements and policies are summarized. The information presented in this chapter is intended for consistency with the City's Comprehensive Plan and Growth Management Act compliance.

Chapter 3 – Flow and Load Projections

The flow and load projections for the WWTP are summarized in this chapter. In addition, wastewater usage is classified by user type and significant users are discussed. A discussion on infiltration and inflow is also provided in this chapter.

Chapter 4 – Performance and Design Criteria

This chapter provides a summary of collection system design criteria as well as reference to Federal and State Regulations relating to WWTP performance criteria.

Chapter 5 – Wastewater Treatment Plant

This chapter describes the condition and existing capacity of the WWTP. A brief history of the improvement projects since 2004 is provided as well as a summary of future planned CIP projects at the WWTP. This chapter was developed by Carollo Engineers, Inc.

Chapter 6 – Collection System

The update of the hydraulic model of the collection system is summarized in this chapter. An evaluation of the existing capacity of the collection system as well as development of a Master Plan for collection system expansion and development is also presented.

Chapter 7 – Capital Improvement Plan

A prioritized list of collection system and WWTP capital projects is provided in this chapter.

Chapter 8 – Financial Plan

An overview of the City's revenues, projections, and plans for financing the projects identified in the CIP is provided in this chapter. This chapter was developed by FCS Group.

Chapter 9 – Operations Program

This chapter includes an overview of the organizational structure and staffing requirements for the Wastewater Utility operations program.

Chapter 10 – Pretreatment

A summary of the City's Pretreatment Program and Fats, Oils, and Grease (FOG) Program are provided in this chapter.

1.4 System Overview

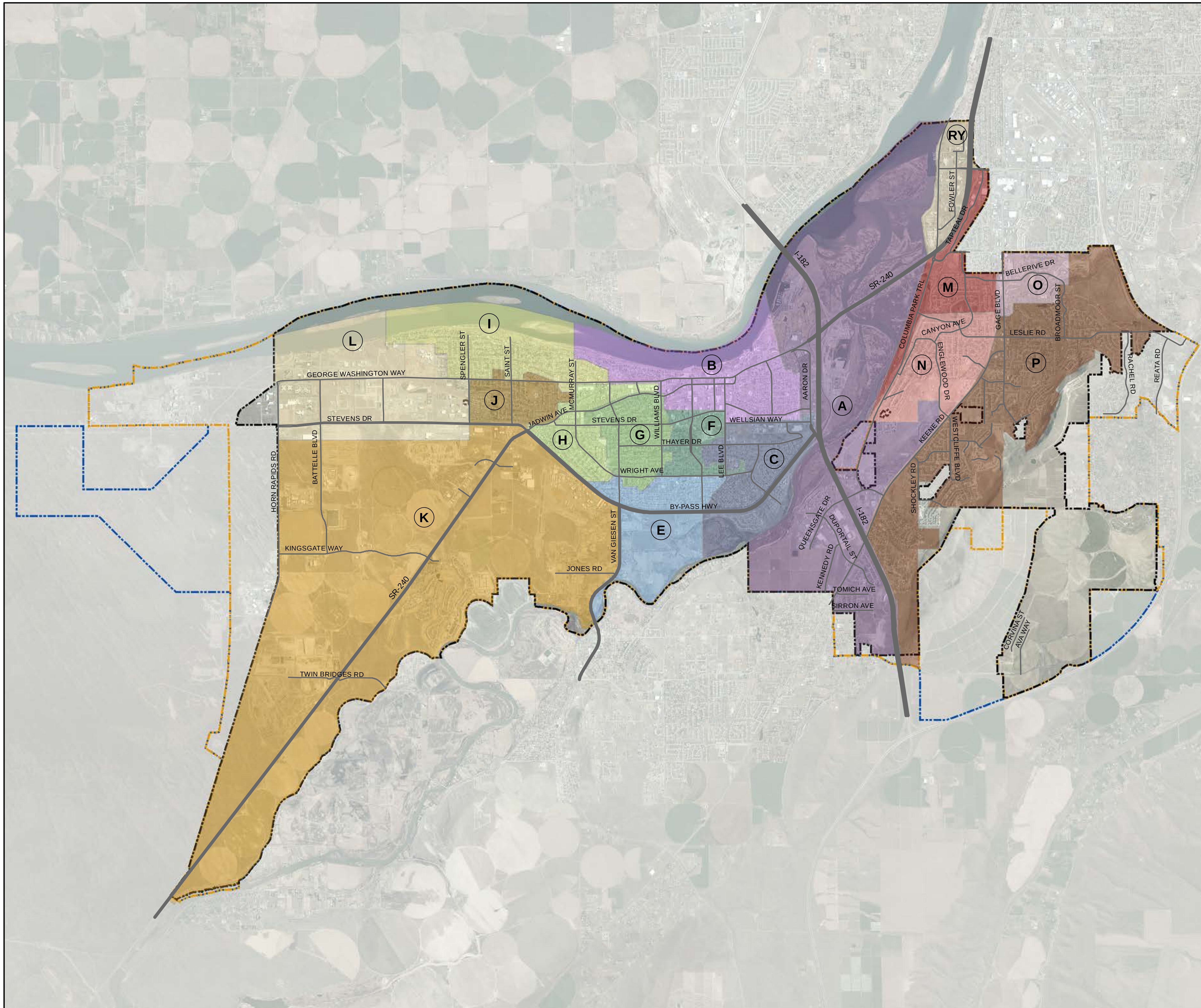
The City of Richland is part of the Tri-Cities urban area, which is an important transportation and trade center for the Columbia Basin of central Washington. The major economic influences in the area include the Hanford Atomic Energy Reservation and irrigation-dependent agriculture.

The original Richland wastewater collection and treatment system was constructed by the federal government to serve the Hanford Reservation in 1942 and 1943. An additional, parallel treatment plant was constructed in 1948 and 1949. These facilities, expected to become obsolete soon after construction, were in use until 1985. In 1976 a study was completed and a plan developed for providing secondary treatment to meet state and federal requirements. A new wastewater treatment facility was recommended to replace the existing plant. Construction of new interceptors was commenced in 1980, and construction of the new wastewater treatment facility began in 1983. The new wastewater treatment facility, the WWTP began operation in September 1985.

The Richland sewer collection system is divided into 17 basins based on topography, configuration, and parcel boundaries – reference **Figure 1-1**. **Table 1-1** lists the sewer basins and the area of each basin.

Table 1-1 – Sewer System Drainage Basins

Sewer Drainage Basin	Area (acres)
A	4,700
B	1,209
C	911
E	869
F	301
G	495
H	447
I	1,071
J	381
K	7,560
L	1,770
M	576
N	738
O	312
P	2,616
Q	1,400
RY	370
TOTAL	**25,726**



City of Richland
2015 General Sewer Plan

Figure 1-1
Sanitary Sewer Drainage Basins

Legend

- City Limits
- UGA
- 50-yr Planning
- Interstate/Highway
- Major Streets

Sewer Basin ID

- A
- B
- C
- E
- F
- G
- H
- I
- J
- K
- L
- M
- N
- O
- P
- RY

0 6,000 12,000 Feet

Date: Apr 1, 2016

J·U·B ENGINEERS, INC.

The City's sewer collection system has expanded from an initial series of pipelines serving the City core to a system containing over 262 miles of gravity pipelines and 14 pumping stations providing public sewer service to a residential population of 53,054. The total area that is provided with public sewer service totals 25,726 acres.

The existing wastewater collection system consists of gravity pipelines ranging in size from 6 inches in diameter up to 54 inches in diameter. Based on the City's sewer GIS information, **Table 1-2** provides a summary of the size and lengths of collection pipes that make up the public collection system.

Table 1-2 – Existing Gravity Collection System Pipes

Diameter (inches)	Length (feet)	Length (miles)
≤6	69,941	13.2
8	965,829	182.9
10	80,282	15.2
12	120,080	22.7
14	5,618	1.1
15	21,153	4.0
16	2,952	0.6
18	30,988	5.9
21	16,601	3.1
24	30,504	5.8
27	411	0.1
30	10,059	1.9
36	1,315	0.2
42	6,058	1.1
54	24,941	4.7
TOTAL	**1,386,732**	**262.6**

Roughly 75 percent of the collection system consists of pipelines 8 inches and smaller in diameter. These pipelines are generally referred to as local collection pipelines and provide service to individual sub-drainage basins located within the Service Area. Pipelines 10 inches and greater in diameter are often referred to as trunk or interceptor pipelines and may provide service to entire drainage basins or more than one drainage basin within the Service Area.

The ability of the collection system to provide gravity sewer service within the Service Area is dependent upon the topography of the Service Area. Much of the Richland sewer service area is flat, which has presented a challenge to constructing sewers with the minimum slopes required to maintain self-cleansing velocities while minimizing pumping. The City has several lift stations and forcemains that are an integral part of the collection system. The City desires to minimize the number of lift stations to reduce operation and maintenance requirements and has eliminated several lift stations with recent interceptor improvements. There are presently 14 sewage pumping stations located throughout the collection system. The name and capacity of these sewage pumping stations are summarized in **Table 1-3**.

Table 1-3 – Sewage Pumping Stations

Name	Rated Flow [a] (gpm)	HP	Pump Type	Pump Manufacturer
Battelle	400	5	Submersible	Flygt
Waterfront	600	15	Centrifugal	Fairbanks Morse
Terminal Dr	150	3	Centrifugal	Fairbanks Morse
Mental Health	260	5	Centrifugal - Chopper	Vaughan
Bradley	180	10	Submersible	Flygt
Columbia Pt	270	6.5	Submersible	Flygt
Wellhouse Loop	100	1.5	Centrifugal	Hydromatic
Duportail	200	7.5	Submersible	Flygt
Montana St	970	30	Centrifugal	Smith & Loveless
Columbia Park Trail	400	10	Submersible	Flygt
Meadows South	100	3	Centrifugal	Hydromatic
Bellerive	260	15	Submersible	Flygt
Meadow Ridge	245	10	Submersible	Flygt
Dallas Rd	260	35	Submersible	Flygt

[a] *The rated flow is based on the operation of one pump. All of the stations have a duplex pump setup.*

The City does not have any connections to other wastewater systems. Richland collects and treats all of the wastewater within its service area and does not receive wastewater from other jurisdictions.

CHAPTER 2

Planning Information

Chapter 2 – **Planning Information**

2.1 Planning Area

The City's UGA and Master Plan (year 2065) Service Areas are presented in **Figure 2-1**. The UGA was established by the City's Planning Department and Citizen Advisory Committee as part of planning activities undertaken to meet the requirements of the State of Washington Urban Growth Management Act (GMA). The current UGA was adopted by the Benton County Commissioners in 2005. The Master Plan Service Area anticipates the development over the next 50 years in the Badger South area toward Interstate 82 and Interstate 182 in the southwest, and reclamation of former 300 Area land in the Hanford Area to the north.

2.2 Service Area

The current Service Area Boundary (commensurate with the UGA) is presented on **Figure 2-1** and represents the area that the existing system of interceptor sewers, trunk sewers, collection system, and pumping stations effectively serve.

Development within the City currently trends toward the south, being limited on the east by the City of Kennewick, the west by the City of West Richland, and the north by the Hanford Area. Future development is expected to continue a south and northwest directional trend. The City's current infrastructure maintenance, rehabilitation and replacement program will help to encourage build-out of the developed territory in the City's interior. Future population increases are anticipated to be significant in the south service area with the addition of the Badger South residential development.

2.2.1 Onsite Sewer Systems

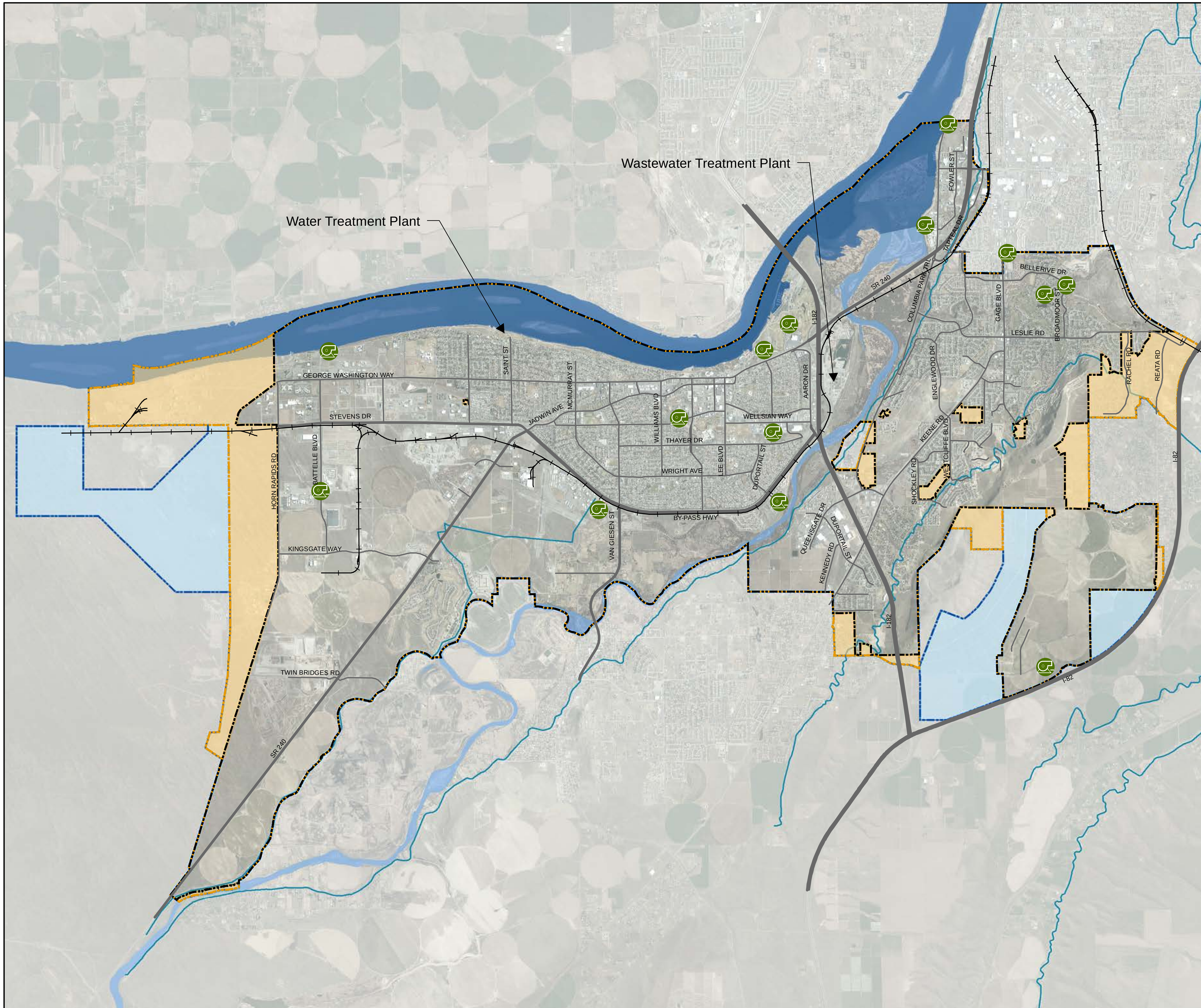
The Benton Franklin Health District (BFHD) has primary responsibility for permitting and policing the residential and small flow commercial dischargers using onsite sewer systems within the City's UGA.

The City's Sewer User Ordinance mandates that residents within the City limits connect to the public sewer system when service is available. In practice, this has only been enforced by requiring the property owner to connect in the event of onsite sewer system failure. Enforcement procedures for onsite sewer system failure are under the jurisdiction of the BFHD. In the event of an onsite sewer system failure, the BFHD Health Officer has the discretion to mandate either hook-up to the public system or onsite sewer system repair or replacement.

The current Service Area and the UGA include only a few small areas in the southern portions of the City where onsite sewer systems are currently the primary means of wastewater disposal. These areas are typically low density developments which predate the southerly expansion of the City's corporate boundaries. An estimated 700 people currently utilize septic tanks and drain fields as a method to dispose of their wastewater.

City of Richland
2015 General Sewer Plan

Figure 2-1
Service Area Boundaries



Legend
- City Limits
- UGA
- 50-yr Planning
- Interstate/Highway
- Major Streets
- Rail Road Track
- Irrigation Canal/Pipeline
- Columbia River
- Yakima River
- City Lift Stations

0 6,000 12,000 Feet

N E S W

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2.3 Service Area Characteristics

2.3.1 Topography

Richland is situated in a river valley between two hill plateaus. The Columbia River forms the City's eastern boundary. The Yakima River runs along the City's western boundary and then east through the City and into the Columbia River. Because Richland is within a river valley, it is relatively flat in the central and north parts of the City. South of the Yakima River, elevations increase significantly (around Badger Mountain).

2.3.2 Climate

The climate of the area is semiarid, characterized by low annual precipitation and large inter-seasonal temperature variations. Strong winds from the west and southwest occur throughout the year and are responsible for localized soil movement and excessive evapotranspiration rates in summer. Annual precipitation seldom exceeds ten inches, with much of the total arriving with summer thunderstorms, which can cause flooding and severe erosion. The recent (2009 - 2013) climatological information for the City is summarized in **Table 2-1**.

Table 2-1 – Climatological Data

Year	Average Temperature (°F)	High Temperature (°F)	Low Temperature (°F)	Rainfall (in)
2009	65	105	5	6.2
2010	65	101	5	9.2
2011	64	97	14	5.0
2012	66	105	17	11.3
2013	68	108	13	7.3
2014	64	104	6	5.0

2.3.3 Geology

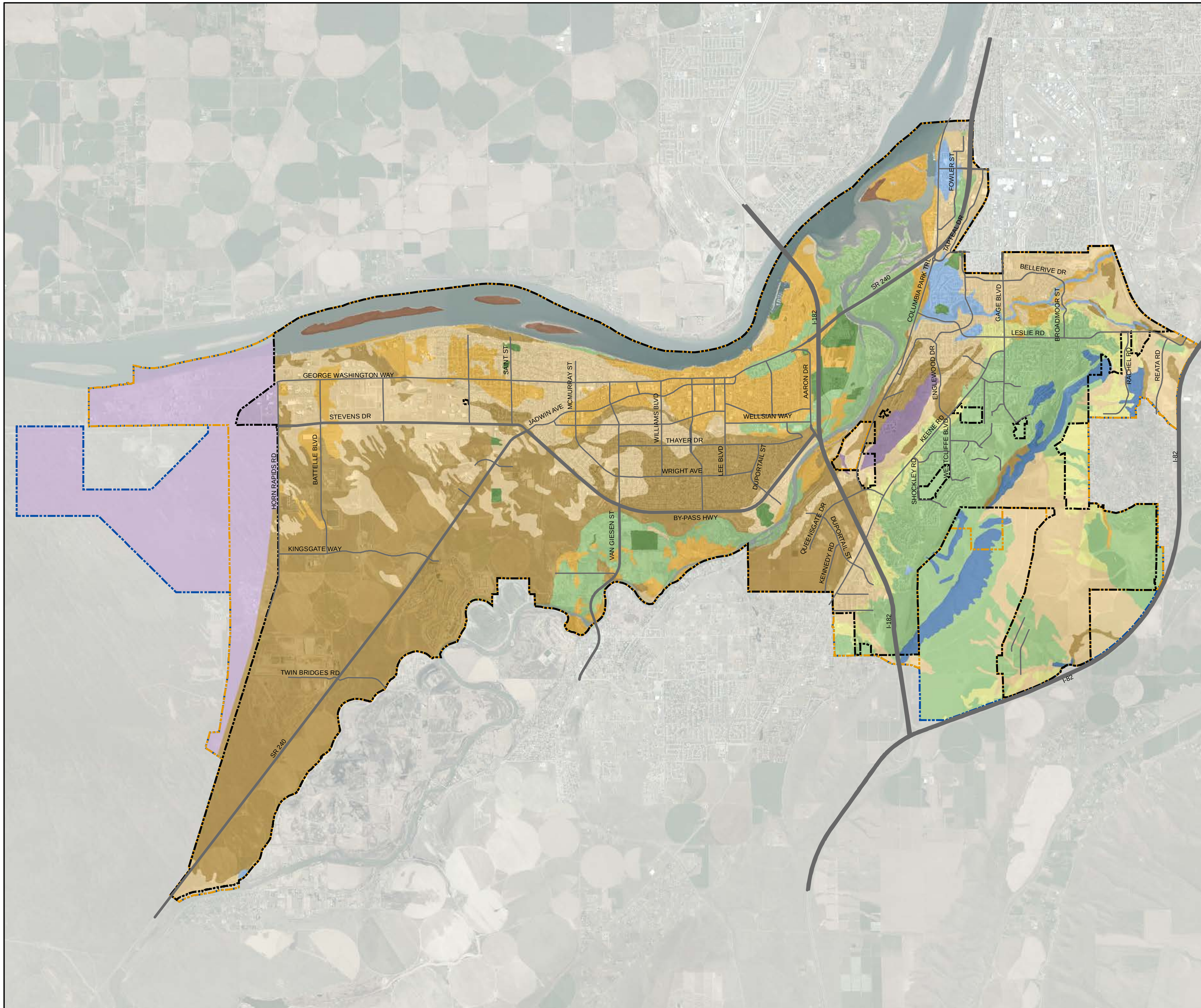
The geology of the Service Area relates to the long history of volcanic activity, which has influenced the Columbia Basin. At the surface is a layer of unconsolidated alluvial and glaciofluvial materials ranging in depth from 0 to 120 feet. The depth of this overburden generally does not exceed 30 feet within the Richland Sewer Service Area. The overburden rests on a thick series of basaltic strata known as the Columbia River basalts, each of which may consist of many distinct basalt flows. These basalts are interbedded with two major and many minor sedimentary strata. The uppermost basalt unit, the Saddle Mountain basalt, crops out in places where the overburden thins in the upper elevations of the Richland planning area. The Saddle Mountain basalt ranges in thickness from 125 to 625 feet, but it is typically about 250 feet thick. It may be interbedded with many sedimentary strata, some of which are up to 50 feet thick. The Saddle Mountain basalt is separated from the Wanapum basalt by the Mabton Interbed. The Mabton Interbed is composed of clay and siltstone and ranges in thickness from 10 to 75 feet, with a typical thickness of 45 feet. The Wanapum basalt ranges in thickness from 600 to 1200 feet, with a typical thickness of 800 feet. Interbedding sedimentary strata are insignificant in the Wanapum basalt. The Vantage sandstone interbed, averaging about 25 feet in thickness, separates the Wanapum basalt from the underlying Grande Ronde basalt. The Grande Ronde basalt has a typical thickness of 5,000 feet, but may range from 2,000 to 12,000 feet thick. The

Grande Ronde basalt contains almost no interbedding sedimentary strata. Under the Grande Ronde basalt lies additional basalt groups, the Pre-Yakima and the Pre-Columbia River basalts.

Locally significant hydrogeologic units occur in the Saddle Mountain and Wanapum basalts, in the Mabton Interbed, and in the overburden where its depth is sufficient.

2.3.4 Soils

The soils in and around Richland are classified by the U.S. Department of Agriculture, Natural Resource Conservation Service (NRCS). Most of the Richland area soils are classified as being silt and sandy loam. **Figure 2-2** depicts the types of soils within the City's sewer service area. The soils are generally well draining.



City of Richland

2015 General Sewer Plan

Figure 2-2

Soils

Legend

- City Limits
- UGA
- 50-yr Planning
- Interstate/Highway
- Major Streets

Soil Type

- Fine Sandy Loam
- Gravel Pits
- Loamy Fine Sand
- Loamy Sand
- Rock Loamy Fine Sand
- Silt Loam
- Stony Fine Sandy Loam
- Very Fine Sandy Loam
- Very Stony Silt Loam
- Riverwash
- No Digital Data Available

Notes:

1. Soil Type is based on Soil Survey Geographic (SSURGO) database for Benton County, by the U.S. Department of Agriculture, Natural Resources Conservation Service (NRCS) Publication dated 09-03-2014.

0 6,000 12,000 Feet

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2.4 Irrigation Districts

Separate irrigation water is only provided for the part of the City that is south of the Yakima River. Those north of the river use City water for irrigation – with a few minor exceptions. There are small irrigation systems serving the following areas:

- Columbia Point: A pump station on the Columbia River provides irrigation to the Columbia Point Golf Course and some multi-family housing units adjacent to the golf course.
- Horn Rapids: A pump station on the Columbia River that serves the Horn Rapids Golf Course, a residential subdivision, sports complex, ORV Park, Landfill, and some farmlands.
- Research District: Two wells owned by the City that serves some commercial and light industrial areas.
- Richland School District: A well located on the grounds of Carmichael Middle School which serves the middle school and adjacent Richland High School.
- Willowbrook: A City owned well located in Claybell Park that provides irrigation to a park and residential subdivision.

In the south, the irrigation network consists of three quasi-municipal agencies: the Badger Mountain Irrigation District (BMID), the Columbia Irrigation District (CID) and the Kennewick Irrigation District (KID). All three maintain separate systems and service areas but each deliver untreated Yakima River water through open and closed conduits to agricultural and residential customers. The extent of this service is generally limited by elevation, as the irrigation systems were designed for gravity operation; however some BMID service areas are served by elevated storage tanks filled by booster pump stations. **Figure 2-3** shows the extent and areas of influence of the three irrigation systems in relation to the City of Richland's utility service area.

2.5 Domestic Water System

The City owns and operates a water system that serves the City of Richland and developments outside of the incorporated area but within the UGA.

The main source of Richland's drinking water is the Columbia River from which water is pumped to the water treatment facility on Saint Street. The treatment facility is rated to produce up to 36 million gallons per day (mgd) and is supplemented by groundwater wells. The City water system includes:

- Fifteen reservoir sites including the WTP clearwell
- Seven active chlorination points
- Twelve booster pump stations
- Three emergency interties with neighboring systems
- Seven pressure zones
- Five well sites for City water supply

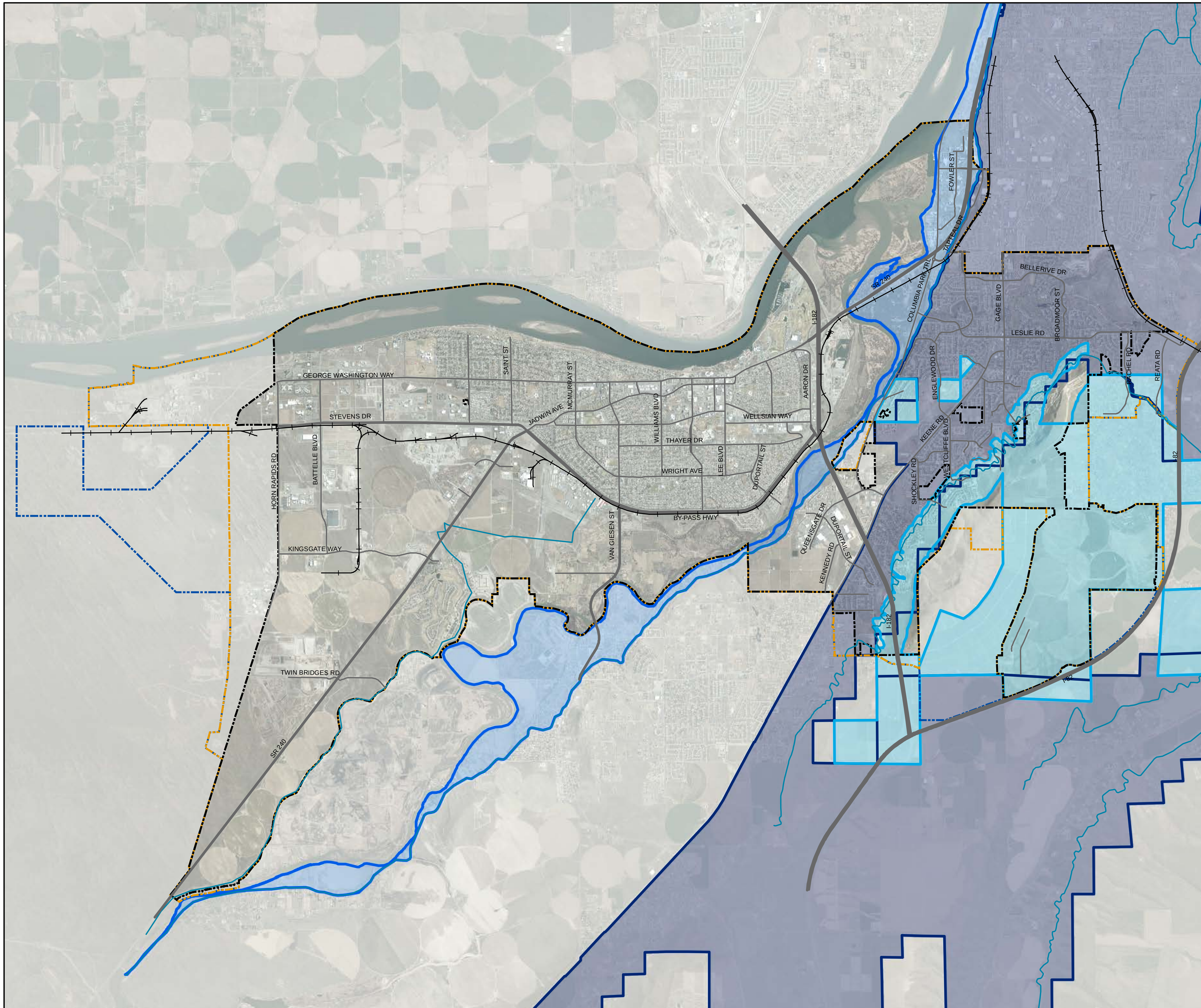
The main water system features are depicted in **Figure 2-4**.

City of Richland

2015 General Sewer Plan

Figure 2-3

Irrigation Districts



Legend

- City Limits
- UGA
- 50-yr Planning
- Interstate/Highway
- Major Streets
- Rail Road Track
- Irrigation Canal/Pipeline

District

- Badger Mountain Irrigation District
- Columbia Irrigation District
- Kennewick Irrigation District

0 6,000 12,000 Feet

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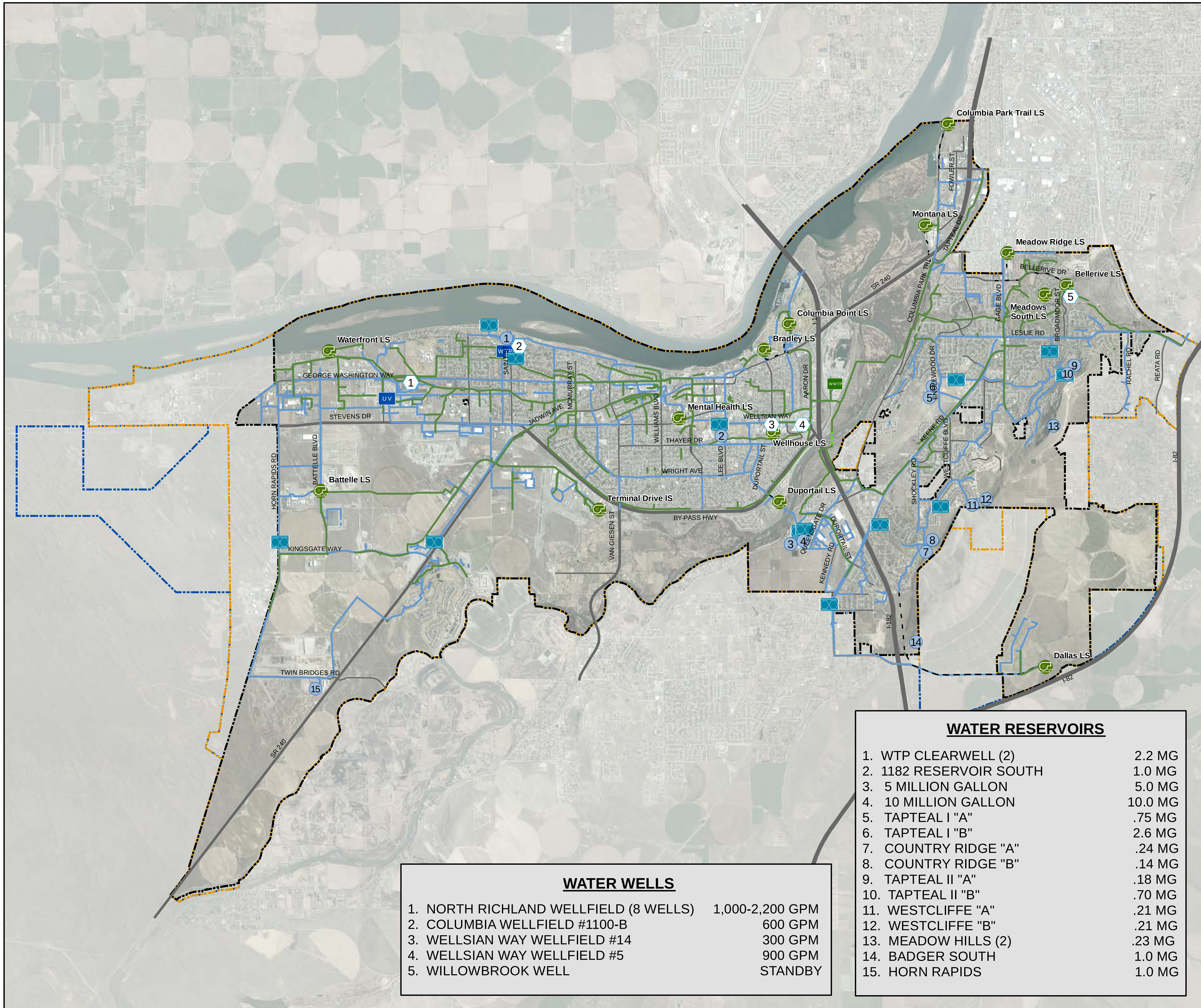
City of Richland
2015 General Sewer Plan

Figure 2-4
Water and Sewer Mainlines



Legend
- City Limits
- UGA
- 50-yr Planning
- Interstate/Highway
- Major Streets
- Water Main (10" and Larger)
- Sewer Main (10" and Larger)
- Inverted Sewer Siphon
- Sewer Forcemain
- Well
- Water Reservoir
- Water Booster Pump
- UV Treatment Facility
- Water Treatment Plant
- Wastewater Treatment Plant
- City Lift Stations

0 6,000 12,000 Feet

WATER WELLS

1. NORTH RICHLAND WELLFIELD (8 WELLS)	1,000-2,200 GPM
2. COLUMBIA WELLFIELD #1100-B	600 GPM
3. WELLSIAN WAY WELLFIELD #14	300 GPM
4. WELLSIAN WAY WELLFIELD #5	900 GPM
5. WILLOWBROOK WELL	STANDBY

WATER RESERVOIRS

1. WTP CLEARWELL (2)	2.2 MG
2. 1182 RESERVOIR SOUTH	1.0 MG
3. 5 MILLION GALLON	5.0 MG
4. 10 MILLION GALLON	10.0 MG
5. TAPTEAL I "A"	.75 MG
6. TAPTEAL I "B"	2.6 MG
7. COUNTRY RIDGE "A"	.24 MG
8. COUNTRY RIDGE "B"	.14 MG
9. TAPTEAL II "A"	.18 MG
10. TAPTEAL II "B"	.70 MG
11. WESTCLIFFE "A"	.21 MG
12. WESTCLIFFE "B"	.21 MG
13. MEADOW HILLS (2)	.23 MG
14. BADGER SOUTH	1.0 MG
15. HORN RAPIDS	1.0 MG

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2.6 Water Reclamation and Reuse

Water reclamation and reuse is a concept gaining considerable recognition in Washington as both a water supply option and a treated wastewater discharge alternative. Reclaimed water can provide an alternative water source for non-potable applications that would otherwise be limited by traditional water supplies. Wastewater effluent reuse can also provide opportunities for an overall decrease in pollution and the ability to meet more stringent water quality requirements when it reduces or removes treated wastewater discharges to sensitive surface waters.

The City's 2010 Water System Plan provides a discussion on Washington State standards and regulations for water reclamation and reuse. As noted in the Water System Plan, the City has a water use efficiency program that is expected to reduce water usage – which will in turn produce some reductions in sewer flows to the WWTP.

Large land areas for agricultural use are located greater than five miles from the WWTP site and the land is either currently not irrigated or is irrigated with untreated surface water delivered from local irrigation districts. Although substituting WWTP effluent for untreated surface water for irrigation would result in a reduction of water diverted from the Columbia River, the restrictions to crop production, public access limitations, and estimated cost of transport and pumping of water would make this alternative infeasible.

2.7 Zoning/Land Use

The Future Land Use Map is presented in **Figure 2-5** and is based on the City of Richland Comprehensive Plan. Within the land use districts, reserve areas are provided to facilitate the orderly expansion of the City's residential, commercial, and industrial base. An analysis of the current utilization of the land use districts in relation to the existing Sewer Service Area and UGA is presented in **Table 2-2**.

Table 2-2 – Land Use Analysis

Land Use	Developed [a] (acres)	Developed (%)	Undeveloped (acres)	Undeveloped (%)	Total (acres)
High Density Residential [b]	435.50	82	94.00	18	529.50
Medium Density Residential [c]	1,187.60	81	271.60	19	1,459.20
Low Density Residential [d]	3,652.20	69	1,648.10	31	5,300.30
Badger Mountain South [e]	1,795.00	68	858.00	32	2,653.00
Commercial	1,882.50	36	3,416.30	64	5,298.80
Industrial	2,246.60	32	4,820.70	68	7,067.30
Open Space/Agricultural	632.50	61	408.70	39	1,041.20
Public Facilities	5,489.00	100	0.00	0	5,489.00
Rights of Way	4.50	0	1,426.50	100	1,431.00
TOTAL	**17,325.40**	**57**	**12,943.90**	**43**	**30,269.30**

(a) *A parcel is developed if it has an addressed structure on it or if it is completely paved.*

(b) *15 units/acre assumed*

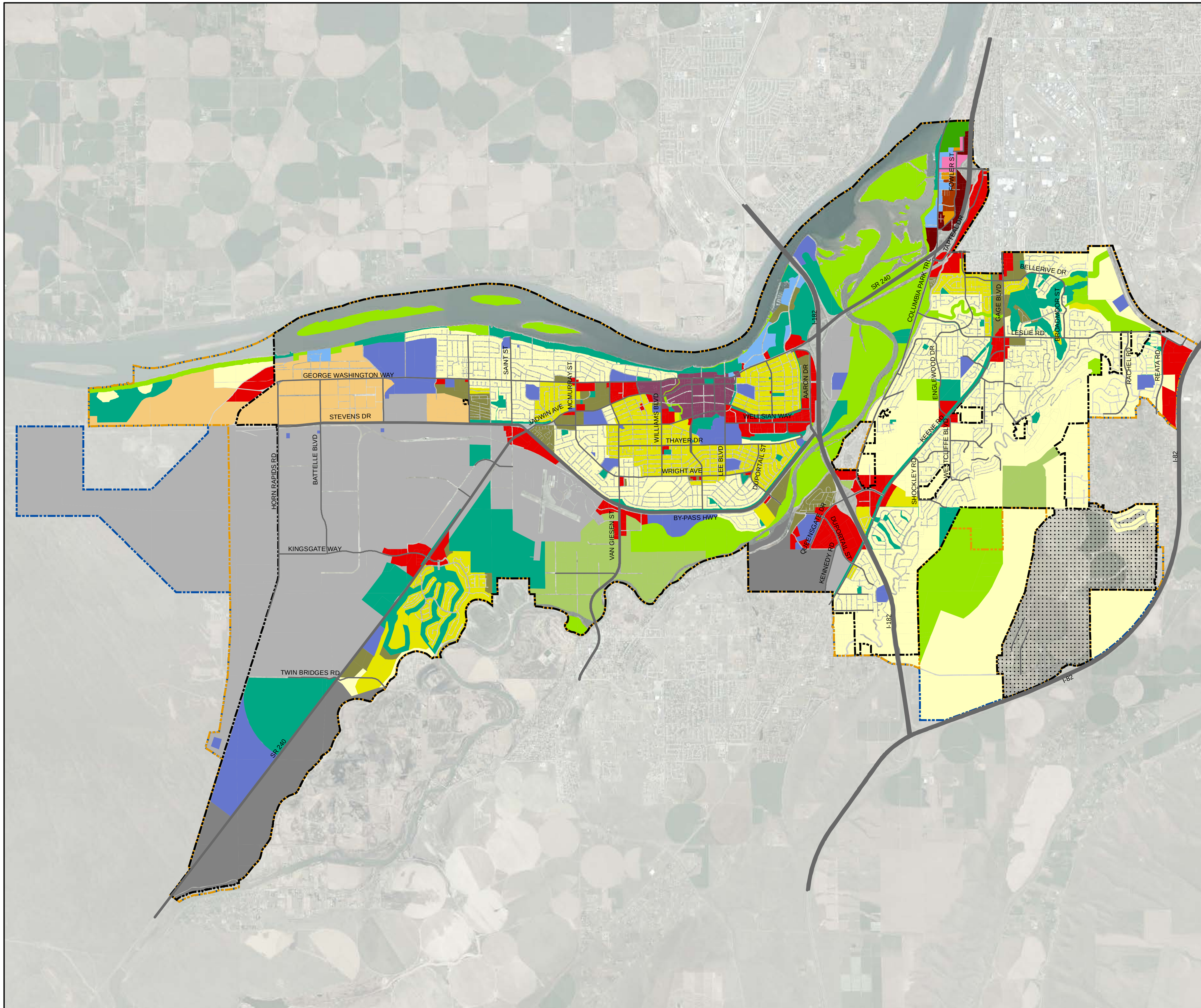
(c) *8 units/acre assumed*

(d) *3.5 units/acre assumed*

(e) *Density varies by development*

City of Richland
2015 General Sewer Plan

Figure 2-5
Land Use



Legend

- City Limits
- UGA
- 50-yr Planning
- Interstate/Highway
- Major Streets

Land Use Type

- Agriculture
- Commercial
- Business/Research Park
- Business Commerce
- General Commercial
- Retail Regional
- Commercial Recreation
- Multifamily Residential Office
- Central Buisness District
- Industrial
- Public Facility
- Low Density Residential
- Medium Density Residential
- High Density Residential
- Waterfront
- Natural Open Space
- Developed Open Space
- Urban Reserve
- Badger Mountain South

0 6,000 12,000 Feet

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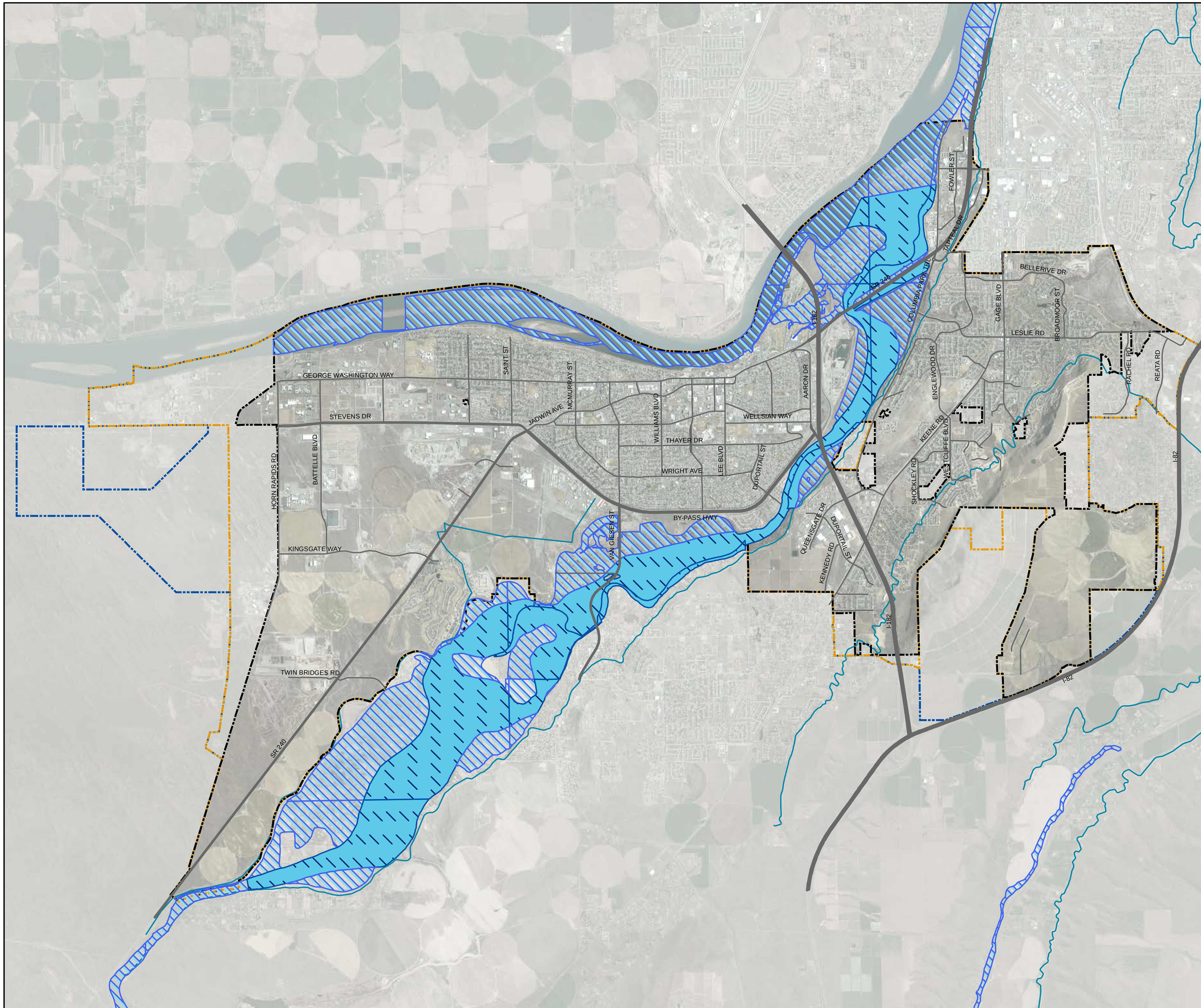

City of Richland
2015 General Sewer Plan

Figure 2-6
FEMA Flood Areas

Legend

- City Limits
- UGA
- 50-yr Planning
- Interstate/Highway
- Major Streets
- Irrigation Canal/Pipeline

Flood Areas

- 100-yr Floodplain
- Floodway

Notes:
1. Flood Data is derived from the Flood Insurance Rate Maps (FIRM) published by the Federal Emergency Management Agency (FEMA).The FIRM is the basis for floodplain management, mitigation, and insurance activities for the National Flood Insurance Program (NFIP).

0 6,000 12,000 Feet

N E S W

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2.8 FEMA Floodway Mapping

The Federal Emergency Management Agency (FEMA), under the National Flood Insurance Program (NFIP), issued Floodway (Flood Boundary and Floodway) Maps and Flood Insurance Rate Maps (FIRM) for Richland in 1982. Subsequently, FEMA restudied and issued revised FIRM maps for select areas within the City in 1984. **Figure 2-6** illustrates the approximate boundary of 100 year flood areas in Richland. In general, sewage facilities constructed within the 100 year flood area must be protected. There are two existing sewage facilities located within the 100 year flood area in the form of two siphon crossings.

2.9 Service Area Policies

Service area policies of the Sewer Utility are defined in Title 17 of the Richland Municipal Code (RMC). This includes provisions for use of the sewer system, prohibited discharges to the public facilities, requirements for pretreatment, general standards for building sewers, general standards for public sewer construction, connection charges and monthly user charges, and requirements for compliance with the Uniform Plumbing Code.

Under paragraph 17.12, all buildings are required to connect to the public sewer system if available within 300' of the owner's property line. Under the current RMC, if a public sewer line is not available to a property (i.e. not located within 300' of the owner's property line), the owner may be allowed to construct a septic tank and drainfield in accordance with rules and regulations of the Benton Franklin Health District.

2.9.1 Wastewater Service Rates

Sewer user charges have been established for the Sewer Utility under Title 17 of the RMC. Under paragraph 17.56.010, residential and multifamily residences are charged monthly rates of $25.60 and $12.40, respectively. Multifamily customers also pay a consumption charge for sewer that is based upon water consumption.

Under paragraph 17.56.020, all non-residential customers are charged monthly rates of $61.50 including a charge of $2.15 per 100 cubic feet of volume as measured at the water service meter.

These monthly sewer service charges apply to all residences and commercial establishments within the City having a sewer on the premise or within 300 feet of the property line, regardless of whether connection to the sewer system has been made.

The monthly sewer rates for sewer furnished to out-of-city customers include a 50% surcharge.

2.9.2 Wastewater Service Connection Fees

Connection fees have been established for the Sewer Utility under paragraph 17.56 of the RMC. Sewer treatment, lift station, interceptor facilities and frontage charges are assessed and collected as a condition precedent to providing sewer service connections based on the water meter size as shown in **Table 2-3**.

Table 2-3 – Connection Fee Charges

Size of Water Meter	Facilities Assessment	Frontage Charge
¾”	$1,995	$15/LF
1”	$1,995	$15/LF
1 ½”	$6,643	$15/LF
2”	$10,633	$15/LF
3”	$19,950	$15/LF
4”	By Contract	$15/LF
6”	By Contract	By Contract

2.10 Growth Management Act Compliance

The Growth Management Act (GMA) was enacted in 1990, and amended in 1991, 1996, and 1997 to ensure coordinated, planned urban growth. The major emphasis is to manage “urban growth” including the type of growth, its intensity, its location, and its demands for utilities and services. Under the GMA, the fastest growing counties in the State (and the cities within them) are required to plan to manage growth. Several other counties, including Benton County and the cities within Benton County, have also opted to plan under the Act.

Comprehensive Plans prepared under the GMA must accommodate a 20-year growth projection. The GMA requires the establishment of “Urban Growth Areas” in order to help guide urban growth into areas that are most appropriate and to reduce urban sprawl. The GMA also requires designation and protection of agricultural lands, forest lands, mineral resource lands, and critical areas. Critical areas include wetlands, critical aquifer recharge areas that provide drinking water, fish and wildlife conservation areas, frequently flooded areas, and geologically hazardous areas.

The City has completed a Comprehensive Land Use Plan under the GMA and is in full compliance with all regulatory mandates. The City’s most recent Comprehensive Plan Update was completed in 2012. This update was conducted as required by GMA to prepare periodic updates. The City has made minor annual amendments to the plan, with the most recent one occurring in 2014.

This General Sewer Plan represents a complementary implementation plan to the Comprehensive Land Use Plan. Significant capital investments will be required for the City to achieve the objectives identified in the Comprehensive Land Use plan. The ability to properly manage wastewater is essential to future residential development and to attract new commerce and industry.

2.11 Population

The City’s 2015 population estimate for the incorporated area is 53,054. This population is estimated based on a Washington State Office of Financial Management (OFM) Benton County population estimate which was updated in April 2014. The 2010 US Census data indicates that there are 2.42 people per single-family residential home.

The County’s most recent Comprehensive Plan estimates a population of 76,533 for the incorporated area by the year 2035. This estimate yields an estimated growth rate of 1.85 percent per year over the next 20 years.

Population projections are shown in **Table 2-4**. These population projections are based on the successful implementation of the City's Growth Management Plan.

Table 2-4 – Population Projections

Year	Service Area Population
2015	53,054
2035	76,533

2.12 Badger Mountain Area Master Planning

South of Badger Mountain is a large planned residential and commercial development known as Badger South. It is the source of the majority of the projected sanitary sewer flow for the area of Richland south of the Yakima River. The Badger South development and vicinity area are shown on **Figure 2-7**. Several sewer planning efforts for this area have been undertaken over the past 15 years. The following is a brief summary of studies completed in 2001, 2004, 2006, and 2010 for the development of the area south of Badger Mountain. The purpose of this summary is to document the history in regards to sewer service concepts for this planned area and to provide background that is applicable to the discussion in the Committed Model and Master Plan Model sections.

<u>2001 Feasibility Study for Wastewater Facilities</u>
This study prepared by J-U-B looked at the planning level feasibility for providing sanitary sewer service to the proposed Badger Mountain Project. In 2001, a lower density of residential development was planned and included a golf course as part of the development. The Study evaluated several different wastewater treatment and disposal options including: onsite sewer systems, evaporation ponds, lagoons, and conventional activated sludge plant. The Study also evaluated connection to the City of Richland sanitary sewer collection system by connecting to an interceptor planned to be extended south from Meadow Springs approximately 2 miles east of the project site (Leslie interceptor). The Study identified three feasible ways to connect to this future sewer trunk:

1. Rachel Road: A lift station would pump sewage via a force main along Clover Road to the top of Rachel Road and then a gravity collection system would follow Rachel Road. A significant amount of pavement restoration, traffic control, and construction impacts to residences were noted as key issues.
2. Canyon Route: A lift station could be avoided if a gravity sewer interceptor was constructed through the canyon that passes through the El Rancho Reata development to Leslie Road. The presence of shallow bedrock was identified as a key issue that would require further evaluation.
3. Bermuda/Reata Road: A lift station would be required to pump sewage to the high point near the Bermuda/Reata Road intersection. The alignment would follow Reata Road east to Leslie Road and then north to Rachel/Leslie Road intersection. The route is not very direct and pavement restoration, traffic control, and construction impacts to residents were identified as key issues.

An alternative point of connection to the Richland sewer system was also identified as the interceptor on Gage Boulevard. This fourth option would follow the alignment of a new road planned to be constructed from Gage Boulevard southwest over the ridge to the Badger Mountain development. A lift station would still be required; however, the pavement restoration, traffic control, and disruption to residences would be minimized.

Because the study area was not part of the City's UGA at the time, the connections to the City system were dropped from further consideration. However, it was noted that connection to a municipal system was the least costly option and the development was encouraged to continue to pursue the possibility of annexing the area in to the City UGA.

2004 General Sewer Plan

The General Sewer Plan update prepared by Brown & Caldwell evaluated four alternative locations for routing flows from the planned Badger Mountain development into the City's collection system. The planning level flows for the development at the time amounted to a total average daily flow of approximately 1.5 million gallons per day (mgd). The four collection system alternatives evaluated were:

1. Willowbrook Basin: This was essentially the Rachel Road alternative that was identified in the J-U-B Study. A lift station would pump flows through a force main along Clover Road and then gravity flow through an interceptor along Rachel Road. Several existing pipes were identified as needing to be upsized for this alternative.
2. West Gage Basin: This was essentially the fourth alternative identified in the J-U-B Study. A new lift station would pump flows over the ridge to the north of the Badger Mountain development and into the trunk on Gage Boulevard. Several existing pipes were identified as needing to be upsized for this alternative.
3. Dallas Road: This alternative considered a lift station that would pump flows around Badger Mountain to the northwest and connect to the City's system in the Country Estates development. Several existing pipes were identified as needing to be upsized for this alternative.
4. Reata Road: This was essentially the third alternative identified in the J-U-B Study. This alternative considered a lift station that would pump flows south and east along Reata Road and connect to the future Bellerive Lift Station. Several existing pipes were identified as needing to be upsized for this alternative.

The General Sewer Plan provided planning level cost estimates and identified the Dallas Road alternative as the least costly alternative. However, it was noted that the City would be required to correct several existing system deficiencies and this option increases the City's operations and maintenance costs – which were not accounted for in the analysis. The General Sewer Plan also evaluated a fifth alternative – a satellite Membrane Bioreactor (MBR) Reclamation plant. This alternative would provide a remote, modular wastewater treatment plant that would provide highly treated water for reuse. It was identified that a use, such as a golf course or playfields, would need to be found for the reuse water in order for this option to be feasible. In addition, further evaluation would need to account for energy, O & M costs, and effluent disposal.

2006 Badger Mountain/Valley View UGA Expansion Capital Facilities Plan

This study prepared by J-U-B looked at capital facilities impacts of expanding the City of Richland UGA boundary to incorporate the proposed Badger Mountain development area. The plan identified two distinct drainage basins in the planning area – the West Basin and the East Basin and identified that a lift station would be needed for each drainage basin. The sanitary sewer plan identified sewer flows from the West Basin that would be pumped northwest through the Country Heights development (alternative 3 identified in the 2004 General Sewer Plan). The East Basin would be pumped via a lift station over the ridge to the north to Meadow Hills Drive and into the interceptor on Gage Boulevard (alternative 2 from the 2004 General Sewer Plan).

2010 Badger Mountain Sub-Area Plan

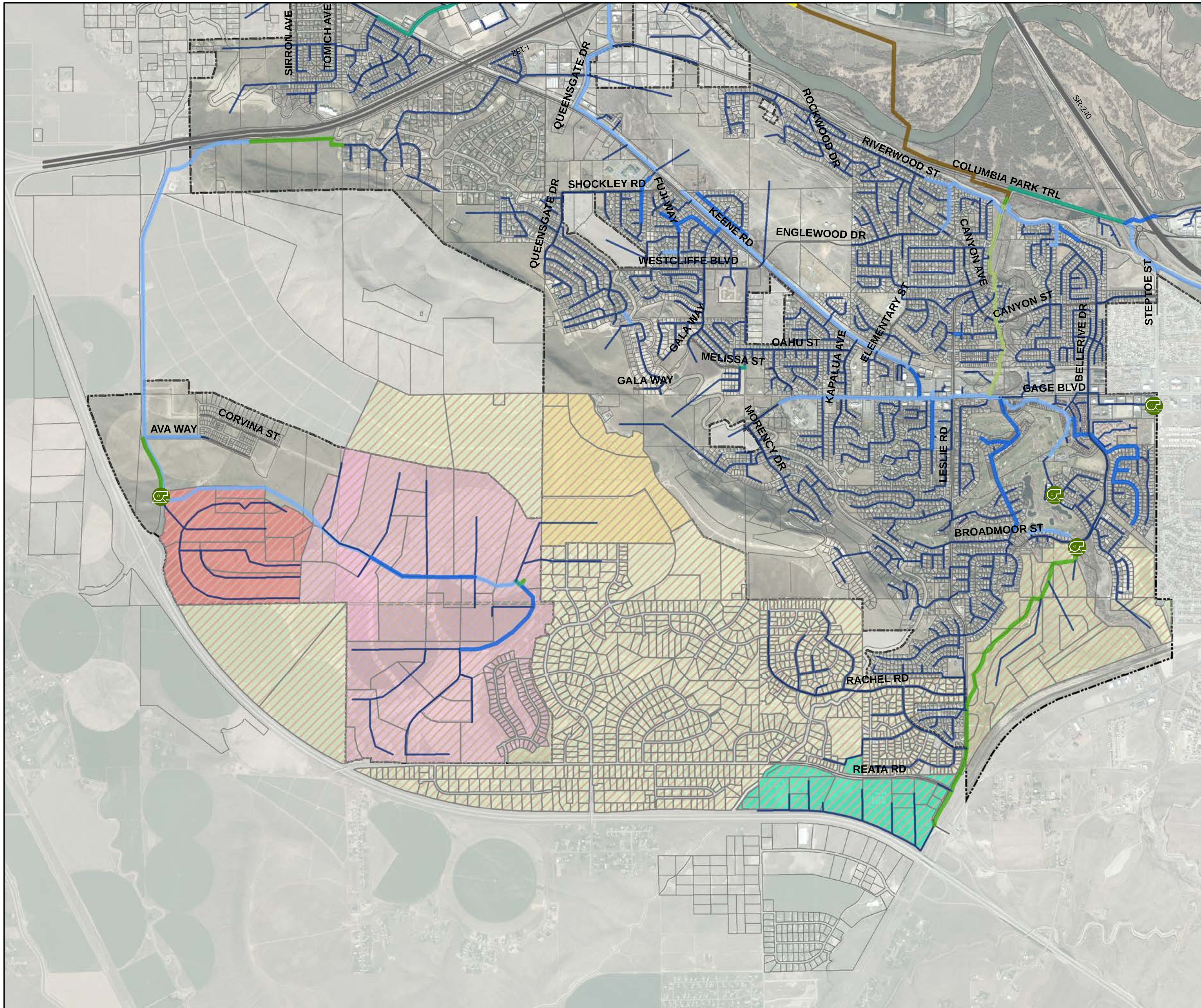
This study prepared by PacWest Engineering developed the master plan for the Badger Mountain development which had become part of the City's UGA at that time. The Plan evaluated three alternatives:

1. West Basin to Country Ridge, East Basin and Wilson Basin to Meadow Hills.
2. West Basin, East Basin, and Wilson Basin all to Meadow Hills.

3. West Basin and East Basin to Country Ridge, and Wilson Basin to Meadow Hills.

Preliminary cost estimates indicated that option 3 was the least cost alternative. However, the costs did not take into the account the relatively high operations and maintenance costs that would be incurred by the City to operate two new, large, regional lift stations. Moreover, the construction cost estimates for the lift stations also appear to be low. Nonetheless, this is the current approved plan for providing sewer service to this area.

This concept as identified in the 2010 Sub-Area Plan has been incorporated into the hydraulic modeling scenarios that are later discussed in this Chapter. There are several off-site improvement projects that will be necessary upon buildout of the Badger Mountain Area – as discussed in subsequent sections.



City of Richland

2015 General Sewer Plan

Richland

Figure 2-7

Badger Mountain Study Area

Legend

- City Limits
- Interstate/Highway
- Major Streets
- City Lift Stations
- Annexation Area
- East-Badger South Dev.
- Part of West-Badger South Dev.
- Wilson Area
- Bellerive LS Basin

Pipe Size (in)

- Collector
- 10
- 12
- 15
- 18
- 21
- 24
- 27
- 30
- 36
- 42
- 54

0 2,500 5,000 Feet

N W E S

Date: Apr 25, 2016

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CHAPTER 3

Flow and Load Analysis

Chapter 3 – Flow and Load Analysis

3.1 Introduction

Influent wastewater to the City of Richland Wastewater Treatment Plant (WWTP) currently consists primarily of residential and commercial dischargers. The City does have industrial dischargers that are regulated through pre-treatment permits – as discussed in **Section 3.5** of this chapter. Data from January 2010 through December 2014 were used for this analysis. Definitions and descriptions of the averaging periods used in this analysis are as follows:

- **Average Day**: The average annual flow rate observed at the facility in a given year. (e.g., total flow for a year divided by 365 days). The average rate is used to estimate annual average pumping and chemical costs, solids production, and organic loading rates.
- **Maximum 3-Month**: The maximum average expected flow or load for three consecutive months in a given year. This condition is typically used to determine when planning for facility upgrades needs to begin (i.e., when this value reaches 85 percent of design capacity).
- **Maximum Month**: The expected flow or load for the peak month in a given year. This condition is typically used to design unit processes for permit compliance.
- **Peak Day**: The expected flow or load for the peak day in a given year. The peak day condition is used to size processes for peak events occurring over a 24-hour period.
- **Peak Hour**: The expected condition occurring during the peak hour in a given year. The peak hour conditions are used to size processes for peak events (e.g. pump stations, oxygen demand).
- **Peaking Factors**: Ratios of maximum events to average events (e.g., a maximum month peaking factor is obtained by dividing the maximum month value for a selected parameter by a baseline value, typically the average day value).

3.2 Existing Influent WWTP Flow & Loads

3.2.1 Flows

Total flow from the City of Richland is measured on the discharge side of the influent pumps with a Panametrics 868 Transient Time Meter. The average day, maximum 3-month average, maximum month, and peak day influent flow for January 2010 through December 2014 are summarized in **Table 3-1**. The daily and monthly average influent flow are shown in **Figure 3-1**.

Table 3-1 – Flow Summary by Year (2010 – 2014)

Item	2010	2011	2012	2013	2014	Probable Existing
Average Day Flow (mgd)	5.76	5.90	5.62	5.48	5.69	5.69 [(a)]
Maximum 3-Month Flow (mgd)	6.12	6.20	5.94	5.72	5.96	6.20 [(b)]
Peaking Factor	1.06	1.05	1.06	1.04	1.05	1.09 [(c)]
Maximum Month Flow (mgd)	6.19	6.25	6.00	5.84	6.07	6.25 [(b)]
Peaking Factor	*1.08*	*1.06*	*1.07*	*1.07*	*1.07*	*1.10* [(c)]
Peak Day Flow (mgd)	7.50	7.34	6.18	7.08	6.90	7.50 [(b)]
Peaking Factor	*1.30*	*1.24*	*1.10*	*1.29*	*1.21*	*1.32* [(c)]
Peak Hour Flow (mgd)	--	--	--	9.41 [(d)]	--	9.41
Peaking Factor	--	--	--	*1.72*	--	*1.65* [(c)]

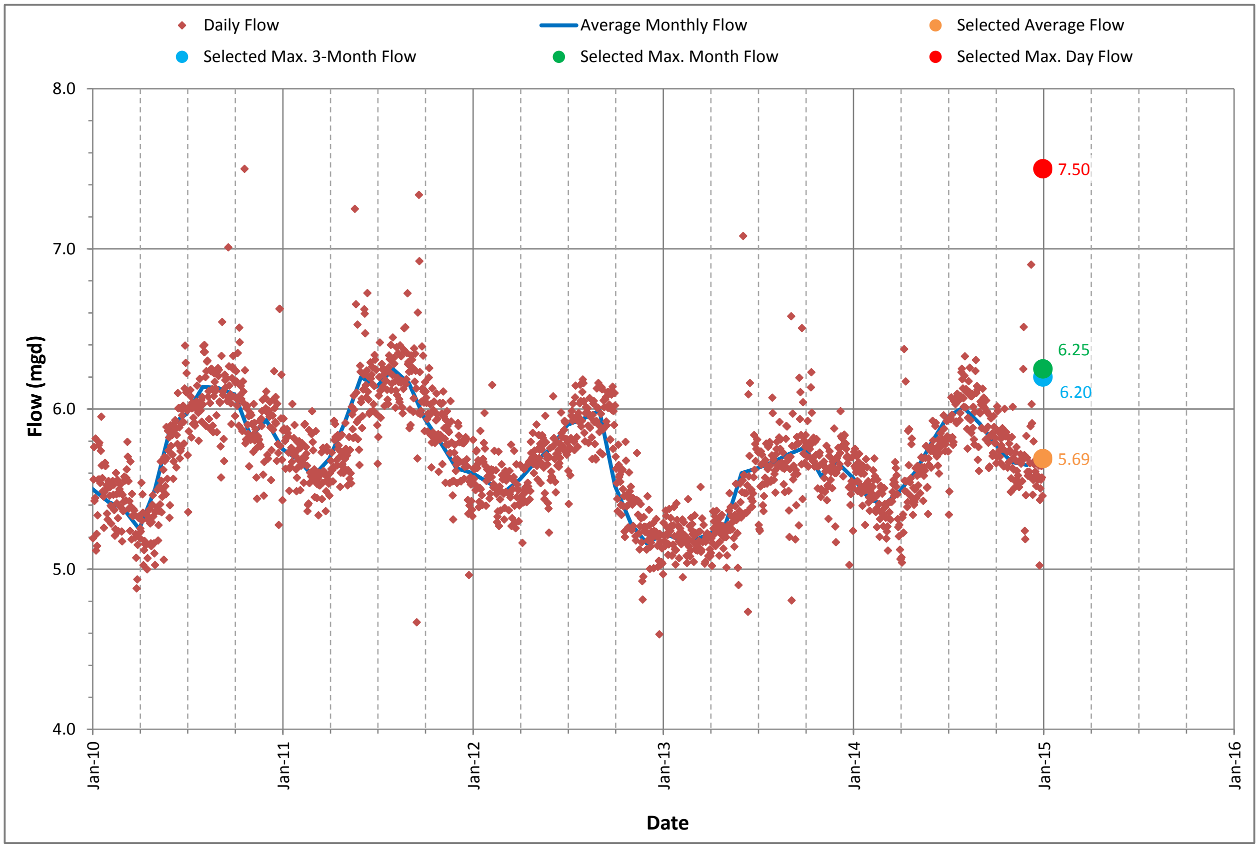
(a) *Selected as the weighted average of data for January 2010 through December 2014.*

(b) *Selected as the observed maximum of the data for January 2010 through December 2014.*

(c) *The peaking factor is calculated as the observed maximum divided by the annual average day condition.*

(d) *Based on hourly flow data available for calendar year 2013, excluding June and September due to construction at the WWTP.*

Figure 3-1 – Flow Summary (2010 – 2014)



The seasonal response of the WWTP flow is likely due to varying degrees of infiltration throughout the year. Higher infiltration rates in late summer and early fall are common in this area and are attributable to irrigation effects. The decrease in flows from 2011 to 2014 is likely due to the ongoing rehabilitation and replacement projects performed each year by the City. A probable existing average flow value of 5.69 mgd was selected for the City of Richland based on the average of average day values for the period of January 2010 through December 2014.

As noted in **Section 2.11**, the 2015 population estimate is 53,054. This results in approximately 107 gallons per capita day (gpcd) using a yearly gross average flow of 5.69 mgd (this does not exclude nonresidential flows). However, during the winter of 2013, flows dropped to approximately 5.20 mgd, or 98 gpcd. The flow per day is slightly higher than a typical range or 50-90 gpcd and is indicative of moderate, year-round infiltration. Infiltration is further discussed in **Section 3.4.**

3.2.2 Biochemical Oxygen Demand (BOD)

The average day, maximum 3-month average, maximum month, and peak day BOD loading for January 2010 through December 2014 are summarized in **Table 3-2**. The daily and monthly average BOD are shown in **Figure 3-2.**

Table 3-2 – BOD Summary by Year (2010 – 2014)

Item	2010	2011	2012	2013	2014	Probable Existing
Average Day Concentration (mg/L)	236	213	243	251	217	232 [(a)]
Average Day Loading (ppd)	11,405	10,503	11,445	11,456	10,352	11,032 [(a)]
Maximum 3-Month Loading (ppd)	12,077	11,410	12,355	13,238	11,373	13,238 [(b)]
Peaking Factor	*1.06*	*1.09*	*1.08*	*1.16*	*1.10*	*1.20* [(c)]
Maximum Month Loading (ppd)	12,847	11,854	14,099	13,802	12,536	14,099 [(b)]
Peaking Factor	*1.13*	*1.13*	*1.23*	*1.20*	*1.21*	*1.28* [(c)]
Peak Day Loading (ppd)	15,093	14,792	25,154 [(d)]	18,870	14,337	18,870 [(b)]
Peaking Factor	*1.32*	*1.41*	*2.20*	*1.64*	*1.29*	*1.71* [(c)]

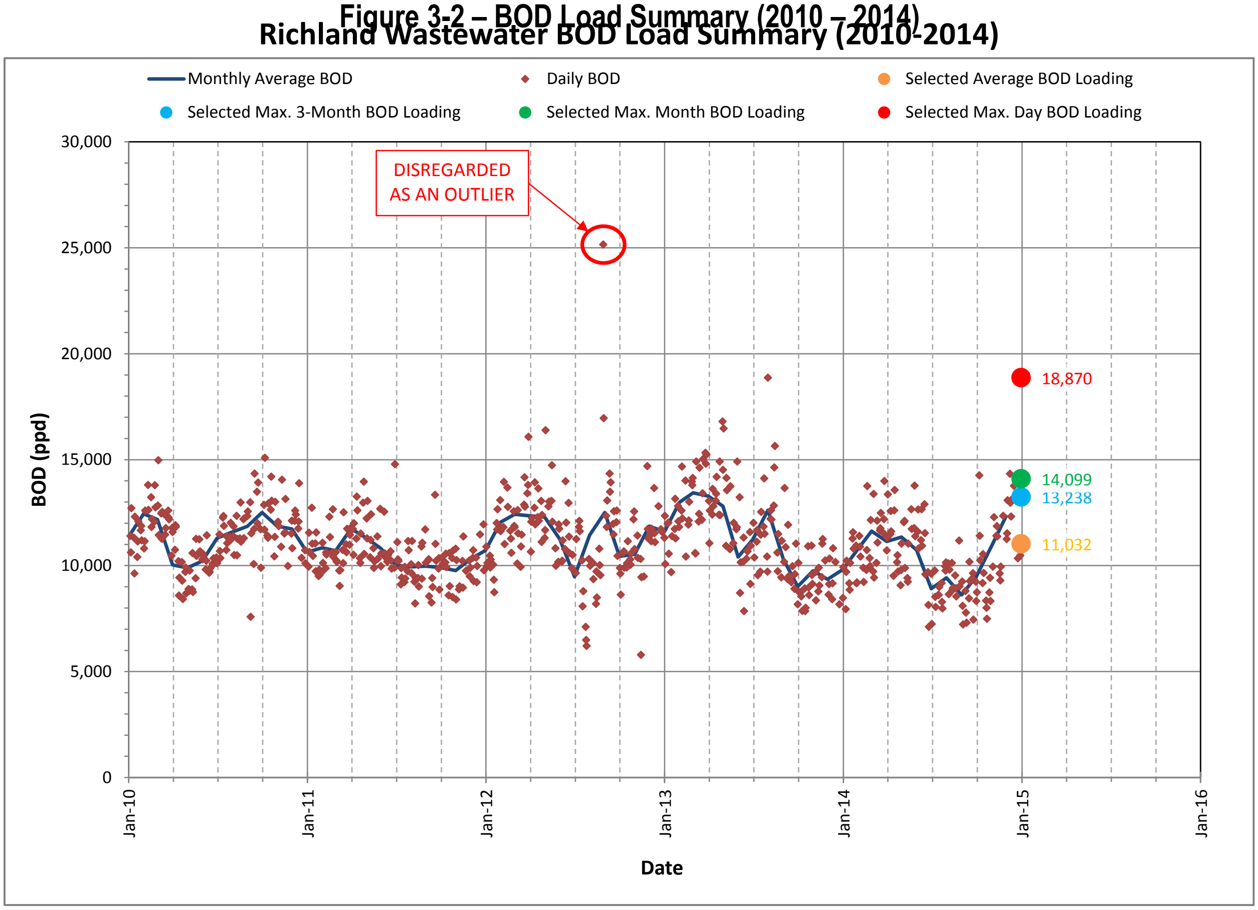
(a) *Selected as the weighted average of data for January 2010 through December 2014.*

(b) *Selected as the observed maximum of the data for January 2010 through December 2014, excluding outliers.*

(c) *The peaking factor is calculated as the observed maximum divided by the annual average day condition.*

(d) *Disregarded as an outliner.*

Figure 3-2 – BOD Load Summary (2010 – 2014)



A probable existing average value for influent BOD loading is 11,032 ppd for a period of January 2010 through December 2014. The BOD loading shows day-to-day variations, but a relatively consistent monthly pattern. However, closer review shows a reverse correlation between BOD and flow (i.e., higher BOD loading is recorded during periods of low flow). This was also brought up as an anomaly in the previous General Sewer Plan. Therefore, the accuracy of the sampling data was called into question. The City noticed in July 2014 that water appeared to be stagnating in the sampling channel during periods of low flow. They subsequently made adjustments to maintain a more steady flow through the channel during low-flow conditions and influent BOD dropped noticeably for the period of July 2014 through October 2014, indicating the City likely discovered a sampling issue at the WWTP that was causing erratic influent data. However, BOD loading increased from October through December 2014, similar to previous years\. The increase in loadings during the fall could be attributed to industrial flows – primarily those of wineries. Continued monitoring and assessment of influent conditions for at least one full calendar year (preferably longer) is recommended to ascertain potential seasonal fluctuations and the true impact of this sampling change. Therefore, probable existing values will be based on influent data from January 2010 through December 2014. Probable plant loading can be revisited, and possibly adjusted, if future data indicates a change is warranted.

An average BOD loading of 11,032 ppd equates to 0.21 pounds per capita per day (ppcd) using an estimated 2015 population of 53,054. This is within the typical range of 0.11 to 0.26 ppcd expected for residential loading (Metcalf and Eddy). The corresponding average BOD concentration over the same time period (i.e. January 2010 through December 2014) is 232 milligrams per liter (mg/L), which is within the typical range of 133 to 400 mg/L reported for domestic wastewater (Metcalf and Eddy). This information is summarized in **Table 3-3**.

Table 3-3 – Selected BOD Loading Compared to Literature Values

Item	City of Richland	Typical Value
Average Day Loading per Capita (ppcd)	0.21	0.11 to 0.26 [(a)]
Average Day Concentration (mg/L)	232	133 to 400 [(b)]

[(a)] *Table 3-13 (page 216), Metcalf and Eddy, 5th Edition*

[(b)] *Table 3-18 (page 221), Metcalf and Eddy, 5th Edition*

3.2.3 Total Suspended Solids (TSS)

The average day, maximum 3-month average, maximum month, and peak day TSS loading for January 2010 through December 2014 are summarized in **Table 3-4**. The daily and monthly average TSS values are shown in **Figure 3-3**.

Table 3-4 – TSS Summary by Year (2010 – 2014)

Item	2010	2011	2012	2013	2014	Probable Existing
Average Day Concentration (mg/L)	269	237	295	298	255	270 (a)
Average Day Loading (ppd)	13,016	11,777	13,952	13,635	12,177	12,911 (a)
Maximum 3-Month Loading (ppd)	13,673	13,199	14,950	16,547	13,797	16,547 (b)
Peaking Factor	*1.05*	*1.12*	*1.07*	*1.21*	*1.13*	*1.28 (c)*
Maximum Month Loading (ppd)	15,822	14,846	16,134	18,146	16,256	18,146 (b)
Peaking Factor	*1.22*	*1.26*	*1.16*	*1.33*	*1.34*	*1.41 (c)*
Peak Day Loading (ppd)	37,339 (d)	21,729	21,297	25,157	23,105	25,157 (b)
Peaking Factor	*2.87*	*1.85*	*1.53*	*1.84*	*1.90*	*1.95 (c)*

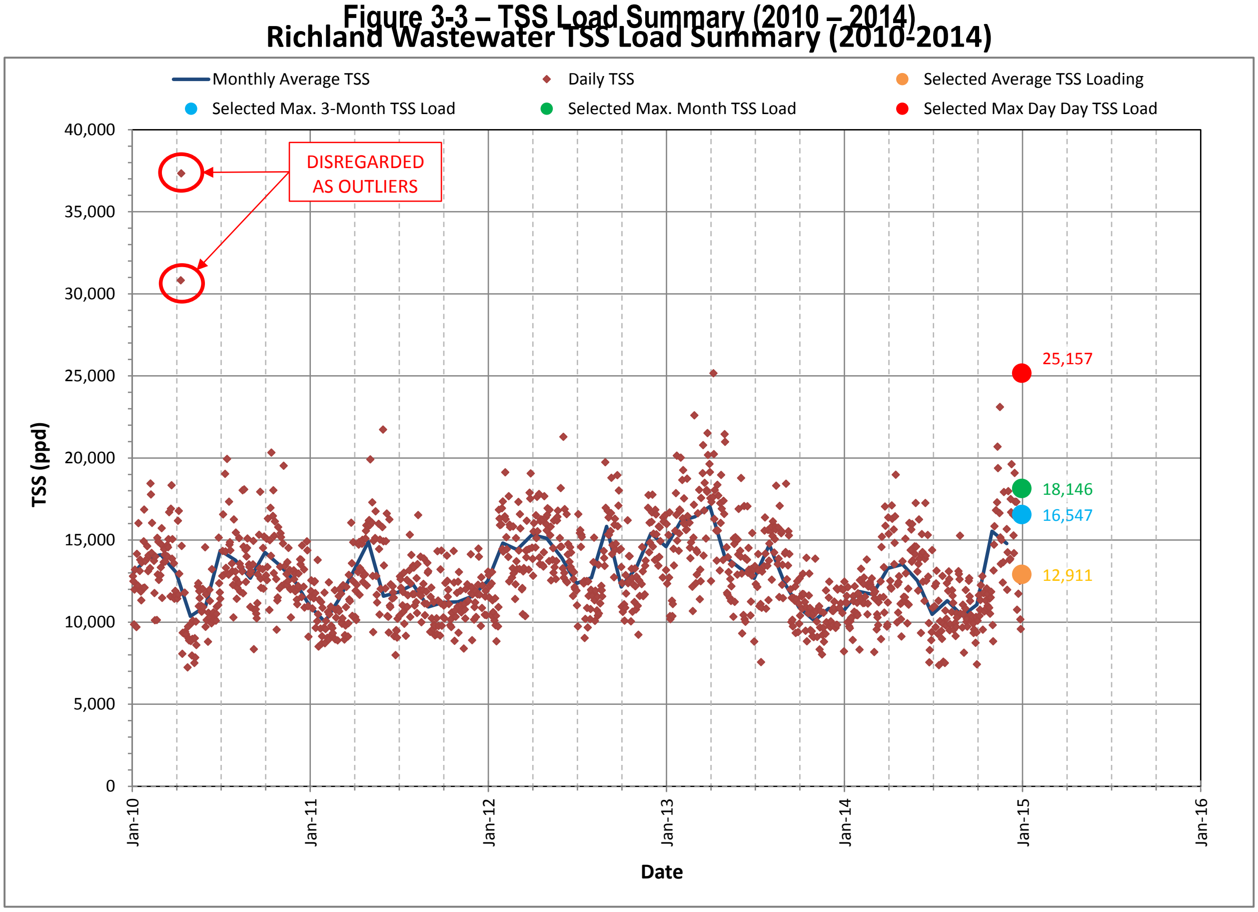
(a) *Selected as the weighted average of data for January 2010 through December2014.*

(b) *Selected as the observed maximum of the data for January 2010 through December 2014, excluding outliers.*

(c) *The peaking factor is calculated as the observed maximum divided by the annual average day condition.*

(d) *Disregarded as an outliner.*

Figure 3-3 – TSS Load Summary (2010 – 2014)



A probable existing average value for TSS loading is 12,911 ppd for a period of January 2010 through December 2014. The TSS loading shows day-to-day variations, but a relatively consistent monthly pattern with a slight upward trend at the beginning of each year. Similar to BOD, the City noticed an unusual correlation between flows and loads during low-flow periods. There was a drop in influent loading values in July 2014 after the influent sampling process was adjusted, and an increase in TSS loading from October through December 2014. Therefore, the probable TSS influent loading, like influent BOD, will be based on influent data from January 2010 through December 2014. Probable plant loading can be revisited, and possibly adjusted, if future data indicates a change is warranted.

An average TSS loading of 12,911 ppd equates to 0.24 ppcd using an estimated 2015 population of 53,054. This is within the typical range of 0.13 to 0.33 ppcd expected for residential loading (Metcalf and Eddy). The corresponding average TSS concentration over the same time period (i.e. January 2010 through December 2014) is 270 mg/L, which is within the typical range of 130 to 389 mg/L reported for domestic wastewater (Metcalf and Eddy). This information is summarized in **Table 3-5**.

Table 3-5 – Selected TSS Loading Compared to Literature Values

Item	City of Richland	Typical Value
Average Day Loading per Capita (ppcd)	0.24	0.13 to 0.33 (a)
Average Day Concentration (mg/L)	270	130 to 389 (b)

(a) *Table 3-13 (page 216), Metcalf and Eddy, 5th Edition*

(b) *Table 3-18 (page 221), Metcalf and Eddy, 5th Edition*

3.2.4 Total Kjeldahl Nitrogen (TKN)

The City of Richland WWTP currently collects weekly samples for influent ammonia. Influent ammonia levels for the period January 2010 through December 2014 ranged from 10.0 to 34.0 mg/L, with an average value of 18.1 mg/L. In comparison to typical literature values, this represents a low- to medium-strength wastewater. Unlike BOD and TSS, the July 2014 influent sampling process change at the WWTP does not seem to have affected influent ammonia data.

Influent Total Kjeldahl Nitrogen (TKN) is typically used for process design, nutrient balances, oxygen demand rates, etc., but this data is currently unavailable. Therefore, the influent ammonia values were converted to total nitrogen using a ratio of typical literature values for medium-strength wastewater (i.e., a ratio of 1.75 based on 35 mg/L of TKN to 20 mg/L of ammonia) (Table 3-18, page 221, Metcalf & Eddy 5th Edition). The estimated influent TKN loading, based on the assumed factor of 1.75 to observed ammonia data, are given in **Table 3-6**. The resulting average concentration is slightly lower than typical values, but likely reflects infiltration occurring in the collection system as noted in **Section 3.4**. The daily and monthly average TKN are shown in **Figure 3-4**.

The assumed TKN values should be revisited and replaced with actual values if TKN data is collected for the WWTP.

Table 3-6 – Probable Existing TKN Loading (2010 – 2014)

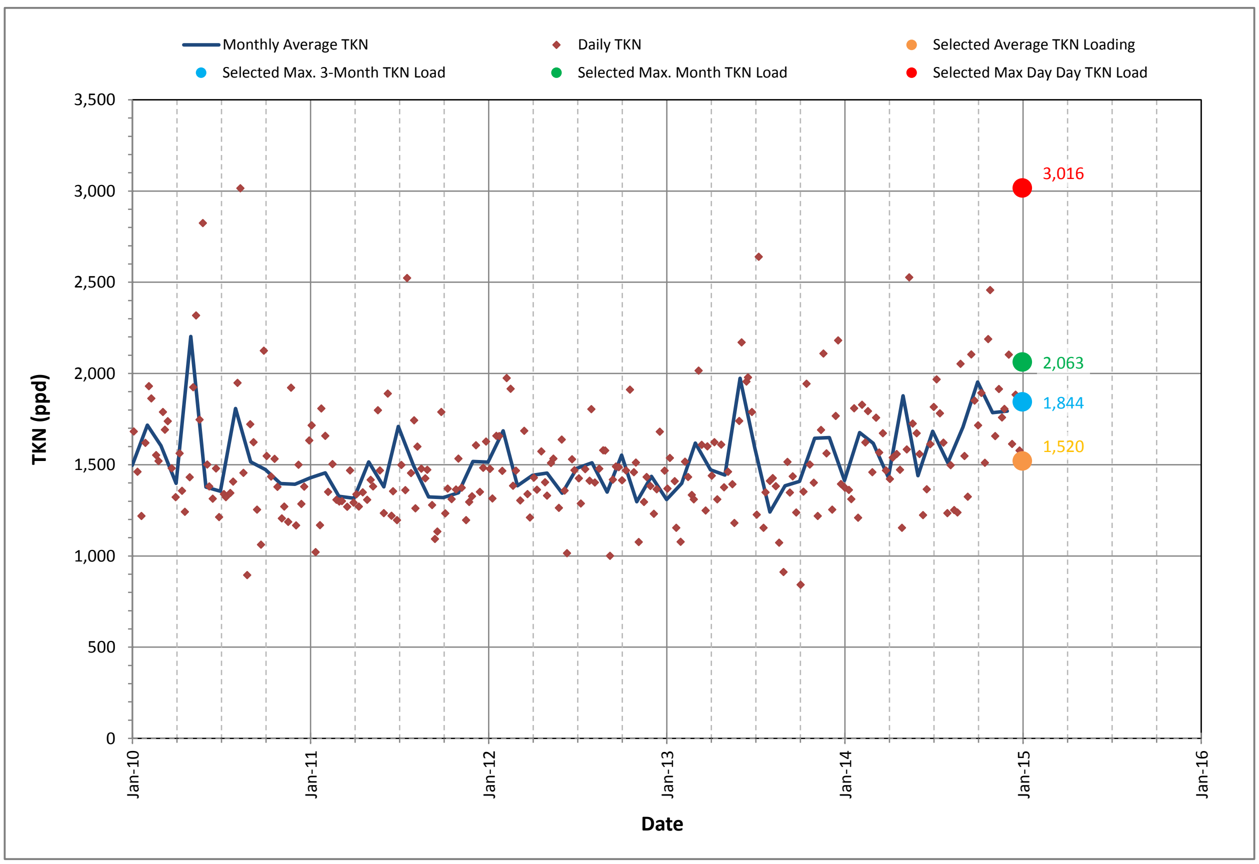
Item	2010	2011	2012	2013	2014	Probable Existing
Average Day Concentration (mg/L)	32.0	28.7	31.0	32.5	34.8	31.8 (a)
Average Day Loading (ppd)	1,560	1,426	1,458	1,500	1,654	1,520 (a)
Maximum 3-Month Loading (ppd)	1,735	1,535	1,572	1,670	1,844	1,844 (b)
Peaking Factor	*1.11*	*1.08*	*1.08*	*1.11*	*1.11*	*1.21* (c)
Maximum Month Loading (ppd)	2,063	1,716	1,734	1,927	1,996	2,063 (b)
Peaking Factor	*1.32*	*1.20*	*1.19*	*1.29*	*1.21*	*1.36* (c)
Peak Day Loading (ppd)	3,016	2,524	1,975	2,640	2,526	3,016 (b)
Peaking Factor	*1.93*	*1.77*	*1.35*	*1.76*	*1.53*	*1.98* (c)

(a) *Selected as the weighted average of data for January 2010 through October 2014.*

(b) *Selected as the observed maximum of the data for January 2010 through October 2014.*

(c) *The peaking factor is calculated as the observed maximum divided by the annual average day condition.*

Figure 3-4 – TKN Load Summary (2010 – 2014)



A probable existing average value of 1,520 ppd for influent TKN loading for the City of Richland was selected based on the average of average day values for the period of January 2010 through December 2014. This equates to 0.029 ppcd using an estimated 2015 population of 53,054. This is within the typical range of 0.020 to 0.040 ppcd expected for residential loading (Metcalf and Eddy). The corresponding average TKN concentration over the same time period (i.e. January 2010 through December 2014) is 31.8 mg/L, which is within the typical range of 23 to 69 mg/L reported for domestic wastewater (Metcalf and Eddy). This information is summarized in **Table 3-7**.

Table 3-7 – Selected TKN Loading Compared to Literature Values

Item	City of Richland	Typical Value
Average Day Loading per Capita (ppcd)	0.029	0.020 to 0.040 [a]
Average Day Concentration (mg/L)	31.8	23 to 69 [b]

[a] *Table 3-13 (page 216), Metcalf and Eddy, 5th Edition*

[b] *Table 3-18 (page 221), Metcalf and Eddy, 5th Edition*

3.2.5 Total Phosphorus (TP)

Influent Total Phosphorus (TP) loading data is currently unavailable for the City of Richland WWTP. Therefore, existing phosphorus loadings will be based on typical literature values (Metcalf and Eddy), as summarized in **Table 3-8.** These values should be confirmed with sampling prior to detailed design.

Table 3-8 – Probable Existing Phosphorus Loading Conditions

Parameter	Value		Current Average Day [c] (ppd)
	Range [a] (mg/L)	Typical [b] (mg/L)	
Total Phosphorus	4-11	6	285

[a] *Table 3-18 (page 221), Metcalf and Eddy, 5th Edition*

[b] *A lower typical value was selected to account for impacts from inflow and infiltration.*

[c] *Based on a current average day flow of 5.69 mgd*

Typical literature values (Metcalf and Eddy) for peaking factors are recommended until sufficient data is collected to define the phosphorus influent loading variability. The maximum month and peak day peaking factors are 1.25 and 1.75, respectively.

3.2.6 Summary of Current Flows and Loads

The existing flow and load data presented above are summarized in **Table 3-9**.

Table 3-9 – Existing Flows and Loads Summary

Item		Value
Flow (mgd)	Average Day	5.69
	Maximum 3-Month	6.20
	Peaking Factor	1.09
	Maximum Month	6.25
	Peaking Factor	*1.10*
	Peak Day	7.50
	Peaking Factor	*1.32*
	Peak Hour	9.41
	Peaking Factor	*1.65*
BOD (ppd)	Average Day	11,032
	Maximum 3-Month	13,238
	Peaking Factor	1.20
	Maximum Month	14,099
	Peaking Factor	*1.28*
	Peak Day	18,870
	Peaking Factor	*1.71*
TSS (ppd)	Average Day	12,911
	Maximum 3-Month	16,547
	Peaking Factor	1.28
	Maximum Month	18,146
	Peaking Factor	*1.41*
	Peak Day	25,157
	Peaking Factor	*1.95*
TKN (ppd)	Average Day	1,520
	Maximum 3-Month	1,844
	Peaking Factor	1.21
	Maximum Month	2,063
	Peaking Factor	*1.36*
	Peak Day	3,016
	Peaking Factor	*1.98*
TP (ppd)	Average Day	285
	Maximum Month	356
	Peaking Factor	*1.25* [a]
	Peak Day	499
	Peaking Factor	*1.75* [a]

(a) *Per typical literature values*

3.3 Projected Flow and Loads for Year 2035

The Benton County Comprehensive Plan lists a projected 2035 population of 76,533 people for the City of Richland. Based on an estimated population in 2015 of 53,054, this results in a growth rate of approximately 1.849 percent per year over the planning period. The average day flow and loading for 2035 was projected based on the estimated growth rate. Maximum month, peak day, and peak hour conditions were estimated based on observed peaking factors noted previously.

The corresponding projected flows and loads for 2035 are summarized in **Table 3-10**. Projected flows are shown in **Figure 3-5**, projected BOD loading is shown in **Figure 3-6**, projected TSS loading is shown in **Figure 3-7**, and projected TKN loading is shown in **Figure 3-8**.

Table 3-10 – Projected Flows and Loads for 2035

Item		Value
Flow (mgd)	Average Day	8.21
	Maximum 3-Month	8.95
	Peaking Factor	1.09
	Maximum Month	9.03
	Peaking Factor	*1.10*
	Peak Day	10.83
	Peaking Factor	*1.32*
	Peak Hour	13.54
	Peaking Factor	*1.65*
BOD (ppd)	Average Day	15,910
	Maximum 3-Month	19,090
	Peaking Factor	1.20
	Maximum Month	20,360
	Peaking Factor	*1.28*
	Peak Day	27,210
	Peaking Factor	*1.71*
TSS (ppd)	Average Day	18,620
	Maximum 3-Month	23,830
	Peaking Factor	1.28
	Maximum Month	26,250
	Peaking Factor	*1.41*
	Peak Day	36,310
	Peaking Factor	*1.95*
TKN (ppd)	Average Day	2,190
	Maximum 3-Month	2,650
	Peaking Factor	1.21
	Maximum Month	2,980
	Peaking Factor	*1.36*
	Peak Day	4,340
	Peaking Factor	*1.98*
TP (ppd)	Average Day	411
	Maximum Month	514
	Peaking Factor	*1.25* [a]
	Peak Day	719
	Peaking Factor	*1.75* [a]

[a] *Per typical literature values*

Figure 3-5 – Flow Projection (2035)

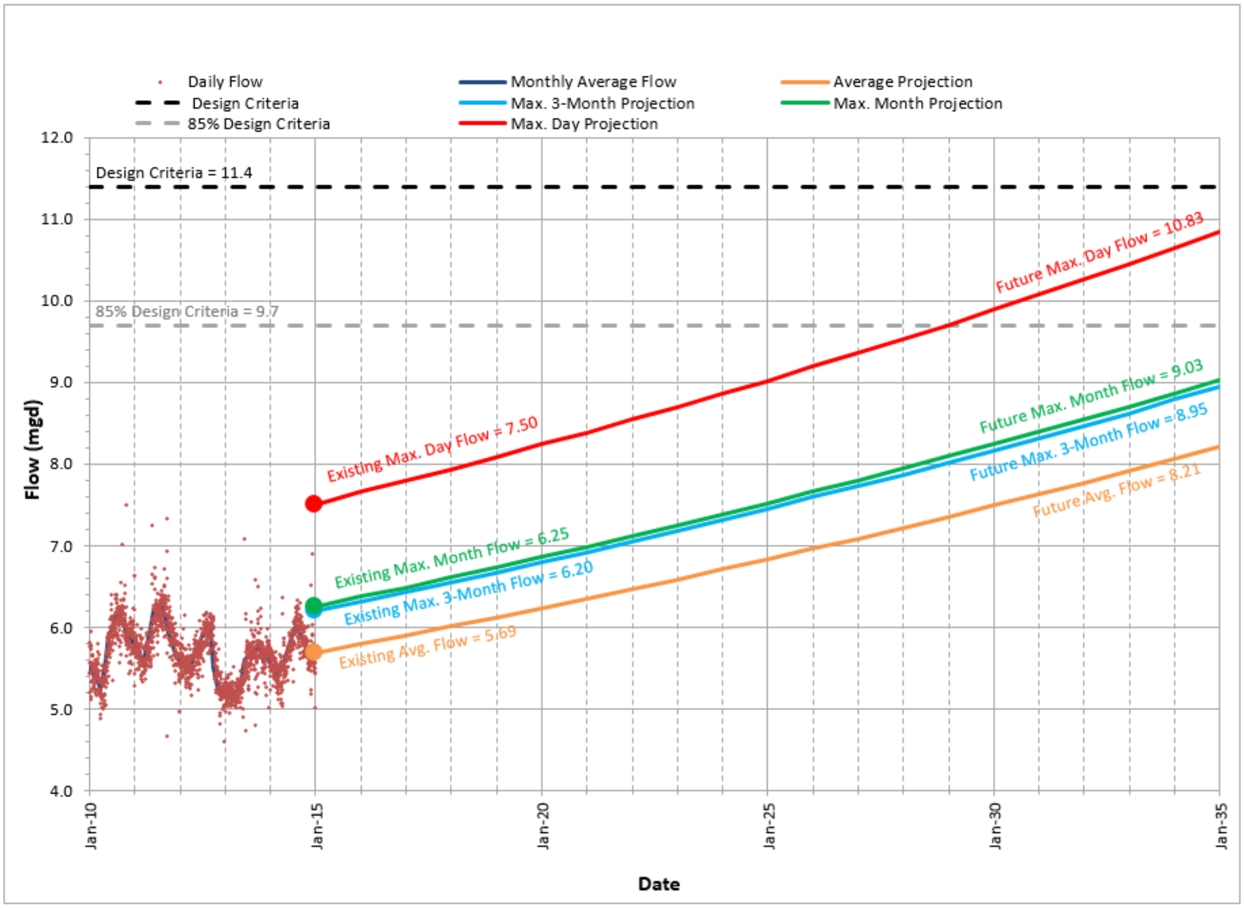


Figure 3-6 – BOD Loading Projection (2035)

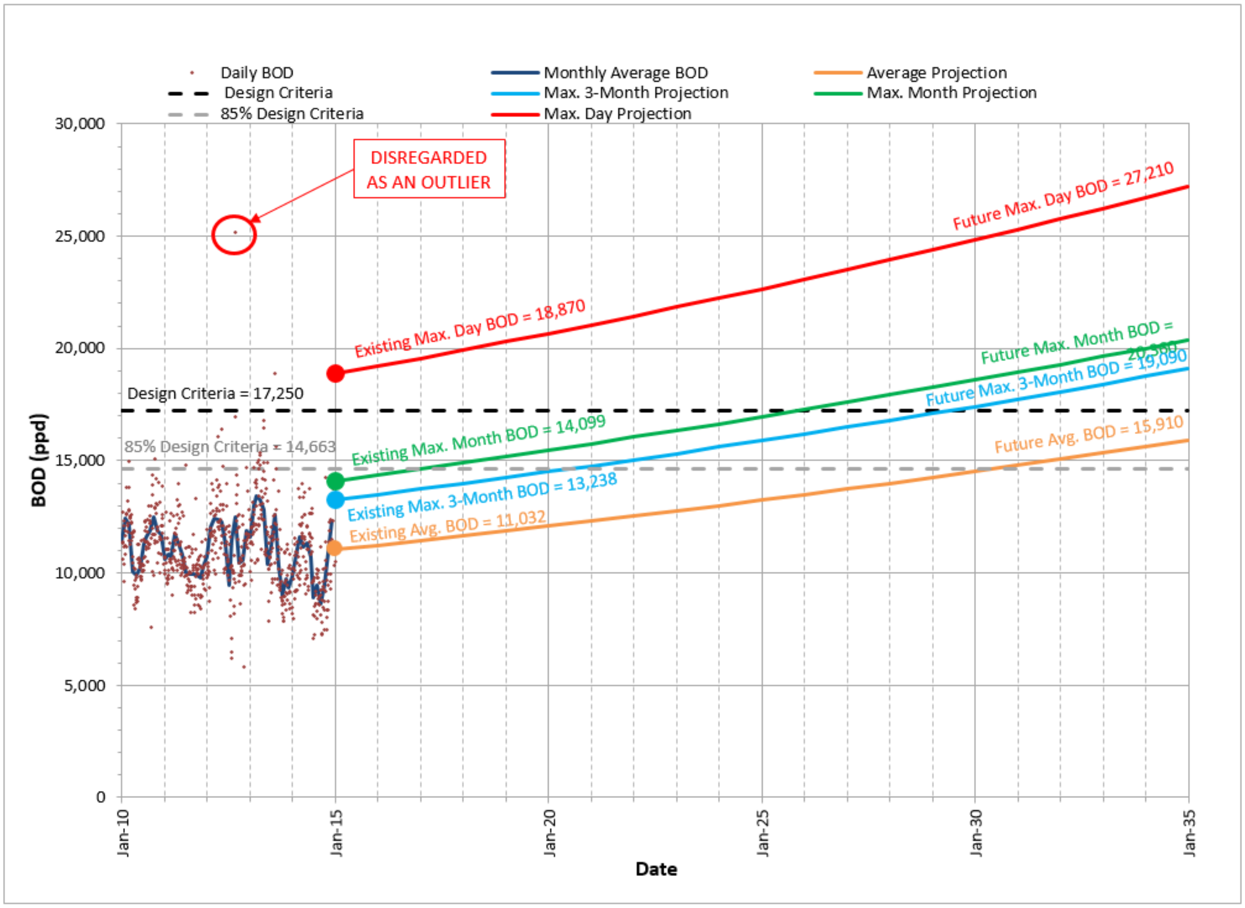


Figure 3-7 – TSS Loading Projection (2035)

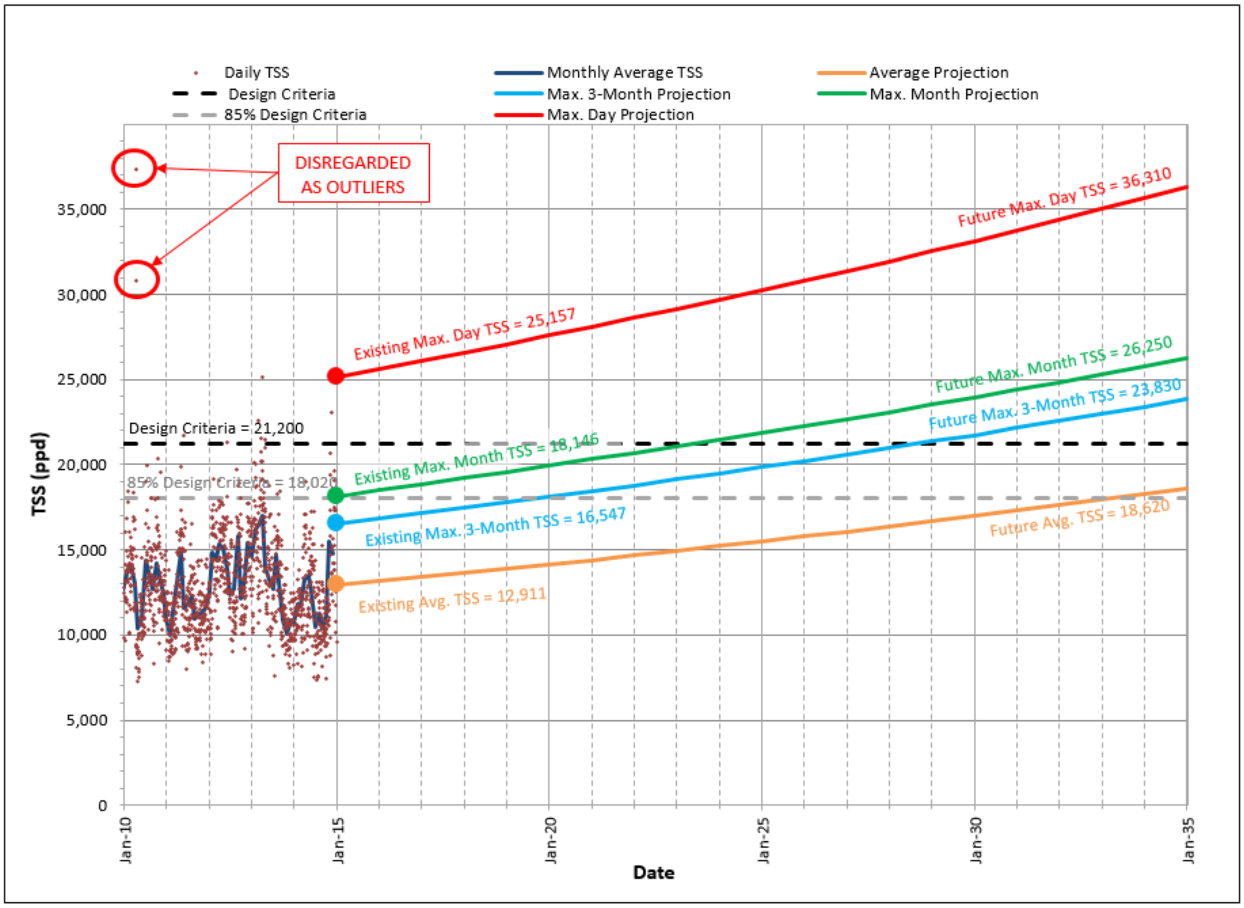
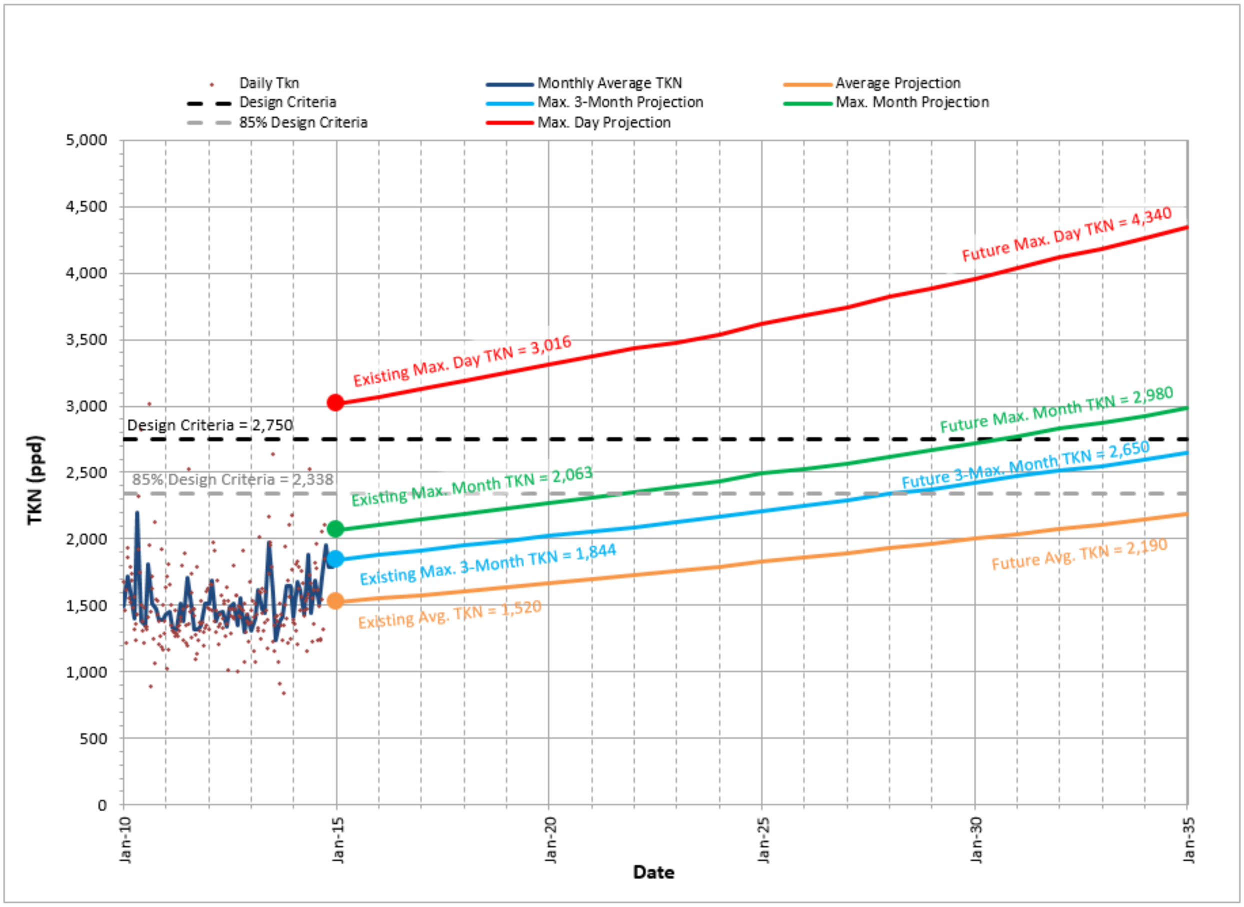


Figure 3-8 – TKN Loading Projection (2035)



3.4 Summary of Flow Contributions and Sources

The City's water service meter billing data from December 2012 through February 2013 was utilized in order to estimate the amount of sewage generated from each parcel within the service area. The parcels were classified according to land use and the summary is provided in **Table 3-11** below.

Table 3-11 – Wastewater Sources & Estimated Flow Contribution (December 2012 – February 2013)

Category	Average Flow (mgd)	Percent of Total
Residential (a)	2.94	49%
Commercial (b)	0.50	8%
Industrial	0.50	8%
Schools (c)	0.07	1%
Other (d)	0.02	<1%
Infiltration (e)	1.91	32%
TOTAL	5.94 (f)	100%

(a) *Includes Low, Medium and High Density Residential land use types, RV and Mobile Home Parks and Assisted Living Facilities.*

(b) *Includes commercial industries, hospitals and hotels*

(c) *Includes colleges/universities, and elementary/middle/high schools*

(d) *"Other" refers to City-owned parks, green spaces, and related facilities*

(e) *Infiltration from calibrated sewer collection system hydraulic model – See Appendix C, Model Assumptions, for more details*

(f) *Average Flow Total is based on the sum of all categories*

The total average flow in **Table 3-11** is based on the sum of all the wastewater categories, including infiltration. This value is slightly greater than the observed WWTP flows shown in **Figure 3-1**, during the same time period, and is a result of calibrating the hydraulic model to individual flow monitor locations throughout the collection system and not directly to the WWTP.

The flow contribution from Residential was further evaluated to identify the unit flow for a single family residence. Based upon the total flow for the Low Density Residential land use type and the amount of single family listings, it was found that that the average daily flow for a single family residence is 160 gpd.

3.5 Large Non-Residential Flows

Industrial and commercial establishments discharging into the City's collection system include: printers, photographic processors, dental and medical facilities, university facilities, industrial laundry facilities, dry cleaners, chemical/biological testing and research laboratories, radiator repair and auto body shops, federal contractors, pesticide applicators, and a nuclear fuel rod manufacturer.

Based on water meter records from winter of 2013, the largest users are presented in **Table 3-12**.

Table 3-12 – Largest Water Users (December 2012 – February 2013)

User	Average Flow (mgd)	Type	Description
Lamb Weston[(a)]	0.52	Industrial	Food Processor
Ingredion	0.15	Industrial	Food Processor
Kadlec Hospital	0.06	Commercial	Medical/Hospital
The Hills Mobile Home Park	0.05	Residential	Residential
ATI – ALLVAC Metal Fabrication	0.04	Industrial	Metal Fabricator
Areva	0.04	Industrial	Nuclear Materials
Richland Mobile Home Park	0.03	Residential	Residential
Red Lion Hotel	0.03	Commercial	Hotel
Washington Closure Hanford	0.03	Industrial	Laboratory/Research
Richland Rehabilitation Center	0.03	Residential	Assisted Living
US Linen	0.03	Industrial	Industrial Laundry
WWTP	0.02	--	City Facility
Alyson Manor Estates	0.02	Residential	Assisted Living
Shilo Inn	0.02	Commercial	Hotel
Battelle	0.02	Industrial	Laboratory/Research
Alterra Assisted Living	0.02	Residential	Assisted Living

(a) *Does not discharge to City sewer system.*

There are currently eleven Significant Industrial Users (SIUs) that are permitted by the City to discharge to the City system:

- Battelle – R & D Lab
- Ingredion (formally Penford Food Ingredients)– Food Processing
- US Linen – Industrial laundry
- Unitech – Nuclear laundry
- Environmental Molecular Sciences Lab – R & D Lab
- Applied Process Engineering Lab – R & D Lab
- ATI-ALLVAC – Titanium Refinery
- AREVA – Nuclear Fuels Manufacturer
- Bioproducts, Science, and Engineering Lab – R & D/Teaching Lab
- 300 Area – R & D Lab
- Physical Sciences Facility – R & D Lab

All flows and loads are projected to grow at the same 1.85% growth rate as projected for the population. No separate growth rates were identified for non-residential flows. The WWTP planning documents in the early 2000s include provisions for a large food processor; however, no such provisions are incorporated into this Plan. The impact of any potential large industrial dischargers should be evaluated on a case by case basis.

3.6 Infiltration and Inflow

Infiltration is the term for groundwater that enters the system through faulty joints, cracks, and service connections as well as through illegal connections of irrigation overflows and foundation drains. Inflow accounts for water that enters the system during a storm event through manhole lids and miscellaneous connections to roof drains and storm drainage structures. Richland experiences a noticeable seasonal variation in infiltration and inflow (I/I) levels that correspond with irrigation season – with peaks occurring in the late summer. The following sources of infiltration have been identified:

- Excessive lawn watering induces percolation into shallow side sewers
- Over-irrigation onto paved areas results in ponding in local drainage ways where it infiltrates
- Perched water tables in areas adjacent to irrigation canals induces infiltration
- The shallow water table in the City's northcentral region (north of McMurray St.), southeasterly region (near the Montana LS) and southcentral region (south of Meadow Springs Golf Course) enters through trunk sewer mains and manholes.

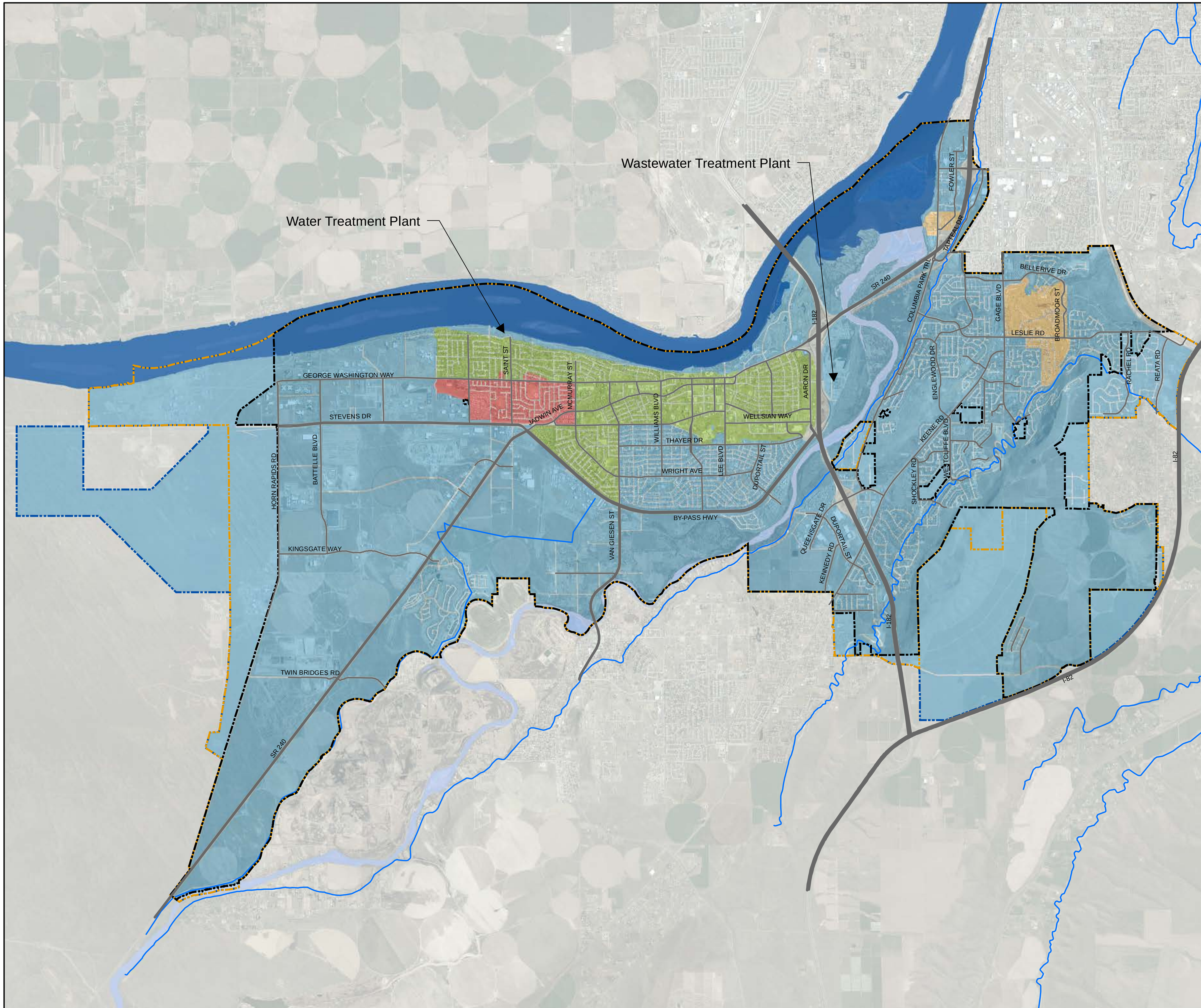
Infiltration and inflow (I/I) affect the sewer system by increasing the volume of flow that must be collected, conveyed, and ultimately treated at the WWTP. This results in reduced efficiency of biological processes and increases the cost of unit processes that are sized based on detention time. Therefore, it is desirable to minimize I/I. The WDOE requires that cities demonstrate that the sewer collection system is not subject to excessive I/I and has established criteria for determining non-excessive I/I.

Special Condition S4.E of the City's NPDES permit requires the annual submission of an I/I Evaluation report. These reports are included in **Appendix O**. This report is a template provided by WDOE that lists average monthly WWTP flows, monthly rainfall amounts, and population served. The difference between the highest and lowest monthly average flow is considered to be the I/I in this report. Although the difference between the highest and lowest monthly average flows indicates a seasonal difference, it does not account for baseline infiltration that may occur throughout the year. As shown in **Table 3-11**, flow monitoring used for calibration of the collection system hydraulic model indicates that infiltration is approximately 1.91 MGD – which is greater than double the 0.83 MGD amount calculated in 2013 using the WDOE template. Therefore, while the WDOE template provides an easy-to-calculate metric that can be used for tracking progress, it is not a true measure of the amount of infiltration in Richland's collection system.

For determining non-excessive infiltration, the City's report references EPA Publication No. 97-03, I/I Analysis and Project Certification. According to the publication, non-excessive infiltration is determined by calculating the average daily flow per capita (excluding major industrial and commercial flows greater than 50,000 gpd). If this value is less than 120 gpcd, the amount of infiltration is considered non-excessive. Using the total average flow listed in **Table 3-11** less the major industrial and commercial flows (totaled as 0.254 mgd) results in a total average flow of 5.69 mgd. Compared to the 2015 population of 53,054, this results in an average daily flow per capita of 107 gpcd, which indicates non-excessive infiltration.

The City has taken aggressive measures to reduce the amount of I/I in recent years. These measures have included inspection of the existing system by both CCTV and manual methods. Based upon the inspections, a prioritized list of rehabilitation projects have been identified which include: storm drainage disconnects, irrigation overflow disconnects, manhole repair/replacement, side service repair, trenchless rehabilitation, and sewer main replacements. This list is designated as the Problems and Maintenance (PM) List and is included in **Appendix K**. Additionally, service area expansion has included gasketed PVC pipe that is pressure tested and inspected by CCTV prior to acceptance. Moreover, care has been taken to ensure that sewer mains are installed within the street right-of-way and outside of areas that are subject to surface water infiltration at the drainage ways. As shown in **Figure 3-9,** calibration of the collection system model indicated there are several areas of relatively high infiltration, while the majority of the system experiences little to no infiltration. The remaining areas believed to be contributing to infiltration seen at the WWTP include the shallow water table in the City's north-central region (north of McMurray St.), southeasterly region (near the Montana LS) and south-central region (south of Gage Boulevard in the Meadow Springs area).

Based on the determination of non-excessive I/I, following the EPA criteria, there is no requirement for the City to engage in a full-scale I/I study. The City should continue its program of flow monitoring, systematically identifying sources of I/I during routine maintenance and inspection, and incorporating repair/replacement projects into the annual budget. However, it would be wise for the City to evaluate addressing the localized areas of medium to high infiltration in an effort to eliminate the nearly 2 mgd of I/I and free up that hydraulic capacity at the WWTP. There is a chance that planned near-term condition rating may preclude the need for this study; however, budget for a future I & I Study has been added to the CIP.



City of Richland

2015 General Sewer Plan

Figure 3-9

Infiltration

Legend

- City Limits
- UGA
- 50-yr Planning
- Interstate/Highway
- Major Streets
- Irrigation Canal/Pipeline
- Columbia River
- Yakima River

Infiltration (gpad)

- None - 0.0
- Low - 175
- Med-High - 1500
- High - 1700

0 6,000 12,000 Feet

N E S W

Date: Apr 1, 2016

J·U·B ENGINEERS, INC.

CHAPTER 4

Performance & Design Criteria

Chapter 4 – Performance and Design Criteria

4.1 City Standard Specifications and Standard Drawings

The City of Richland Public Works Department has developed standards which provide minimum construction criteria for Public Works within the City or for which the City will take ownership. The City maintains a set of Standard Provisions based upon the WSDOT Standard Specifications which define the minimum Sanitary Sewer construction standards, and is supplemented by the Standard Drawings. A copy of the City of Richland Standard Specifications and Standard Drawings for Sanitary Sewer Construction is included in **Appendix N**.

The City has also developed standard specifications and design drawings for submersible sewage lift stations. The Guidelines and Standard Specifications and Details for Sewage Pump Stations is provided in **Appendix N**.

4.2 Collection System Design Criteria for Master Planning

The design criteria listed in **Table 4-1** is to be used for future system planning. The design criteria is based on the City's water meter data that was collected and grouped by land use, between the months of December 2012 and March 2013. Water usage during the months of December-March is a good indication of sewer use because little to no water is used for irrigation. The usage was averaged over these three months to provide an average daily flow based on land use. Additional design criteria and assumptions can be found in **Appendix C**.

Table 4-1 – Collection System Planning Criteria

Parameter	Value
Residential Unit Flows[a]	160 GPDU[b]
Commercial Unit Flows	625 GPAD[c]
Industrial Unit Flows	1,250 GPAD[d]
Manning Pipe Roughness Coefficient	0.012
Minimum sewer velocity	2 feet per second

[a] *Based on 2.42 people per dwelling*
[b] *Gallons per dwelling unit*
[c] *Gallons per acre per day*
[d] *Note that the City of Richland unit flow analysis identified 60 GPAD as average flow for small, dry industries, and 3,000 GPAD as the average flow for large, permitted industries. It was determined that with a large range of industry types and resulting flows, a gross area flow for areas zoned industrial was based on a reference value of 1,250 GPAD, from Wastewater Engineering: Treatment and Reuse, by Metcalf & Eddy.*

4.3 Discharge Standards

4.3.1 Federal Water Quality Standards

The principal authority for the water pollution control programs is the Clean Water Act (33 U.S.C. 1251 et seq.). The aim of the act is to "restore and maintain the chemical, physical, and biological integrity of the nation's waters." This act set forth the following national goals:

- Eliminate the discharge of pollutants into navigable waters by 1985.
- Set interim goals of water quality which will protect fish and wildlife and will provide for recreation by July 1, 1983.
- Prohibit the discharge of pollutants in quantities that might adversely affect the environment.
- Construct publicly owned waste treatment facilities with federal financial assistance.
- Establish waste treatment management plans within each state.
- Establish the technology necessary to eliminate the discharge of pollutants.
- Develop and implement programs for the control of non-point sources of pollution to enable the goals of the act to be met.

These goals were to be achieved by a legislative program which includes permits under the National Pollutant Discharge Elimination System (NPDES). Key provisions of the act include the development of such permit systems and effluent standards as well as state and local responsibilities.

The Clean Water Act emphasizes that state governments are to use the minimum federal standards, guidelines, and goals, and establish individual pollution control programs and enforcement procedures. When the state has completed its programs for waste treatment management, its implementation plans for preserving or restoring water quality, and the Environmental Protection Agency (EPA) has approved those programs, the state assumes enforcement responsibilities. The Washington Department of Ecology (WDOE) has been delegated with these responsibilities by EPA.

4.3.2 Washington State Surface Water Quality Standards (WQS)

The State of Washington's surface water quality standards are given in the Washington Administrative Code (WAC) Chapter 173-201A, the Water Quality Standards for Surface Waters of the State of Washington, and WAC Chapter 173-204, Sediment Management Standards.

WAC 173-201A strives to establish surface water quality criteria which are consistent with public health and public enjoyment, and the propagation and protection of fish, shellfish, and wildlife, pursuant to the provisions of chapter 90.48 of the Revised Code of Washington (RCW). The surface water quality standards establish specific water quality criteria based on the Aquatic Life and Recreational Use designations. Use designations for the Columbia River (river mile 309.3 to 596.6), the reach in which Richland's outfall is located, are defined in WAC 173-201A Table 602 as follows:

- Aquatic Life Uses: Non-Core Salmon/Trout
- Recreational Uses: Primary Contact
- Water Supply Uses: Domestic Water, Industrial Water, Agricultural Water, and Stock Water
- Miscellaneous Uses: Wildlife Habitat, Harvesting, Commerce/Navigation, Boating, and Aesthetics

In accordance with the direction of EPA, WDOE has pursued compliance with surface quality standards based on a watershed management approach. The emphasis of watershed management is to monitor, analyze, and protect water quality on a geographic basis. The watershed management strategy was implemented as a means to:

- Identify and address high priority water quality issues.
- Tie NPDES permit conditions more closely to localized water quality conditions.
- Improve coordination among state, tribal and local environmental programs.
- Target activities to attain state water quality standards.

In 1970, under WAC 173-500-040 and the Water Resources Act of 1971 (RCW 90.54), WDOE partitioned the state into 62 Water Resource Inventory Areas (WRIAs). These WRIAs are the administrative underpinning of WDOE's business activities and provide the framework for the watershed approach which is embodied in the Section 303(d) process. The Columbia River at Richland is located between WRIA 31 (Rock/Glade) and WRIA 36 (Esquatzel Coulee). However, due to its size, the Columbia River Basin is managed as its own watershed area.

In addition, in July 1993, WDOE designated 23 Water Quality Management Areas (WQMAs). The City of Richland is located within WQMA 31 (Horseheaven/Klickitat). These WQMAs are water quality management basins for which coordinated and integrated science, permitting and water pollution control, and prevention measures are implemented to meet State water quality standards.

The WDOE program undertakes the following five activities in each WQMA over a five year, rotating cycle period:

Year 1 Scope water quality.
Year 2 Conduct water quality monitoring and special studies.
Year 3 Analyze water quality and the effects of pollution.
Year 4 Develop technical reports that record water quality, areas of concern, and strategies to respond to these concerns.
Year 5 Issue wastewater discharge permits and implement other pollution prevention and pollution control actions that respond to priority water quality issues.

4.3.2.1 303(d) List

The Federal Clean Water Act (Section 303(d)) and federal regulation 40 CFR Part 130.7 require states to develop a 303(d) list. The primary purpose of the 303(d) listing is to describe the health of rivers, coastal waters, estuaries and lakes. In Washington, WDOE submits this listing of "troubled waters" to EPA for approval and uses it to monitor water quality trends and establish priorities for protection. Water bodies must meet two criteria to be placed on the 303(d) list:

- Current water quality does not meet the state water quality requirements.
- Technology-based controls are not sufficient to achieve water quality requirements.

Monitoring data to determine which water bodies should be identified on the 303(d) list are gathered from several sources, including WDOE's own monitoring, and project-specific monitoring conducted by resource agencies, tribes, and other sources. Monitoring information submitted to the WDOE is evaluated to ensure that the data was collected and analyzed using quality assurance/quality control methods and that data was tested by a state accredited laboratory.

Water body protection involves the setting of Total Maximum Daily Load (TMDL) limits. The TMDL assignment process, as described in the Federal Clean Water Act, is used to establish allowable pollutant concentrations to be apportioned to both point and non-point sources of pollutants that may discharge to a water body while still supporting beneficial use and meeting water quality standards. TMDLs are often referred to as Water Cleanup Plans.

WDOE's current listing process is much more comprehensive than the early 303(d) lists that were developed in 1996 and 1998. The current process assigns Water Quality Assessment Categories to water bodies ranging from Category 1 (clean waters) to Category 5 (polluted waters that require a TMDL). Since the categories are pollutant-specific, a single water body may be listed in multiple categories.

Most of the Columbia River Mainstem fails to meet state and/or tribal Water Quality Standards for critical periods of time, mainly in the spring and summer months, for both water temperature and total dissolved gas. Therefore, this water body has been "303(d) listed" for these two pollutants. The status of the TMDLs for these pollutants and the potential for future TMDLs for other pollutants are discussed below.

4.3.2.2 Temperature TMDL

Development of the temperature TMDL for the Columbia River from the Canadian border to its mouth at the Pacific Ocean was initiated by EPA in 2002 and was expected to be finalized by May 2003. However, due to public concerns regarding the EPA's conclusions regarding the impacts of hydroelectric dams and other technical issues, the TMDL has been delayed indefinitely. According to conversations with WDOE staff, no schedule for completing the TMDL has been established. However, if this TMDL is finalized, it may have a significant impact on the City's discharge, especially during the warmer months.

4.3.2.3 Dissolved Gas TMDL

EPA approved WDOE's submittal of the Total Dissolved Gas (TDG) TMDL for the Mid-Columbia River and Lake Roosevelt on July 27, 2004. The area covered by this TMDL includes the Columbia River Mainstem from the Canadian border to the Oregon/Washington border. Since the primary source of TDG pollution is hydroelectric dams, this TMDL is expected to have minimal impact on municipal wastewater discharges such as the City of Richland's.

4.3.3 Future TMDLs

The WDOE Surface Water Quality Standards website includes Current Rule Activities with the recent update on 'Human Health Criteria and Implementation Tools Rulemaking.' This rulemaking is focusing on water quality standards for toxics. Ecology issued a draft rule for public comment in February 2016 and has committed to a final Rule adopted in August 2016. This Rule could reduce allowable concentrations of toxins in the effluent primarily due to an increase in fish consumption rates. Most toxins accumulate in the fatty portions of edible fish. For example, the current Washington WQS for Polychlorinated Biphenyls (PCBs) is 170 pg/l. This concentration is difficult to consistently meet at a conventional wastewater treatment plant with advanced secondary treatment. There is not currently any data on Richland's influent concentrations, but typical influent concentrations range from 1,000 – 10,000 pg/l. There is also limited to no data on the concentration of toxics in the Columbia River. If the new rule requires extensive treatment to remove toxics, it could trigger advanced oxidation processes after secondary treatment with filtration – which could double or triple the cost of treatment.

If the new rules are adopted, TMDLs must be prepared to allocate loading. WDOE does not appear to have immediate plans to develop these TMDLs, but this process could be completed in the next five years. After the TMDL is established, it typically takes two permit cycles to gather effluent data, update facilities plans, obtain necessary financing authority, design, and construct new treatment facilities to meet the TMDL allocated effluent limits for the wastewater treatment plants. Although the entire process could take three or more permit cycles before it is realized as an effluent limit, it would have dramatic effects on the City's WWTP and would require a significant increase in the cost of treatment. This proposed toxics rulemaking is one the City should follow closely.

Once the new rules are finalized, there are compliance strategies that the City should consider. For example, the Spokane River dischargers are developing source control programs in addition to membrane treatment at the end of the pipe. WDOE has been encouraging this form of pollution reduction which may help with permit compliance only if the dischargers that provide the source control get credit for the cleanup.

4.3.4 Existing Discharge Standards

The City of Richland WWTP operates under NPDES Waste Discharge Permit No. WA-002041-9. A copy of both the current permit and fact sheet are included in **Appendix O**. The permit was effective on August 1, 2009 and expired on July 31, 2014. The City has applied for renewal of the permit and has been following the terms and conditions of the existing permit in the interim. A draft version of the new permit was submitted to the City in fall 2015. The City has an excellent track record of meeting permit requirements – with no violations in recent history. In fact, in 2014 the City received its 4th consecutive "Outstanding Performance Award" from WDOE.

The permit requires the City to submit a plan and a schedule for continuing to maintain capacity whenever actual flow or load reaches 85% of any one of the design criteria for three consecutive months. The flows and loads for the permitted facility are based upon the design criteria as listed in the permit and included in **Table 4-2**:

Table 4-2 – Design Criteria – 2009 NPDES Permit

Parameter	Design Criteria	85% of Design
Average flow for maximum month	11.4 mgd	9.7 mgd
BOD_5 loading for maximum month	17,250 lbs/day	14,663 lbs/day
TSS loading for maximum month	21,200 lbs/day	18,020 lbs/day
NH3-N loading for maximum month	2,750 lbs/day	2,338 lbs/day

The only effluent limitations in the permit relate to BOD_5, TSS, fecal coliform, pH, residual chlorine, and ammonia. The existing effluent limits are presented in **Table 4-2**.

Table 4-3 – Effluent Limits – 2009 NPDES Permit

Parameter	Average Monthly	Maximum Average Weekly
BOD_5	30 mg/L, 2,588 lbs/day, 85% removal of influent BOD_5	45 mg/L, 3,882 lbs/day
TSS	30 mg/L, 2,852 lbs/day, 85% removal of influent TSS	45 mg/L, 4,278 lbs/day
Fecal Coliform Bacteria	200/100 mL	400/100 mL
Parameter	**Average Monthly**	**Maximum Daily**
NH_3-N	18.5 mg/L, 1,759 lbs/day, 85% removal of influent NH3-N	27.7 mg/L, 2,634 lbs/day
Residual Chlorine	N/A	0.5mg/L, 48 lbs/day
Parameter	**Daily**	
pH	$6.0 \leq pH \leq 9.0$	

4.4 Expected Future Discharge Standards

Based upon inquires made to the WDOE Staff, the new discharge permit is expected to be issued in 2016. A draft was submitted to the City in fall of 2015 and it appears that the permit will remain largely unchanged. There are some minor clarifications to the mixing zone and there is a requirement for an Outfall Evaluation. It should be noted that statewide trending for discharge permits includes various levels of water quality and source control testing beyond what existing permit holders have experienced in the past. For example, Walla Walla and College Place are now required to conduct PCB testing and develop toxic management control plans. The City should review the existing pretreatment program and source control programs with an eye towards reducing the compliance effort to meet future discharge limits for toxics. While the permit conditions that will result from current rule-making efforts are far from clear, the evidence points toward more stringent standards. Involvement with the rule-making is critical to provide as much compliance flexibility as possible plus reasonable compliance schedules for any required upgrades.

CHAPTER 5

Wastewater Treatment Plant

Chapter 5 – **Wastewater Treatment Plant**

5.1 Introduction

This Chapter of the General Sewer Plan evaluates the capacity and condition of the existing facilities at the City of Richland Wastewater Treatment Plant (WWTP) to adequately treat current and projected flow and loads to meet current NPDES permit discharge requirements. The chapter relies heavily on both previous work and plant input to determine the reliable treatment capacity of each individual unit process. A list of short-term recommended improvements and subsequent process evaluations is the primary outcome of the chapter.

The WWTP treats primarily municipal wastewater through primary sedimentation and secondary activated sludge process. Chlorine is injected prior to discharge to the Columbia River for disinfection. Solids are thickened with dissolved air flotation, mesophillicaly digested, dewatered on belt presses and transported to the City composting facility to attain a Class A compost which is sold to the public through wholesale distributors.

Except for the aeration basins and air delivery system, the facilities at WWTP have not been substantially upgraded since original design. The original surface aerated basins were converted in 2000 and 2003 to plug flow staged aeration basins with disc diffuser aeration.

5.1.1 Approach

Documentation review was used as the basis of the evaluation of the hydraulic and treatment capacity of the unit processes. This documentation review included previous sewer comprehensive plans, design documentation of plant facilities, and process capacity evaluations. Primary sources include the following:

- WWTP Capacity Assessment Report (2003)
- General Sewer Plan Update (2004)
- Original Design documentation (1988)
- Operations and Maintenance Manual (2008)

Current and projected flow and loads were adopted from the analysis presented in **Chapter 3**. Additional analysis on the solids stream was conducted in this Chapter to identify any gross incongruities in the WWTP solids balance, and analyze digester loadings under different solid thickening scenarios than those currently in use. The current and projected loadings were compared to the estimated unit capacities developed from previous documentation to estimate available capacity in each unit process.

Subsequent to the preliminary unit process capacity analysis, a workshop was held with City staff to review the initial findings and assess the condition of existing facilities. The workshop presentation is included as **Appendix L**. A two-hour walkthrough with WWTP plant staff of the facilities identified plant reliability concerns with existing equipment. Plant staff highlighted the criticality of existing equipment for reliable operation of plant facilities. Both the condition and criticality of equipment to plant operations were inventoried and used to determine the recommended upgrades.

5.1.2 Current Plant Upgrades

The Solids Upgrade Project, currently in construction, includes replacement of thickening and dewatering equipment, and the waste activated sludge (WAS) pumps. The Disinfection Upgrade Project replaces the existing chlorine gas system with a hypochlorite generation and supply system. The City is procured and installed the new hypochlorite production and injection system in June 2015.

5.1.3 NPDES Permit

A summary of the 2009 NPDES permit is shown in **Tables 4.1** and **4.2** which identify the allowed influent flows and loads as well as the discharge requirements. As can be noted by the ammonia discharge limit, which is below the influent ammonia concentration, partial nitrification in the secondary process is required. The permit and fact sheet are included in **Appendix O**.

5.1.4 Influent Flows and Loads

The current influent flow and loads are shown in **Table 5-1** and the projected influent flow and loads are shown **Table 5-2**. As described in **Chapter 3**, these are based on measured influent loads with projections calculated using current peaking factors and projected population growth. **Table 5-3** presents the current influent values compared to rated plant capacity. **Table 5-4** presents the projected influent values compared to rated plant capacity. The current TSS and BOD loadings are nearly 80 percent of rated plant capacity and the projected 20-year BOD and TSS loadings are over 100 percent of plant rated capacity. Both current and projected influent flows remain below plant rated capacity. Both the BOD and TSS concentration have increased over 50 percent from the original design criteria. The increases in concentration can be due to water conservation measures and sewer conveyance system upgrades. However, the highest loadings tend to be during the lowest flows, which points to the influent sampler measuring settled as well as suspended material. In the 2004 General Sewer Plan the accuracy of influent sampling of BOD and TSS was questioned. The solids balance analysis, as described in **Section 5.3** of this chapter, also reveals discrepancies between the influent loadings and primary solids production. To verify the accuracy of the influent sampler, plant staff increased the flow rate near the sampler. BOD and TSS loadings initially decreased to concentrations more in-line the plant effluent permit, but in the wet weather months of 2014 the BOD and TSS loadings were in-line with previous years data analyzed in Chapter 3. Continued monitoring is required to verify the influent loadings continue to trend downward over seasonal variations in flow and load conditions.

In addition to the requisite flow and load parameters of Average Day, Maximum Month, Peak Hour and Peak Event which are considered in determining the hydraulic and treatment capacity of each unit process, a Maximum Three Month value has been calculated for current and projected flow and loads. Washington Administrative Code (WAC) requires that a WWTP upon exceeding 85% of the permitted flows and loads for three consecutive months must undertake either improvements to increase treatment capacity or an engineering report to reevaluate or "rerate" the plant capacity.

Table 5-1 – Current Flow and Loads

	Flow (mgd)	BOD (ppd)	TSS (ppd)	BOD (mg/L)	TSS (mg/L)
Average Day	5.70	11,032	12,911	235	273
Maximum 3 Month	6.20	13.238	16.547	260	321
Maximum Month	6.25	14,099	18,146	270	348
Peak Day	7.50	18,870	25,157	302	419
Peak Hour	9.41	-	-	-	-

Table 5-2 – Projected Flow and Loads

	Flow (mgd)	BOD (ppd)	TSS (ppd)	BOD (mg/L)	TSS (mg/L)
Average Day	8.21	15,910	18,620	235	273
Maximum 3 Month	8.95	19,090	23,830	260	321
Maximum Month	9.03	20,360	26,250	269	348
Peak Day	10.83	27,210	36,310	301	418
Peak Hour	13.54	-	-	-	-

Table 5-3 – Current Flow and Load Compared to Plant Capacity

	Flow (mgd)	BOD (ppd)	TSS (ppd)	BOD (mg/L)	TSS (mg/L)
Maximum 3 Month	6.25	13,238	16,547	270	348
Permitted Max Month	11.4	17,250	21,500	181	226
% of Permitted Capacity	52%	77%	77%	-	-

Table 5-4 – Projected 20-Year Flow and Load Compared to Plant Capacity

	Flow (mgd)	BOD (ppd)	TSS (ppd)	BOD (mg/L)	TSS (mg/L)
Maximum 3 Month	8.78	19,090	23,830	260	321
Permitted Max Month	11.4	17,250	21,500	181	226
% of Permitted Capacity	77%	111%	111%	-	-

Table 5-3 and **Table 5-4** indicate that in the near future (approximately 2020) the three month loadings will exceed 85% of plant rate capacity assuming a continuation of the current sources to the WWTP. At that time an engineering report will be required by the WAC as stated in the NPDES permit to reassess the capacity of plant and either rerate the capacity or develop plant improvements to provide more capacity. The trigger for this evaluation will be determined when three consecutive monthly DMRs show 85% capacity. We estimate this to be approximately 2020; however, recent changes in influent sampling may delay the realization of this requirement further.

5.2 Unit Process Capacity and Condition Evaluation

A process flow chart of existing facilities is provided in **Figure 5-1**. Key parameters of major unit processes are shown in **Table 5-5**. The following unit processes were evaluated in this Chapter:

- Influent Screening
- Influent Pumping
- Secondary Screening
- Aerated Grit Removal
- Primary Clarifiers
- Aeration Basins
- Secondary Clarification
- Digestion

Figure 5-1 – Process Flow of Existing Facilities

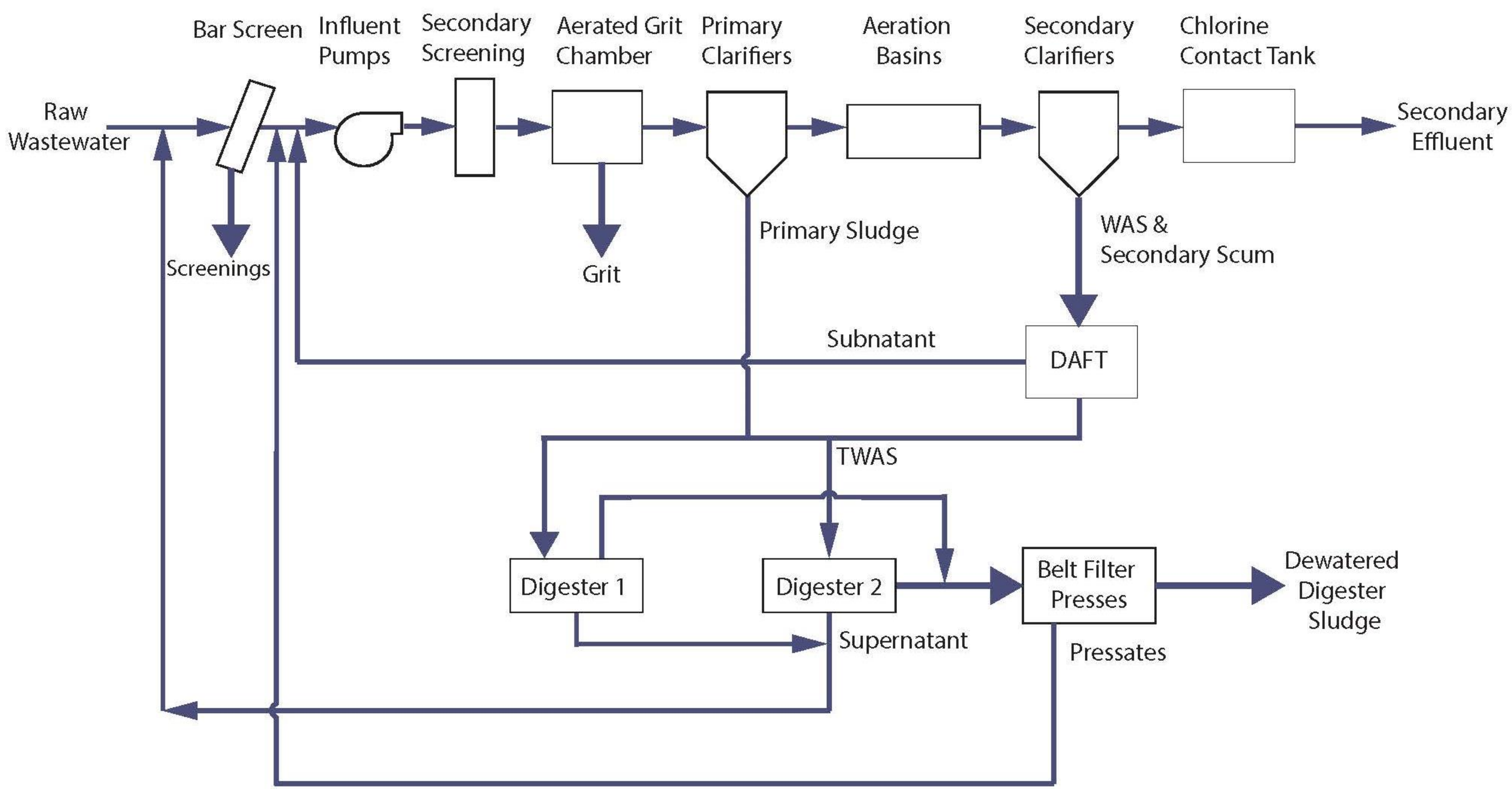


Table 5-5 – Major Unit Process Key Parameters

Process Element	Number of Units	Design Data
Bar Screens	1	
Openings Between Bars, inch		3/8”
Influent Pumps	4	
Power, hp		1@75, 3@100
Total firm capacity, mgd		24
Aerated Grit Chambers	1	
Length, feet		38
Width, feet		22
Depth, feet		16.5
Total volume, gallons		130,000
Primary Clarifiers	2	
Diameter, feet		85
Sidewater depth, feet		10
Surface area per clarifier, square feet		5,674
Aeration Basins	2	
Length, feet		145
Width, feet		100
Sidewater depth, feet		20
Volume per basin, MG		1.8
Secondary Clarifiers	2	
Diameter, feet		85
Sidewater depth, feet		15
Surface area per clarifier, square feet		14,314
Chlorine Contact Chamber	1	
Volume per chamber, cubic feet		34,000
Dissolved Air Flotation Thickener	1	
Diameter, feet		35
Depth, feet		10
Surface area, square feet		962
Anaerobic Digester	2	
Diameter, feet		60
Depth, feet		28
Volume per digester, gallons		617,000
Belt Filter Press	2	
Belt width, meter		2

5.2.1 Influent Screening

Influent screening consists of one automated 3/8-inch bar screen and one manual bar screen, one screenings washer-compactor, and a screenings bin. Odor treatment facilities are no longer in-service. Plant staff directs all flow to the automated bar screen when it is in-service.

Performance and Capacity: In previous documentation, neither the hydraulics nor the condition of the bar screen facility were analyzed: therefore, in this report a hydraulic model was developed to assess the capacity of the bar screen. Assuming 50 percent blockage, 14 mgd produces a hydraulic loss that matches the peak hydraulics conditions indicated on the original design drawings.

Plant staff reports the performance of the bar-screen to be unsatisfactory. It is over 30 years old and replacement parts are difficult to find. Several major mechanical failures have occurred recently to bearings, motor, and frame structure. Therefore, it is scheduled for replacement in 2017.

Redundancy: The manual bar screen provides 100 percent redundancy on failure of automated bar screen.

Condition: The automated bars screen is nearing the end of its useful life. Deterioration of metal components due to exposure to hydrogen sulfide as well as 20 years of uninterrupted service has reduced the reliability of this critical unit process. The washer/compactor is also at the end of its useful life. Staff report that the automated bar screen has been out of service for repairs multiple times in the past year. This requires use of the manual bar screen and results in less effective screening and a higher demand on plant personnel.

The odor control equipment in the screening facility has been out-of-service for multiple years and is no longer functioning. Ventilation facilities do not adequately remove odors from the screenings building exposing staff and equipment to hydrogen sulfide.

5.2.2 Influent Pumping

Influent pumping system includes four vertical turbine pumps with associated valving. Typical operation is one to two pumps in-service with the additional pumps providing redundancy.

Performance and Capacity: Three pumps, each with a rated capacity of 9 mgd and one with a 6 mgd capacity provide 24 mgd of reliable capacity, which is well above the peak flow values projected over the next 20 years. The combination of smaller and larger pumps allows for diurnal variations in flow to be met without the need to store influent in the conveyance system.

Redundancy: The multiple pumps available for service provide suitable redundancy for current and future flows.

Condition: The pumps are well-maintained by plant staff, but are over 20 years old and will require major rebuilds in the future. The discharge valves had evidence of leaking and corrosion and may need to be replaced in the next 10 years. Although the pumps are nearing the end of their useful life, the substantial redundancy in the pumping capacity reduces the criticality of an influent pump failure to plant operations.

5.2.3 Secondary Screening

The secondary screening consists of fine screening, washing and compacting. These facilities were added after initial plant construction and designed to remove a higher level of screenings material to improve the quality of biosolids.

5.2.4 Aerated Grit System

The aerated grit system consists of an aerated grit chamber, aeration blowers, grit pumps, classifiers, and grit bins. It is located directly upstream of the primary clarifiers.

Performance and Capacity: The capacity of the aeration grit tank, blowers, pumps and classifiers exceeds the rated plant maximum month capacity of 11.5 mgd, applying industry standards and Department of Ecology Orange Book design criteria. The equipment is well-maintained and has suitable redundancy. Plant staff reports the aerated grit removal system operates reliably, and excessive grit deposition has not been evident upon cleaning of the aeration basins or primary clarifiers.

Redundancy: The aerated grit pumps and blowers have suitable redundancy under maximum hydraulic loadings as noted in the referenced publications.

Condition: Plant reports all equipment operates reliably. Equipment was not directly observed in the plant walk-through; therefore, the condition assessment relies on plant staff observations. Plant staff have noted the need to update the odor control in this room.

5.2.5 Primary Clarifiers

Primary clarification consists of two circular primary clarifiers, with scum removal, and two air diaphragm pumps, which pump both primary sludge and primary scum by alternating the pump suction location. The liquid stream continues from primary clarification to the aeration basins. The primary sludge is thickened in the clarifiers then transferred with the scum to the digesters.

Performance and Capacity: The Process Capacity Evaluation in 2002 estimated the removal efficiency of primary clarifiers under the hydraulic and solids loading of 7.46 mgd and 15,000 ppd to be around 60 percent for total suspended solids (TSS) and 40 percent for BOD which is in the expected range of performance. Estimates of the primary clarifier hydraulic capacity from previous reports indicate the discharge weir will be submerged at 14 mgd, well below the projected plant hydraulic peak loadings. At current flow and loads plant staff operate only one of the two clarifiers year-round; however, the plant did start operating both clarifiers in December 2015 to alleviate some problems that were occurring in the aeration basins.. The reduction in removal efficiency of the primary clarifiers at higher hydraulic loadings, than previously assessed, places a larger treatment burden on the aeration basins. However, as described in the subsequent section, only one of two aeration basins is currently operated year-round. Current and projected overflow rates with two clarifiers in-service are compared in **Table 5-6** to Orange Book recommended values.

Table 5-6 – Projected Overflow Rates Compared to Orange Book

	Current Flow (mgd)	Overflow Rate (gpd/sf)	Projected Flow (mgd)	Overflow Rate (gpd/sf)	Orange Book (gpd/sf)
Average Annual	5.70	494	7.99	693	800
Maximum Month	6.25	542	8.79	762	1,200
Peak Day	7.50	651	10.50	911	2,000

The primary sludge diaphragm pumps reliably convey thickened primary sludge to the digesters. However primary sludge flow measurement based on pump stroke and volume calculations may be inaccurate, as was the case with WAS flow calculation prior to replacement of the air diaphragm pumps with rotary lobe pumps and magnetic flow meters.

Redundancy: At current flows and loads, 100 percent year round redundancy of the primary clarification system exists. At projected loads, plant staff will have the flexibility to run a primary clarifier during higher flow periods to increase the removal efficiency of the system and reduce the loading to the secondary system.

Condition: The primary clarifiers are well-maintained, but will require recoating to prevent deterioration of the mechanism and extend the equipment's useful life. The gear drives are scheduled to be rebuilt in 2017. The air diaphragm pumps are near the end of their useful life, but continue perform reliably.

5.2.6 Aeration Basins

The aeration basins consist of two aeration basins, two 300-hp turbo blowers and four 125-hp multi-stage centrifugal blowers. Each aeration basin has seven zones with automated dissolved oxygen control at each stage. Typically the first stage is unaerated and operated as a biological selector. The remaining zones are aerated under normal operation although the second zone can also be run as unaerated in a larger selector is required.

Performance and Capacity: The performance and the capacity of the aeration basins has been extensively studied in previous documentation including the WWTP Capacity Assessment Report (2004). This analysis relies on the biological modeling presented in those studies. Currently the plant operates year-round with one aeration basin in-service and one 300-hp blower in-service. The projected flows and loads over the planning period increased by only 50 percent. Sufficient aeration and treatment capacity will exist over the planning period, assuming current solids retention times are maintained.

Condition: The aeration basins and blowers are in good condition. However, staff have noted some warping of the marine plywood separating the five individual cells – this may be causing some by-passing of mixed liquor from Cell 1 directly to Cell 5. Repair or replacement of the plywood should be considered. Maintenance and repair of the diffuser system and blowers is ongoing to maintain reliable operation.

5.2.7 Secondary Clarifiers

The secondary clarifier system consists of two secondary clarifiers, two return activated sludge (RAS) pumps and two waste activated sludge pumps.

Performance and Capacity: A single secondary clarifier reliably treats current flows and loads year-round. Sludge volume indexes (SVI) vary from 300 to 120 over the data period, but more recently have been in the range of 180 to 120. At these lower SVIs and assuming a MLSS of less than 2,500 mg/L previous studies, including the WWTP Capacity Assessment Report (2004), have shown the clarifiers have nearly 21 mgd of treatment capacity. This will accommodate even the projected peak hydraulic loadings. The hydraulic analysis included in the WWTP Capacity Assessment Report (2004) indicated the discharge weir will be submerged at 14 mgd. Secondary Clarifier overflow rates at current and future flows and loads are compared to applicable Orange Book values in the **Table 5-7**.

Table 5-7 – Secondary Current and Future Flows and Loads Compared to Orange Book

	Current Flow (mgd)	Overflow Rate (gpd/sf)	Projected Flow (mgd)	Overflow Rate (gpd/sf)	Orange Book (gpd/sf)
Average Annual	5.70	199	7.99	279	600
Maximum Month	6.25	218	8.79	307	800
Peak Day	7.50	262	10.50	367	1,200

One operational parameter that may need to be addressed in the future is the low RAS and WAS concentration which is often below 5,000 mg/L. The inability to effectively control or increase the RAS and WAS concentration results in a higher return flow rate as the loadings increase. This higher return flow increases both the hydraulic and solids loading on the secondary clarifier system. During the plant walk-through the City discussed upgrading the RAS rate control by implementing variable frequency drives (VFD) on the RAS pumps. Currently valves at the Aeration Basin Distribution Structure are modulated to adjust the RAS flow rate and thus the concentration. In addition to increased loading to the secondary clarifier, lack of RAS flow control can result in swings in the solids retention time (SRT) of the diurnal flows if wasting rates remain constant. Finally, the WAS concentration affects the cost and performance of the thickening process. Higher WAS concentrations could result in lower polymer costs, greater digester capacity and operational flexibility, and potentially higher dewatered solids concentration.

Capacity of the WAS pumps is also of concern to plant staff. The Solids Upgrade Project will replace existing WAS pumps with larger capacity rotary lobe pumps.

Redundancy: Currently the secondary clarifier system operates with 100 percent redundancy year-round. At projected flow and loads plant staff will have the flexibility to operate two secondary clarifiers as flow and loads warrant, or as SVI increases.

Condition: The secondary clarifiers and RAS and WAS pumping systems are well-maintained and in good condition. The WAS pumps are of different manufacturers and require multiple part inventories. Coating of clarifiers and possible future mechanism replacement will be required to maintain reliable operation in the future. The gear drive is scheduled to be rebuilt in 2017.

5.2.8 Digestion System

The digestion system consists of two parallel operating mesophilic digesters each with a pump mix system. A single boiler which runs on biogas with diesel fuel back-up provides heat to two sludge/water heat exchangers to maintain mesophilic temperatures within the digesters. Additional equipment within the digester control building includes the dewatering feed pumps, thickened WAS pumps and heat exchanger recirculation pumps.

Performance and Capacity: With both digesters in-service year-round in parallel feed mode, the digestion system operates reliably with volatile acids being steadily below 100 mg/L and alkalinity above 5000 mg/L. The current SRT value of 38 days is above the 15 days required to meet Class B and maintain reliable digester operation. The solids loading on the digestion system due to the lower concentration of the thickened WAS and RAS is below 0.10 pounds per cubic foot per day under current and projected conditions.

The current parallel feed mode, where both digesters are fed thickened WAS and primary sludge has been adopted to replace the series feed mode intended in the original design. Historically, foaming issues limited the loading to the digesters and required both digester volumes be used in parallel. Currently, the lower concentration of thickened primary sludge and thickened WAS decreases the SRT for single digester operation below recommended values during maximum month conditions so parallel digester operation is required. The assumed digester feed concentration during original design was 6-7 percent. Current operation feeds solids at 3-4 percent, which produces a volumetric increase of the 100 percent compared to the original design. The current Solids Upgrade Project will replace existing thickening equipment and may be able to produce a higher concentration of thickened WAS. The ability of the plant to operate on one digester during digester cleaning activities is unknown. A digester cleaning event scheduled for the summer of 2014 has been postponed to 2016.

Heating efficiency has been improved through insulation of the two digester roofs. Mesophilic temperatures are maintained throughout the year using biogas as the primary heating fuel.

SRT and volatile solids loading rates under current and projected loads is shown in **Table 5-8**. In calculating the SRT it is assumed the digesters are fed over a 24 hour period. Values shown are for two digesters operating in series.

Table 5-8 – Digestion System Operational Evaluation

Flow Condition (mgd)	Current SRT (days)	Current vs. Loading Rate (ppd/cf)	Projected SRT (days)	Projected vs. Loading Rate (ppd/cf)
Maximum Month	38	0.05	27	0.08

The capacity of the digestion system is highly dependent on the operating parameters of the solids thickening processes which determine the SRT and the operation of the selector in the aeration basins to remove the microbial organisms that cause foaming.

Condition: The digesters and associated pumping equipment are in good condition. Relocation of digester gas piping from the digester control building is desirable to plant staff for current code compliance and safety reasons. The relocation of the piping would improve the equipment life of the motor control centers. The current motor control centers are scheduled for replacement in the near future. Coating of the digester interiors and improvements to the Digester Control Building HVAC will enhance the digestion systems reliability and ease of operation (one was coated three years ago). Additional controls on the boiler were completed last year and will improve monitoring and energy efficiency, and lessen O&M costs associated with boiler operation. Wholesale replacement of the boiler is being completed now as it is nearing the end of its useful life.

5.3 Solids Stream Unit Process Loading Analysis

To better determine the effect of variation in the operation of the solids thickening process and to estimate the effect of projected flows and loads on the loadings to the digestion system, a solids balance model was developed as shown in **Figure 5-2**, which is included to demonstrate the process stream evaluated. This model uses solids stream flow and loading inputs along with adjustable performance criteria of the solids stream unit processes to assess performance of the digestion system at the selected flows and loads. The figure does not represent a current or projected operational point. Plant data from 2009-2014 was used to determine current solids stream flow and loads. Projected conditions were estimated by increasing solids stream flow and loads by the same percentage as the projected influent flow and loads. Although this approach does not account for variations in the primary and secondary operational protocols, it does provide a reliable assessment assuming solids production processes do not vary substantially over the planning period.

5.3.1 Data Analysis

In developing the current solids stream flow and loads, three discrepancies in the data were observed as listed below along with probable explanations for the deviations. These deviations occur less frequently after the adjustment to the flow rate near the influent sampler – reference **Section 3.2.2**.

1. The measured thickened WAS flow increased dramatically in 2013 by up to 30 percent. Plant staff identified this change as a result of replacing the air diaphragm pumps with rotary lobe pumps and installing a magnetic flow meter.
2. The measured thickened primary total solids in pounds per day was less than 30 percent of expected given the influent solids loading and expected removal percentages for the primary clarifiers. Two factors may account for this under measurement.
    - The influent TSS and BOD measurements are higher than the actual concentrations especially during low flow periods. The location of the intake to the sampling unit measures settled material, increasing the measured concentration. TSS and BOD concentrations dropped when plant staff increased flows near the BOD and TSS sampler intake but increased again to expected levels.
    - The flow calculation from the primary sludge pumps may be lower than the actual flow, as was the case with the thickened WAS flow rate calculation prior to replacing the pumps with rotary lobe pumps. With ODS pumps, the flow is calculated from the volume of each stroke and the number of strokes.
3. The measured flow to dewatering is 15 to 20 percent higher than the sum of the measured thickened primary sludge and thickened WAS flows. Digestion will reduce the solids concentration but should not increase or decrease the digester feed flow rate compared to the digested sludge flow rate. Again this discrepancy may be explained by the under calculation of the thickened primary sludge.

In the data from July 2014-October 2014 the measured values for the primary sludge production (ppd basis) were more representative of expected removal rates for primary clarifiers operating at the reported overflow rates. Although the solids balance between the influent solids and the sum of the primary sludge and primary effluent did not consistently match, this could be explained by the weekly frequency of TS measurements for the primary sludge. The sum of digestion influent flow rates over a month of operation matched consistently the flows reported from dewatering. In summary this data set has less discrepancies than previously observed in the data from 2011 to 2013.

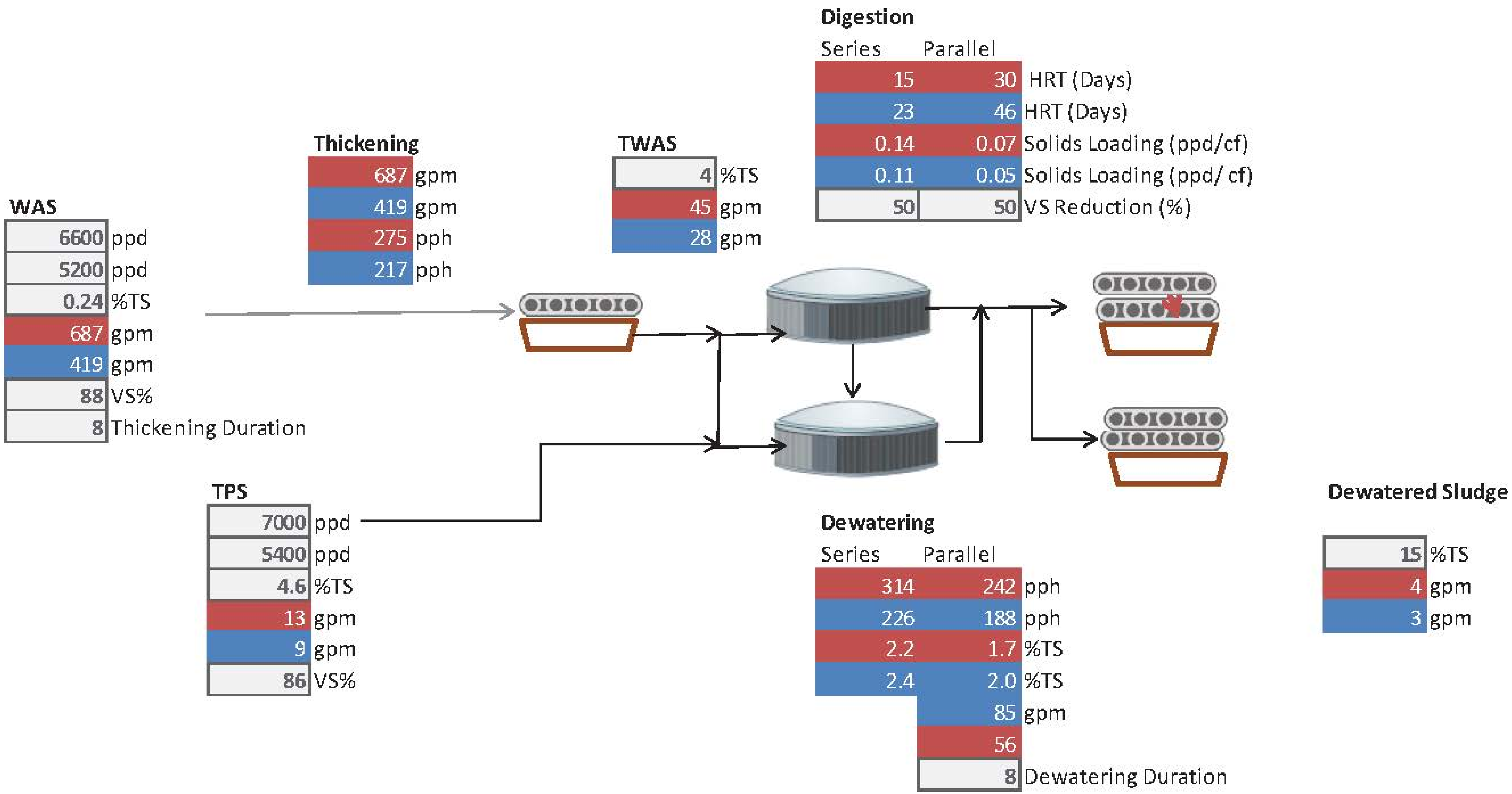
Although these measurement discrepancies do not in themselves reduce treatment capacity, the inability to accurately assess loading and treatment performance affects process reliability, especially during periods when the treatment process is operating near capacity. Accurate measurement of key parameters allows for the treatment train to more easily absorb unplanned treatment loads and for the City to have a clear understanding of their operational capacity. Therefore it is suggested the plant perform, at least quarterly, a solids balance to assure the calculated loadings match the actual process loadings.

Table 5-9 shows the current and projected loads for the solids stream. These values depend on the process management of the primary and secondary clarifiers. Operational choices will affect these loading rates, especially those associated with Peak Day and Maximum Month.

Table 5-9 – Solids Stream Flow Rates

Flow Condition	Primary Sludge Total Solids (ppd)	WAS Total Solids (ppd)
Current		
Average Annual	4,000	3,900
Maximum Month	4,800	4,800
Peak Day	6,500	5,600
Projected		
Average Annual	5,400	5,500
Maximum Month	6,500	6,700
Peak Day	9,200	7,900

Figure 5-2 – Solids Balance Model



5.4 Recommended Improvements

The following discussion identifies the recommend improvements or actions to assure reliable operation of the WWTP to meet the projected loads over the planning period for the unit processes analyzed in this report. The improvements detailed below primarily address current conditions at the WWTP which affect reliability, redundancy and ease of operation and maintenance. To address future limitations in rated capacity it is suggested that the City budget for an engineering report be undertaken in approximately 2020. This report will not only adjust the current influent loading limitations, but also provide the City with an increase in rated capacity to treat additional industrial loads.

Influent Screenings Facility: Due to the condition of the existing screening facilities and the lack of automated redundancy, a renovation of the screenings facilities including replacement of the existing screen and the addition of another automated screen, new compacting and washing equipment, upgrades and replacement to the ventilation and odor treatment systems is required. This upgrade should be implemented in 2-5 years to assure continued reliable screenings and protection

Influent Solids Loading: Plant should continue to monitor the influent solids loading data for an entire year to better substantiate that the observed reduction in solids loading is due to low flow at the sample location. The City should plan in the next 5 years to undertake either a rerating study to rerate the treatable influent BOD and TSS loadings or an Engineering Report to develop plant upgrades to treat the projected BOD and TSS loadings, until a year of influent data under the new sample conditions verifies the actual loadings are lower than those measured over the past 5 years.

RAS Pump Station: Plant Staff should test the efficacy of controlling RAS with the existing butterfly valves under all loading conditions to assure that the RAS concentration can be increased to a minimum of 0.7 percent TS. Controlling the thickening of the RAS and WAS in the secondary clarifier to a more standard 0.7-1.0 percent total solids will increase the digester SRT and increase the solids concentration to the dewatering process.

Other improvements, identified by plant staff, will maintain the exceptional level of treatment performance demonstrated at the WWTP historically. **Table 5-10** summarizes the recommended improvements for the next 1 to 5 years. Dollar values are 2014 probable construction costs. Engineering, administrative, legal and construction management costs are not included.

In addition to the capital improvements listed in **Table 5-10**, the City should budget for ongoing O&M activities not associated with Capital Improvements. Typically O&M cost to maintain plant facility are estimated based on the facility capacity, age, and complexity. For a facility such as the Richland the range is between $75,000 and $100,000 per mgd and average annual conditions, which annually equates to between approximately $425,000 and $590,000. $500,000 is a reasonable value to adopt considering the new solids improvements and solids pump upgrades.

Table 5-10 – Near Term Capital Improvements

Description	Cost [a]
–Influent Screening	$2,000,000
Plant Wide HVAC Improvements	$290,000
Digester Building MCC	$80,000
Primary Clarifier #2 Coating	$160,000
Digester #1 Tank Coating	$320,000
Secondary Clarifier #2 Coating	$220,000
Clarifier Gear Drive Replacements	$305,000
Plant Pump & Piping Replacement - 2017	$75,000
TOTAL	$3,450,000

(a) *Based on Wastewater Treatment Facility Renewal & Replacement List from City of Richland*

5.5 Plant Staffing

Plant is staffed by the following: one Operations Manager, four shift operators (Group III), three operators in training, and two laboratory technicians. There are also two plan mechanical craftsmen and a part-time electrical and instrumentation technician. Even with current tight market for qualified operators the Plant has been able to structure the plant personnel to train and promote young operators to overtake lead responsibilities. Staffing is consistent with other treatment plants of this size considering the number of shifts and level of automation. Futher discussion on staffing is provided in Chapter 9.

Until recently the WWTP was staffed 24 hours a day, seven days a week. Automation of certain plant processes substantially reduced plant staff demands and allowed current staffing levels to suitably operate and maintain the plant, which resulted in eliminating the graveyard shift.

Chapter 6

Collection System

Chapter 6 – **Collection System**

6.1 Introduction

The following chapter details the evaluation process and findings for the City's existing sewer collection system. Generally the existing hydraulic model was updated to reflect the existing collection system and calibrated using flow monitoring data recorded by City-owned equipment deployed for this sewer plan update. This update occurred in two steps: Step One – South Richland Sewer Review (discussed further in **Section 6.2.2.1**); Step Two – Remaining Richland Sewer Review. The hydraulic model was then used to evaluate the City's sewer collection system under the following three scenarios, each of which is discussed in this chapter:

- Existing Model and Analysis (**Sections 6.2 & 6.3**): Represents the current collection system within the City limits. Capacity issues identified at this stage indicate problems under today's conditions.
- Committed Model (**Section 6.5**): Represents the development within the entire UGA boundary; essentially everything the City has committed to serve and includes all parcels. Capacity issues identified at this stage indicate there may be problems upon build-out to the City's UGA boundary.
- Master Plan Model (**Section 6.6**): Represents the ultimate build-out of the future wastewater service area by including additional study areas identified by the City.

The above three scenarios also identify the modeling sequence that was followed for evaluating the City's sewer collection system: Existing System followed by the Committed Model and then the Master Plan Model. With the completion of the first two scenarios, an issues list was prepared to highlight the findings and is included in each discussion section.

The City's sewer collection system consists of manholes, gravity pipes, clean-outs, lift stations and force mains. The gravity collection pipes range in diameter from 8-inch to 54-inch and comprise a network of nearly 263 miles. The City's 14 lift stations act as local collection points for sewer flows within an area that cannot be served by extension of the existing gravity collection system. Lift station pumps convey wastewater through force main pipes that have a combined total length of greater than 5.5 miles. The lift stations are discussed in more depth in **Section 6.9**.

6.2 Existing System Model

6.2.1 General

The existing hydraulic model in this study was built using the City's GIS data and survey records and then analyzed using InfoSWMM modeling software. The City's previous comprehensive plan update (2004) utilized Hydra modeling software. For this update, InfoSWMM was chosen to provide a GIS-based modeling platform well suited to integrate the existing City data and the hydraulic model and to also provide a more sophisticated hydraulic modeling engine. The existing hydraulic model's primary purposes are to:

- Provide a snapshot of the current collection system flows
- Identify potential existing capacity issues
- Provide a platform for use in the Committed and Master Plan Models

The existing hydraulic model consists of two layers – 1) the System Layer and 2) the Flow Generation Layer. Each layer includes multiple parameters and corresponding assumptions that characterize the area and system being modeled. The assumptions are coupled with surveyed pipe inverts, record drawing data, flow monitoring data, characteristics learned from the physical system, similar studies done in the region, and general and historical knowledge gained through previous work for the City. Key assumptions used in the existing model are documented in **Appendix C**.

6.2.2 System Layer

The existing hydraulic model System Layer consists of the manholes, gravity sewer pipes, force mains, and lift stations in the collection system. A map of the Existing System is found on **Figure A1** and a map of the Sewer Collection System Basins (as described in **Section 1.4**) is found on **Figure 1-1**.

6.2.2.1 Existing Collection System Layer

The existing collection system layer was first updated from the previous hydraulic model (2004 update) in April of 2014 for the South Richland Sewer Review (SRSR) project. The objective of the SRSR project was to conduct hydraulic modeling for the gravity sewer system downstream of the UGA annexation area and the Badger South planned development in South Richland and to identify master plan improvements and alternatives. (A technical memorandum discussing the SRSR project results is included in **Appendix B**.) The previous hydraulic model was used as the main source of information for rim elevations, invert elevations, pipe sizes, and pipe lengths. The City's record drawings from improvement projects after 2004 were used to update the hydraulic model to current conditions. As depicted on **Figure A2**, there were five main areas of improvement projects that were added to the model: 1) Physical Sciences Facility at PNNL, 2) Logston Sewer Interceptor, 3) RY Basin Improvements (new lift station and abandon two existing lift stations), 4) Leslie Sewer Trunk, and 5) Badger Mountain South Development. Any missing or questionable data was reviewed with the City and then supplemented with record drawings, field checks, or survey if necessary. Missing or questionable data for trunk pipes was resolved by using data from the previous model or by straight-grading individual sections of pipe (i.e., interpolating an invert based on upstream and downstream inverts). These manholes and pipes are tagged in the model accordingly.

A portion of the collector pipes (8-inch and less) were added into the existing hydraulic model to facilitate flow routing; however, invert data was not verified or resolved because none of the 8-inch pipes were analyzed for capacity. The 8-inch pipes in the model perform the sole function of routing flows into the model and no physical data or capacity data on the 8-inch pipes should be utilized for decision making purposes. Only pipes 10 inches in diameter and larger were analyzed for capacity in the hydraulic model.

6.2.2.2 Lift Stations

Lift station and force main data were added to the existing hydraulic model based on the previous model and by record drawings and discussions with City staff. **Table 6-1** lists the current 14 lift stations that are operating in the City and that are represented in the model. **Figure A1** depicts the locations of these lift stations. Also included in **Table 6-1** are the design operating point's which were obtained from the City and from the previous General Sewer Plan. The lift stations are discussed in more depth in **Section 6.9**.

Table 6-1 – Existing Lift Stations

Lift Station	Year Constructed/ Last Major Rehabilitation	Design Operating Point (one pump)	Pump Description	Wet Well Dimensions
Battelle	2013	400 gpm at 20-ft	Flygt, 5 hp, NP-3102 MT 3~ 465	12-ft Diam.
Waterfront	1977	600 gpm at 43-ft	Fairbanks Morse, 15 hp	6-ft Diam.
Terminal Dr	1981	150 gpm at 24-ft	Fairbanks Morse, 3 hp	8-ft Diam.
Mental Health	2009	260 gpm at 21-ft	Vaughan, 5 hp, SP4C, 1170 rpm, 8.9in Imp	6-ft Diam.
Bradley	1999	180 gpm at 59-ft	Flygt, 10 hp, NP-3127	10-ft Diam.
Columbia Pt	2010	270 gpm at 52-ft	Flygt, 6.5 hp, NP-3102 SH 3~ 256	10-ft Diam.
Wellhouse Loop	1978	100 gpm at 12-ft	Hydromatic, 1.5 hp	6-ft Diam.
Duportail	1995	200 gpm at 45-ft	Flygt, 7.5 hp, NP-3127 HT 3~ 489	6-ft Diam.
Montana St	2015	970 gpm at 105-ft	Smith & Loveless, 30 hp, 4B3 1760 rpm	8-ft Diam.
Columbia Park Trail	2012	400 gpm at 50-ft	Flygt, 10 hp, NP-3127 MT 3~ 438	8-ft Diam.
Meadows South	1970's	100 gpm at 20-ft	Hydromatic, 3 hp	7-ft Diam.
Bellerive	2005	260 gpm at 80-ft	Flygt, 15 hp, NP 3153.180 HT, 229mm Imp	6-ft Diam.
Meadow Ridge	2007	245 gpm at 55-ft	Flygt, 10 hp, CP 3127.090 HT, 217mm Imp	6-ft Diam.
Dallas Rd	2012	260 gpm at 172-ft	Flygt, 35 hp, NP 3171 SH 3~ 277	8-ft Diam.

6.2.3 Flow Generation Layer

6.2.3.1 Water Meter Usage Data

Previous sewer modeling efforts assumed typical sanitary sewer unit flows based upon land use designation; however, a more precise method was utilized for the update of the existing hydraulic model. Sanitary unit flows for the existing model were based on recorded City water meter data from the period between December 2012 and February 2013. During these winter months, the vast majority of metered water used by customers is for potable use only (i.e. no irrigation) and discharged to the collection system. Therefore, use of meter data is a good indicator of base sanitary flow contribution. Since potable water service meter data was used, it provided actual usage data to generate sewer flows in the model rather than relying on typical unit flow data. This method yields a more precise representation of the existing flows in the system.

The average daily flow for each water meter was calculated from the average winter monthly volume recorded by each meter, yielding an average water use of 160 gpd per residential dwelling unit. Therefore an ERU was defined as 160 gpd. Based on 2.42 persons per household (reference **Section 2.11**), this yields a per capita flow of 66.1

gpd. Average daily flows were then adjusted by assumed peaking factors to reflect weekend and weekday diurnal curves. These diurnal curves were specific to each land use type and were adjusted during calibration of the model (See **Section 6.2.4**).

A majority of the City is characterized by residential flows. Since the highest average and peak residential flows usually occur on weekends, the majority of the trunk pipes will experience peak flows on the weekend; however, smaller basins with a high percentage of non-residential flows may experience peak flows during the weekdays. For example, a school generates the majority of its wastewater during the week, so the daily average was adjusted so that the majority of the flow is distributed throughout the week, and very little flow is distributed over the weekend. Therefore, the existing hydraulic model was built using factors to adjust the average daily flows from the water meters to average weekday and weekend flows to capture both maximum peak possibilities.

Currently, the City collects water meter data on a monthly basis, and the data is reported as a volume in hundreds of cubic feet. In the case of not being able to access a water meter for any reason (blocked by car, covered in snow, other) the City estimates the monthly value (on the lower end). The next month's meter reading might require a correction, however typically not a negative value.

6.2.3.2 Land Use

The land use types used in the existing hydraulic model are listed in **Table 6-2** and were generated from the land use codes provided with City water meter data.

Table 6-2 – Existing Model Land Use Types

➢ Assisted Living	➢ Open Space
➢ Church	➢ Public
➢ Commercial	➢ Residential - High Density
➢ Hospital	➢ Residential - Medium Density
➢ Hotel	➢ Residential - Low Density
➢ Industrial	➢ Restaurant
➢ Office	➢ School

The Residential Low Density land use type consists of all single-family dwelling units. Residential Medium Density consists of multi-family dwelling units with between two and four dwelling units, as well as condominiums, townhomes, mobile homes and RV parks. Residential High Density includes all apartments and multi-family dwelling units with greater than four dwelling units. Residential and non-residential unit flows are shown in **Table 6-3**.

Table 6-3 – Unit Flows

Parameter	Value
Residential Unit Flows [a]	
Low Density (Single family homes)	160 GPDU
Medium Density (Multi-family – 2 to 4-plex and condo/patio homes)	147 GPDU
High Density (Multi-family – >4-plex and apartments)	147 GPDU
Non-Residential Unit Flows [b]	
Assisted Living	3,300 GPAD
Church	150 GPAD
Commercial [c]	350 GPAD
Composite Commercial [d]	625 GPAD
Hospital	5,500 GPAD
Hotel	3,000 GPAD
Industrial [e]	60 GPAD
Industrial-Heavy [f]	3,000 GPAD
Office	350 GPAD
Public	540 GPAD
Restaurant	2,500 GPAD
School	170 GPAD

(a) *Based on 2.42 people per dwelling*
(b) *Based on winter water meter records divided by the net parcel area*
(c) *Includes range of commercial businesses, from convenience stores to big box stores.*
(d) *Combines commercial, hotel, restaurant, hospital, office, and public land-uses.*
(e) *Based on dry, non-significant, industrial flows only.*
(f) *Based on the 11 permitted City significant industrial users.*

Water meter data was coupled with land use to complete the flow generation layer. Although, the City's water meter data was supplied in spreadsheet format, the representing parcel number was also included and used to link the water meter to the parcel it served. Diurnal curves (the typical 24-hour shape of the flow) were then developed for each land use type. The initial diurnal curve patterns for each land use type were based on historical modeling experience and flow monitoring results. Diurnal curves for each land use type were then adjusted during calibration efforts to match the flow monitoring results. The diurnal curves used in the model can be found in **Appendix D**.

6.2.3.3 Infiltration and Inflow

Infiltration is groundwater entering the sewer through cracked pipes, faulty service connections or other deficiencies in the collection system. This can be groundwater from a high water table or rainfall induced groundwater. As mentioned in **Section 3.5**, the City has incorporated several inspection and rehabilitation measures to minimize infiltration. During calibration for the Existing Model, it was determined that there were specific areas within the City that accounted for more infiltration than others (as shown in **Figure 3-9**). This observation was confirmed by City staff.

Inflow is the flow of water directly into the sewer during and after a rainfall event due to direct connection to the sewer from storm drains, roof drains, parking lots, manholes, etc. The City has a low quantity of cross connections where storm drains or roof drains are connected to the sewer system. Therefore, inflow was assumed as 50 gpad.

6.2.4 Calibration

Calibration is the process of modifying various parameters and their assumed values in order to match flow monitoring data collected from multiple locations. Sewer flows were monitored at a total of eight locations in the collection system as shown on **Figure A3**. For the SRSR project (as described in **Section 6.2.2.1**) sewer flows were monitored at two locations, south of the Yakima River, between July 20th and August 8th, 2013. Six additional locations were selected and sewer flows were monitored between March 26th and May 12th, 2014. At each location, the City installed their flow monitoring equipment in the collection system manholes. During both flow monitoring periods there were no wet weather events with any measureable amounts of rainfall and therefore only dry weather flows were captured.

The flow monitoring data set, itself, has limitations that prevent a 'perfect' calibration between model output and real flows. Some of the factors affecting calibration include the level of uncertainty of the flow monitoring data and the low resolution of the water meter usage data. Additional limitations, not related to the flow monitoring data, include the diurnal curve patterns used, routing assumptions, normal fluctuations in wastewater flows from day-to-day, and the overall quantity of wastewater production. Considering these limitations, the model calibrated well, without significant changes to base assumptions or parameters, providing a high level of confidence in the existing model results and in the subsequent development of the master plan models. A complete listing of model assumptions and parameters are included in **Appendix C**.

6.2.4.1 Dry Weather Calibration

As discussed in **Section 6.2.3.1**, the model was calibrated to both weekend and weekday recorded flows. Recorded flows for individual days were plotted on a XY graph as the measured flow (in mgd) versus time (48 hour period of weekend and weekday) to show the uncertainty and variability of flow at any given point in the system. The flow monitoring data for the larger service areas showed less variability in flow values than the smaller service areas, which is a result of the number of customers upstream. An average weekend diurnal curve and average weekday diurnal curve were produced for each monitoring site based on the 30 minute average flow from the flow monitoring data. Final calibration graphs for dry weather flows are included in **Appendix D**.

6.2.4.2 Wet Weather Calibration

There were no wet weather events with any measureable amounts of rainfall during the flow monitoring period. Capturing a rainfall event during flow monitoring provides information regarding the impact the event has on the collection system at each flow monitoring site. Without a measureable event, a 3-hour short duration thunderstorm with a 2-year return period and a total rainfall amount of 0.424 inches (as per the DOE Stormwater Management Manual for Eastern Washington (SMMEW)) was used as the design storm. The short duration thunderstorm has a higher peak flow as compared to the 24-hour design storm. To simulate a worst-case condition in the model, the peak inflow from the storm event was aligned with the peak in the sanitary flow on the weekend. This results in a larger net return period for the storm event. The existing model includes a simulated rainfall event from the design storm. Final graphs for an assumed wet weather event are included in **Appendix D**.

6.3 Existing Model Analysis

6.3.1 Existing Model Analysis

The design storm discussed in **Section 6.2.4.2,** was incorporated into the calibrated model for analysis of the existing system capacity. Two measures of flow conditions in the collection system were used for evaluation of the existing model: flow depth over pipe diameter (d/D) and reserve capacity of the pipe. Based on these measures, there was only one minor instance of pipe surcharging and therefore, the existing collection system appears to have adequate capacity at today's flows.

Figures A4 and **A5** in **Appendix A** show Depth over Diameter (d/D) and Reserve Capacity for the Existing Model, respectively. Depth over diameter can be used to identify the extents of surcharging, and includes backwater effects from downstream pipe segments. Reserve capacity can be used to identify individual pipes that could be the root cause of the surcharging or limited capacity, but does not include the backwater effects from downstream pipe segments. This is evident when looking at both figures side by side. **Figure A5** shows several small instances of pipes being over capacity; however these issue pipes are not seen on **Figure A4** – these overcapacity pipes are pipes that were constructed nearly flat or with a reverse grade.

Table 6-4 contains a list of the areas where issues were identified in the system for the Existing Model. **Appendix E** has additional information and hydraulic grade line plots for each issue. **Appendix F** contains results from the Existing Model Analysis. Note that all existing model results and figures include the design storm event.

Table 6-4 – Existing Model Issues

Location	Issue	Reference	Recommended Action
Country Ridge Collector at Queensgate Drive	Surcharging ~ 0.10'	Appendix E, Section 2.1	Do Nothing / Monitor Pipe Flow Depth and Conditions

Table 6-5 contains a summary of each lift station and its remaining capacity given existing conditions. The lift stations have sufficient existing capacity.

Table 6-5 – Existing Model Lift Station Summary

Lift Station	Design Capacity (GPM)	Existing Peak Flow (GPM)(a)	Existing Peak Flow (% of Capacity)	Remaining Capacity (GPM)
Battelle	400	349(b)	87%	51
Waterfront	600	237	40%	363
Terminal Drive	150	19	13%	131
Mental Health	260	7	3%	253
Bradley	180	25	14%	155
Columbia Pt	270	58	21%	212
Wellhouse Loop	100	7	7%	93
Duportail	200	78	39%	122
Montana St	970	290	30%	680
Columbia Park Trail	400	41	10%	359
Meadows South	100	18	18%	82
Bellerive	260	82	32%	178
Meadow Ridge	245	7	3%	238
Dallas Rd	260	20	8%	240

(a) *Peak Flow values include a 10% factor of safety to reduce the potential for overloading the station. See Appendix C for further detail.*
(b) *Assumes Areva (SIU CR-IU008) is discharging at its MPL of 0.40 mgd.*

6.4 Committed Model

6.4.1 Introduction

The analysis of the Existing Model shows that the existing collection system is capable of handling existing design flows. The next step is to identify how the system will perform with future flows from areas to which the City has committed to provide service. The Committed Model represents the development within the entire UGA boundary; essentially everything the City has committed to serve and generally includes all parcels, developed or not. **Figure A6** identifies all the areas within the UGA that are currently undeveloped that were added to the Committed Model and developed based on their current land use. Additionally, the occupancy rates for multi-family housing are set equal to 100 percent to maximize sewer flows. The Committed Model is a tool to guide growth and expansion of the collection system and to also identify potential future deficiencies in the current collection system. The Committed Model's primary purposes are, therefore, as follows:

- Show the "uncommitted" capacity remaining in the collection system.
- Provide the size, approximate location, and depth for future sewer pipes 10 inches and greater in size to serve the UGA boundary.
- Identify potential capacity issues that may arise in the existing collection system as the City develops new areas beyond the City limits but within the UGA.
- Develop a base model to use in evaluating future wastewater service scenarios.

6.4.2 Committed Model System Layer

6.4.2.1 Trunk Pipes

The Committed Model system layer was developed to take advantage of existing and future public right-of-way and the low-lying areas along natural drainages. During the development of the system, the following information was taken into consideration:

- Selected master plan improvements identified in the SRSR project were included: the Bellerive Lift Station pump upgrade (600 gpm pump replacement) and East Badger South Lift Station.
- Development of the Badger South planned development was based on specific planning values provided by the developer's consulting engineer.
- The area west of the By-Pass Highway (SR 240) and southwest of the Richland Airport, currently zoned Agriculture, was not developed.
- Stevens Drive divides the upper North Richland area into two separate drainage basins with different land uses: the west area includes the Horn Rapids industrial park while the east area includes facilities related to the Pacific Northwest National Laboratory (PNNL).
- Undeveloped industrial area in and around the Horn Rapids Industrial Park (HRIP) was developed using a value of 1,250 gpad (as selected by the City) – this includes the Bechtel laydown yards generally south of Battelle Boulevard and west of Stevens Drive.
- Previous General Sewer Plan routing.

To reduce capital construction costs and operation and maintenance costs, the depth of future trunk pipes (10 inches and larger) was held to a minimum cover of 10-feet where possible while still providing service and minimizing the number of lift stations. It should be noted that these are planning level depths based upon the City's GIS contour layer data of 2-foot intervals. Detailed topographical survey will be needed on a project by project basis in order to refine pipe depths.

Sizing of future pipes was accomplished using the design parameters listed in **Table 6-6**. A portion of undeveloped land within the UGA has sufficient slope to allow trunk pipes to be constructed at steeper than minimum grade, thereby allowing for a reduction in trunk pipe sizes. In order to ensure that pipe downsizing, due to steeper than minimum slope, does not result in physical obstruction bottlenecks, care must be taken that trunks are designed and installed at the same or steeper slope than those listed in the results of the Committed Model. **Appendix G** lists the proposed sizes, inverts, and slopes of the future trunk pipes, and denotes any trunk pipes that require steeper than minimum slopes.

Table 6-6 – Future Pipe Design Parameters

Pipe Diameter (in)	Maximum Allowed Depth/Diameter	Minimum Slope
8	0.50	0.40%
10	0.55	0.28%
12	0.60	0.22%
15	0.65	0.15%
18	0.75	0.12%
21 to 30	0.75	0.10%
≥36	0.85	0.10%

The following is a list of assumptions used in the Committed Model:

- Each dwelling unit houses an average of 2.42 people based on the 2010 US Census data.
- The average sanitary flow per dwelling unit was 160 gpd, the residential unit flow determined during the calibration of existing model.
- Infiltration and inflow for future trunk pipes will be zero.
- Significant Industrial User's (SIU's), permitted by the City, are discharging at the Maximum Permitted Limit (MPL)
- Undeveloped residential areas were addressed by land use type and the following criteria (as described in **Appendix C**):
    - Low Density Residential – level of development based on parcel size
        - ≤ 1 acre = leave parcel flow as-is / no further development for Committed Model
        - > 1 acre = first reduce (by 23%) parcel size for non-buildable area, then subdivide parcel into 3.5 du/ac and multiply by the Low Density Unit Flow (160 gpdu) to calculate the Committed Model flows
    - Medium Density Residential – evaluate the density of each parcel based on the value of the following ratio: (Water Meter flow) / (number of du) / (Low Density Unit Flow)
        - ≥ 0.75 = leave parcel flow as-is for Committed Model
        - < 0.75 = update the parcel's Committed Model flow by the product (number of du)*(Low Density Unit Flow)
    - High Density Residential – same process as for Medium Density Residential

6.4.3 Committed Model Flow Generation Layer

6.4.3.1 Land Use and Unit Flows

Future flows were developed for the Committed Model which assume 100% development within the UGA boundary. **Figure A6** depicts the locations of all the assumed infill development.

Land use designations for the Committed Model were determined from existing land use and zoning designations as well as discussions with the City. The unit flows identified in **Table 6-3** were used to generate the Committed Model flows.

6.4.3.2 Flow Allocation

Similar to the Existing System Model, each parcel in the Committed Model service area was modeled by injecting flow into the nearest upstream manhole in the system layer. Some large master plan parcels were divided according to the proposed land use configurations and flows then injected into multiple locations based on topography and trunk pipe serviceability.

It is important to note that the service area boundaries for each trunk pipe within the Committed Model are based on aerial mapping and City/County contours, and therefore are approximate. Individual service area boundaries may change slightly as field survey is performed and development occurs. While safety factors built into the model allow for these minor changes, significant proposed changes or the cumulative effect of minor changes should be analyzed to prevent over-allocation of trunk capacity in the future.

6.5 Committed Model Analysis

6.5.1 Committed Model Analysis

The Committed Model analysis provides results assuming development within the entire UGA boundary, without the addition of any relief pipes or the correction of existing system deficiencies. This helps identify the priorities for Capital Improvement Projects. It should be noted that the Committed Model predicts that the total average daily flow at the WWTP will be 12.64 mgd, while in **Table 3-10** the average predicted flow to the WWTP in 20-years will be 8.24 mgd. As noted in the table, the 20-year flow is based on specific growth rate values for residential, commercial and industrial land uses; therefore, the Committed Model represents growth beyond the 20-year planning projections.

Figures A7 and **A8** show Committed Model results for Depth over Diameter and Reserve Capacity, respectively. As previously noted, depth over diameter can be used to identify the extents of surcharging, and includes backwater effects from downstream pipe segments; while reserve capacity can be used to identify individual pipes that could be the root cause of the surcharging or limited capacity, but does not include the backwater effects from downstream pipe segments.

Appendix G contains a tabular layout, by pipe model ID, of the results from the Committed Model analysis. The results list the upstream and downstream manhole information including rim and invert elevation and the data source, the pipe length, diameter, and slope, and Committed Model results including flow, velocity, d/D and reserve capacity. All Committed Model results and figures include the design storm event.

Table 6-7 contains a list of the areas where issues were identified in the system for the Committed Model. Each problem reach is identified by the general location and is discussed in detail in **Appendix E**. The issues are also grouped into the applicable CIP project number used in **Table 7-1** of **Chapter 7**.

Table 6-7 – Committed Model Pipe Capacity Issues

Location	Issue	Identified Under Existing Model Analysis	Reference	Recommended Action
Country Ridge Collector to Yakima River	Overflow		Appendix E, Section 3.1	Replace Pipe with Larger Diameter – Developer Driven Improvement
Leslie Rd Trunk Near Col. Park Trail	Pipe Nearing Capacity d/D = 0.95		Appendix E, Section 3.2	Replace Pipe Section – See *CIP **CP.1** for details*
Keene Rd Collector At Keene/Gage Int.	Surcharge ~ 0.10-ft		Appendix E, Section 3.3	Replace Pipe Section – See *CIP **CP.2** for details*
Upper North Interceptor	Surcharging of Local Collectors and Residential Services		Appendix E, Section 3.4	Replace Pipe Section – See *CIP **CP.3** for details*
Bellerive LS Downstream Piping	Surcharge ~ 3.0-ft		Appendix E, Section 3.5	Replace Pipe Section – See *CIP **CP.4** for details*
Logston Interceptor Logston Blvd	Pipe Nearing Capacity d/D = 0.86		Appendix E, Section 3.6	Do Nothing / Monitor Pipe Flow Depth and Conditions
Airport Collector On Hagen Rd	Pipe Nearing Capacity d/D = 0.82		Appendix E, Section 3.7	Do Nothing / Monitor Pipe Flow Depth and Conditions
Hwy 240 Interceptor Highway Crossing	Flat & Reverse Grade, Pipe Nearing Capacity d/D = 0.80		Appendix E, Section 3.8	Do Nothing / Monitor Pipe Flow Depth and Conditions

The City's lift stations, with the Committed Model scenario peak flow and remaining capacity are listed in **Table 6-8.**

Table 6-8 – Committed Model Lift Station Summary

Lift Station Name	Design Capacity (GPM)	Committed Model Peak Flow (GPM)[a]	Committed Model Peak Flow (% of Capacity)	Remaining Capacity (GPM)
Battelle[b]	400	349[b]	87%	51
Waterfront	600	260	43%	340
Terminal Drive	150	22	15%	128
Mental Health	260	7	3%	253
Bradley	180	84	47%	96
Columbia Pt	270	84	31%	186
Wellhouse Loop	100	10	10%	90
Duportail[c]	200	222[c]	Exceeds Capacity	-22
Montana St	970	298	31%	672
Columbia Park Trail	400	52	13%	348
Meadows South	100	20	20%	80
Bellerive[d]	260	540[d]	Exceeds Capacity	-280[d]
Meadow Ridge	245	11	4%	234
Dallas Rd[e]	260	2,450[e]	Exceeds Capacity	-2190[e]

(a) *Peak Flow values include a 10% factor of safety to reduce the potential for overloading the station. See Appendix C for further detail.*

(b) *See Section 6.6.2.1 for a discussion of build-out of the Battelle Lift Station drainage basin and peak influent flow conditions.*

(c) *See Section 6.6.2.2 for a discussion of build-out of Duportail Lift Station drainage basin and peak influent conditions.*

(d) *See Section 6.2.2.3 for a discussion of build-out of the Bellerive LS drainage basin and peak influent flow conditions.*

(e) *See Section 6.2.2.4 for a discussion of build-out of the Badger Mountain South development, the Dallas Road LS drainage basin and peak influent flow conditions.*

6.5.2 Committed Model Lift Station Analysis

Table 6-8 provides the results of the Committed Model scenario peak flows into the City's 14 existing lift stations. The difference between the existing lift station capacity and the peak flows is listed. Note that the peak flow values as predicted by the model include a 10% factor of safety (See **Appendix C** for further detail). As noted in the table, four lift stations have negative values which indicate additional capacity will be required to meet the Committed Model scenario peak flows. Each of these four lift stations are discussed in further detail in the following sections.

6.5.2.1 Battelle Lift Station

During the Committed Model scenario, it was assumed that Areva (SIU CR-IU008) would be discharging at its maximum permitted limit (MPL) of 0.40 mgd and a new gravity collection pipe would be constructed along Battelle Blvd, east of the lift station, to serve current and undeveloped areas. It was assumed the undeveloped areas along Battelle Boulevard would be developed at the industrial unit flow of 1,250 gpad. This new gravity pipe would provide a bypass around the existing lift station and reduce influent flows to only those from Areva. The model identified that peak flows into the Battelle Lift Station would be roughly the same as during the Existing Model scenario, at 350 gpm

(0.50 mgd), since other flows are relatively minor. Note that in order to connect to existing sewer services and maintain a minimum of 8-ft of cover, the pipe should be 12-in in diameter and constructed at a slope of 0.10%.

In January 2016, Areva has agreed to a modified permit limit of 0.258 mgd; lower than the current MPL.

6.5.2.2 Duportail Lift Station

During the Committed Model scenario, it was assumed that the apartments within the drainage basin are all at full (100%) occupancy and that a currently undeveloped area (44 acres) north of the lift station would be developed at a density of 5 units/acre using a unit flow of 160 gpdu (Note that this also excludes 23% of the area for roads and landscaping). Based on these assumptions, the model identified that peak flows into the Duportail Lift Station would be roughly 200 gpm (0.29 mgd), which matches the capacity of one of the existing pumps. It should be noted that the existing lift station is within the proposed alignment of the Duportail Bridge and Roadway extension project and is planned for relocation. (See **Figure 6-3** for additional detail) At this time it is not known when this improvement will occur, although it is predicted to take place within the next 5 to 10 years. At that time, larger pumps with additional capacity should be selected for the new lift station.

As noted in **Table 6-8**, the factored peak flows into the lift station are approximately 222 gpm and the flows are then pumped through an existing 6-inch force main. At this peak flow the flow velocity through the force main will be approximately 2.5 fps, therefore greater than the minimum velocity required for self-cleaning (2 fps) but less than the maximum recommended velocity (8 fps) as listed in Ecology's Orange Book reference. Downstream of the forcemain discharge manhole is the 12-inch Bypass Highway Interceptor and the Committed Model scenario identifies this pipe has 0.60 mgd of reserve capacity for this additional flow.

6.5.2.3 Bellerive Lift Station

During the Committed Model scenario, all undeveloped area within a lift station drainage basin was developed at its current zoning and routed to the lift station to account for future development – this also includes converting homes currently on septic systems. Within the Bellerive Lift Station drainage basin is the existing Rancho Reata neighborhood; therefore to identify ultimate build-out conditions for the lift station two scenarios were considered: Rancho Reata *not-included*, and Rancho Reata *included*. Given these two scenarios, the model identified that peak flows into the lift station were roughly 425 gpm (0.61 mgd) and 490 gpm (0.71 mgd), respectively. Each of the existing lift station pumps only has a capacity of 260 gpm (0.37 mgd) and therefore additional capacity will be required with build-out of this drainage basin area. (See **Figure 6-4** for additional detail)

As noted in **Table 6-8**, the factored peak flows into the lift station are approximately 550 gpm (assuming Rancho Reata *included*) and the flows are then pumped through an existing 6-inch force main. At this peak flow the flow velocity through the force main will be approximately 5.6 fps, therefore greater than the minimum velocity required for self-cleaning (2 fps) but less than the maximum recommended velocity (8 fps) as listed in Ecology's Orange Book reference. The forcemain currently discharges into 8-inch gravity sewer collection piping along Bellerive Drive, north of Gage Boulevard. At this location the gravity sewer pipe has a pipe slope of approximately 1.5% with a reserve capacity of approximately 0.40 mgd, however further downstream, the pipe slope flattens to approximately 0.50% and the reserve capacity decreases to 0.30 mgd. Surcharging occurs here and continues upstream for roughly 3,500-feet. Replacing the 8-inch piping with 12-inch piping (and matching the existing slope) will resolve the surcharging of the downstream collection system.

6.5.2.4 Dallas Road Lift Station

During the Committed Model scenario, all of the Badger South development was considered developed per its master plan. To handle the master plan flows, the developers planned to route sewer flows, generated on the east half of the development, to an east Badger South lift station that would be positioned near the northeast corner of the Badger South development. The East Badger South Lift Station is shown as a master planned lift station on **Figure 6-5**. The wastewater would then be pumped to the west and to the West Badger South Lift Station, a master plan lift station, adjacent to the existing Dallas Road lift station. The developer intends for the West Badger station to be a central lift station to service the entire Badger South development. Wastewater flows would then be pumped along the same alignment the Dallas Road forcemain follows – eventually discharging into the gravity collection system in Country Ridge. (See **Figure 6-5** for additional detail)

As noted in **Table 6-8**, the factored peak flows for build-out of the development at the west, central lift station are approximately 2,450 gpm (3.5 mgd). Influent flows at the lift station are from the West Badger South area and from the East Badger South lift station. Peak flows for West Badger South area are approximately 1,000 gpm (1.44 mgd) and the capacity of the East Badger South lift station pumps are approximately 1,300 gpm (1.87 mgd). At the West Badger South lift station flows are pumped through an existing 12-inch force main. At the factored peak flow the flow velocity through the force main will be approximately 7.0 fps, therefore greater than the minimum velocity required for self-cleaning (2 fps) but less than the maximum recommended velocity (8 fps) as listed in Ecology's Orange Book reference. As previously noted, the forcemain discharges into 8-inch gravity sewer collection piping through Country Ridge. At this location the gravity sewer pipe is roughly constructed at minimum slope and has less than 0.75 mgd of reserve capacity. Routing the Badger South flows through this existing collection system causes surcharging which then leads to localized flooding at several manhole locations. The surcharging continues as the collector crosses Keene Road and Queensgate Drive for a total of roughly 12,000-lineal feet of 8-inch piping that will need replacement.

Figure 6-1 – Intentionally Left Blank

City of Richland
2015 General Sewer Plan

Figure 6-2
Battelle Lift Station

Legend

- City Limits
- Interstate/Highway
- Major Streets
- City Lift Stations
- Manhole

Pipe Size (in)

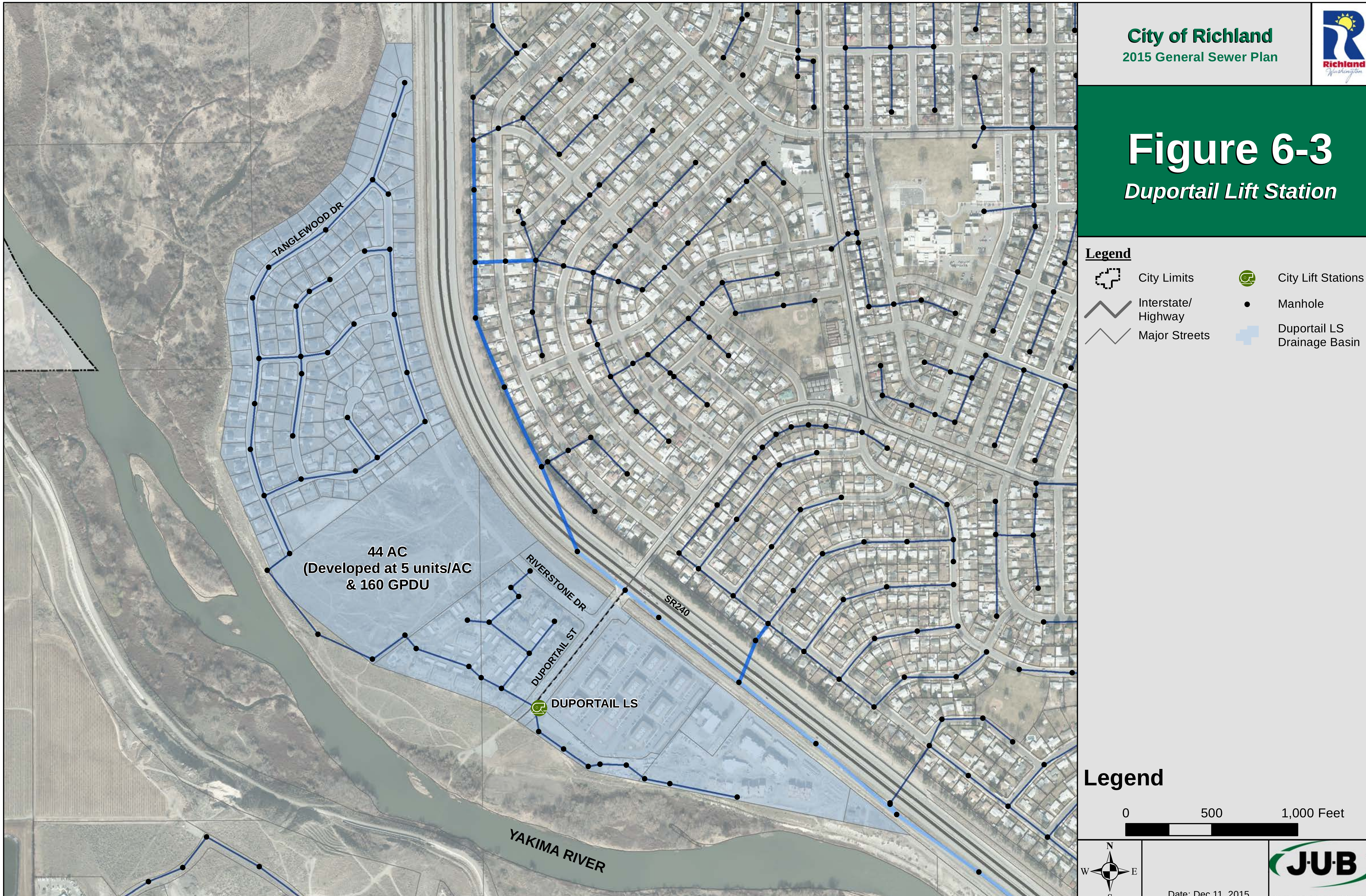
- Collector
- 10
- 12
- 15
- 18
- 21
- 24
- 27
- 30
- 36
- 42
- 54
- Forcemain

0 500 1,000 Feet

N S W E

Date: Dec 11, 2015

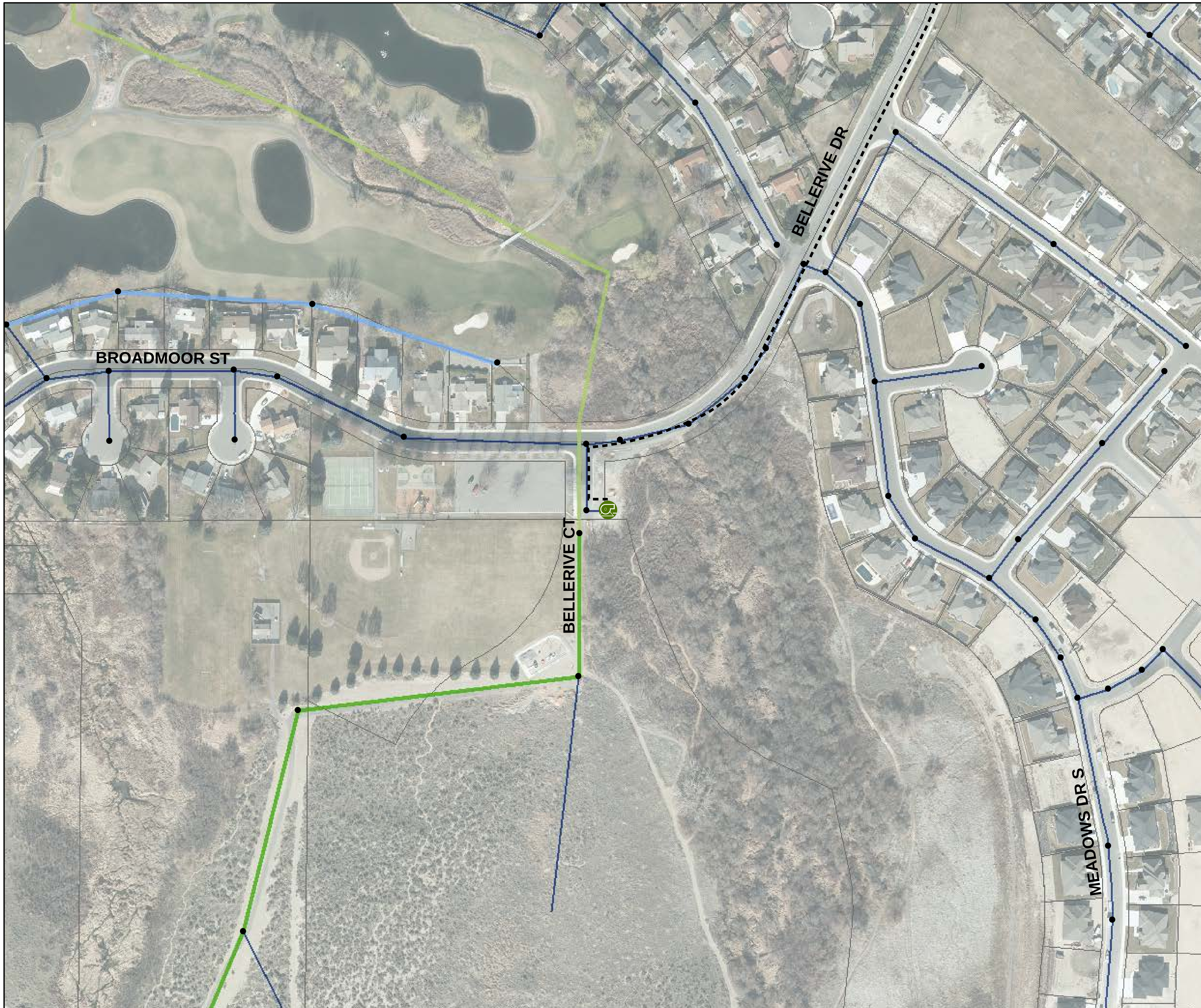
J·U·B ENGINEERS, INC.



City of Richland
2015 General Sewer Plan
Richland Washington
Figure 6-3
Duportail Lift Station
Legend
City Limits
Interstate/ Highway
Major Streets
City Lift Stations
Manhole
Duportail LS Drainage Basin
TANGLEWOOD DR
44 AC
(Developed at 5 units/AC
& 160 GPDU
RIVERSTONE DR
DUPORTAIL ST
SR240
DUPORTAIL LS
YAKIMA RIVER
Legend
0
500
1,000 Feet
N
W
E
S
Date: Dec 11, 2015
J·U·B
J·U·B ENGINEERS, INC.

City of Richland
2015 General Sewer Plan

Figure 6-4
Bellerive Lift Station



Legend

- City Limits
- Interstate/Highway
- Major Streets
- City Lift Stations
- Manhole

Pipe Size (in)

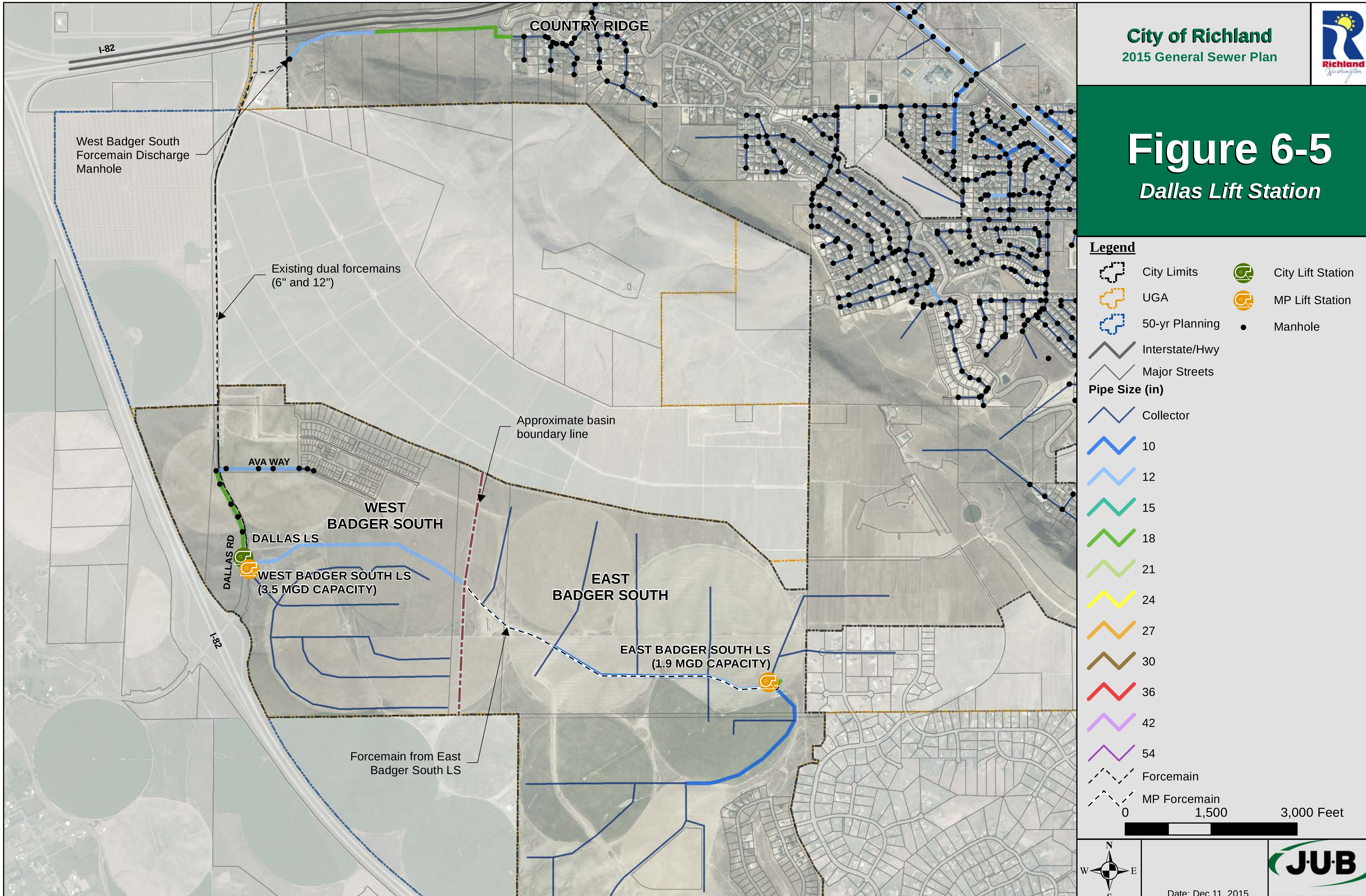
- Collector
- 10
- 12
- 15
- 18
- 21
- 24
- 27
- 30
- 36
- 42
- 54
- Forcemain

0 200 400 Feet

N W E S

Date: Dec 2, 2015

J·U·B ENGINEERS, INC.


City of Richland
2015 General Sewer Plan
Figure 6-5
Dallas Lift Station
Legend
City Limits
UGA
50-yr Planning
Interstate/Hwy
Major Streets
City Lift Station
MP Lift Station
Manhole
Pipe Size (in)
Collector
10
12
15
18
21
24
27
30
36
42
54
Forcemain
MP Forcemain
0
1,500
3,000 Feet
Date: Dec 11, 2015
J·U·B ENGINEERS, INC.
COUNTRY RIDGE
I-82
West Badger South Forcemain Discharge Manhole
Existing dual forcemains (6" and 12")
Approximate basin boundary line
AVA WAY
WEST BADGER SOUTH
DALLAS LS
DALLAS RD
WEST BADGER SOUTH LS (3.5 MGD CAPACITY)
EAST BADGER SOUTH
EAST BADGER SOUTH LS (1.9 MGD CAPACITY)
Forcemain from East Badger South LS

6.6 Master Plan Model

6.6.1 Master Plan Model Analysis

With modern materials and construction methods, it is expected that sewer pipes will exceed fifty years of service before rehabilitation or replacement is necessary. Therefore, a Master Plan Analysis is prepared as a model scenario by planning to a 50-year boundary. This ensures that all projects identified for the CIP (at the Committed Model stage) are further upsized to handle planned build-out flows – thus ensuring the pipes will provide reserve capacity for their design life. The 50-year boundary was identified by the City as the UGA boundary along the south part of Richland and the UGA boundary along the north part of Richland including a 1,350 acre portion of a land transfer from the Department of Energy's 300 Area at the request of TRIDEC.

Table 6-9 contains a list of the issues identified in the Master Plan Model. Similar to the existing and committed model results, each issue is identified by the interceptor name or general location and is discussed in detail in **Section 6.7.2** and included in **Appendix E**. The issues also reference the applicable CIP project number used in **Table 7-1** of **Chapter 7**. **Figure A10** identifies the Master Plan Model pipe sizes.

Between the initial conceptual layout and the final model results, several alignment changes were made to provide service to the study area extents, minimize the sewer depths, and eliminate the need for lift stations. The Master Plan scenario was developed so that the majority of the future trunk pipes are at planned depths of less than 20 feet below the existing ground surface, as shown in **Figure A11**, with the exception of portions of the collection system extension along Horn Rapids Road and adjacent to SR 240. In these basins, the ground topography varies widely with existing drainage depressions; however, as planned development extends to these areas, the pipe depths are expected to decrease.

Figures A12 and **A13** show the Depth over Diameter and the Reserve Capacity, respectively, for the Master Plan Model based on the existing pipe sizes. As previously noted, depth over diameter can be used to identify the extents of surcharging, and includes backwater effects from downstream pipe segments; while reserve capacity can be used to identify individual pipes that could be the root cause of the surcharging or limited capacity, but does not include the backwater effects from downstream pipe segments.

Appendix H contains results from the Master Plan Model Analysis. All Master Plan Model results and figures include the design storm event.

Table 6-9 – Master Plan Model Issues

Location	Issue	Identified Under Committed Model Analysis	Reference	Recommended Action
Country Ridge Collector to Yakima River	Overflow	X	Appendix E, Section 3.1	Replace Pipe with Larger Diameter – Developer Driven Improvement
Leslie Rd Trunk Near Col. Park Trail	Surcharge ~ 2.0-ft	X	Appendix E, Section 3.2	Replace Pipe Section – See *CIP **CP.1** for details*
Keene Rd Collector At Keene/Gage Int.	Surcharge ~ 0.10-ft	X	Appendix E, Section 3.3	Replace Pipe Section – See *CIP **CP.2** for details*
Upper North Interceptor	Surcharging of Local Collectors and Residential Services	X	Appendix E, Section 3.4	Reconfigure Interceptor and New Lift Station – See *CIP **CP.3** for details*
Bellerive LS Downstream Piping	Surcharge ~ 3.0-ft	X	Appendix E, Section 3.5	Replace Pipe Section – See *CIP **CP.4** for details*
Logston Interceptor Logston Blvd	Several Segments at Full Flow d/D ~ 1.00	X	Appendix E, Section 4.1	Do Nothing / Monitor Pipe Flow Depth and Conditions
Hwy 240 Interceptor Highway Crossing	Flat & Reverse Grade, Pipe Nearing Capacity d/D = 0.85	X	Appendix E, Section 3.7	Do Nothing / Monitor Pipe Flow Depth and Conditions
Airport Collector On Hagen Rd	Pipe Nearing Capacity d/D = 0.82	X	Appendix E, Section 3.8	Do Nothing / Monitor Pipe Flow Depth and Conditions

6.6.2 Assessment of Master Plan Model Results

The following sections discuss the issues identified in the Master Plan Model and provide further detail. Several of the issues are grouped together and discussed as a whole based on their location in the collection system. It should be noted that each of these system issues were discussed in the "Committed Model" discussion and are recommended to be further upsized in order to provide capacity for the Master Plan flows.

6.6.2.1 Country Ridge Collector and Badger South Development

As development continues within the Badger South area, more flow will be routed through the Country Ridge Collector. The existing collector pipe is 8-inch diameter both through the Country Ridge development and downstream to the intersection of Queensgate Dr and Jericho Rd (approximately 7,600 LF). It then increases to a mix of 12-inch and 15-inch diameter pipe and connects to the Yakima River inverted siphon crossing (approximately

4,500 LF). The bottleneck in this collection system is through the 8-inch piping, which was mainly constructed at minimum slope (0.40%) and only has a reserve capacity of 0.40 mgd at existing flows. Note that the capacity of the Dallas Road lift station is 260 gpm (0.37 mgd); therefore improvements to the Country Ridge Collector should precede a pump upgrade to the lift station.

The Master Plan Model identifies at build-out that the peak flow from the Badger South development will be approximately 2,300 gpm (3.3 mgd). This flow is out of the large planned lift station to be constructed adjacent to the existing Dallas Road lift station. Downstream pipe improvements through Country Ridge and to the Yakima River (approximately 12,000 LF) should be sized to accommodate this future peak flow. Using the Master Plan Model, it was determined that 18-inch gravity collection pipe (matching the existing pipe grades) would convey the future lift station discharge.

6.6.2.2 Leslie Road Trunk

In South Richland, drainage basins M, N, O, and the majority of P (see **Figure 1-1** for basin reference) drain half the City through a 21-inch trunk sewer pipe on Leslie Road – making this a critical part of the collection system. The trunk pipe follows Leslie Road northward and downhill toward Columbia Park Trail at a 6% grade before flattening to a 0.50% slope at the bottom of the hill. At this grade change the trunk pipe also decreases in diameter from 21-inch to 18-inch for a length of 120-feet before increasing in size to 30-inch. Both the Committed and Master Plan Model identify that surcharging will occur during these scenarios. Peak flows through the trunk pipe are approximately 4.7 mgd and surcharging of up to 2-feet occurs in the 18-inch section of piping.

6.6.2.3 Keene Road Collector

The Keene Road Collector is a 12-inch pipe that drains residential developments, on either side of Keene Road, and routes flows to the east and eventually to the Leslie Road trunk. From record drawings and previous sewer model data it was determined that the collector decreases in diameter to 10-inch pipe for a short 900-LF section before increasing to 12-inch. This bottleneck is located on Keene Road, just north of the intersection of Keene Road and Gage Boulevard. Both the Committed and Master Plan Model identify that the amount of surcharging that occurs here is minor (0.10-ft); however a decrease in diameter can prevent larger objects in the collection system from passing through.

6.6.2.4 Upper North Interceptor and Service Backups

The Upper North Interceptor (UNI) is located in a residential area of North Richland, generally north of McMurray Street and east of George Washington Way (G-Way). It is approximately 15,000 of 18-inch and 24-inch concrete interceptor pipe that drains from the diversion structure near the intersection of G-Way and University Drive down to a connection with the Lower North Interceptor (54-inch pipe) at the intersection of McMurray Street and G-Way. (See **Figure A1** for reference) The lower half (18-inch and 24-inch) of the UNI was constructed in the 1970's while the upper half (24-inch) was constructed in 1997. City sewer crews' note that there are several dropped pipe joints along the UNI that have been observed during routine CCTV inspection. Crews also note that homeowners, in specific areas along the UNI, have complained about backups or overflows into their basements when the UNI flows at or greater than half full. For that reason the diversion structure has been adjusted to keep flows below half pipe flow.

As part of this General Sewer Plan two alternatives were considered to resolve the service backups caused by the UNI. However, the Master Plan Model was first used to confirm that all master plan flows can be diverted away from

the UNI through control of the diversion structure. This was confirmed using the Master Plan Model and no surcharging was observed.
Working with City staff, the affected neighborhoods within this area of North Richland were identified and two improvement alternatives were considered:

A. Construct New Connection Piping on Davison Avenue for UNI & Construct New Lift Station at the Water Treatment Plant (WTP) (See **Figure 6-5**)

   With this alternative approximately 2,400-LF of new 18-inch piping would be constructed along Davison and connect two existing UNI manholes. The existing UNI piping at each manhole would be plugged to route flows down the new piping – bypassing a neighborhood area that has had service backups. This "disconnected" area would instead drain to the new lift station located in an open space adjacent to the WTP. The new lift station would serve several small cul-de-sac's near the intersection of Saint St and Davison Avenue.

B. Construct a Compact Lift Station near McArthur & Alexander & Construct New Lift Station at the Water Treatment Plant (WTP) (See **Figure 6-5**)

   With this alternative two lift stations would be constructed to alleviate service backups. A compact lift station with a small footprint would be located along Alexander Avenue, between MacArthur and Spengler Streets. This lift station would serve approximately 15 homes that are at a greater likelihood of a service backup. It would discharge back into the UNI. The new lift station at the WTP is also a part of this alternative, however fewer homes would be connected and therefore a smaller pump size may be considered.

The City did not choose an alternative at this time, but will revisit both alternatives and better identify the affected homes at the time this project begins design.

6.6.2.5 Bellerive Lift Station Drainage Basin

As previously noted in **Section 6.6.2.3,** within the Bellerive Lift Station drainage basin is the existing Rancho Reata neighborhood which is not currently in the City limits but is within City's UGA and the overall area of where this lift station can serve; therefore to identify ultimate build-out conditions for the lift station two scenarios were considered: Rancho Reata *not-included*, and Rancho Reata *included*. Given these two scenarios, the model identified that peak flows into the lift station were roughly 425 gpm (0.61 mgd) and 490 gpm (0.71 mgd), respectively. Each of the existing lift station pumps only has a capacity of 260 gpm (0.37 mgd) and therefore additional capacity will be required with build-out of this drainage basin area. The forcemain currently discharges into 8-inch gravity sewer collection piping along Bellerive Drive, north of Gage Boulevard. At this location the gravity sewer pipe has a pipe slope of approximately 1.5% with a reserve capacity of approximately 0.40 mgd, however further downstream, the pipe slope flattens to approximately 0.50% and the reserve capacity decreases to 0.30 mgd. Surcharging occurs here and continues upstream for roughly 3,500-feet. Replacing the 8-inch piping with 12-inch piping (and matching the existing slope) will resolve the surcharging of the downstream collection system.

It is important to note that previous planning for the Badger South development identified approximately half the developed flow to be routed east and to the Bellerive Lift Station. This planning discussion is included in the SRSR memo included in **Appendix B**. The concept included the design of the East Badger South Lift Station with a discharge forcemain routed along I-82 and down to the current endpoint of the Leslie Interceptor, near the intersection of Leslie Road and E Reata Road. This increase of sewer flows to the existing lift station would surpass a minor pump upgrade and instead require the construction of a larger lift station facility with a trench style design. An alternative to a new lift station was considered mainly in an effort to minimize long term O&M. The alternative project was the Meadow Springs Interceptor, which was an extension of the Leslie Interceptor. The alignment generally followed northward and across the Meadow Springs golf course, under Gage Boulevard and through the Canyon Terrace neighborhood to connect to existing 21-inch sewer north of the intersection of Leslie Road and Canyon Street. The Meadow Springs Interceptor was planned as 21-inch pipe through the golf course, then increasing in pipe size, north of Gage Boulevard, to 24-inch.

6.6.2.6 Logston Interceptor

The Logston Interceptor currently extends from Logston Boulevard north to Battelle Boulevard and serves the Horn Rapids Industrial Park area. The 9,000-LF interceptor pipe is 24-inch diameter and was constructed at 0.08% slope with several sections constructed at slopes (0.06%) flatter than the design. The capacity of a 24-inch pipe flowing full and at 0.08% slope is approximately 4.48 mgd. The Committed Model scenario identified that with the build-out of the Industrial Park and areas within the City's UGA (at the unit flow of 1,250 gpad), the interceptor would have an average depth over diameter (d/D) value of 0.72 and a d/D value of 0.86 at any bottlenecks (where 0.06% slope). The average reserve capacity during this scenario is 1.05 mgd with a value of 0.15 mgd at its bottleneck.

Under the Master Plan scenario, an additional 1,350 acres of area north of Horn Rapids Road and the UGA is expected to be added as part of a TRIDEC request area from the Department of Energy. The City plans for this area to be zoned industrial with the same unit flow (1,250 gpad). To best serve this additional area, the Logston Interceptor was extended at minimum slope (0.10%) northward over existing ground. The proposed area is generally flat with the existing ground sloping approximately 0.50% to the south. The proposed alignment for the Logston extension generally follows the existing ground low points and would first constructed to the west along Battelle Boulevard, approximately 1,000-feet, to the western properly line of the Areva facility. Construction would then turn northward, crossing Horn Rapids Road, and following the existing low areas of ground. The Master Plan Model scenario identified that with the additional TRIDEC area the Logston Interceptor would be at full flow capacity where constructed at 0.08% with areas of minor surcharging. It is important to note that the trunk pipe in this area has an average bury depth of 8-feet and there are no sewer services directly connected which would allow for a minimum amount of surcharging during only peak flow conditions.

6.6.2.7 SR 240 Crossing

Both the Committed and Master Plan Model identified the existing 18-inch pipe crossing the SR 240 highway was constructed at nearly a flat grade and that a connecting pipe was constructed with a reverse grade; causing wastewater to collect and puddle at low points along the piping. The affected piping includes a 200-LF bore crossing under the highway and a 220-LF section of reverse grade piping. It appears that additional 18-inch piping may be required to "catch" the existing piping invert elevation using the minimum pipe slope for 18-inch (0.12%).

6.6.2.8 Airport Collector

Both the Committed and Master Plan Model identified a 400-LF section of existing 12-inch pipe, just north of the Richland Airport, was nearing capacity (d/D = 0.82). In this area the pipe slope transitions from steeper than minimum slope to minimum slope. A hydraulic jump occurs at the manhole with the transition of pipe slope; however given the location of where this issue occurs (industrial area and near an airport) the City may choose to simply monitor future conditions for any issues.

6.7 Renewals and Replacements

As documented in **Section 6.3**, the hydraulic analysis of the sewer collection system trunks (10-inch and larger) indicated very few hydraulic bottlenecks in the existing collection system. However, the City has an aged collection system with known condition issues. Therefore, the City's Capital Improvement Plan (CIP) for the next ten years will included a renewed focus on a repair/replacement program for older portions of the collection system based upon condition and risk of failure factors. The replacement program prioritizes sewer pipes with the greatest need for replacement each budget year. The City's goals for the replacement plan included the following:

- Utilize and leverage existing GIS data, existing pipe condition assessment data and hydraulic model results.
- Develop a prioritizing ranking of all pipes in the collection system.
- Develop comprehensive prioritization tool to help City staff make informed engineering decisions about which sewer pipes to target with roadway and water system repair/replacement planning efforts.
- Provide ability for City staff to update the prioritization 'in-house'.
- Summarize priority pipes into projects that include budget level costs and preliminary construction method recommendations.

Through discussions and meetings with the City a prioritization method combining pipe condition, risk and hydraulic capacity was developed. This method is included in **Appendix Q**. Manhole condition was not included in the prioritization at this time, but could be added in the future as standardized manhole condition rating methods are adopted and applied (e.g. NASSCo-MACP).

6.7.1 Prioritization Criteria and Weighting

The prioritization method is composed of three main categories including likelihood of failure (pipe condition), consequence of failure (risk), and hydraulic capacity. The majority of the criteria in each category involved processes and/or data that are already collected by the City. Through workshops with City staff, each category and criteria was assigned a certain weighting value to reflect relative importance. The scoring results using this weighting helped to provide the City with an initial evaluation of their collection system and because these weights can be easily modified, they will likely be adjusted and fine-tuned over time as the City implements the replacement and rehabilitation program.

The Likelihood of Failure (LoF) category generally represents the condition of the pipe. It includes the following criteria:

- PACP Rating – standardized condition rating score from City's Granite XP scoring software.
- Pipe Material – clay, concrete, and steel pipes are given a higher weight, while PVC is given a lower weight.
- Pipe Age – pipe age is based on GIS data or estimated, with older pipes given a higher weight
- Time Since Last CCTV Inspection – pipes with a greater number of years since the last CCTV inspection are given a higher score.

- Cleaning Frequency – based on the City's Problem & Maintenance (PM) list where pipes with a higher frequency of cleaning are given a higher score.

The Consequence of Failure (CoF) category represents the risk and impact of a pipe failure and includes the following criteria:

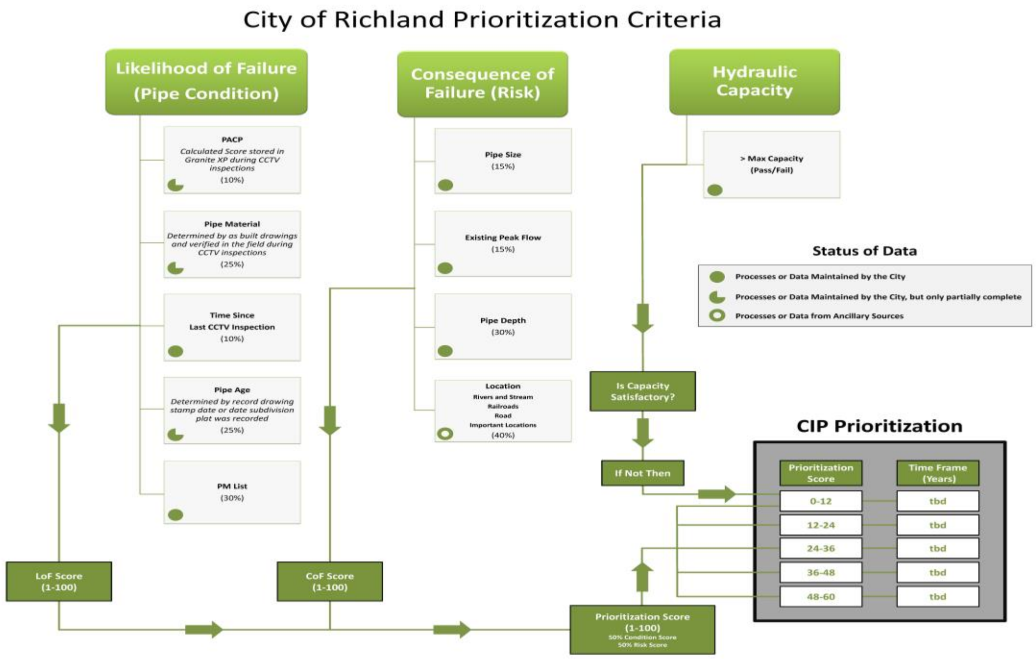
- Pipe Size – larger pipes generally serve a greater number of customers and have greater impact on disruption of service.
- Peak Flow – higher flows are given a higher weight because of the potential for overflows and backups to larger portions of town.
- Location – pipes that cross or run nearby existing water sources (well fields) and waterways are given a higher weight due to the potential environmental risks. Pipes that cross or are located in main arterial roads or cross railroads are given a higher weight due to the increased nuisance to citizens and disruption of service. Pipes that provide service to important locations such as hospitals, schools and major business centers that are sensitive to disruptions of service are given higher weights as well.
- Pipe Depth – deeper sewers are given higher weights due to the additional time required for repairs and increased disruption of service.

The Hydraulic Capacity category utilizes the current sewer model results to assign the highest priority to sewer lines that are over capacity – as these are automatically targeted for replacement independent of condition rating. As noted in **Section 6.3**, the City does not have any existing pipes that are over capacity and therefore this category was not used.

A score (0-100) was then assigned to each pipe for each scoring criteria. **Appendix Q** includes a detailed explanation of each criterion and how scores are calculated. Data field calculations are performed using functionality built into ESRI ArcMap. Each criterion score is then multiplied by the weighting factor and the two categories are summed individually. This results in a CoF score (0-100) and a LoF score (0-100). These two categories are then combined using the category weighting factors to establish a priority score for each pipe (0-100). An explanation of how the weighting factors are determined for both the overall categories (CoF and LoF) and each category criterion is included in the following paragraph. The higher the overall priority score the greater priority should be given for pipe rehabilitation or replacement. Several iterations were conducted by varying the weighting for various criteria. **Figure 6-6** outlines the final weighting criteria and prioritization process that was used.

In regards to the weighting factors used for both the LoF and CoF categories and the criterion within each category, generally little weight was assigned to pipe condition data criterion of the LoF category because the City does not have a significant amount of reliable pipe condition data. As noted in **Section 6.8**, developing an inventory of existing pipe condition is a goal for the City's replacement program. A significant amount of weight was associated to the pipe materials and age criterion of the LoF category because there is a reliable source of this information in the City's GIS system. A significant amount of weight was also associated to areas where the operations staff visits regularly for routine maintenance (i.e. Cleaning Frequency in the LoF category). In the CoF category, more weight was given to the location/proximity and pipe depth criterion and less to the flow and size.

Figure 6-6 – Prioritization Criteria



Figures summarizing the prioritization scoring are included in **Appendix Q**. The City can easily modify the individual criteria weighting or LoF/CoF weighting to further refine and customize the prioritization over time. This can be done using simple ArcMap field calculations without expending significant time.

6.7.2 Developing Pipe Condition Ratings

The City has not been actively performing CCTV assessment of existing pipes and does not have reliable information in regards to current condition. This results in a significant gap in data as the condition rating should be the backbone of a renewal/replacement program. Therefore, the City is budgeting for a large-scale effort to acquire this vital condition information over a three year period. The current pipe scoring and prioritization list that resulted from the effort in this plan provides a general guideline and target to identify those portions of the system that the City should immediately focus on gathering condition data for. Results are shown on **Figure Q2.**

Because most PVC pipe has been recently constructed, the City should focus on the assessment of all non-PVC pipe in the collection system. The City GIS data indicates that there is approximately 725,000 LF of non-PVC pipe in the system. A planning level cost for CCTV and pipe condition rating is approximately $2/LF. Therefore, this effort is expected to cost approximately $1.5 million. A typical rate of assessment is 2,000 LF per day. The City is planning on implementing this project over three years; therefore, approximately $500,000 of the renewals/replacement budget will be dedicated to CCTV and pipe condition rating. The existing prioritization scoring should be utilized to identify which areas of the City should immediately be targeted when developing the contracts for the CCTV and pipe condition rating effort.

Another benefit of the pipe condition scoring effort will be a database listing the types of defects that are found in the various pipes of the collection system. This data will then be valuable in determining the likely construction methods of rehabilitation. A preliminary construction method can be determined by evaluating the type of defects associated with each pipe segment. The City utilizes CUES Granite XP protocols during the CCTV process to code pipe defects. By joining the City's current CUES data table (CCTV defects table) and the City's current sewer pipe dataset in GIS and using tools within ArcMap, a preliminary construction method can be determined for each pipe segment. The City should ensure that this pipe defect data is included in the deliverable for the CCTV and pipe condition rating effort.

6.7.3 Updating the Prioritization

Once the pipe condition rating data has been gathered by the City, it can easily be incorporate into the overall pipe scoring system. Once completed, the backbone of the sewer rehabilitation and replacement program will be the City's existing sewer GIS data and CCTV results (CUES condition rating). This prioritization tool utilizes existing City datasets and the power of GIS to visualize LoF (condition rating) and CoF (risk rating) over the entire City. This tool will allow the City to make informed engineering decisions about each replacement or rehabilitation project.

Each pipe segment can be given an updated LoF score (condition) by joining the City's current CUES data table (CCTV results) to the City's current sewer pipe dataset in GIS and running a simple field calculation. This can be done as often as the City desires, whether it is yearly, quarterly, or more frequently. Additional LoF (condition) criterion that the City may wish to update regularly could include Time Since Last CCTV Inspection and O & M Frequency.

The CoF scores (risk) requires less frequent updates, because the criteria used in this category are less dynamic than the LoF (condition) criteria. Most of the data is derived from the existing sewer model, which is only updated and recalibrated approximately every five years. The 'Location' criteria are also nearly static with updates every five years being adequate.

Appendix Q details procedures to update all of the LoF and CoF criteria. We recommend that the LoF scores (condition) and overall prioritization scores be updated on a yearly basis, prior to establishing the CIP for the coming year. We recommend the CoF (risk) be updated when the existing sewer model is recalibrated, approximately every five years. As the sewer rehabilitation and replacement program is refined in coming years, the City can easily update the LoF, CoF, and overall prioritization scores more or less frequently as required.

6.7.4 Collection System Replacement Analysis

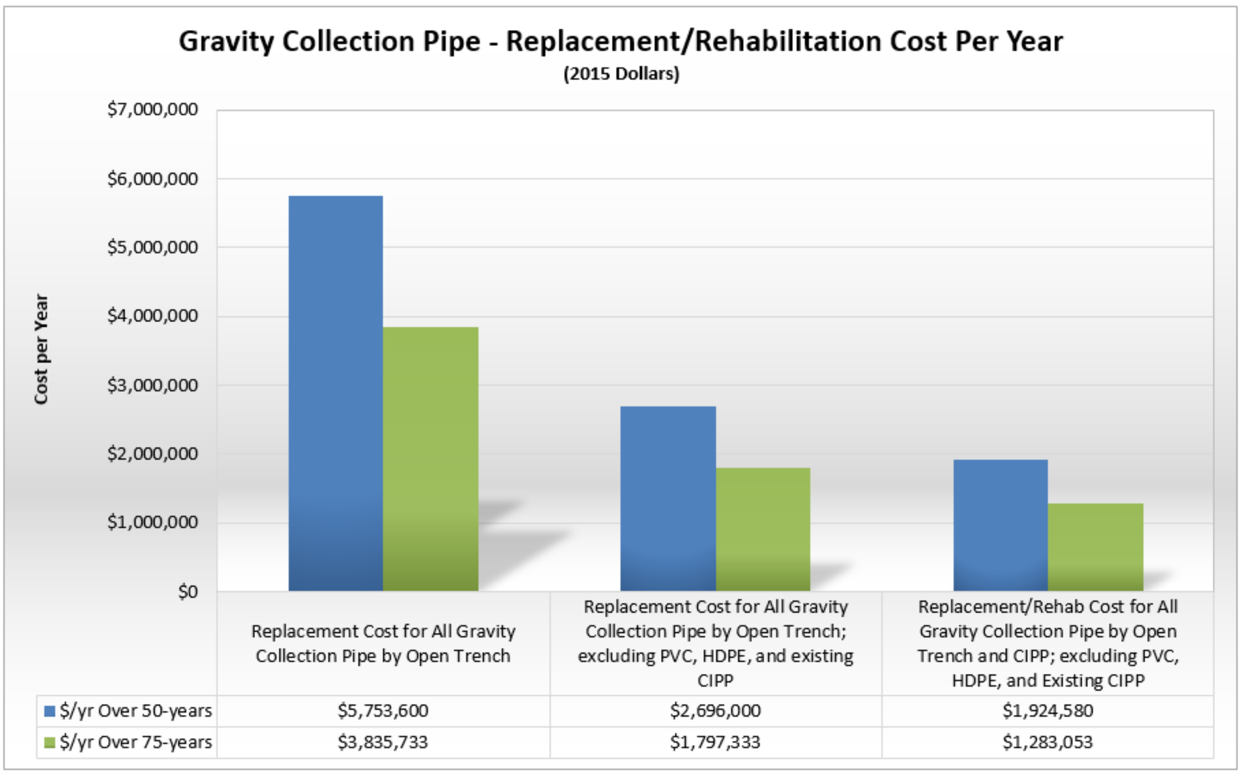
To determine a range that should be targeted for annual budgeting for collection system renewal/replacement, a total system replacement cost was calculated. The cost of total system replacement for the entire existing gravity sewer collection system via trench and replacement construction methods in the City is estimated to be approximately $288 million. This number assumes sewer collection mains only and not the service connections to each parcel.

Sewer collection system pipes have an expected lifetime that ranges from 50 to 75 years. Therefore, assuming that the collection system would need to be replaced every 50 years, beginning today the annual cost for system replacement in 2015 dollars is approximately $5.8 million per year. Assuming replacement every 75 years, the cost is approximately $3.8 million per year.

PVC pipe is expected to have a lifetime of approximately 100 years – and the oldest PVC pipe in the ground today is approximately 25 years old. Therefore, it is reasonable to assume that none of the existing PVC pipe will need to be replaced in the next 75 years. Similarly, it is reasonable to assume that all pipe that has been rehabilitated with CIPP methods will not require additional rehabilitation in the next 75 years – the City has been conducting CIPP rehabilitation on approximately 130,000 LF of pipe since1997. If we exclude all PVC pipe and CIPP pipe from the replacement cost analysis, the total collection system replacement cost decreases to approximately $135 million – assuming trench and replacement construction methods. Assuming that this non-PVC and non-CIPP portion collection system would need to be replaced every 50 years, the annual cost for system replacement in 2015 dollars is approximately $2.7 million per year. Assuming replacement every 75 years, the cost is approximately $1.8 million per year.

To take the analysis one step further, it is reasonable to assume that of the non-PVC pipe and non-CIPP pipe to be replaced, rehabilitation costs will decrease if the pipe can be rehabilitated with CIPP lining versus traditional trench and replace methods. Assuming that approximately half of this non-PVC pipe can be rehabilitated with CIPP while the other half is replaced via trenching, the total system replacement cost in 2015 dollars is approximately $96 million. Implementing the rehabilitation/replacement plan over 50 years is a cost of approximately $1.9 million per year. Assuming rehabilitation/replacement every 75 years, the cost is approximately $1.3 million per year.

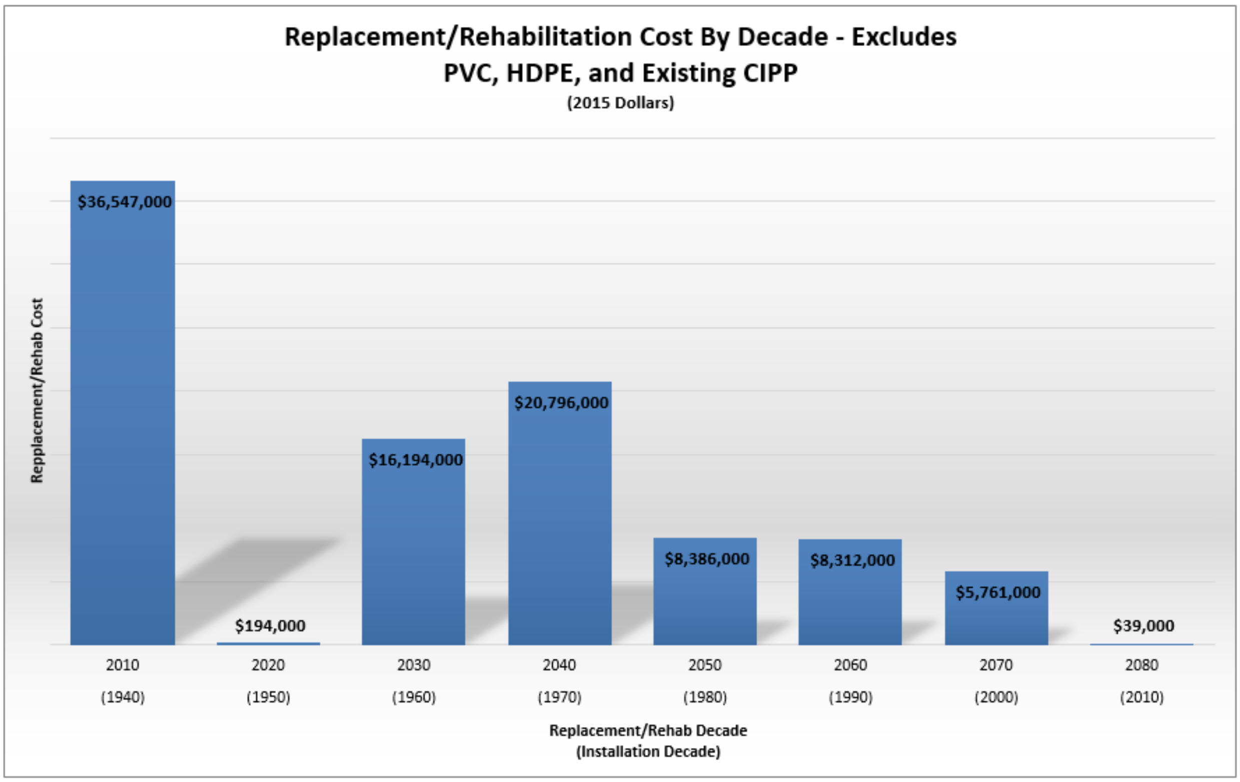
Figure 6-7 – System Replacement Costs



Based upon this analysis, the City should be budgeting somewhere between $1.3 million and $1.9 million (2015 dollars) annually for renewal and replacement of the sewer collection system. In the CIP presented in **Chapter 7**, $1.5 million dollars is budgeted annually beginning in 2017. **Chapter 8** addresses the financial impacts of the renewal/replacement program

It is worth noting that the above analysis does not take into account the age of the existing pipes. The City has limited data on pipe age; however, an estimate of pipe installation by the decade was developed in order to identify the potential timing of replacement. Figure 6-8 depicts potential cost of replacement per decade for the next several decades. This assumes a 75-year lifespan for the non-PVC pipe that has not yet been rehabilitated. Because a significant portion of the City was constructed in the 1940s, replacement of a large portion of the City is likely required soon. This emphasizes the need for CCTV inspection and condition rating of the system in order to verify if the pipes are near the end of their service life. As the City updates the GIS records on pipe installation years, this analysis can be further refined.

Figure 6-8 – Potential Timing of System Replacement Costs



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One other noteworthy item is in regards to service connections. The numbers above are based upon rehabilitation of sewer main pipes only and does not account for rehabilitation of the sewer service connections. As noted above, assuming non-PVC and non-CIPP pipes are replaced with a combination of open-trench and CIPP, this is approximately $96 million in 2015 dollars. If the City decided to also rehabilitate the approximately 9,000 sewer service laterals associated with these projects, it is estimated to add another $23 million to the project cost – or an increase of about 25%.

6.8 Lift Stations

This section discusses the existing condition of the 14 lift stations that the City maintains and includes the existing pump capacities. In addition, this section identifies any needs and upgrades for each lift station.

6.8.1 Description of Existing Facilities

The City currently maintains 14 lift stations. The existing lift stations are generally classified as belonging to one of two different categories, local service or interceptor service. In general, the interceptor lift stations receive flows from large service areas and their operation is important to the overall performance of the collection system. If an interceptor service lift station were to fail, it could have significant impacts as measured by the area affected by flooding. The Montana St lift station serves all of the RY sewer basin and is necessary to convey flows west of SR 240. All other City lift stations serve smaller drainage basins and are therefore classified as local service. For this reason, special attention and prioritization is given to the Montana St station. The City's 14 lift stations are shown in **Figure A1** and are listed in **Table 6-10**.

Table 6-10 – Existing Lift Stations

Lift Station Name
Battelle
Waterfront
Terminal Drive
Mental Health
Bradley
Columbia Pt
Wellhouse Loop
Duportail
Montana St
Columbia Park Trail
Meadows South
Bellerive
Meadow Ridge
Dallas Rd

The following is a general description of each of the lift station facilities.

Battelle Lift Station

The Battelle Lift Station is located in and serves an industrial area in the upper North area of Richland off Battelle Blvd. The station was originally constructed in 1995 as a duplex submersible pump style lift station with 25 hp pumps to serve a drainage basin of approximately 3,400 AC. In 2013, as a part of the Logston Sewer Interceptor project, the drainage basin was significantly decreased in size and routed to the new interceptor. The Battelle Lift Station drainage basin is now approximately 330 AC in size, serving mainly the area east of the lift station and fronting Battelle Blvd. In 2013 the original submersible pumps were replaced with smaller 5 hp Flygt pumps to convey the lower flows through a new, and much shorter, forcemain to the nearby interceptor pipe. The wet well is a 12-ft diameter precast manhole with a depth of that is approximately 18 feet. A 6-inch forcemain (130 LF) is used to transport the pumped sewage to the nearby gravity system discharge manhole located near the intersection of Battelle Blvd and Logston Blvd. The station is reported to have a capacity of 400 gpm (0.58 mgd) with one pump in operation.

Fencing is provided around the station and an intrusion alarm is also provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system.

Waterfront Lift Station

The Waterfront Lift Station is located north of the Penford processing facility in upper North Richland, adjacent to the Columbia River. It serves the processing facility and a small area of residential development, north of the lift station and east of Richardson Rd. The station was constructed in 1977 and has a typical wet-pit/dry-pit configuration. The dry well has a 3.5-ft diameter entrance tube and a 7-ft diameter factory built station that rests on a common foundation slab with the wet well. The wet well is a 6-ft diameter, 19 foot deep structure. Two 15 hp vertical non-clog Fairbanks Morse sewage type pumps are used in a duplex configuration to remove the sewage from the wet well. A 6-inch forcemain (260 LF) is used to transport the pumped sewage to the gravity system discharge manhole located near the intersection of Richardson Rd and Lindberg St. The station is reported to have a capacity of 600 gpm (0.86 mgd) with one pump in operation.

Fencing is provided around the station and an intrusion alarm is also provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system. The station is also equipped with a small ventilation fan, heater, and dehumidifier in the dry well. A sump pump is provided and the station does have an alarm in case a leak occurs inside the pump chamber.

No station improvements since 1977, although it is scheduled for replacement by a City standard, submersible style lift station in 2017.

Terminal Drive Lift Station

The Terminal Drive Lift Station is located near the south end of the Richland Airport. It serves the Columbia Basin Racquet Club facility and a small area of commercial parcels along Terminal Dr. The station was constructed in 1981 and has a typical wet-pit/dry-pit configuration. The dry well has a 3.5-ft diameter entrance tube and a 7-ft diameter factory built station that rests on a common foundation slab with the wet well. The wet well is an 8-ft diameter, 15 foot deep structure. Two 3 hp vertical non-clog Fairbanks Morse sewage type pumps are used in a duplex configuration to remove the sewage from the wet well. A 4-inch forcemain (390 LF) is used to transport the pumped sewage to the gravity system discharge manhole located Northeast of the lift station. The station is reported to have a capacity of 150 gpm (0.22 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system. The station is also equipped with a small ventilation fan, heater, and dehumidifier in the dry well. A sump pump is provided and the station does have an alarm in case a leak occurs inside the pump chamber.

Mental Health Lift Station

The Mental Health Lift Station is located on Stevens Drive, just north of the Kadlec Hospital campus. The lift station was originally constructed in 1973 as a submersible pump style lift station. In 2009 the lift station pumps were replaced with two 5 hp Vaughan chopper style pumps to address the constant ragging issues the City was experiencing in the flows to the lift station. The wet well is a 6-ft diameter precast manhole that has a depth of approximately 15 feet. A 4-inch forcemain (195 LF) is used to transport the pumped sewage to the gravity system discharge manhole located in Stevens Drive. The station is reported to have a capacity of 260 pm (0.37 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is provided. The alarms are transmitted to the wastewater treatment plant by the telemetry system.

Bradley Lift Station

The Bradley Lift Station is located east of George Washington Way, at the intersection of Comstock St and Bradley Blvd. It serves the commercial area along Bradley Blvd, mainly north of the Columbia Point Golf Course. The lift station was constructed in 1999 as a submersible pump style lift station. The wet well is a 10-ft diameter precast manhole that is approximately 24 feet deep. Two 10 hp Flygt submersible pumps are used in a duplex configuration to remove the sewage from the wet well. A 6-inch forcemain (1,774 LF) is used to transport the pumped sewage to the intersection of George Washington Way and Columbia Point Drive where it combines with the 6-inch forcemain from the Columbia Point Lift Station. The combined forcemain is 8-inch in size and discharges into the gravity system discharge manhole located near the intersection of Aaron Drive and Abbot Street. The station is reported to have a capacity of 180 gpm (0.26 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is present. The alarms are transmitted to the wastewater treatment plant by the telemetry system.

Columbia Point Lift Station

The Columbia Point Lift Station is located east of George Washington Way on Columbia Point Drive. It serves the commercial and multi-family area along Columbia Point Drive mainly north and east of the Columbia Point Golf Course. The lift station was constructed in 1999 as a submersible pump style lift station. The wet well is a 10-ft diameter precast manhole that is approximately 20 feet deep. Two 6.5 hp Flygt submersible pumps are used in a duplex configuration to remove the sewage from the wet well. A 6-inch forcemain (1,800 LF) is used to transport the pumped sewage to the intersection of George Washington Way and Columbia Point Drive where it combines with the 6-inch forcemain from the Bradley Lift Station. The combined forcemain is 8-inch in size and discharges into the gravity system discharge manhole located near the intersection of Aaron Drive and Abbot Street. The station is reported to have a capacity of 270 gpm (0.39 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is present. The alarms are transmitted to the wastewater treatment plant by the telemetry system.

Wellhouse Loop Lift Station

The Wellhouse Loop Lift Station No. is located at the intersection of Wellhouse Loop and Wyman St. It serves a mainly commercial area fronting Wellhouse Loop. The station was constructed in 1978 and has a typical wet-pit/dry-pit configuration. The dry well has a 3.5-ft diameter entrance tube and a 7-ft diameter factory built station that rests on a common foundation slab with the wet well. The wet well is a 6-ft diameter, 17 foot deep structure. Two 1.5 hp vertical non-clog Hydronix sewage type pumps are used in a duplex configuration to remove the sewage from the wet well. A 4-inch forcemain (50 LF) is used to transport the pumped sewage to the gravity system discharge manhole located at the same intersection. The station is reported to have a capacity of 100 gpm (0.14 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system. The station is also equipped with a small ventilation fan, heater, and dehumidifier in the dry well. A sump pump is provided and the station does have an alarm in case a leak occurs inside the pump chamber.

Duportail Lift Station

The Duportail Lift Station is located at the end of the cul-de-sac on Duportail Street, north of the Yakima River. It serves both the single family and the multi-family developments west of the By-Pass Highway (SR 240) and north of the River. The lift station was constructed in 1995 as a submersible pump style lift station. The wet well is a 6-ft diameter precast manhole that is approximately 23 feet deep. Two 7.5 hp Flygt submersible pumps are used in a duplex configuration to remove the sewage from the wet well. A 6-inch forcemain (860 LF) is used to transport the pumped sewage to the gravity system discharge manhole located near the intersection of Duportail Street and the By-Pass Highway. The station is reported to have a capacity of 200 gpm (0.29 mgd) with one pump in operation.

Fencing is provided around the station and an intrusion alarm is also provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system.

Montana St Lift Station

The Montana Lift Station is located at the end of Montana Street, north of Columbia Park Trail in the Richland Wye area. It serves all of the RY drainage basin which consists of all the area north and east of SR 240 and south of the Yakima River. The station was constructed in 1968 and has a typical wet-pit/dry-pit configuration. The dry well has a 3-ft diameter entrance tube and an 8-ft diameter factory built station that rests on a common foundation slab with the wet well. The wet well is a 6-ft diameter, 23 foot deep structure. Two 30 hp vertical non-clog Smith & Loveless sewage type pumps are used in a duplex configuration to remove the sewage from the wet well. The forcemain consists of both 8-inch AC (1,850 LF) and 10-inch PVC (866 LF) pipe and conveys sewage flows to the gravity system discharge manhole located west of the roundabout at Columbia Park Trail and N Steptoe Street. The station is reported to have a capacity of 970 gpm (1.40 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system. The station is also equipped with a small ventilation fan, heater, and dehumidifier in the dry well. A sump pump is provided and the station does have an alarm in case a leak occurs inside the pump chamber.

Columbia Park Trail Lift Station

The Columbia Park Trail Lift Station is located on Columbia Park Trail, adjacent to the Columbia River. It serves the Hanford Reach Interpretive Center and the Fowler and Tapteal Lift Station drainage basins (both these older style lift stations were abandoned). The lift station was constructed in 2012 as a submersible pump style lift station. The wet well is an 8-ft diameter precast manhole that is approximately 13 feet deep. Two 10 hp Flygt submersible pumps are used in a duplex configuration to remove the sewage from the wet well. A 6-inch forcemain (1,320 LF) is used to transport the pumped sewage to the gravity system discharge manhole located near the intersection of Columbia Center Blvd and Columbia Park Trail. The station is reported to have a capacity of 400 gpm (0.58 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system.

Meadows South Lift Station

The Meadows South Lift Station is located at the end of Meadows Drive South and adjacent to the Meadow Springs Golf Course. It serves an area of single family and multi-family development off Meadows Drive, generally west of Bellerive Drive. The station was constructed in the 1970's and has a typical wet-pit/dry-pit configuration. No record drawings were available, although the City clarified that the wet well is a 7-ft diameter, 21 foot deep structure. Two 3 hp vertical non-clog Hydromatic sewage type pumps are used in a duplex configuration to remove the sewage from the wet well. A 4-inch forcemain (90 LF) is used to transport the pumped sewage to the gravity system discharge manhole located in Blalock Court, north of the lift station. The station is reported to have a capacity of 100 gpm (0.14 mgd) with one pump in operation.

Fencing is provided around the station and an intrusion alarm is also provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system. The station is also equipped with a small ventilation fan, heater, and dehumidifier in the dry well. A sump pump is provided and the station does have an alarm in case a leak occurs inside the pump chamber.

Bellerive Lift Station

The Bellerive Lift Station is located on Bellerive Ct, east of Claybell Park in South Richland. It serves single family residential development along Broadmoor St (to the west) and the Heights at Meadow Sprints development (to the east). The lift station was constructed in 2005 as a submersible pump style lift station. The wet well is a 6-ft diameter precast manhole that is approximately 28 feet deep. Two 15 hp Flygt submersible pumps are used in a duplex configuration to remove the sewage from the wet well. A 6-inch forcemain (1,980 LF) is used to transport the pumped sewage to the gravity system discharge manhole located north of Gage Blvd, on Bellerive Dr. The station is reported to have a capacity of 260 gpm (0.37 mgd) with one pump in operation.

In 2011 the Leslie Sewer Trunk was constructed to the south and connected to the lift station. The 18-inch trunk pipe was designed to collect all flows south of the lift station, including the east half of the Badger Mountain South planned development. As discussed in the SRSR (see Appendix B) the increased sewer flows to the lift station will exceed the current capacity. The recommended improvements included extending a 21-inch pipe to the north, across the Meadow Springs Golf Course, and connecting to an existing 21-inch trunk pipe on Leslie Road, near the north edge of the Canyon Terrace subdivision. At the lift station there is a 13-ft vertical difference between the Leslie Sewer Trunk influent pipe invert and the local residential collector influent pipe invert and therefore, as part of the improvements, the City may choose to leave the lift station in place, to serve the local collector, in lieu of deepening the trunk pipe. Shallow groundwater was encountered during the lift station construction.

No fencing is provided around the station, but an intrusion alarm is present. The alarms are transmitted to the wastewater treatment plant by the telemetry system.

Meadow Ridge Lift Station

The Meadow Ridge Lift Station is located on Steptoe St, just south of the intersection with Gage Blvd. It serves commercial development south of Gage, between Bellerive Dr and Steptoe St. The lift station was constructed in 2007 as a submersible pump style lift station. The wet well is a 6-ft diameter precast manhole that is approximately 13 feet deep. Two 10 hp Flygt submersible pumps are used in a duplex configuration to remove the sewage from the wet well. A 6-inch forcemain (1,080 LF) is used to transport the pumped sewage to the gravity system discharge manhole located west of the lift station. The station is reported to have a capacity of 245 gpm (0.35 mgd) with one pump in operation.

No fencing is provided around the station, but an intrusion alarm is present. The alarms are transmitted to the wastewater treatment plant by the telemetry system.

Dallas Rd Lift Station

The Dallas Road Lift Station is located on Dallas Road, north of the I-82, Dallas Rd exit. It serves the west half of the mixed use development of Badger Mountain South. The lift station was constructed in 2012 as a submersible pump style lift station. The wet well is an 8-ft diameter precast manhole that is approximately 25 feet deep. Two 35 hp Flygt submersible pumps are used in a duplex configuration to remove the sewage from the wet well. The lift station has dual forcemain pipes (9,510 LF of 6-inch and 12-inch) to accommodate increased sewer flows to the station. Currently only the 6-inch forcemain pipe is in use and transports the pumped sewage to the gravity system discharge manhole located generally southeast of the I-182/Dallas Rd undercrossing. The station is reported to have a capacity of 260 gpm (0.37 mgd) with one pump in operation.

Record drawings identify a future bypass of this lift station, to a second lift station that is not currently constructed. No phasing information was available at this time.

Fencing is provided around the station and an intrusion alarm is also provided. The alarms are transmitted to the wastewater treatment plant by the radio telemetry system.

6.8.2 Proposed Improvements and Future Replacement Schedule

The City has been proactively upgrading the lift stations by replacing pumps and piping and electrical control systems. The following is a list that identifies the scheduled improvements for the lift station upgrades:

2014 – Montana Lift Station Standby Generator:
Installation of an on-site generator to operate the lift station during power outages.

2015 – Columbia Park Trail Lift Station Standby Generator:
Installation of an on-site generator to operate the lift station during power outages.

2017 – Waterfront Lift Station Replacement:
Replace this deficient wet-pit/dry-pit station with a submersible pump style lift station.

Lift stations must also be rehabilitated and replaced as necessary. Mechanical rehabilitation is often required every 15 to 30 years, while electrical upgrades are often required every 15 to 20 years. A major rehabilitation or replacement should be expected every 50 years. As shown in **Table 6-11,** several lift stations will need mechanical and/or electrical upgrades within the next 10 years. None of the lift stations are expected to undergo a major rehabilitation or replacement within the next 10 years, other than those identified in the previous section. For budgetary purposes, the following costs are assumed: a mechanical upgrade is $30,000 to $80,000 and an electrical upgrade is $25,000 to $55,000, depending on the lift station size (2015 dollars).

Table 6-11 – Lift Station Rehabilitation/Replacement Expectations

Lift Station Name	Year Constructed/ Last Major Rehabilitation	Comments	Rehabilitation Expected In…		
			Mechanical (15 to 30 years)	Electrical (15 to 20 years)	Major Rehabilitation/ Replacement (50 years ±)
Battelle	2013		20 - 30 years	20 - 30 years	40 - 50 years
Waterfront[1]	1977		0 - 5 years	0 - 5 years	5 - 15 years
Terminal Drive	1981		0 - 5 years	0 - 5 years	15 - 20 years
Mental Health	2009		10 - 15 years	10 - 15 years	40 - 50 years
Bradley	1999		5 - 10 years	5 - 10 years	30 - 40 years
Columbia Pt	1999		5 - 10 years	5 - 10 years	30 - 40 years
Wellhouse Loop	1978		0 - 5 years	0 - 5 years	5 - 15 years
Duportail[2]	1995		0 - 5 years	0 - 5 years	25 - 35 years
Montana St	1968		0 - 5 years	0 - 5 years	5 - 10 years
Columbia Park Trail	2012		20 - 30 years	20 - 30 years	40 - 50 years
Meadows South	1970's		0 - 5 years	0 - 5 years	5 - 10 years
Bellerive	2005		10 - 15 years	10 - 15 years	40 - 50 years
Meadow Ridge	2007		10 - 15 years	10 - 15 years	40 - 50 years
Dallas Rd	2012		20 - 30 years	20 - 30 years	40 - 50 years

(1) *Waterfront Lift Station is scheduled for a complete station replacement in 2017.*

(2) *Duportail Lift Station is scheduled for relocation and replacement as a part of the Duportail Bridge project. No schedule at this time.*

Chapter 7

Capital Improvement Plan

Chapter 7 – Capital Improvement Plan

7.1 CIP Overview

The Capital Improvement Plan (CIP) prioritizes the improvements that are necessary in the near term to relieve capacity issues, replace deteriorated segments of the collection system, and implement improvements that will be needed as infill occurs in the City and as the wastewater service area is expanded to the future boundary. The CIP is organized into the following categories:

- Capacity Projects – Required to address insufficient hydraulic capacity of existing pipes in the near future.
- System Expansion – Required to serve new areas within the UGA.
- Collection System Improvements – Required to address components of the collection system needing an upgrade.
- Rehabilitation/Replacement – Required to maintain the integrity of the existing system.
- WWTP Improvements – As identified by the Plant Staff.
- WWTP Rehabilitation/Replacement – As identified by Plant Staff.
- Developer Driven Growth Projects – To serve growth both inside and outside the UGA.

Figure A14 shows the location and type of each project in the CIP. **Appendix I** contains detailed opinions of probable cost and a CIP summary/figure for each project. All capital costs are in 2015 dollars. The opinions of probable cost are for budgetary purposes only and further refinement of the cost opinions will be required during subsequent preliminary engineering and design phases for each CIP project.

The timeframe for implementing CIP projects not related to rehabilitation/replacement will ultimately depend on realized growth and non-residential development. The timeframes for the CIP projects shown on **Table 7-1** are based on review with City staff.

7.2 CIP Projects

Table 7-1 includes a summary of all identified projects in the CIP capital cost and recommended timeframe for completion.

Table 7-1 – CIP Projects

ID	Description/System Name	Recommend Action	Timeframe and Capital Cost										
			2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	With Growth[1]
Capacity Projects – Funded by Connection Fees													
CP.1	Leslie Rd Trunk Replacement	Replace 18-inch bottleneck section											$329,000
CP.2	Keene Rd Collector Replacement	Replace 10-inch bottleneck section							$329,000				
CP.3	Upper North Interceptor Improvements	New lift station and piping to address neighborhood surcharging										$2,238,000	
CP.4	Bellerive LS Pump Upgrade & Downstream Improvements	New lift station pumps and downstream pipe replacement to address surcharging										$1,785,000	
System Expansion – Funded by Connection Fees													
SE.1	Leslie Interceptor Extension	Collection system expansion to extend utility service	$800,000										
Collection System Improvements – Funded by a split of Connection Fees and Rates													
CS.1	Montana Lift Station Standby Generator	Generator installation to operate lift station during power outages	$40,000										
CS.2	Columbia Lift Station Standby Generator	Generator installation to operate lift station during power outages	$25,000										
CS.3	Waterfront Lift Station Replacement	Replace deficient lift station			$608,000								
Rehabilitation and Replacement Projects – Funded by Rates													
RR.1	Renewals and Replacement	10-yr rehabilitation and replacement program based on Condition Assessment	$250,000	$258,000	$1,599,000[2]	$1,652,000[2]	$1,705,000[2]	$1,761,000	$1,818,000	$1,878,000	$1,939,000	$2,002,000	

ID	Description/System Name	Recommend Action	Timeframe and Capital Cost										
			2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	With Growth[1]
RR.2	Annual Street Overlay Areas	Annual repair and replacement of sewer deficiencies in areas scheduled for re-paving	$100,000	$103,000	$107,000	$110,000	$114,000	$117,000	$121,000	$125,000	$129,000	$133,000	
RR.3	Infiltration and Inflow Study								$200,000				
WWTP Improvements – Funded by Rates/Connection Fees													
WWTP. 1	Influent Upgrades	Influent Upgrades			$2,133,000								
WWTP. 2	Engineering Report	Re-Rating Study for Design Criteria						$411,000					
WWTP Rehabilitation and Replacement – Funded by Rates													
WWTP. RR.1	WWTP Renewals and Replacements	General rehabilitation and replacement				$551,000	$568,000	$587,000	$606,000	$626,000	$646,000	$667,000	
WWTP. RR.2	Plant Wide HVAC Improvements	System improvements to current HVAC equipment	$290,000										
WWTP. RR.3	Digester Building MCC	Replace obsolete and failing motor control center hardware	$80,000										
WWTP. RR.4	Primary Clarifier #2 Coating	Recoat primary clarifier #2 to protect from corrosion		$165,000									
WWTP. RR.5	Digester #1 Tank Coating	Recoat digester #1 tank		$330,000									
WWTP. RR.6	Secondary Clarifier #2 Coating	Recoat secondary clarifier #2 to protect from corrosion		$227,000									
WWTP. RR.7	Clarifier Gear Drive Replacements	Replace obsolete and failing gear drive on the clarifier			$325,000								
WWTP. RR.8	Plant Pump and Piping Replacement	Annual pump and piping maintenance			$80,000								
Annual Capital Improvement Plan Total													
Yearly Totals			$1,585,000	$1,083,000	$4,852,000	$2,313,000	$2,387,000	$2,876,000	$3,074,000	$2,629,000	$2,714,000	$6,825,000	

(1) All capital costs are in 2015 dollars.

(2) $500,000 will be allocated to CCTV and Pipe Condition Rating

Table 7-2 – Developer Driven Growth Projects

ID	Description/System Name	Recommend Action	Timeframe and Capital Cost										
			2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	With Growth[1]
Developer Driven Growth Projects – Projects to serve growth both inside and outside the UGA													
DD.1	Country Ridge Downstream Improvements	Upgrade downstream pipe to provide for future lift station upgrades and additional pumping capacity											$4,070,000
DD.2	East Badger South Lift Station	Construction required for development within the East Badger South Basin – SRSR CIP #1 (AHBL est.)											$5,500,000
DD.3	West Badger South Lift Station	Construction required for build-out of West Badger South and East Badger South											$3,180,000
DD.4	Horn Rapids Interceptor Extension	From Kingsgate Sports Complex to Village Pkwy/Construction as required with growth											$450,000
DD.5	SR 240 Interceptor	From Village Pkwy to Horn Rapids Rd/Construction as required with growth											$3,214,000
DD.6	600 Area (South) Interceptor	From Battelle Blvd to Horn Rapids Rd & North/Construction as required with growth											$3,467,000
Developer Driven Growth Project Total													
													$19,881,000

[1] *All capital costs are in 2015 dollars.*

7.3 Budgeting CIP Projects

The costs associated with each CIP project were grouped by time and are summarized in **Table 7-3**. Refer to **Appendix I** for a detailed breakdown of each project. The timeframes listed are intended to begin in Calendar Year 2016. The additional CIP costs identified herein for lift station replacement/rehabilitation should be reviewed and integrated as budget permits. If this work is not completed in the designated year timeframe, the work should be carried forward into the following year timeframe and the budgets revised accordingly.

Table 7-3 – CIP Cost Summary

CIP Project Timeframe	10-YR Capital Cost
0 – 6 Years	$15,096,000
6 – 10 Years	$15,242,000
As Needed with Growth(1)	$20,210,000
Totals	**$50,548,000**

Chapter 8

Financial Plan

Chapter 8 – Financial Plan

8.1 Introduction

This chapter was prepared by FCS GROUP to provide a financial program that allows the sewer utility to remain financially viable during the planning period. This financial viability analysis considers the historical financial condition, current and recommended financial and policy obligations, operation and maintenance needs, and the financial impact of completing the capital projects identified in this General Sewer Plan (GSP) Update. Furthermore, this chapter provides a review of the utility's current rate structure with respect to rate adequacy and customer affordability.

8.2 Past Financial Performance

This section includes an historical summary of financial performance as reported by the City on the fund resources and uses arising from cash transactions, as well as an historical summary of comparative statements of net position.

8.2.1 Comparative Financial Statements

Financial operations of the sewer utility are managed within Fund 403, the Wastewater Utility Fund. **Table 8-1** shows a summary of fund resources and uses arising from cash transactions for the previous 6 years (2008 through 2013). **Table 8-2** shows a summary of assets and liabilities, with the difference between the two reported as "net position". Increases or decreases in net position are useful indicators of the financial position of the City's utility fund. Noteworthy findings and trends are discussed to demonstrate the historical performance and condition of the City's utility fund.

Table 8-1 – Summary of Historical Fund Resources and Uses Arising from Cash Transactions

	2008	2009	2010	2011	2012	2013
OPERATING REVENUES						
Charges for services:						
Sewer	$ 7,481,709	$ 7,929,743	$ 8,333,342	$ 8,582,408	$ 8,574,149	$ 8,777,356
Other operating revenues	-	-	182,445	-	-	-
Total operating revenues	7,481,709	7,929,743	8,515,787	8,582,408	8,574,149	8,777,356
OPERATING EXPENSES						
Maintenance and operations	3,021,477	3,260,519	3,378,611	4,051,372	3,509,728	3,596,509
Administrative and general	1,123,676	1,256,281	1,229,288	1,206,980	1,233,823	1,275,978
Taxes	875,640	922,798	1,024,946	1,032,075	1,027,269	1,043,607
Depreciation	1,305,060	1,369,951	1,373,424	1,431,552	1,606,031	1,605,507
Total operating expenses	6,325,853	6,809,549	7,006,269	7,721,979	7,376,851	7,521,601

Operating income (loss)	1,155,856	1,120,194	1,509,518	860,429	1,197,298	1,255,755
NONOPERATING REVENUES/(EXPENSES)						
Investment earnings	241,703	49,168	35,906	241,004	233,860	(36,231)
	2008	**2009**	**2010**	**2011**	**2012**	**2013**
Interest expense	(896,096)	(841,406)	(815,001)	(778,249)	(778,844)	(739,289)
Other interest earnings	29,411	7,220	-	31,814	5,089	685
Debt costs	(208,664)	(190,513)	(193,920)	(193,920)	(188,291)	(31,131)
Misc. nonoperating rev/(exp)	16,404	53,587	1,313,155	321,850	3,415	(86,325)
Total nonoperating rev (exp)	(817,242)	(921,944)	340,140	(377,501)	(724,771)	(892,291)
Net income before contributions and transfers	338,614	198,250	1,849,658	482,928	472,527	363,464
Capital contributions	330,484	479,859	1,095,437	954,386	1,885,014	1,351,619
Transfers in	-	10,629	-	300,000	25,597	-
Transfers out	-	(52,722)	-	(8,073)	(55,295)	(101,000)
Change in net position	669,098	636,016	2,945,095	1,729,241	2,327,843	1,614,083
Net position – beginning	35,981,807	36,629,518	37,014,375	41,743,937	43,498,602	45,723,919
Prior period adjustments	(21,387)	(251,159)	1,784,467	25,424	(21,297)	42,862
Net position – ending	$36,629,518	$37,014,375	$41,743,937	$43,498,602	$45,805,148	$47,380,864
O&M Coverage Ratio	118.3%	116.5%	121.5%	111.1%	116.2%	116.7%
Net Operating Income as % of Operating Revenue	15.4%	14.1%	17.7%	10.0%	14.0%	14.3%
Debt Service Coverage Ratio	2.75	2.34	2.62	2.01	2.36	2.34

8.2.2 Findings and Trends

- The City's sewer sales increased by 11.4 percent from 2008 to 2011, and an additional 2.3 percent from 2011 to 2013. The lower increases in later years were likely due to the depressed economy. Total expenses increased each year through 2011; in 2012, lower maintenance and operations expenses assisted with net operating income increasing again.
- The O&M Coverage Ratio (total operating revenue divided by total operating expenses) began 2008 at 118.3 percent, declined to 111.1 percent in 2011 and ended 2013 at 116.7 percent. A ratio of 100 percent or greater shows that revenue will successfully cover expenses and the City has remained above this for the past six years.
- Net Operating Income as a percent of Operating Revenue in 2008 was 15.4 percent, increasing to a high of 17.7 percent in 2010, then lowering to 14.3 percent in 2013. Similar to the O&M Coverage Ratio, these trends help to show how successfully operating revenue actually covered operating expenses, with higher positive numbers being the best and negative numbers showing need for improvement.

- The Debt Service Coverage Ratio is required by bond covenants to remain above 1.25 during the life of the loans. This ratio is calculated by dividing cash operating income (revenue less expenses before depreciation) by annual revenue bond expenses. This ratio remains above the target, beginning 2008 at a high of 2.75, decreasing to 2.01 in 2011 and climbing again to 2.34 in 2013.

Table 8-2 – Summary of Historical Comparative Statement of Net Position

	2008	2009	2010	2011	2012	2013
ASSETS						
Current:						
Cash and cash equivalents	$1,678,177	$1,470,480	$2,462,350	$ 616,151	$ 340,373	$ 326,778
Deposits with third parties	-	2,650	2,650	2,650	2,650	2,650
Investments	730,550	1,663,517	454,738	1,973,661	4,119,215	3,872,216
Receivables:						
Customer accounts (net)	449,863	436,378	442,395	481,943	497,462	638,253
Due from other funds	-	2,790	-	-	-	-
Due from other governments	-	101,163	493,100	942,608	-	-
Interfund loans	600,000	65,871	65,871	65,871	141,153	-
Prepaid items	-	-	-	252	-	3,196
Inventory	4,303	4,285	4,342	1,113	1,113	1,113
Total current assets	3,462,893	3,747,134	3,925,446	4,084,249	5,101,966	4,844,206
Noncurrent:						
Restricted cash and cash equivalents	2,103,159	2,571,144	4,819,944	85,477	4,845,982	578,181
Restricted investments	1,339,450	3,342,993	1,346,929	4,751,072	-	2,599,878
Receivables:						
Interfund loans	327,200	329,356	272,895	216,434	-	-
Deferred charges	267,348	168,850	157,055	145,260	81,229	-
Capital:						
Depreciated assets (net)	11,145,641	10,795,986	10,475,539	13,165,616	12,797,345	12,535,229
Infrastructure	34,140,777	34,324,834	36,067,858	40,343,437	41,393,170	44,433,011
Construction in progress	2,028,916	1,184,396	4,760,372	140,508	132,129	56,210
Total capital assets (net)	47,315,334	46,305,216	51,303,769	53,649,561	54,322,644	57,024,450

Total noncurrent assets	51,352,491	52,717,559	57,900,592	58,847,804	59,249,855	60,202,509
Total assets	54,815,384	56,464,693	61,826,038	62,932,053	64,351,821	65,046,715
DEFERRED OUTFLOWS OF RESOURCES						
Deferred amount on debt funding	-	-	-	-	-	362,237
Total deferred outflows of resources	-	-	-	-	-	362,237
LIABILITIES						
Current liabilities:						
Accounts payable and accrued expenses	264,512	212,292	336,888	398,730	586,050	904,189
Payable to other governments	13,828	16,055	19,039	(304)	7,102	102
Due to other funds	-	28,032	-	-	4,501	-
Deposits payable	11,083	4,623	11,215	13,480	9,280	4,440
Compensated absences-current	86,665	107,507	118,270	105,004	102,698	119,073
Notes and contracts payable-current	-	-	7,827	38,219	60,551	62,330
Revenue bonds payable-current	893,965	1,062,390	1,100,321	1,142,371	1,185,952	1,222,281
Total current liabilities	1,270,053	1,430,899	1,593,560	1,697,500	1,956,134	2,312,415
Noncurrent liabilities:						
Compensated absences	86,665	107,507	118,270	105,004	102,697	119,073
Notes and contracts payable	-	50,582	1,271,137	1,491,209	1,400,039	1,337,709
Revenue bonds payable	16,829,148	17,861,330	16,943,134	15,983,738	14,931,803	14,102,891
Unearned revenue	-	-	156,000	156,000	156,000	156,000
Total noncurrent liabilities	16,915,813	18,019,419	18,488,541	17,735,951	16,590,539	15,715,673
Total Liabilities	18,185,866	19,450,318	20,082,101	19,433,451	18,546,673	18,028,088

NET POSITION						
Net investment in capital assets	29,592,221	27,381,496	33,658,557	34,994,024	36,656,660	40,488,015
Restricted for:						
Debt service	1,339,450	1,342,993	1,346,929	4,751,072	1,276,076	999,878
Capital improvements	2,103,159	4,335,487	4,663,944	3,246,906	3,413,906	2,022,181
Unrestricted	3,594,688	3,954,399	2,074,507	506,600	4,458,506	3,870,790
Total Net Position	$36,629,518	$37,014,375	$41,743,937	$43,498,602	$45,805,148	$47,380,864
Current Ratio	2.73	2.62	2.46	2.41	2.61	2.09
Debt to Net Position Ratio	0.48	0.51	0.43	0.39	0.35	0.32
Debt to Noncurrent Capital Assets Ratio	0.37	0.41	0.35	0.32	0.30	0.27

8.2.3 Findings and Trends

- The Current Ratio is calculated by dividing unrestricted current assets by current liabilities and measures a company's ability to pay short-term obligations. This ratio ranges from a high of 2.7 in 2008 to a low of 2.1 in 2013. Anything above 2.0 for this liquidity ratio is good.
- The Debt to Net Position Ratio compares total debt to total net position, which is the difference between current assets and liabilities. This ratio begins at 0.48 or 48 percent debt in 2008, increases to 0.51 in 2009 and decreases to end 2013 at 0.32. For City utilities, 50 to 60 percent is within an industry target range..
- The Debt to Noncurrent Capital Asset Ratio compares total debt to noncurrent assets, which are also known as property, plant and equipment. This ratio begins at 0.37 or 37 percent debt to 63 percent noncurrent assets in 2008. Noncurrent capital assets increase $9.7 million throughout the six year history while debt decreases $2.4 million and the ratio lowers to 0.27 in 2013. A ratio of 60 percent debt to 40 percent equity is a general industry target.

8.3 Current Financial Structure

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

8.3.1 Financial Plan

The sewer utility is an enterprise fund, meaning it is self-sufficient and rates and fees collected for sewer service support the financial obligations of the utility. The primary source of funding is derived from ongoing monthly charges for service, with additional revenues coming from annual permits, late fees, and other miscellaneous revenue. The City controls the level of user charges and, subject to statutory authority, can adjust user charges as needed to meet financial objectives.

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

1. **Capital Funding Plan.** Identifies funding sources for the total capital improvement plan (CIP) obligations during the planning period. The plan defines a strategy for funding annual CIP costs based on an analysis of available resources from rate revenues, existing reserves, connection charges, debt financing, and any special resources that may be readily available (e.g. grants, developer contributions, etc.). The capital funding plan impacts the financial plan based on use of debt financing (resulting in annual debt service) and the level of cash-funding of capital costs from annual rate revenues.

2. **Financial Forecast.** Combines the total annual capital impact with operating, maintenance and administration of the sewer system. Included in the financial plan is a reserve analysis that forecasts cash flow and fund balance activity along with testing for satisfaction of minimum fund balance policies. The financial plan ultimately evaluates the sufficiency of utility revenues in meeting all obligations, including cash uses such as operating expenses, debt service, capital outlays, and reserve contributions, as well as any coverage requirements associated with long-term debt. Based on the total annual revenue requirement to support the utility, the financial plan identifies the adjustment to rates required to complete the financial plan.

8.3.2 Capital Funding Plan

The CIP developed for this GSP identifies $15.1 million in project costs over the 6-year planning horizon, escalated to year of spending. The 10-year period totals $30.3 million.

A summary of the ten-year CIP is shown in Table 8-3. As shown, each year has varied capital cost obligations depending on construction schedules and infrastructure planning needs. Approximately 50 percent of the capital costs are within the 6-year planning period. Table 8-4 provides more detail for the 6-year CIP.

Table 8-3 – 10-Year CIP

Year	Inflated
2015	$1,585,000
2016	$1,083,000
2017	$4,852,000
2018	$2,313,000
2019	$2,387,000
2020	$2,876,000
6-Year Total	**$15,096,000**
2021-2024	$15,242,000
10-Year Total	**$30,338,000**

Table 8-4 – Six-Year Detailed CIP (inflated $)

Project	2015	2016	2017	2018	2019	2020
Leslie Rd Trunk Replacement						*future*
Keene Rd Collector Replacement						*future*
Upper North Interceptor Improvements						*future*
Bellerive LS Pump Upgrade and Downstream Improvements						*future*
Leslie Interceptor Extension	800,000					
Montana Lift Station Standby Generator	40,000					
Columbia Lift Station Standby Generator	25,000					
Waterfront Lift Station Replacement			608,000			
Renewals and Replacement	250,000	258,000	1,599,000	1,652,000	1,705,000	1,761,000
Annual Street Overlay Areas	100,000	103,000	107,000	110,000	114,000	117,000
Infiltration and Inflow Study						*future*
Influent Upgrades			2,133,000			
Engineering Report						411,000
WWTP Renewals and Replacements				551,000	568,000	587,000
Plant-wide HVAC Improvements	290,000					
Digester Building MCC	80,000					
Primary Clarifier #2 Coating		165,000				
Digester #1 Tank Coating		330,000				
Secondary Clarifier #2 Coating		227,000				
Clarifier Gear Drive Replacements			325,000			
Plant Pump and Piping Replacement			80,000			
Total Annual CIP Costs	**$ 1,585,000**	**$ 1,083,000**	**$ 4,852,000**	**$ 2,313,000**	**$ 2,387,000**	**$ 2,876,000**

8.3.3 Capital Financing Strategy

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

- Facility Fee reserves for identified growth projects
- Other accumulated cash reserves
- Transfers of excess cash (over minimum balance targets) from the Operating Fund

- Annual cash from rates earmarked for system reinvestment funding
- Interest earned on Capital Fund balances and other miscellaneous capital resources
- Revenue bond financing

Based on information provided by the City, the sewer utility began 2014 with $3.91 million in the Operating Fund and $2.02 million in the Facilities Fee Fund. Additional funds beyond the Operating Fund target of forty five days of cash operating expenses are transferred to the Capital Fund. Table 8-5 presents the corresponding 10-year capital financing strategy.

Table 8-5 – 10-Year Capital Financing Strategy

Year	Capital Expenditures Inflated	Revenue Bond Financing	Cash Funding	Total Financial Resources
2015	$1,585,000	-	$1,585,000	$1,585,000
2016	1,083,000	-	1,083,000	1,083,000
2017	4,852,000	-	4,852,000	4,852,000
2018	2,313,000	469,585	1,843,415	2,313,000
2019	2,387,000	1,129,594	1,257,406	2,387,000
2020	2,876,000	1,251,527	1,624,473	2,876,000
Subtotal	**$15,096,000**	**$2,850,705**	**$12,245,295**	**$15,096,000**
2021-2024	15,242,000	-	15,242,000	15,242,000
Total	**$30,338,000**	**$2,850,705**	**$27,487,295**	**$30,338,000**

The 10-year capital funding plan indicates the City's cash reserves are sufficient to meet 91% of the total capital funding need. Revenue bond proceeds of $2,850,000 complete the funding plan for both the 6 and 10 year planning periods.

The capital funding plan assumes a consistent growth rate among financial and system capacity planning. It is assumed that if growth is not occurring at the planned rate, the timing of capital projects would be adjusted accordingly and revenue impacts evaluated.

8.4 Available Funding Assistance and Financing Resources

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

8.4.1 City Resources

Resources appropriate for funding capital needs include accumulated cash in the construction fund, rate revenues designated for capital spending purposes, and capital-related charges such as the Connection Fee. The first two resources will be discussed in the Fiscal Policies section (8.5.2) of the Financial Forecast. Capital-related charges are discussed below.

8.4.1.1 Connection Fees (Facility Fees)

A connection fee refers to a one-time charge imposed on new customers as a condition of connecting to the sewer system. The City refers to this charge as a facility fee. The purpose of the connection fee is to promote equity between new and existing customers. Revenue can only be used to fund utility capital projects or to pay debt service incurred to finance those projects. The City currently charges all new customers a Connection Fee based on water meter size, with a base rate of $1,995 for a 3/4" meter.

8.4.1.2 Local Facilities Charges

While a connection charge is the manner in which new customers pay their share of general facilities costs, local facilities funding is used to pay the costs of local facilities that connect each property to the system's infrastructure. Local facilities funding is often overlooked in rate forecasting because it is funded up-front by either connecting customers, developers, or through an assessment to properties, but never from rates.

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

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

The developer extension is a requirement that a developer install onsite and sometimes offsite improvements as a condition of extending service. These are in addition to the connection charge required and must be built to City standards. Part of the agreement between the City and the developer planning to extend service might include a late-comer agreement, resulting in a late-comer charge to new connections to the developer extension.

Latecomer charges are a variation of developer extensions whereby new customers connecting to a developer-installed improvement make a payment to the City based on their share of the developer's cost (RCW 35.91.020). The City passes this charge on to the developer who installed the facilities. As part of the developer extension process, a later comer agreement between the City and developer defines the allocation of costs and records latecomer obligations on the title of affected properties. No interest is allowed, and the reimbursement agreement cannot exceed 20 years in duration, except under special circumstances.

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

8.4.2 Outside Resources

This section outlines various grant, loan and bond opportunities available to the City through federal and state agencies to fund the CIP identified in the GSP.

8.4.2.1 Grants and Low Cost Loans

Historically, federal and state grant programs were available to local utilities for capital funding assistance. However, these assistance programs have been mostly eliminated, substantially reduced in scope and amount, or replaced by loan programs. Remaining miscellaneous grant programs are generally lightly funded and heavily subscribed. Nonetheless, even the benefit of low-interest loans makes the effort of applying worthwhile. Grants and low-cost loans for Washington State utilities are available from the Department of Commerce including two assistance programs that the City may be eligible for.

Public Works Trust Fund (PWTF) – Cities, counties, special purpose districts, public utility districts, and quasi-municipal governments are eligible to receive loans from the PWTF. Eligible projects include repair, replacement, and construction of infrastructure for domestic water, sanitary wastewater, stormwater, solid waste, road, and bridge projects that improve public health and safety, respond to environmental issues, promote economic development, or upgrade system performance. Currently the Public Works Board has suspended the non-Construction Programs and significantly reduced funding to the construction loan program. The Public Works Board website notes that the next funding cycle is to be determined by funding levels in early 2016-17.

When the program is funded and available, PWTF loans are available at interest rates ranging from 1.28 percent to 2.55 percent depending on the repayment term, with reduced interest rates available for all projects located in “distressed” communities. The standard loan offer is 2.55 percent interest repaid over a 5 to 20 year term. All loan terms are subject to negotiation and Board approval. Currently no local match is required and the maximum loan amount is $7 million per jurisdiction per biennium.

Information regarding the application process as well as rates and terms are posted on the PWTF website in early spring. The next application cycle is planned for the spring of 2016.

Further detail is available at http://www.pwb.wa.gov.

8.4.2.2 Bond Financing

General Obligation Bonds – General Obligation (G.O.) bonds are bonds secured by the full faith and credit of the issuing agency, committing all available tax and revenue resources to debt repayment. With this high level of commitment, G.O. bonds have relatively low interest rates and few financial restrictions. However, the authority to issue G.O. bonds is restricted in terms of the amount and use of the funds, as defined by Washington constitution and statute. Specifically, the amount of debt that can be issued is linked to assessed valuation.

RCW 39.36.020 states:

> *"(ii) Counties, cities, and towns are limited to an indebtedness amount not exceeding one and one-half percent of the value of the taxable property in such counties, cities, or towns without the assent of three-fifths of the voters therein voting at an election held for that purpose.*
>
> *(b) In cases requiring such assent counties, cities, towns, and public hospital districts are limited to a total indebtedness of two and one-half percent of the value of the taxable property therein."*

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

Revenue Bonds – Revenue bonds are commonly used to fund utility capital improvements. The debt is secured by the revenues of the issuing utility. With this limited commitment, revenue bonds typically bear higher interest rates than G.O. bonds and also require security conditions related to the maintenance of dedicated reserves (a bond reserve) and financial performance (added bond debt service coverage). The City agrees to satisfy these requirements by resolution as a condition of bond sale.

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

8.5 Financial Forecast

The financial forecast, or revenue requirement analysis, forecasts the amount of annual revenue that needs to be generated by user rates. The analysis incorporates operating revenues, O&M expenses, debt service payments, rate-funded capital needs, and any other identified revenues or expenses related to operations. In addition to annual operating costs, the revenue needs also include debt covenant requirements and specific fiscal policies and financial goals of the City. The objective of the financial forecast is to evaluate the sufficiency of the current level of rates.

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

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

Coverage Test – The coverage test is based on a commitment made by the City when issuing revenue bonds and some other forms of long-term debt. As a security condition of issuance, the City would be required per covenant to agree that the revenue bond debt would have a higher priority for payment (a senior lien) compared to most other expenditures; the only outlays with a higher lien are O&M expenses. Debt service coverage is expressed as a multiplier of the annual revenue bond debt service payment. For example, a 1.0 coverage factor would imply that no additional cushion is required. A 1.25 coverage factor means revenue must be sufficient to pay O&M expenses, annual revenue bond debt service payments, plus an additional 25 percent of annual revenue bond debt service payments. The excess cash flow derived from the added coverage, if any, can be used for any purpose, including funding capital projects. Targeting a higher coverage factor can help the City achieve a better credit rating and provide lower interest rates for future debt issues.

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

8.5.1 Current Financial Structure

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

8.5.2 Fiscal Policies

A brief summary of the key financial policies employed by the City, as well as those recommended and incorporated in the financial program are discussed below.

Operating Fund – Operating reserves are designed to provide a liquidity cushion to ensure that adequate cash working capital will be maintained to deal with significant cash balance fluctuations such as seasonal fluctuations in billings and receipts, unanticipated cash expenses, or lower than expected revenue collections. The City's current policy is to maintain a minimum balance in the Operating Fund equal to 45 days of O&M expenses.

Capital Fund – A capital contingency reserve is an amount of cash set aside in case of an emergency should a piece of equipment or a portion of the utility's infrastructure fail unexpectedly. The reserve also could be used for other unanticipated capital needs including capital project cost overruns. Industry practices range from maintaining a balance equal to 1 to 2 percent of fixed assets, an amount equal to a 5-year rolling average of CIP costs, or an amount determined sufficient to fund equipment failure (other than catastrophic failure). The final target level should balance industry standards with the risk level of the City. The City's does not currently maintain a capital contingency reserve. It is recommended for consideration in future policy review and rate planning.

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

The City's Utility Financial Operating Policy states that "traditional convention is to rate-finance a portion of capital additions at a level equal to annual depreciation expense". In this analysis, the routine capital expense for system reinvestment is funded based on the existing policy. These monies are put directly into the Capital Fund and are made available for capital project costs. A phase-in approach is applied to this policy in 2017 through 2019 to bring the utility up to a fully funded level.

Debt Management – It is prudent to consider policies related to debt management as part of broader utility financial policy structure. Debt management policies should be evaluated and formalized including the level of acceptable outstanding debt, debt repayment, bond coverage and total debt coverage targets. The City's existing bond covenants require a 1.25 debt coverage test, which is met throughout the forecast.

8.5.2.1 Financial Forecast

The financial forecast is developed from 2014 budget documents along with other key factors and assumptions to develop a complete portrayal of the City's annual financial obligations. The following is a list of the key revenue and expense factors and assumptions used to develop the financial forecast:

- **Revenue** – The City has two general revenue sources: revenue from charges for service (rate revenue) and miscellaneous (non-rate) revenue. In the event of a forecasted annual shortfall, rate revenue can be increased to meet the annual revenue requirement. Non-rate revenues are forecast to escalate based on the nature of the revenue.
- **Connection Fee Revenue** – The current connection fee of $1,995 is expected to increase based on the connection fee update, however connection fee revenue has been forecast in the rate study based on the current connection fee to be more conservative. The current connection fee is expected to generate between $597,000 in 2015 and just under $985,000 in 2024, collected from 300 to 500 new annual residential equivalent connections. This money is used to fund growth related capital projects.
- **Growth** – Rate revenue is escalated based on an annual growth rate of 1.85% beginning in 2017, provided in **Section 2.11** of this GSP. Revenue projections in 2015 and 2016 are based on the actual 2014 growth rate of 1.3%.
- **Expenses** – O&M expense projections are based on the 2014 budget and are forecast to increase with general cost inflation of 2.29 percent, construction cost inflation of 3.26 percent, labor cost inflation of 2.22 percent, and benefit cost inflation of 4.26 percent. Budget figures were used for 2014 taxes; future taxes are calculated based on forecasted revenues and prevailing tax rates.
- **Existing Debt** – The City currently has a total of six outstanding sewer debt issues, including five revenue bonds and one American Recovery and Reinvestment Act (ARRA) loan. Revenue bond annual payments range from $1.88 million decreasing to $240,000 when two revenue bond issues are eliminated. ARRA annual payments are about $103,000 per year and expire in 2031.

- **Future Debt** – The capital financing strategy developed for this GSP indicates that borrowing will be required in years 2018 through 2020 to complete the CIP, resulting in new debt service repayment obligations beginning in 2018.
- **Transfer to Capital** – Any Operating Fund balance above the minimum requirement is assumed to be available to fund capital projects and is projected to be transferred to the Capital Fund each year. The 2014 Operating Fund balance is expected to end the year at 62 days of O&M expenses, which includes cushion above the minimum target for the rate-smoothing strategy. The Capital Fund balance is expected to end the year at approximately $3.6 million.

Although the financial plan is completed for the 10-year time horizon of this GSP, the revenue requirement forecast focuses on the shorter term planning period 2015 through 2020. It is important that the City revisit the forecast every 2 to 3 years to ensure that the rate projections developed remain adequate. Any significant changes should be incorporated into the financial plan and future rates should be adjusted as needed.

Table 8-6 summarizes the annual revenue requirements based on the forecast of revenues, expenditures, fund balances and fiscal policies.

Table 8-6 – 6-Year Financial Forecast

Revenue Requirement	2015	2016	2017	2018	2019	2020
Revenues						
Rate Revenues Under Existing Rates	$ 8,809,133	$ 8,924,538	$ 9,089,641	$ 9,257,800	$ 9,429,069	$ 9,603,507
Non-Rate Revenues	274,116	274,030	273,870	273,833	273,935	274,131
Total Revenues	**$ 9,083,249**	**$ 9,198,568**	**$ 9,363,512**	**$ 9,531,633**	**$ 9,703,005**	**$ 9,877,638**
Expenses						
Cash Operating Expenses	$ 6,560,926	$ 6,722,825	$ 6,899,246	$ 7,076,872	$ 7,259,531	$ 7,447,382
Existing Debt Service	1,974,153	1,972,424	1,982,185	1,965,750	1,958,470	1,956,064
New Debt Service	-	-	-	39,752	135,377	241,323
Rate Funded System Reinvestment	600,000	600,000	731,348	1,124,777	1,537,895	1,979,889
Total Expenses	**$ 9,135,079**	**$ 9,295,249**	**$ 9,612,779**	**$ 10,207,152**	**$10,891,273**	**$ 11,624,658**
Net Surplus (Deficiency)	**$ (51,830)**	**$ (96,681)**	**$ (249,267)**	**$ (675,518)**	**$ (1,188,268)**	**$ (1,747,020)**
Additions to Meet Coverage	-	-	-	-	-	-
Total Surplus (Deficiency)	**$ (51,830)**	**$ (96.681)**	**$ (249,267)**	**$ (675,518)**	**$ (1,188,268)**	**$ (1,747,020)**

The financial forecast indicates that there is an existing deficiency at current rate levels, and that sewer rates will need to increase to meet the total annual financial requirement in all years. Rates would need to increase a total of 21.5 percent by 2020 to achieve revenue sufficiency. The City is currently in the process of completing a rate study to adopt a near-term rate plan that will establish annual rate increases. The remaining summaries are based on 5 percent annual rate increases in 2017 through 2020 to achieve the cumulative 21.5 percent increase.

8.5.3 City Funds and Reserves

Table 8-7 shows a summary of the projected Operating Fund, Capital Fund, and Facilities Fee Fund ending balances through 2020 based on the rate forecasts presented above. The operating fund is maintained at a minimum of 45 days of operating expenses, the capital fund balance is depleted through funding the CIP, and the Facilities Fee Fund is used only for qualifying CIP projects, dipping in 2015, then building up as Facilities Fee revenue exceeds annual qualified CIP project spending.

Table 8-7 – 6-Year Financial Forecast

Ending Fund Balance	2015	2016	2017	2018	2019	2020
Operating Fund	$ 969,404	$ 872,723	$ 850,592	$ 872,491	$ 895,011	$ 918,170
Capital Fund	2,926,237	2,449,072	1,112,068	132,629	86,578	36,196
Facilities Fee Fund	1,793,343	2,401,871	531,194	1,414,448	2,315,488	3,234,645
Total	**$ 5,688,984**	**$ 5,723,667**	**$ 2,493,853**	**$ 2,419,568**	**$ 3,297,076**	**$ 4,189,011**

8.6 Existing and Projected Rates

8.6.1 Existing (2015) Rates

The City's current rate structure consists of two rate components, a fixed monthly charge based on rate class, which is charged to all customers, and a monthly usage charge per hundred cubic feet (ccf) that is charged to multifamily and commercial customers. **Table 8-8** shows the existing rate structure.

Table 8-8 – 2015 Existing Rate Structure

	Existing
Residential	$ 25.60
Multifamily	
Base Charge	$ 12.40
Usage Charge (per ccf)	$ 2.15
Commercial	
Base Charge	$ 61.50
Usage Charge (per ccf)	$ 2.15

8.6.2 Projected Rates

While the City' annual rate strategy to achieve revenue sufficiency is being addressed in the rate study currently underway, the cumulative adjustment by 2020 of 21.5 percent is applied to the existing rate structure to project rates in 2020. **Table 8-9** shows the projected rates as applied uniformly to all rate components in all classes. A cost of service analysis is a part of the rates study and changes to the rates might result from those findings as well.

Table 8-9 – 6-Year Projected Rates

	Existing	2020
Residential	$ 25.60	$ 31.10
Multifamily		
Base Charge	$ 12.40	$ 15.07
Usage Charge (per ccf)	$ 2.15	$ 2.61
Commercial		
Base Charge	$ 61.50	$ 74.72
Usage Charge (per ccf)	$ 2.15	$ 2.61

Table 8-10 shows the residential monthly bill impact.

Table 8-10 – Monthly Bill Comparisons

Single-Family	Monthly Bill
Existing Monthly Bill	$ 25.60
Projected to 2020	$ 31.10
$ Difference	$ 5.50
Total Rate Increase	21.5%

8.7 Connection Fee

The Connection Fee, or Facilities Fee, is imposed as a condition of service on new customers connecting to the system. In addition to any other costs related to physically connecting a customer to the system, the connection fee is typically based on a blend of historical and planned future capital investments in system infrastructure.

The purpose of the connection fee is two-fold: 1) to provide a source for capital financing and 2) to equitably recover a proportionate share of utility plant-in-service costs from new customers. In the absence of connection fees, growth-related costs would be borne in large part by existing customers. The cost of the system to be recovered by connection fees can be defined in two parts: an existing cost portion based on historical investments in existing infrastructure, and a future cost portion that recovers costs related to planned capital projects. Revenues generated from the connection fees can be used to fund capital projects or debt service incurred to finance capital projects, but should not be used to pay for operating and maintenance costs.

The existing cost basis is intended to recognize the current ratepayers' net investment in the original cost of system assets. The total cost of the sewer system reflects:

- ***Utility Plant-In-Service:*** The majority of the existing cost basis is composed of the original cost of plant-in-service, as documented in the City's 2013 fixed asset schedule. 2014 asset additions were available before completion of this analysis and were added in lieu of 2013 Construction Work in Progress.

- ***Less: Contributed Capital:*** Assets funded by developers or grants are excluded from the cost basis on the premise that the connection fee should only recover costs actually incurred by the City. Assets funded by special assessments are also excluded from the cost basis to avoid double charging customers for assets that were funded through those assessments. City staff provided records of historical annual capital contributions since 2010. Data on contributions in previous years was not available.

- ***Plus: Interest on Utility-Funded Assets***: RCW 35.92.025 and subsequent legal interpretations provide a guideline for connection charges which suggests that such charges can include interest on an asset at the rate applicable during the time of construction. Using the historical Bond Buyer Index for 20-year term bonds, interest can accumulate for a maximum of ten years from the date of construction for any particular asset, and cannot exceed an interest earnings rate above 10% in any given year. Conceptually, this interest provision attempts to account for opportunity costs that the City's customers incurred by supporting investments in infrastructure rather than having it available for investment or other uses.

- ***Less: Net Debt Principal Outstanding:*** Another adjustment to the existing system cost basis is to deduct the net liability of outstanding utility debt, recognizing that new customers will bear a proportionate share of this debt related to existing assets through their utility rates. Therefore, the cost of those assets charged to new development is offset to some degree by the remaining debt liability. Since the utility typically has cash resources that are not included in the system cost basis, the net debt load is defined as total debt minus outstanding cash and investments.

Development of the existing system cost basis is shown in **Table 8-11.**

Table 8-11 – Existing Cost Basis – Connection Fee

Existing System Cost Basis	$
Sewer Capital Assets	$86,501,954
Contributed Assets	(4,434,612)
Interest Accrued on Assets	42,783,253
Net Outstanding Debt Calculation::	
Outstanding Debt Principal	(5,341,640)
Cash Balances Y-E 2013	3,909,616
Net Outstanding Debt	(1,432,024)
Net Existing System Cost Basis	**$123,418,571**

The future cost basis can include utility capital projects planned for construction and identified in the comprehensive system planning documents. Each project in the 2015 – 2024 capital improvement program was allocated as either "upgrade/expansion" or "repair/replacement." Totals for each utility are listed below:

- ***Repair and Replacement Projects:*** Projects costs allocated to the repair/replacement category are excluded from the cost basis. The cost of the utility asset being replaced is included in the existing cost basis. Excluding repair/replacements avoids double-counting the cost of a utility asset by including it in both existing and future cost totals.

- ***Upgrade and Expansion Projects:*** Projects that are planned to serve system growth by expanding system capacity, or are planned to improve existing service levels and/or meet new regulations are included in the upgrade/expansion allocation.

- ***Less: Outside Funding Sources:*** Projects directly funded by developers or special property assessments are not included in the calculation.

The future system cost allocation results are summarized in **Table 8-12.**

Table 8-12 – Future Cost Basis – Connection Fee

Future System Cost Basis	$
Total Capital Improvement Program (2015$)	$25,520,000
Less: Repair and Replacement Projects	(21,164,500)
Net Future System Cost Basis	**$4,355,500**

In order to calculate an equitable share of the system costs for new connections, the connection fee cost basis is divided by the number of Meter Capacity Equivalents (MCEs) the system can serve when the CIP is complete. Total connection fee cost basis, divided by total capacity served by the system, determines the equitable unit cost of system buy-in as a basis for setting the connection fee. Projected 2024 ERUs of 42,500 is the maximum capacity the system can serve based on the facilities in place at CIP completion based on JUB capacity analysis. Applying the same growth rate used to arrive at the ERU capacity level to the existing MCE total, results in the MCE capacity served at CIP completion. Calculation of the unit cost connection fee is shown in **Table 8-13.**

Table 8-13 – Unit Cost - Connection Fee

Connection Fee Unit Cost Calculation	$
Existing Cost Basis	$123,418,571
Future Cost Basis	4,355,500
Total Cost Basis	$127,774,071
Total System Capacity Served (MCEs)	27,116
Unit Cost of System Capacity – Connection Fee per MCE	**$4,712**

The updated fee is $2,717 more than the current $1,995 per MCE. The City's existing connection fee is based on the water meter size of new customers. The updated charge would represent the fee for the standard single family meter size. The meter capacity ratio for larger meter sizes would be applied to the updated base fee to determine the connection fee.

8.8 Affordability

The Department of Health and the Department of Commerce Public Works Board use an affordability index to prioritize low-cost loan awards depending on whether rates exceed 2.0 percent of the median household income for the service area. The median household income for the City of Richland was $68,744 in 2008 – 2012 according to the U.S. Census Bureau. The 2012 figures are escalated based on the assumed 2.22 percent labor cost inflation to show the median household income in future years. **Table 8-14** presents the City's rates projected to 2020, tested against the 2.0 percent monthly affordability threshold.

Table 8-14– Affordability Analysis

Year	Labor Inflation	Median HH Income	2% Monthly Threshold	Projected Monthly Bill	% of Median HH Income
2012		$ 68,744	$ 114.57		
2013	2.22%	70,270	117.12		
2014	2.22%	71,830	119.72	25.60	0.43%
2020	**2.22%**	**81,945**	**136.57**	**31.10**	**0.46%**

Applying the 2.0 percent test, the City's rates are forecast to remain within the indicated affordability range through 2020.

8.9 Conclusion

The results of this analysis indicate that rate increases are necessary to fund ongoing operating needs and future debt requirements to fund the CIP, as well as meet financial policy targets. Implementation of a rate plan that achieves the 21.5% cumulative increase to rates by 2020 should provide for continued financial viability while maintaining generally affordable rates.

It is important to remember that the analysis performed in this chapter assumes growth rates from **Section 2.11** of this GSP. If the future growth rates change, the proposed annual rate increases may need to be updated and revised.

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

Chapter 9

Operations Program

Chapter 9 – Operations Program

9.1 Introduction

The City maintains 23 full-time equivalent (FTE) staff to perform standard operation and maintenance for more than 262 miles of collection system, 14 sewage lift stations, and the City's WWTP. Standard operation and maintenance includes a preventative maintenance program for maintaining the Wastewater Utility. This chapter presents a description of the Wastewater Utility management and the elements of the preventative maintenance program for operating and maintaining the Wastewater Utility.

9.2 Wastewater Utility Management

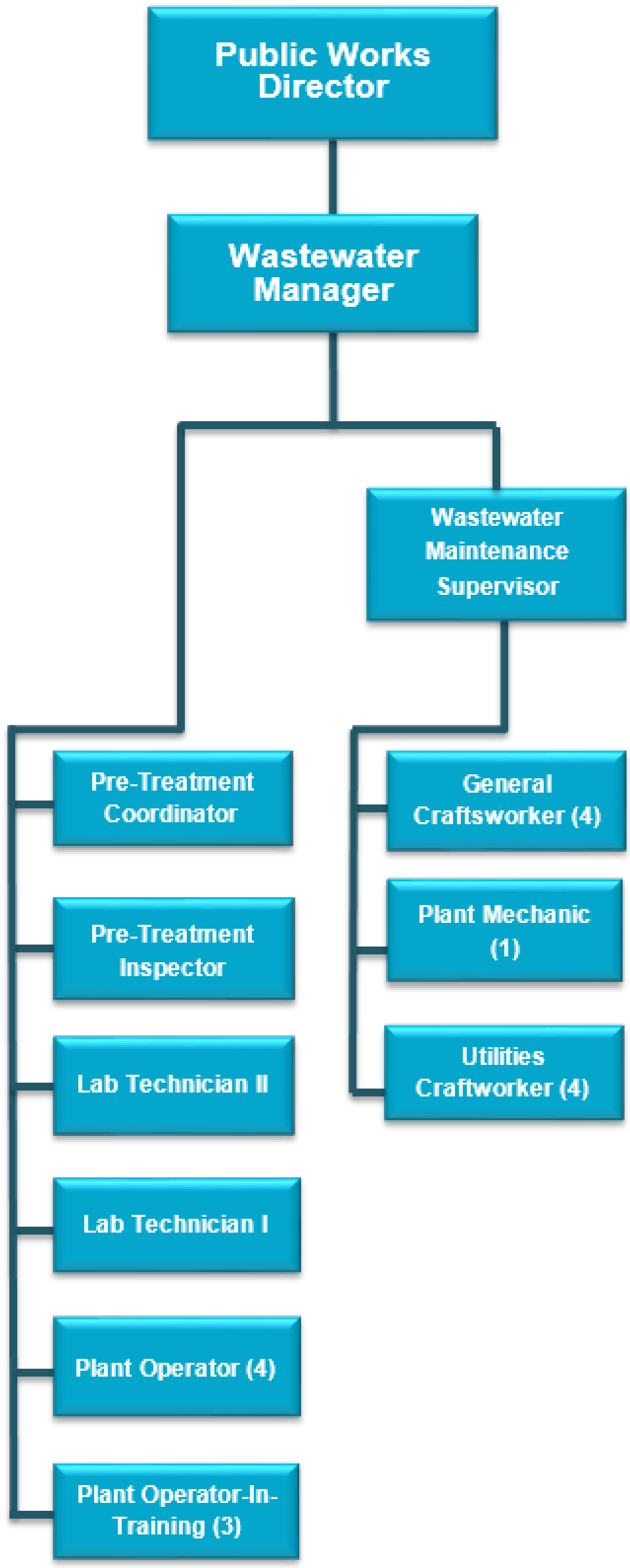
9.2.1 Organizational Structure

The Wastewater Utility operates under the direction of the Public Works Department. The Wastewater Manager, reports to the Public Works Director, and manages the activities of the Wastewater Utility. **Figure 9-1** illustrates the organizational structure of the Wastewater Utility.

The wastewater maintenance unit is responsible for the operation and maintenance of the collection system including the lift stations. The wastewater maintenance supervisor and nine craft workers are assigned to this service unit.

The WWTP is staffed with those responsible for the daily operations and maintenance of equipment and facilities at the WWTP, including conveyance facilities associated with the plant, and the City's pretreatment program. The pretreatment staff, two lab technicians, and plant operators are assigned to this service unit.

Figure 9-1 – Wastewater Utility Organizational Chart



9.2.2 Staffing

As previously indicated, the Wastewater Utility has a total of 23 FTE staff. In 2005, the EPA published a guide, which includes a table with suggested manpower guidelines for public wastewater collection systems based on city population. As per **Section 2.11**, the City's 2015 population estimate is 53,054 and therefore a manpower estimate for a population of 50,000 was used. The recommendations are based on EPA documents with a publication date of 1973 and 1974 (over 40 years old); however they provide a point of comparison. As a disclaimer, EPA notes that the manpower values may not take into account technological advances that have occurred since the publication date that might reduce staffing requirements. The suggestions are listed in **Table 9-1**.

Table 9-1 – Staffing Comparison [(a)]

Occupational Title	Est. number of personnel	Est. total man-hours per week
Superintendent	1	40
Assistant Superintendent		
Maintenance Supervisor	1	40
Foreman	1	40
Maintenance II	1	40
Maintenance I	3	120
Mason II [(b)]	1	40
Mason I		
Maintenance Equipment Operator	2	80
Construction Equipment Operator	1	40
Auto Equipment Operator		
Photo Inspection Technician		
Laborer	2	80
Dispatcher	1	40
Clerk Typist	1	40
Stock Clerk	1	40
STAFF TOTAL	**16**	**640**

(a) *Presented in Guide for Evaluating Capacity, Management, Operation, and Maintenance (CMOM) Programs at Sanitary Sewer Collection Systems by EPA.*

(b) *Originally included for systems constructed of brick, the position can be replaced with a system-specific position.*

The total staffing requirements, based on the EPA guidelines, is 16 FTE's. The City currently has 23 FTEs and appears to be well staffed according to these guidelines.

9.2.3 Training

Well-trained staff are an essential part of an effective operation and maintenance program. In addition to the workforce possessing the education and skills necessary to operate and maintain a utility system that is becoming increasingly complex with automation and computerization, staff training and education is seen as an important aspect of workforce retention and recruitment.

Maintenance personnel should be familiar with current equipment and procedures, as well as having access to review all applicable regulations. Training criteria should be established for each job description with periodic reviews conducted accordingly. The Wastewater Utility currently budgets $9,000 per year for technical training of the staff – this includes travel expenses, tuition and conference costs, safety pamphlets, and in-house safety training. The staff also attend non-cost PNCWA regional section meetings several times per year. The Wastewater Utility staff is allocated 30-40 full training days per year.

In addition to staff training, a succession plan to document and record system knowledge from aging staff, nearing retirement, is also of value. Updating maintenance procedures and collection system records by experienced senior staff and implementing skill-based training and knowledge transfer should be incorporated.

9.2.4 Customer Service

An effective customer service and public relations program ensures that the City and the Wastewater Utility address all incoming inquiries, requests, and complaints in a timely fashion. From this customer information, the City and the Wastewater Utility may further develop or revise programs to better address areas of concern. Currently, when a problem or customer complaint is received the responder updates the Wastewater Supervisor. The Supervisor then contacts a crew in the field to respond. Typically crews respond within 30 minutes of receiving the contact call. Complete details of the problem or complaint are entered to the City's Sewer Incident Reports. The program then returns a job ticket with an assigned work order number. This information is stored in a computer database. Awareness of past issues will help the Wastewater Supervisor to determine how the amount and types of inquiries, requests, or complaints are trending.

During daily routine and complaint-related activities, the collection system field crews and their activities are the most visible segment of any Wastewater Utility. Workers project a public image and therefore staff should be aware of how to respond to the public and familiar with any existing easements that might require access onto private property to service facilities. Vehicles should also be equipped with adequate emergency lighting, flashers, and signs for visible notification and traffic control. As appropriate, operators should notify homeowners prior to maintenance or construction work.

9.2.5 Maintenance Management and Record Keeping

The Wastewater Utility uses the Hansen Information Technologies (now Infor Hansen) software for maintenance management recording and record keeping. To maintain access to the most current collection system information all maintenance work performed on the collection system, including new construction, labelling of manholes and clean-outs, closed circuit television (CCTV) inspections, preventative maintenance including cleanings and repairs, sanitary sewer backups, and sanitary sewer overflows (SSO) is recorded on a log sheet that is kept on file at the WWTP.

9.3 Operation and Maintenance Activities and Programs

This section presents elements of the preventative maintenance program for operating and maintaining the Wastewater Utility. It includes operations and maintenance routines with preventative and corrective routines, wastewater-related programs and an inventory of equipment.

9.3.1 Collection System Maintenance

9.3.1.1 Gravity Sewer Pipe

The gravity sewer pipe collection system maintenance program consists of cleaning and flushing pipes, root removal and treatment, CCTV inspection, and construction repairs (both minor and major). To address these maintenance requirements the Wastewater Utility maintains a staff of six utility workers and one maintenance supervisor. The utility workers are divided into three two-man crews, each assigned to a specific maintenance task: 1) hydraulic jet flushing crew for scheduled preventative maintenance of City pipes; 2) a second hydraulic jet crew that also completes hydraulic sawing and rodding; 3) a CCTV crew for inspection of all sewer pipes 6-inches and larger. Each crew is also available to make repairs to sewer pipes, manholes and clean-outs and respond to emergency sewer back-up calls as necessary. Each maintenance program is described below.

Cleaning and Flushing

A principal goal of maintaining public support and system reliability of the wastewater collection system is to ensure that sewers remain clear of stoppages and free of odor. The City has a goal to clean all sewer pipes (262 miles) at least once every three years. Problem areas are identified and placed on a routine cleaning schedule until system repairs are made to eliminate the restrictions or problem areas.

Roots, grease and deposited solids are the most common cleaning problems. Effective control of these problems necessitates understanding how they develop. Grease builds up in a pipe over time as waste oils from foods float on the surface of the wastewater and coat the inside of the pipe. Repeated coatings can restrict a pipe to a fraction of its original size and inhibit flow. The grease coating hardens over time and becomes difficult to remove. This problem is usually found near restaurants and commercial food processors. Household garbage disposal units also affect the character of residential wastewater and can lead to grease problems. Rodding and hydraulic cleaning are used to treat root and grease problems.

The City uses a Trailer Mounted Hand Rodder and Power Rodder with motor to turn the rod. The rod, which is stored on a reel, is fed into the sewer pipe to the point of obstruction and turned automatically. It can be set so that the machine functions with little operator effort. Rodders are used to clear obstructions such as root intrusions and grease accumulations and retrieve rags and other materials. With proper tools, roots and obstructions in six inch and up to fifteen inch pipes can be removed. Set up time for rodders is longer than is required for hydraulic cleaners and greater operator skill is needed. All equipment is limited to use on driveable surfaces – although equipment must sometimes be pushed by hand behind homes or at access points off of the street. Because the skill level required for this equipment is higher than for hydraulic cleaners, safe and effective operation of the rodder requires thorough training. Experience is necessary to operate rods without damaging them.

Hydraulic cleaning uses high velocity water to clean the pipe. A water pump delivers water through a nozzle at high pressure and velocity, moving most materials through the pipe.

Currently two, two-man crews are performing the cleaning and flushing maintenance in the City. To provide the recommended level of maintenance (once every three years) a total of 461,120 lineal feet (LF) (1/3 of 262 miles) of sewer pipe is required to be cleaned and flushed per year. A total of 2,500 man-hours is required to provide this recommended level of maintenance with the current collection system.

Root Control and Treatment

Chemical treatment with RootX is used by maintenance staff in many locations throughout the City.

CCTV Inspection

Inspection by CCTV is an effective method of determining the nature and extent of internal problems in the City's collection system. The City's CCTV inspection crew consists of a two-man crew using a portable TV camera mounted on rubber wheels. The City has a goal to inspect all sewer pipes (262 miles) at least once every ten years. Required inspection is performed on both existing and new sewer pipes and where problems have been reported. All inspections are captured on video and all information recorded during inspection is uploaded into the Infor Hansen maintenance database. Not only can reports be generated with the inspection, but a permanent visual record can be made for subsequent review. In addition, television inspection of the entire system would provide an inventory of all system conditions that could be used to prioritize rehabilitation options for the City system.

Currently a two-man crew performs all the CCTV inspections in the City. To provide the recommended level of maintenance (once every ten years) a total of 138,336 LF (1/10 of 262 miles) of sewer pipe is required to be inspected per year.

System Repairs

Collection system repairs include both major and minor repairs. Only minor system repairs such as manhole repair, manhole cover replacement and adjustments, and repairs involving only limited excavations (spot repair on shallow lines) is typically performed by City staff. Major system repairs are contracted through the Public Works Department to trenching and piping contractors with the equipment and personnel to perform this work.

9.3.1.2 Manholes

The majority of the existing City manholes are concrete and overtime can deteriorate due to age or chemical attack. Manhole rehabilitation options include concrete-based linings, chemical lining systems, and chemical grouting. As the City's collection system ages, manhole rehabilitation may be considered compared to structure replacement, especially in areas that are difficult to access or in high use. Rehabilitation options are listed below:

1. Concrete-Based Linings:
    a. Goal: To maintain existing concrete, no chemical attack resistance
    b. Surface Prep: High pressure wash to remove debris
    c. Result: Similar to a non-shrink grout that can include additives to reduce shrinkage cracking

2. Chemical Lining System:
    a. Goal: To halt the chemical attack on the concrete surface
    b. Surface Prep: Range of options including pressure wash to sand blasting
    c. Result: Final coating thickness is product specific and can vary but will line the walls and channel

3. Chemical Grouting:
    a. Goal: To halt groundwater intrusion cause by infiltration of groundwater
    b. Surface Prep: None
    c. Result: Chemical is injected in the intrusion area to fill voids, multiple injections may be required

9.3.1.3 Lift Stations

The City currently maintains 14 lift stations spaced throughout the collection system. Six are located south of the Yakima River with the remaining eight located to the north. To maintain each lift station, the Wastewater Utility conducts site visits approximately once per week. During these visits, preventative maintenance tasks are performed and recorded in each lift station logbook/recordbook. Any maintenance issues are recorded at that time. These issues are typically addressed and resolved once the correct tools and parts are obtained.

Eight of the lift stations are submersible pump style lift stations. Preventative maintenance at these lift stations includes, routinely inspecting the concrete wet wells for surface striping or spalling caused by hydrogen sulfide and a submersible pump drawdown test. The wet wells are cleaned out as needed based on visual inspection. Five of the City's lift stations are the wet pit/dry pit style lift stations. Preventative maintenance at these lift stations includes, checking logs and runtime hours for each pump, test pump operation, check valve operation, test alarm functions, and visual inspection of condition and need for maintenance

Currently backup power provisions for one the lift stations is provided (Montana LS).The collection system renewal and replacement list identifies that standby generators will be installed at the Columbia Park Trail lift station in 2016.

9.3.2 WWTP Maintenance

The wastewater treatment plant maintenance unit is responsible for the maintenance at the WWTP. Routine tasks at the WWTP include valve, pump, and telemetry maintenance and any preventative maintenance routines on all equipment.

A detailed list of maintenance requirements and schedule is provided in the WWTP Operations and Maintenance Manual (updated 2015).

9.3.2.1 New Facilities

Recent facility upgrades include the following:
1. Conversion of chlorine gas to on-site generated sodium hypochlorite for final wastewater disinfection.
2. Replacement of the variable frequency drives on the influent lift station pumps.
3. Position gauges for the floating digester gas covers.

9.3.3 Wastewater Programs

9.3.3.1 Pretreatment Program

The City's pretreatment program regulates the quality of wastewater discharged to the WWTP through the Richland Municipal Code (RMC) section 17.30, which lists general provisions, discharge requirements, industrial permit requirements, sampling requirements, and enforcement actions. The pretreatment program includes the industrial permitting program and the fats, oils, and grease (FOG) program.The pretreatment program is currently staffed with one Pretreatment Coordinator and one Pretreatment Inspector. Pretreatment is discussed more in **Chapter 10**.

9.3.4 Wastewater Equipment

The City has several types of equipment for operations and maintenance procedures. An inventory list of the equipment in included in **Table 9-2**.

Table 9-2 – Existing Wastewater Utility Equipment

Quantity	Equipment
1	CCTV Inspection Truck
2	Vactor Truck
1	Pretreatment Truck
2	Service Truck (4WD)
3	Service Vehicle (2WD)
1	Jet Truck
3	Flow Monitoring Devices

9.3.5 Preventative Maintenance

Operating equipment includes all plant and pumping stations, and wastewater collection maintenance equipment. Each lift station should be visited once per week with a complete cleaning (wash-down) and lubrication of the facility mechanical systems as necessary. Electrical equipment should be tested once per week to establish operational conditions.

9.4 Performance Indicators

Performance ratings use measures of system performance to provide a quantitative basis for characterizing utility performance. Below are performance measures that can be used to help evaluate sewer collection system infrastructure performance:

- Pipe failures (in failures per mile per year)
- Flushing efforts (in feet of pipe flushed annually) and number of problem sections
- Customer complaints on collection system performance
- Lift Station failures
- Dollar amount in claims payout (annual basis)
- Root treatment efforts (in feet cleaned annually)

Additionally, the parameters described below can also be used to help evaluate collection system performance.

9.4.1 Collection System

By implementing and adhering to a preventative maintenance program, the need for reactive maintenance routines should decrease.

9.4.2 NPDES Permitting Requirements

The City's NPDES permit covers the WWTP and the collection system. The current permit (WA0020419) was issued June 17th, 2009 and on July 2nd, 2014 it was given an administrative extension, until further notice, due to limited DOE staffing. The City was sent a draft permit and fact sheet in October, 2015 for review and comment prior to finalizing. A copy of the permit, fact sheet, and letter of administrative extension are included in **Appendix O**.

The City's NPDES permit requires the City to perform sampling and submit annual reports. On a monthly basis, the parameters set forth in the permit monitoring schedule, which includes both WWTP influent and effluent flows, are tracked and reported to DOE by the Discharge Monitoring Report (DMR).

9.5 Current Operation and Maintenance Issues

9.5.1 Collection System

There are many locations in the collection system (approximately 91,000 LF) where 8-inch and 6-inch local collector or service pipes are cleaned on a routine basis (semi-annual to annual) due to FOG buildup, roots or other obstruction, aging and brittle pipe material (clay or concrete), or low or inconsistent slopes. Sewer service pipes with FOG buildup or low slopes do not produce adequate flushing velocities to transport wastewater solids. In some areas the aging and brittle pipe is rehabilitated, instead of being replaced, with a cast-in-place (CIP) liner.

There are also several locations in the collection system where homes with basements have a deeper service lateral. These service pipes are then directly connected, downstream, to the bottom invert of an interceptor pipe, instead of the top of the pipe. As the flow depth in the main rises, it causes surcharging in the local collector pipes and backups in the service lateral pipes and has the potential to lead to spills in homes with basements.

Issues with larger diameter interceptor pipes include dropped joints, groundwater infiltration at the pipe joints, and low slopes (<0.10%) which results in low flow velocities (notably the 42-inch Horn Rapids sewer trunk often has dark organic material floating on the wastewater surface giving it a "salad bar" appearance – flushing the trunk only temporarily relieves this issue).

Groundwater seepage into the collection system at manhole and pipe joints is also an ongoing issue. The City has developed an annual Manhole Rehabilitation Program that it funds through its annual renewals and replacements project list.

The City also has two sewer siphons (Leslie Rd Interceptor Inverted Siphon and Richland West Sewer Inverted Siphon) that convey flows from South Richland under the Yakima River and to the WWTP. Additional inspection and maintenance should be provided to these two structures since they provide the only method for draining South Richland, they cross the Lower Yakima River and any type of leak or spill would be a significant event, and they are uncased ductile iron (DI) pipe installed over 20 years ago. Record drawings show inlet and outlet structures on either side of the river crossing. These are weir structures that house isolation valves for each siphon barrel, although the current valve status (open or closed) for each pipe is not known. Design standards for sewer siphons note that these inlet and outlet structures are prime environments for hydrogen sulfide release, which results in an aggressive corrosion attack on concrete and iron materials. Therefore frequent inspection and any necessary corrosion control is important to maintain these facilities. A brief description of each siphon crossing is listed below.

9.5.1.1 Leslie Rd Interceptor Inverted Siphon

According to record drawings, the Leslie Rd Interceptor Inverted Siphon was constructed in 1979. It consists of three separate, uncased, ductile iron siphon barrels measuring 10-inch, 12-inch and 16-inch in diameter, with an equivalent diameter of 22.4”. Based on record drawings, the bury depth is approximately 5-feet below the river bottom and the siphon crossing has a length of 426-ft from inlet structure to outlet structure. This siphon conveys flows from the 30-inch trunk pipe along Columbia Park Trail which drains the central and eastern portions of South Richland (generally all of Richland south of Columbia Park Trail and east of Shockley Blvd). The siphon barrels have not been flushed or CCTV inspected and therefore the current condition of each barrel is not known and if there is any accumulation of heavy solids that might be affecting the overall siphon capacity. Although conducting routine maintenance on this existing siphon is difficult due to limited site access, large existing average flow (1.40 mgd) and submerged pipe conditions, developing a maintenance schedule is critical given the large drainage basin and because it is a river crossing. Any scheduled maintenance should take place between the hours of 2 am to 5 am when existing flows are at the lowest (0.60 mgd), according to the calibrated hydraulic model, and flow depths are roughly 6-inches.

9.5.1.2 Richland West Sewer Inverted Siphon

According to record drawings, the Richland West Sewer Inverted Siphon was constructed in 1994. It consists of three separate, uncased, ductile iron siphon barrels measuring 8-inch, 14-inch and 16-inch in diameter, with an equivalent diameter of 22.7”. Based on record drawings, the bury depth is approximately 5.5-feet below the river bottom and has a crossing length of 876-ft from inlet structure to outlet structure. The siphon conveys flows from the 24-inch trunk pipe along I-182 which drains the west portion of South Richland (generally west of Shockley Blvd). The siphon barrels are jetted twice per year but have not been CCTV inspected and therefore the current condition of each barrel is not known and if there is any accumulation of heavy solids that might be affecting the overall capacity. Although conducting routine maintenance on this existing siphon is difficult due to limited site access and submerged pipe conditions, the existing average flow (0.24 mgd) is not large and maintenance is critical given the large drainage basin and because it is a river crossing. Any scheduled maintenance should take place between the hours of 4 am to 6 am when existing flows are at the lowest (0.07 mgd) according to the calibrated hydraulic model, and flow depths are roughly 2-inches.

9.5.2 Lift Stations

Montana Lift Station. City crews previously noted the accumulation of sand and rock in the wet well and consistent ragging and clogging of both the pumps and check valves. Also noted was that the lift station pumps cycle frequently, up to 10 times per hour per pump, instead of a more typical value of 6. The lift station forcemain pipe also consists of both 8-inch AC pipe and 10-inch PVC pipe. The exact location and alignment of the 10-inch section is not known and was considered to be buried during the 2007 construction of the roundabout at Columbia Park Trail and Steptoe Street.

Wellhouse Loop Lift Station. City crews previously noted wet well turbulence while pumping leading to pump plugging and also ragging issues with this lift station.

9.5.3 WWTP

Current plant upgrades are discussed in **Section 5.1.2**.

Chapter 10

Pretreatment

Chapter 10 – **Pretreatment**

10.1 Program Overview and Components

The City's pretreatment program regulates the quality of wastewater discharged to the WWTP through the Richland Municipal Code (RMC) section 17.30, which lists general provisions, discharge requirements, industrial permit requirements, sampling requirements, and enforcement actions. The intent is to control the entry of pollutants into the waste stream where they could result in damage to the collection system and/or interfere with the biological treatment process. Pollutants also include trace contaminants, such as heavy metals and residual synthetic organic chemicals, which accumulate in the environment and concentrate in the food chain until reaching threshold levels which disrupt the ecological system.

The pretreatment program includes the industrial pretreatment program, the fats, oils, and grease (FOG) program, and the biosolids composting facility at the Horn Rapids Landfill. The pretreatment program is currently staffed with two FTE's, one Pretreatment Coordinator and one Pretreatment Inspector.

10.2 Industrial Pretreatment Program

The City was delegated directly by the EPA in 1984 which allows it to manage and issue industrial wastewater permits to businesses identified as a significant industrial users (SIU's). Since that time, the City has updated its industrial pretreatment program consistent with federal and state requirements and as listed in their NPDES permit. The following section describes this program.

10.2.1 Source Identification

To maintain a current database of all industries that discharge non-domestic waste, the City sends an industrial waste survey (IWS) with a copy of the current pretreatment standards to all new businesses. The goal of the IWS is to identify the volume and character of the pollutants discharged by the user. Depending on the type of business and information obtained, the pretreatment staff then determines if more information is needed (e.g., sampling laboratory or dental office where the specific type of wastes will be discharged). Restaurants and businesses providing food services (food service establishments or FSE's) receive a specific survey requesting information regarding grease traps, grease trap maintenance, and rendering service contracts. Industries identified as SIU's are required to complete an industrial permit application. The completed application is then reviewed by the pretreatment coordinator to determine if an industrial wastewater permit is required. In some cases a pretreatment inspection can be made to inspect the facilities and verify survey information or obtain additional information.

Once the survey process is complete, the information is then uploaded to the City's intranet database that contains all current business licenses. This information is valuable when diagnosing treatment problems at the WWTP and tracing the source. It also can be used to develop local limits for problem dischargers, to determine the sampling requirements (for both industrial user and the City) and to estimate manpower and equipment requirements. When completed, the IWS helps to then categorize businesses by one of the following user types listed in **Table 10-1**.

Table 10-1 – Industrial User Types

Category	Description	User Count [a]
Significant Industrial User (SIU)	Discharge a non-domestic waste stream of ≥25,000 gpd (0.025 mgd) or a non-domestic waste stream ≥5% of the average dry weather flow or organic capacity of the WWTP	10
Categorical Industrial User (CIU)	Subject to categorical standards as defined in 40 CFR Part 403.3(t)	1
Minor Industrial User (MIU)	Small industries with discharge flows that do not significantly impact the treatment system or contaminate the biosolids	792
Food Service Establishment (FSE)	Restaurants and businesses providing food services	164
Insignificant Industrial User (ISU) (also known as Zero Discharger)	Do not discharge to the sewer collection system or do not discharge non-domestic waste	2,655

(a) *Based on end of year 2014 records.*

In 2013 the business license database included 4,695 licensees located in the City. Any new businesses under construction or under a remodel are inspected by pretreatment personnel to ensure compliance with pretreatment standards. For FSE's this includes the installation of appropriate grease removal devices and interceptors. For SIU's and CIU's this includes appropriate pretreatment facilities and sampling equipment.

10.2.2 Industrial Permitting

The industrial wastewater permit has the following purpose: to prevent pass through or interference, protect the quality of the surface water receiving the WWTP's effluent, protect worker health and safety, facilitate sludge management and disposal, and protect against damage to the collection system and WWTP. The City's permitting process for industrial users follows the guidelines outlined in EPA's Industrial User Permitting Guidance Manual. The manual provides a framework for drafting and issuing industrial user permits. A brief description of the City's industrial permitting process is outlined in this section.

As previously described, a permit application is required to be completed by any new or existing SIU. The permit application contents include the following:

- Identifying information
- List of any existing permits
- Description of operations, including SIC code, raw chemicals and materials used, and a process diagram
- Flow measurement (average and maximum daily flow)
- Pollutant Measurements of representative samples of daily operations, including identifying applicable pretreatment standards for the wastewater discharge

- Compliance Schedule, where necessary to identify how an industry's operations and maintenance will be implemented to meet the City's pretreatment program
- Certification statement signed by an authorized representative that the pretreatment standards are being met

Once a completed permit application is submitted to the City it is reviewed for completeness and a pretreatment inspection of the facility is conducted. During this inspection, the permit application information can be evaluated for completeness and accuracy. The inspector can also verify the production processes, the presence of any toxic or hazardous waste, the identification of all waste streams, and the potential for spills and leaks. Following the inspection, the Public Works Director reviews all data furnished by the user and determines if a wastewater discharge permit shall be issued. During this time, a public announcement regarding the permit application is published in the local newspaper for a two week period. The announcement period is followed by a 30-day public comment period. The Public Works Director reviews all public comments and the pretreatment staff will respond generally or to specific comments as necessary. The Public Works Director then prepares a justification for the decisions made during the permit review process which are summarized in an industrial user Fact Sheet. The Fact Sheet describes the principal facts and policy decisions considered in preparing the industrial wastewater permit. The draft Fact Sheet is then sent to the applicant for review and to submit review comments as necessary. The WDOE will also receive a copy of the draft permit during the public review period.

Each permit is valid for a maximum of five years and is non-transferable without approval by the City. The permit also lists the required sampling and monitoring requirements, including submittal of technical reports and compliance schedules.

10.2.3 Enforcement

As part of the City's current NPDES permit, an enforcement response plan (ERP) was developed to provide consistent enforcement responses for similar violations and circumstances for all entities discharging non-domestic waste to the collection system and the WWTP. The ERP includes detailed procedures indicating how to respond to instances of industrial user noncompliance, a description of an escalating enforcement response, and time frames for enforcement responses. **Table 10-2** lists the descriptions of enforcement actions listed in the ERP.

Table 10-2 – Violation Enforcement Actions

Notice Type Category	Subcategory	Action
Informal Notice		
	Verbal Notification	By phone or in person to provide an immediate notification of violation. Typically used for minor instances
	Warning Letter	Typically as a follow-up to a verbal notification or in lieu of
	Informal Meeting	Used to gather information and discuss steps to alleviate noncompliance. Also used to determine the level of commitment by the industrial user [a]

Notice of Violation (NOV)		Written notice to the industrial user that a pretreatment violation has occurred. The NOV documents the legal authority, violation description, and date of the violation. Requires a response from the industrial user detailing the violation and corrective actions taken.
Administrative Order (AO)		Direct the industrial user to undertake and/or cease specified activities by specified deadlines. Terms of an AO may or may not be negotiated with the industrial user.
Show Cause Hearing		Formal meeting with the industrial user for explanation of noncompliance and to determine if more severe enforcement is required.
Termination of Service		Applied when the discharge from an industrial user presents imminent endangerment to the health or welfare or persons, or the environment, or as an escalating enforcement action due to failure to respond adequately to previous enforcement actions.
Administrative Fines		Punitive monetary fine assessed by the City to recover the economic benefit of noncompliance and deter future violations.
Civil Litigation		Formal process where the City files a lawsuit against the industrial user to secure court ordered action to correct violations and secure penalties for actions.

(a) *Any source that introduces pollutants in the collection system and WWTP from any non-domestic source.*

10.3 Grease Control Program

The disposal of grease into the City's wastewater collection system has caused problems in both the collection system and the WWTP. The regulatory community generally refers to the collection of fats, oils, and grease as "FOGs". "Grease" is the solid or semi-solid fraction of FOGs, and "oils" are the liquid fraction. Most of the FOGs that accumulate in the wastewater collection system are derived from food waste products. FOGs tends to coagulate and coat the walls of collection system pipes creating flow obstructions. Over time, clumps of grease slough off the pipe walls and can accumulate on the concrete shelf above manhole channels and in lift station wet wells. It also collects in both suction and discharge piping of lift stations, reducing pumping rates and increasing energy costs. Floating oils on the surface of wastewater can reduce oxygen transfer and lead to septic conditions which can produce odors and lead to corrosion of some pipe materials.

The City recently updated their RMC to strengthen the FOG removal requirements for FSE's. Restaurant management and owners must retain maintenance records for their grease removal devices and interceptors. The cause for any sewer back-up at a FSE must also be documented to identify a potential grease-related issue. Current RMC limit for the prohibited discharge of FOG is 100 parts per million (mg/L).

The pretreatment inspector conducts site visits for FSE's once every 5 years and as necessary. During these visits, the grease removal device or interceptor is checked to observe the current condition and remaining capacity and if the contents need cleaning. The cleaning frequency may need adjustment to meet City ordinance. A post-inspection form is then completed by the pretreatment inspector to summarize the inspection findings and is signed by the FSE owner or representative.

If necessary, CCTV inspection can be used to check the condition of the sewer service lateral pipe on the outlet side of the grease removal device. This may be required for those FSE's needing more intensive monitoring or if any complaints were received from the City's wastewater maintenance crews.

Education through public awareness on proper oil and grease disposal is provided to all sewer system customers. The City's residential customers are informed as to the effect of grease and oil on the sewer system. The informational material emphasizes the potential for coalescing grease and oil plugging the side sewer for which the property owner is responsible. It includes information on the use of garbage grinders. Ongoing education of the FSE's regarding the City's purpose of the grease control plan is also a valuable tool. Educational information includes:

- Copy of the City's pretreatment ordinance
- Grease trap/interceptor maintenance
- Garbage disposal and sewer service cleaning checklist
- Large laminated posters on FOG control for posting in their kitchens
- List of rendering and plumbing contractors
- Diagrams of how grease traps function

10.4 Biosolids Program

The biosolids composting facility was first permitted in April 2011 by WDOE and the BFHD. The facility is located at the City's Horn Rapids Landfill. The composting facility utilizes Class B biosolids (digested sludge thickened to about approximately 18% solids by belt filter press) produced at the WWTP in addition to ground yard waste and wood chips to produce Class A Exceptional Quality (EQ) biosolids as per EPA Regulation Part 503.

The composting operation to produce the Class A EQ biosolids consists of two phases, the active phase and the cured phase. During the active phase, biosolids at the compost facility are managed in a windrow arrangement and mechanically turned at least 5 times in a 15 day period to mix the compost feedstock. Water is also added, as necessary to maintain the correct moisture level for active composting. The windrow arrangement of biosolids are monitored for the required temperature and depth. Incoming Class B biosolids are monitored for nutrient content and weight and also are tested quarterly for pathogens, metals, inorganic and organic content and other regulatory tests. Following the active phase, the biosolids are moved to the curing section of the composting facility to allow to cure and fully compost for several months. Fully composted biosolids are then tested quarterly for metals, inorganics, nutrient amounts, pathogens and other regulatory tests to ensure quality and safety. The final Class A EQ biosolid is produced once the fully composted and tested biosolids are rescreened to remove larger material. The larger material is returned to the start of the composting process.

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Appendices