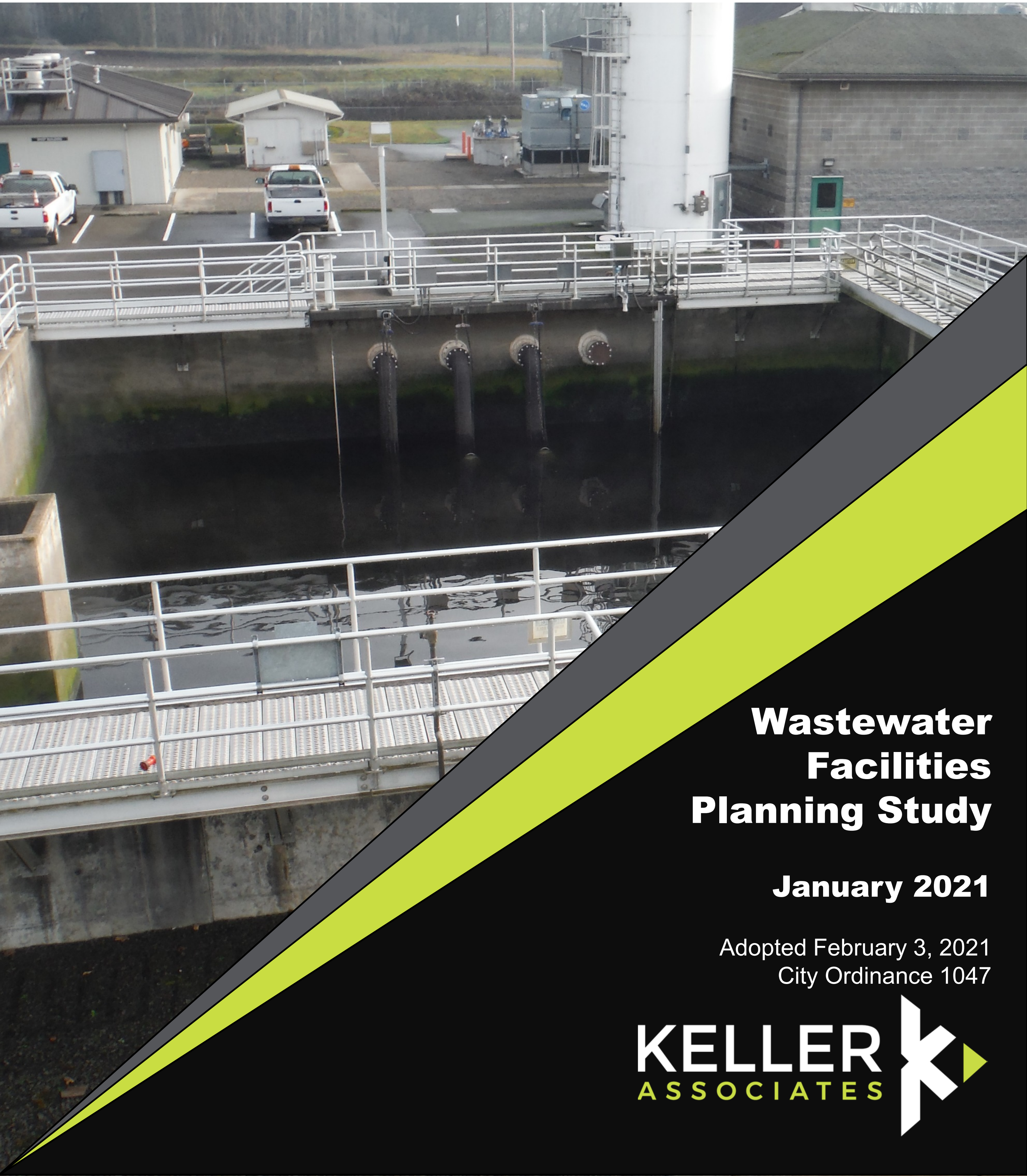
City of Stayton OREGON



Wastewater Facilities Planning Study

January 2021

Adopted February 3, 2021
City Ordinance 1047

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CITY OF STAYTON
WASTEWATER MASTER PLAN

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ACRONYMS, ABBREVIATIONS, AND SELECTED DEFINITIONS

AADF	average annual daily flow
ac	acre
AGS	aerobic granular sludge
ATS	automatic transfer switch
BID	business improvement district
BLM	Bureau of Land Management
BOD_5	5-day biochemical oxygen demand
BOR	Bureau of Reclamation
CCTV	closed circuit television
CDBG	community development block grants
CFR	Code of Federal Regulations
CIP	Capital Improvement Plan
CIPP	cured-in-place pipe
DEQ	Idaho Department of Environmental Quality
DMR	discharge monitoring report
DO	dissolved oxygen
EDU	equivalent dwelling unit
EPA	Environmental Protection Agency
FEMA	Federal Emergency Management Agency
fps	feet per second
ft	feet or foot
ft^2	feet squared or foot squared
ft^3	cubic feet or cubic foot
GIS	geographic information system
GPAD	gallons per acre per day
gpcd	gallons per capita per day
gpd	gallons per day
gpm	gallons per minute
HOA	hand/off/auto
HP	horsepower
hrs	hours
HRT	hydraulic retention time
IDAPA	Idaho Administrative Procedures Act
I/I	infiltration and inflow
in	inch
IPDES	Idaho Pollutant Discharge Elimination System
KW	kilowatt
kwh	kilowatt hour
LF	linear foot
LID	local improvement district
MBR	membrane bioreactor
MG	million gallons
MGD	million gallons per day
mg/L	milligrams per liter
mL	milliliter
MLSS	mixed liquor suspended solids

mm	millimeter
MMF	maximum month flow
MPN	most probable number
N	nitrogen
NFPA	National Fire Protection Association
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollution Discharge Elimination System
NPS	National Park Service
NTS	natural treatment system
NTU	Nephelometric turbidity units
O&M	operation and maintenance
OH&P	overhead and profit
PDF	peak day flow
PHF	peak hour flow
pH	Hydrogen ion concentration (measure of the acidity or basicity)
PLC	programmable logic controller
ppcd	pounds per capita per day
ppd	pounds per day
psi	pounds per square inch
PVC	polyvinyl chloride
RAS	return activated sludge
SBR	sequencing batch reactor
SCADA	supervisory control and data acquisition
SCFM	standard cubic feet per minute
sf	square feet or square foot
SRF	state revolving fund
SRT	solids retention time
SU	standard unit
TDH	total dynamic head
TKN	total Kjeldahl nitrogen
TMDL	total maximum daily load
TN	total nitrogen
TP	total phosphorus
TSS	total suspended solids
US	United States
USA	United States of America
USDA	US Department of Agriculture
USDA-RUS	US Department of Agriculture, Rural Utilities Services
USFS	United States Forest Service
USFWS	US Fish and Wildlife Service
USGS	US Geological Survey
UV	ultraviolet radiation
VFD	variable frequency drive
VSS	volatile suspended solids
WAS	waste activated sludge
WWTP	wastewater treatment plant

EXECUTIVE SUMMARY

In 2020, the City of Stayton, Oregon (City), contracted with Keller Associates, Inc. (Keller) to complete a wastewater facility planning study for the City's sanitary sewer collection system and wastewater treatment plant (WWTP). The study area consists of all areas within the City of Stayton Urban Growth Boundary (UGB). This section summarizes the major findings of the facilities plan, including brief discussions of alternatives considered and final recommendations.

1.1 PLANNING CRITERIA

City-defined goals and objectives, Public Works Design Standards (PWDS), engineering best practices, and regulatory requirements form the basis for planning and design. Applicable regulatory requirements include the National Pollutant Discharge Elimination System (NPDES) permit, Total Maximum Daily Loads (TMDLs), State Water Quality Standards, Recycled Water (Reuse) Regulations, and Land Use and Comprehensive Plan Requirements. Additional discussion of planning criteria is included in Sections 2.6 and 2.7.

1.2 PLANNING CONDITIONS

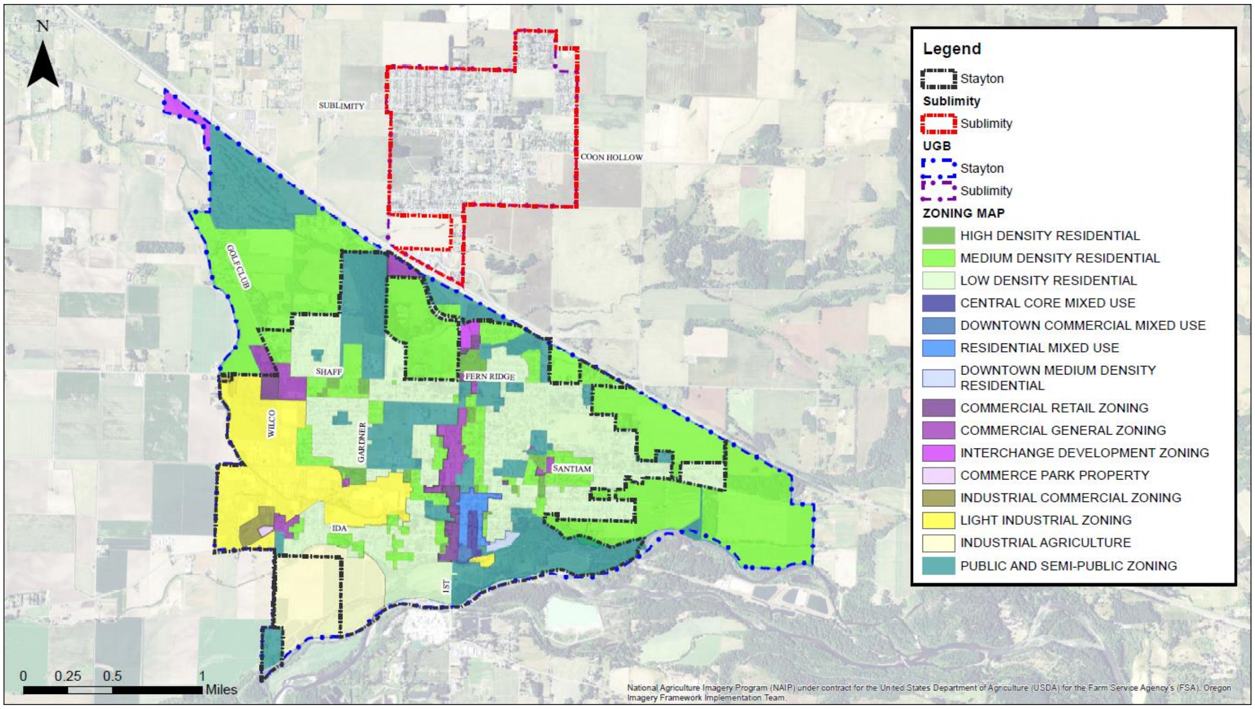
1.2.1 Study Area and Land Use

The study area, consisting of the City of Stayton urban growth boundary (UGB), is shown in Figure 1-1 on the following page. The study area slopes generally to the north toward Mill Creek on the north end of the City and to south toward the WWTP and eventually to the North Santiam River on the south side of the City. The City of Sublimity owns and operates a wastewater collection system within its UGB. The Sublimity collection system discharges to the City of Stayton's collection system and flows to the Stayton WWTP for treatment. Figure 1-1 shows the City of Sublimity's UGB for reference. Evaluation of the Sublimity system, aside from the impacts of population growth and infiltration and inflow (I/I) on the Stayton system, is not included in the scope of this study. Figures 2 through 6 in Appendix A present the topography, mapped floodplains, wetlands, and historic sites. Soil data for the study area are included in Appendix B. The wastewater system currently serves only areas within the Stayton and Sublimity UGBs. Further expansion of the UGB was not considered in this report. It is recommended that future development and capital improvements within the UGB provide adequate conveyance for the full build-out of the upstream sewer basins within the UGB.

1.2.2 Demographics

The City's population has been increasing at a steady rate over the past few decades. Historic populations for the City of Stayton and City of Sublimity were obtained from the U.S. Census and Marion County in cooperation with Portland State University (PSU). PSU analyzes historical trends and anticipates growth patterns to develop growth rates for 5-year increments. The most current population estimate provided by PSU for the combined area of Stayton and Sublimity was 10,840 in 2019. The PSU coordinated growth rates provide a population projection for 2040 of 12,697 (combined Stayton and Sublimity). These growth rates were reviewed and approved by the technical advisory committee for this planning study. Additional details about growth calculations can be found in Section 2.3. The overall estimated population growth rate from 2019 to 2040 is approximately 0.76% annually.

FIGURE 1-1: STUDY AREA



1.2.3 Wastewater Flows

Historical wastewater flows were evaluated to develop planning flows and provide flow projections for the planning period. Observed flows for each year from 2015–2019 and planning flows are summarized in Table 1-1 below.

TABLE 1-1: OBSERVED HISTORICAL FLOWS

	Historical Flows (MGD)					Planning Flow (MGD)
Year	2015	2016	2017	2018	2019	2019
Population	10,480	10,500	10,525	10,700	10,840	10,840
ADWF[1]	0.96	1.19	1.15	1.00	1.03	**1.07**
$MMDWF_{10}$	1.07	2.16	1.52	2.06	1.14	**1.92**
AADF[1]	1.51	1.72	1.78	1.38	1.32	**1.67**
AWWF[1]	2.07	2.26	2.43	1.78	1.93	**2.09**
$MMWWF_5$	4.02	4.09	3.22	2.55	2.46	**4.09**
PWkF	5.15	3.44	3.90	3.33	3.64	**5.15**
$PDAF_5$	6.70	4.28	4.97	4.27	4.64	**7.17**
PIF_5[2]	7.20	4.83	5.40	4.97	5.83	**8.35**
Total Rainfall (in/yr)	51	56	63	38	37	
Total Flow (MGY)	551	630	648	374	420	

[1] Spring 2018 and Summer 2019 data omitted from Planning Flow calculations because of inaccurate readings at WWTP.

[2] PIF5 flow was adjusted based on continuous flow data from peak days between 2015 and 2019.

To project the planning flows derived from the analysis, a projected flow per capita (reported in gallons per capita per day, gpcd) was developed. Projected planning flows (MGD) are based on

2019 planning flows with the addition of the product of projected unit flows (gpcd) and projected population increase (Table 1-2). Actual future flows will depend on several variables and could potentially be decreased through aggressive infiltration and inflow (I/I) reduction efforts.

TABLE 1-2: PROJECTED PLANNING FLOWS

	Planning Flow (MGD)	Planning Unit Flow (gpcd)	Projected Unit Flow (gpcd)[1]	Projected Planning Flow (MGD)				
Year	2019	2019	---	2020	2025	2030	2035	2040
Population	10,840	10,840	---	10,927	11,371	11,833	12,295	12,697
ADWF	**1.07**	98	98	1.07	1.12	1.16	1.21	1.25
$MMDWF_{10}$	**1.92**	177	177	1.94	2.01	2.10	2.18	2.25
AADF	**1.67**	154	154	1.68	1.75	1.82	1.89	1.96
AWWF	**2.09**	193	185	2.11	2.19	2.28	2.36	2.43
$MMWWF_5$	**4.09**	378	240	4.11	4.22	4.33	4.44	4.54
PWkF	**5.15**	475	285	5.17	5.30	5.43	5.56	5.67
$PDAF_5$	**7.17**	662	350	7.20	7.36	7.52	7.68	7.82
PIF_5[2]	**8.35**	770	450	8.38	8.58	8.79	9.00	9.18

1 Projected unit flow scaled down to reflect reduced I/I in future developments.

2 PIF5 flow calculated using continuous flow data from peak storm events between 2015 and 2019.

1.2.4 Wastewater Composition

The wastewater influent loading analysis followed a similar methodology used for the influent flows. Plant influent data from the DMRs for January 2015 through December 2019 was evaluated to evaluate dry weather (May 1 – October 31) and wet weather (November 1 – April 30) loads (pounds per day). The pounds per day loading data was used to calculate the pounds per capita per day (ppcd) for the corresponding populations; these values were used to estimate the 2040 design year loadings using the 2040 population of 12,697 (see Section 2 for further details).

1.3 COLLECTION SYSTEM EVALUATION

The wastewater collection system consists of approximately 36 miles of gravity sewer mains, three miles of force main, and four pump stations.

1.3.1 Pump Station Evaluation

There are four pump stations and approximately three miles of force main operated and maintained by the City in its wastewater collection system (Figure 10 in Appendix A). Pump stations are generally named by their locations in the City: Industrial, Mill Creek, Wilco, and Gardner. Onsite facility evaluations were completed in December 2019 and February 2020 with City operations personnel to review conditions of the pump station facilities, current maintenance activities, and known operational problems encountered by City staff.

Industrial and Wilco pump stations are both equipped with dry well pumps next to wet wells. Mill Creek pump station is equipped with submersible pumps. Gardner pump station was not evaluated as a part of the study as it will be taken offline within the planning period. Table 1-3 below provides a summary for the three pump stations evaluated. Appendix D includes pump curves for the three pump stations.

TABLE 1-3: PUMP STATION INVENTORY

	Industrial	Mill Creek	Wilco
PUMP STATION			
Type	Dry well, duplex pump station	Wet well, triplex pump station	Dry well, duplex pump station
Pump Type	Self-priming, non-clog centrifugal (Smith & Loveless 4B2Y)	Submersible, VFD (set for soft start), non-clog (Flygt NP 3202-090/640)	(Smith & Loveless 6C3A)
Capacity[1] (gpm)	Each pump: 150 gpm @ 21 ft TDH	Two pumps: 3,170 gpm @ 77 ft TDH; One pump: 2,220 gpm @68 ft TDH	Each pump: 800 gpmat 48 ft TDH
Pump (each)	2 hp @ 900 rpm (230 V, 60 Hz, 3 ph)	60 hp @ (460 V, 60 Hz, 3ph)	20 hp@ 1175 rpm (230 V, 3 ph)
Level Control Type	Air bubbler to be replaced with Ultrasonic	Pressure Transducer	Ultrasonic
Overflow Point	Influent MH	Influent MH	Influent MH
Overflow Discharge	Stormwater swale with drain	Stormdrain	Stormdrain
Auxiliary Power Type	Portable generator	Permanent diesel generator	Permanent diesel generator
Location	At WWTP	Onsite	Onsite
Output (kW)	85	150	80
Fuel Tank Capacity (gal)	170	196	100
Transfer Switch	Manual	Automatic	Automatic
Alarm Telemetry Type	Radio, operator call-out	Radio, operator call-out	Radio, operator call-out
Originally Constructed	1980's	2006	1975
Year Upgraded	2016 (pumps), 2020 (controls)	2016	2007 (electrical/controls)
Wet Well Diameter (ft)	6	12	8
Wet Well Net Storage (gal)	4,652	30,032	8,271
FORCE MAIN			
Length, Type	Approx. 525 ft. of 6-inch	Approx. 8715 ft. of 18-inch PVC and 20-inch HDPE	Approx. 75 ft. of 12-inch PVC
Profile, Continuously Ascending (Yes/No)	Yes	No	No
Discharge Location	Manhole at W Deschutes Drive and Willamette Avenue	Manhole on Jetters Way north of WWTP	Mill Creek discharge force main
Combination Air Release/Vaccuum Valves	No	Yes	No

[1]Capacity as reported in record drawings and O&M Manuals

This evaluation presents general observations and recommendations, along with specific recommendations for individual pump station sites. General recommendations are provided as a guideline to allow the City to maintain the lift stations for the 20-year planning period. Functionality and any items of concern observed during the onsite evaluation are noted in Section 3.2.

Overall, the Industrial, Mill Creek, and Wilco pump stations are in good condition. Deficiencies noted for Industrial and Mill Creek pump stations can be addressed through the recommended short-term improvements discussed in Section 6.

1.3.2 Pipeline Capacity Evaluation

A wastewater collection system model (InfoSWMM) was developed to evaluate existing and 20-year collection system capacity. Continuous flow monitoring was completed during the wet weather period between February and April 2020. The collected data was analyzed along with continuous precipitation data to establish typical 24-hour patterns, average flows at each site, and gauge rainfall influence in the system. Both dry weather and wet weather periods were used for loading and calibration efforts.

Gravity pipelines were evaluated according to City's 2015 Public Works Design Standards. Pipe size was determined by using one-half (1/2) of the maximum gravity flow capacity of the pipe for pipes 15 inches in diameter and less and two-thirds (2/3) for pipes larger than 15 inches in diameter. Sewage pump stations were evaluated based on the capacity to handle flows with the largest pump out of service (defined as firm capacity).

The calibrated model was used to assess the effects of a 5-year, 24-hour design storm event on the existing system. Figures 12a and 12b of Appendix A illustrate potential overflow sites and pipe capacity limitations.

For the 20-year capacity evaluation, future loads were distributed based on PSU population projections (Section 2) and City projected future residential, commercial, and industrial growth. Figures 14a and 14b in Appendix A illustrate the potential overflow sites and capacity limitations identified by the 20-year model analysis. Overall, problem areas identified in the 20-year evaluation reflect the same areas identified in the existing system analysis.

1.3.3 Collection System Improvement Alternatives

If a conveyance deficiency (identified in Sections 3 and 4) had one clear preferred solution, then the improvement is not discussed here, but is included in the Capital Improvement Plan (CIP) described in Section 6.

<u>Mill Creek Pump Station</u>

Two alternatives were identified to address the flow meter deficiency at Mill Creek: replace the flow meter vault with installation of a bypass pipeline and isolation valves or install a clamp-on ultrasonic flow meter within the existing flow meter vault.

<u>Conveyance</u>

While the conveyance system deficiencies discussed in Section 4 do not have multiple feasible alternatives, installation of parallel facilities or taking no action could be considered. The City could choose to construct parallel facilities in areas with limited remaining capacity. This alternative would increase the system's capacity and generally costs less than full replacements. Another advantage of constructing parallel facilities is that existing infrastructure could be left in service while the parallel facilities are constructed. The disadvantages of this alternative are the long-term increase in maintenance costs associated with maintaining parallel facilities and the potential higher life-cycle costs associated with the eventual replacement or rehabilitation of the original pipeline / pump station.

Taking no action is not a viable option because surcharging and the potential for overflows would only worsen. This could result in negative impacts to human health and the environment, in addition to fines from the DEQ.

1.3.4 Recommended Collection System Improvements

Recommendations and cost estimates are in Section 6 of this report.

Lift Stations

Short-term Priority 1 pump station improvements address existing deficiencies at the Mill Creek and Industrial pump stations. The total estimated cost for these improvements is $270,000, which includes a full replacement of the flow meter vault with bypass pipeline at Mill Creek pump station.

Long-term Priority 3 improvements assume that Gardner pump station is displaced with other CIP projects. The total estimated cost for these improvements is $486,000 and includes converting Industrial and Wilco pump stations from dry well to submersible pump stations at the end of the stations' useful life.

Pipelines

Priority 1 improvements address potential overflows near the downtown core of the City. Improvements include upsizing gravity mains on Jetters Way, W Ida Street, and N Evergreen Avenue.

Priority 2 improvement projects will alleviate remaining existing and future capacity limitations. Extension of the Mill Creek force main south on Jetters Way will address capacity issues in the gravity main on Jetters Way. Displacing the Gardner pump station and rerouting wastewater flows north to the Mill Creek trunk line, would alleviate capacity issues as well as long term operations and maintenance costs. Upsizing gravity mains on N Evergreen Avenue and W Ida Street, upstream of Priority 1 improvements, will address existing and future capacity issues.

Prioritization was evaluated as a part of the hydraulic capacity analysis. Results of the simulations indicate there is flexibility in how the City chooses to phase Priority 2 projects.

Operations and Maintenance

The costs associated with funding an on-going replacement and rehabilitation program are summarized in Section 6. Pipelines should be cleaned approximately every three to five years (frequency can be adjusted based on pipe material plus scour conditions and observations by City staff). Manhole rehabilitation and service line repairs should be coordinated with pipeline rehabilitation work. Emphasis should be placed on areas where pipe conditions pose the largest threat of sanitary sewer surcharging or a more immediate threat of collapse.

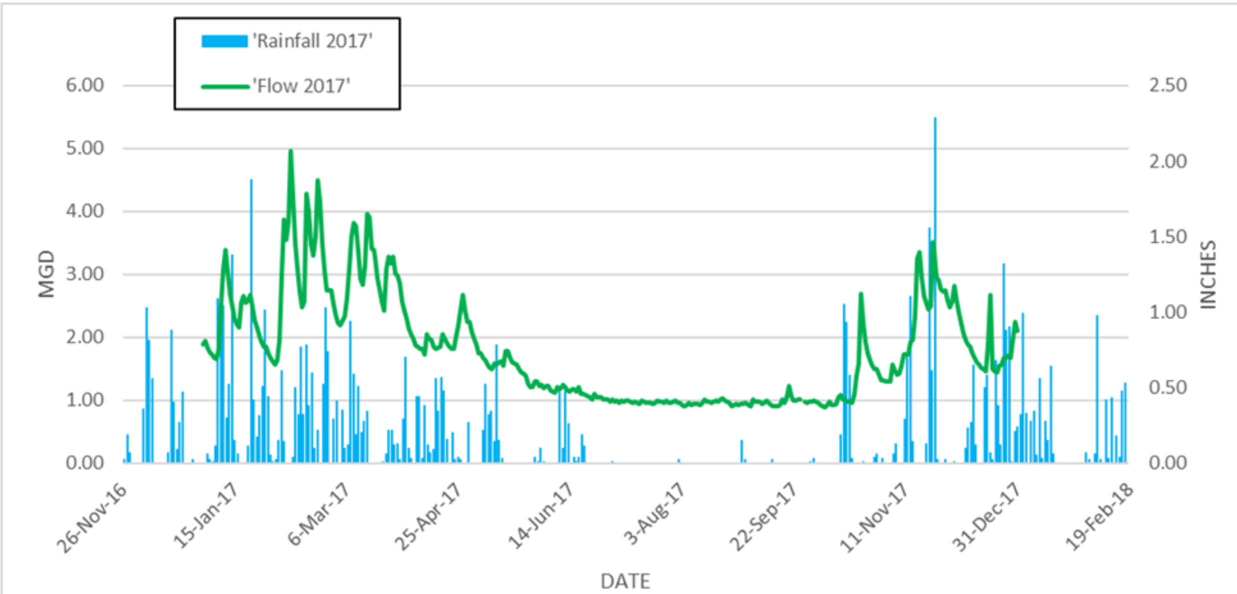
1.3.5 Infiltration & Inflow

I/I is a concern in the Stayton collection system. The rapid response between precipitation events and increased flows suggests that a significant component of peak flow is from storm water inflow. The sustained increase in flow over several days following a large storm event suggests that groundwater is also infiltrating into the City's wastewater collection system.

Recent sanitary sewer infiltration and inflow studies which included a pump run time analysis, extensive flow monitoring, CCTV inspections, night-time flow monitoring, and smoke testing to generate a prioritized list of the top 25 I/I reduction projects in the study area, as well as a list of cross connections found while smoke testing, and spot repair needs identified through CCTV inspections have confirmed the excessive I/I.

Pump run time analysis was completed at each of the four City-owned lift stations (Gardner, Industrial, Mill Creek, and Wilco). When daily run times are compared with rainfall events, a close correlation between high rainfall months and monthly increase in run times is evident (see Chart 1-1 below). This correlation indicates that I/I is the likely cause of the increase in flow. Continuous flow monitoring data was used to better characterize the nature and distribution of I/I in the system.

CHART 1-1: 2017 DAILY FLOW AND PRECIPITATION



Cleaning and CCTV inspection of the entire City pipeline has been incorporated in this master plan analysis. The National Association of Sewer Service Companies' (NASSCO) pipeline assessment certification program (PACP) was used again to record defects and grade pipe condition during CCTV inspections as a method of standardization.

Smoke testing was completed on approximately 18.9 miles of pipe. Smoke introduced into the sanitary system should only be released from nearby manholes, cleanout pick holes, and building plumbing vents; smoke emitted anywhere else indicates a potential source of I/I (See Figures 20-21, Appendix A).

Throughout the inspections, the most common operations and maintenance (O&M) defects found were infiltration, roots, intruding taps, and dirt or gravel in the pipe and laterals. The most frequent structural defects were cracks, fractures, and holes or breaks.

It is recommended that the City establish a routine cleaning schedule for cleaning of the collection system. After completing replacement or rehabilitation of pipes in the priority CIP areas or on the spot repairs list, it is recommended that the City re-inspect the pipes using CCTV. Additionally, continuous flow monitoring should continue to take place in the system and at the headworks of the wastewater treatment facility.

1.3.6 Recommended Infiltration & Inflow Improvements

It is recommended the City continue improvements on the system, broken into three categories: prioritized improvements for pipelines, spot repair/cross connection fixes, and development of an ongoing I/I reduction plan. Identifying, monitoring, and eliminating I/I is an ongoing and dynamic process.

Prioritized Improvements are detailed in Section 8 and in Figure 22 of Appendix A. In Table 8-1, 41 of the top 100 deficient pipe segments were considered by score and grouped by location to create logical rehabilitation projects for the City.

Some pipelines may be in relatively good condition but have one or two locations where there are severe defects. Rather than replace the entire pipeline reach, localized spot repairs may be more appropriate for these locations. A priority list for spot repairs was compiled into Table 8-2 of Section 8.

It is also recommended the City continue to identify and monitor sources of I/I system wide. Part of this ongoing process is continuous inspection, improvement, and progress tracking. It is recommended the City plan out routine CCTV inspections. The City should try to inspect 42,000 linear feet of pipe every year to complete the entire system on a 5-year rotation.

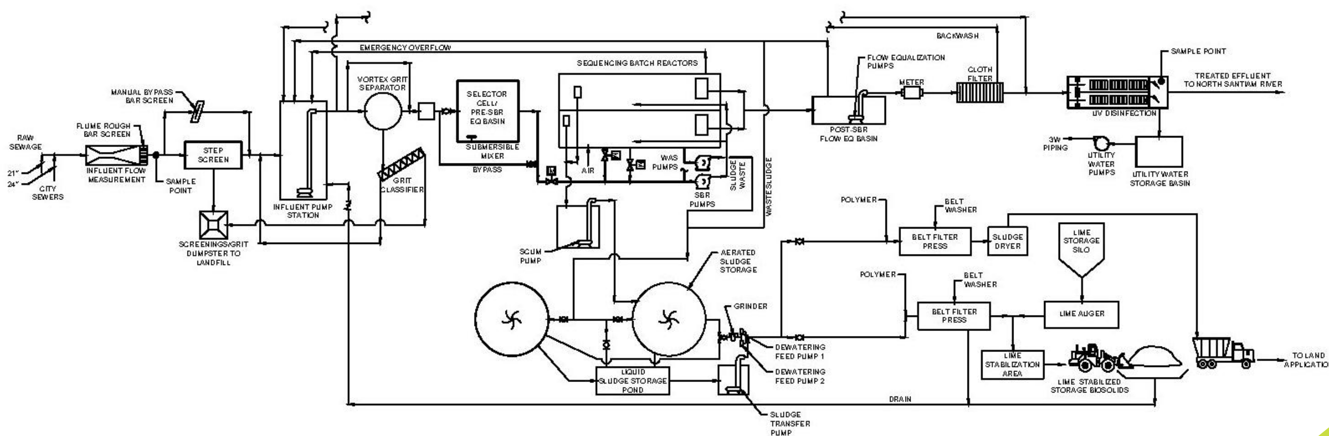
It is also recommended the City continues using the PACP format for future video inspections. The PACP format provides the City an industry standard, objective analysis and allows the condition of the same pipe to be compared over time. This could be helpful in tracking the deterioration of pipes, completing preventative maintenance activities, and identifying and correcting problems before a pipe fails.

1.4 WASTEWATER TREATMENT PLANT EVALUATION

1.4.1 Existing Facilities

The City owns and operates the Wastewater Treatment Plant (WWTP) located at 950 Jetters Way. Wastewater from the entire collection system, including the City of Sublimity, is collected and enters on the north side of the WWTP. Septage disposal is not allowed at the WWTP. The plant includes a sequencing batch reactor (SBR) process; operations building; headworks with Parshall flume, influent composite sampler, and step screen; influent pump station; vortex grit removal and grit washing/compacting; blower building; equalization basin; tertiary filters; UV disinfection; utility water system; and sludge drying. A simplified schematic process layout of the WWTP is shown in Figure 1-2.

FIGURE 1-2: EXISTING WWTP PROCESS SCHEMATIC



1.4.2 Capacity Assessment

To identify potential hydraulic and treatment capacity issues, each plant component was evaluated. The capacities are summarized in Table 1-4. Entries in red indicate process elements that are at or near to their individual capacities.

TABLE 1-4: PLANT CAPACITY SUMMARY (MGD)

Component	Governing Flow	Firm Capacity Provided	Current Capacity Needed	2040 Capacity Needed	Comments
Influent Screen	PIF_5	10.2	8.35	9.18	Bar screen redundancy – not a fine screen
Influent Pump Station	PIF_5	9.3	8.35	9.18	Room for future pumps
Grit Removal/Classifier	PIF_5	9.3	8.35	9.18	Performance may decrease above 5 MGD
SBR Basins	**$MMWWF_5$**	**4.1**	**4.09**	**4.54**	**Three Basin Rule limits capacity**
Post-SBR Equalization	**$PDAF_5$**	**7.2**	**7.17**	**7.82**	**Pump and basin capacity**
Filtration	75% of $PDAF_5$	6.0	5.38	5.87	Can add more disks to existing units
UV Disinfection	PIF_5	10.2	8.35	9.18	Redundancy bank in each channel

1.4.3 Recommended Treatment Plant Improvements

Recommended treatment plant alternatives are summarized below. Additional discussion on alternatives and recommendations is included in Section 11. If a WWTP deficiency (identified in Sections 9 and 10) had one clear preferred solution (such as installing an additional screen, purchasing critical spare pump motors, repairing the sludge storage pond, etc.), then the improvement is not discussed here, but is included in the Capital Improvement Plan (CIP) and individual project summary sheets found in Appendix D.

<u>Effluent Discharge</u>

The City of Stayton currently discharges treated effluent under the National Pollutant Discharge Elimination System (NPDES) Permit No. 101601 (see Appendix B) into the North Santiam River. Several different discharge alternatives were evaluated and discussed with the City in this facilities plan. The recommended alternative is Winter Influent Equalization followed by River Discharge.

<u>Post-SBR Equalization (EQ)</u>

The current Post-SBR EQ system is currently at capacity. If Winter Influent Equalization is utilized, the flow to the Post-SBR EQ system may remain unchanged; however, this is based on the SBR continuing to operate without issues. Due to either selecting a different discharge recommendation, or due to risks of SBR upsets, the City desired to evaluate different Post-SBR EQ alternatives. The recommended alternative is to add piping to combine the Selector Cell and the Post-SBR EQ basin.

Biosolids Drying

The City of Stayton currently provides their community with Class A EQ (exceptional quality) biosolids. However, the existing dryer has been challenging, requiring significant amount of expertise to operate and expensive emergency repairs to keep the dryer system running. Several different alternatives were evaluated and discussed with the City in this facilities plan. The City desires to continue to produce Class A EQ biosolids. Due to the high capital cost for a new dryer system, it is recommended to begin budgeting for a new dryer system to replace the existing. The selection of the type of new dryer should be made during the predesign phase after visiting installations and further discussions with operators; however, based on the evaluation performed during this planning study, the City's preference is a belt dryer due to its performance, safety, reliability, longevity, and controls.

1.5 CAPITAL IMPROVEMENT PLAN

1.5.1 Summary of Costs

The cost summary of the projects is listed in Table 1-5 (Capital Improvement Plan). Capital costs developed for the recommended improvements are Class 4 estimates as defined by the Association for the Advancement of Cost Engineering (AACE). The costs are based on experience with similar recent collection system and WWTP upgrade projects. Equipment pricing from manufactures of the large equipment items was also used to develop the estimates. The total estimated probable project costs include contractor markups and 30% contingencies, which is typical of a planning-level estimate. Overall project costs include total construction costs, costs for engineering design, construction management services, inspection, as well as administrative costs. For the collection system projects, the contractor's overhead and profit are worked into the line items. Priorities are set for today and will be re-evaluated when there is a need for re-assessment. The CIP is based on modeling data that was available during the completion of this facilities plan. When projects are carried forward, the model, data, assumptions, etc., should be re-evaluated to make any necessary adjustments to the basis of the project. An estimated schedule for the next six years is shown in Table 1-6 on page 1-11.

TABLE 1-5: SUMMARY OF COSTS (20-YEAR CIP)

ID#	Item	Primary Purpose(s)	Total Estimated Cost (2020)	SDC Growth Apportionment		City's Estimated Portion
				%	Cost	
Priority 1 Improvements						
1.1	Pipeline Upsizing on Jetters and Ida	Capacity	$ 2,943,000	6%	$ 170,213	$ 2,772,787
1.2	Short Term Pump Station Upgrades	Operations, Safety	$ 270,000	22%	$ 59,772	$ 210,228
1.3	Winter Equalization	Permit Compliance, Capacity, Operations	$ 12,050,000	14%	$ 1,687,000	$ 10,363,000
1.4	Influent Pump Control	Permit Compliance, Operations	$ 103,000	14%	$ 14,420	$ 88,580
1.5	Post-SBR Equalization	Permit Compliance, Capacity, Operations	$ 120,000	14%	$ 16,800	$ 103,200
1.6	Miscellaneous Parts	Redundancy, Operations	$ 202,000	14%	$ 28,280	$ 173,720
1.7	Turbo Blower Replacement	Operations	$ 990,000	14%	$ 138,600	$ 851,400
1.8	Misc. SBR Improvements	Operations	$ 167,000	14%	$ 23,380	$ 143,620
Total Priority 1 Improvements (rounded)			**$ 16,845,000**		**$ 2,139,000**	**$ 14,707,000**
Priority 2 Improvements						
2.1	Mill Creek Force Main Extension	Capacity	$ 1,190,000	22%	$ 263,442	$ 926,558
2.2	Gardner Pump Station Displacement	Capacity, Operations	$ 781,000	14%	$ 111,053	$ 669,947
2.3	Pipeline Upsizing on Evergreen	Capacity	$ 1,406,000	10%	$ 142,438	$ 1,263,562
2.4	Pipeline Upsizing on Ida	Capacity	$ 1,480,000	4%	$ 64,149	$ 1,415,851
2.5	Influent Screen	Redundancy, Operations	$ 466,000	14%	$ 65,240	$ 400,760
2.6	Dryer Replacement	Operations	$ 7,770,000	14%	$ 1,087,800	$ 6,682,200
2.7	Utility Water Storage	Operations	$ 1,160,000	14%	$ 162,400	$ 997,600
2.8	Generator	Operations	$ 1,050,000	14%	$ 147,000	$ 903,000
2.9	Sludge Storage Pond Repairs	Operations	$ 516,000	14%	$ 72,240	$ 443,760
Total Priority 2 Improvements (rounded)			**$ 15,820,000**		**$ 2,120,000**	**$ 13,710,000**
Priority 3 Improvements						
3.1	Long Term Pump Station Upgrades	Operations	$ 486,000	14%	$ 69,106	$ 416,894
Total Priority 3 Improvements (rounded)			**$ 490,000**		**$ 70,000**	**$ 420,000**
TOTAL WWTP AND COLLECTION SYSTEM IMPROVEMENTS COSTS (rounded)			**$ 33,155,000**			

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2020 dollars and does not include escalation to time of actual construction. Keller Associates has no control over variances in the cost of labor, materials, equipment, services provided by others, contractor's methods of determining prices, competitive bidding or market conditions, practices or bidding strategies. Keller Associates cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the cost presented herein.

TABLE 1-6: PRIORITY 1 CIP SCHEDULE

ID#	Item	Cost (2020)	Opinion of Probable Costs					
			2021	2022	2023	2024	2025	2026
Priority 1 Improvements								
1.1	Pipeline Upsizing on Jetters and Ida	$ 2,943,000	$ 371,401	$ 2,722,432				
1.2	Short Term Pump Station Upgrades	$ 270,000		$ 53,524	$ 237,468			
1.3	Winter Equalization	$ 12,050,000		$ 1,270,859	$ 4,332,368	$ 7,613,667		
1.4	Influent Pump Control	$ 103,000			$ 111,558			
1.5	Post-SBR Equalization	$ 120,000		$ 126,559				
1.6	Miscellaneous Parts	$ 202,000	$ 103,723	$ 106,520				
1.7	Turbo Blower Replacement	$ 990,000						$ 1,161,357
1.8	Misc. SBR Improvements	$ 167,000					$ 190,762	
Total (rounded)		**$ 16,845,000**	**$ 475,000**	**$ 4,280,000**	**$ 4,681,000**	**$ 7,614,000**	**$ 191,000**	**$ 1,161,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2020 dollars and includes a 2.7% annual escalation based on historic ENR data. Keller Associates has no control over variances in the cost of labor, materials, equipment, services provided by others, contractor's methods of determining prices, competitive bidding or market conditions, practices or bidding strategies. Keller Associates cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the cost presented herein.

SECTION 2 – PROJECT PLANNING

The City of Stayton owns and operates a municipal wastewater collection system and wastewater treatment plant (WWTP). The study's purpose is to determine the City's wastewater collection and treatment needs and provide a plan to implement improvements to meet the needs. This study describes the conditions, flows, and problems in the existing system and provides recommendations for improvements to the collection system and WWTP.

2.1 LOCATION

The study area illustrated in Figure 1 in Appendix A consists of all the areas within the City of Stayton's Urban Growth Boundary (UGB). The City of Sublimity owns and operates a wastewater collection system within its UGB. The Sublimity collection system discharges to the City of Stayton's collection system and flows to the Stayton WWTP for treatment. Figure 1 shows the City of Sublimity's UGB for reference. No evaluation of the Sublimity system, aside from the impacts of population growth and infiltration and inflow (I/I) on the Stayton system, are included in the scope of this study. Figure 2 (Appendix A) includes topographic contours for the study area.

2.2 ENVIRONMENTAL RESOURCES PRESENT

This section describes the existing environmental resources present in this area that might be impacted by wastewater facilities. The components analyzed in this section include land use, prime farmland, floodplains, wetlands, cultural resources, coastal resources, and socio-economic conditions. Discussion of environmental impacts of specific alternatives is covered later in the report.

2.2.1 Land Use / Prime Farmland

The City of Stayton zoning includes residential, commercial, industrial, and public zoning within the city limits. The City's comprehensive plan establishes future zoning categories for the area between the city limits and the UGB. A zoning map for the study area is in Figure 3 (Appendix A). Table 2-1 provides a detailed breakdown for each zoning and land use category.

TABLE 2-1: SUMMARY OF STAYTON LAND USE

	CITY ZONING		OUTSIDE CITY		TOTAL COMBINED	
Zone/Designation	Acres	% of Total	Acres	% of Total	Acres	% of Total
RESIDENTIAL	**975**	**49.81%**	**849**	**73.63%**	**1,824**	**58.64%**
High Density Residential	44	2.22%			44	1.40%
Medium Density Residential	229	11.70%			229	7.36%
Low Density Residential	702	35.88%			702	22.58%
DOWNTOWN	**43**	**2.18%**	**0**	**0%**	**43**	**1.37%**
Central Core Mixed Use	8	0.42%			8	0.27%
Downtown Commercial Mixed Use	5	0.24%			5	0.15%
Downtown Residential Mixed Use	23	1.16%			23	0.73%
Downtown Medium Density Residential	7	0.36%			7	0.23%
COMMERCIAL	**107**	**5.48%**	**25**	**2.17%**	**132**	**4.25%**
Commercial Retail	34	1.73%			34	1.09%
Commercial General	63	3.20%			63	2.01%
Interchange Development	8	0.42%			8	0.26%
Commerce Park	2	0.13%			2	0.08%
INDUSTRIAL	**402**	**20.56%**	**112**	**9.71%**	**514**	**16.54%**
Industrial Commercial	15	0.77%			15	0.48%
Light Industrial	320	16.36%			320	10.29%
Industrial /Agricultural	67	3.44%			67	2.16%
PUBLIC/SEMI PUBLIC	**430**	**19.20%**	**167**	**14.48%**	**597**	**19.20%**
Total Acreage	1,956	100%	1,153	100%	3,109	100.00%

Half of zoning within the city limits is residential (low, medium, and high density). Currently, 36% of the land within the City as Low Density Residential. Medium Density Residential zoning accounts for 12% of the City; while High Density Residential comprises only 2% of the City. Of the area within the Urban Growth Boundary, but not yet annexed, 74% of the land is designated as residential. Upon full annexation and build-out of the UGB, residential land will comprise 56% of the City. Industrial areas are concentrated in the southwest portion of the City. Some areas of public land use are expected northwest of the City.

2.2.2 Floodplains

Information on the floodplains in the study area is available from the Federal Emergency Management Agency (FEMA) Map Service Center. These maps show portions of the planning area lie within the 100-year floodplain adjacent to the floodway of the Santiam River and several other small drainages. Figure 4 of Appendix A shows the flood areas within the study area obtained from the FEMA website. This figure is for display purposes only. For specific projects in these areas, the individual FEMA Flood Insurance Rate Map (FIRM) Panels would be referenced.

2.2.3 Wetlands

Stayton completed a Local Wetlands Inventory (LWI) in 1998 that was accepted by DSL and is referenced in the 2013 Comprehensive Plan. In the 2013 Comprehensive plan the City takes inventory and map their wetlands to assess their functions in order to determine “Locally Significant Wetlands” that contribute to wildlife habitat, fish habitat, water quality, floodwater retention, recreational opportunities, and/or educational opportunities. Approximately 245 acres of wetlands were identified within the study area. The following descriptions from the 2013 Comprehensive Plan describe the different wetland types in the area (Figure 5, Appendix A):

• **Open water wetlands -** A wetland consisting of waters less then 6.6 feet in depth; submerged or floating plants may inhabit shallower areas.

• **Emergent wetlands -** Wetlands dominated by erect, rooted herbaceous plants that can tolerate flooded soil conditions, but cannot tolerate being submerged for extended periods, e.g. cattails, reeds, and pickerelweeds.

• **Scrub-shrub wetlands** - A wetland dominated by shrubs and woody plants less than 20 feet. Water levels can range from permanent to intermittent flooding.

• **Forested wetlands** - A wetland with soil that is saturated and often inundated, and is dominated by woody plants taller than 20 feet. Water-tolerant shrubs and herbaceous plants are often beneath the forest canopy.

• **Forest mosaic wetlands** – (no description given).

• **Emergent mosaic wetlands** – (no description given).

• **Filled wetlands** – (no description given).

Locally Significant Wetlands criteria developed by the Division of State Lands (DSL) were applied to the wetland units within the city; According to the Comprehensive Plan, 16 wetland units met the criteria and are considered Locally Significant Wetlands. In 2007 the Land Use Development Code was amended to regulate activities in and surrounding wetlands. The Code currently prohibits development activity, including fill, within the locally significant wetlands.

2.2.4 Historic Sites, Structures, and Landmarks

The National Register of Historic Places lists four historic sites for Stayton: Charles and Martha Brown House, Hobson-Gehlan General Merchandise Store, Deidrich Building, and the Beauchamp Building. All four of these locations have local significance. Furthermore, the 2013 Comprehensive Plan lists nine additional historic sites being of local historic significance. A map all sites can be found in Figure 6, Appendix A.

2.2.5 Biological Resources

The U.S. Fish and Wildlife Service (USFWS) produces a database that lists endangered and threatened plants throughout the country. A database search for the study area returned seven types of plants listed as endangered or threatened (Appendix B). The USFWS also provides lists of endangered/threatened species (see Appendix B for the January 30, 2020 summary).

The 2013 Stayton Comprenhensive Plan states that the North Santiam River, Mill Creek, Salem Ditch, and the Stayton Power Canal have been inventoried as significant to fish by Oregon Department of Fish and Wildlife. The North Santiam River has been identified as spawning habitat for Summer Steelhead, Spring Chinook, and Fall Chinook and migration habitat for Coho Salmon. Department of State Lands shows both Salem Ditch and Stayton Ditch as being Essential Salmonid Habitat.

2.2.6 Water Resources

The North Santiam River and Mill Creek flow through the study area. The WWTP outfalls to the North Santiam River. The 2006 Willamette Basin TMDL (total maximum daily load) includes the

North Santiam River Subbasin. The North Santiam River is 303(d) listed for temperature. Other parameters of concern in the larger Willamette River basin include mercury, bacteria, and dissolved oxygen. Additional discussion on water quality and regulatory requirements is in Section 2.7.

2.2.7 Coastal Resources

There are no coastal areas within the study area.

2.2.8 Socio-Economic Conditions

According to the City Economic Development Strategy and Action Plan, the City has experienced steady growth, but growth has slowed substantially during the past ten years. The median household income is $49,500, which is 14% less than the national average. The action plan states utility rates for the City are more expensive compared to those of similar neighboring cities for both industrial and retail businesses. Higher rates can be a challenge for economic growth.

All areas of the City have access to the City collection system, which delivers the City designated level of service to all users. Recommended improvements in this plan will help achieve the same level of service throughout the collection system for all users. The wastewater treatment plant does not impact one area of town more or less, therefore recommended improvements will benefit/impact all residents equally. City Council holds a public meeting to review and adopt the wastewater facilities planning study.

2.2.9 Climate, Geologic Hazards, and Soils

Climate

Stayton lies within the Willamette Valley, which has a relatively mild climate throughout the year, characterized by cool, wet winters and warm, dry summers. Table 2-2 summarizes the climate data for Stayton (National Oceanic and Atmosphere Administration (NOAA) Monthly Normals).

TABLE 2-2 CLIMATOLOGICAL DATA (1981-2010)

	Jan	Feb	Mar	Apr	May	June	July
Precipitation (in)	7.31	5.83	5.41	4.48	3.36	2.38	0.71
Mean Temp. (F)	40.7	12.8	16.9	50.6	56.1	61.2	66.7
Snowfall (in)	1.0	3.0	0.0	0.0	0.0	0.0	0.0
	Aug	**Sep**	**Oct**	**Nov**	**Dec**	**Average**	
Precipitation (in)	0.9	1.71	4.14	7.93	8.21	4.36	
Mean Temp. (F)	66.6	62	53.3	45.3	39.8	47.67	
Snowfall (in)	0.0	0.0	0.0	0.2	2.0	0.52	
*Snow values taken from Salem Mcnary Field, Or US due to no available data from Stayton							

Geologic Hazards

Potential geologic hazards in the Stayton area include landslides and earthquakes. There are no known volcanoes in this area to cause a volcanic hazard. According to the Geologically Hazardous Areas Overlay Zone map by Marion County, there are low-hazard of landslides in this area. Furthermore, the Oregon Department of Geology and Mineral Industries (DOGMI) lists Stayton in the low-to-moderate landslide susceptibility range. According to the Marion County Natural Hazards Mitigation Plan, Stayton is susceptible to low/intermediate earthquake hazards. However, the Marion County steering committee rated the probability of an earthquake occurring as high, meaning that it is likely Marion County will be affected by a damaging earthquake within a 10- to 35-year period. Appendix B includes hazard maps for landslides and seismic activity.

Soils

In general, the soils within the Stayton area are either silty clay loam or silty loam, and the slopes vary from zero to thirty percent, according to the NRCS website. Appendix B includes soil maps for the Stayton area.

2.2.10 Air Quality

Stayton lies within the Willamette Valley air shed. The valley is bordered on the east by Cascade Mountain Range and the west by the Coast Mountain Range. The valley is closed off on the north and south as the two ranges come together. The prevailing wind direction is from the southwest in the winter and from the north in the summer. Due to these geologic features, air pollution generated in the valley can become trapped. Air pollution sources include automobile emissions, field burning, slash burning, and other agricultural practices. The City does not lie within an EPA nonattainment area.

2.3 POPULATION TRENDS

The official population projections for the City of Stayton and City of Sublimity reflect the collaborative efforts of Marion County and Portland State University (PSU). These agencies published a document in June 2017, establishing the official coordinated population rates for all the cities in Marion County. The document is titled “Coordinated Population Forecast for Marion County, its Urban Growth Boundaries (UGB), and Area Outside UGBs 2017-2067”, and includes a summary of historical populations from the U.S. Census.

Table 2-3 presents the historical populations from the referenced document. Each year, PSU establishes a preliminary population estimate in November, which is sent to state and local jurisdictions and community partners. PSU then sends a certified population estimate in December. For this wastewater planning study, the base starting point for population projections was the 2019 certified population estimate. The PSU referenced document provided the future population estimates. At the end of the planning period, it is anticipated for the growth rates to decrease. The overall estimated population growth from 2019 to 2040 for the combined areas of Stayton and Sublimity (from 10,840 to 12,697) reflects an annual average growth rate of 0.76%.

TABLE 2-3: POPULATION HISTORY AND PROJECTIONS

Year	Stayton Population	Sublimity Population	Total Population	Source
1960	2,108	--	2,108	U.S Census, Population Research Center: Portland State University
1970	3,170	--	3,170	U.S Census, Population Research Center: Portland State University
1980	4,396	--	4,396	U.S Census, Population Research Center: Portland State University
1990	5,011	--	5,011	U.S Census, Population Research Center: Portland State University
2000	6,816	2,148	6,816	U.S Census, Population Research Center: Portland State University
2010	7,644	2,681	10,325	U.S Census, Population Research Center: Portland State University
2017	7,770	2,755	10,525	PSU Certified Population
2018	7,810	2,890	10,700	PSU Certified Population
2019	7,870	2,970	10,840	PSU Certified Population
2020	7,933	2,994	10,927	Projected Using Coordinated Growth Rate of 0.8% Stayton and 0.8% Sublimity
2025	8,255	3,115	11,371	Projected Using Coordinated Growth Rate of 0.8% Stayton and 0.8% Sublimity
2030	8,591	3,242	11,833	Projected Using Coordinated Growth Rate of 0.8% Stayton and 0.8% Sublimity
2035	8,931	3,364	12,295	Projected Using Coordinated Growth Rate of 0.7% Stayton and 0.5% Sublimity
2040	9,248	3,449	12,697	Projected Using Coordinated Growth Rate of 0.7% Stayton and 0.5% Sublimity

2.4 FLOWS

The wastewater flows analysis reviews historical wastewater flows and provides projected flows for the planning period. This section summarizes the results of the analysis. The City's projected flows were estimated using the methods recommended by the Oregon Department of Environmental Quality (DEQ) in "Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon." A few of the values developed from the DEQ methods were adjusted based on observed flow events at the WWTP. For months with inflated or incorrect influent flows due to maintenance activities at the WWTP, effluent data was substituted. Influent and effluent trends were compared to evaluate that effluent data was a suitable substitution.

Average Annual Daily Flow (AADF)

The average annual daily flow (AADF) is the average daily flow for the entire year. An AADF was calculated for each year of data. Years with a complete data set (2015– 2019) were averaged to obtain the AADF.

Average Dry-Weather Flow (ADWF)

The average dry-weather flow (ADWF) is the average daily flow for the period of May 1 through October 31. An ADWF was calculated for each year of data. Years with a complete data set (2015–2019) were averaged to obtain the ADWF.

Average Wet-Weather Flow (AWWF)

The average wet-weather flow (AWWF) is the average daily flow for the periods encompassing November 1 through April 30. An AWWF was calculated for each year of data. Years with a complete data set (2015– 2019) were averaged to obtain the AWWF.

Maximum Monthly Dry-Weather Flow ($MMDWF_{10}$)

The maximum monthly dry-weather flow ($MMDWF_{10}$) represents the month with the highest flow during the summer months. DEQ's method for calculating the $MMDWF_{10}$ is to graph the January through May monthly average flows for the most recent year against the total precipitation for each month. DEQ states that May is typically the maximum monthly flow for the dry-weather period (May through October). Selecting the May 90% precipitation exceedance most likely corresponds to the maximum monthly flow during the dry-weather period for a 10-year event. The May 90% precipitation exceedance value (4.88 inches for Stayton) is extrapolated from the NOAA Summary of Monthly Normals from 1981 to 2010.

Data from 2015–2019 was used according to the DEQ guidance to produce Chart 2-1. The data points from March-May 2018 were excluded as outliers because they do not follow the trend of the rest of the data set. Table 2-4 summarizes the data points illustrated in the chart.

Maximum Monthly Wet-Weather Flow ($MMWWF_5$)

The maximum monthly wet-weather flow ($MMWWF_5$) represents the highest monthly average during the winter period. DEQ's method for calculating the $MMWWF_5$ is to graph the January through May average daily flows against the monthly precipitation. DEQ states that January is typically the maximum monthly flow for wet weather (November through April). Selecting the January 80% precipitation exceedance value (9.93 inches for Stayton as obtained from the NOAA Summary of Monthly Normals) most likely corresponds to the maximum monthly flow during the wet-weather period for a 5-year event. The DEQ method and $MMWWF_5$ result are illustrated in Chart 2-1 and summarized in Table 2-4.

CHART 2-1: MONTHLY AVERAGE FLOW VS. RAINFALL ($MMDWF_{10}$ AND $MMWWF_5$)

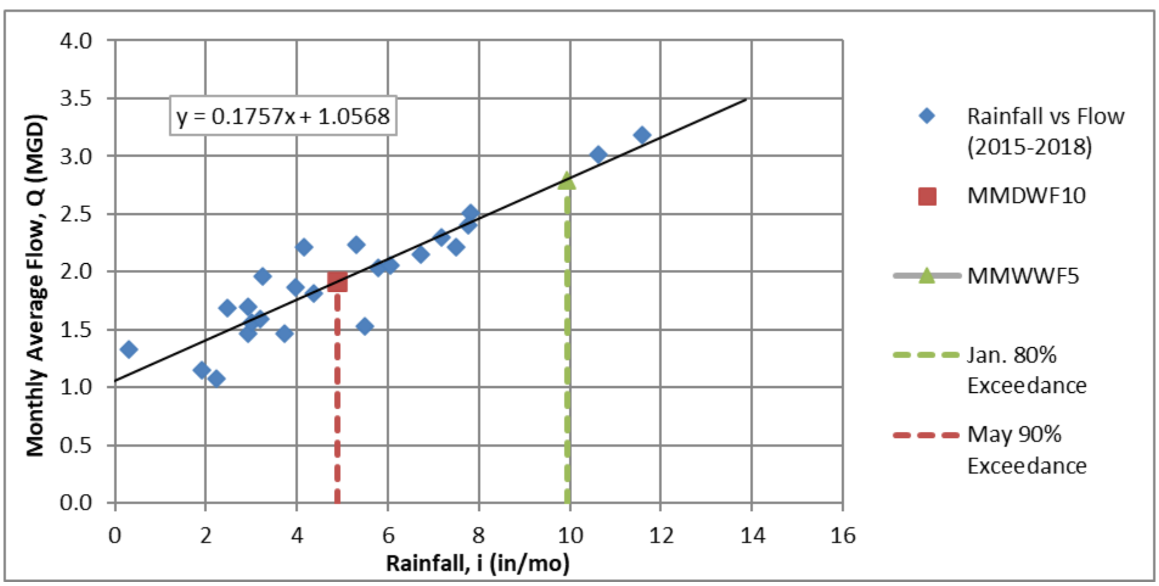


TABLE 2-4: MONTHLY AVERAGE FLOW VS. RAINFALL ($MMDWF_{10}$ AND $MMWWF_5$)

Month	Monthly Average Flow (MGD)					Rainfall (in/mo)				
	2015	2016	2017	2018	2019	2015	2016	2017	2018	2019
January	1.96	2.50	2.23	2.30	1.60	3.26	7.82	5.30	7.18	0.31
February	1.87	2.21	3.19	1.69	2.22	3.97	4.17	11.60	2.48	3.20
March	1.53	2.40	3.01	2.91	1.70	5.49	7.76	10.64	4.36	6.72
April	1.47	1.46	2.03	4.22	2.16	2.93	3.72	5.78	6.05	1.92
May	1.08	1.17	1.56	2.74	1.15	2.24	1.28	3.01	0.31	1.92
MMDWF10	1.92 MGD					4.88 in/mo				
MMWWF5	2.80 MGD					9.93 in/mo				

To confirm the validity of the DEQ method, a 30-day rolling average of the available flow data (January 1, 2015, through December 31, 2019) was evaluated. The maximum observed 30-day rolling average flow was 4.09 MGD. This average flow occurred from December 7, 2015 through January 2, 2016. An $MMWWF_5$ of 4.09 MGD was used because this observed flow was higher than the DEQ estimated flow.

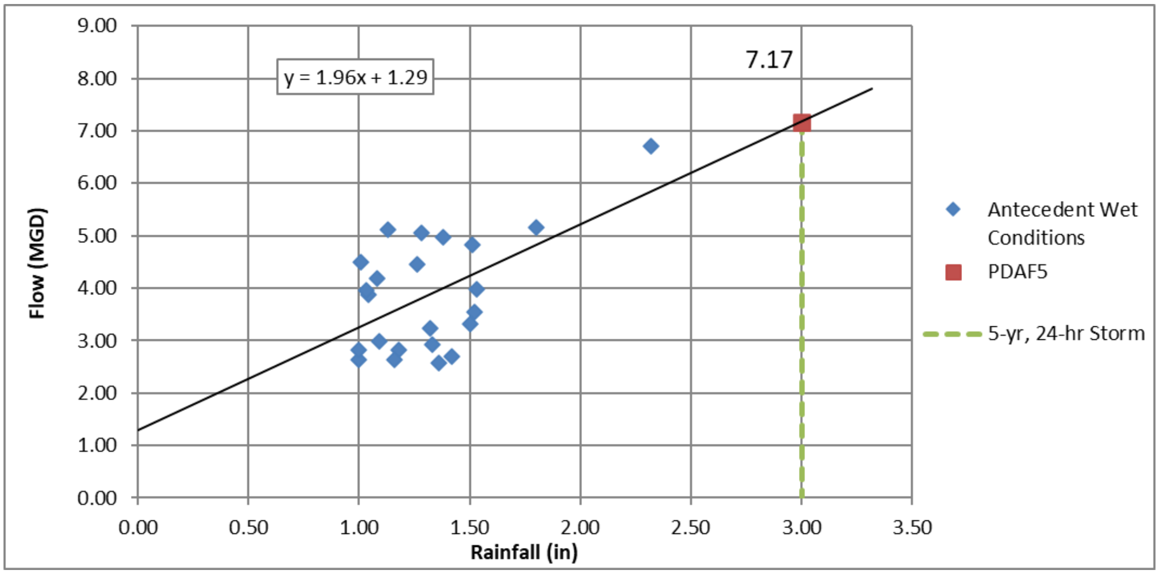
Peak Week Flow (PWkF)

The PWkF was calculated using a 7-day rolling average for each year. The maximum of all the year PWkF values was used as the PWkF.

Peak Daily Average Flow ($PDAF_5$)

As outlined by the DEQ, the peak daily average flow ($PDAF_5$) corresponds to a 5-year storm event. The DEQ's method for determining $PDAF_5$ is plotting daily plant flow against daily precipitation for significant storm events, using data only for wet-weather seasons when groundwater is high. The $PDAF_5$ is the 5-year, 24-hour storm event (3.0 inches per the NOAA isopluvial maps for Oregon) from a trend line fitted to the data. A significant storm event was considered more than 1-inch of rainfall in 24-hours. Antecedent conditions were evaluated on a case-by-case basis, and wet conditions were assumed if any day in the preceding three had a storm event of 0.5-inches or larger. Data was also considered based on cumulative rainfall for 30 days before the storm event. The cutoff for 30-day cumulative rainfall (for purposes of this analysis) was 5.5-inches. Chart 2-2 below shows the results of the DEQ analysis.

CHART 2-2: FLOW VS. RAINFALL ($PDAF_5$)



In analyzing the data, peak flows at the WWTP occurred on the same day or the following day as the storm. The $PDAF_5$ developed using DEQ's method was compared with the top five peak day flow events from 2015-2019 (see Table 2-5 below). The $PDAF_5$ developed using DEQ's method was selected as the value to use during this planning study because it estimates a higher $PDAF_5$ than the top five observed flow events.

TABLE 2-5: TOP FIVE FLOW EVENTS

Date	DMR Flow (MGD)	Rain (in/d)	Peak Inst. Flow (MGD)	60 day rainfall (in)
December 18, 2015	6.70	2.32	7.20	24.28
December 19, 2015	5.43	0.17	5.76	24.21
December 8, 2015	5.29	0.66	5.40	17.14
February 9, 2017	4.97	1.38	5.40	14.10
February 16, 2017	4.29	1.88	4.75	15.13

Peak Instantaneous Flow (PIF_5)

The peak instantaneous flow (PIF_5) represents the peak flow recorded at the WWTP. The DEQ recommends evaluating hourly or instantaneous flow data for high-flow days if available. The peaking factor (peak instantaneous to average daily ratio) will be less during heavy flows than during normal flowrates because of infiltration influence from high groundwater. The City provided continuous flow data for high-flow days in the last five years to evaluate this peaking factor. The average peaking factor was 1.16 during these high-flow events (data summarized in Appendix C). Using a peaking factor of 1.16 and the $PDAF_5$, a PIF_5 of 8.35 MGD was selected.

Infiltration and Inflow (I/I)

I/I is an issue in the collection system, and results in the high peak flows experienced at the WWTP during wet weather (Appendix C). The City has been working to characterize and evaluate I/I throughout the collection system. The I/I work completed previously, and for this study, is discussed in Section 7. The City's ongoing efforts to reduce I/I in its collection system will reduce flows to the treatment plant.

Observed Historical Flows and Projected Design Flows

Table 2-6 summarizes the observed flows for each year from 2015-2019. The historical flows were derived as described in the preceding paragraphs, with the exception of the PIF_5. The peak instantaneous flow used the $PDAF_5$ and the peaking factor of 1.16.

TABLE 2-6: OBSERVED HISTORICAL FLOWS

	Historical Flows (MGD)					Planning Flow (MGD)
Year	2015	2016	2017	2018	2019	2019
Population	10,480	10,500	10,525	10,700	10,840	10,840
ADWF[1]	0.96	1.19	1.15	1.00	1.03	**1.07**
$MMDWF_{10}$	1.07	2.16	1.52	2.06	1.14	**1.92**
AADF[1]	1.51	1.72	1.78	1.38	1.32	**1.67**
AWWF[1]	2.07	2.26	2.43	1.78	1.93	**2.09**
$MMWWF_5$	4.02	4.09	3.22	2.55	2.46	**4.09**
PWkF	5.15	3.44	3.90	3.33	3.64	**5.15**
$PDAF_5$	6.70	4.28	4.97	4.27	4.64	**7.17**
PIF_5 [2]	7.20	4.83	5.40	4.97	5.83	**8.35**
Total Rainfall (in/yr)	51	56	63	38	37	
Total Flow (MGY)	551	630	648	374	420	

[1] Spring 2018 and Summer 2019 data omitted from Planning Flow calculations because of inaccurate readings at WWTP.

[2] PIF5 flow was adjusted based on continuous flow data from peak days between 2015 and 2019.

To project the planning flows to future populations, a projected flow per capita (reported in gallons per capita per day, gpcd) was developed. As shown in Table 2-7, the per capita per day flows are different for existing for projected growth. This method recognizes the existing effects of I/I on the current system, and the assumed reduced I/I influence on wet-weather flows in the future as better construction methods and materials are utilized. Table 2-7 summarizes the projected planning flows. Actual future flows will depend on several factors and could potentially decrease through aggressive I/I reduction efforts. It is recommended that flows be reviewed periodically and future capital projects phased where practical. It is also recommended that the City consider updating the Public Works Design Standards wastewater peak factors (Section 503.02.D) to be consistent with the design flows presented in Table 2-6.

TABLE 2-7: PROJECTED PLANNING FLOWS

	Planning Flow (MGD)	Planning Unit Flow (gpcd)	Projected Unit Flow (gpcd)[1]	Projected Planning Flow (MGD)				
Year	2019	2019	---	2020	2025	2030	2035	2040
Population	10,840	10,840	---	10,927	11,371	11,833	12,295	12,697
ADWF	**1.07**	98	98	1.07	1.12	1.16	1.21	1.25
$MMDWF_{10}$	**1.92**	177	177	1.94	2.01	2.10	2.18	2.25
AADF	**1.67**	154	154	1.68	1.75	1.82	1.89	1.96
AWWF	**2.09**	193	185	2.11	2.19	2.28	2.36	2.43
$MMWWF_5$	**4.09**	378	240	4.11	4.22	4.33	4.44	4.54
PWkF	**5.15**	475	285	5.17	5.30	5.43	5.56	5.67
$PDAF_5$	**7.17**	662	350	7.20	7.36	7.52	7.68	7.82
PIF_5[2]	**8.35**	770	450	8.38	8.58	8.79	9.00	9.18

1 Projected unit flow scaled down to reflect reduced I/I in future developments.

2 PIF5 flow calculated using continuous flow data from peak storm events between 2015 and 2019.

Future Flow Projections & Model Scenarios

See Section 4.2 for information on future flow projections and the model evaluation of future system expansion.

2.5 LOADINGS

The wastewater influent loading analysis follows a similar methodology used for the influent flows. The historical wastewater loading data was used to develop future loading projections for the planning period. This section summarizes the results of the carbonaceous 5-day biochemical oxygen demand ($cBOD_5$), 5-day biochemical oxygen demand (BOD_5), and total suspended solids (TSS) load analysis. Dry weather (May 1 – October 31) and wet weather (November 1 – April 30) loads were evaluated. The following definitions summarize the terminology of the loading conditions:

Average Daily Load (ADL)

The average daily load (ADL) was calculated for both dry weather (DWADL) and wet weather (WWADL) for each year of data. Data from 2015-2019 were averaged to obtain the ADLs.

Maximum Month Load (MML)

The maximum month load (MMDL) was calculated for both dry weather (DWMML) and wet weather (WWMML) for each year of data. Data from 2015-2019 were averaged to obtain the MMDLs. The maximum month data is from the discharge monitoring reports (DMRs) and represents the samples taken during the month rather than a 30-day rolling average.

Maximum Daily Average Load (MDL)

The maximum daily load (MDL) was calculated for both dry weather (DWMDF) and wet weather (WWMDF) for each year of data. Data from 2015-2019 were averaged to obtain the MDLs.

2.5.1 Observed Historical and Projected cBOD, BOD and TSS Loadings

The $cBOD_5$, BOD_5 and TSS loadings (ppd) are summarized in Table 2-8 and Table 2-9, respectively.

TABLE 2-8: OBSERVED HISTORICAL CBOD5 AND BOD LOADING

Parameter	2015	2016	2017	2018	2019	Avg.	Max.
Population	*10,480*	*10,500*	*10,525*	*10,700*	*10,840*	--	--
BOD_5 and $CBOD_5$ ppd							
DWADL ($CBOD_5$)	1,691	1,519	1,498	1,604	1,484	1,559	1,691
DWMMDL ($CBOD_5$)	1,892	1,618	1,796	2,619	1,633	1,912	2,619
DWMDL ($CBOD_5$)	2,406	2,988	3,063	4,324	2,025	2,961	4,324
WWADL (BOD_5)	1,791	1,616	1,547	2,608	2,432	1,999	2,608
WWMMDL (BOD_5)	2,021	1,887	1,884	3,453	2,763	2,401	3,453
WWMDL (BOD_5)	2,553	2,466	2,898	5,626	3,635	3,436	5,626

TABLE 2-9: OBSERVED HISTORICAL TSS LOADING

Parameter	2015	2016	2017	2018	2019	Avg.	Max.
Population	*10,480*	*10,500*	*10,525*	*10,700*	*10,840*	--	--
TSS ppd							
DWADL	2,782	2,483	2,919	3,265	3,247	2,939	3,265
DWMMDL	3,336	3,210	4,062	4,888	3,957	3,891	4,888
DWMDL	5,125	5,436	8,699	8,292	6,176	6,746	8,699
WWADL	2,494	2,187	2,476	3,226	2,824	2,641	3,226
WWMMDL	3,383	2,524	3,183	3,777	3,076	3,189	3,777
WWMDL	6,491	4,001	7,688	6,630	5,348	6,031	7,688

Unit loadings in pound per capita per day (ppcd) were calculated for each year of data analyzed. Projected unit loadings are the maximum of the individual 2015 to 2019 unit loads. It is assumed the unit loadings do not change during the planning period due to decreases in I/I or additional industrial loadings. The projected loads in pounds per day are the product of the projected unit load (ppcd) and the population. Projected $cBOD_5$, BOD_5, and TSS loads are summarized in Table 2-10 and Table 2-11, respectively.

TABLE 2-10: PROJECTED $CBOD_5$ AND BOD_5 LOAD

Parameter	Planning Criteria (ppcd)	Loading Projections (ppd)				
		2020	2025	2030	2035	2040
Projected Population		*10,927*	*11,371*	*11,833*	*12,295*	*12,697*
BOD_5 and $CBOD_5$						
DWADL ($CBOD_5$)	**0.161**	1,764	1,835	1,910	1,984	2,049
DWMMDL ($CBOD_5$)	**0.245**	2,674	2,783	2,896	3,009	3,107
DWMDL ($CBOD_5$)	**0.404**	4,415	4,595	4,781	4,968	5,131
WWADL (BOD_5)	**0.244**	2,663	2,771	2,884	2,996	3,094
WWMMDL (BOD_5)	**0.323**	3,526	3,670	3,819	3,968	4,097
WWMDL (BOD_5)	**0.526**	5,745	5,979	6,222	6,464	6,676

TABLE 2-11: PROJECTED TSS LOAD

Parameter	Planning Criteria (ppcd)	Loading Projections (ppd)				
		2020	2025	2030	2035	2040
Projected Population		*10,927*	*11,371*	*11,833*	*12,295*	*12,697*
TSS						
DWADL	**0.305**	3,334	3,469	3,610	3,751	3,874
DWMMDL	**0.457**	4,992	5,195	5,406	5,617	5,801
DWMDL	**0.827**	9,032	9,399	9,781	10,162	10,495
WWADL	**0.302**	3,295	3,429	3,568	3,707	3,828
WWMMDL	**0.353**	3,858	4,014	4,177	4,340	4,482
WWMDL	**0.730**	7,981	8,306	8,643	8,980	9,274

2.6 PLANNING CRITERIA

2.6.1 Collection System

The City's conveyance system will be sized for the projected 20-year peak instantaneous flow rates associated with the 5-year, 24-hour storm event. Projected growth areas, as discussed in section 4 , will be as shown in Figure 7 (Appendix A). Where appropriate, new lines will be sized one nominal pipe size larger than needed for areas that may not be at buildout by the end of the planning period. Additionally, it should be noted, efforts to reduce I/I in the collection system could further extend the service population. When sizing gravity collection systems, pipelines will be sized according to City's 2015 Public Works Design Standards. Pipe size shall be determined by using one-half (1/2) of the maximum gravity flow capacity of the pipe for pipes 15 inches in diameter and less, and shall be two-thirds (2/3) for pipes larger than 15 inches in diameter. Sewage pump stations will be designed to handle these flows with the largest pump out of service (defined as firm capacity).

The evaluations performed as part of this planning study are used to prioritize recommended improvements to address deficiencies in the collection system. These improvements are organized into the Capital Improvement Plan (CIP) and included in the System Development Charge (SDC) evaluation. For the collection system model evaluation, pipe surcharging was not allowed.

2.6.2 Wastewater Treatment Plant

The future WWTP influent flows and loading were developed using 2015 to 2019 historical data and population forecasts described above. A summary of the planning conditions for the 20-year planning period is listed in Table 2-12.

TABLE 2-12: WWTP LOADING PROJECTIONS FOR 2040

Parameter	Planning Criteria (ppcd)	Loading Projections (ppd)				
		2020	2025	2030	2035	2040
Projected Population		*10,927*	*11,371*	*11,833*	*12,295*	*12,697*
BOD_5 and $CBOD_5$						
DWADL ($CBOD_5$)	**0.161**	1,764	1,835	1,910	1,984	2,049
DWMMDL ($CBOD_5$)	**0.245**	2,674	2,783	2,896	3,009	3,107
DWMDL ($CBOD_5$)	**0.404**	4,415	4,595	4,781	4,968	5,131
WWADL (BOD_5)	**0.244**	2,663	2,771	2,884	2,996	3,094
WWMMDL (BOD_5)	**0.323**	3,526	3,670	3,819	3,968	4,097
WWMDL (BOD_5)	**0.526**	5,745	5,979	6,222	6,464	6,676
TSS						
DWADL	**0.305**	3,334	3,469	3,610	3,751	3,874
DWMMDL	**0.457**	4,992	5,195	5,406	5,617	5,801
DWMDL	**0.827**	9,032	9,399	9,781	10,162	10,495
WWADL	**0.302**	3,295	3,429	3,568	3,707	3,828
WWMMDL	**0.353**	3,858	4,014	4,177	4,340	4,482
WWMDL	**0.730**	7,981	8,306	8,643	8,980	9,274

2.7 REGULATORY REQUIREMENTS

Regulations, existing constraints, and water quality impacts directly affect the requirements for wastewater infrastructure, as discussed below.

2.7.1 Collection System

Pump Station Regulatory Requirements

Pump stations lift wastewater and convey it to a discharge point. Pump stations must meet the DEQ's requirements, such as the following:

- Redundant Pumping Capacity – The DEQ design criteria requires the pump station firm capacity to be capable of conveying the larger of the 10-year dry-weather or 5-year wet-weather event. For Stayton, due to the I/I, this means that the pump stations must pump the 5-year, 24-hour storm event peak instantaneous flows with the largest pump out of service.

- Hydrogen Sulfide Control – Hydrogen sulfide can be corrosive (especially to concrete materials) and lead to odor problems. Where septic conditions may occur, provisions for addressing hydrogen sulfide should be in place.

- Alarms – The alarm system should include high level, overflow, power, and pump fail conditions. The DEQ also requires an alarm condition when all pumps are called on (loss of redundancy alarm) to keep up with inflow into the pump station.

- Standby Power – Standby power is required for every pump station because extended power outages may lead to wastewater backing up into homes and sanitary sewer overflows. Ideally, a dedicated gen-set, with automatic transfer switch, is located at each pump station to meet redundancy requirements. However, mobile generators or portable trash pumps may be acceptable for some pump stations, depending on the risk of overflow, available storage in the wet well and pipelines, alarms, and response time.

- The DEQ has also established guidelines for wet well volumes, overflows, maximum force main velocities, and location/elevation relative to mapped floodplains.

Pipeline Regulatory Rules (CMOM Guidance)

CMOM refers to Capacity Management, Operation, and Maintenance of the entire wastewater conveyance system. The vast majority of all sanitary sewer overflows originate from three sources in the collection system: 1) I/I, 2) roots, and 3) fats, oil, and grease (FOG). I/I problems are best addressed through a program of regular flow monitoring, T.V. monitoring, and pipeline rehabilitation and replacement. Blockages from roots or FOG are also addressed via a routine cleaning program. A FOG control program may also involve public education and City regulations (e.g. requirements for installation and regular maintenance of grease interceptors). All new facilities believed to contribute FOG should be equipped with grease interceptors.

The DEQ prohibits all sanitary sewer overflows. The Oregon sanitary sewer overflow rules include both wet-weather and dry-weather design criteria. The DEQ has indicated that they have enforcement discretion and that fines will not occur for overflow resulting from storm events that exceed the DEQ design criteria (i.e. greater than a winter 5-year storm event or a summer 10-year storm event).

In December 2009, the DEQ developed a Sanitary Sewer Overflow Enforcement Internal Management Directive that provides guidance for preventing, reporting, and responding to sanitary sewer overflows. The DEQ updated this document in November 2010. The City's discharge permit also includes requirements for an Emergency Response and Public Notification Plan.

Excessive Infiltration and Inflow

EPA defines excessive I/I as the quantity that can be economically eliminated from a sewer system by rehabilitation. Some guidelines for determining excessive I/I were developed in 1985 by EPA based on a survey of 270 standard metropolitan statistical area cities (EPA Infiltration/Inflow Analysis and Project Certification, 1985). Non-excessive numeric criteria for infiltration was defined as average daily dry-weather flows that are below 120 gpcd. Similarly, a guideline of 275 gpcd average wet-weather flow was established as an indicator below which is considered non-excessive storm water inflow.

Pipeline Surcharging

Pipeline surcharging occurs as flows exceed the capacity of a full pipe, causing wastewater to back up into manholes and services. Surcharging of gravity pipelines is generally discouraged because of: 1) the increased potential for backing up into residents' homes, 2) the increased potential of exfiltration, and 3) health risks associated with sanitary sewer overflows.

Illicit Cross Connections

Any illicit cross connections from the City's storm water system should be removed.

2.7.2 Wastewater Treatment Plant

The City of Stayton's WWTP currently operates under the 2016 National Pollutant Discharge Elimination System (NPDES) permit, which expires January 31, 2021 (Permit Number 101601). Oregon Department of Environmental Quality (DEQ) is the regulatory agency charged with the administration of the NPDES permit program established under the Clean Water Act (CWA). Appendix B includes a copy of the permit. The City has submitted its permit renewal application to the DEQ for review, and DEQ is currently working on the renewal. According to DEQ, the current permit will be administratively extended and remain in effect until DEQ takes action on the renewal application and renews the permit.

Current NPDES Permit Discharge Requirements

The City of Stayton currently discharges treated effluent under the NPDES permit to the North Santiam River at River Mile 14.9. Table 2-13 summarizes the existing effluent limits.

TABLE 2-13: EXISTING NPDES PERMIT LIMITS

Parameter	Unit	Monthly Average Limit	Monthly Geometric Mean Limit	Weekly Average Limit	Seven Day Rolling Average Limit	Daily Maximum Limit	Instantaneous Maximum Limit
$CBOD_5$ (May 1 - October 31)	mg/L	10	--	15	--	--	--
	ppd	110	--	160	--	220	--
	% removal	85 (minimum)	--	--	--	--	--
TSS (May 1 - October 31)	mg/L	10	--	15	--	--	--
	ppd	110	--	160	--	220	--
	% removal	85 (minimum)	--	--	--	--	--
BOD_5 (November 1 - April 30)	mg/L	30	--	45	--	--	--
	ppd	340	--	510	--	680	--
	% removal	85 (minimum)	--	--	--	--	--
TSS (November 1 - April 30)	mg/L	30	--	45	--	--	--
	ppd	340	--	510	--	680	--
	% removal	85 (minimum)	--	--	--	--	--
Temperature (June 16 - August 31)	million Kcals/day	--	--	--	57	--	--
Temperature (April 1 - April 30)	million Kcals/day	--	--	--	129	--	--
Temperature (May 1 - June 15; September 1 - October 31)	million Kcals/day	--	--	--	89	--	--
E. coli	#/100 mL	--	126	--	--	--	406
pH	SU	Range of 6.0 - 9.0					

The City does not currently operate a recycled water program but could develop one as explained in the permit. If the City developed a recycled water program, a Recycled Water Use Plan meeting the requirements of OAR 340-055 would need to be submitted and approved by the DEQ.

Biosolids

Both federal and state regulations apply to land application of biosolids from wastewater treatment plants. Title 40 of the Code of Federal Regulations, Part 503 (40 CFR §503) discusses standards for the use and disposal of biosolids. Oregon regulations include OAR 340-50. The state biosolids regulations were most recently revised in July 1995. They reference many of the federal technical biosolids regulations (40 CFR §503), including limits on trace pollutants and pathogens. Under state regulations, the City must keep a Biosolids Management Plan (BMP) and Land Application Plan. The City revised the BMP in 2014 to reflect the changes to the solids treatment that produced Class A biosolids.

Under normal circumstances, the City treats all solids removed in the wastewater treatment process by dewatering and drying. All biosolids meet the requirements for Class A EQ biosolids designation. As such, the biosolids have no restrictions on their use. The biosolids produced are given away in bulk at the WWTP. All off-site transportation is done by those receiving the biosolids.

Mixing Zone

The current permit provides for a mixing zone that consists of the portion of North Santiam River contained within a band extending out 66 feet from the north bank of the river and extending from a point 10 feet upstream of the outfall to a point 200 feet downstream of the outfall. The Zone of Immediate Dilution (ZID) is the portion of the allowable mixing zone located within 20 feet of the point of discharge. The most recent mixing zone study was conducted in October 2006 by West Yost Associates.

Emerging and Future Water Quality Regulations

In the 20-year planning period, it is possible that water quality regulations could become more stringent. However, in discussions with the DEQ, they suggest that no significant changes are expected. The Three Basin Rule (OAR 340-041-0350) apply to the North Santiam River Basin. As a result of the Three Basin Rule, the City of Stayton must stay within their current mass loads for all pollutants. Growth within the City of Stayton and Sublimity, and the resulting increases influent flow and load, must be addressed by increases in efficiency and/or non-discharging alternatives. The BOD_5 and TSS loading limits are technology-based effluent limits based on the Basin Standards of OAR 340-041 and the design flows to calculate the mass loads.

The temperature requirements are set by the TMDL on the North Santiam River. The requirements are derived from a waste load allocation (WLA). Similar to other pollutants, the current thermal loading is not anticipated to change. The pH requirements for the North Santiam River (6.5 to 8.5) are met at the edge of the mixing zone.

This section discusses some of the potential parameters that could be regulated over the planning period.

Ammonia Rule

In August 2015, EPA approved revisions to Oregon's ammonia water quality standards for the protection of aquatic life. This standard identifies that mussels and snails are the most sensitive species. DEQ did not adopt criteria for ammonia, based on the absence of snails/mussels, but current information indicates that they are (or historically were) present through most of Oregon. DEQ did not preclude the development of site-specific criteria.

Currently, ammonia discharge is not regulated at the WWTP, although monitoring is required per Schedule B. A reasonable potential analysis (RPA) was performed during the last permit evaluation. The RPA indicated no reasonable potential for exceeding the new criteria. The previous permit contained a limit for ammonia, and a permit modification in 2008 removed that limit.

Nutrients

Nitrogen and phosphorus are the typical concerns for nutrient impaired receiving water bodies. The North Santiam River Subbasin, where the WWTP outfall is located, is not water quality limited for nutrients.

Dissolved Oxygen

The North Santiam River Subbasin has stream segments that are listed under the CWA 303(d) list for dissolved oxygen. At this time there is not a TMDL for the subbasin. There is potential for a TMDL to be developed for dissolved oxygen in the future, but the timeline is unknown at this time, and it is not clear if a TMDL would impact the City of Stayton WWTP discharge limits. When a TMDL is completed, the City will be assigned a WLA within the TMDL specific to the City's discharge to the North Santiam River.

Effluent Reuse

An alternative to direct river discharge of treated effluent is using effluent reuse for beneficial purposes. The WWTP does not currently use recycled water within the WWTP property to offset potable water use but has the facilities to begin this practice. The storage basin for utility water is approximately 20,000 gallons, while daily potable water use is nearly 140,000 gallons on peak days. The standards for effluent reuse in Oregon are established under OAR 340-055. Planning considerations may include increasing the utility water storage for onsite reuse.

Oregon Human Health Water Quality Criteria

Discharges must be evaluated for toxic pollutants of concern (POCs) that might cause an exceedance of the water quality standard in the receiving water body. The current water quality criteria for aquatic toxicity are listed in OAR 340-41 pollutant Tables 20, 33A and 33B, and for human health water quality criteria in OAR 340-41 pollutant Table 40.

Mercury is a contaminate of concern throughout the Willamette Basin, of which the North Santiam River is a subbasin. Total mercury was found to be above detection in the City of Stayton effluent, and a mercury minimization plan was required during the last NPDES permit evaluation process. Implementation of the plan is to be started within one month of DEQ approval of the plan.

Reliability and Redundancy

The EPA Technical Bulletin EPA-430-99-74-001: *Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability* (1973) requires new or expanding wastewater treatment plants that discharge to a receiving stream to meet minimum standards for mechanical, electrical, and component reliability. Redundancy and reliability refer to the level of protection required for the environment and receiving stream. The standards are divided into three, increasingly stringent, classes of reliability:

- Reliability Class I: Works that discharge, or potential discharge, (1) into public water supply, shellfish, or primary contact recreation waters, or (2) as a result of its volume and/or character, could permanently or unacceptably damage or affect the receiving waters or public health if normal operations were interrupted.

- Example: discharging near drinking water intakes or into shellfish waters.
- Reliability Class II: Works that discharge, or potential discharge, as a result of its volume and/or character, would not permanently or unacceptably damage or affect the receiving waters or public health during periods of short-term operations interruptions, but could be damaging if continued interruption of normal operations were to occur (on the order of several days).
- Example: discharging into recreational waters
- Reliability Class III: Works not otherwise classified as Class I or Class II.

The DEQ has indicated that all WWTPs within the Willamette Valley are Class I facilities. Class I and Class II requirements are outlined in Table 2-14. In addition to these standards, unit operations must be designed to pass the peak hydraulic flow with one unit out of service. Also, mechanical components in the facility must be designed to enable repair or replacement without violating the effluent limitations or causing control diversion.

TABLE 2-14: EPA REQUIREMENTS FOR RELIABILITY

<table>
<tr><th>Component</th><th>Reliability Class I</th><th>Reliability Class II</th></tr>
<tr><td>Raw sewage pumps, lift stations</td><td colspan="2">Peak flow with largest unit out of service. Peak flow is defined as the maximum wastewater flow expected during the design period.</td></tr>
<tr><td>Mechanical bar screens</td><td colspan="2">One backup with either manual or mechanical cleaning shall be provided. Facilities with only two screens shall have at least one manually cleaned bar screen.</td></tr>
<tr><td>Grit removal</td><td colspan="2">Overflow shall be sufficient to pass peak flow with all grit units out of service.</td></tr>
<tr><td>Primary sedimentation</td><td colspan="2">50% of design flow capacity with the largest unit out of service. Design flow is defined as the flow used as the design basis of the component.</td></tr>
<tr><td>Activate sludge process</td><td colspan="2">A minimum of two equal volume basins shall be provided. No backup basin required.</td></tr>
<tr><td>Aeration blowers</td><td colspan="2">Supply the design air capacity with the largest unit out of service shall be provided. A minimum of two units.</td></tr>
<tr><td>Air diffusers</td><td colspan="2">With the largest section of diffusers isolated or out of service, oxygen transfer capacity shall not be measurably impaired.</td></tr>
<tr><td>Secondary sedimentation</td><td>The units shall be sufficient in number and size so that, with the largest unit out of service, the remaining units have capacity for at least 75% of the design flow.</td><td>The units shall be sufficient in number and size so that, with the largest unit out of service, the remaining units have capacity for at least 50% of the design flow.</td></tr>
<tr><td>Filters/advanced treatment</td><td>The units shall be sufficient in number and size so that, with the largest unit out of service, the remaining units have capacity for at least 75% of the design flow.</td><td>No backup required.</td></tr>
<tr><td>Disinfection basins</td><td colspan="2">50% of design flow capacity with the largest unit out of service. Design flow is defined as the flow used as the design basis of the component.</td></tr>
<tr><td>Effluent pumps</td><td colspan="2">Peak flow with largest unit out of service. Peak flow is defined as the maximum wastewater flow expected during the design period.</td></tr>
<tr><td rowspan="2">Electrical power</td><td colspan="2">Provisions of two separate and independent sources of electrical power, either from two separate utility substations or from a single substation and a works-based generator shall be provided. Designated backup source shall have sufficient capacity to operate all vital components, critical lighting, and ventilation during peak flow conditions.</td></tr>
<tr><td>The provision of backup power capacity for secondary treatment, final clarification, and advanced treatment is required. The provision of capacity for degritting and sludge handling and treatment is optional.</td><td>The provision of backup power capacity for secondary treatment, final clarification, and advanced treatment is optional. The provision of capacity for degritting and sludge handling and treatment is not required.</td></tr>
<tr><td>Sludge holding tanks</td><td colspan="2">The volume of the holding tank shall be based on the expected time necessary to perform maintenance and repair of the component in question.</td></tr>
<tr><td>Anaerobic digestion</td><td colspan="2">At least two digestion tanks shall be provided. Backup sludge mixing equipment shall be provided or the system shall be flexible enough such that with one piece of equipment out of service, total mixing capacity is not lost. Backup equipment may be uninstalled.</td></tr>
<tr><td>Aerobic digestion</td><td colspan="2">A backup basin is not required. At least two blowers or mechanical aerators shall be provided. Isolation of largest section of diffusers without measurably impairing oxygen transfer is allowed.</td></tr>
<tr><td>Sludge pumping</td><td colspan="2">Pumps sized to pump peak sludge quantity with one pump out of service. Backup pump may be uninstalled.</td></tr>
</table>

Source: EPA Technical Bulletin EPA-430-99-74-001: Design Criteria for Mechanical, Electric, and Fluids System and Component Reliability (1973).

Regulatory Summary

Based on the discussion above, Table 2-15 provides a summary of the assumed WWTP treatment requirements for the WWTP 20-year planning period. Average effluent concentrations for $cBOD_5$, BOD_5 and TSS were estimated using the current mass loading (ppd) and corresponding dry or wet weather max month, week, or day influent flow.

TABLE 2-15: WWTP 20-YEAR (2040) PLANNING CRITERIA

Parameter	Unit	Influent	2040 Planning Effluent Requirements					
			Monthly Average Limit	Monthly Geometric Mean Limit	Weekly Average Limit	Seven Day Rolling Average Limit	Daily Maximum Limit	Instantaneous Maximum Limit
Annual Average Daily Flow (AADF)	MGD	1.96	--	--	--	--	--	--
Dry Weather Avg. Flow (ADWF) (May 1 - Oct. 31)	MGD	1.25	--	--	--	--	--	--
Max. Month Flow ($MMDWF_{10}$) (May 1 - Oct. 31)	MGD	2.25	--	--	--	--	--	--
Max. Week Flow (DPWkf) (May 1 - Oct. 31)	MGD	3.89	--	--	--	--	--	--
Max. Day Flow (DPDF) (May 1 - Oct. 31)	MGD	4.38	--	--	--	--	--	--
Wet Weather Avg. Flow (AWWF) (Nov. 1 - Apr. 30)	MGD	2.43	--	--	--	--	--	--
Max. Month Flow ($MMWWF_5$) (Nov. 1 - Apr. 30)	MGD	4.54	--	--	--	--	--	--
Max. Week Flow (PWkF) (Nov. 1 - Apr. 30)	MGD	5.67	--	--	--	--	--	--
Max. Day Flow ($PDAF_5$) (Nov. 1 - Apr. 30)	MGD	7.82	--	--	--	--	--	--
Peak Instantaneous Flow (PIF_5) (Nov. 1 - Apr. 30)	MGD	9.18	--	--	--	--	--	--
Temperature (May 1 - Oct. 31)	°C	15-21	--	--	--	--	--	--
Temperature (Nov. 1 - Apr. 30)	°C	12-18	--	--	--	--	--	--
$CBOD_5$ (May 1 - Oct. 31)	mg/L ppd % removal	3,107	10 110 85 (minimum)	-- -- --	15 160 --	-- -- --	-- 220 --	-- -- --
TSS (May 1 - Oct. 31)	mg/L ppd % removal	5,801	10 110 85 (minimum)	-- -- --	15 160 --	-- -- --	-- 220 --	-- -- --
BOD_5 (Nov. 1 - Apr. 30)	mg/L ppd % removal	4,097	30 340 85 (minimum)	-- -- --	45 510 --	-- -- --	-- 680 --	-- -- --
TSS (Nov. 1 - Apr. 30)	mg/L ppd % removal	4,482	30 340 85 (minimum)	-- -- --	45 510 --	-- -- --	-- 680 --	-- -- --
Temperature (Jun. 16 - Aug. 31)	million Kcals/day		--	--	--	57	--	--
Temperature (Apr. 1 - Apr. 30)	million Kcals/day		--	--	--	129	--	--
Temperature (May 1 - Jun. 15; Sept. 1 - Oct. 31)	million Kcals/day		--	--	--	89	--	--
E. coli	#/100 mL	--	--	126	--	--	--	406
pH	SU	6.8	Range of 6.0 - 9.0					

2.8 COMMUNITY ENGAGEMENT

The City provided opportunities for the community to engage in the planning process and provide comments or ask questions through the City website, a Citizen Advisory Committee (CAC), and City Council meeting. The CAC met five times throughout the planning study process to review draft sections and provide comments. Input from the committee was incorporated into the final documents. The City posted draft portions of the planning study on the City website for community review and comment. The community also had the opportunity to engage in the planning process by participating in a City Council meeting that was held before the Council voted to approve the planning study. The Oregon Department of Environmental Quality (DEQ) was part of the Technical Adivsory Committee, reviewed the final document, and provided comments. Responses to the DEQ comments were incorporated into the final report.

SECTION 3 – COLLECTION SYSTEM EXISTING FACILITIES

This section contains a description and evaluation of the existing wastewater collection system (including pump stations and pipelines) for the City of Stayton.

3.1 SYSTEM DESCRIPTION

The wastewater collection system consists of approximately 36 miles of gravity sewer mains, three miles of force main, and four pump stations. The pipelines range from 6 to 24 inches in diameter. Figure 8 (Appendix A) illustrates the pipe diameters and Figure 9 illustrates the pipe material in the City's collection system. There are over 760 manholes in the City's collection system. Pump station locations and their basins are shown in Figure 10.

3.2 CONDITION OF EXISTING FACILITIES

3.2.1 Pump Stations and Force Mains

There are four pump stations and approximately three miles of force main operated and maintained by the City in its wastewater collection system (Figure 10 in Appendix A). Pump stations are generally named by their locations in the City: Industrial, Mill Creek, Wilco, and Gardner.

Onsite facility evaluations were completed in December 2019 and February 2020 with City operations personnel to review conditions of the pump station facilities, current maintenance activities, and known operational problems encountered by City staff. Pump drawdown tests were conducted with help from maintenance personnel to observe the pumps' operation. Gardner pump station is scheduled to be taken out of service and was not evaluated as a part of this planning study.

Industrial and Wilco pump stations are both equipped with dry well pumps next to the wet wells. Mill Creek pump station is equipped with submersible pumps. Each pump station alternates pumps between lead/lag (duplex systems) or lead/lag/standby (triplex systems) for targeting equal runtime between pumps. Level control is either through air bubbler, ultrasonic, or pressure transducer sensors. Float switches are used for high-level alarms. The floats are a redundant system to the main level control and provide a reliable system for the high-level alarm. Table 3-1 contains summary information for the three pump stations evaluated. Appendix D includes pump curves for the three pump stations described below.

TABLE 3-1: PUMP STATION INVENTORY

	Industrial	Mill Creek	Wilco
PUMP STATION			
Type	Dry well, duplex pump station	Wet well, triplex pump station	Dry well, duplex pump station
Pump Type	Self-priming, non-clog centrifugal (Smith & Loveless 4B2Y)	Submersible, VFD (set for soft start), non-clog (Flygt NP 3202-090/640)	(Smith & Loveless 6C3A)
Capacity[1] (gpm)	Each pump: 150 gpm @ 21 ft TDH	Two pumps: 3,170 gpm @ 77 ft TDH; One pump: 2,220 gpm @68 ft TDH	Each pump: 800 gpm at 48 ft TDH
Pump (each)	2 hp @ 900 rpm (230 V, 60 Hz, 3 ph)	60 hp @ (460 V, 60 Hz, 3ph)	20 hp@ 1175 rpm (230 V, 3 ph)
Level Control Type	Air bubbler to be replaced with Ultrasonic	Pressure Transducer	Ultrasonic
Overflow Point	Influent MH	Influent MH	Influent MH
Overflow Discharge	Stormwater swale with drain	Storm drain	Storm drain
Auxiliary Power Type	Portable generator	Permanent diesel generator	Permanent diesel generator
Location	At WWTP	Onsite	Onsite
Output (kW)	85	150	80
Fuel Tank Capacity (gal)	170	196	100
Transfer Switch	Manual	Automatic	Automatic
Alarm Telemetry Type	Radio, operator call-out	Radio, operator call-out	Radio, operator call-out
Originally Constructed	1980's	2006	1975
Year Upgraded	2016 (pumps), 2020 (controls)	2016	2007 (electrical/controls)
Wet Well Diameter (ft)	6	12	8
Wet Well Net Storage (gal)	4,652	30,032	8,271
FORCE MAIN			
Length, Type	Approx. 525 ft. of 6-inch	Approx. 8715 ft. of 18-inch PVC and 20-inch HDPE	Approx. 75 ft. of 12-inch PVC
Profile, Continuously Ascending (Yes/No)	Yes	No	No
Discharge Location	Manhole at W Deschutes Drive and Willamette Avenue	Manhole on Jetters Way north of WWTP	Mill Creek discharge force main
Combination Air Release/Vaccuum Valves	No	Yes	No

[1]Capacity as reported in record drawings and O&M Manuals

This evaluation presents general observations and recommendations, along with specific recommendations for individual pump station sites. General observations and some recommendations are presented first for the pump station sites, wet wells, electrical systems, instrumentation, telemetry, drawdown tests, housekeeping, maintenance, safety equipment, emergency generators, and security. General recommendations are provided as a guideline to allow the City to maintain the pump stations for the 20-year planning period. Any items of concern observed during the onsite evaluation are also noted. Pump station specific observations and recommendations follow.

A. General Observations

Sites

The pump station sites are easily accessible from streets throughout the City. Industrial and Wilco pump stations are not fenced. Bollards around the controls and onsite generator of Wilco pump station provide some protection. The Industrial pump station does not have these protection measures, although it is at the end of a low-trafficked industrial drive. Mill Creek pump station has fencing and a locked building.

Wet Wells

The wet wells are concrete, in good condition and do not show signs of significant deterioration. There was minimal buildup of grease and debris in the wet wells during the site visits. City operators indicate that grease and debris buildup is not an issue and regular maintenance prevents significant buildups.

Electrical Systems

Electrical systems at all pump stations are in fair condition. The controls at Industrial and Wilco pump stations were replaced and moved above ground since the original construction of the pump stations. Electrical equipment becomes obsolete in time due to changes in technology. Parts and service for outdated equipment become more difficult to obtain in time, requiring replacement with new equipment. Most of the electrical equipment at the pump stations will become obsolete and require replacement within twenty years. The equipment can be replaced when this occurs.

Instrumentation

Instrumentation consists of pressure gauges, pressure transducers for analog transmission of pressures, ultrasonic sensors for digital transmission of incremental levels in the wet well, and float switches. Level control, alarms, and flow are typically the only instruments pump stations require. Monitoring flow at pump stations is recommended for maintenance and operational benefits. A record of flow from a pump station can provide information on pump, sewer, and inflow conditions; unauthorized inflow; and future planning for expansion or replacement.

Telemetry

Wilco and Industrial pump stations have radio-based telemetry systems with communication to a central location, the WWTP. Mill Creek pump station is connected to the WWTP SCADA via fiber optic cable. The telemetry systems are currently functioning adequately and use SCADA programmable logic controller (PLC) systems. Currently, the City uses a general frequency, but has been discussing getting their own frequency. The stations are programmed with a variety of call-out alarms, which trigger a notification at the WWTP and a call to the on-call operator phone. Each of the pump stations have the following call-out alarms:

- High level
- Low level
- Power out
- Station running on emergency power (Mill Creek pump station)
- Individual pump faults
- VFD failure (Mill Creek pump station)
- Communication failure

It is recommended that a call-out alarm be added that notifies operators if all of the pump in a pump station turn on (an indication of no redundancy). In addition to these alarms, the stations are equipped with backup level sensors or floats except for Industrial pump station. The backup floats at Wilco pump station are not connected to the SCADA and are not functional at this time. It is recommended that the floats be investigated and repaired or serviced as need as well as connected to the SCADA to send out unique alarms for the high/low water levels.

Drawdown Tests

During the site visit, drawdown pump tests were completed to determine approximate pump flow rates. Each pump was tested at the three pump stations evaluated, and pumping combinations were tested at Mill Creek pump station. Depth readouts, available on each PLC, were used to record depth over time. Estimates for average pump flow rates were calculated using the pump test data. These estimated flow rates, along with the rated pump capacities, are shown in Table 3-2.

TABLE 3-2: MEASURED PUMP FLOW RATES

Pump Station	Average Flow Rate (gpm)	Rated Pump Capacity (gpm)
Industrial	100	150
Mill Creek (one pump)	1,700	2,220
Two Pumps	2,500	3,170
Wilco	750	800

Wilco pump station had inconsistent average flow rates between the two pumps during pump testing. Pump 2 (north pump) was running at approximately half the flow rate of Pump 1 (south pump). The onsite display indicated that Pump 2 ran approximately 1.5x longer the day before and had run approximately 1.25 longer that day so far. Weekly pump run logs for the past five months show that Pump 2 typically has 1-2 hours more of daily run time than Pump 1. Maintenance logs indicated that the pumps have had air lock issues in the last 10 years. It is possible that Pump 2 has an air lock at the time of the site visit that caused the decreased pumping capacity. The pump impeller was replaced in September 2020 and could improve the flow rate. The City is waiting on a quote to replace the motor and possibly the rotating assembly as part of the maintenance on Pump 2. It is recommended that the City have Pump 1 inspected at the same time the Pump 2 maintenance is performed. After the maintenance and inspections, it is recommended additional pump field tests be performed to assess field-rated firm capacity of the Wilco pump station.

Housekeeping/Maintenance

The pump stations are kept in clean and orderly condition. The interior of the Mill Creek pump station is in good condition. Floors and walls are clean, painted, and maintained. Two sources of wash-down water are provided inside and outside the building. A yard hose connection is available at Mill Creek pump station, with a reduced pressure backflow preventer valve is installed in the plumbing upstream of it. Water can be transported to the Industrial and Wilco pump stations on the City's Vac-Con and Crane trucks as needed.

Mill Creek and Wilco pump stations wet well interiors were clean, with only small amounts of floating debris and FOG buildup. Industrial pump station receives industrial flow that does not contain significant amounts of FOG. Operators inspect pump stations weekly, recording pump run times and perform FOG wash-downs and vacuum of the wet wells as needed. Operators indicate this maintenance schedule is sufficient to prevent any larger backups or problems in the collection system from FOG. Other major monthly maintenance activities include pump checks, high-level alarm checks, observation of vandalism or other problems, and generator checks.

Safety Equipment

The Mill Creek pump station wet well has fall protection installed under the solid covers. The fall protection consists of a steel grating on hinges that covers the opening to prevent falling into the wet well. The grates can be hinged up should access to the well be required. The other two wet wells have manhole cover access to the wet well. There are no fire extinguishers, first aid kits, or eye wash stations at the pump stations. However, operators carry fire extinguishers and first aid kits in their trucks. Onsite wash-down water and hoses could be used if an operator were to be exposed to contaminated material at any of the pump stations.

Emergency Generators and Backup Power

All permanent generators are located outside in weatherproof enclosures. Mill Creek and Wilco pump stations have emergency diesel generators; these run on diesel fuel stored in an above-ground tank at each generator. The fuel tanks are located under the generator frame skid (referred to as a sub-base fuel tank with a double wall containment) and fuel is pumped directly from the tank. The Industrial pump station has a connection available for a portable 85 kW generator stored at the WWTP. The generators are automatically exercised weekly for a short period of time, including the portable generator at the WWTP. All generators were fully serviced in Fall 2019.

Security

The Mill Creek pump station site is fenced, with a gate that locks. Industrial and Wilco pump stations have electrical panels and access manholes that are locked. No intrusion alarm system nor video equipment were observed at the sites. Use of video security provides a deterrent to vandalism, improved public safety, and a higher level of confidence in the reliability of the system. Mill Creek and Wilco pump stations have outdoor site lighting.

HVAC

The Mill Creek pump station building has ventilation fans and louvers for ventilation and air cooling, in addition to inside electric unit heaters.

Cross Connection Control

Cross connections occur when the pump station discharge or wet well is allowed to be connected to a potential source of potable water. The main locations of cross connection potential at pump stations are wash-down hoses and air release valve discharges. The other potential for cross connection is storm water surcharging of pump station sewer overflow systems that then flow into the wet wells. The profiles of overflows connected to the storm system were not evaluated as part of this master plan. There were no known or observed cross connection issues found during the site visits.

B. Industrial Pump Station

Industrial pump station is located at the end west end of Deschutes Drive, in an industrial park area adjacent to a stormwater detention pond. It was originally installed in the 1980's, with installation of new pumps in 2016 and above-ground controls in 2020. The site does not have a building and lacks fencing and bollards. The electrical control enclosure and dry well access are locked. The wet well manhole access is not locked or bolted. The dry well access hatch seal is degrading and missing in spots. The electrical enclosure has an antenna for its SCADA system but lacks permanent outdoor lighting. There is a hookup for a portable, standby generator and the City's crane truck has external lights for work performed at the station after hours. Washdown water is not available onsite but can be brought to the site as need through City vacuum trucks.

The pump station has a combination dry and wet well system, with duplex pumps installed in a circular dry well. The level in the wet well is currently monitored with an air bubbler level sensor, although the air bubbler is scheduled to be replaced with an ultrasonic sensor in 2020 due to repeated maintenance issues. An Allen-Bradley pump controller is used for pump operation. Both pumps were installed in 2016 and are controlled using a lead on, lag on, and pump off operational strategy. The lead and lag pumps are automatically switched to maintain approximately equal run hours on each pump. For pump maintenance, the pumps, motors and isolation valves must be accessed by entering the confined space of the dry well, approximately 25 feet below ground. There is no flow meter or discharge pressure gauge (there are ports for gauges) for this pump station.

The pump station has provisions for a portable generator for emergency operation. The suitable portable generator is kept at the WWTP. An onsite generator would provide better reliability than a portable generator. Many issues can arise during an emergency that would prevent use of a portable generator; blocked access, washed-out or damaged roads, generator failure, and greater need for the portable unit elsewhere are possibilities to consider. However, at this time the influent to the pump station is low and there is adequate storage in the wet well for the portable generator to be brought and connected in the case of a power outage.

The Industrial pump station serves nearby industrial facilities (i.e. farm equipment, machining, and truck repair) and no residences. Influent wastewater flows have little to no FOG. Each pump has a capacity of 150 gpm at 21 TDH. The pump station is intended to be operated so that it cycles through each pump. Based on pump run data recorded by operators from Fall 2019 to Winter 2020, this appears to be how the pump station is operating. There have been no known issues with the pump station overflowing. If the pump station were to overflow, it is anticipated flow would come out of the top of the wet well and flow into the adjacent stormwater pond. The 6-inch effluent force main discharges into a manhole on Deschutes Drive.

Deficiencies:

- Dry well hatch seal has deteriorated and may allow water intrusion
- Equipment is below ground in confined space and is difficult to access, inspect and maintain
- No bollards to protect access hatches and electrical/control panels

Recommendations:

- Replace dry well hatch seal
- At end of useful life of pumps, install submersible pumps and move equipment above ground
- Install bollards

C. Mill Creek Pump Station

Mill Creek pump station is located on the east side of Golf Club Road, just south of Bear Place and 0.7 miles north of Shaff Road. It was installed in 2006 with two pumps, and in 2016 a third pump was added. These pump station upgrades allowed for the displacement of two preexisting pump stations. The pump station conveys flows from Stayton as well as City of Sublimity. There is an onsite metering station to monitor the flows from Sublimity. The site has a small building, influent manhole, wet well, effluent force main vault, flow meter and chemical injection vault, combination air/vacuum valve, bioxide chemical storage tank, storage shed, and generator. A chain link fence topped with barbed wire and locked gate secure the site equipment. Electrical controls, a water bladder tank, chemical feed panel, and restroom are inside the building.

The building can be accessed through a standard swinging door or coiling door, while the equipment in the yard can be accessed through a code operated rolling gate. The building is concrete masonry and was constructed in 2006. Masonry buildings have a long service life and require very little maintenance (other than routine cleaning). Roofs are the main source of building maintenance over time. Mill Creek has asphalt shingles with some moss growth and will likely require repair or replacement over the next 10 years. There are no windows, so future window maintenance or replacement is not required. The doors are painted steel. The doors will be usable for many years, but some maintenance will be required, likely due to paint deterioration over time.

Water is supplied by an on-site well and bladder tank system. The bladder tank and two hose bibbs are housed inside the pump station building. The wash water is available next to the water bladder as well as in the yard.

The level in the wet well is monitored with an ultrasonic level sensor and a pressure transducer system. A Flygt MiniCAS pump controller is used for pump operation. Variable frequency drives are programmed to operate as soft starts. Influent wastewater is routed through a 72-inch manhole before continuing through a 24-inch PVC pipe to the 12-foot diameter wet well. The Operations and Maintenance (O&M) Manual (Tetra Tech, March 2006) states there is no piped overflow at the station. If an overflow were to occur, the wastewater would backup into the influent manhole and then overflow at the wet well and/or influent manhole; the rim elevations are approximately the same. Pumps are mounted on steel pipe rails in the wet well to allow for their removal without entering the wet well. Fall protection is installed at the wet well entrance. The discharge piping has visible corrosion. Coating piping after a station has already been in service is not recommended, as it is a major project that does not provide benefits in comparison to the work involved. If fittings start to corrode, they should be replaced as feasible.

The three 10-inch ductile iron discharge force mains leaving the wet well combine into one 12-inch ductile iron force main, before transitioning to an 18-inch PVC force main and leaving the pump station site. Once offsite, the force main transitions to 20-inch HDPE and back to 18-inch PVC before discharging into a manhole on Jetters Way. A valve vault next to the wet well provides access to the 10-inch discharge force mains plug and check valves. There are pressure gauges on each pipe, but they are were not functioning at the time of the facility visit and are only visible from inside the vault. A 4-inch combination air/vacuum valve is connected through an offset pipe to the 12-inch force main. It is heavily rusted and should be serviced to assess whether it needs to be replaced. The valve manufacturer recommends servicing at least once per year. An 8-inch bypass line connects the 72-inch manhole to the offset pipeline. There is no isolation valve on the line to the wet well. A valve vault on the 12-inch force main contains a flow meter followed by chemical injection. The flow meter is no longer functional and needs to be replaced. There is no bypass line on the discharge force main downstream of the flow meter. There is no isolation on the downstream side of the flow meter. This makes maintenance or isolation and replacement of the flow meter difficult for the City to achieve due to the high volume of wastewater being pumped through the Mill Creek pump station. Both the flow meter and valve vaults are hard piped and do not contain dismantling joints.

A 3,650-gallon bioxide chemical tank is connected to both the 12-inch discharge flow main and the wet well. It is currently set up to continuously feed bioxide at the force main and is effective at controlling odor. The system runs continuously during summer months and is often turned off in the winter months. The tank does not have a drain and thus is impossible to clean. The tank has not been cleaned or inspected in at least one year. The discharge piping has a small leak at the base of the tank near the ball valve. The tank lacks a level reader so operators must monitor the level visually to assess when it must be refilled, which is typically three times per year. Additionally, the fill line PVC piping has degraded due to UV radiation. The chemical pump feed lines from the tank to the building are exposed to weather. The chemical feed panel is inside the pump station building. There are two chemical pumps on the chemical panel, but only one was operable at the time of the site visit. There is currently a damaged seal on the chemical piping adjacent to the panel that has a leak and resulting corrosion buildup. The odor control system was installed with the pump station because while the station was built to handle future flows, low flows at the time of installation caused odor issues. Flows to the pump station have since increased and the City performed sulfide testing and monitored gases at the treatment plant headworks in 2020 to assess whether continued operation of the odor control system is necessary. Results indicate that septic conditions do not exist, and the City plans to phase the system out of service.

A permanent diesel standby generator is controlled through an automatic switch inside the pump station building. It is equipped with external noise attenuation.

Mill Creek pump station serves the northern area of Stayton and the entire population of Sublimity. Each pump has a capacity of 2,220 gpm at 68 TDH, while two pumps in service have a capacity of 3,170 gpm at 77 TDH. There have been no known issues with the pump station overflowing. The pumps were most recently serviced in Spring 2019. The pump station is intended to be operated so that it cycles through each pump. Based on pump run data recorded by operators from Spring 2019 to 2020, Pump 3 tends to run longer than Pumps 1 and 2.

Deficiencies:

- Building roof has moss growth accumulating, which accelerates roof degradation
- Pressure gauges not functioning
- Corroded discharge lines in wet well
- Heavily rusted combination air/vacuum valve
- Discharge flow meter does not function
- No bypass option for flow meter work
- No dismantling joints in flow meter and valve vaults

- No isolation or check valve downstream of flow meter
- No isolation valve on wet well inlet line from 72-inch manhole

Recommendations:

- Regular moss removal of building roof
- Install new discharge pressure gauges
- Service combination air/vacuum valve
- When odor control system is taken offline permanently, chemicals on site should be stored or disposed of properly for operational safety.
- Replace discharge flow meter
- Install bypass option for work on flow meter
- Install isolation valve for flow meter and replace flow meter
- Install isolation valve on wet well inlet line from 72-inch manhole

D. Wilco

Wilco pump station is located on Wilco Road, just north of W Locust Street, in an industrial yard. It was originally installed in 1975 and previously conveyed wastewater flows from Sublimity. With the 2006 construction of Mill Creek pump station, Wilco's influent flows decreased considerably. The pump station lacks a building and fence but has several bollards around the dry well, control enclosure, and standby generator. A small portable heater is stored onsite in the enclosure. Electrical upgrades to the pump station were completed in 2007. The electrical control enclosure, dry well access and wet well access are all locked. The electrical enclosure has an antenna for its SCADA system and outdoor lighting.

The pump station has a combination dry and wet well system, with duplex pumps installed in a circular dry well. The level of the wet well is monitored with an ultrasonic sensor. An Allen-Bradley pump controller is used for pump operation. Both pumps were initially installed in 1975 and are controlled using a lead on, lag on, and pump off operational strategy. The lead and lag pumps are automatically switched. One pump has since been replaced (Pump 1), the year of installation is unknown at this time. Pump 2 is scheduled to have the impeller and possibly a complete motor and rotating assembly replaced in the near future. For pump maintenance, the pumps, motors and isolation valves must be accessed through the dry well, approximately 25 feet below ground. There is no discharge flow meter or pressure gauge at the site. A bypass pump connection is available onsite connecting to the discharge force main. There is an access manhole to an abandoned, heavily rusted air release valve. It is assumed that the ARV was connected to the abandoned 10-inch asbestos-lined discharge force main but could not be verify with record drawings.

The 8-foot diameter wet well is monitored for FOG buildup to avoid impedance of the ultrasonic sensor. No FOG buildup was evident at the time of the facility visit. Wash water is available onsite through a hose bib. The 12-inch effluent force main connects directly to the 18-inch Mill Creek discharge force main on the west side of Wilco Road. A permanent diesel standby generator is operated with an automatic transfer switch installed as part of the 2007 electrical upgrades. It is equipped with external noise attenuation. The generator was most recently serviced in November 2019.

Wilco pump station serves a small, primarily residential area. Each pump has the capacity of 800 gpm at 48 TDH. There have been no known issues with the pump station overflowing. If the pump station were to overflow, it is anticipated the wastewater would backup in the influent lines and overflow at the manhole at the intersection of Wilco and Shaff Roads based on rim elevations in the City GIS system. The pump station is intended to be operated so that it cycles through each pump. Based on daily pump start data during the site visit, it appears to be operating in this manner. However, it appears that Pump 2 consistently has higher daily run times. As discussed previously, the Wilco pumps had inconsistent average flow rates between the two pumps during onsite pump testing. Pump 2 (north pump) had a much lower flow rate than Pump 1 (south pump). Pump run times recorded from November 2019 through February 2020 indicates that Pump 2 has consistently higher run times. The difference in run times between the two pumps has increased in 2020 with Pump 2 running approx. 34% more each month. It is possible that Pump 2 impeller is significantly worn or has an air lock that is causing the decreased pumping capacity. Operators are waiting on motor and rotating assembly quotes before moving forward with these and impeller replacements. At this time, Pump 1 has not been inspected recently.

Deficiencies:

- General O&M and equipment information is not documented in one location for the operators to access.
- Pump 2 has a significantly lower field-tested pumping rate than Pump 1
- Pump 2 impeller, motor, and rotating assembly show signs of wear
- Pump 1 has not been inspected recently
- Equipment is below ground in confined space and is difficult to access, inspect and maintain

Recommendations:

- Document equipment and O&M at pump station and WWTP for easy access to operators
- Replace Pump 2 impeller and motor and rotating assembly as needed upon inspection
- Inspect Pump 1 and document condition; perform maintenance as needed upon inspection
- Perform field capacity tests on both pumps after maintenance has been completed
- At end of useful life of pumps, install submersible pumps and move equipment above ground

3.4.2 Gravity Mains

Please refer to Section 7 for gravity main discussion.

3.3 COLLECTION SYSTEM OPERATION & MAINTENANCE SUMMARY

See Section 6.3 for Operation and Maintenance Summary and Recommendations

SECTION 4 – COLLECTION SYSTEM HYDRAULIC EVALUATION

4.1 COLLECTION SYSTEM COMPUTER MODEL

This section summarizes the wastewater collection system model development process and existing and 20-year collection system analysis. It outlines the model construction and calibration process, and also documents existing deficiencies. Improvements to address these deficiencies are discussed in Section 5.

4.1.1 Model Construction

InfoSWMM Suite 14.7 Update #1 was selected as the modeling software for this project. InfoSWMM is a fully dynamic model which operates in conjunction with Esri ArcGIS and allows for evaluation of complex hydraulic flow patterns. The previous master plan collection system model was completed in XPSWMM (Keller Associates, 2006).

The City maintains a Stayton GIS database. Pipe diameter and invert elevation data for the model were populated from this database as well as the previous XPSWMM model and available record drawings. As part of model construction, 20 spot elevation locations were surveyed throughout the City, along trunk lines, to verify GIS database elevations. In places where survey data was unable to be collected, record drawings were referenced.

Pipelines with diameters of 8-inches and larger were modeled. Figure 7 in Appendix A shows the modeled lines in the system. After all manholes and pipes were created, and data populated in the model, several queries were conducted to reveal anomalies in the data. These included reverse slope pipes, unusual changes in pipe size, and uncommon configurations in the pipe network. Anomalies were discussed with City personnel and appropriate changes were made to the model.

All four pump stations (Industrial, Mill Creek, Wilco, and Gardner) are included in the existing system model. Pump station wet well dimensions and operational set points were provided by the system operators or taken from the operations and maintenance (O&M) manuals. Average pump capacities were verified by field tests and O&M manual pump curves were used to characterize the pump station pumps when available. Pump field tests were not performed at Gardner pump station and pump capacity from O&M documents was used for the model. All pump stations were modeled with their firm capacities (capacity with largest pump offline).

It is important to note that one of the basic assumptions of the hydraulic model is that all pipelines are free from physical obstructions such as roots and accumulated debris. Such maintenance issues, which certainly exist, must be discovered and addressed through consistent maintenance efforts. The control structure on Jetters Way was assumed to be completely open during model simulations. The modeled capacities discussed in this chapter represent the capacity assuming the sewer lines are in good working order.

4.1.2 Model Calibration

Model loads refer to the wastewater flows that enter the sewer collection system. These loads are comprised of wastewater collected from individual services (base flows), plus groundwater infiltration and storm water inflows (I/I). As part of this study, flow monitoring was completed during the wet weather period from February to the first week of April 2020. Flow monitoring data was collected at six manholes throughout the system to help calibrate the model. The six monitoring sites divided the system into six basins. Figure 11 (Appendix A) shows flow monitoring locations and basins used for model calibration. The basins were used to characterize flows throughout the system. The collected data was analyzed along with continuous precipitation data to establish typical 24-hour patterns, average flows at each site, and gauge rainfall influence in the system. Both dry weather and wet weather periods were used for loading and calibration efforts. Loads for the model were developed and calibrated in several stages as described below.

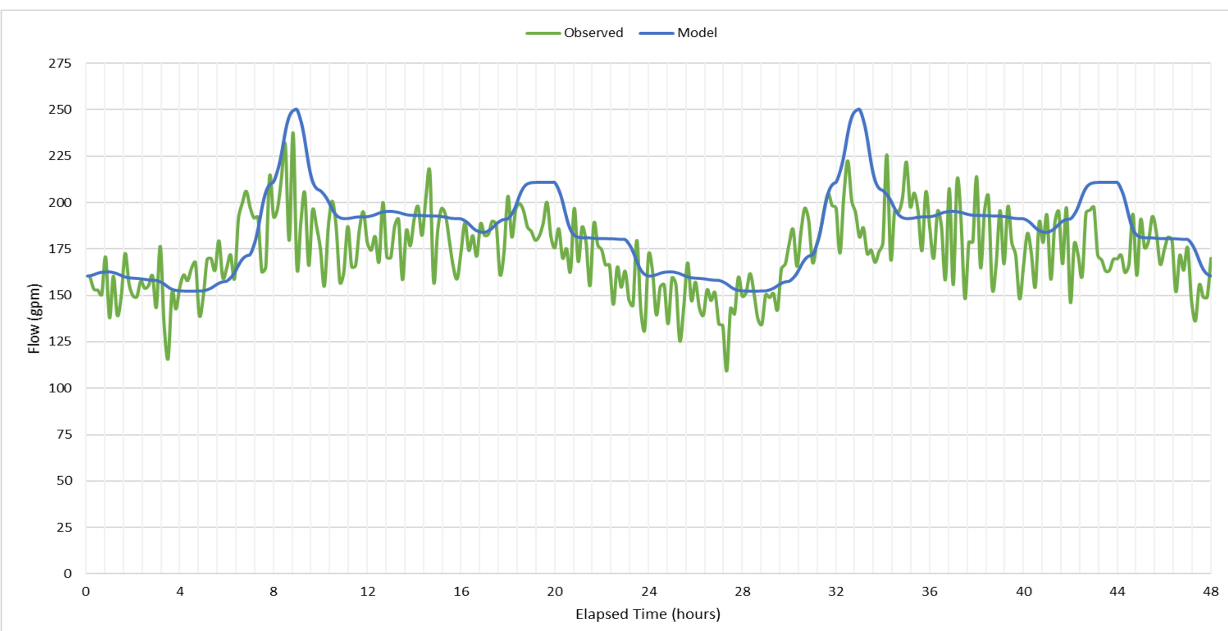
Dry Weather Flow (DWF) Calibration

As a starting point, base flows were estimated using Discharge Monitoring Report (DMR) data from 2015 to 2019 to calculate a per capita flow rate (see Table 2-7 in Section 2 for unit flows). Individual water meter locations for customers in Stayton were linked to the sewer model using GIS to provide a highly accurate distribution of wastewater loads. An average flow was assigned to each modeled manhole based on the equivalent dwelling units (EDU) and per capita flow rate. City Planning utilizes 2.6 people/EDU per 2010 U.S. Census data in its calculations.

A period of two dry days (none or trace amounts of rainfall) was analyzed from the flow monitoring data to select a typical day for each site, which was utilized to develop a diurnal flow pattern for the basin. This dry period was preceded by two days of none or trace amounts of rainfall. Diurnal patterns for each monitoring site were assigned to all dry weather flows within the corresponding basin.

The model was calibrated at the flow monitoring locations within the collection system and total modeled influent flow at the Wastewater Treatment Plant (WWTP) was compared to the targeted design average dry weather flow. Appendix E contains a summary of the data and analysis used for modeling purposes. An example of DWF calibration results are shown below in Chart 4-1. The blue line shows the model results and the green line show flow monitoring data collected.

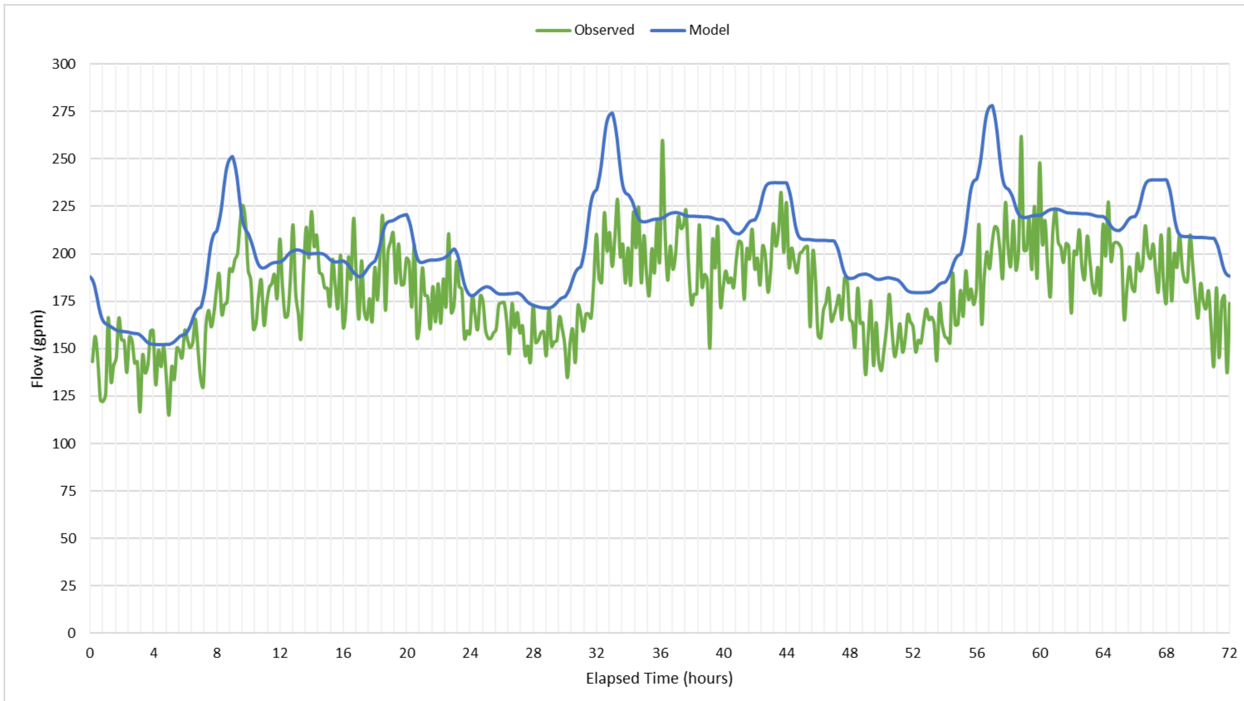
CHART 4-1: SAMPLE DRY CALIBRATION SITE 3



Wet Weather Flow (WWF) Calibration

The RTK method was used for rainfall-derived infiltration and inflow (RDII) prediction. Rainfall data for a 72-hour period with the highest cumulative (0.94 in) rainfall during the period of flow monitoring was utilized to calibrate wet weather flows. The storm event rainfall was entered into InfoSWMM. RTK parameters were then adjusted to calibrate the model with flow monitoring data. Again, total modeled influent flows at the WWTP were compared to the targeted design average daily flow and influent flow data in addition to calibrating the model at various locations within the collection system and at the WWTP influent. An example of wet weather flow calibration results is shown below in Chart 4-2.

CHART 4-2: SAMPLE WET CALIBRATION SITE 3



Design Storm

The design storm for model evaluation was the 5-year, 24-hour storm event. A standard 24-hour NRCS rainfall distribution for a Type 1A storm was used. The rainfall for the 5-year, 24-hour storm event from NOAA isopluvial maps is 3.0 inches. This was used as the multiplier for the Type 1A storm hyetograph. The existing system calibrated model was run with the design storm event. The modeled peak instantaneous and the peak of the daily average flows at the WWTP were compared to the design PIF_5 and $PDAF_5$ (Table 2-7). The maximum day and peak flows were lower than the planning criteria. Reviewing flow monitoring and DMR data, the wet weather storm event used for calibration had dry antecedent conditions and the RDII response matched these conditions. The 5-year storm event accounts for wet antecedent conditions and high groundwater infiltration as is typical for large storm events in the area. Additional groundwater infiltration was added to the overall system to represent wet antecedent conditions and further calibrate the 5-year design storm response.

4.1.3 Existing Capacity Limitations

The calibrated model was used to assess the effects of a 5-year, 24-hour design storm event on the existing system. Figure 12a in Appendix A illustrates the potential overflow sites and pipe capacity limitations identified during the existing system peak instantaneous flow model analysis. The figure is color-coded to show a gradation of pipes based on utilized capacity (e.g., red = flowing at >100% capacity, orange = flowing at 85-99% of capacity, yellow = flowing at 75-84% capacity, etc.). As stated in Chapter 2, the planning criteria for undersized pipelines is if the flow is equal or greater than 85% of full capacity based on maximum depth of flow. The manholes shown in red experience overflow in the simulation and represent the greatest risk for backing up services and possible overflow sites.

Most of the undersized pipes in the system are located upstream of the W Ida Street trunk line. In evaluating the model results, surcharging upstream of the trunk line is likely caused by undersized pipes on the downstream end of the system. Figure 12b in Appendix A illustrates this in term of pipe flow capacity. The pipes in red and orange in Figure 12b indicate undersized pipes in the system causing the extensive surcharging illustrated in Figure 12a.

Pipes upstream and downstream of Gardner pump station also have capacity limitations. The peak flow entering the wet well for the pump station during the design storm event was approximately 520 gpm. The reported capacity of each pump is 300 gpm. The firm capacity of the duplex system is lower than the existing peak inflow and is the source of the pipe capacity limitations upstream of the pump station are caused by undersized pumps. The City has plans to displace the Gardner pump station in the near future and have flows redirected north towards the Mill Creek pump station, which will alleviate this capacity limitation.

Three manholes located on W Fir Street, N First Avenue, and N 2nd Avenue indicate potential overflow due to pipe surcharging downstream. The manhole rim elevations are relative low points in the pipe network and were verified by City staff. If these areas have historically overflowed during high rainfall events, it is recommended that City staff monitor and investigate to determine the actual extents of any overflow that may occur.

4.1.4 Critical Slope Areas

The City's 2015 Public Works Design Standards (PWDS) provide minimum pipe slopes for sanitary sewer flow (Table 4-1). Modeled gravity main slopes were compared with these recommended minimum slopes. Pipes that are less than their recommended minimum slope are shown as orange in Figure 13 of Appendix A. Low slopes can cause capacity issues and require higher than normal O&M. These mains should be monitored for capacity, odor, and solids buildup problems. Pipes with low slopes may need to be cleaned more frequently to prevent solids buildup and flow disruption. The City currently cleans and inspects approximately 1/5 of the pipes in the collection system every year. It is recommended the City continue this regular maintenance schedule and note if areas have consistent solids buildup or flow disruption issues. These areas may need to be cleaned more frequently.

TABLE 4-1: MINIMUM PIPE SLOPES

Pipe Size (Inches)	Minimum Slope in Percent (feet per 100 feet)
8	0.40
10	0.28
12	0.22
15	0.20
18	0.20
21 and larger	0.20

Source: 2015 City PWDS, 503.03.C

4.2 FUTURE COLLECTION SYSTEM PERFORMANCE

This section summarizes future flow projections, the model evaluation of future system expansion, and documents anticipated future deficiencies for the 20-year planning period. Alternative improvements to address these deficiencies are presented in Section 5.

4.2.1 Future Flow Projections & Model Scenarios

Future flows were distributed based on PSU population projections (Section 2) and City projected future residential, commercial, and industrial growth. Flows per capita for projected population growth were assumed to be similar to existing flows per capita. Residential flows were projected using future growth areas, City zoning, projected number of equivalent dwelling units, and ADWF per capita attributed with residential contributions. Projected industrial and commercial development is anticipated to grow proportionally to the projected residential growth. The City-identified commercial growth location is estimated to produce an average base flow of 350 gallon per lot based on typical base flow for a car dealership (Metcalf and Eddie, 3rd Edition). The City-identified industrial growth location is estimated to have average base flows of 1,500 gpad. This corresponds to Industrial Commercial Average Day Flow Allowance in Section 503.02.D of the City's 2015 Public Works Design Standards. Projected flows per zoning designation for the 20-year planning period are presented in Table 4-2.

TABLE 4-2: 20-YEAR PROJECTED FLOWS BY ZONING

Zoning	Estimated EDUs	Estimated Acreage	Estimated Unit Flow[1] (gpcd/gpad)	Estimated Flow[2] (gpd)
Residential	530	-	98	135,000
Sublimity	184	-	98	46,900
Commercial	N/A	-	-	350
Industrial	N/A	59	1,500	84,000
Totals:	**714**	**59**		**266,250**

[1]Residential flows based on Design ADWF per capita from Table 2-7 (98 gpcd). Industrial and commercial values from 2015 Public Works Design Standards and Metcalf and Eddie, 3rd Edition.

[2]Assume 2.6 people/dwelling unit (2010 US Census).

The City provided projected near-term development properties for the 20-year planning period as shown in Figure 7 in Appendix A. Flow associated with each development area identified in Figure

7 was added to the closest modeled manhole to allocate future flows. Where applicable, future infrastructure was added to the model for future system evaluation. The future model was run to analyze the effects of future development on the system for the 20-year planning horizon.

4.2.2 20-Year Capacity Limitations

Approximately 6,000 linear feet of gravity main were added to the model as future infrastructure to support development indicated by the City in the next 20-years (Figure 7). The model was run to evaluate the effects of a 2040 peak day flow event on the existing system and the future infrastructure. Figure 14a in Appendix A illustrates the potential overflow sites and capacity limitations identified by the 20-year model analysis. Overall, the problem areas identified in the 20-year evaluation are similar to the areas identified in the existing system analysis (Ida trunk line and Gardner pump station). The model indicates the volume of overflows and duration of overflows or surcharging does increase in these areas compared to the existing scenario. Similar to Figure 12b, Figure 14b indicates pipes undersized for the future simulated storm event.

Future analysis of the Wilco pump station reveals that the peak flow entering the wet well for the pump station during the design storm event was approximately 825 gpm. The capacity of each pump is estimated to be 800 gpm based on O&M manuals. Field testing indicated a wide range of flows in the duplex pump system (1,500 gpm to 480 gpm). It is recommended that additional field tests be performed following upgrades to pump impellers and motors to assess firm capacity. See Section 3 for a more detailed discussion of Wilco pump station.

SECTION 5 – COLLECTION SYSTEM IMPROVEMENT ALTERNATIVES

This section describes alternatives considered to address the collection system deficiencies presented in Sections 3 and 4.

5.1 PLANNING CRITERIA

The planning criteria used for this collection system facilities planning effort are summarized as follows. Improvements to the City's conveyance system will be sized for the projected 2040 peak instantaneous flow rates associated with the 5-year, 24-hour storm event (PIF_5 in Table 2-7). Criteria for requiring improvements is when the max flow depth/full depth (d/D) of a pipe is greater than 85%. Collection systems pipeline improvements will be sized according to City's 2015 Public Works Design Standards. Gravity pipe size shall be determined by using one-half (1/2) of the maximum gravity flow capacity of the pipe for pipes 15 inches in diameter and less and shall be two-thirds (2/3) for pipes larger than 15 inches in diameter.

5.2 PUMP STATION ALTERNATIVES

Pump station existing conditions were summarized in Section 3. The deficiencies highlighted in Section 3 require relatively minor improvements to resolve. Two alternatives for replacing the Mill Creek pump station flow meter are discussed in Section 5.2.1 below. Recommended short- and long-term pump station condition improvements are summarized in Section 6.

5.2.1 Mill Creek Pump Station

The flow meter downstream of the Mill Creek pump station wet well is broken and cannot easily be replaced by City staff due to the lack of isolation valves or fittings in the valve vault. The vault cannot easily be bypassed due to the influent flow rate and lack of accessible downstream manhole (the downstream force main extends for approximately 4,700 linear feet (LF) before a cleanout is accessible). The following alternatives were considered for replacing the flow meter:

- Alternative 1: Install a bypass line to isolate the existing flow meter vault via plug valves and tapping sleeves. Remove and replace the flow meter vault in place, with additional isolation valves and new magnetic flow meter. The bypass line would remain in place for future maintenance access on the flow meter vault and an end cap on the bypass line would enable future connection.

- Alternative 2: Install a clamp-on ultrasonic flow meter on the discharge force main inside the existing flow meter vault. Straight lengths of pipe are needed upstream (typically 15 LF) and downstream (typically 3 LF) of the flow meter, which pump station asbuilts indicate are available. While a relatively less intrusive and inexpensive option compared to Alternative 1, clamp-on flow meters are difficult to calibrate and often less accurate than in-line magnetic flow meters.

5.3 CONVEYANCE ALTERNATIVES

Conveyance system deficiencies discussed in Section 4 reflect potential overflow and capacity issues. The City has already preliminarily invested in two conveyance improvement projects that were identified in the previous facilities planning study (see Section 6 for discussion of these projects). Alternatives to these projects were not considered.

While the remaining deficiencies do not have multiple feasible alternatives, installation of parallel facilities or taking no action could be considered. The City could choose to construct parallel facilities in areas with limited remaining capacity. This alternative would increase the system's capacity and generally costs less than full replacements. Another advantage of construction parallel facilities is that existing infrastructure could be left in service while the parallel facilities are constructed. The disadvantages of this alternative are the long-term increase in maintenance costs associated with maintaining parallel facilities and the potential higher life-cycle costs associated with the eventual replacement or rehabilitation of the original pipeline / pump station.

Taking no action is not a viable option because surcharging and the potential for overflows would only worsen. This could result in negative impacts to human health and the environment, in addition to fines from the DEQ.

See Section 6 for discussions of cost, environmental impacts, land requirements, potential construction challenges, and sustainability considerations.

SECTION 6 – RECOMMENDED COLLECTION SYSTEM IMPROVEMENTS

This section consists of the recommended plan to address the wastewater collection system deficiencies. The recommended projects presented here have been incorporated into the City Capital Improvement Plan (CIP) in Section 12.

6.1 RECOMMENDED PUMP STATION IMPROVEMENTS

Recommended pump station condition improvements summarized in Section 6.1 account for deficiencies summarized in Section 3.2, including the recommended Gardner pump station displacement. Costs presented in the following tables are planning level estimates and are in 2020 dollars. Actual costs may vary and should be refined further in the pre-design process. Engineering costs assume that multiple pump station projects will be grouped together for project administration efficiencies.

6.1.1 Priority 1 – Address Existing Deficiencies

Priority 1 pump station improvements address existing, short-term condition deficiencies that should be completed in the next six years. Improvement costs are summarized by pump station in Table 6-1. Cost estimate details can be found in Appendix F.

The recommended improvement for Industrial pump station is to install two bollards to protect the access hatches and controls equipment. The recommended project for the Mill Creek pump station is Alternative 1, installing a bypass line and completely replacing the flow meter vault. While more expensive than Alternative 2, it provides better long-term resiliency in terms of flow meter quality and maintenance access to the pump station infrastructure. In addition to the flow meter vault, Priority 1 improvements to Mill Creek pump station include new discharge pressure gauges, 24-inch plug valve, and 4-inch combination air/vacuum valve.

There are no recommended short-term improvements for the Wilco pump station.

TABLE 6-1: PUMP STATION RECOMMENDED SHORT-TERM IMPROVEMENTS

Site	Improvements Cost (rounded)
Industrial Pump Station	$3,000
Mill Creek Pump Station	$267,000
Total Project Costs (rounded)	***$270,000***

6.1.2 Priority 3 – Address Future Deficiencies

The following table summarizes recommended, long-term Priority 3 improvements by pump station (Table 6-2). These projects are identified as Priority 3 projects, not Priority 2, because they are long-term improvements and not urgent deficiencies. These recommended improvements assume that the Gardner pump station is displaced and therefore no additional long-term improvements are necessary for the Gardner pump station during the 20-year planning period. Cost estimate details can be found in Appendix F.

Recommended Priority 3 improvements for both Industrial and Wilco pump stations are to convert the pump stations from dry well to submersible pump stations at the end of the stations' useful life. This will improve future maintenance access to pumps and associated piping and valving.

TABLE 6-2: PUMP STATION RECOMMENDED LONG-TERM IMPROVEMENTS

Site	Improvements Cost (rounded)
Industrial Pump Station	$206,000
Wilco Pump Station	$280,000
Total Project Costs (rounded)	***$486,000***

6.2 RECOMMENDED CONVEYANCE IMPROVEMENTS

This section summarizes the recommended pipeline improvements to address deficiencies from Section 4. All existing system deficiencies increase in the 20-year scenario. Improvements alleviate potential wastewater overflow and surcharging through 20-year planning period. Pipelines are sized based on maximum flow in accordance with City design standards 502.03.E. All pipelines that are replaced, at a minimum, to match the upstream pipeline size. This is considered an industry good practice.

The pipeline replacements described below assume open cut construction unless otherwise stated. Alternatively, the City could utilize trenchless rehabilitation technologies such as pipe bursting, cured-in-place-pipe installation, or slip lining. Under the right circumstances, these approaches can be less costly than the open cut construction approach. Evaluation of the appropriate installation method should be completed as a part of the concept or pre-design phase of pipeline replacement projects.

Improvements are organized by location and are shown in Figure 15 of Appendix A. More detailed planning level cost estimates for recommended improvements can be found in Appendix F.

6.2.1 Priority 1 – Reduce Risk of Overflow

Priority 1 improvements address potential overflows near the downtown core of the City, highlighted in Section 4. The recommended improvements are to upsize approximately 5,400 linear feet (LF) of gravity main on Jetters Way, W Ida Street, and N Evergreen Avenue. Pipes would be upsized from 18-inch and 21-inch to 30-inch from the existing control vault on Jetters Way to W Ida Street at N Gardner Avenue; from 15-inch to 21-inch on W Ida between N Gardner Avenue and N Evergreen Avenue; and from 10-inch to 18-inch on N Evergreen Avenue between W Ida Street to north of High Street. Completion of this project would alleviate potential overflows upstream. A summary of the cost estimate is provided in Table 6.3 below.

TABLE 6-3: PRIORITY 1 COST ESTIMATE

Project	Total Project Cost (rounded)
Upsizing pipeline on Jetters, Ida, and Evergreen	$ 2,943,000

Even after Priority 1 improvements are completed, the hydraulic evaluation indicates surcharging will still occur in parts of the existing conveyance system due to undersized pipes upstream of the

Priority 1 improvements. Given the large expense and historical lack of damage, surcharging in these areas were designated Priority 2 improvements.

6.2.2 Priority 2 – Address Existing and Future Capacity Limitations

Priority 2 improvement projects will alleviate remaining existing and future capacity limitations. Projects are described in detail below based on location and shown in Figure 15.

Priority 2.1 Mill Creek Force Main Extension

The existing Mill Creek force main discharges on Jetters Way, just upstream of the control vault. As identified in Section 4, capacity issues exist for the gravity mains on Jetters Way. Extension of the force main was identified in the previous facilities planning study as means of alleviating these capacity issues on Jetters Way. The City has a completed plan set for installation of approximately 2,750 LF of 26-inch force main from the existing force main to a discharge vault upstream of the WWTP headworks. Design includes boring a segment of the force main under the Power Canal. The cost estimate completed for this plan set was used as a basis for the cost estimate shown in Table 6-4 below.

TABLE 6-4: MILL CREEK FORCE MAIN EXTENSION COST ESTIMATE

Project	Total Project Cost (rounded)
Mill Creek Force Main Displacement	$ 1,190,000

Priority 2.2 Gardner Pump Station Displacement

Capacity issues were identified in the gravity main both upstream and downstream of Gardner pump station in Section 4. Displacing the pump station and rerouting wastewater flows north, to the Mill Creek trunk line, would alleviate capacity issues as well as long term operations and maintenance costs. The pump station would be demolished and approximately 2,170 LF of 12-inch gravity main would be installed along N Gardner Ave to an existing manhole on Shaff Road to route wastewater flows north. The cost estimate for this project is in Table 6-5 below.

TABLE 6-5: GARDNER PUMP STATION DISPLACEMENT COST ESTIMATE

Project	Total Project Cost (rounded)
Gardner Pump Station Displacement	$ 781,000

Priorities 2.3 and 2.4 Evergreen and Ida East Pipeline Upsizing

While the Priority 1 project described in Section 6.1 addresses potential overflow locations, pipelines upstream of this project north on N Evergreen Avenue and east W Ida Street are also undersized. Existing and projected capacity issues can be addressed through upsizing gravity main along these road segments. The north section of pipeline upsizing includes approximately 2,720 LF of 15-inch pipe from the end of Priority 1 project on N Evergreen Avenue north and east to W Locust Street and N First Avenue. The east section of pipeline upsizing includes approximately 2,780 LF of 18-inch pipe from the W Ida Street and N Evergreen Avenue to E Marion Street and N Fourth Avenue. Project extents are shown on Figure 15. Note that each of these pipeline segments involves crossing the Salem Ditch and the cost estimate assumes boring under the waterway.

Modeling indicates that Priorities 2.3 and 2.4 may be phased in either order. Cost estimates for both projects are presented in Table 6-6 below.

TABLE 6-6: EVERGREEN AND IDA EAST UPSIZING

Project	Total Project Cost (rounded)
Upsizing pipeline on Evergreen	$ 1,406,000
Upsizing pipeline on Ida	$ 1,480,000

Priority 2 Phasing

Prioritization was evaluated as a part of the hydraulic capacity analysis. While available capacity in the downstream portion of the gravity system along Jetters Way decreases prior to the force main extension project, no additional overflows result from simulations of the Gardner pump station displacement or Evergreen and Ida East gravity main upsizing occurring before the force main extension project. This indicates that the City may phase the Priority 2 projects based on other selection criteria, such as operation and maintenance costs.

Additional Improvement Projects

The City will continue to budget annually for I/I related improvements. This work will continue to be directed by the I/I based priority improvements highlighted in Section 8 and any additional I/I evaluations completed. Continued coordination with other utility projects could provide cost savings for the City. This work is considered part of the annual replacement budget work for pipelines and manholes. Further discussion of annual replacement budgets is included in Section 12.

6.3 MAPS

Maps of the existing collection system are provided in Figures 8 and 9 of Appendix A. The recommended improvements are shown in Figure 15 of Appendix A.

6.4 ENVIRONMENTAL IMPACTS

Potential impacts of the alternatives to environmental resources presented in Section 2 are described below.

6.4.1 Land Use / Prime Farmland / Formally Classified Lands

The Mill Creek force main extension will require construction through farmland adjacent to a City road that is on the edge of the UGB. It is recommended that impacts to the farmland be minimized during construction of the force main.

6.4.2 Floodplains

As shown in Figure 4, some portions of the study area (including the wastewater treatment plant) are located inside the 100- and 500-year floodplains of the North Santiam River or Mill Creek. None of the alternatives would create new obstructions to these floodplains.

6.4.3 Wetlands

None of the improvements are in wetland areas (Figure 5). Note the Department of State Lands Wetland Determination Request is typically recommended for publicly financed projects.

6.4.4 Cultural Resources

None of the alternatives would interfere with the above-ground cultural resources identified by the National Register of Historic Places or the 2013 Comprehensive Plan. Two improvements (Priorities 2.3 and 2.4) would cross the Salem Ditch, a City-designated historic resource. It is recommended that when these improvements occur, boring or another trenchless method be used to limit impacts to this waterway. Note that State Historic Preservation Office (SHPO) and Tribal consultation is often required for publicly financed projects.

6.4.5 Biological Resources

The IPAC Report in Appendix B provides a general list of threatened and endangered species within the City limits. It is unlikely that any of the plants exist on the proposed project sites since the areas have been previously disturbed and paved or landscaped. Environmental consultations would be required for publicly financed projects.

As Section 2 discusses, the North Santiam River, Mill Creek, Salem Ditch, and Stayton Power Canal have ODFW-listed fish species and/or protected fish habitat. No in-stream work is proposed. It is recommended that boring or another trenchless methods be used for pipeline installation across the Power Canal and Salem Ditch such that impacts to fish species or habitat is limited.

6.4.6 Water Resources

Modifications to the collection system would reduce the risk of overflows and potential to spill into waterways. Design for extension of the Mill Creek force main includes jack and boring under the Power Canal to minimize impacts. It is recommended that sections of the pipeline upsizing projects on N Evergreen Avenue and N First Avenue (Priorities 2.3 and 2.4) be bored so that impacts to the Salem Ditch are minimized. There are no other alternatives that involve stream crossings.

6.4.7 Socio-Economic Conditions

None of the alternatives would have a disproportionate effect on any segment of the population. Equitable wastewater facilities would be provided to all people within the City, limited only by physical geography and overall City budget – rather than by economic, social, or cultural status of any individual or neighborhood.

6.5 LAND REQUIREMENTS

The City would need to purchase easements for construction of the Mill Creek force main extension.

6.6 POTENTIAL CONSTRUCTION PROBLEMS

The depth of the water table and subsurface rock may affect construction of the alternatives. Gravels and sands combined with high groundwater will require extensive dewatering. However, subsurface investigations were not within the scope of this project.

The project area's soil is typical for the area and would require construction techniques normally used to effectively manage excavation, dewatering, and sloughing issues that may arise in Marion County. Construction plans for any of the alternatives would also include provisions to control dust and runoff.

6.7 SUSTAINABILITY CONSIDERATIONS

Sustainable utility management practices include environmental, social, and economic benefits that aid in creating a resilient utility.

6.7.1 Water and Energy Efficiency

Displacement of the Gardner pump station will decrease energy usage as it eliminates two pumps and associated electrical instrumentation from the collection system. Flow would be diverted by gravity to the Mill Creek pump station, which would likely translate to an increase in energy usage at this pump station.

Extension of the Mill Creek force main could potentially increase energy use at the Mill Creek pump station as flow would be pumped an additional 2,750 LF.

Reducing I/I in the collection system will result in a decrease in water and energy usage at the wastewater treatment plant due to an overall reduction in flow needing to be treated.

6.7.2 Green Infrastructure

No new green infrastructure has been proposed with the collection system alternatives.

6.7.3 Other

No other considerations are discussed here.

6.8 OPERATION AND MAINTENANCE RECOMMENDATIONS

6.8.1 Pipeline Cleaning and CCTV

Cleaning and CCTV inspection work has been subcontracted out in the past. Pipelines should be cleaned approximately every three to five years (frequency can be adjusted based on pipe material plus scour conditions and observations by City staff), because a scum buildup will typically form within two years of operation and is a precursor to corrosion. Approximately 63,360 feet/year should be cleaned to cover the entire system every three years. As a general recommendation, concrete pipelines should be CCTV inspected about every five years, as they are more susceptible to corrosion. PVC pipelines should be CCTV inspected about every 10 years, primarily to check for any bellies or sags that may have formed or, pipeline joints that may have separated. Problematic areas may be cleaned and inspected every year or two, or more regularly as required. Areas with adverse grades or large sags may require more frequent attention.

6.8.2 Service Lines

Service lines can be a major source of I/I. Identifying leaky service lines should be a part of regular CCTV inspection work. Additional evaluations of service line conditions should be completed in anticipation of mainline rehabilitation work.

6.8.3 Flow Monitoring

In addition to CCTV inspection, it is recommended the City begin a flow monitoring program to better pinpoint I/I sources and further calibrate the sewer model. Keller Associates recommends that the City complete periodic flow monitoring for areas where I/I is suspected. Flow monitoring could also include night-time monitoring during storm events.

6.8.4 Pipeline Replacement Program

As degrading pipe sections and I/I problems are identified through CCTV monitoring and flow monitoring, Keller Associates recommends that these areas be corrected. Pipeline and manhole replacement and rehabilitation needs are likely to increase as the sanitary sewer collection system ages.

Keller Associates recommends the City begin budgeting for replacement/rehabilitation of an average of 2,750 feet of the collection pipeline system each year. This amount would allow replacement of the entire system within approx. 75 years, the estimated useful life of pipelines. Concrete pipes in the system should be replaced first. The linear feet of pipeline and number of manholes replaced annually is an average and should be adjusted based on future CCTV and other maintenance records. The costs associated with funding an on-going replacement and rehabilitation program are summarized in Table 6.7.

TABLE 6.7: REPLACEMENT BUDGETS

Item	Lifespan	Cost/Year
Pipelines	75 year	$642,000
Manholes	50 year	$116,000
Cleanouts	50 year	$8,000
Laterals/Cleanouts	50 year	$40,500
Total		***$806,500***

Manhole rehabilitation and service line repairs should be coordinated with pipeline rehabilitation work. Priority pipeline replacements/rehabilitation work identified in the CCTV inspections could be funded from this program. Emphasis should be placed on areas where pipe conditions pose the largest threat of sanitary sewer surcharging or a more immediate threat of collapse. Wherever possible, coordinate construction activities with planned roadway projects to minimize construction costs.

SECTION 7 – INFILTRATION AND INFLOW (I/I)

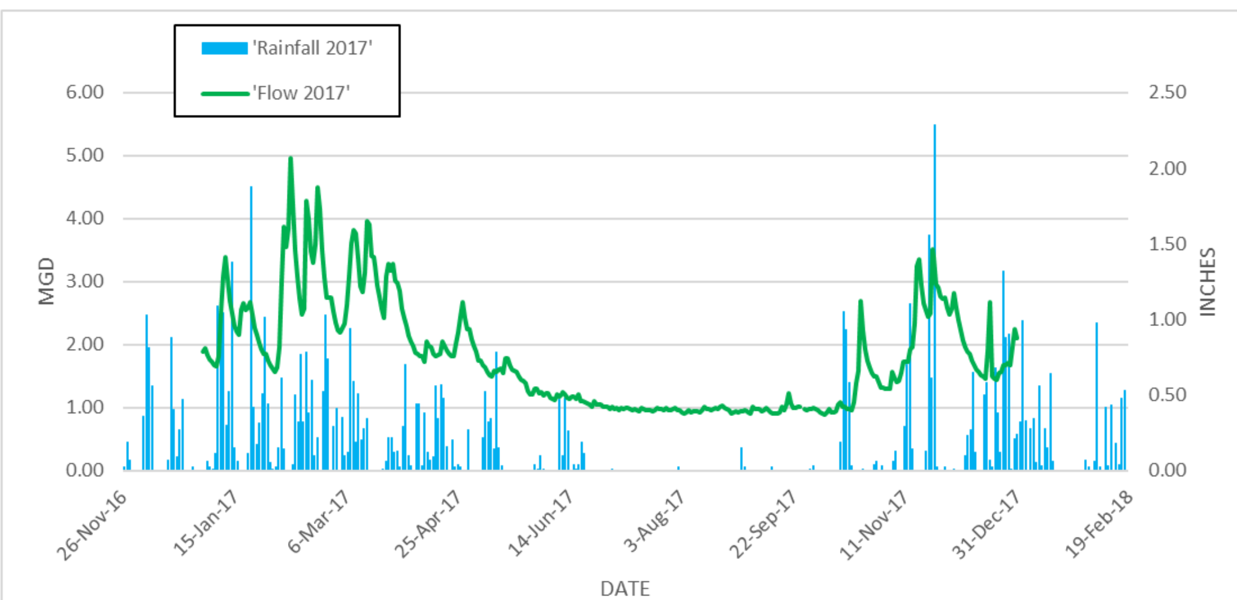
This section summarizes past and current City efforts to evaluate and reduce infiltration and inflow (I/I) from the collection system. Infiltration refers to groundwater that enters the wastewater collection system through leaky pipes and manholes. Inflow refers to storm water that enters the sewer system through any number of sources, including the holes in manhole lids as well as roof drains and storm catch basins connected to the sewer system. The data collection and analysis completed as part of the facilities planning study was completed in stages to prioritize efforts and identify areas with high I/I, ultimately identifying priority rehabilitation projects.

7.1 BACKGROUND

The City of Stayton wastewater collection system consists of over 36 miles of gravity pipelines ranging from 6 to 24 inches. The 2008 Wastewater Facilities Planning Study identified a noticeable trend between average daily precipitation and flow through a comparison of WWTP flows and rainfall events. It was concluded that this close correlation could indicate either inflow or shallow groundwater infiltration. This study includes a pump run time analysis, extensive flow monitoring, CCTV report analysis, night-time flow monitoring, and smoke testing to generate a prioritized list of the top 25 I/I reduction projects in the study area, as well as a list of cross connections found while smoke testing, and spot repair needs identified through CCTV inspections.

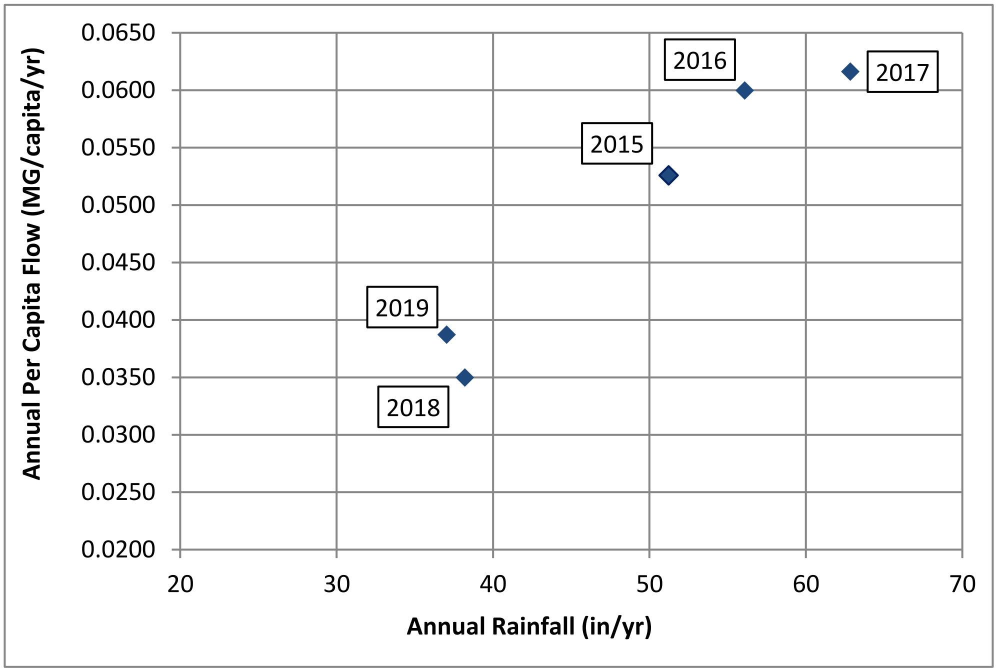
Visual evidence of I/I influence in the system can be seen in Chart 7-1, which shows the 2017 daily flows and precipitation recordings at the treatment plant site. The rapid response between precipitation events and higher flows suggests that a significant component of peak flow is from storm water inflow. The sustained increase in flow over several days following a large storm event suggests that groundwater is also infiltrating into the City's wastewater collection system. Flows for 2017 are representative of previous years.

CHART 7-1: 2017 DAILY FLOW AND PRECIPITATION



Evidence of I/I influence can also be seen by comparing annual rainfall against annual per capita flow. Chart 7-2 below shows a positive linear relationship between rainfall and normalized flow over the range of rainfall observed for 2015–2019 The following sections detail the I/I efforts completed for this planning study.

CHART 7-2: ANNUAL RAINFALL VS. PER CAPITA FLOW



7.2 FLOW MONITORING

Continuous flow monitoring was completed for 9 weeks during January - April 2020 to better characterize the nature and distribution of I/I in the system. Six flow monitors were placed throughout the system (See Figure 11 Appendix A) based on City staff recommendations, previous I/I study data collected, and land use considerations. Flow monitoring equipment provided by Keller was used to collect level, velocity, and flow data in 10-minute intervals. Rainfall data was collected at the WWTP weather station in 15-minute intervals.

Appendix C shows flow and precipitation data over time for all the flow monitoring sites. Flow monitoring basins 1 & 3 are sub-basins of Site 2. Chart 7-3 illustrates the flow vs precipitation for Site 2, which is located at the southern end of the City. Basin 2 (considering the influence of basins 1 and 3) was determined to have the most influence from I/I. Chart 7-4 is for Site 4, the Washington St basin just downstream of the Gardner Pump Station. This was the second highest flow during a rainfall event and the other basin with highest indication of I/I. As a result of this analysis, subsequent phases of monitoring (e.g., night-time monitoring and analysis, CCTV, and smoke testing) were focused in the service area upstream of these basins.

CHART 7-3: FLOW MONITORING BASIN 2 IDA ST

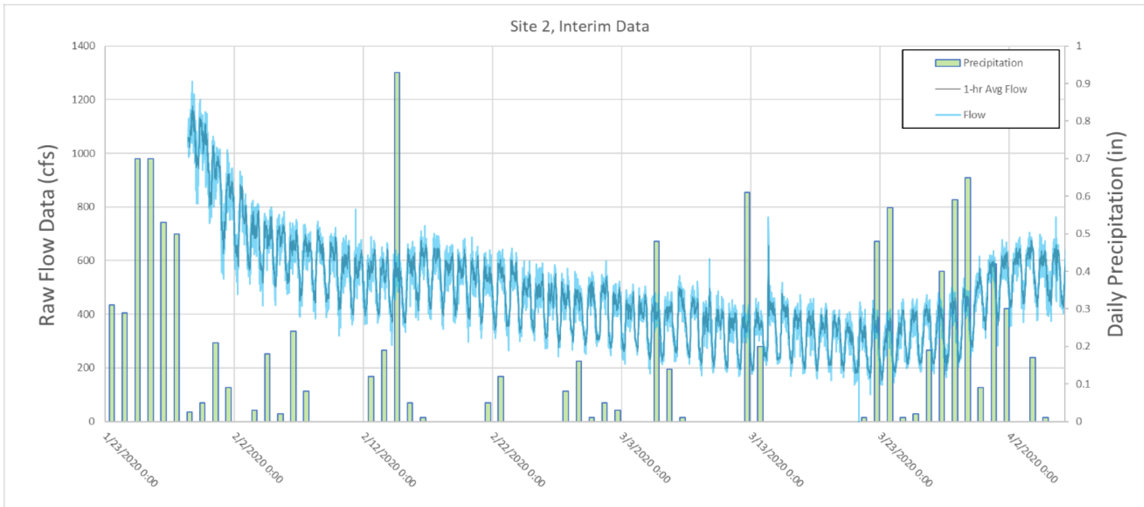
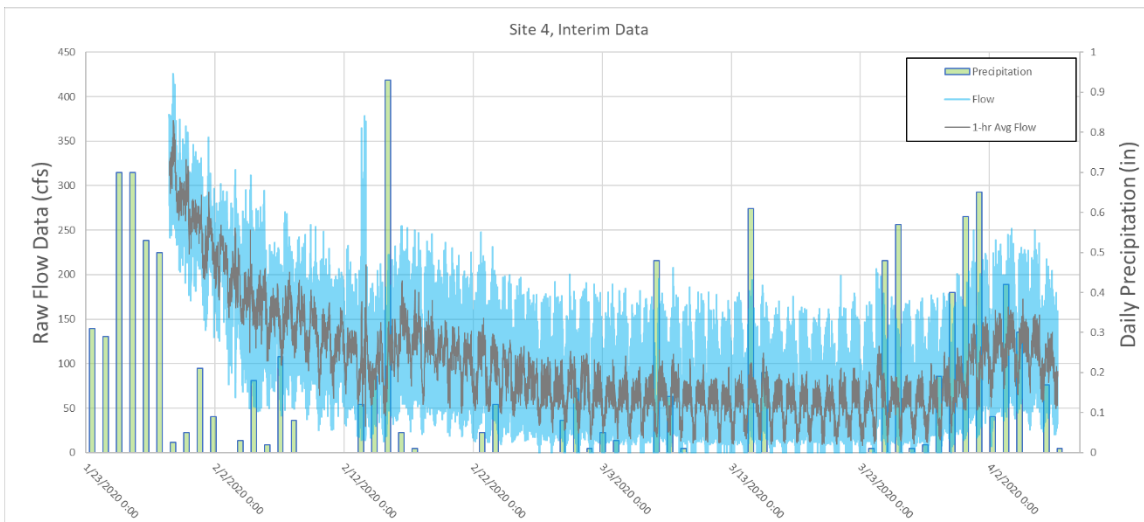


CHART 7-4: FLOW MONITORING BASIN 4 WASHINGTON ST

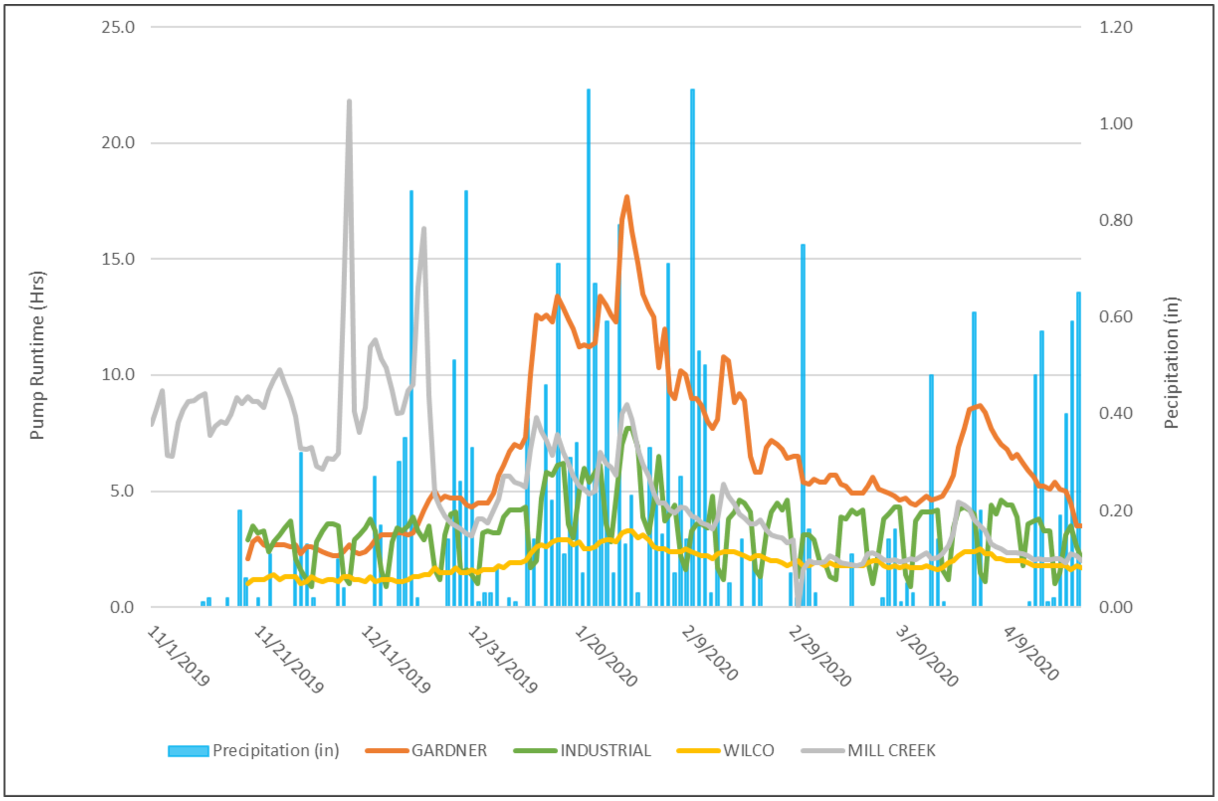


7.3 PUMP RUN TIME ANALYSIS

Three of the four City-owned pump stations (Gardner, Industrial Area, Mill Creek, and Wilco) were visited to complete pump flow tests and facility evaluations. Gardner pump tests and evaluation were not included in the scope of this study because displacement of the pump station is planned for the near future. City staff provided pump station history and anecdotal performance records. The pump stations and their service areas are shown in Figure 10 (Appendix A).

The daily run times for all four public pump stations were analyzed. Chart 7-5 shows the results for all four pump stations. The City does not currently trend daily run time data, so data collected for November 2019 – April 2020 was used to perform this pump run time analysis. When daily run times are compared with rainfall events, a close correlation between high rainfall months and monthly increase in run times is evident. This correlation indicates that I/I is the likely cause of increase in flow.

CHART 7-5: STAYTON PUMPSTATION RUN TIME VS PRECIPITATION



In order to compare high daily run times caused by I/I against average daily flows, several peaking factors (PF) were calculated. Peaking factors compare flows during dry periods to flows during rainfall events. A higher peaking factor indicates more I/I in the pump station service area. The results of these analyses for November 2019 –April 2020 are summarized in Table 7-1. The peaking factors are color scaled from red (highest I/I ratio) to green (lowest I/I ratio). Of the four pump stations, Gardner had the highest peaking factors, which suggests the highest ratio of I/I to average flow in its service area.

TABLE 7-1: PUMPSTATION PEAKING FACTORS

	Gardner	Industrial	Wilco	Mill Creek
Pump Rate	300 gpm	150 gpm	900 gpm	3,170 gpm
PF AVERAGE	5.3	2.1	2.4	1.0
PF MAX	6.4	3.0	2.6	1.3

The highest daily pump run time at the Gardner pump Station was 6.4 times the average daily pump run time for November 2019 – April 2020. The Wilco pump Station was second for I/I based on run time peaking factors, followed closely by Industrial.

It is recommended that the City continue to record daily pump run time data and review the data every couple of years to establish trends and prioritize rehabilitation efforts. It is also suggested the City install permanent flow meters and pressure gauges at all pump stations to better track I/I and pump performance. These instruments should be connected to the SCADA system to allow for continuous monitoring, recording, and trending.

7.4 CCTV CLEANING AND INSPECTION

The City cleans and closed-circuit television (CCTV) inspects approximately 1/5 of the pipes in the collection system each year. This equates to the entire system being cleaned and inspected approximately every five years. The City uses the National Association of Sewer Service Companies' (NASSCO) pipeline assessment certification program (PACP) to record defects and grade pipe condition during closed-circuit television (CCTV) inspections. This program creates a comparable baseline for the pipelines studied and allows for the tracking of pipe condition over time. The program operates by creating specific codes for the various defects found in pipelines. In theory, if multiple operators were to inspect the same pipeline they would generate "identical" PACP reports. While this may not be exact, it is a method of standardization of CCTV inspections. PACP defects are separated into two categories: structural and O&M. The types of defects for each category are listed in Table 7-2.

TABLE 7-2: PACP STRUCTURAL AND O&M DEFECTS

Structural	O&M
Cracks	Settled and Attached deposits
Fractures	Root intrusion
Break in pipe/Holes	Defective taps/laterals
Collapse	Infiltration
Deformation	Sags
Defective joint	Obstacles/Obstructions
Surface Damage/Corrosion	Vermin
	Angled alignments

Operators record and code observations and defects during the CCTV inspection. From this coding, the PACP software assigns a grade of 1 to 5 to each defect, with 1 being a minor defect and 5 being the most significant defects. Table 7-3 indicates the general assignment of condition grades from NASSCO.

TABLE 7-3: GENERAL ASSIGNMENT OF PIPE CONDITION GRADES

Grade 5	Collapsed or collapse imminent
Grade 4	Collapse likely in foreseeable future
Grade 3	Collapse unlikely in near future
Grade 2	Minimal collapse risk
Grade 1	Acceptable structural condition

After grading all the defects, the software generates a PACP report, which has three different types of ratings: pipe rating, quick rating, and rating index. The pipe rating is the sum of the number of each grade of defect multiplied by the defect grade. For example, a pipe with four grade 5 defects would have a rating of 20. The quick rating is a four-digit number that indicates the highest-grade defect (first digit) and the number of its occurrences (second digit), the second highest grade defect (third digit), and the number of its occurrences (fourth digit). For example, a pipe with two grade 5 defects, no grade 4 defects, one grade 3 defect, and any number of grade 2 and 1 defects would have a quick rating of 5231. If there are more than nine of a defect grades then a letter code is used to the number of defects as follows: A – 10 to 14, B – 15 to 19, C – 20 to 24, etc.

The rating index is an average severity of defects along the pipe. The rating index is calculated by the pipe rating divided by the total number of defects. Each of the three ratings are separated into structural, O&M, and overall ratings, resulting in a total of nine (9) ratings per inspection. The structural ratings are calculated using only the structural defects, and the O&M ratings are calculated using only the O&M defects (Table 7-3).

The last five years of CCTV inspections, approximately the entire collection system, were reviewed and analyzed to develop a list of recommended spot repairs and combined with other I/I analysis to prioritize I/I reduction projects in the system. City staff have individual records of each of the PACP reports analyzed, which are not included in the Appendix of this study. Some inspections were abandoned because a defect (such as a root ball or protruding tap) made it impossible for the camera to continue. Reverse inspections were performed for some abandoned inspections, but not all. Reverse inspections were noted in the data, and PACP scores from the two inspections were combined.

Throughout the inspections, the most common O&M defects found were roots, intruding taps, infiltration, and dirt or gravel in the pipe and laterals. The most frequent structural defects were cracks, fractures, and holes or breaks.

Figure 16 (Appendix A) shows the highest-grade defect along a pipe length. There are 26 pipes that have at least one grade 5 defect, and an additional 42 pipes that have at least one grade 4 defect. These pipelines have partially collapsed/failed segments or segments that are near collapse/failure. Most of the grade 5 and 4 defects discovered during the 2015-2019 inspections were O&M defects: infiltration or root intrusion. Some of these are located within pipeline segments where full length rehabilitation or replacement is recommended. A localized spot repair may be appropriate for other grade 5 defects. All grade 5 defects should be repaired in the immediate future. Recommended I/I improvements are discussed in Section 8.

It is recommended the City continues using the PACP format for future video inspections. The PACP format provides the City an industry standard, objective analysis and allows the condition of the same pipe to be compared over time. This could be helpful in tracking the deterioration of pipes, completing preventative maintenance activities, and identifying and correcting problems before a pipe fails.

7.5 NIGHT-TIME MONITORING

Visual night-time investigations were performed to better identify sources of I/I. Night-time monitoring was focused in Basins 2, 4, and 6; based on results of flow monitoring, pump run time, and CCTV scoring. The period of low flow (2:00am - 6:00am) was approximated from the flow monitoring graphs. Monitoring was performed during this time because flow in the sewer is almost entirely from I/I as there is minimal contribution from users during this time. Night-time monitoring was carried out following consecutive days of precipitation, which maximized the shallow groundwater levels (infiltration) and to view the highest levels of I/I.

A field visit was carried out by Keller Associates between 2:00am – 5:00am, on Tuesday June 9th, 2020. It rained 0.84 inches during the previous three days (6/6-6/8). Table 7-4 lists the daily rainfall for the week prior to night-time monitoring. It rained intermittently during night-time monitoring on June 9th.

TABLE 7-4: DMR RAINFALL 6/6/2020 – 6/9/2020

Date	Daily Rainfall (in)
6/6/2020	0.1
6/7/2020	0.76
6/8/2020	0.07
6/9/2020	0.32

Observations were performed beginning at the farthest downstream manhole in a pipeline system. The velocity and depth data were visually approximated from manhole to manhole, moving upstream until the I/I flow was essentially zero. Manholes where the line branches were targeted so that visual estimations could identify which branch(es) were contributing the most I/I flow. The branch(es) that had the most I/I flow were then followed upstream to narrow the basin contributing the most I/I. Additional velocity and flow depth data were estimated for manholes in between two inspected manholes when a significant decrease in flow was observed.

Depth data was approximated using a survey rod and a level placed across the top of the manhole. The survey rod was lowered into the manhole to the bottom of the inlet. The depth was measured by reading the height of the level when the rod was at the bottom of the inlet, and then again when the rod was raised so the bottom of the rod was at the top of the flow. Velocity was then estimated visually using the time for particles in the flow to travel through the manhole width. Approximate depths and velocities were recorded from these observations.

Night-time monitoring is visual and a relative indication of areas of high I/I. These priorities will be considered with the results of the other testing done for the study to make recommendations for system-wide I/I rehabilitation project prioritization.

7.6 PRIORITIZATION

The first step in developing a score for the prioritization process utilizes the CCTV inspection results. After reviewing the overall PACP ratings for the pipelines inspected in Stayton's system, it was found that many of the overall ratings were skewed for O&M items that could be managed by the City (e.g. removal of gravel in pipeline). Therefore, in developing the initial prioritization scoring, the PACP structural ratings were weighted more heavily than the O&M ratings. An adjusted overall PACP rating for each pipe segment was calculated by giving the structural rating 60% of the weight and operational rating a 40% weight. This rating was then normalized by dividing the total PACP rating by the pipeline length. A segment's score was reduced by 50% if the failure has been spot repaired. Pipeline segments were then given a 1-10 score using breaks in score distribution and review of inspections near scoring thresholds. Figure 17 (Appendix A) Illustrates this step.

The second step in the prioritization process was to consider the pipeline age/materials. A 60-year-old pipeline with similar deficiencies to a pipeline that is 30 years old should have a higher priority for replacement. Most pipelines of a given material were installed in the similar time period. Age and material were incorporated into the pipe condition score as additional points added to the PACP score. Table 7-5 shows the points added to a pipeline's score based on material data

TABLE 7-5: MATERIAL / AGE ADJUSTMENTS

Material	Adjustment
Asbestos Concrete	+2
Concrete	+1
PVC / Fiberglass Pipe	+0.5

The third step in prioritizing improvements was to consider night-time monitoring observations. The night-time monitoring estimated flow rate was accounted for in a score adjustment increase. Table 7-6 shows the points added to a pipeline's score based on night-time flow rate in gallons per minute (gpm).

TABLE 7-6: NIGHT-TIME MONITORING FLOW ADJUSTMENTS

Night-time Flow (gpm)	Adjustment
Flow > 35	+3
35 > Flow > 10	+1
10 > Flow	+0.5

The condition score of a pipe was the sum of the PACP score and the two score adjustments. The range for this score is 0 to 15 (0 to 10 for PACP, 0 to 2 for material/age, and 0 to 3 for night-time flow). The highest score a pipeline in the study received was 12.5. Figure 18 (Appendix A) shows the condition score of pipe segments. After completion of the comparison checks, it was determined that night-time flow observations had minimal- to no-impact on the condition scores of the pipe segments that had the largest existing condition scores.

The risk of failure to the City is a function of both the likelihood of failure (pipeline condition) and the consequence of failure. For example, a pipeline failure that services a small residential cul-de-sac will have a much smaller impact than a larger interceptor that services a business district or school/hospital. Consequence of failure was incorporated into the prioritization process by using multiplying factors. Table 7-7 shows the parameters and factors applied for consequence of failure. If one of the parameters applied to a pipeline segment, that pipeline segment's condition score was multiplied by the corresponding factor. If multiple parameters applied to a pipeline, it was multiplied by each factor. For example, an 18-inch interceptor pipeline that runs through the commercial zone would have its condition score multiplied by 1.2 and then by 1.1 to calculate its final I/I impact score. The I/I impact score -- the condition score multiplied by all applicable consequence factors -- was used to develop the preliminary pipeline rehabilitation/replacement prioritization schedule for the pipe segments in the study area. Figure 19(Appendix A) illustrates I/I prioritization rankings based on pipe segment impact scores.

TABLE 7-7: CONSEQUENCE OF FAILURE FACTORS

Parameter	Factor
If commercial or industrial zone	x 1.1
If school trunkline	x 1.1
If next to creek	x 1.2
If hospital trunkline	x 1.3
If interceptor >/= 18"	x 1.2

It is believed that a CCTV inspection is a critical component in making a final pipeline condition assessment for recommending pipeline rehabilitation/replacement near to the time the work will be completed. This study assumes that the City of Stayton inspects their entire system on a 5-year recurring basis.

7.7 SMOKE TESTING

Keller Associates smoke tested approximately 100,000 linear feet of the sanitary sewer mainlines system on August 24th-September 3rd, 2020 (Figure 20 in Appendix A). The City of Stayton notified all property owners within the smoke testing area one week in advance of testing. City staff hung notifications on doors for those areas that would be affected by the smoke testing prior to August 24, 2020 emergency services and dispatch were notified one week prior to and again each day with updates as to the daily location of smoke testing.

Keller Associates provided the smoke testing equipment, which consisted of two Hurco Power Smokers, LiquiSmoke, and road signs. The smoker introduces smoke in the sanitary sewer system through the top of a manhole. The two smoker assemblies were run at the same time, approximately two manholes apart. Smoke introduced into the sanitary system should only be released from nearby manholes, cleanout pick holes, and building plumbing vents; smoke emitted anywhere else indicates a potential source of I/I.

Throughout the 18.9 miles of pipe smoke tested, 40 total problem locations were noted. There were no illegal vents, 9 cross-connections, 33 cleanouts, 2 possible laterals, and 1 other problem noted during smoke testing. These sites and concerns are summarized in Table 7-8 below and Figure 21 in Appendix A. Photos and field notes of each problem are also presented in Appendix G. The main problems found, reason for concern, and recommended actions are listed below:

- Broken or open cleanouts (C/O)
  - Can collect localized storm water, especially if located near a low point
  - Notify property owner and seal C/O
- Leaking laterals
  - Allow high infiltration into the sewer system
  - Notify property owner and repair lateral
- Cross-connections
  - Consist of direct connections to the sewer system that should be connected to the storm water system instead, such as roof drains and storm water catch basins
  - For cross-connections on private property, notify property owner and have cross-connection removed
  - For cross-connections on City property, investigate to confirm cross-connection, remove cross-connection
- Additional observations from smoke testing:
  - New manhole discovered.

TABLE 7-8: RECORD OF SMOKE TESTING PROBLEM LOCATIONS

Picture ID	Defect Type	MH Tested	Smoke Intensity	Recommended Action	Photo
1	Cleanout	1010-26	Mild	Seal Cleanout	Y
2	Cleanout	1010-26	Mild	Seal Cleanout	Y
3	Cleanout	1008-12	Mild	Seal Cleanout	Y
4	Cleanout; Broken Lateral	1112-13	Mild	Seal Cleanout; Investigate	Y
5	Cross-Connection	1112-16	Heavy	Investigate	Y
6	Cleanout	1015-24	Mild	Cap Cleanout	Y
7	Cleanout	1015-15	Heavy	Cap Cleanout	Y
8	Cross-Connection / Indoor Plumbing	1015-15	Mild	Investigate	Y
9	Cleanout	1015-30	Light	Seal Cleanout	Y
10	Cleanout	1002-01	light	Seal Cleanout	Y
11	Cleanout	1015-23	Mild	Cap Cleanout	Y
12	Cleanout	504-03	Light	Seal Cleanout	Y
13	Cross-Connection	1013-03	Light	Investigate	Y
14	Cleanout	1013-03	Heavy	Seal Cleanout	Y
15	Cleanout	916-03	Mild	Cap Cleanout	Y
16	Cleanout	1007-17	Light	Investigate	Y
17	Cross-Connection / Indoor Plumbing	1009-10	Mild	Investigate	Y
18	Cross-Connection	1009-10	Mild	Investigate	Y
19	Smoke from Crawl Space	1007-12	Very Light	Investigate	Y
20	Cleanout	1007-04	Light	Seal Cleanout	Y
21	Cleanout	1007-04	Light	Seal Cleanout	Y
N/A	Cleanout	1006-08	Heavy	Seal Cleanout	N
22	Cleanout	1004-03	Mild	Seal Cleanout	Y
23	Cleanout; Cross-Connection	1001-06	Heavy	Investigate	Y
24	Street Cleanout	1104-20	Light	Seal Cleanout	Y
25	Cleanout	1106-10	Light	Cap Cleanout	Y
26	Cleanout	1106-16	Light	Seal Cleanout	Y
27	Cleanout; Broken Lateral	1007-15	Heavy	Seal Cleanout	Y
28	Cleanout	1008-04	Mild	Seal Cleanout	Y
29	Cleanout	1007-13	Low	Seal Cleanout	Y
30	Cleanout	1015-15	Light	Seal Cleanout	Y
31	Cleanout	1006-08	Low	Seal Cleanout	Y
32	Cleanout	316-05	Heavy	Cap Cleanout	Y
33	Cleanout; Cross-Connection	316-02	(1) Low (2) Heavy	Seal Cleanout, Investigate Cross-Connection	Y
34	Cleanout	1001-06	Low	Seal Cleanout	Y
35	Cross-Connection	1106-07	Light	Investigate	Y
36	Cleanout	901-11	Mild	Seal Cleanout	Y
37	Manhole	901-15	Mild	Seal Manhole	Y
38	Cleanout	1004-13	Mild	Cap Cleanout	Y
39	Cleanout	908-28	Mild	Seal Cleanout	Y
40	Cleanout	1004-03	Mild	Cleanout; Broken Lateral	Y

Estimations of the cost and associated benefits of removing cross-connections identified by smoke testing are addressed in the Potential I/I Reductions in Section 7.8. Recommended actions to reduce I/I from defects identified through smoke testing are discussed in Section 8.

7.8 POTENTIAL I/I REDUCTIONS

The first course of action that can reduce I/I in a system is to repair defects in the collection system. During storm events or day-to-day activities, water can infiltrate into pipes through defects such as breaks, cracks, holes, or other structural defects. If many defects are discovered in a single pipe, replacement or rehabilitation of the full pipe should be considered. Options for full pipe repair include open trench repair/replacement or trenchless rehabilitation. Both options should be considered for their ease of use and overall cost to the City, explained in Section 7.7 of this report. If the overall pipe is in good condition, but contains single or a small number of defects, then a spot repair may be more appropriate.

Additionally, elements such as cleanouts, swales, house drains, and catch basins may be directly connected to the collection system. During smoke testing, sources of storm water inflow were identified, and the storm water runoff methodology referred to as the rational method was used to determine inflow. Table 8-3 lists these cross connections, their estimated inflow, and estimated cost per gpm to eliminate the cross-connections.

Five cross-connections were identified as potential storm drains into the sanitary sewer system. The driveway, area, and roof drains are the most cost-effective to repair. Owners whose roof drains were found to be connected to the sewer system should be notified and required to disconnect them from the sewer system, rerouting them to the yard or street or reconnecting them to the storm system per Stayton Municipal Code 13.24 930. There should be minimal cost to the City to have property owners disconnect their roof drains from the sewer.

The City should disconnect the area drain connected to the sewer system. These connections should be verified by the City with tracer dye tests and video inspections. Improvement costs for each of these repairs have been estimated in Table 8-3 in Section 8. The benefit of removing these sources of storm water inflow is primarily capacity related. Reduced flows result in lower risk of sanitary sewer overflows and have the potential to offset or delay capital expansion projects for the collection and treatment systems that are triggered by hydraulic capacity.

The data available covered 2018 through 2020 for the wastewater fund, including the budgeted line items and the actual costs incurred. The total expenditures including debt service were approximately $3 million for both years. The wastewater expenses can be separated into two categories: fixed and variable. The fixed costs are those that remain the same whether I/I is removed (i.e. most equipment, personnel, etc.). Variable costs are those that can be reduced if I/I were reduced (i.e. chemicals to treat, electrical bills, equipment repair, supplies, etc.).

Line items for the Operations (WWTP) and the WW Collection were reviewed to determine those that include a variable component. The percentage of the line item attributable to variable flows was estimated, and all variable costs were summed up and then divided by an approximated average daily flow to arrive at a cost per gpm due to variable costs. On average, it costs approximately $251 for every gpm of the average annual flow. The City can evaluate for varying payback periods, but if using 10 years, a repair cost should be less than $2,510 per gpm to be justified. If a longer payback period is used, a higher repair cost can be justified.

This is a planning level evaluation of the cost to convey and treat inflow and infiltration. If the City desires, a much more detailed evaluation can be performed to break out the variable costs more accurately. At the end of the day, the City needs to identify and repair I/I where feasible and practical. The cost to convey and treat should only be used to limit the amount of money spent on I/I reduction if the system had very limited amounts of I/I. Please note, due to the potential offset or delay to treatment plant or other capital improvements if I/I flows are reduced, the evaluation summarized above does not account for these savings. These savings have the potential to be much larger than pipeline and lateral rehabilitation costs.

7.9 REPLACEMENT / REHABILITATION COST ESTIMATES

Planning level costs were developed for replacement projects based on the length of pipe. The budget estimate of $235 per linear foot assumes open trench installation, 8-inch to 12-inch pipeline replacement as well as lateral replacement (within the right-of-way), installation of cleanouts at the property line, and manhole replacements. The cost also includes a 20% contingency and a 15% cost for engineering and construction management services.

If open trenching proves to be too disruptive in certain areas, there are alternative trenchless rehabilitation techniques. Two of the more common techniques are pipe bursting and use of cured-in-place-pipe (CIPP). Pipe bursting is often used if a pipe needs to be upsized by one nominal size (e.g. 10-inch in diameter to 12-inch in diameter). CIPP involves the use of a textile liner tube and liquid resin, which cures in place, and is more common when the pipe does not need upsizing. Depending on the application, the City could realize a potential project saving of 20-40+% by using trenchless technologies instead of open trench replacement.

Trenchless technologies may be ideal for areas with high traffic because there is no trenching and the process can often be done in one night. However, trenchless technologies may not be recommended where there are many laterals that need to be replaced, pipeline sags, or other large defects that require spot repairs. For CIPP projects, roots and intruding taps must be removed before using CIPP. Spot repairs can also be done using CIPP. As part of the project pre-design, the City should perform further CCTV inspection to gather the most current information and evaluate each project and defects present to decide the most appropriate rehabilitation technique. For example, while trenchless rehabilitation may be cheaper than replacement, it is not possible to CIPP a collapsed pipe.

7.10 RECOMMENDED OPERATIONAL AND ADMINISTRATIVE PRACTICES

After completing replacement or rehabilitation of pipes in the priority CIP areas or on the spot repairs list, it is recommended that the City re-inspect the pipes using CCTV. One common mistake in I/I projects is that it is assumed the new or rehabilitated pipe completely fixes the inflow or infiltration problem, and then efforts are focused elsewhere. However, it is not uncommon to see new inflow/infiltration problems into the pipe arise at a different portion of the pipe after one problem is addressed, especially in cases of spot repairs or where the pipe is below the groundwater table. Often, water that was leaking into the pipe through one defect will migrate to other defects and continue infiltrating. Continued CCTV monitoring after project completion will help identify if the project repaired the defects and identify any new defects, so that efforts can be appropriately directed towards defects in the system.

Additionally, continuous flow monitoring should continue to take place in the system and in the influent of the wastewater treatment plant. As peaking factors are a primary indicator of I/I, it is important to collect data and track flow. Comparing flow in the collection system during drier periods to wetter periods will provide a peaking factor. One indication of a successful I/I program is a continuous decrease of the peaking factor as more defects are corrected. Through continuous monitoring and data collection, the City should be able to determine the effectiveness of its I/I program in the coming years.

It is recommended that the City continue the established routine pipeline cleaning and inspection schedule of the collection system. Routine cleaning of the pipes can remove debris buildup, which can cause unnecessary pressure/strain on the pipes and remove root intrusions. Routine cleaning can break off root intrusions before the root itself grows and expands the existing defect in the pipe, potentially saving the cost of replacement or rehabilitation in the future. A more detailed description of operation and maintenance recommendations including staffing recommendations can be referenced in Section 6 of this report.

Finally, it is highly recommended that the City continues open interaction and involvement with its constituency about the nature of projects and work being completed. Public forums, town halls, flyers, and bulletins are potential methods to disseminate information and receive feedback from the public. Prior notice should be given informing residents of disturbances from projects including approximate timeline of the repairs, especially in cases of pipeline work on busy streets or in commercial areas.

SECTION 8 – RECOMMENDED INFILTRATION AND INFLOW (I/I) IMPROVEMENTS

8.1 RECOMMENDED IMPROVEMENTS

Tracking and identifying sources of I/I was completed through pump run time tests, continuous flow monitoring, video inspections, smoke testing, and night-time monitoring. The top priorities for rehabilitation/replacement/spot repair, and cross-connections identified during smoke testing, are contributors to the I/I in the system. Recommendations and top priority projects are summarized in the following sections. Prior to replacement or rehabilitation, it is recommended that all trunk lines be video inspected, and the results compared with recommendations in this section and Section 6 of this master plan to re-evaluate project priorities for these pipelines.

8.1.1 Prioritized Improvements for Pipelines

Using the methodology described in Section 7, the top 100 pipe segments from the 2015-2019 inspections were considered by score and grouped by location to create logical rehabilitation projects for the City. The recommended improvements were compared to recent rehabilitation/replacement projects (within the last four years) provided by the City and recommended CIP improvements that address capacity deficiencies (see Section 6 for collection system CIP descriptions). Projects that had been replaced or rehabilitated recently were not included in these I/I recommendations. This study assumes that the complete collection system has been CCTV inspected in the past 5 years in accordance with the City's current inspection schedule. The top priority projects and associated pipe segments are listed in Table 8-1 below. Figure 22 (Appendix A) shows the location of each project. Noted on Figure 22, I/I Projects 1 and 2 overlap with CIP Priority 1. I/I Projects 12 and 15 overlap with CIP Priority 2.2, and I/I Project 4 overlaps with CIP Priority 2.3. The available data was reviewed for each of the priority projects to create a project sheet, which highlights the defects found for the pipes, makes some suggestions for rehabilitation techniques (if applicable), and gives a conceptual level opinion of probable costs (Appendix G)

During the most recent round of inspections, some pipes did not receive a full-length inspection. These inspections are considered "abandoned," and should be cleaned and re-inspected by the City to ensure that there are no defects along the remainder of the pipe length.

TABLE 8-1: PROJECT PRIORITIZATION FOR PIPE SEGMENTS

Project Priority	Pipe Segment	Manhole	Manhole	Material	Diameter (in)	Risk Score	Legnth (ft)	Total Length (ft)	CIP Year
1	L2574B	916-07	916-05	RCP	18	18.9	292	1104	1
1	L2574A	916-08	916-07	RCP	18	12.2	209		
1	L24	1013-04	1013-03	RCP	15	11.4	173		
1	L25	1013-03	916-08	RCP	15	11.3	430		
2	L32	915-16	915-02	RCP	21	15.8	134	134	1
3	L145	1011-10	1011-08	RCP	8	13.8	240	778	1
3	L144	1011-09	1011-08	RCP	8	11.4	83		
3	L146	1011-11	1011-10	RCP	8	4.0	95		
3	L147B	1010-14	1010-11	RCP	8	6.4	360		
4	L77	1015-26	1015-25	RCP	8	13.2	94	94	2
5	L87	1015-11	1015-01	RCP	8	15.2	349	723	2
5	L78	1015-01	1015-18	RCP	8	9.8	374		
6	G54	907-08	902.12	AC	8	12.5	351	351	2
7	L408	1004-06	1004-24	RCP	8	11.5	34	795	2
7	L405	1004-12	1004-11	AC	8	10.1	386		
7	L406	1004-11	1004-07	RCP	8	6.1	152		
7	L409	1004-24	1004-09	PVC	8	3.6	223		
8	L9	914-10	914-02	RCP	6	11.1	225	348	3
8	L10	914-02	914-03	RCP	8	4.9	123		
9	L174	1008-07	1008-06	RCP	8	11.1	29	29	3
10	L620B	901-24	901-09	PVC	8	11.0	15.5	15.5	3
11	L376	1006-01	1011-10	FRP	8	11.0	459	459	3
12	L411	1004-17	1004-05	RCP	8	10.2	130	204	3
12	L410	1004-09	1004-17	RCP	8	9.2	74		
13	L272	1001-06	1001-05	RCP	8	10.1	440	648	3
13	L274	1001-04	1001-05	RCP	8	5.0	208		
14	L436	908-16	908-04	AC	6	2.5	265	265	4
15	L413	1004-02	1004-23	RCP	8	9.6	77	77	4
16	L576	915-22	915-14	RCP	8	9.2	57	57	4
17	L354	1002-08	1007-08	RCP	8	8.9	379	379	4
18	L464	907-10	907-11	AC	6	8.8	358	592	4
18	L465	907-11	907-12	AC	6	4.9	234		
19	L505	915-07	915-06	RCP	8	8.6	167	167	4
20	G73	415-13	415-12	PVC	8	6.0	226	226	4
21	L504	915-04	915-03	RCP	10	8.3	225	372	4
21	L19	914-08	915-03	RCP	8	5.8	147		
22	L47	1015-15	1014-01	RCP	8	8.7	428	428	5
23	L95	1015-03	1015-02	RCP	8	5.2	157	157	5
24	L3043	1001-07	1104-14	RCP	8	6.7	77	503	5
24	G71	1001-26	1001-07	RCP	8	3.5	426		
25	L140	1011-07	1011-06	RCP	10	6.6	303	303	5

Using the project sheets in Appendix G, it is recommended that the City begin to remedy the priority pipe segments in Table 8-1. Additionally, the City can use this document as a resource to identify future pipe rehabilitation projects and can be used as a reference when making future infrastructure improvements to provide potential cost savings to the City by grouping infrastructure projects. For example, if a roadway containing a defective pipe segment is being improved or replaced, combining the two efforts into one project could save the City time and lower costs.

8.1.2 Spot Repairs / Cross Connections

Some pipelines may be in relatively good condition but have one or two locations where there are severe defects. Rather than replace the entire pipeline reach, localized spot repairs may be more appropriate for these locations. For this analysis, any pipeline with a PACP grade 4 or 5 defect that was not included in the top priority pipeline rehabilitation/replacement projects is included in the spot repair priority list in Table 8-2 below.

TABLE 8-2: SPOT REPAIR LIST

Pipe Segment	Manhole From	Manhole To	Description	Material	Diameter (in)
L184	1112-01	1112-06	Class 5 Structural Defect	RCP	8
L380	1006-05	1006-04	Class 5 Structural Defect	RCP	8
CDT_57	411-03	411-02	Class 5 O&M Defect	PVC	24
CDT_65	409-02	409-03	Class 5 O&M Defect	PVC	18
G66	901-27	901-26	Class 5 O&M Defect	PVC	6
L123	1009-02	1009-03	Class 4 O&M Defect	RCP	8
L151	1010-15	1010-13	Class 4 O&M Defect	RCP	8
L204	1112-07	1112-08	Class 4 O&M Defect	RCP	8
L3014	1005-06	1005-01	Class 4 O&M Defect	RCP	8
L371	1006-09	1006-08	Class 4 O&M Defect	RCP	8
L447	902-06	902-15	Class 4 O&M Defect	AC	8
L81	1015-18A	1015-18	Class 4 O&M Defect	RCP	8
L97	1009-05	1016-03	Class 4 O&M Defect	RCP	12
L536	416-01	901-26	Class 4 O&M Defect	PVC	8
L364	1007-06	1007-01	Class 4 O&M and Structural Defect	RCP	8

Recommended actions to reduce I/I defects identified through smoke testing are found below in Table 8-3. Estimates of the cost and associated benefits of removing cross-connections identified by smoke testing are addressed in this table and discussed in more detail in Potential I/I Reductions, Section 7.7

TABLE 8-3: ESTIMATED INFLOWS AND IMPROVEMENT COSTS FOR CROSS-CONNECTIONS

Picture ID	Address	Inflow Source	Area of Inflow, A (ac)	Runoff Coefficient, C	Rainfall Intensity, i (in/hr)	Inflow, Q (cfs)	Inflow, Q (gpm)	Estimated Improvement City Cost	Cost per GPM
5	1352 Burnett st	Driveway Drain	0.009	0.9	2	0.02	7.4	$ 1,500.00	$202.57
13	818 Ida st	Roof Gutter	0.075	0.9	2	0.13	60.4	$ 1,300.00	$ 21.51
18	1200 Sixth ave	Lawn Grate	0.093	0.17	2	0.03	14.2	$ 1,500.00	$105.68
33	555 Summerview dr	Front Yard Drain	0.020	0.17	2	0.01	3.1	$ 1,500.00	$483.07
35	1684 Mountain ct	Front Yard Drain	0.014	0.17	2	0.00	2.2	$ 1,500.00	$684.16

8.1.3 Ongoing I/I Reduction Plan

It is recommended that the City continue to identify and monitor sources of I/I system wide. Tables 8-1 through 8-3 should be considered dynamic tables and thus should be updated periodically to reflect new information found in ongoing I/I investigations.

Part of this ongoing process is continuous inspection, improvement, and progress tracking. It is recommended the City plan out routine CCTV inspections. The City should try to inspect 41,766 linear feet of pipe ever year to complete the entire system on a 5-year rotation. Pipes should have their risk scores continuously updated after inspections, and Table 8-1 should dynamically change to reflect updates in the system and prioritize new pipes as defective ones are rehabilitated, replaced, or repaired.

It is estimated, based on 39.5 miles of pipeline and a 100-year life cycle, that the City should be replacing 2,088 linear feet of pipeline a year. With an approximate cost of $220 per linear foot (approximate based on mix of open trench, pipe bursting, and CIPP rehab/replacement), this means that the City should budget approximately $460,000 per year just for pipeline replacement. This budget number was considered in grouping projects and estimating how many years it would take to complete the rehabilitation or replacement of the pipelines.

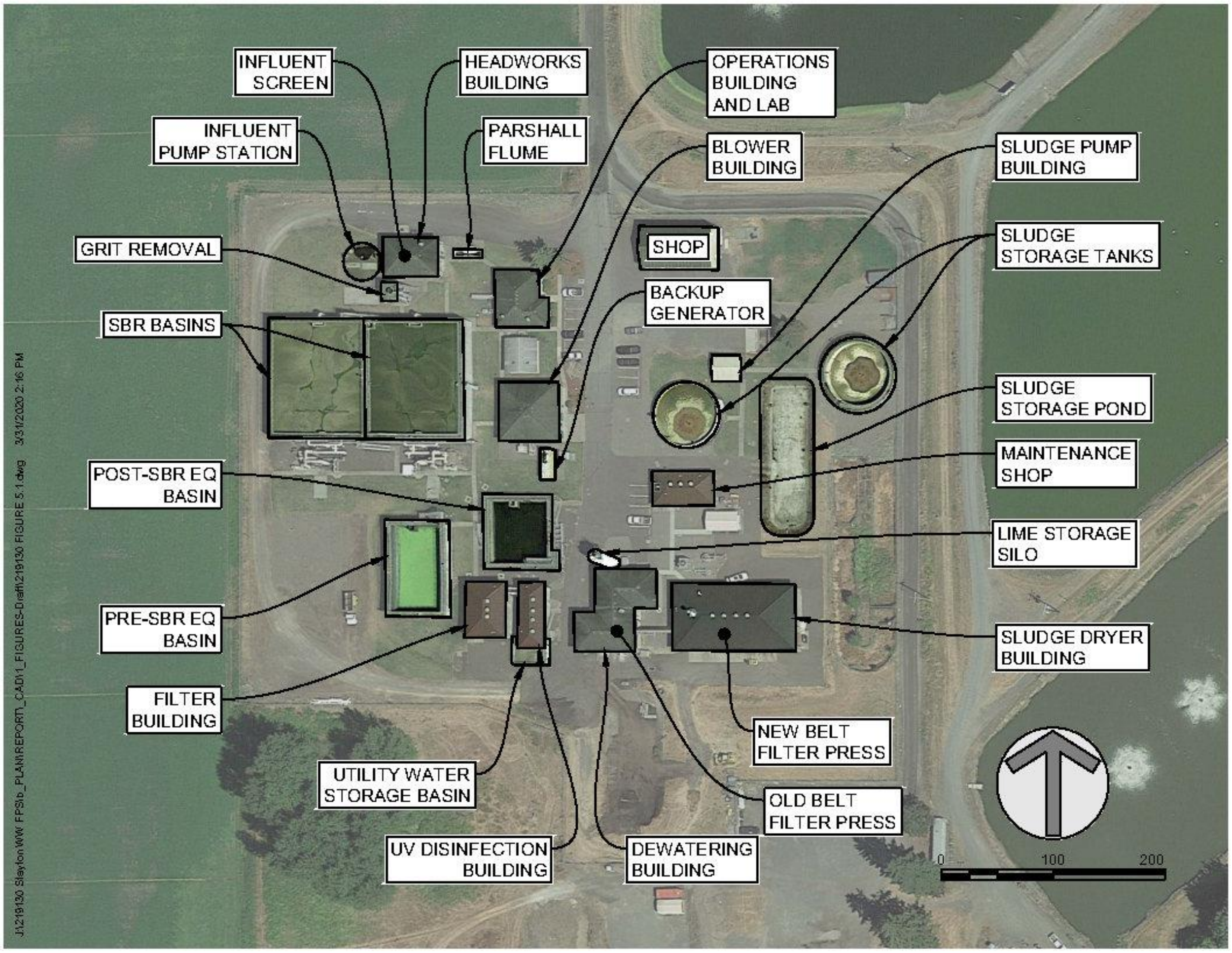
Continued monitoring in areas that have been studied and where improvements are made is important for tracking I/I in the system and for estimating the effect rehabilitation/replacement efforts have on I/I. This information can help identify effective methods of reducing or eliminating I/I in areas of the system monitoring will also help to track I/I over time and allow the City to identify areas where I/I is getting worse. Investigations and improvement work can be focused on those areas to reduce system I/I. Identifying, monitoring, and eliminating I/I is an ongoing and dynamic process. It is recommended the City continue rehabilitation/replacement efforts and continue to monitor and track I/I throughout the sanitary sewer system.

SECTION 9 – WWTP CONDITIONS ASSESSMENT

9.1 LOCATION MAP

A map of the existing wastewater treatment plant (WWTP) is shown in Figure 9.1. Full size figures are included in Appendix A (Figures 24-26).

FIGURE 9.1 – EXISTING WWTP MAP

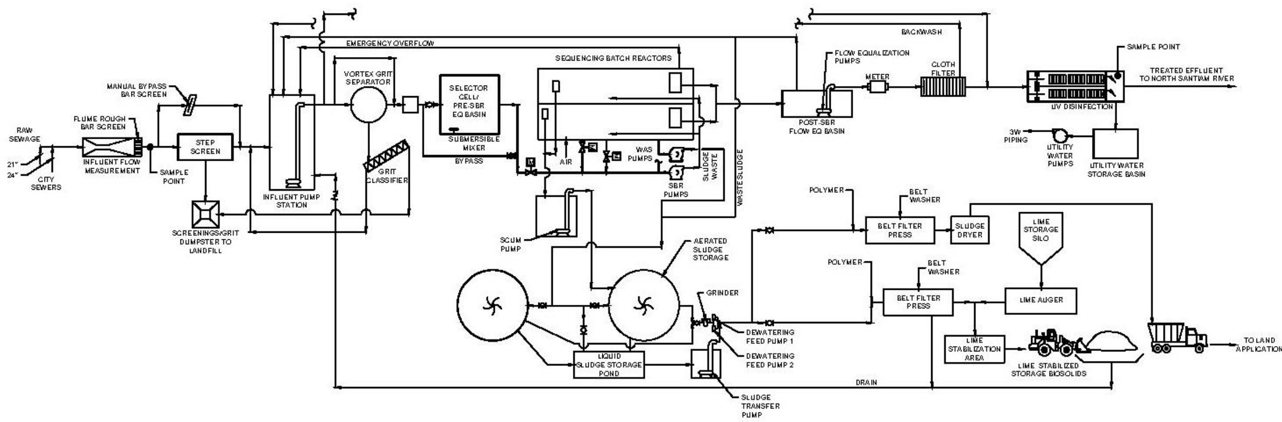


9.2 HISTORY

The City of Stayton has provided wastewater treatment for the Stayton-Sublimity area since 1962. The original WWTP was an oxidation ditch type plant which operated until 1997. Most of the original WWTP equipment has been abandoned except for the aerated sludge storage tank and sludge storage pond. The plant expansion in 1997 included a sequencing batch reactor (SBR) process; new operations building; headworks with Parshall flume, influent composite sampler, and inclined drum screen; influent pump station; vortex grit removal and grit washing/compacting; blower building; equalization basin; UV disinfection; and belt press with lime stabilization.

In 2009 the inclined drum screen, which had been experiencing a lot of issues, was replaced with the step screen. In 2013, the WWTP was upgraded with tertiary filtration; new UV disinfection; an additional equalization basin; new SBR pumps and blowers; additional sludge holding capacity, an expanded utility water system; and sludge drying. A simplified schematic process layout of the WWTP is shown in Figure 9.2.

FIGURE 9.2 – EXISTING WWTP PROCESS SCHEMATIC



9.3 WWTP DESCRIPTION

Wastewater from the entire collection system, including the City of Sublimity, is combined and enters on the north side of the WWTP. Septage is not allowed at the WWTP. Wastewater flows by gravity through the headworks, which consists of a Parshall flume for influent flow measurement, composite sampler, a step screen with manual bar screen bypass channel, screen washer/compactor, and grit classifier. The flow then enters the influent pump station where four submersible pumps in a wet well lift the wastewater up to the elevated vortex grit removal system. The screen and grit removal systems help protect the downstream equipment from large objects and also smaller nuisance particles that can damage the equipment.

From the grit removal system, the wastewater flows by gravity and is directed into one of the two SBR basins. Biological treatment occurs in the SBR basins as mixed liquor in the basins is intermittently aerated during a treatment cycle. Aeration to the SBR basins is provided by three (3) blowers located in the process control / blower building at the northwest corner of the site. At the end of the cycle, the mixed liquor settles in the basin prior to decanting the treated supernatant top water layer. Periodically the generated solids are removed by a waste activated sludge (WAS) pump and directed to one of the two aerated sludge storage tanks. Treated supernatant from the SBR basins flows to an equalization basin by gravity. The equalization basin is used for storage and to equalize the flow to the downstream process. The wastewater is pumped by the equalization pumps to the cloth filters for tertiary solids removal. The treated wastewater then flows by gravity to the UV system for disinfection. The UV disinfection deactivates bacteria, viruses, and other microorganisms to permissible levels for discharge. The effluent is currently discharged into the North Santiam River.

9.4 CONDITION OF EXISTING FACILITIES

9.4.1 Headworks


Influent Screen

Prior to entering the headworks building, the influent passes through a Parshall flume. The flume has a 12-inch throat. An ultrasonic level sensor is used to measure the water level in the flume and convert the level to an influent flow measurement. An American Sigma STREAMLINE refrigerated composite sampler is used to collect influent samples downstream of the Parshall flume. There is also a manual bar screen with 2-inch spacings downstream of the Parshall flume to protect the downstream screen equipment.

Inside the headworks building is a step-type bar screen with ¼-inch openings (HUBER SSL4900). The step screen was installed in 2009 after the drum screen experienced mechanical and flow capacity issues. The step screen operates automatically based on the water surface elevation difference upstream and downstream of the screen. There is also a backup timer in the screen control panel that will automatically turn on the screen after a delay.

There is a manual bar screen with ¾-inch openings in a separate channel. If the flow going through the step screen channel backs up significantly, the flow will automatically bypass the step screen and flow through the manual bar screen in this other channel. Stop logs are in place to isolate a screen channel if extensive maintenance is required. The screenings from the step screen are automatically washed and compacted in a wash press (HUBER WAP2), and then dropped into a trash container.

The control panels are located in a separate, unclassified electrical room of the headworks building. The screen and wash press can be controlled by a hand / off / auto (HOA) switch on the control panel. The headworks does not have a combustible gas detector; however, one has been purchased and is being installed.

Deficiencies

- The manual bar screen openings are too large; during high flow events large particles are able to pass through to the downstream processes.

Recommendations

- Install a second automatic influent screen to protect the downstream processes during high flow events.

9.4.2 Influent Pump Station


Influent Pump Station

After passing through the screens, the wastewater flows by gravity to the influent pump station wet well. Four (4) submersible pumps are located in the wet well. Two of the pumps are 35 HP and the other two pumps are 60 HP. Most of the influent pumps have been rebuilt in the past several years – the most recent was Pump #1 in May 2019. The pump station includes an ultrasonic level transducer to measure the level in the wet well, as well as backup float switches. Pumps are controlled based on the water level using a lead, lag, lag-lag, standby operational strategy. The pump designations are automatically rotated daily. The influent pump station was designed to accommodate a fifth influent pump in the future.

The City staff noted that the check and plug valves on the influent pumps are leaking and need to be replaced. The pumps are not controlled by variable frequency drives (VFD) to ramp up flow based on the level in the wet well. VFDs could save energy and help with process control of the SBRs. The Energy Trust of Oregon has been contacted and is planning to determine if energy incentives are available for the improvements.

The City's supervisory control and data acquisition (SCADA) system is used to track the status of the headworks equipment and send alarms. A backup generator provides power in the event of a power loss.

Deficiencies

- The valves on the pump discharge are leaking.
- There is little flow control capability on the influent pumps, which can cause fluctuations in the SBR process.

Recommendations

- Replace the leaking pump valves.
- Change out the pump starters to VFDs and add a control strategy to control the flow.

9.4.3 Grit Removal

Wastewater from the headworks force main is pumped to the grit removal located near the SBR basins. Removal of grit helps protect the equipment downstream by reducing wear caused by the grit particles. It also reduces the amount of inert material before it takes up space in the downstream SBR basins. The grit removal equipment was installed in 1997 and is comprised of a single H.I.L. Technology (now Hydro International) Grit King® vortex grit separator, recessed impeller grit pump, and grit cyclone/classifier. The grit cyclone/classifier is a WEMCO Hydrogritter and is located in the headworks building.

Grit Facility

Pumped grit is sent to the grit cyclone/classifier which dewaters the grit and deposits it into a dumpster. The grit removal system is operated by a timer system. A flush valve is opened periodically to fluidize the accumulated grit in the bottom of the vortex grit chamber to allow easier removal. The City staff report the solenoid flush valve had been having issues and was replaced in November 2019.

The cyclone/classifier starts at the same time as the grit pump but continues to operate until the grit is deposited in the dumpster. The grit facility operates continuously but does have bypass piping and valves if the equipment needs to be taken off-line for service or repairs.

There is only one pump which discharges to the grit cyclone/classifier. If the pump or classifier are out of service, grit would accumulate in the grit chamber and eventually overflow into the SBR basins. Grit accumulation in the SBR basins could cause issues with the SBR equipment. There are no spare motors for the equipment or spare pumps. Additionally, City staff have noticed that the stairs leading up to the vortex grit chamber are not secure. Securing the ladder has been planned and is occurring in 2020.

Deficiencies

- There is not a redundant grit unit or a spare grit pump and motors for the grit removal equipment.

Recommendations

- Purchase a spare grit pump and spare motors for the other equipment.

9.4.4 Selector Cell (Pre-SBR Equalization Basin)

Pre-SBR Equalization

The WWTP had been experiencing process upsets. As part of the 2013 upgrades, a 500,000-gallon equalization basin was added to control the influent to the SBRs. This pre-SBR equalization basin can store the influent until the SBRs are ready; providing the SBRs additional time for treatment, which is helpful during high flow periods. A 15 HP submersible high-speed mixer was installed to mix the basin. The SBR basin mixing pump and automatic valves transfer contents from the pre-SBR equalization basin to the correct SBR. The concept of using the basin as a “selector cell” is to hold influent and then send a slug load of high organic material to the SBRs with a low dissolved oxygen content. This would create an environment in the SBRs that would select for microorganisms with good settling characteristics. However, the slug feed mode was difficult for the City staff to program and so has never really been used.

Deficiencies

- Slug feed mode has been difficult for the City staff to operate. Using the flow proportional mode, the SBR mixing pumps transfer the flow too fast and shortens the treatment time in the SBR basins.

Recommendations

- Look at improving the slug feed controls or adding new pumps and piping to improve the flow control to the SBR basins.

9.4.5 SBR Basins

SBR Basins

The SBRs perform the combined functions of activated sludge biological treatment and secondary clarification in the same basin by operating the system in a batch process. Process operation is control by the SBR programmable logic control (PLC). Wastewater leaving the grit removal facility is directed to the SBR basins, entering through the two influent distribution headers in the bottom of each basin. Each SBR basin has a maximum volume of approximately 1.3 million gallons.

Basin mixing is performed by two headers in each basin and an external 125 HP centrifugal pump, one for each basin. Aeration is accomplished using two (2) 200 HP K-Turbo blowers (now owned by Aerzen) in a lead, lag arrangement. Air is sent to the SBR basins by air control valves, as programmed by the SBR control panel. The K-Turbo blowers are difficult to maintain as there is only one technician in the US who can work on these machines. Also, there may be more efficient blowers for the varying water levels in the SBRs. Originally, rotary lobe positive displacement blowers were used. One of the original 200 HP positive displacement blowers remains as a backup, but it is old, very noisy and does not working particularly well according to City staff. Air from the blowers is directed to the SBRs in two (2) 14-inch pipelines (one per SBR basin).

Two floating decanters in each SBR basin are used to decant the treated supernatant following a settle period. The pneumatic decant valves are leaking as the seats are worn. Air is supplied to these valves from two air compressors in the Blower Building. The floating decanters are configured to withdraw effluent approximately 18 inches below the top water surface, so floating scum has not typically been an issue. The water level in the SBR basin is monitored using an ultrasonic level sensor. The change in water level typically controls the aeration, settling and decant portions of the cycle. There is a backup float switch to alert the City staff of a high-water level condition.

Each SBR basin is equipped with a 15 HP centrifugal WAS pump outside of the basin. The WAS pump removes settled sludge from the bottom of the basin through the same pipe that is used to bring influent into the SBR basin. The WAS pump is typically operated during the idle or decant portions of the SBR cycle. The flow of sludge is measured via a magnetic flow meter. The WAS pumps are operated via a VFD for either an operator-adjustable period of time or number of gallons. As a backup to the WAS pumps, the SBR mixing pumps could also be used for periodic, manual wasting. When operating with only one basin, the current SBR program is not able to automatically waste sludge, and instead must be manually operated.

There is a floating scum skimmer in each basin, similar to the floating decanters. The skimmer pipes lead to a scum wet well where a 3 HP submersible scum pump is used to transfer the scum back to a sludge storage tank. The scum pump is controlled by a float switch. The City staff reports that they have not been successful with operating the scum skimmers.

The SBR basins were recently drained and inspected. The City staff reported that the amount of grit in the basins was less than expected as the basins had not been inspected for approximately 8 years. The pumps and piping around the SBR basins are exposed to the weather rather than under a cover.

Deficiencies

- The K-Turbo blowers are difficult to maintain and may not be as efficient as other blowers.
- The SBR decant valves are leaking.
- Scum removal is not working.
- Pumps and piping are exposed to the weather.
- When only one SBR basin is in operation, the current program does not allow for automatic sludge wasting.

Recommendations

- Look at energy incentives to replace the blowers. The last of the original rotary lobe positive displacement blowers is being replaced by an Inovair geared centrifugal blower in 2020. Energy incentives have been approved for the replacement.
- Replace the decant valves.
- Replace the scum removal system.
- Cover the pumps and piping to protect them from corrosion.
- Update the SBR program to include automatic sludge wasting, even when only one SBR basin is in operation.

9.4.6 Post-SBR Equalization Basin

Post-SBR Equalization Basin

Treated wastewater from the SBRs is directed to the post-SBR equalization basin via a 30-inch pipe. The purpose of the flow equalization basin is to equalize the decanted SBR effluent so that it does not overwhelm the filtration and UV disinfection systems downstream. Additionally, the equalization basin pumps are used to lift the water up to those systems.

The equalization basin has a capacity of 215,000 gallons. Three (3) submersible pumps are installed in the basin, with space for a fourth pump. Two of the pumps were replaced in September 2019.

A magnetic flow meter on the pump discharge line measures the flow. This is also the effluent flow measurement. The water level in the equalization basin is monitored via an ultrasonic level sensor. There is also a 5 HP submersible sump pump (with shelf spare) to drain the basin. City staff report it is difficult to clean this basin because the floor is not sloped sufficiently or the sump is not deep enough, so solids tend to settle in the center. However, this is not a big issue for City staff as it can be washed down with fire hoses.

9.4.7 Filtration

The filtration system was installed in 2013 to further reduce solids in the WWTP effluent. The filtration system includes three (3) 10-Disk AquaDisk cloth media filters. A weir on each filter is used to equally distribute the influent to the three (3) filters.

Cloth Media Filter

During filtration, solids build up on the media and backwashing (by reversing flow through the filter) is needed to clean the media. Solids also settle to the bottom of the filter basin and must be periodically removed via perforated pipes at the bottom of the basin. Backwash pumps are used to remove solids that accumulate on the cloth and underneath the disks. The backwash pumps are operated based on the water level in the basin (as measured by a pressure transducer), elapsed timer, or manual activation. A drive rotates the disks past backwash shoes to clean the media.

There is one turbidity meter for the influent channel and one turbidity meter for the effluent channel. The piping to the meters was only ¼ inch and tended to clog. It was recently replaced with ¾ inch piping. The cloth media for Filter #2 was recently changed. The media on Filters #1 and #3 is scheduled for replacement.

Deficiencies

- Cloth media for Filters #1 and #3 is old and performance is decreasing. The cloth media for Filter #1 has been ordered and will be replaced in 2020.

Recommendations

- Replace the cloth media on Filter #3.

9.4.8 UV Disinfection

The UV disinfection system inactivates pathogens and other microorganisms before the effluent is discharged into the North Santiam River. The filtered water flows by gravity through a 24-inch pipe from the Filter Building to the UV disinfection system. The UV disinfection system is comprised of two rectangular channels with three (3) banks of five (5) horizontal UV modules in each channel. Both channels are operated simultaneously during normal conditions. The dose and number of lamps in use is automatically adjusted based on signals received from the SBR PLC.

The UV disinfection system was installed in 2013. In May 2019 the system was refurbished with genuine Trojan factory parts and is working well according to City staff.

There is an automatic isolation gate at the head of each channel. At the end of each channel, there is an ultrasonic level sensor, a low-level float switch, and an automatic level control weir gate. The weir gate elevation is automatically controlled to maintain a minimum water depth over the UV lamps. An American Sigma STREAMLINE refrigerated composite sampler in the UV Building is used to collect effluent samples. The UV channels empty into a common effluent drop box that supplies water to the utility water basin and flows through a 30-inch effluent pipe to the outfall into the North Santiam River.

No deficiencies were noted for the UV disinfection system.

UV Disinfection System

9.4.9 Utility Water System

Treated and disinfected effluent flows from the UV disinfection system to the outfall and also to the Utility Water Storage Basin. The basin has a capacity of approximately 20,000 gallons. There are four (4) 5 HP centrifugal utility water storage pumps. The utility water is further disinfected with sodium hypochlorite via a chemical feed pump. The utility water system discharges into a 6-inch pipeline and supplies water throughout the WWTP.

Utility Water Pumps

No mechanical deficiencies were identified by City staff; however, the City staff, due to the limited storage capacity in the basin have not been able to operate the utility water system. There is a connection between the potable water system and the utility water system. The connection is protected by a reduced pressure backflow preventer valve.

9.4.10 Solids Handling

Sludge removed from the SBR basins via WAS and scum pumps is sent to the sludge storage tanks. The sludge storage tanks both have diameters of 55 feet. Sludge Storage Tank #1 has a fairly flat bottom with a maximum water surface elevation of approximately 12.7 feet. Sludge Storage Tank #2 has a sloped bottom and a maximum water surface elevation of approximately 11.5 feet. The working volume for Sludge Storage Tanks #1 and #2 is 172,400 gallons and 213,250 gallons, respectively.

Sludge Storage Tank

Aeration is provided by a 20 HP, floating mechanical aerator in each tank. A spare motor for the surface aerators is kept onsite. An ultrasonic level sensor is used to shutoff the aerator if the sludge level drops below a certain level to prevent damaging the aerator.

In the event the sludge storage tanks are full, the sludge would overflow to a non-aerated sludge storage pond adjacent to the sludge storage tanks. The capacity of the concrete-lined sludge storage pond is approximately 225,000 gallons. A 7.5 HP, non-clog submersible pump can be used to transfer the sludge from the storage pond back to Sludge Storage Tank #1. The sludge storage pond is concrete lined and according to City staff needs repair.

Deficiencies

- The sludge storage pond needs repair.

Recommendations

- Repair and recoat the sludge storage pond.

9.4.11 Solids Dewatering

The dewatering system includes a 3 HP inline sludge grinder, two (2) 7.5 HP rotary lobe dewatering feed pumps, and two belt filter presses. The flow to the belt filter presses is measured using a magnetic flow meter. Each of the belt filter pressess has an associated washwater pump and polymer feed unit. The belt filter press installed in 2013 is the primary unit and is used for dewatering sludge that is sent to the dryer. The other belt filter press is used temporarily if necessary and can be used in conjunction with lime. Dewatered sludge can be conveyed from either of the belt filter presses to the sludge dryer.

Belt Filter Press

The belt filter press feed pumps are used to pump WAS from the sludge storage tanks to either of the belt filter presses.

Prior to dewatering in either system, the WAS is conditioned with a lquid polymer and then fed onto the belt filter press. The filtrate is collected and returned to the influent pump station. The dewatered cake is conveyed to the sludge dryer or the lime stabilization, depending on the belt press in use.

The City staff report that a polymer motor is in need of repair, but the replacement part has been purchased. Also, the area around the polymer is a slip hazard.

Deficiencies

- Polymer area is a slip hazard.

Recommendations

- Redesign polymer area to reduce slip hazard.

9.4.12 Sludge Drying

The main units of the sludge drying system include the sludge dryer, conveyors, and the cooling system. Additional components of the system include:

- two (2) feed conveyors
- dryer feed hopper with auger
- thermal fluid (hot oil) unit that provides the dryer heat source
- cooling auger
- scrubber condenser
- odor control (carbon scrubber fan)
- dried sludge transfer conveyor
- cooling tower with fan
- cooling tower feed wet well with two (2) vertical turbine pumps
- dryer cooling water feed wet well with two (2) vertical turbine pumps.

Sludge Dryer

The dryer indirectly heats the sludge to remove excess water. The dried material is discharged into a cooling auger (cooled with water from the cooling system), and then conveyed via the transfer conveyor to the dried sludge process room. The steam generated from the drying process is liquefied in a scrubber condenser, and then cooled in the cooling tower before being recycled to the headworks. Air from the scrubber condenser is treated in the odor control system. There is no redundancy for the dryer and parts are very expensive.

The dried biosolids are stored in the processing room and removed by the general public for free. The City has been successful in attracting attention for the biosolids, and the demand has steadily increased. Since the biosolids are a Class A product, if there is insufficient storage in the processing room, the biosolids can be stored elsewhere, including a nearby 40-acre tract of land owned by the City.

Deficiencies

- There is no redundancy for dryer system.
- Dryer system parts are expensive.
- The BCR (fka Therma-Flite) dryer has varying lead-times for spare parts.

Recommendations

- Purchase spare parts to store onsite for quicker repairs.
- Evaluate different dryer options.
- Request a list of spare parts that should be on-hand, and those that have reasonable lead-times.

9.4.13 Lime Stabilization


Lime Storage

The lime stabilization process applies crushed pebble lime to dewatered sludge cake at a rate to produce a stabilized sludge with a pH of 12 for a minimum of two hours and 11.5 over a 24-hour period. The stabilized sludge is then transferred to cake storage or truck loading for disposal. The lime stabilization system includes a lime storage silo with volumetric feeder and feed screw conveyor, a dewatered cake feed screw conveyor, a mixer, a cake transfer conveyor and a truck loading conveyor. Dewatered cake is dropped from the belt filter press into the cake feed screw conveyor and transferred to the lime stabilization paddle mixer.

The dewatered cake is mixed with pebble lime delivered from the lime silo, located outside of the Dewatering Building. The lime storage silo receives bulk crushed pebble lime from truck delivery. Lime is withdrawn from the silo to the stabilization mixer at a rate determined by the operator and delivered to the mixer by the lime feed screw conveyor. The cake and lime are thoroughly mixed and discharged to the cake transfer conveyor. The stabilized cake is then transferred to the truck loading or cake storage area for sampling and disposal.

No deficiencies were noted by City staff; however, the lime storage system is not in use at this time.

9.4.14 Ancillary Equipment (Emergency Power, Operations Building, Lab, and SCADA)

There is one CAT 1,250 kW diesel powered emergency standby generator at the WWTP. When power is lost, the generator is started automatically. Power is switched to the generator automatically via the automatic transfer switch. This generator powers the critical equipment including the influent screen, influent pumps, SBR equipment, post-SBR equalization basin pumps, and UV disinfection equipment. The dryer is not connected to the generator, and power loss can lead to major maintenance issues if the dryer is in operation. The generator has a 12-hour fuel capacity. The generator is in good condition, but was installed in 1997 and is expensive to repair.

Backup Generator

The Operations Building and Lab is generally in good repair; however, the HVAC system in the laboratory should be separate from the rest of the building for health and safety reasons. Additionally, it may be beneficial to have a small biosolids lab space in the Dewatering Building.

The SCADA system has been maintained by The Automation Group (TAG) and is in good condition. The City is considering changing to a new SCADA system with better connectivity throughout the WWTP as funds become available.

Deficiency

- Not all of the WWTP equipment is connected to the generator, so effluent compliance may be more difficult during a power outage.
- The standby generator is reaching the end of its useful life and is becoming more expensive to repair.
- The HVAC system in the Operations Building and Lab struggles with the lab components.
- There is no area in the current Dewatering Building to process lab samples.

Recommendation

- Connect all of the WWTP equipment to backup power. The City is currently exploring connecting the dryer motor control center to the emergency generator.
- Provide backup power for all of the WWTP equipment.
- Separate the HVAC systems for the lab from the rest of the Operations Building. This is currently being investigated by Santiam Heating.
- Create a location for biosolids sampling and analysis in the Dewatering Building. This is currently being worked on by City staff.

9.4.15 Site Security and Roads

The main access to the WWTP is a gravel road (Jetters Way) off W Ida Street. In order to enter the WWTP from this direction, there is a one-lane bridge that crosses Power Canal. A detailed structural analysis of the bridge has not been performed.

All roads and most of the drivable surfaces within the WWTP are paved. During the site visit it was noted that the drivable areas within the WWTP site where in good condition. The entire WWTP site is enclosed by a chain link fence with three strands of barbed wire. Lockable gates are located at the entrances to the WWTP.

City staff did not note any deficiencies with regards to security or roads at the plant.

9.4.16 Stormwater

The WWTP collects site stormwater and drains it into the Headworks. There is a plant overflow but that flows to a designated overflow area. The City does not have a general storm water permit since they collect and treat the stormwater. City staff did not note any deficiencies with regards to the stormwater drainage.

9.5 WATER / ENERGY / WASTE AUDITS

The City has not conducted any water, energy or waste audits.

SECTION 10 – WWTP CAPACITY EVALUATION

This chapter provides a summary of the performance and capacity of the existing Stayton WWTP.

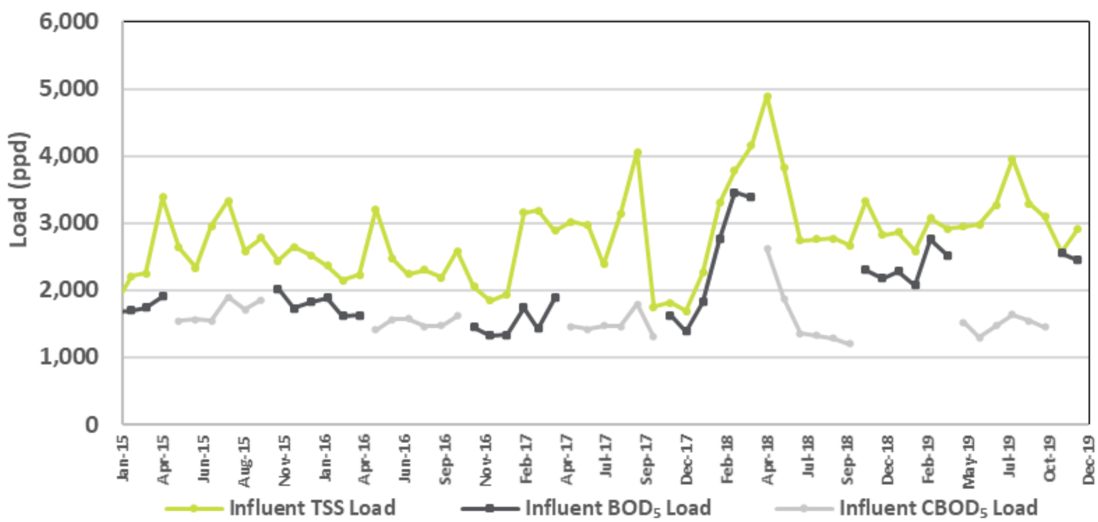
Plant influent data taken from the DMRs for January 2015 through December 2019 were analyzed. The plant influent was monitored for 5-day biochemical oxygen demand (BOD_5), 5-day carbonaceous biochemical oxygen demand ($CBOD_5$), total suspended solids (TSS), and pH. The City collected influent 24-hour composite samples at least two times per week to test for BOD_5, $CBOD_5$, and TSS, and at least three grab samples per week for pH. Although not required by the permit, the City also measured influent temperature.

The plant effluent was monitored for BOD_5, $CBOD_5$, TSS, temperature, *E. coli* bacteria, pH, ammonia, total Kjeldahl nitrogen (TKN), nitrite-nitrate (NO_2+NO_3), total phosphorus (TP), and UV dose. The City collected 24-hour composite samples at least two times per week to periodically test for BOD_5, $CBOD_5$, TSS, and ammonia. At least once per week, from May through October, 24-hour composite effluent samples were taken to test for TKN, NO_2+NO_3, and TP. The effluent temperature was measured at least twice per hour throughout the year. Grab samples were collected to test the effluent *E. coli* bacteria (at least two times per week) and pH (at least three times per week).

10.1 INFLUENT WATER QUALITY

Influent BOD_5, $CBOD_5$, and TSS concentrations and loadings into the WWTP are summarized in Chapter 2. Chart 10.1 shows these influent constituents from 2015 through 2019.

CHART 10.1: INFLUENT TSS, BOD_5, AND $CBOD_5$ LOADS



The waste strength (load) appears to be increasing slightly, likely due to population growth during the reporting period.

Influent ammonia (as N), total nitrogen (TN), total phosphorus (TP), and alkalinity were not measured at the WWTP. For this facility planning study evaluation, typical values based on the influent BOD_5, $CBOD_5$, and TSS loads were utilized, and adjustments were made using the actual effluent monitoring results.

The monthly average influent pH and temperature measurements are shown in Charts 10.2 and 10.3, respectively. To determine the average pH, the hydrogen ion measurements were averaged. The minimum monthly influent temperature was 12.6°C. The maximum monthly influent temperature was 20.8°C.

CHART 10.2: INFLUENT pH

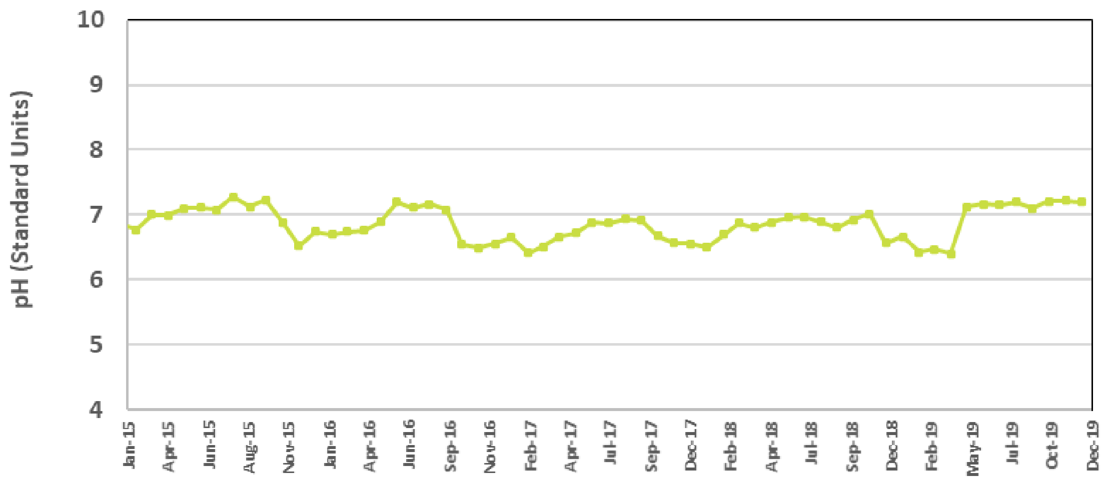
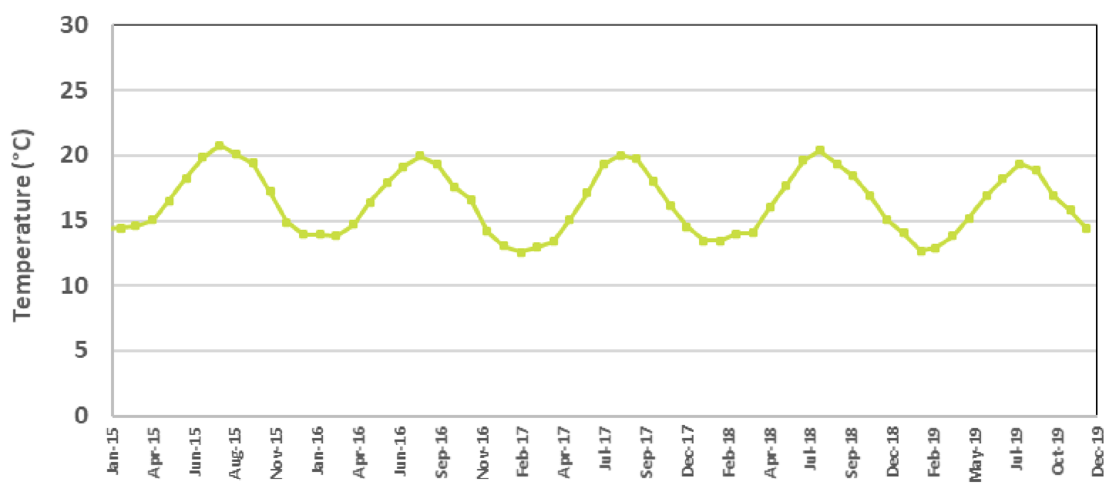


CHART 10.3: INFLUENT TEMPERATURES



10.2 EFFLUENT WATER QUALITY

This section evaluates the effluent water quality from 2015 through 2019. In the charts below, the effluent sampling results were compared to the planning level limits from Chapter 2.

Effluent TSS and BOD_5 – November 1 through April 30

As discussed previously, the effluent requirement from November 1 through April 30 is BOD_5 rather than $CBOD_5$. The monthly average TSS and BOD_5 effluent concentrations and loads are shown in Charts 10.4 and 10.5, respectively. The WWTP experienced exceedances for these parameters in early 2018, due to a level sensor error in the SBR basin, an electrical outage with power loss to the SBR PLC, screen repair, and a broken filter chain. As shown, due to the high flows during this period, it is easier to achieve compliance with the effluent concentrations (which are established based on OAR Chapter 340, Division 41), than the effluent loads (which are per the Three Basin Rule).

CHART 10.4: EFFLUENT TSS AND BOD_5 MONTHLY AVERAGE CONCENTRATIONS

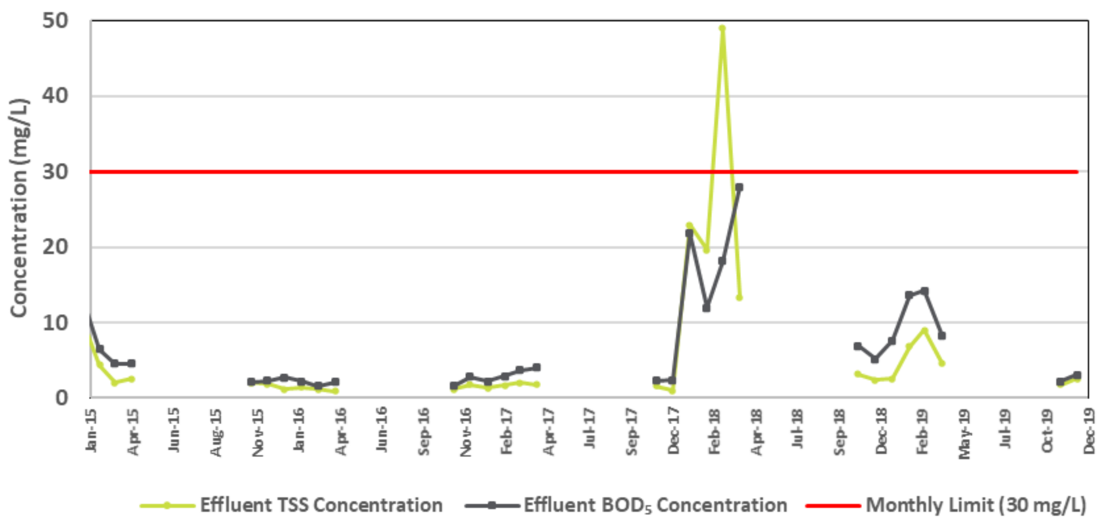
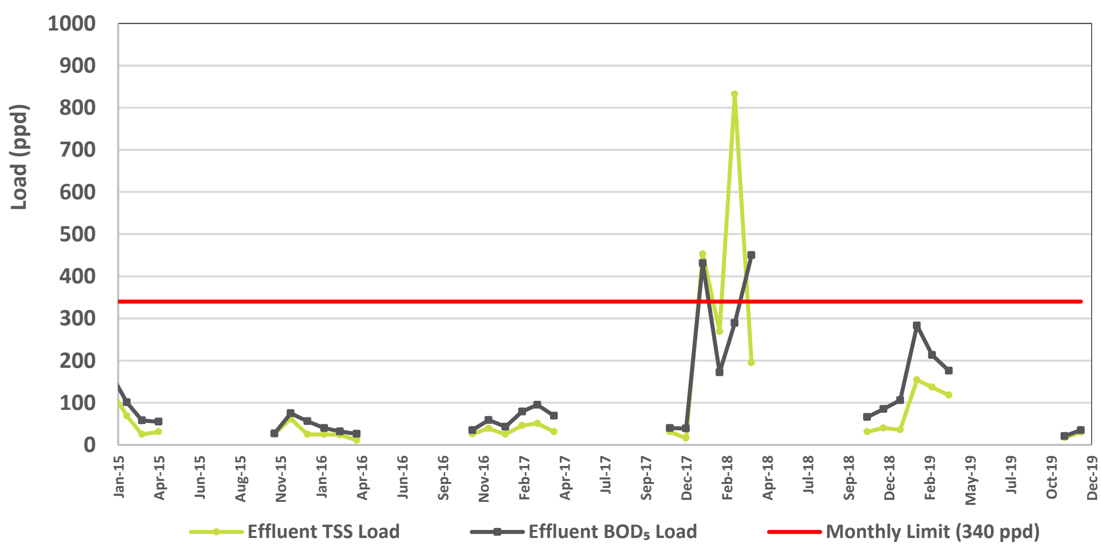


CHART 10.5: EFFLUENT TSS AND BOD_5 MONTHLY AVERAGE LOADS



Effluent TSS and $CBOD_5$ – May 1 through October 31

Monthly average TSS and $CBOD_5$ effluent concentrations and loads are provided in Charts 10.6 and 10.7, respectively. No exceedances were observed during this dry weather period.

CHART 10.6: EFFLUENT TSS AND $CBOD_5$ MONTHLY AVERAGE CONCENTRATIONS

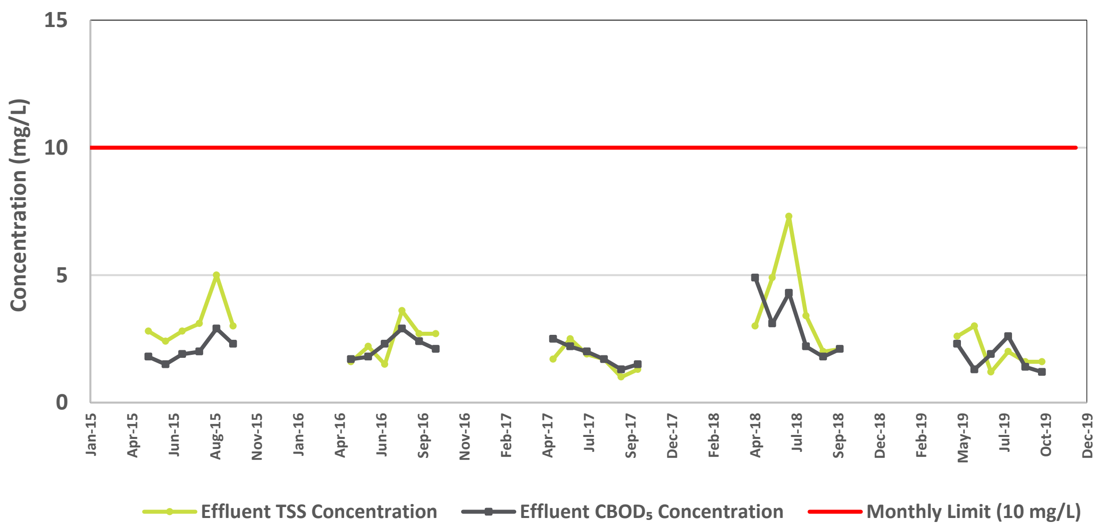
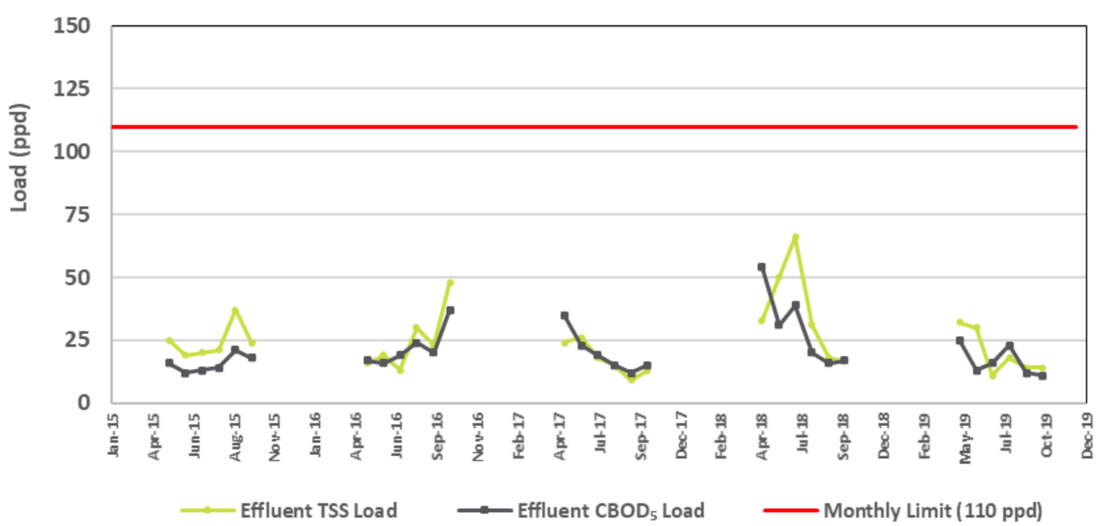


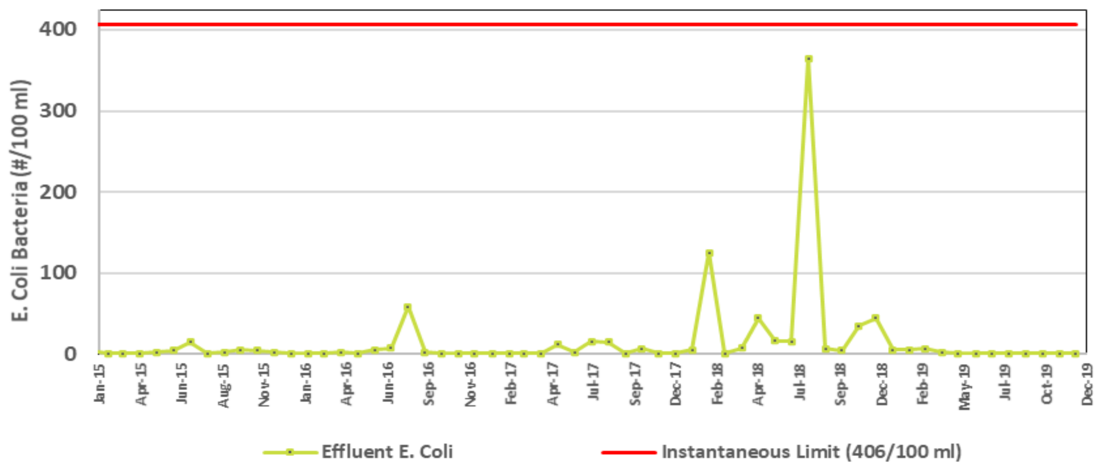
CHART 10.7: EFFLUENT TSS AND $CBOD_5$ MONTHLY AVERAGE LOADS



Effluent *E. coli* Bacteria

Instantaneous *E. coli* bacteria effluent data is shown in Chart 10.8. During the process upset in early 2018, a spike in effluent *E. coli* was measured. Additionally, in August 2018, there was another high count of bacteria. Despite these few events, there have not been any permit exceedances during this period.

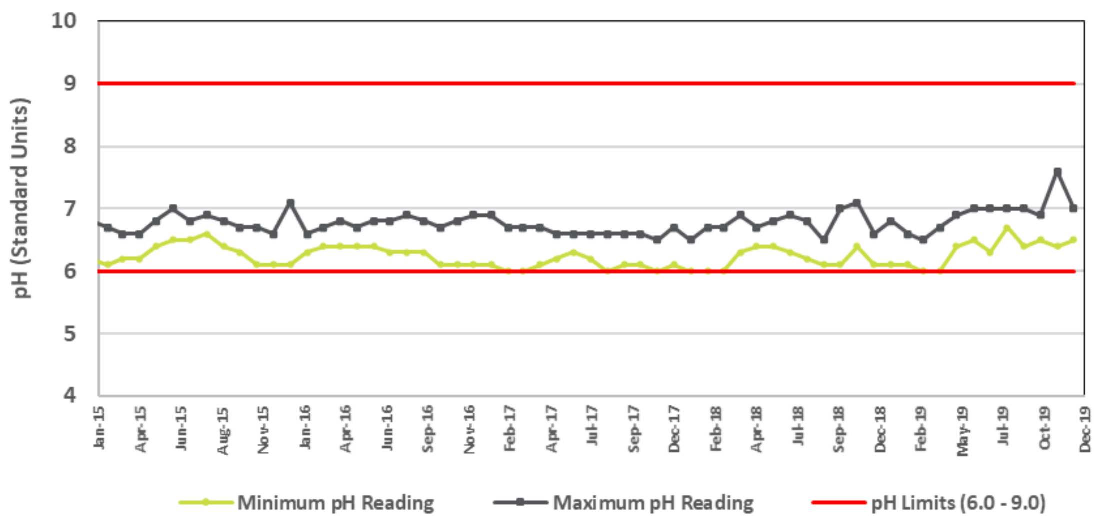
CHART 10.8: EFFLUENT E. COLI INSTANTANEOUS MAXIMUM



Effluent pH

Chart 10.9 shows the pH effluent data from 2015 through 2019. The City has experienced some low pH events; however, no violations were recorded. The City also recently replaced its pH probe, and since then, the pH measurements have not been as close to the lower limit.

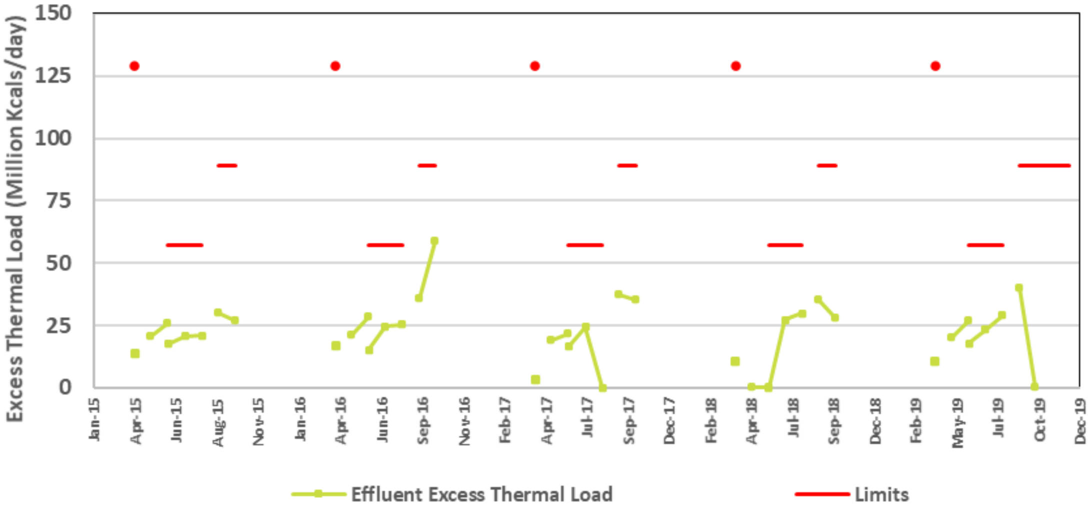
CHART 10.9: EFFLUENT PH



Effluent Temperature

The excess thermal load is calculated using a 7-day rolling average of the maximum effluent temperature during decant and the 7-day rolling average daily WWTP flow. The excess thermal load requirement varies depending on the time of year. A comparison of the excess thermal load to the limits is shown in Chart 10.10. The excess thermal load has been below the permit limits.

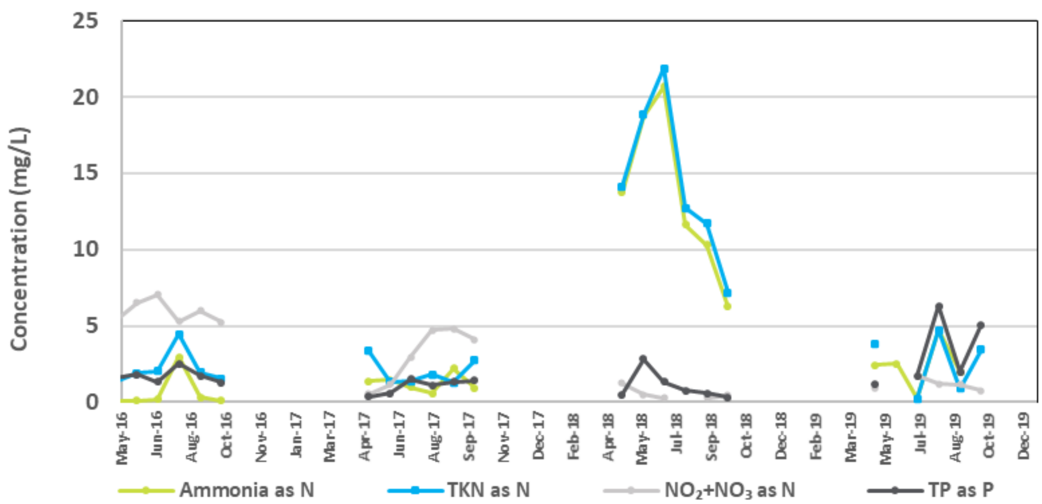
CHART 10.10: EFFLUENT EXCESS THERMAL LOAD



Effluent Nitrogen and Phosphorus

Although there are no nutrient permit limits, ammonia, TKN, NO_2+NO_3, and TP are periodically measured during the dry weather months. Monthly effluent concentrations are shown in Chart 10.11. The effluent ammonia and TKN concentrations were higher than usual in 2018. This is likely due to some carryover in lost mixed liquor due to the WWTP upset. Based on the 2019 data, the WWTP appears to have recovered to previous treatment levels.

CHART 10.11: EFFLUENT AMMONIA, TKN, NO_2+NO_3, AND TP MONTHLY AVERAGE CONCENTRATIONS



10.3 RELIABILITY EVALUATION

An essential criterion for WWTP planning is the reliability of unit processes, which generally relates to providing redundant equipment. For the highest level of reliability (Reliability Class I per EPA guidance, EPA 430-99-74-001), at least two units are required for screens, pumps, aeration basins, blowers, and disinfection. The EPA reliability criteria also requires the capacity, with the largest piece of equipment out of service, be sufficient to provide for:

1. Screens – peak design flow
2. Pumps – peak design flow
3. Blowers – design oxygen transfer
4. Filters – 75% of the design flow

Ten States Standards, (a well-known industry resource, although not formally adopted by Oregon as a standard), also recommends that UV disinfection facilities be able to provide full treatment with one bank out of service.

Table 10.1 provides a summary of the WWTP equipment reliability. This summary table includes ratings for redundancy, criticality, and equipment condition for each major unit process.

TABLE 10.1: UNIT PROCESS RELIABILITY SUMMARY

Equipment	Built	Redundancy Rating	Criticality Rating	Equipment Condition Rating
Influent Screen/Washer	2009	3 (Manual Screen)	S/H, EQ, PF, CC	M - Overall
Influent Pump Station	1997 (pumps rebuilt 2017-2019)	1	S/H, PF, CC	LN – Pumps; M – Wet Well; R – Valves
Grit Removal/Classifier	1997	4 (Bypass Pipe)	PF, CC	M – Overall
SBR Basins	1997	1	S/H, EQ, PF, CC	M – Overall; R – Decant Valves and Scum Removal
SBR Blowers	1997 PD blower; 2013 Turbo blowers	1	S/H, EQ, PF, CC	W – PD blower; M – Turbo blowers
Post-SBR Eq. Basin	1997 (2 of 3 pumps new 2019)	1	S/H, EQ, PF	LN – Pumps; M – Basin
Filtration	2013	1	S/H, EQ, PF, CC	M – Overall; R – Cloth Media
UV Disinfection	2013	1	S/H, EQ, PF, CC	M – Overall
Utility Water System	2013	1	PF	M – Overall
Sludge Storage	1997 Tank 1 & Pond; 2013 Tank 2	4 (aerator motor)	S/H, PF	M – Tanks; M – Aerators; R – Pond
Sludge Dewatering	1997 and 2013	1	S/H, PF	M – Overall; R – Polymer Motor
Sludge Treatment	1997 Lime stabilization; 2013 Dryer	4 (lime stabilization)	S/H, PF, CC	W – Dryer parts
Redundancy Rating				
1	**One level of "in-kind" redundancy** (Identical piece of equipment is available)			
2	**Two+ levels of "in-kind" redundancy** (More than one identical piece is available)			
3	**Equipment alternative** (An alternative piece of equipment is provided)			
4	**Procedural alternative** (An alternative operating procedure is used)			
5	**No Backup** (Failure of equipment will shut entire process down)			
Criticality Rating				
S/H	**Safety and Health Risk** (Would create a safety risk to plant personnel or others)			
EQ	**Effluent Quality Risk** (Would create effluent permit risk)			
PF	**Process Functionality Risk** (Would affect the function of other processes)			
CC	**Cost Critical** (Would cost a significant amount to repair/replace in an emergency)			
Equipment Condition Rating				
N	**New** (Equipment is new, or replaced in last 12 months)			
LN	**Like New** (Equipment is operated very little or recently overhauled)			
M	**Used But Maintained** (Equipment showing expected wear, but is maintained)			
W	**Heavily Worn** (Equipment close to the end of useful life; not performing well)			
R	**Needs Replacement** (Equipment needs replacement or repair)			

All of the major unit processes have a type of redundancy. Additional details on equipment conditions can be found in Chapter 9.

10.4 CAPACITY LIMITATIONS

WWTP capacity considers both hydraulic and treatment limitations. A model was created using Visual Hydraulics (Version 4.2) to investigate WWTP hydraulics. The treatment process was investigated using a BioWin 6.0 process model. The models assumed all WWTP components were online and functioning. As-built drawings were used during model development, and calibration of the process model was performed using plant operational data and the DMRs. However, as part of any design, a survey should be undertaken to confirm the elevations in the hydraulic profile. The existing hydraulic profile for the 2040 PIF_5 (9.18 MGD) is included in Appendix A. The model results for each area of the WWTP are discussed below.

10.4.1 Headworks

The rated flow capacity of the influent Parshall flume is 10.4 MGD. This is greater than the 2040 peak instantaneous flow, so the Parshall flume should be able to measure the peak flow events during the planning period.

The capacity of the influent step screen (according to HUBER (screen manufacturer)) is 10.2 MGD, which is greater than the 2040 peak instantaneous flow. Therefore, there appears to be sufficient capacity to handle future flows. However, when screen maintenance is required, the manual bar screen with large openings must be used. A second mechanical screen would allow the existing step screen to be serviced while providing adequate screening into the WWTP.

The wash press receives the screenings from the fine screen. It automatically washes and compacts the screenings before placing them into a trash container. The wash press has a capacity of 70 cubic feet per hour, which is acceptable.

10.4.2 Influent Pump Station

Following the influent screens, wastewater flows into the adjacent influent pump station wet well. Four (4) submersible pumps are located in the wet well. Two of the pumps are 35 HP, with a rated capacity of 1,390 gpm per pump. The other two pumps are 60 HP, with a rated capacity of 3,680 gpm per pump. The firm capacity (the capacity with one of the 60 HP pumps offline) is 6,460 gpm (9.3 MGD). A pump test of the pump station was not included as part of the evaluation. Based on the rated capacity, it appears that the pumps have enough capacity for the planning period. There is space in the pump station for an additional pump to be added in the future.

The capacity of the 24-inch section of pipe between the influent pump station and the vortex grit removal system is also sufficient for the planning period.

10.4.3 Grit Removal

The capacity of the vortex grit removal system is 9.27 MGD, which is greater than the 2040 PIF_5. However, the removal system is designed to remove 95% of grit 150 microns and larger at flows only up to 5.0 MGD. This means that grit removal may not be satisfactory during high flow events.

The grit classifier is rated for the same capacity as the grit removal pump. A test was not performed as part of this project. However, the SBRs were recently cleaned, and the amount of grit removed was less than expected, which indicates the grit removal system is functioning as designed.

10.4.4 Selector Cell (Pre-SBR Equalization Basin)

The Pre-SBR Equalization Basin has a capacity of approximately 500,000 gallons. Assuming an influent flow of 7.82 MGD (2040 $PDAF_5$), the equalization basin could be used to store the flow for 92 minutes. If one of the SBR basins is down for maintenance, the minimum storage time needed to not advance a 6-hour cycle is 180 minutes (3 hours). Therefore, the Pre-SBR equalization basin would not be large enough to handle the influent flow during a peak day flow event in the winter. However, an SBR basin could still be taken down for maintenance in the summer when the flows are much lower.

An 8.3 HP submersible high-speed mixer provides mixing for the equalization basin. This equates to a mixing intensity of approximately 16 HP per million gallons, which is acceptable for this basin.

10.4.5 SBR Treatment

The Three Basin Rule (OAR 340-041-0350) prohibits an increase in the permitted effluent mass loading to the North Santiam River. As shown in Charts 10.4 and 10.6, excluding the WWTP upset, wintertime monthly average effluent BOD_5 concentrations are typically as high as approximately 10 mg/L, and in the summertime effluent $CBOD_5$ concentrations are typically as high as approximately 5 mg/L. Based on the historical data from when the WWTP is operating well (10 mg/L BOD_5 concentrations and 5 mg/L $CBOD_5$ concentrations), to continue to meet the Three Basin Rule monthly average loading limits (340 ppd BOD_5 in winter; 110 ppd $CBOD_5$ summer), the maximum monthly flows must stay below approximately 4.1 MGD in the winter and 2.7 MGD in the summer. The Three Basin Rule does not affect the effluent permit concentrations, and DEQ has stated the effluent permit concentration limits will remain at 30 mg/L for BOD_5 and 10 mg/L for $CBOD_5$ for the foreseeable future. The current monthly flows are 4.09 MGD in the winter and 1.92 MGD in the summer, and the 2040 planning monthly flows are 4.54 MGD in the winter and 2.25 MGD in the summer. Therefore, it will be difficult for the WWTP to comply with the Three Basin Rule loading restrictions without reductions in wintertime I/I or additional treatment processes.

A process evaluation of the SBR was also conducted without considering the Three Basin Rule. The SBR basins were originally designed for the following flows: 1.37 MGD ADWF, 2.18 MGD $MMDWF_{10}$, 1.96 MGD AWWF, 2.71 MGD $MMWWF_5$, 3.91 MGD $PDAF_5$, and 6.87 MGD PIF_5. Current wet weather flows exceed these design values, although the 2040 dry weather flows are approximately equal to these values. Reductions in I/I would maintain flows closer to the SBR's original design values, thereby delaying significant expansions to the WWTP.

The SBR basins were designed for a BOD_5 and TSS loading (both identical) of 2,254 lbs./day annual average and 4,760 lbs./day maximum month. The current BOD_5 and TSS loads are approximately at the annual average level; however, the maximum month loadings have been less than the design values, and there appears to be additional loading capacity in the SBR as observed by the operating mixed liquor suspended solids (MLSS) concentrations. Historically, the average MLSS concentrations required for optimum treatment have ranged between approximately 1,200 and 1,800 mg/L. SBRs can typically be operated at much higher MLSS concentrations (up to 5,000 mg/L), so there may be additional treatment capacity in the existing SBRs (although the SBRs may be limited in their hydraulic or settling capacity).

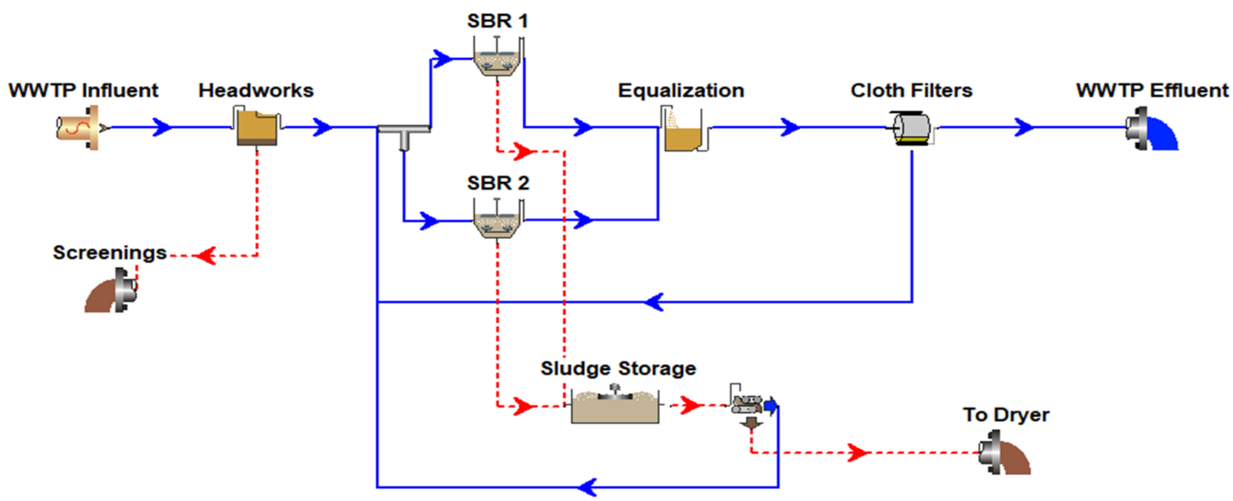
Maintaining an appropriate hydraulic retention time (HRT), dissolved oxygen (DO) level, and food to mass ratio (F:M) while still settling solids (low sludge volume index (SVI)) are the keys to a well-functioning SBR. The SBR basins operate from a maximum water level of 22 feet down to a minimum water level of 15 feet; however, currently, the minimum water level is set at 17 feet. The maximum basin volume in each SBR is approximately 1.3 million gallons. Based on the 2040 $MMWWF_5$ (4.54 MGD), the SBR basins would have a combined HRT of roughly 14 hours, which is acceptable, but on the lower end of what is recommended.

A 2.0 mg/L DO concentration ordinarily is desirable to ensure oxygen is available for metabolism of the influent organic matter (BOD_5) and nitrification. (Nitrification is typically unavoidable due to the detention time in an SBR, so adequate aeration is essential for the removal of both BOD_5 and ammonia). The aeration system (blowers and diffusers) has a firm capacity (with one of the 200 HP blowers out of service) of approximately 14,800 lbs. oxygen per day. Using the 2040 influent BOD_5 loading of 4,097 ppd, and assuming an influent TKN of 540 ppd, peaking factor of 1.25 – and aeration requirements of 1.2 lbs. oxygen per lb. BOD_5, and 4.6 lbs. oxygen per lb. TKN – the required aeration is approximately 9,250 lbs. oxygen per day in 2040. Therefore, the existing aeration system has the firm capacity to handle the aeration requirements, with one of the blowers out of service.

As observed in the Stayton Wastewater Treatment Plant Process Analysis and Modeling Technical Memorandum dated January 2019 (Tech Memo), the F:M ratio has typically been in an optimum range between 0.04 and 0.1 lbs. BOD_5/lb. MLVSS/day, except during the equipment issues and plant upset in early 2018. During the plant upset, the F:M ratio and SVI spiked, and the effluent permit limits were exceeded for TSS and BOD_5. Normally the SVIs have been below 300 mL/g.

A BioWin model was developed to evaluate further the capacity of the SBR (without consideration of the Three Basin Rule). Figure 10.1 shows a schematic of the BioWin model.

FIGURE 10.1: BIOWIN MODEL SCHEMATIC



Inputs into the model included dimensions from the record drawings and cycle times from the WWTP staff. The model utilized the influent flows and loads from Chapter 2 as well as additional parameters, as discussed in Section 10.1. A diurnal peak hourly flow peaking factor of 1.1 from the Tech Memo was used. VSS analysis results from December 2018 showed an influent VSS fraction of 94% of the influent TSS. In the mixed liquor, the MLVSS fraction was approximately 89% of the MLSS. Alkalinity results from October 2018 showed influent alkalinity of 200 mg/L as $CaCO_3$. A total recycle of flow and load of 7% was assumed in the model to account for backwash from the cloth filters and filtrate from the belt filter press.

The model was calibrated using data from December 2019 and then validated using data from April 2019. Based on the BioWin model, the current basins are nearing capacity, and an additional SBR would be necessary to meet the 2040 planning criteria. However, these results are based on the SBR providing optimal performance. Optimal performance, similar to that currently achieved, will not be sufficient to comply with the Three Basin Rule as the flows increase.

Air to each SBR basin travels through a dedicated 14-inch-diameter pipe. With the dedicated blower operating at full power (3,400 SCFM), the flow rate through the 14-inch pipeline would be approximately 53 fps, which is acceptable.

Basin mixing is performed by a 125 HP centrifugal pump for each basin; each pump has a capacity of 11,000 gpm. This corresponds to a mixing intensity of approximately 100 HP per million gallons, which is more than adequate to mix the SBR basins.

There are two decanters in each SBR basin. Each decanter is designed for a maximum flow of 5,600 gpm, so the combined maximum flow from an SBR basin is 11,200 gpm. For the 2040 maximum day flow of 7.82 MGD, a minimum of nine (9) cycles per day would be required to move the water through the SBR basins. It would be very difficult to meet the treatment requirements for several days with this many cycles per day. However, for the 2040 max. month flow of 4.54 MGD, a minimum of approximately five (5) cycles per day are required, which is acceptable.

Following the SBR basins, the wastewater flows through a 30-inch diameter pipe by gravity. At the peak flow of 11,200 gpm per SBR, the flow rate through the 30-inch section of the pipe would be approximately 5.1 fps, which is acceptable. The flow at the beginning of each decant is typically sufficient to scour the pipe of solids, and no issues have been observed by WWTP staff.

Each 15 HP centrifugal WAS pump has a capacity of 900 gpm (1.3 MGD), which is acceptable for the size of the SBR basin. There is no installed redundant WAS pump; however, the mixing pumps can be used to waste. The WAS is pumped through an 8-inch pipe, which has a capacity of approximately 1,600 gpm, so as long as both WAS pumps do not operate at the same time, the line should be adequate.

10.4.6 Post-SBR Equalization Basin

The equalization basin volume is approximately 215,000 gallons. The current minimum water level in each SBR is set by the WWTP staff at 17 ft., which equates to a decant volume of approximately 300,000 gallons. The equalization basin is not sized to hold an entire maximum decant; however, as the decant flow enters the equalization basin, the submersible equalization pumps turn on. These pumps are each rated for 2,410 gpm (3.5 MGD), so the firm capacity with one pump out of service is 4,820 gpm (7.0 MGD). If, as the maximum decant flow enters the equalization basin the equalization pumps are turned on (with one of the pumps on standby), the equalization basin and pumps could theoretically handle the current maximum decant flow. However, the current $PDAF_5$ is 7.17 MGD, which is the limit of the existing equalization basin and pump capacity. This evaluation assumes that two maximum decants do not occur within approximately 30 minutes or less from each other.

As the flows to the SBR increase, either the number of cycles per day or the working water levels would need to change. Each SBR has a possible minimum water level of 15 ft., which corresponds to a maximum decant volume from an SBR of approximately 420,000 gallons. As the flows increase, the SBR minimum water level could be decreased to 15 ft., but the equalization basin would need a storage capacity of approximately 240,000 gallons. Therefore, either the equalization pumps or the equalization basin are undersized for the maximum SBR decant volume.

The 18-inch magnetic flow meter on the equalization pump discharge has a rated capacity of 10.8 MGD, which is more than adequate for the planning period. Similarly, the 18-inch discharge pipe is more than adequate for the 2040 PIF_5 of 9.18 MGD and would be capable of handling the combined flow from the three equalization pumps.

10.3.7 Filtration

Each of the three (3) 10-Disk AquaDisk units has a surface area of 538 square feet (1,614 square feet total) and is rated for a peak flow of 3 MGD. There is space on each of the AquaDisk units for two additional disks, which would increase the capacity of each of the units to 4 MGD. The existing filters have sufficient redundancy, since with one unit out of service the remaining filters can handle 75% of the 2040 max. day flow (7.82 MGD), which is 5.87 MGD. If all three of the equalization basin pumps were operated at the same time, the hydraulic loading rate through the three AquaDisk units would be 4.48 gpm/ft^2, which is less than their rated capacity of 6 gpm/ft^2.

10.4.8 UV System

The UV disinfection system has a rated capacity of 10.2 MGD with two of the banks in operation and a redundant bank in each channel. This is sufficient for the planning period. Similarly, the 24-inch diameter pipe between the filter building and the UV building is sufficient for the 2040 PIF_5 and can accommodate the flow from all three equalization pumps operating concurrently.

10.4.9 Outfall

The outfall pipe and diffusers were evaluated for the 20-year planning flows using the Visual Hydraulics model. Following the UV system, the wastewater exits the WWTP and flows toward the North Santiam River. The outfall is comprised of a 30-inch concrete pipe, and it is approximately 920-feet long. The effluent flow discharges through a two-port submerged diffuser into the North Santiam River. The diffuser ports are each 30 inches long with 8-inch nozzles, and a flanged connection allows for future expansion. The effluent pipe is sufficient for the 20-year planning period.

A mixing zone analysis was made in October 2006. The analysis used an ADWF of 1.90 MGD for the evaluation, which is greater than the 2040 ADWF of 1.25 MGD. The mixing zone analysis found that the existing outfall was sufficient and did not recommend any improvements.

10.4.10 Utility Water System

The Utility Water Storage Basin stores approximately 20,000 gallons; however, the WWTP uses approximately 140,000 gallons on peak days. Each of the four (4) 5 HP utility water system pumps has a rated capacity of 60 gpm. The size of the utility water line is 6 inches. The main water uses come from the belt filter press as well as the dryer cooling tower and boiler. The water demand, batch nature of the SBR process, and inadequate equalization in the Post-SBR Equalization Basin lead to water shortages in the storage basin.

Additionally, the dryer cooling tower and boiler function with fewer problems if potable water is used rather than utility water. For this reason, the plant water system has been switched over from utility water to potable water.

10.4.11 Solids Handling

The working volume for Sludge Storage Tanks #1 and #2 is 172,400 gallons and 213,250 gallons, respectively. In the event the sludge storage tanks are full, the sludge would overflow to a non-aerated sludge storage pond with a capacity of approximately 225,000 gallons. The expected maximum month sludge production during the 20-year period is approximately 60,000 gpd, so the peak month holding capacity would be at least ten days. This is sufficient for a shutdown in the dewatering equipment.

The maximum water surface elevations for Sludge Storage Tank #1 and #2 is 12.7 and 11.5 feet, respectively. Aeration and mixing are provided by a 20 HP floating mechanical aerator in each tank. This equates to a horsepower per million gallons (HP/MG) of 116 and 93, respectively, which is greater than the 40-50 HP/MG typically recommended.

There is no aeration or mixing in the sludge storage pond; however, it is used only in emergencies. A non-clog submersible transfer pump can be used to transfer the sludge back to Sludge Storage Tank #1. The 7.5 HP transfer pump has a rated design capacity of 500 gpm, which is satisfactory for operational purposes.

10.4.12 Solids Dewatering

Belt filter press operation is a function of several variables, including hydraulic loading rate (HLR) and solids loading rate (SLR). The SLR is typically the more limiting of the two. The proper sludge feed rate is essential in obtaining optimum performance of the belt filter press. The proper HLR is dependent on the sludge feed concentration and overall performance requirements. The HLR, as well as the belt speed, must be maintained such that adequate detention time is provided. The SLR is a function of the HLR and the waste activated sludge concentration. Operator experience is the best guide for press loading. When the proper loading rates are exceeded, the effectiveness of the belt filter press is decreased, and the solids capture efficiency is reduced.

The newer belt filter press prior to the dryer has a 1.5 m belt width. The older belt filter press prior to lime stabilization has a 1.7 m belt width. The satisfactory operation for the newer belt filter press will typically be obtained with a SLR of up to 1,000 dry pounds per hour. For the older belt filter press prior to lime stabilization, a SLR of up to 900 dry pounds per hour will typically provide satisfactory operation. Based on the expected maximum month sludge production during the 20-year period of 2,400 dry lbs./day, the belt filter press capacities are acceptable.

Sludge is pumped from the sludge storage tanks to the belt filter presses using one of the two (2) 7.5 HP rotary lobe pumps. Each of the rotary lobe pumps has a capacity of 150 gpm. The HLR capacity for each belt filter press is able to handle this flow rate. Immediately upstream of the pumps is a sludge grinder with a capacity of 600 gpm.

10.4.13 Sludge Drying

The biosolids dryer has a capacity of 1,000 dry pounds per hour. As mentioned previously, the expected maximum month sludge production is 2,400 dry pounds per day, so the biosolids dryer can achieve Class A biosolids for the 20-year planning period. However, the backup system is lime stabilization, which is unable to produce Class A biosolids.

10.4.14 Plant Capacity Summary

A summary of the existing treatment capacity at the plant is provided in Table 10.3. Entries in red indicate process elements that are at or near to their individual capacities.

TABLE 10.3: WASTEWATER PLANT CAPACITY SUMMARY (MGD)

Component	Governing Flow	Firm Capacity Provided	Current Capacity Needed	2040 Capacity Needed	Comments
Influent Screen	PIF_5	10.2	8.35	9.18	Bar screen redundancy – not a fine screen
Influent Pump Station	PIF_5	9.3	8.35	9.18	Room for future pumps
Grit Removal/Classifier	PIF_5	9.3	8.35	9.18	Performance may decrease above 5 MGD
SBR Basins	**$MMWWF_5$**	**4.1**	**4.09**	**4.54**	**Three Basin Rule limits capacity**
Post-SBR Equalization	**$PDAF_5$**	**7.2**	**7.17**	**7.82**	**Pump and basin capacity**
Filtration	75% of $PDAF_5$	6.0	5.38	5.87	Can add more disks to existing units
UV Disinfection	PIF_5	10.2	8.35	9.18	Redundancy bank in each channel

SECTION 11 – WWTP IMPROVEMENT ALTERNATIVES

There are many different alternatives to meet the wastewater facility deficiencies discussed in this master plan. The alternatives with the highest likelihood of being used by the City were considered for evaluation. The goals of the alternatives were to:

- Find solutions that are practical and cost-effective
- Provide facilities capable of reliably meeting permit limits
- Maximize use of existing facilities
- Select facilities that can be constructed without negatively impacting effluent quality
- Identify solutions that could be phased to reduce debt and minimize user rate increases

If a WWTP deficiency discussed in the previous chapters had one clear preferred solution (such as installing an additional screen, purchasing critical spare pump motors, repairing the sludge storage pond, etc.), then the solution is not discussed here, but is included in the individual project summary sheets found in Appendix D.

The advantages, disadvantages, and comparative costs of the alternatives are presented in this chapter. The cost estimates are a Class 5 cost opinion, as defined by the Association for the Advancement of Cost Engineering. They include estimated construction costs with markups of 10% for general conditions, a contingency of 30%, 15% contractor overhead and profit (OH&P), and general and administrative services (including design administration, construction observation, loan support, legal services, etc.) of 25% based on total construction cost.

In addition to project capital costs, annual operation and maintenance (O&M) costs are compared to arrive at a more complete picture of the alternative costs. A 20-year life-cycle cost analysis is provided for most of the alternatives, based on a real discount rate (inflation removed) of 0.3%. The equipment (unless a short-lived asset) is assumed to have a 20-year useful life so no depreciation or salvage value is included for comparing the alternatives. To estimate maintenance costs, an electricity rate of $0.10 per kWh was used for power costs, $0.90 per therm was used for natural gas costs, and a labor cost of $30 per hour was used.

11.1 DISCHARGE ALTERNATIVES

The current method of discharge is into the North Santiam River. Several different discharge alternatives were discussed with the City. Regionalization, due to the political complexity, physical distance, and pipeline cost with another city of similar size or larger to share wastewater facilities, was not cost-effective and not of interest to the City currently.

<u>Discharge Alternative 1: Continue River Discharge</u>

Under this alternative, the City would continue discharging all flows received to the North Santiam River following treatment. However, improvements would be needed to remove additional loading to the North Santiam River to comply with the Three Basin Rule. The improvements at the WWTP would include adding membrane filters between the cloth media filters and the UV disinfection system to further reduce loadings to the river. Since the SBRs and Post-SBR EQ are at capacity, as shown in Section 10, we have assumed one new SBR basin and a larger Post-EQ SBR basin (by adding piping between the Selector Cell and existing Post-SBR EQ) is included.

Discharge Alternative 2: Add Winter Equalization and Continue River Discharge of All Flows

This alternative would require equalization storage to divert influent received above a certain treatment threshold. It is anticipated that equalization storage would be used primarily during the winter to store excess flow volume until it can be treated at a rate which complies with the Three Basin Rule. Assuming a 4.1 MGD flow treatment threshold (based on the current SBR capacity from Section 10) and using the 2019 average wet-weather design flow, the historical monthly precipitation data from the National Climatic Data Center (NCDC) station in Stayton, and evaporation data from the Western Regional Climate Center – North Willamette Research and Extension Station; the total required equalization storage volume based on current flows is approximately 17 million gallons. Estimating the future storage need based on the projected increase in the average wet-weather design flow from 2019 to 2040, the total required storage volume increases to approximately 34 million gallons. It is worth noting that precipitation has a significant impact on the amount of storage required/available.

For this alternative, the influent would continue to be screened in the existing headworks. New pumps would be placed in the existing influent pump station to send the peak flows to the equalization lagoons. For Alternative 2, it was assumed that land would be purchased near the WWTP at a price of $30,000 per acre, and new lagoons with surface aerators would be installed. A return pump station was included in this alternative; however, it might be possible to utilize an automatic control valve to return the equalized water to the WWTP without a return pump station. The equalization system will be automated. As high influent flow rates are measured in the headworks, the new pumps in the influent pump station will turn on and pump the peak flows to the equalization lagoons. The return flows from the equalization will also be programmed to happen automatically if the maximum water level is reached in the equalization basins. The WWTP staff will also use the return pump station (or control valve) to move the water back to the WWTP when the influent flows are lower.

Typically, some water will remain in the equalization storage (either influent or rainwater) to avoid septic conditions. However, the lagoons will also have a slope for draining and cleaning if the City wants to drain the lagoons periodically. The WWTP staff will probably adjust how they operate and how frequently they wash the equalization lagoons to avoid odors and nuisance conditions. Aeration equipment will help keep the influent from becoming septic.

Another alternative (Alternative 2a) would be to purchase the existing lagoons near the WWTP (old NORPAC facility). It is possible that the existing lagoons could be used for the equalization as the combined storage volume of the three lagoons is approximately 37 million gallons, which is more than is needed for the planning period. The old NORPAC facility has surface aerators, which could be used to keep the influent mixed and aerated to avoid septic conditions. However, the condition of the existing NORPAC lagoons and aerators are unknown, so a range of costs for the storage lagoons and purchase price are shown for Alternative 2a. A due diligence line item was included to investigate the lagoons and land prior to purchasing.

Discharge Alternative 3: Winter Effluent Equalization, Steady River Discharge, and Irrigate with Peak Winter Flows

In this alternative, the City would store peak effluent flows during the winter so that the Three Basin Rule is not exceeded. The treated and stored water would be land applied in the summer. The treated water would need to meet Class C recycled water regulations. Permit requirements for agricultural reuse are likely to continue to be less stringent as surface water discharge requirements to North Santiam River as the nutrients in the WWTP effluent are useful for plants.

Use of treated wastewater outside of the WWTP is governed by recycled water regulations, as outlined in Oregon Administrative Rules (OAR) 340-055. The April 2008 revisions to Oregon's Recycled Water Use Rules allow the use of recycled water for beneficial purposes, if the use provides a resource value and protects public health and the environment. Replacing another water source that would be used under the same circumstances or supplying nutrients to a growing crop are considered as resource values and beneficial purposes. OAR 340-055 defines five categories of effluent, identifies allowable uses for each category, and provides requirements for treatment, monitoring, public access, and setback distances. Irrigation of fodder, fiber, and seed crops not for human consumption is allowed for any class of effluent. Fewer restrictions are imposed for higher quality effluent, as shown in Table 11-1. Stayton's effluent would be required to achieve Class C recycled water requirements, which it is currently capable of meeting.

TABLE 11-1: REUSE REQUIREMENTS BY EFFLUENT CATEGORY

	Class A	Class B	Class C	Class D	Non-disinfected
Treatment[1]	O,D,F	O,D	O,D	O,D	O
Total coliform, 7-day median #/100 mL	2.2 [2]	2.2 [2]	23 [3]	- [4]	Per permit
Turbidity, NTU	2	-	-	-	
Public access [5]		Limited	Limited	Controlled	Prevented
Setback to property line [6]		10 feet	70 feet	100 feet	Per permit
Setback to water supply source		50 feet	100 feet	100 feet	150 feet

[1] O = oxidized, D = disinfection, F = filtration, RWUP = Recycle Water Use Permit
[2] Must not exceed 23 total coliform organisms per 100 milliliters (ml) in any single sample
[3] Must not exceed 240 total coliform organisms per 100 ml in any two consecutive samples
[4] Rather than total coliform, Class D Recycled Water is required to sample for E. coli. E. coli is a subgroup of the total coliform organisms, so a total coliform analysis includes the E. coli organisms. For Class D Recycled Water, the 30-day log mean must not exceed 126 E. coli organisms per 100 ml; and must not exceed 406 E. coli organisms per 100 ml in a single sample
[5] Limited public access: no direct contact during irrigation cycle
[6] Sprinkler irrigation assumed

For recycled water use, groundwater must be protected in accordance with the requirements of OAR 340-040. For agricultural use, this typically translates to irrigating at agronomic rates to match the net irrigation requirements of the crops. While the Willamette Valley is widely known for grass seed crops, alfalfa is more favorable as it uses more water per acre. Typical water application rates for grass seed crops are approximately 15.5 inches per acre per year while alfalfa hay will require 20 inches or more (Oregon Crop Water Use and Irrigation Requirements, 1992, OSU ext. Pub. 8530). Therefore, the theoretical area of irrigated farmland needed to irrigate the projected 2040 winter seasons excess flow during the growing season, based on alfalfa hay requirements and assuming a 90% irrigation efficiency, is approximately 80 acres.

There are several considerations when selecting land which include topography, groundwater levels, groundwater pollutant concentrations, general soil conditions, climate, land use, well locations, and distance to water bodies. Nutrient applications would vary from year to year depending on the amount of effluent applied; however, it could be expected that approximately 70 pounds per acre of nitrogen and 14 pounds per acre of phosphorus could be provided which would account for approximately 30-40% of the nitrogen and 100% of the phosphorus requirement depending on the crop grown.

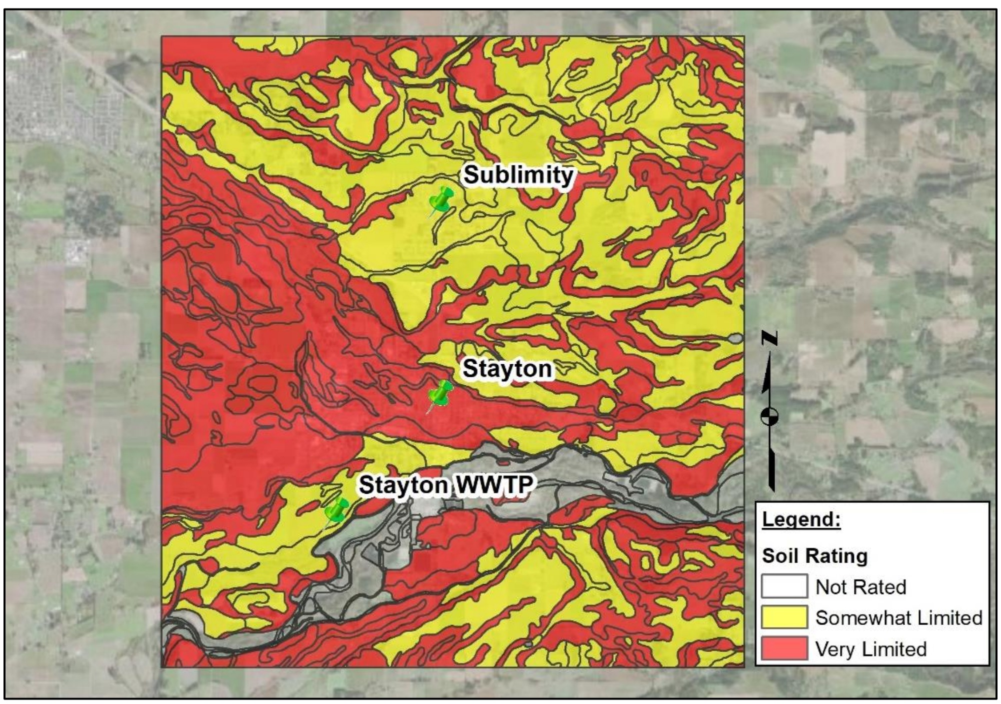
A preliminary assessment of the feasibility of land application was done based on soil suitability ratings from the NRCS Soil Data Explorer. The rating class terms, as defined by NRCS, are summarized in Table 11-2.

TABLE 11-2: NRCS SOIL RATING FOR REUSE

Rating	Suitability for Specified Use	Ability to Overcome Limitations	Expected Performance	Expected Maintenance
Not Limited	Very favorable	NA	Good	Very low
Somewhat Limited	Moderately favorable	Can be minimized by special planning, design, or installation	Fair	Moderate
Very Limited	One or more unfavorable features	Generally, cannot be overcome without major soil reclamation, special design, or expensive installation procedures	Poor	High

Figure 11-1 shows the NRCS rating map for disposal of wastewater by land application. Much of the land in and around Stayton is somewhat or very limited (yellow and red areas on Figure 11-1).

FIGURE 11-1: NRCS LAND SUITABILITY FOR REUSE WATER



A typical arrangement for most communities is to have a farmer handle operation of the land application site, including crop management and irrigation equipment maintenance. The farmer may also be asked to pay for pumping costs from storage to the irrigation site, and for use of the site based on a flat rate per acre or crop yield. Any agreement with the farmer must include conformance with reuse permit requirements (e.g. no ponding or runoff, application at rates not to exceed published irrigation water requirements, etc.). The City would likely be responsible for all costs of monitoring (soils, crops, and groundwater) required by the reuse permit. It should be noted that, if the farmland used for effluent disposal is privately owned, the City may have limited control over when the effluent is used. For this evaluation it was assumed that the City would purchase the land due to the need to maintain control for land application permitting purposes.

For this evaluation it was assumed that the treated water would be pumped to land up to 1 mile away from the WWTP and that the storage site would be located next to the WWTP in new lagoons. A cost of $30,000 per acre was assumed to purchase a suitable application property and land for the lagoons. Recycled water would need to be pumped to the lagoons during the winter and then to the application property during the summer. For this evaluation it was assumed the City would be responsible for the irrigation and reuse system maintenance but that harvesting, and handling of the crop would be managed through a partnership with an area farmer.

Since the equalization is downstream of the WWTP, improvements to the SBR and Post-SBR equalization are needed like the first alternative. We have included one new SBR basin and a larger Post-EQ SBR basin.

Discharge Alternative Evaluation

A summary of the advantages and disadvantages of the discharge alternatives are shown in Table 11-3. With the addition of an ultrafiltration system, Alternative #1 would also likely increase the WWTP classification to a Class IV Facility which may require additional certifications for City operators.

TABLE 11-3: SUMMARY OF DISCHARGE ALTERNATIVES ADVANTAGES / DISADVANTAGES

Alternative	Advantages	Disadvantages
Discharge Alt. 1 – Continued River Discharge	• Same discharge method as currently used.	• High operating costs for membranes. • Requires additional and ongoing plant upgrades. • Higher risk of permit violations. • WWTP classification increases to Class IV
Discharge Alt. 2 – Winter Influent Equalization and River Discharge	• Influent can be held to avoid process upsets. • Same discharge method as currently used. • Reuses existing infrastructure.	• Additional complexity of operation and maintenance. • Treating rainwater captured in equalization lagoons
Discharge Alt. 3 – Winter Effluent Equalization and Land App.	• Flexibility to meet permit limits. • Beneficial reuse of the City's wastewater.	• Highest capital costs since WWTP capacity expansion is needed. • Risk of transmission failures. • Complexity of operating and maintaining additional infrastructure.

A preliminary cost comparison of the alternatives is shown in Table 11-4. Other improvements at the WWTP that were common for the alternatives (e.g. replacing the filter cloth in Filter #3, generator upgrades, etc.) were not included in the capital or operating costs. A due diligence line item was included to investigate the lagoons and land (depending on the alternative) prior to purchasing.

TABLE 11-4: EFFLUENT DISCHARGE COMPARISON COSTS (2020)

Item	Discharge Alt. 1: Continue River Discharge	Discharge Alt. 2: Add Winter Eq., Continue River Discharge All Flows	Discharge Alt. 2a: Same as Alt. 2 Except Purchase Old NORPAC Lagoons	Discharge Alt. 3: Winter Eff. Eq., Steady River Discharge, Irrigate Peak Winter Flows
Project Costs				
SBR Basins and Equipment	$4,100,000	$0	$0	$4,100,000
Post SBR Equalization	$40,000	$0	$0	$40,000
Membrane Filters	$8,270,000	$0	$0	$0
Storage Lagoons	$0	$4,900,000	$0 to $4,900,000	$4,900,000
Lagoon Pumping & Transmission	$0	$430,000	$430,000	$1,900,000
Application Site	$0	$0	$0	$140,000
Subtotal	***$12,410,000***	***$5,330,000***	***$430,000 to $5,330,000***	***$11,080,000***
General Conditions (10%)	$1,250,000	$540,000	$50,000 to $540,000	$1,110,000
Subtotal	***$13,660,000***	***$5,870,000***	***$480,000 to $5,870,000***	***$12,190,000***
Contingency (30%)	$4,100,000	$1,770,000	$150,000 to $1,770,000	$3,660,000
Subtotal	***$17,760,000***	***$7,640,000***	***$630,000 to $7,640,000***	***$15,850,000***
Contractor OH&P (15%)	$2,670,000	$1,150,000	$100,000 to $1,150,000	$2,380,000
Total Construction Cost	**$20,430,000**	**$8,790,000**	**$730,000 - $8,790,000**	**$18,230,000**
General and Administrative Costs (25%)	$5,110,000	$2,200,000	$190,000 to $2,200,000	$4,560,000
Land or Lagoon Purchase Price	$0	$525,000	$2,500,000 to $5,000,000	$2,925,000
Due Dillegence for Purchase	$0	$15,000	$200,000	$30,000
Total Project Cost	**$25,540,000**	**$11,530,000**	**$3,620,000 to $16,190,000**	**$25,715,000**
Estimated Annual O&M	$294,700	$71,900	$71,900	$100,000
20-Year Life Cycle Cost	**$31,260,000**	**$12,930,000**	**$5,020,000 to $17,590,000**	**$27,660,000**

Annual O&M Costs				
Electricity	$74,800	$42,300	$42,300	$61,500
Chemicals	$15,400	$0	$0	$0
Parts	$184,700	$21,800	$21,800	$29,500
Personnel	$19,800	$7,800	$7,800	$9,000
Estimated Annual O&M	**$294,700**	**$71,900**	**$71,900**	**$100,000**
20-Year Life Cycle Cost	**$31,260,000**	**$12,930,000**	**$4,820,000 to $17,390,000**	**$27,660,000**

Discharge Recommendation

The recommended alternative is Winter Influent Equalization followed by River Discharge (Alternative #2). If the I/I is significantly reduced, then the lagoons may not be needed, but they would still provide the City with flexibility to maximize WWTP operations as steady flows can be helpful for overall WWTP performance, especially SBR operation. Depending on the condition and price of the old NORPAC lagoons, it may be advantageous for the City to purchase the existing lagoons (Alternative 2a). No impact to the WWTP Facility Classification is anticipated from implementing this alternative.

11.2 POST-SBR EQUALIZATION (EQ) ALTERNATIVES

The current Post-SBR EQ system is currently at capacity. If Winter Influent Equalization is utilized (Discharge Alternative #2), the flow to the Post-SBR EQ system may remain unchanged; however, this is based on the SBR continuing to operate without issues. Due to either selecting a different discharge recommendation, or due to risks of SBR upsets, the City desired to evaluate different Post-SBR EQ alternatives.

Post-SBR EQ Alternative 1: Pipe Decant to Selector Cell and Combine Selector with Existing EQ

With this alternative, the decant piping from the SBRs would go to both the Selector Cell and the Post-SBR EQ basin. This would eliminate the Selector Cell for use before the SBR system; however, it would increase the storage capacity of the Post-SBR EQ system. The Selector Cell would likely fill up a maximum of 6 feet, which equates to a maximum storage volume in the Selector Cell of approximately 150,000 gallons. With the existing Post-SBR EQ basin, the combined storage volume would be approximately 365,000 gallons. As discussed in Section 10, the maximum allowable decant volume from the SBR (dropping the minimum water level to 15 ft.) would require a storage volume of 240,000 gallons with the existing pumps, so this alternative would meet this requirement. This alternative would also require the least changes to the system.

Post-SBR EQ Alternative 2: Add Decant Pump Station, Pump to Selector Cell, and Pump from Selector to Filters

The intent of this alternative is to utilize the maximum storage volume in the existing Selector Cell. The maximum storage volume is approximately 500,000 gallons. In this alternative a lift station would be added between the SBRs and the Selector Cell, and second lift station would be added to pump the equalized effluent to the filters.

Post-SBR EQ Alternative 3: Add Decant Pump Station, Pump to Selector Cell and Combine Selector with Existing EQ

This alternative is a combination of Alternatives 1 and 2. The alternative would use the maximum storage capacity of the Selector Cell and the maximum storage capacity of the existing Post-SBR EQ basin. The combined storage capacity would be approximately 715,000 gallons. This alternative would include one new lift station, which would be between the SBRs and the Selector Cell.

Post-SBR EQ Alternative 4: Enlarge the Existing Post-SBR EQ

In this alternative, the existing Post-SBR EQ basin would be increased to increase the total capacity to approximately 240,000 gallons. Another alternative would be to increase the pumping capacity; however, to limit the effect on the downstream filters and UV disinfection systems, enlarging the Post-SBR EQ was more attractive. Space around the existing Post-SBR EQ basin is limited. Also, construction within the existing basin would be difficult, so modifications would take place adjacent to the existing Post-SBR EQ prior to a limited shutdown period.

Post-SBR EQ Alternative Evaluation

A summary of the advantages and disadvantages of the alternatives are shown in Table 11-5.

TABLE 11-5: SUMMARY OF POST-SBR EQ ALTERNATIVES ADVANTAGES/DISADVANTAGES

Alternative	Advantages	Disadvantages
Post-SBR EQ Alt. 1 – Pipe Decant to Selector Cell and Combine Selector with Existing EQ	• Lowest capital costs. • Least changes. • Low maintenance costs. • No additional electrical or controls.	• Would not be able to use the Selector Cell as pre-SBR equalization. • Would provide additional capacity, but not as much as other alternatives.
Post-SBR EQ Alt. 2: Add Decant Pump Station, Pump to Selector Cell, and Pump from Selector to Filters	• Maximize existing storage volume. • Capable of holding the maximum decant volume.	• Highest capital cost. • High operating costs. • Additional power and controls for lift stations. • Does not use existing Post-SBR EQ basin and pumps.
Post-SBR EQ Alt. 3: Add Decant Pump Station, Pump to Selector Cell and Combine Selector with Existing EQ	• Maximize existing storage volume. • Highest storage capacity – capable of holding decants from two SBRs simultaneously	• Highest capital cost. • High operating costs. • Additional power and controls – lift station and modulating valve to existing Post-SBR EQ.
Post-SBR EQ Alt. 4: Enlarge the Existing Post-SBR EQ	• Low maintenance costs. • No additional electrical or controls. • Keeps the Selector Cell available.	• Bypass pumping needed during changeover. • Least additional storage capacity.

A preliminary cost comparison of the alternatives is shown in Table 11-6.

TABLE 11-6: POST-SBR EQ COMPARISON COSTS (2020)

Item	Post-SBR EQ Alt. 1 – Pipe Decant to Selector Cell and Combine Selector with Existing EQ	Post-SBR EQ Alt. 2: Add Decant Pump Station, Pump to Selector Cell, and Pump from Selector to Filters	Post-SBR EQ Alt. 3: Add Decant Pump Station, Pump to Selector Cell and Combine Selector with Existing EQ	Post-SBR EQ Alt. 4: Enlarge the Existing Post-SBR EQ
Project Costs				
Site Work	$ 20,000	$ 80,000	$ 60,000	$ 100,000
Piping	$ 20,000	$ 100,000	$ 40,000	$ 10,000
Decant Lift Station	$ -	$ 350,000	$ 350,000	$ -
Control Valve	$ -	$ -	$ 10,000	$ -
Post-SBR EQ Lift Station	$ -	$ 350,000	$ -	$ -
Additional Post-SBR EQ Storage	$ -	$ -	$ -	$ 300,000
Electrical/Controls	$ -	$ 150,000	$ 90,000	$ -
Subtotal	***$ 40,000***	***$ 1,030,000***	***$ 550,000***	***$ 410,000***
General Conditions (10%)	$ 10,000	$ 110,000	$ 60,000	$ 50,000
Subtotal	***$ 50,000***	***$ 1,140,000***	***$ 610,000***	***$ 460,000***
Contingency (30%)	$ 20,000	$ 350,000	$ 190,000	$ 140,000
Subtotal	***$ 70,000***	***$ 1,490,000***	***$ 800,000***	***$ 600,000***
Contractor OH&P (15%)	$ 20,000	$ 230,000	$ 120,000	$ 90,000
Total Construction Cost	**$ 90,000**	**$ 1,720,000**	**$ 920,000**	**$ 690,000**
General and Administrative Costs (25%)	$ 30,000	$ 430,000	$ 230,000	$ 180,000
Total Project Cost	**$ 120,000**	**$ 2,150,000**	**$ 1,150,000**	**$ 870,000**
Annual O&M Costs				
Electricity	$ 5,300	$ 10,600	$ 10,600	$ 5,300
Parts	$ 4,200	$ 8,300	$ 8,300	$ 4,200
Personnel	$ 4,200	$ 8,400	$ 8,400	$ 4,200
Estimated Annual O&M	**$ 13,700**	**$ 27,300**	**$ 27,300**	**$ 13,700**
20-Year Life Cycle Cost	**$ 390,000**	**$ 2,680,000**	**$ 1,680,000**	**$ 1,140,000**

Post-SBR EQ Alternative Recommendation

Even though the additional capacity is not the largest, the recommended alternative is to add piping to combine the Selector Cell and the Post-SBR EQ basin (Alternative #1). This alternative has the least impact to current operations, lowest capital cost, and does not require any programming. This additional capacity will benefit downstream filter and UV operation.

11.3 BIOSOLIDS DRYING ALTERNATIVES

The City of Stayton is pleased to provide their community with Class A EQ (exceptional quality) biosolids. However, the existing dryer has been challenging and requires a significant amount of expertise to operate. Additionally, emergency repairs to keep the dryer system running have been very expensive. The City is interested in options to keep producing Class A EQ dried biosolids but would like to make it less expensive and challenging to operate.

Biosolids Drying Alternative 1: Purchase Spare Parts for Existing Dryer

Having spare parts on hand could help reduce the cost of repairs, since they could be used to perform preventative maintenance. Replacing parts prior to the part failing is significantly less expensive than an emergency repair. The parts can typically be purchased at a lower cost, shipping expenses are significantly lower, and repairs can be made during regular hours with a trained team.

Biosolids Drying Alternative 2: Replace Dryer

The number of U.S. municipalities that utilize thermal drying continues to increase as landfilling and land application of biosolids becomes more difficult. In 2016 the number of thermal drying facilities operating or under construction in the U.S. was 105 according to the Sixth Edition of WEF Manual of Practice No. 8. The City is interested in replacing the dryer if another system can be found that is safer, more reliable, and has better manufacturer support. Additionally, the City would like better controls and the ability to hold and continue treating biosolids until certain parameters are met. Ideally the dryer would operate unattended using shutdown processes and alarms in the event of an emergency to improve the throughput of the system, increase the drying efficiency, and decrease the wear on parts from frequent shutdowns and startups.

Biosolids dryers are differentiated mainly by their method of heat transfer – either direct or indirect. Direct dryers typically use hot air to heat the biosolids directly in the dryer. There is no separation of the hot air from the biosolids. Some common types of direct dryers are fluidized bed dryers, flash dryers, belt dryers, and rotary drum dryers. Of this group, belt dryers are becoming more popular as they use a lower temperature. One of the major issues with dryers is the large amount of natural gas or electricity needed, so if a belt dryer can utilize a lower temperature, that can be a major advantage. Due to the belt dryer footprint and height, the existing roof would either need to be raised or the building expanded to accommodate a belt dryer.

A second type of biosolids dryer is an indirect dryer. With an indirect dryer, metal walls separate the biosolids from the heat source (usually hot water, steam, or oil). Examples of indirect dryers include paddle dryers, fluidized bed dryers, rotary chamber dryers, pressure filter/vacuum dryers, vertical tray dryers, and hollow-flight dryers. Stayton's current dryer is a hollow-flight dryer that uses hot oil in the screw flights to heat the biosolids. The oil temperature for a paddle dryer is typically around 380°F, which is less dangerous and less expensive to heat than the 500°F used for Stayton's existing dryer. A paddle dryer would be a fairly simple changeout for the dryer as most of the existing auxiliary equipment could be reused.

Another type of dryer is a solar dryer. Solar dryers use the radiant energy of the sun to heat the biosolids. They are capable of meeting Class A EQ biosolids; however, they require previous stabilization prior to drying (e.g., aerobic digestion, lime stabilization, etc.). Therefore, solar dryers were not considered to replace the existing dryer due to the additional capital expense.

Biosolids Drying Alternative Evaluation

For this evaluation, a belt dryer and paddle dryer were chosen as alternatives to replace Stayton's existing dryer. To help the City decide on the biosolids drying alternative, the following evaluation criteria were selected and assigned a weight factor. The weighted evaluation is in Table 11-9.

- Safety – 30%; measure of the ability for operators to easily maintain and deal with issues (highest grade (1) provides the most safety)
- Longevity – 25%; life expectancy of the dryer (highest grade (1) means longest life)
- Reliability – 20%; ability to run unattended (highest grade (1) means the equipment can run constantly unattended)
- On-Going Support – 15%; technicians and parts are readily available, and the company is committed to the product (highest grade (1) means qualified support and parts available)
- Controls – 10%; level of automation to allow for operator control and optimization (highest grade (1) means controls provide operators with the most process automation)

Table 11-7 shows a high-level summary of the advantages and disadvantages of the alternatives.

TABLE 11-7: SUMMARY OF DRYING ALTERNATIVES ADVANTAGES / DISADVANTAGES

Alternative	Advantages	Disadvantages
Biosolids Drying Alt. 1 – Purchase Spare Parts for Existing Dryer	• Lowest capital costs. • Operator familiarity.	• Cannot operate unattended. • Operator's will continue to have ongoing safety and support issues. • Equipment longevity is a concern. • Hot oil temperature is higher than alternatives.
Biosolids Drying Alt. 2a – Replace Dryer with a Belt Dryer with a Proven Track Record	• Can operate unattended. • Lower operating temperature, so lower operating costs and safety hazards. • Can have higher quality end-product (more nitrogen). • Dried biosolids would be more granular (less dust). • Requires less time to heat up and cool down.	• Highest capital cost. • More air / odor treatment required. • Larger footprint – would need to either raise the roof or build a separate room. Placing the belt dryer in another room could allow the existing dryer to be used for redundancy.
Biosolids Drying Alt. 2b – Replace Dryer with a Paddle Dryer with a Proven Track Record	• May address safety, support, and longevity issues. • Likely fits in existing footprint. Can use some existing equipment. • Low odors. • Lower capital cost than belt dryer.	• Like existing – takes more time to warm up and cool down than belt dryer.

A preliminary cost comparison of the alternatives is shown in Table 11-8. The life-cycle cost comparison assumed there would be no hauling expenses as the volume and weight of the dried biosolids from the three alternatives would be similar.

TABLE 11-8: BIOSOLIDS DRYING COMPARISON COSTS (2020)

Item	Biosolids Drying Alt. 1 – Purchase Spare Parts for Existing Therma-Flite Dryer	Biosolids Drying Alt. 2a – Replace Therma-Flite with a Belt Dryer	Biosolids Drying Alt. 2b – Replace Therma-Flite with a Paddle Dryer
Project Costs			
Spare Parts	$ 250,000	$ -	$ -
Equipment (including Installation)	$ -	$ 3,420,000	$ 2,300,000
Room Modifications	$ -	$ 200,000	$ -
Electrical/Controls	$ -	$ 150,000	$ 150,000
Subtotal	***$ 250,000***	***$ 3,770,000***	***$ 2,450,000***
General Conditions (10%)	$ -	$ 380,000	$ 250,000
Subtotal	***$ 250,000***	***$ 4,150,000***	***$ 2,700,000***
Contingency (30%)	$ -	$ 1,250,000	$ 810,000
Subtotal	***$ 250,000***	***$ 5,400,000***	***$ 3,510,000***
Contractor OH&P (15%)	$ -	$ 810,000	$ 530,000
Total Construction Cost	**$ 250,000**	**$ 6,210,000**	**$ 4,040,000**
General and Administrative Costs (25%)	$ -	$ 1,560,000	$ 1,010,000
Total Project Cost	**$ 250,000**	**$ 7,770,000**	**$ 5,050,000**
Annual O&M Costs			
Electricity / Fuel	$ 55,100	$ 41,900	$ 49,600
Parts	$ 55,200	$ 8,900	$ 8,900
Personnel	$ 62,900	$ 16,800	$ 42,000
Estimated Annual O&M	**$ 173,200**	**$ 67,600**	**$ 100,500**
20-Year Life Cycle Cost	**$ 3,610,000**	**$ 9,090,000**	**$ 7,000,000**

The biosolids drying alternatives were graded on how well they met these criteria in Table 11-9.

TABLE 11-9: BIOSOLIDS DRYING EVALUATION CRITERIA

Evaluation Criteria	Weight	Biosolids Drying Alt. 1 – Purchase Spare Parts for Existing Therma-Flite Dryer	Biosolids Drying Alt. 2a – Replace Therma-Flite with a Belt Dryer	Biosolids Drying Alt. 2b – Replace Therma-Flite with a Paddle Dryer
Safety	0.30	0.2	1.0	0.5
Longevity	0.25	0.2	0.8	0.6
Reliability	0.20	0.3	1.0	0.5
On-Going Support	0.15	0.3	0.7	0.7
Controls	0.10	0.4	1.0	0.5
Total Rating	1.00	0.26	0.91	0.56

Biosolids Drying Alternative Recommendation

Due to the high capital cost for a new dryer system, it is recommended to begin budgeting for a new dryer system to replace the existing. The selection of the type of new dryer should be made during the predesign phase after visiting installations and further discussions with operators; however, based on the evaluation performed during this planning study, the City's preference is a belt dryer due to its safety, reliability, longevity, and controls.

11.4 ENVIRONMENTAL IMPACTS

The potential environmental impacts of the selected alternatives are summarized provided below.

Land Use / Prime Farmland / Formally Classified Lands

It is not anticipated that any of the selected alternatives or other improvements will disrupt prime farmland. The improvements will take place at the WWTP or existing nearby lagoons.

Floodplains

As shown in Section 2 and Appendix A, some improvements would be located inside the 100-year and 500-year floodplains. While none of the alternatives would create new obstructions to the floodplain, there may be some permitting/exclusion requirements for these projects.

Wetlands

The alternatives are not located in wetland areas as they will be in already disturbed areas of the WWTP or in the existing nearby lagoons.

Cultural Resources

It is not anticipated that any of the alternatives will interfere with cultural resources.

Biological Resources

The improvements are on previously disturbed lands, so there are no expected new impacts on biological resources. ESA/EFH consultations may be required for publicly funded projects.

Water Resources

Modifications to the WWTP to reduce flow fluctuations should improve treatment efficiency and effluent quality.

Socio-Economic Conditions

It is not anticipated that these improvements will have a disproportionate effect on any segment of the community.

11.5 LAND REQUIREMENTS

The winter equalization alternative would require land to be purchased. All other improvements would be located within the existing WWTP site.

11.6 POTENTIAL CONSTRUCTION PROBLEMS

The depth of the water table and subsurface rock may affect the construction of the alternatives. However, subsurface investigations were not within the scope of this project. Construction techniques to effectively manage excavation, dewatering, and sloughing issues should be required of any construction plans. Construction plans for any of the alternatives should also include provisions to control dust and runoff.

11.7 SUSTAINABILITY CONSIDERATIONS

Sustainable utility management practices include environmental, social, and economic benefits that aid in creating a resilient utility.

Water and Energy Efficiency

Adding equalization would result in a more balanced use of equipment, which would reduce energy consumption, as well as improve treatment reliability and water quality. Additionally, the City is in the process of replacing its remaining positive displacement blower with a more efficient blower. The City intends to continue to look for opportunities to reduce energy and decrease water use.

SECTION 12 – CAPITAL IMPROVEMENT PLAN (CIP)

This section outlines the recommended plan to address the wastewater facility deficiencies identified in previous sections. The alternative evaluation conducted in Chapters 5 and 11 helped the City make decisions for the wastewater system improvements.

12.1 BASIS FOR ESTIMATE OF PROBABLE COST

Capital costs developed for the recommended improvements are Class 4 estimates as defined by the Association for the Advancement of Cost Engineering (AACE). Actual construction costs may differ from the estimates presented, depending on specific design requirements and the economic climate when a project is bid. An AACE Class 4 estimate is normally expected to be within -50 and +100 percent of the actual construction cost. As a result, the final project costs will vary from the estimated presented in this document. The range of accuracy for a Class 4 cost estimate is broad, but these are typical accuracy levels for planning work.

The costs are based on experience with similar recent collection system and WWTP upgrade projects. Equipment pricing from manufactures of the large equipment items was also used to develop the estimates. The total estimated probable project costs include contractor markups and 30% contingencies, which is typical of a planning-level estimate. Overall project costs include total construction costs, costs for engineering design, construction management services, inspection, as well as administrative costs. For the collection system projects, the contractor's overhead and profit are worked into the line items. The amount of engineering for the Mill Creek Pump Station is anticipated to be a more significant percentage of the total project cost than the pipeline improvement project.

12.2 SUMMARY OF COSTS (20-YEAR CIP)

The cost summary of the projects is listed in Table 12-1 (Capital Improvement Plan). The system development charge (SDC) eligibility was factored using the expected growth of the existing peak flow to the projected 2040 peak flow. The amount of capacity that can be utilized for future connections up to the projected 20-year planning period is used as the percentage for SDC eligibility. Priority 1 projects are the short-term projects to be completed in the next six years. Costs shown are planning-level estimates and can vary depending on market conditions. These costs should be updated as the project is further refined in the pre-design and design phases. Individual project sheets for Priority 1 projects are included in Appendix H. Each project sheet consists of a project objective, description, location map, and cost estimate.

The primary driver/s for each CIP project is identifed in the third column of Table 12-1. Priorities are set based on modeling performed as part of this facilities planning study and discussions with City staff. Priority 1 collection system improvements address identified potential overflows in the system and more immediate needs of the existing pump stations. Priority 2 collection system projects address remaining surcharging identified in the hydraulic evaluation. Priority 1 WWTP improvements are focused on the capability to meet permit requirements, operate safely, and handle flow increases as the City continues to grow. Priority 2 WWTP improvements address needs that will reduce O&M costs, decrease operational risks, and address concerns that will arise later in the 20-year planning period. When projects are undertaken, the models, data, assumptions, etc. are re-evaluated to ensure any necessary adjustments to the project's basis are incorporated.

TABLE 12-1: SUMMARY OF COSTS (20-YEAR CIP)

ID#	Item	Primary Purpose(s)	Total Estimated Cost (2020)	SDC Growth Apportionment %	SDC Growth Apportionment Cost	City's Estimated Portion
Priority 1 Improvements						
1.1	Pipeline Upsizing on Jetters and Ida	Capacity	$ 2,943,000	6%	$ 170,213	$ 2,772,787
1.2	Short Term Pump Station Upgrades	Operations, Safety	$ 270,000	22%	$ 59,772	$ 210,228
1.3	Winter Equalization	Permit Compliance, Capacity, Operations	$ 11,530,000	100%	$ 11,530,000	$ -
1.4	Influent Pump Control	Permit Compliance, Operations	$ 103,000	100%	$ 103,000	$ -
1.5	Post-SBR Equalization	Permit Compliance, Capacity, Operations	$ 120,000	100%	$ 120,000	$ -
1.6	Miscellaneous Parts	Redundancy, Operations	$ 202,000	14%	$ 28,280	$ 173,720
1.7	Turbo Blower Replacement	Operations	$ 990,000	14%	$ 138,600	$ 851,400
1.8	Misc. SBR Improvements	Operations	$ 167,000	14%	$ 23,380	$ 143,620
Total Priority 1 Improvements (rounded)			**$ 16,325,000**		**$ 12,174,000**	**$ 4,152,000**
Priority 2 Improvements						
2.1	Mill Creek Force Main Extension	Capacity	$ 1,190,000	22%	$ 263,442	$ 926,558
2.2	Gardner Pump Station Displacement	Capacity, Operations	$ 781,000	14%	$ 111,053	$ 669,947
2.3	Pipeline Upsizing on Evergreen	Capacity	$ 1,406,000	10%	$ 142,438	$ 1,263,562
2.4	Pipeline Upsizing on Ida	Capacity	$ 1,480,000	4%	$ 64,149	$ 1,415,851
2.5	Influent Screen	Redundancy, Operations	$ 466,000	100%	$ 466,000	$ -
2.6	Dryer Replacement	Operations	$ 7,770,000	14%	$ 1,087,800	$ 6,682,200
2.7	Utility Water Storage	Operations	$ 1,160,000	14%	$ 162,400	$ 997,600
2.8	Generator	Operations	$ 1,050,000	14%	$ 147,000	$ 903,000
2.9	Sludge Storage Pond Repairs	Operations	$ 516,000	14%	$ 72,240	$ 443,760
Total Priority 2 Improvements (rounded)			**$ 15,820,000**		**$ 2,520,000**	**$ 13,310,000**
Priority 3 Improvements						
3.1	Long Term Pump Station Upgrades	Operations	$ 486,000	14%	$ 69,106	$ 416,894
Total Priority 3 Improvements (rounded)			**$ 490,000**		**$ 70,000**	**$ 420,000**
TOTAL WWTP AND COLLECTION SYSTEM IMPROVEMENTS COSTS (rounded)			**$ 32,635,000**			

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2020 dollars and does not include escalation to time of actual construction. Keller Associates has no control over variances in the cost of labor, materials, equipment, services provided by others, contractor's methods of determining prices, competitive bidding or market conditions, practices or bidding strategies. Keller Associates cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the cost presented herein.

12.3 OTHER ANNUAL COSTS

In addition to the capital improvement costs presented in Table 12-1, the following expected annual operating costs are recommended for consideration in setting annual budgets for the collection system:

- Additional collection system replacement/rehabilitation needs: Based on linear feet of pipeline, and number of manholes and cleanouts, the City should budget a total of $806,500/year for pipeline replacement/rehabilitation (to be either contracted out or completed using City crews).
- The City should target the infiltration and inflow (I/I) projects discussed in Section 8 as a part of the annual pipeline replacement/rehabilitation budget. Prioritizing these projects should help to reduce I/I flows into the system and potentially delay capital improvements triggered by increased system flows.
- Pump station annual operation and maintenance costs will go down as the City prepares to abandon Gardner pump station.

- Collection system cleaning and CCTV needs: City maintenance staff currently follow a five-year timeline to clean and CCTV inspect the entire system. No change is recommended to the current practice of cleaning and CCTV inspection.
- Annual O&M costs for the collection system may increase slightly due to the increase in linear feet of pipeline.

Priority improvements at the treatment plant should decrease the overall O&M for the City staff. Many of the projects were based on the condition and reliability of the existing equipment. As these issues are addressed by these improvements, the associated O&M should decrease. The following provides a brief description of the impacts of the treatment plant projects on O&M:

- Winter Equalization – equalizing the WWTP influent would require additional pumps and controls; however, the SBRs and other downstream components would benefit from more consistent flows, leading to fewer process upsets and more consistent operation. It would also provide the City staff with the flexibility to perform maintenance even during high flow events.
- Influent Pump Drives – the current pumps are either at full-capacity or off. A variable speed drive and control panel will help avoid issues with the SBR operating based on inconsistent influent flows.
- Post-SBR Equalization – increasing the size of the equalization downstream of the SBR would help the SBR maximize its flexibility and consistent operation. The equalization would allow the maximum decant volume to increase, improving the SBR’s ability to deal with changing conditions.
- Miscellaneous Spare Parts – the lead times for some parts can be long, which can have long-term effects on the WWTP. It is recommended that critical spare parts be on hand, especially for long lead time parts without installed redundancy.
- Turbo Blower Replacement – the existing turbo blowers are difficult to maintain. Rather than struggling when issues come up, the City’s overall O&M would decrease with new blowers.
- Influent Screen – having a second mechanical screen installed would reduce the amount of debris that enters the WWTP when the existing mechanical screen is down.
- Dryer Replacement – due to the high oil temperatures, the existing dryer is dangerous to maintain. Also, replacement parts are expensive, wear out rapidly, and have long lead times befor delivery. The City’s overall O&M would decrease with a new dryer.
- Utility Water Storage – the WWTP is one of the biggest water users in the City. The City would recoup the lost revenue if the WWTP had a larger utility water tank.
- Generator – the O&M for the City staff increases dramatically in the event of an extended loss of power. Having a backup generator that could power the entire WWTP would remove a significant amount of O&M.

Overall, the projected increase in influent flows and loadings will increase the total O&M of the system; however, the routine O&M required to keep the equipment in good working condition will be decreased significantly by these improvements.

12.4 SCHEDULE

An estimated schedule for the next six years is shown in Table 12-2. Again, the costs presented here are planning-level estimates using current (2020) dollar values. The actual cost for each project is further refined in the pre-design and design phases.

TABLE 12-2: PRIORITY 1 CIP SCHEDULE

ID#	Item	Cost (2020)	Opinion of Probable Costs					
			2021	2022	2023	2024	2025	2026
Priority 1 Improvements								
1.1	Pipeline Upsizing on Jetters and Ida	$ 2,943,000	$ 371,401	$ 2,722,432				
1.2	Short Term Pump Station Upgrades	$ 270,000		$ 53,524	$ 237,468			
1.3	Winter Equalization	$ 11,530,000		$ 1,216,017	$ 4,332,368	$ 7,093,112		
1.4	Influent Pump Control	$ 103,000			$ 111,558			
1.5	Post-SBR Equalization	$ 120,000		$ 126,559				
1.6	Miscellaneous Parts	$ 202,000	$ 103,723	$ 106,520				
1.7	Turbo Blower Replacement	$ 990,000						$ 1,161,357
1.8	Misc. SBR Improvements	$ 167,000					$ 190,762	
	Total (rounded)	**$ 16,325,000**	**$ 475,000**	**$ 4,225,000**	**$ 4,681,000**	**$ 7,093,000**	**$ 191,000**	**$ 1,161,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2020 dollars and includes a 2.7% annual escalation based on historic ENR data. Keller Associates has no control over variances in the cost of labor, materials, equipment, services provided by others, contractor's methods of determining prices, competitive bidding or market conditions, practices or bidding strategies. Keller Associates cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the cost presented herein.

12.5 OTHER FINANCIAL CONSIDERATIONS

The City of Stayton currently has two loans for the sewer utility, one from USDA and the other from US Bank. The USDA loan has a 3.25% interest rate and will mature in 2053. The annual required payment on this loan is $375,000. The US Bank loan has a 2-4% variable interest rate and will mature in 2028. The annual required payment is $422,025. Neither loan has a holding or reserve requirement.

It is recommended the City complete a full-rate study for the wastewater utility to evaluate the potential user rate and system development charge (SDC) impacts of the recommended CIP. Estimated SDC eligibility for each identified capital improvement is included in Table 12-1 for use in completing a full rate study. Keller Associates recommends the City actively pursue opportunities for grant funds, low-interest loans, or principal forgiveness funding sources to mitigate user rate impacts. As the City begins to prepare and proceed on CIP projects, it is recommended they setup a one-stop meeting with Business Oregon to identify and assess potential funding sources for the sewer projects.