3.3. REASONABLE GROWTH

Wastewater facility improvements are needed to stay ahead of growth due to the potential increased population and new construction. Chapter 1 discussed population growth projections, including customers served. The wastewater system must accommodate growth in the planning period. The new growth should proportionately fund necessary improvements using system development charges (SDCs).

Keller Associates conducted a Future Growth Meeting with the City of Willamina staff. This meeting aimed to identify and discuss areas of potential future growth for the 20-year planning period. The future growth discussed during the meeting is summarized in Table 1-3.

3.3.1. Influent Flows

The influent flow analysis looks at historic wastewater flows and provides flow projections for the planning period. This section summarizes the results of the flow analysis in accordance with DEQ's "Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon" for determining design flows in the City's system. Due to concerns with the integrity of flow data reported at the WWTP, flow data was obtained from the pump station run time data. WWTP influent flow was calculated by multiplying the individual pump run times by the measured pump flows measured in the field and summing the calculated flows for a given day.

- 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 from 2018-2022. The AADF for each of these years was then averaged to obtain the AADF for the planning criteria.

- Average Dry-Weather Flow (ADWF)

  The average dry-weather flow (ADWF) is the average daily flow from May through October. An ADWF was calculated for each year of data and then averaged.

- Average Wet-Weather Flow (AWWF)

  The AWWF was calculated as the average daily flow for the period encompassing January-April and November-December for each year of data. Five years' worth of data (2018-2022) was averaged to obtain the AWWF.

- Max Month Dry-Weather Flow ($MMDWF_{10}$)

  The maximum monthly dry-weather flow ($MMDWF_{10}$) represents the month with the highest flow during the summer. 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 (4.14 inches) was extrapolated from the National Oceanic and Atmospheric Administration (NOAA) Summary of Monthly Normals from 1981 to 2010 using a NOAA station in Willamina. Data from 2018–2022 was used to produce Figure 3-9. Table 2 summarizes the data points illustrated in the chart.

- Max Month Wet-Weather Flow ($MMWWF_5$)

  The maximum monthly wet-weather flow ($MMWWF_5$) represents the highest monthly average during the winter. 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 most likely corresponds to the maximum monthly flow during the wet-weather period for a 5-year event. The January 80% precipitation exceedance value (10.90 inches) was extrapolated from the NOAA Summary of Monthly Normals from 1981 to 2010 using a NOAA station in Willamina. The DEQ method and $MMWWF_5$ result are illustrated in Figure 3-9 and summarized in Table 3-3.

FIGURE 3-9: FLOW VS. RAINFALL ($MMDWF_{10}$ AND $MMWWF_5$)

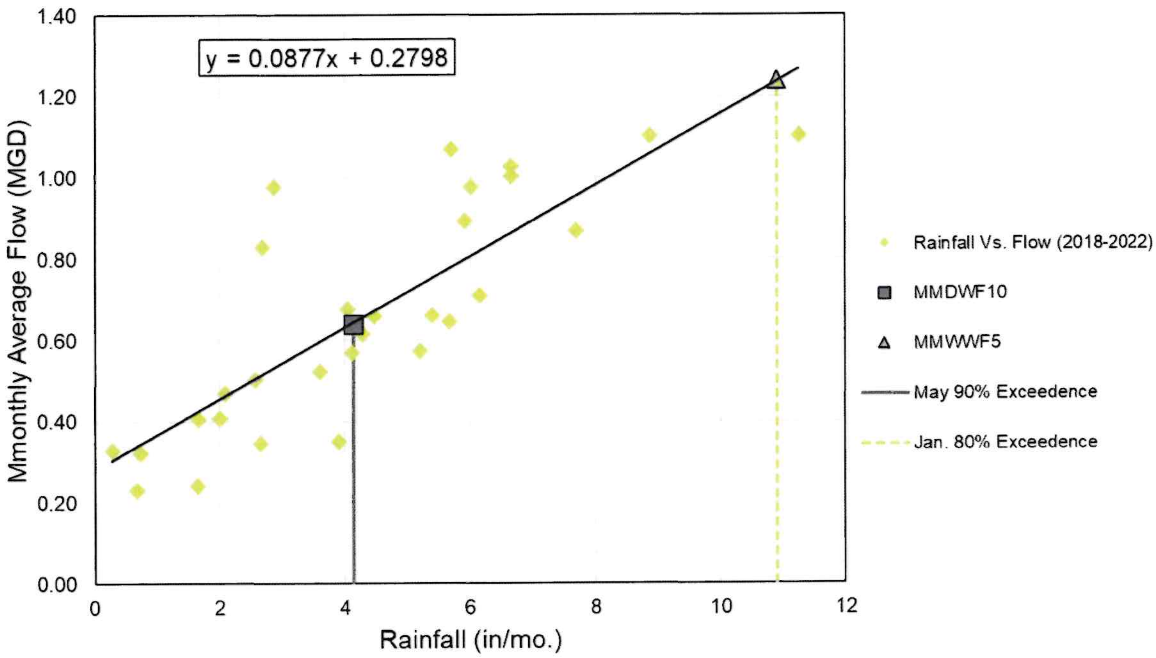


TABLE 3-3: MONTHLY AVERAGE FLOW VS. RAINFALL ($MMDWF_{10}$ AND $MMWWF_5$)

Month	Monthly Average Flow (MGD)					Rainfall (in/mo.)				
	2018	2019	2020	2021	2022	2018	2019	2020	2021	2022
January	0.87	0.66	1.10	1.10	0.89	7.7	5.4	11.3	8.9	5.9
February	0.52	0.83	0.98	0.98	0.50	3.6	2.7	2.9	6.0	2.6
March	0.57	0.41	0.35	0.47	0.66	5.2	2.0	2.7	2.1	4.5
April	0.61	0.65	0.41	0.32	No Data	4.3	5.7	1.7	0.7	6.2
May	0.33	0.24	0.35	0.23	No Data	0.3	1.6	3.9	0.7	4.1
$MMDWF_{10}$	0.64 MGD					4.14 in/mo.				
$MMWWF_5$	1.24 MGD					10.90 in/mo.				

A 30-day rolling average of the available dry weather flow data (May 1, 2018 through October 31, 2022) was reviewed to confirm the validity of the DEQ method. The maximum observed dry weather 30-day rolling average flow was 0.45 MGD (May 25, 2018 through June 23, 2018). The precipitation during these 30 days was 0.81 inches. Since this is lower than the value predicted by DEQ's method, 0.64 MGD was selected as the $MMDWF_{10}$. Similarly, a 30-day rolling average of the available wet weather flow data was evaluated to compare with the $MMWWF_{10}$ calculated by the DEQ method. The maximum observed wet weather 30-day rolling average flow was 1.41 MGD (December 18, 2020 through January 16, 2021). The precipitation during this 30-day rolling average was 13.15 inches. The observed event of 1.41 MGD was selected as the $MMWWF_{10}$.

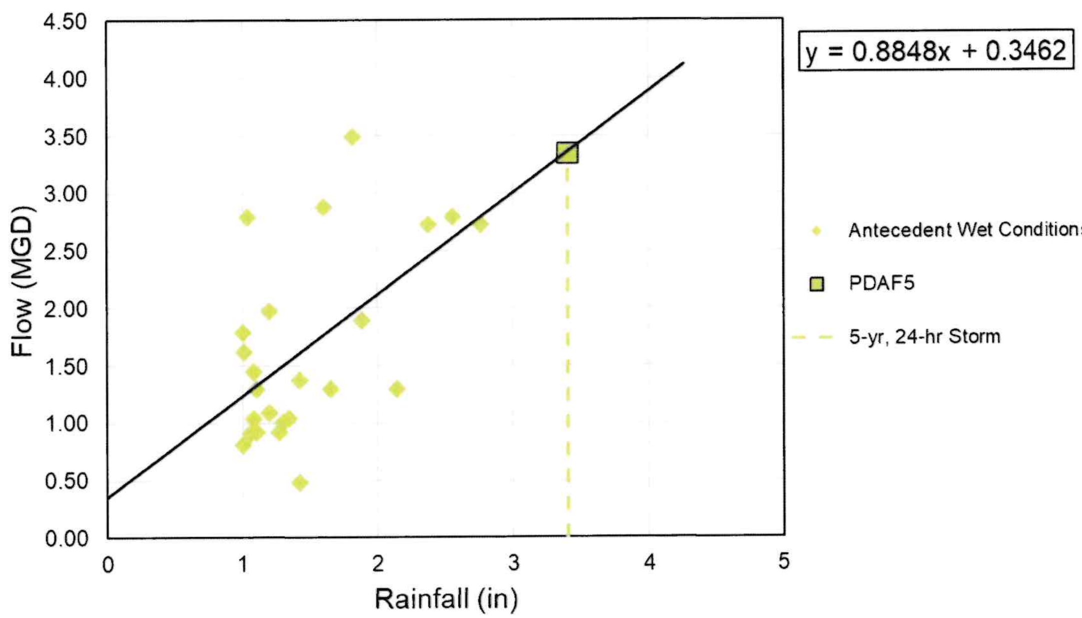
- Peak Week Flow (PWkF)

The PWkF was calculated using a 7-day rolling average for each year.

- Peak Daily Average Flow (PDAF5)

As outlined by the DEQ, the peak daily average flow ($PDAF_5$) corresponds to a 5-year storm event. 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 (3.4 inches per the NOAA isopluvial maps for Oregon) from a trend line fitted to the data. A significant storm event was considered more than 0.5 inches 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 1.5 inches. Figure 3-10 below shows the results of the analysis.

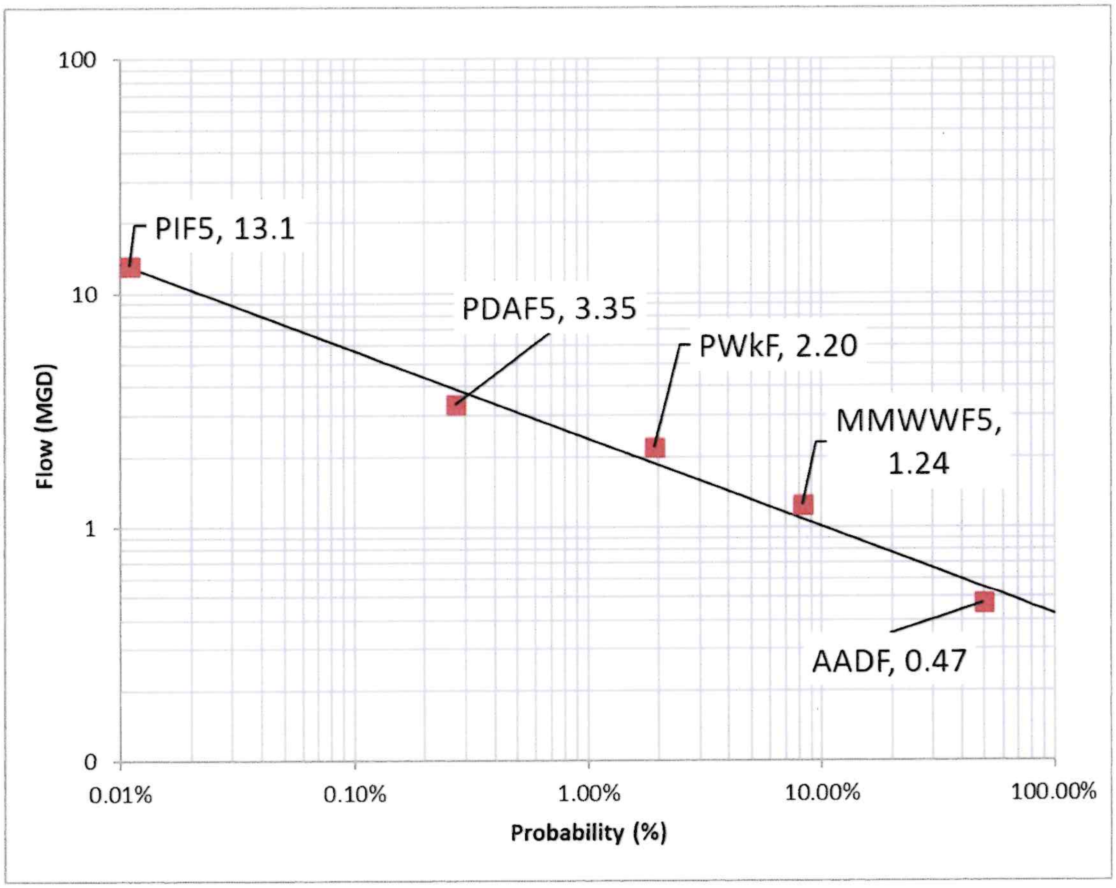
FIGURE 3-10: FLOW VS. RAINFALL ($PDAF_5$)



- Peak Instantaneous Flow (PIF5)

The peak instantaneous flow (PIF_5) represents the peak flow recorded at the WWTP. If available, the DEQ recommends evaluating hourly or instantaneous flow data for high-flow days. The City does not record instantaneous flow data. As an alternative, DEQ recommends estimating PIF_5 by extrapolation. A probability graph was produced, where the PIF_5 was extrapolated from a known $PDAF_5$. Figure 3-11 (next page) shows the results.

FIGURE 3-11: FLOW VS. PROBABILITY (PIF_5)



The PIF_5 was found to be 13.1 MGD using the DEQ extrapolation method. This PIF_5 appears unreasonable compared to the population served, operator knowledge of the system, and other Oregon cities with similar systems. Table 3 presents the PIF_5 to $PDAF_5$ peaking factor of several reference cities in Oregon. Using DEQ's extrapolation method, the PIF_5 to $PDAF_5$ peaking factor would be 3.91.

TABLE 3-4: $PIF_5/PDAF_5$ PEAKING FACTORS

City	$PIF_5/PDAF_5$
Newberg, Oregon	1.30
Amity, Oregon	1.54
Sheridan, Oregon	1.46
Average	**1.43**

Applying a peaking factor of 1.43 to the Willamina $PDAF_5$ yields a PIF_5 of 4.79 MGD for 2022.

➢ Observed Historical Flows

Table 3-5 summarizes the observed flows for each year from 2018-2022. The historical flows were derived as described in the preceding paragraphs. The total rainfall in inches per year (in/yr) and the total flow in million gallons per year (MGY) are summarized.

TABLE 3-5: OBSERVED HISTORICAL FLOWS

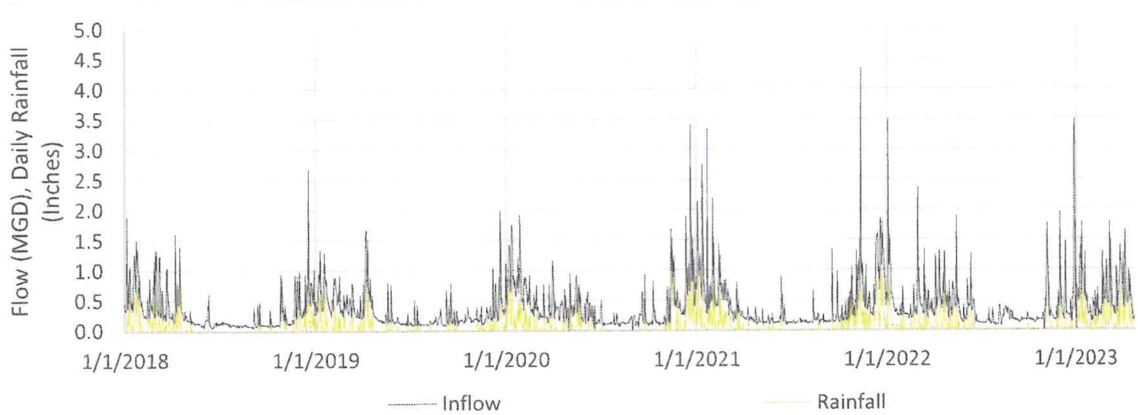
Historical Flows (MGD)						Planning Flow (MGD)
Year	2018	2019	2020	2021	2022	---
Population	2,160	2,250	2,270	2,276	2,282	2,282
ADWF	0.21	0.20	0.22	0.24	0.22	0.22
$MMDWF_{10}$	0.58	0.30	0.53	0.31	0.64	0.64
AADF	0.39	0.36	0.43	0.54	0.47	0.47
AWWF	0.57	0.53	0.64	1.37	0.68	0.76
$MMWWF_5$	1.04	1.04	1.17	1.41	1.24	1.24
PWkF	1.25	1.46	1.6	2.03	2.20	2.20
$PDAF_5$	1.5	2.48	2.68	2.88	3.35	3.35
PIF_5	2.16	3.55	3.84	4.13	4.79	4.79
Total Rainfall (in/yr)	37	33	47	46	47	---
Total Flow (MGY)	142	133	157	197	179	---

3.3.2. Observed Historical Inflow and Infiltration (I/I)

Inflow refers to stormwater that enters the sewer system through several sources, including the holes in manhole lids and cross-connections. Infiltration refers to groundwater that enters the wastewater collection system through leaks in pipes and manholes. Excessive I/I can contribute to overwhelming the collection, conveyance, and treatment systems.

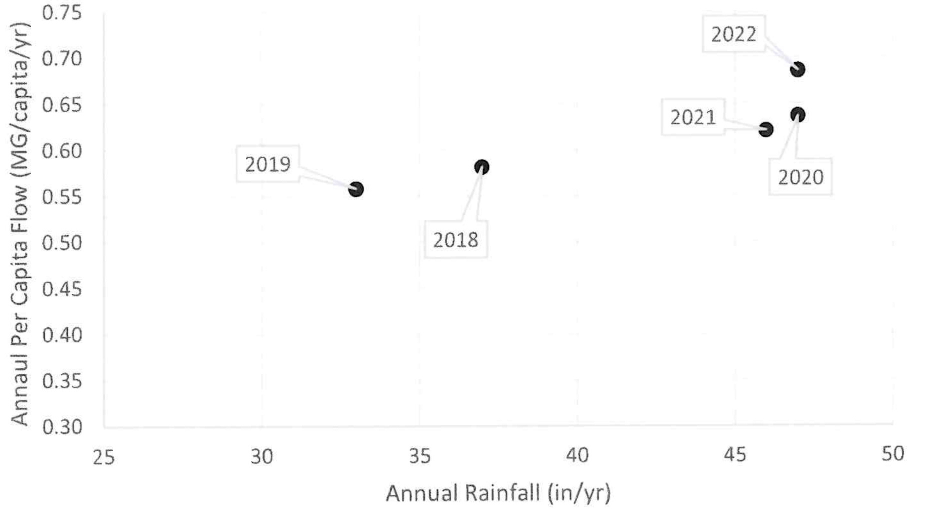
Evidence of I/I can be seen when plotting flow and rainfall on the same graph. Rapid increases in flow following precipitation events suggest a high influence from inflow. A slower, sustained increase in flow following precipitation events would suggest a high amount of infiltration. Rapid increases in flow and large tails that slowly taper down suggest strong influence from both inflow and infiltration. FIGURE 3-12 illustrates the observed I/I data for the study years provided.

FIGURE 3-12: DAILY FLOW AND PRECIPITATION



Evidence of I/I can be further demonstrated by comparing annual rainfall against annual per capita flow. Figure 3-13 shows a positive linear relationship between rainfall and average annual flow.

FIGURE 3-13: ANNUAL RAINFALL VS PER CAPITA FLOW



Several peaking factors were calculated to compare wet-weather flows against average flows from 2018 through 2022. Table 3-6 summarizes these peaking factors and compares them to other nearby communities. Stayton and Newberg both have documented challenges with high I/I influence. Aurora is a newer system, and for a gravity collection system in western Oregon, it has low I/I

TABLE 3-6: WET-WEATHER PEAKING FACTORS COMPARISON

Peaking Factors	Willamina, Oregon						Stayton, OR	Newberg, OR	Aurora, OR	Amity, OR
	2018	2019	2020	2021	2022	Planning Criteria	Planning Flows	Planning Flows	Planning Flows	Planning Flows
PIF_5/ADWF	10.29	17.75	17.45	17.21	21.77	**21.77**	7.8	12.3	3.1	16.8
$PDAF_5$/ADWF	7.14	12.4	12.18	12	15.23	**15.23**	6.7	9.5	2.4	10.9
$MMWWF_{10}$/ADWF	4.95	5.2	5.32	5.88	5.64	**5.64**	1.8	2.0	1.0	1.4

- I/I Mitigation Techniques

The City may use various techniques to identify sources of I/I and rehabilitate pipes to decrease I/I. Chapter 6 includes a discussion of I/I mitigation techniques. A pump run time analysis and CCTV system inspection can also be beneficial. The City conducted a nighttime I/I analysis to identify potential locations for sources of I/I. The results from this analysis are shown in Appendix F.

Table 3-7 summarizes the annual average base flow and the ratio of peak flow to the base flow for the 2018-2022 data sets. The peak flow compared to the base flow indicates the I/I influence in the system. In 2018-2022, the peak flow ranged from approximately 1.44 to 2.70 times the base flow. I/I exists in the system but is not excessive. Some communities experience peak flows more than ten times the base flow.

TABLE 3-7: ANNUAL PEAK DAY FLOW/AVERAGE BASE FLOW

Year	Average Base Flow (MGD)	Peak Flow (MGD)	Peak Flow/Average Base Flow
2018	1.04	1.5	1.44
2019	1.04	2.48	2.38
2020	1.17	2.68	2.29
2021	1.41	2.88	2.04
2022	1.24	3.35	2.70

In addition, future new construction should reduce I/I due to newer, more watertight sewer components.

3.3.3. Future Flow Projections

A projected flow per capita (reported in gallons per capita per capita per day, gpcd) was developed to project the planning flows. Table 3-8 summarizes the projected planning flows. Actual future flows depend on several factors and could decrease through aggressive I/I reduction efforts. Flow should be measured and reviewed periodically, and future capital projects should be phased where practical. The 2045 projected flows will be used for treatment system design.

TABLE 3-8: PROJECTED PLANNING FLOWS

	Planning Flow (MGD)	Planning Unit Flow (gpcd)	Projected Design Flow (MGD)					
Year	---	---	2025	2030	2035	2040	2045	2065
Population	2,282	2,282	2,182	2,314	2,459	2,604	2,749	3,384
ADWF	0.22	96	0.21	0.22	0.23	0.25	0.26	0.32
$MMDWF_{10}$	0.64	280	0.61	0.65	0.69	0.73	0.77	0.95
AADF	0.47	205	0.45	0.47	0.50	0.53	0.56	0.69
AWWF	0.76	332	0.72	0.77	0.82	0.87	0.91	1.12
$MMWWF_5$	1.24	542	1.18	1.25	1.33	1.41	1.49	1.83
PWkF	2.20	964	2.10	2.23	2.37	2.51	2.65	3.26
$PDAF_5$	3.35	1,470	3.21	3.40	3.61	3.83	4.04	4.97
PIF_5	4.79	2,100	4.58	4.86	5.16	5.47	5.77	7.11

3.3.4. Future Load Projections

The wastewater influent loading analysis follows a methodology similar to that of 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 influent BOD_5 and TSS load analysis. The City is not required to record effluent nitrogen levels. However, the City sampled Total Kjeldahl Nitrogen (TKN) in May 2023. The result of this sampling is discussed below. 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 2018-2022 were averaged to obtain the ADLs.

- Maximum Month Load (MML)

The maximum month load (MML) was calculated for both dry weather (DWMML) and wet weather (WWMML) for each year of data. The maximum month data is from the discharge monitoring reports (DMRs) from 2018 to 2022 and represents the samples taken during the month rather than a 30-day rolling average.

- BOD_5, TSS, and TKN Loading

The BOD_5 and TSS loadings are summarized in Table 3-9 and Table 3-10, respectively.

TABLE 3-9: OBSERVED HISTORICAL BOD_5 LOADING

Year	2018	2019	2020	2021	2022	Avg.	Max.	Planning Criteria
Population	2,160	2,250	2,270	2,276	2,282	---	---	2,282
BOD_5 (ppd)								
DWADL	523	806	655	727	536	649	806	---
DWMML	885	1,010	1,220	1,000	790	981	1,220	---
WWADL	397	657	787	782	668	658	787	---
WWMML	468	1,088	1,166	1,150	968	968	1,166	---
BOD_5 (ppcd)								
DWADL	0.242	0.358	0.289	0.319	0.235	0.289	0.358	0.358
DWMML	0.410	0.449	0.538	0.440	0.346	0.436	0.538	0.538
WWADL	0.184	0.292	0.347	0.344	0.293	0.292	0.347	0.347
WWMML	0.216	0.483	0.514	0.505	0.424	0.429	0.514	0.514

TABLE 3-10: OBSERVED HISTORICAL TSS LOADING

Year	2018	2019	2020	2021	2022	Avg.	Max.	Planning Criteria
Population	2,160	2,250	2,270	2,276	2,282	---	---	2,282
TSS (ppd)								
DWADL	309	418	295	305	302	326	418	---
DWMML	741	479	393	406	480	500	741	---
WWADL	316	457	392	520	417	420	520	---
WWMML	532	789	634	873	532	672	873	---
TSS (ppcd)								
DWADL	0.143	0.186	0.130	0.134	0.132	0.145	0.186	0.186
DWMML	0.343	0.213	0.173	0.178	0.210	0.224	0.343	0.343
WWADL	0.146	0.203	0.173	0.229	0.183	0.187	0.229	0.229
WWMML	0.246	0.351	0.279	0.383	0.233	0.299	0.383	0.383

The City sampled TKN, TSS, and BOD_5 in May 2023, and the results are provided in Table 3-11 below.

TABLE 3-11: MAY 2023 SAMPLING RESULTS

Parameter	mg/L	ppd	ppcd
BOD_5	183	405	0.178
TSS	156	345	0.151
TKN	34.2	76	0.033

The sampling results revealed a BOD_5:TKN ratio of approximately 5.35. Future TKN loadings to the plant were projected based on the future projected BOD_5 loadings and a BOD_5 to TKN ratio of 5.35.

Unit loadings in pounds per capita per day (ppcd) were calculated for each year of data analyzed. Projected unit loadings are the maximum of the individual 2018-to-2022-unit loads. The projected loads in pounds per day are the product of the projected unit load (ppcd) and the population. The projected BOD_5, TSS, and TKN loads are summarized in Table 3-12.

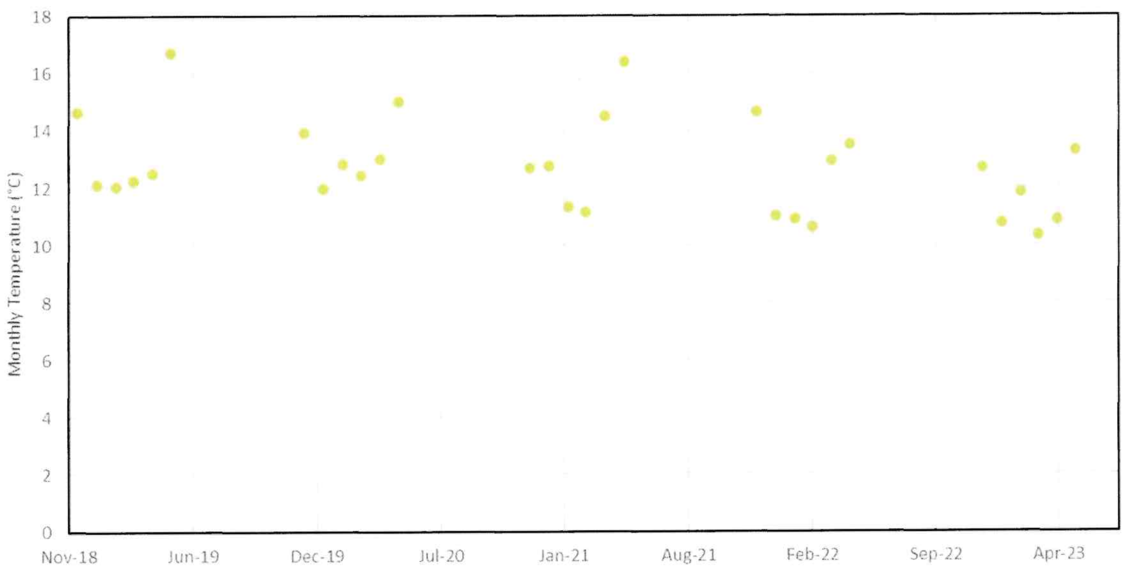
TABLE 3-12: PROJECTED BOD_5, TSS, AND TKN LOADS

Parameter	Year	2025	2030	2035	2040	2045
	Population	2,182	2,314	2,459	2,604	2,749
	BOD (ppcd)	BOD (ppd)				
WWADL	0.35	756	802	852	903	953
WWMML	0.51	1,121	1,189	1,264	1,338	1,413
DWADL	0.36	782	829	881	933	985
DWMML	0.54	1,173	1,244	1,322	1,400	1,478
Parameter	TSS (ppcd)	TSS (ppd)				
WWADL	0.23	500	530	563	596	630
WWMML	0.38	836	886	942	997	1,053
DWADL	0.19	406	430	457	484	511
DWMML	0.34	748	794	843	893	943
Parameter	TKN (ppcd)	TKN (ppd)				
WWADL	0.06	141	150	159	169	178
WWMML	0.10	209	222	236	250	264
DWADL	0.07	146	155	165	174	184
DWMML	0.10	219	232	247	261	276

- Temperature

The City has also collected effluent temperature readings during the discharge period. The monthly average effluent temperatures are shown in Figure 3-14. The minimum monthly temperature was approximately 10°C. The maximum monthly temperature was approximately 17°C.

FIGURE 3-14: WWTP EFFLUENT TEMPERATURES



3.3.5. Capacity Limitations

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 potentially discharge (1) into the public water supply, shellfish, or primary contact recreation waters, or (2) as a result of its volume or character, could permanently or unacceptably damage or affect the receiving waters or public health if normal operations were interrupted. For example, they discharge near drinking water intakes or into shellfish waters.

Reliability Class II: Works that discharge, or potential discharge, as a result of its volume 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). For example, it is discharging into recreational waters.

Reliability Class III: Works not otherwise classified as Class I or Class II.

- Pump Station

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 require the pump station's firm capacity to convey the larger of the 10-year dry-weather or 5-year wet-weather event. For Willamina, 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. Mobile generators or portable trash pumps may be acceptable for pump stations, depending on the risk of overflow, available storage in the wet well and pipelines, alarms, and response time.

- Pipeline

CMOM refers to the Capacity Management, Operation, and Maintenance of the entire wastewater conveyance system. The vast majority of 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 installing and regularly maintaining 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 to provide 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 were defined as average daily dry-weather flows below 120 gpcd. Similarly, a below 275 gpcd average wet-weather flow guideline was established for non-excessive stormwater inflow.

- Pipeline Surcharging

Pipeline surcharging occurs as flows exceed the capacity of a full pipe, causing wastewater to back up into manholes and service laterals. 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 stormwater system should be removed. 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 3-13. 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 3-13: UNIT PROCESS RELIABILITY EVALUATION

Equipment	Backup Rating	Criticality Rating	Equipment Condition Rating
Influent Screen	5	S/H, EQ, PF, CC	R
Aerated Lagoon	4	S/H, EQ, PF, CC	M
Aerated Lagoon Aeration	1	S/H, EQ, PF, CC	R
Effluent Storage Lagoon	4	S/H, EQ, PF	M
Chlorine Feed Pump	1	S/H, EQ, PF	LN
Declorination Feed Pump	1	S/H, EQ, PF	M
Chlorine Contact Basin	5	EQ, PF	M
River Pump	1	EQ, PF	M

Backup rating	
1	**One level of "in kind" redundancy** (identical peice of equipment is available to replace
2	**Two of more levels of "in kind" redundancy** (More than one piece of equipment is
3	**Equipment alternative** (An alternative piece of equipment is provided)
4	**Procedural alternative** (An alternative operating procedure is required to provide
5	**No Backup** (Failure of equipment will shut entre process down)

Criticality Rating	
S/H	**Safty and Health Risk** (Loss would create risk to saftey and health of plant personnel
EQ	**Effluent Quality Risk** (Loss would create risk to WWTP effluent quality and could result
PF	**Process Functionality Risk** (Loss would affect the function and/or efficiency of the affected process
CC	**Cost Critical** (Loss would have significant cost imact in short term or long term)

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 to a condition like new)
M	**Used but Maintainted** (Equipment showing expected wear, but is adequately maintained and functions well)
W	**Heavily Worn** (Equipment close to end of useful life, needs overhaul, difficulty in preforming intended functions)
R	**Needs Replacement** (Equipment does not acceptably preform, beyond cost-effective repair)

TABLE 3-14: EPA REQUIREMENTS FOR RELIABILITY

Component	Reliability Class I	Reliability Class II
Raw sewage pumps, lift stations	Peak flow with the largest unit out of service. Peak flow is the maximum wastewater flow expected during the design period.	
Mechanical Bar Screens	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.	
Grit Removal	Overflow shall be sufficient to pass peak flow with all grit units out of service.	
Primary sedimentation	50% of design flow capacity with the largest unit out of service. Design flow is the flow used as the design basis of the component.	
Activated sludge process	A minimum of two equal-volume basins shall be provided. No backup basin is required.	
Aeration blowers	Supply the design air capacity with the largest unit out of service shall be provided. A minimum of two units.	
Air diffusers	With the largest section of diffusers isolated or out of service, oxygen transfer capacity shall not be measurably impaired	
Secondary sedimentation	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.	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.
Filters/advanced treatment	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.	No backup is required.
Disinfection basins	50% of design flow capacity with the largest unit out of service. Design flow is the flow used as the design basis of the component.	
Effluent pumps	Peak flow with the largest unit out of service. Peak flow is the maximum wastewater flow expected during the design period.	
Electrical power	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. The designated backup source shall have sufficient capacity to operate all vital components, critical lighting, and ventilation during peak flow conditions.	
	The provisions of backup power capacity for secondary treatment, final clarification, and advanced treatment are required. The provisions of capacity for degritting and sludge handling and treatment are optional.	The provisions of backup power capacity for secondary treatment, final clarification, and advanced treatment are optional. The provision of capacity for degritting and sludge handling and treatment is not required.
Sludge holding tanks	The volume of the holding tank shall be based on the expected time necessary to perform maintenance and repair of the component in question.	
Anaerobic digestion	At least two digestion tanks shall be provided. Backup sludge mixing equipment shall be provided, or the system shall be flexible enough to not lose total mixing capacity with one piece of equipment out of service. Backup equipment may be uninstalled.	
Aerobic digestion	A backup basin is not required. At least two blowers or mechanical aerators shall be provided. Isolation of largest section diffusers without measurably impairing oxygen transfer is allowed.	
Sludge pumping	Pumps sized to pump peak sludge quantity with one pump out of service. The backup pump may be uninstalled.	

3.3.6. Collection System Evaluation

This section summarizes the construction of the wastewater collection system model with the calibration process and documents identified deficiencies. Alternatives to address these deficiencies are discussed in Chapter 4.

- Model Construction

An accurate computer model of the wastewater system is an important planning tool and provides the basis for identifying existing and future collection system deficiencies. No previous hydraulic model was available; therefore, the model inputs, including elevations, pipe diameters, alignment, connectivity, pump station characteristics, etc., were input based on information provided by the City. InfoSWMM Suite 14.7 Update #2 was selected as the modeling software for this project. InfoSWMM is a fully dynamic model that operates in conjunction with Esri ArcGIS and allows for evaluating complex hydraulic flow patterns.

A survey of approximately 100 manholes was completed as a part of this master plan to check the ground surface and pipe invert elevations throughout the collection system. The survey data was used in conjunction with record drawings of the collection system to assign elevations for the pipes and manholes in the model. The two lift stations in the collection system were included in the hydraulic 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 and record drawings. Field pump tests were completed for both the lift stations as described in Section 2.3. The lift stations were modeled with design points equal to the flows observed in the pump tests.

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 modeled capacities discussed in this chapter represent the capacities assuming the wastewater collection lines are in good working order.

- Flow Monitoring

Flow monitoring for this study was conducted for several weeks in the winter, from January 12 to March 22, 2022. Temporary flow monitoring sensors were installed at four locations throughout the collection system, as shown in Figure 3-15. The sensors measure the depth of water in the pipe, and the velocity of the wastewater flows to calculate the flow through the pipe. The flow monitors were programmed to record data points every 15 minutes to capture fluctuations in system flows. While flow monitoring is an effective way of measuring sewer flows, there is a potential error in the data. Malfunctioning equipment and debris accumulation on the sensor are the most common errors encountered during the flow monitoring process. The flow data was checked a minimum of once per week to identify poor data quality and address issues if possible.

Scattergraphs were developed by plotting velocity on the x-axis and depth on the y-axis to evaluate the data quality and identify potential issues. The theoretical Manning Equation was also plotted on the scattergraph to indicate if the observed data matches the theoretical values. Tightly clustered data points along the Manning curve indicate good quality data. Where the Manning Equation did not match well, an adjusted version of the Manning Equation called the Stevens-Schutzbach Method was used. This method applies to pipes with downstream obstructions, offset joints, debris, or other related conditions that cause standing water in the pipe even if there is no flow. This method was used when the scattergraph showed no velocity but a depth greater than zero. A brief description of the flow monitoring results is provided below:

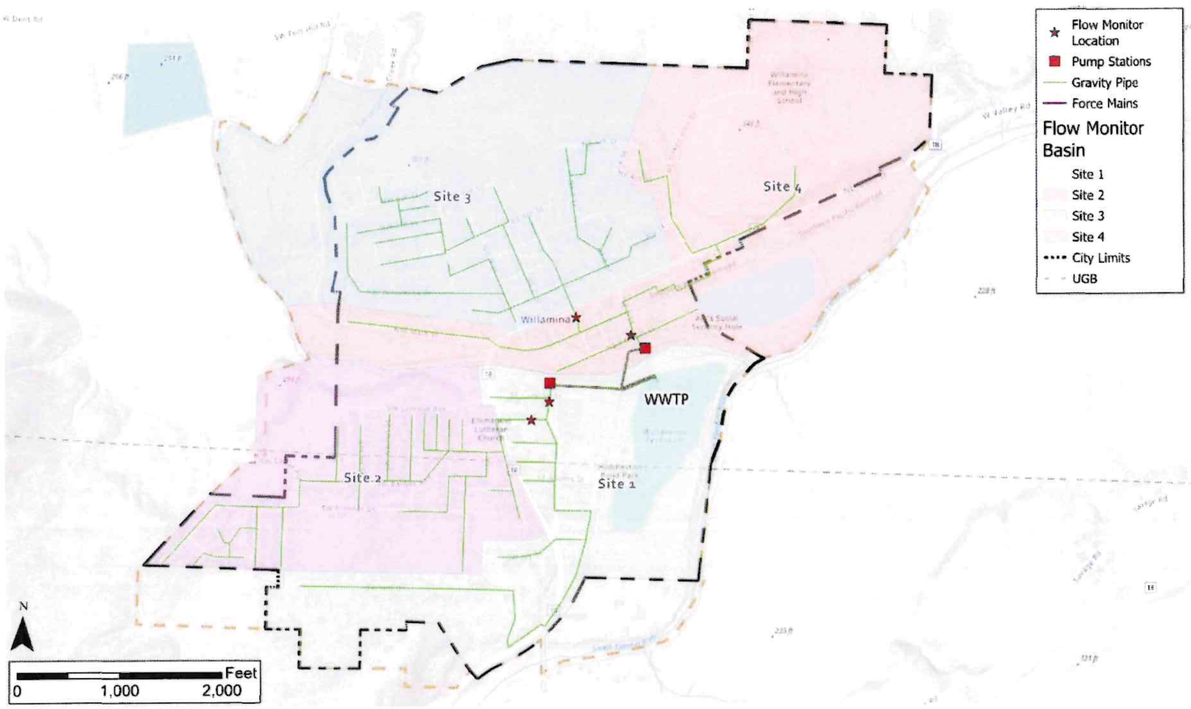
- Site 1 was installed upstream of the Washington Pump Station. The data quality at this site was particularly good with tightly clustered data points in the scattergraph. The average flows were around 220 gpm, showing a clear diurnal pattern, which is relatively consistent each day. There were no significant concerns with the data gathered at this site.

- Site 2 was installed upstream of Site 1. The data quality at this site was poor for the first couple of weeks; therefore, the sensor was replaced and recalibrated during one of the site visits. After recalibrating the sensor, the data quality was good, with average flows of around 30 gpm. Once the equipment was replaced, there were no major concerns with the data gathered at this site.

- Site 3 was installed to capture flows in the E Street Pump Station Basin and was installed upstream of Site 4. The data quality at this site was reasonably good, with tightly clustered data points in the scattergraph and only some outliers. The average daily flows were around 50 gpm. The diurnal pattern was clear, and there were no major concerns with the data gathered from this site.

- Site 4 was installed just upstream of the E Street Pump Station. The scattergraph indicates very good data quality with an average daily flow of around 180 gpm. There were no major concerns with the data gathered at this site.

FIGURE 3-15: FLOW MONITORING LOCATIONS



- Model Calibration

  Model loads refer to the wastewater flows that enter the wastewater collection system and comprise wastewater collected from individual services (base flows), groundwater infiltration (GWI), and stormwater I/I. The four monitoring sites divided the system into four areas. The collected data was analyzed along with continuous precipitation data to establish typical diurnal patterns, average base flows, GWI, and gauge rainfall influence at each site. 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.

  The model calibration results are summarized in Table 3-15, and details are provided in Appendix D. The green values represent flows calibrating within +10% or -5%. The red values indicate flows outside of this range. In summary, the model was calibrated well, and most peak flows were matched within the targeted ranges.

Calibrating the hydraulic model is critical to building confidence in the output model results. The calibration process aims to adjust the model inputs to match the observed data points from the flow monitoring. Two flow monitor periods were selected for the calibration scenario. First is the DWF calibration scenario, which used data from March 10, 2022. The DWF scenario is to establish the base flows and adjust the diurnal patterns from each flow monitor basin. This date was selected because there was minimal rainfall before and after, which resulted in less I/I influence. The second calibration scenario is the WWF scenario, which utilized data from February 28 through March 4. There was approximately three inches of cumulative rainfall during this period. This represents a wet weather flow event, and model inputs can be adjusted to match the I/I observed during this time. A rain gauge was installed in Willamina during the flow monitoring period, and the rainfall distribution was also input into the model.

Diurnal curves were developed for the four flow monitor sites based on the observed flows. These diurnal curves were assigned to the junctions in their respective basins to simulate the flow changes throughout the day. The base flows were adjusted globally up or down from the initial flow allocation based on the water meters, so the average flows matched the system flows observed on the calibration day. The model was then exercised, and the output results were compared to the observed data from the monitoring period. The base flows, diurnal patterns, and other model inputs were adjusted with each model run until the outputs matched the observed data. The model peak flows were targeted to be within -5% to +10% of the observed flows to be considered calibrated. It should be noted that matching the peak flows was given more weight than the daily volume because the peak flows are the primary evaluation criteria to determine if there are capacity deficiencies.

TABLE 3-15: MODEL CALIBRATION RESULTS

Location[1]	Observed Daily Flow Volume (gal)	Modeled Daily Flow Volume (gal)	Percent Difference	Observed Peak Flow (gpm)	Modeled Peak Flow (gpm)	Percent Difference
Site 1 DWF	348,724	354,283	1.6%	284	277	-2.5%
Site 2 DWF[2]	30,215	30,227	0.0%	47	46	-1.2%
Site 3 DWF	55,347	57,666	4.2%	55	60	8.7%
Site 4 DWF	213,105	230,490	8.2%	203	204	0.5%
Site 1 WWF #1	549,026	417,443	-24.0%	945	1030	9.0%
Site 2 WWF #1	252,147	227,169	-9.9%	335	366	9.4%
Site 3 WWF #1	115	118	3.0%	476	521	9.5%
Site 4 WWF #1	358	299	-16.5%	780	839	7.6%

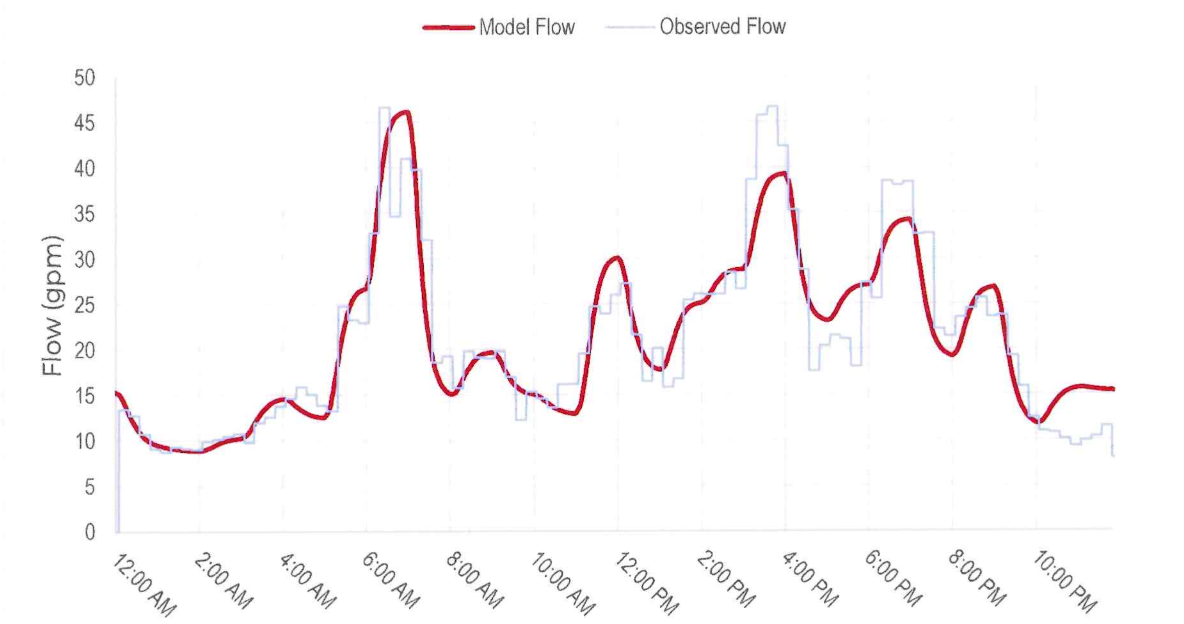
1) Calibration scenarios consisted of one dry weather flow period (DWF) and two wet weather rainfall events. Emphasis was given to the wet weather flow (WWF) event #1 because it was a larger rainfall event. The second WWF event was used as a check.

2) Site 2 DWF flow event had very volatile readings that were not representative of actual flows. A rolling 1-hour average was used to calibrate to this event at this site.

➢ Base Flow Calibration

Potable water consumption data was used to establish a base flow for the existing sewer system. The total residential consumption was divided by the number of residential connections to establish the residential base flows. These residential base flows were then allocated based on the location of residential parcels. The non-residential areas were allocated based on where commercial zone areas were located and the percentage of total consumption that was non-residential. Additionally, the top ten largest water users were manually assigned to the nearest junction. The original base flows allocated in the model resulted in much lower flows than observed during the flow monitor DWF event. This is likely because flow monitoring was conducted during the wet season, and although there was no rainfall during this period, infiltration was likely occurring. The DWF scenario was exercised in the model, and the base flows were adjusted up or down to match the flow monitoring data. Figure 3-16 illustrates an example calibration from Site 2.

FIGURE 3-16: SAMPLE BASE FLOW CALIBRATION SITE 2

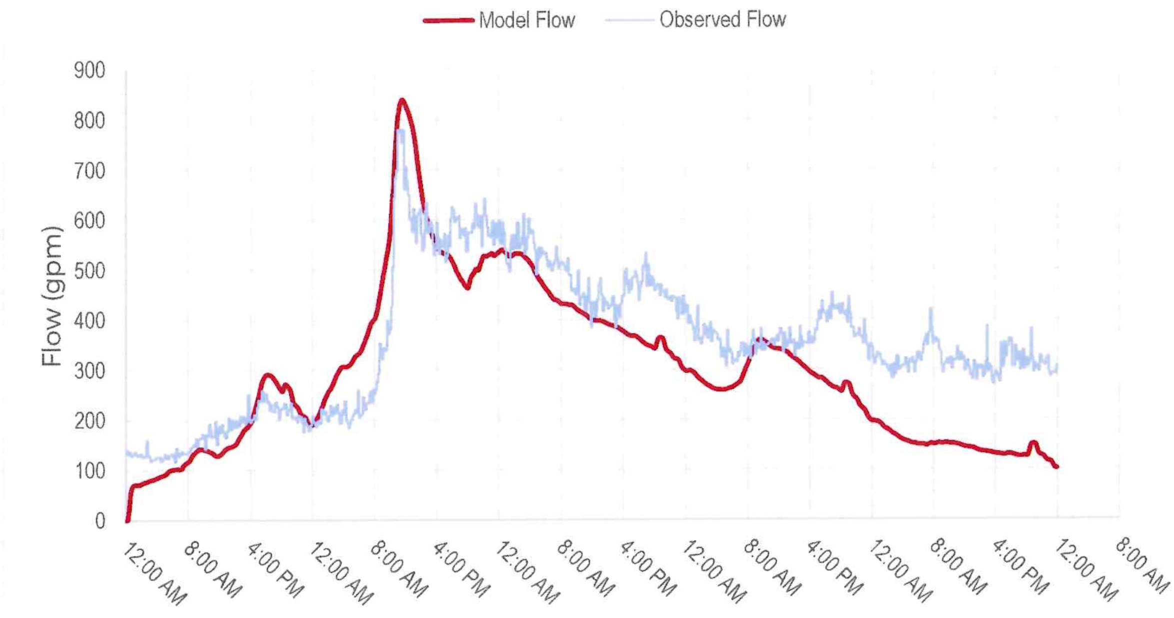


- Wet Weather Flow (WWF) Calibration

The RTK method was used to predict rainfall-derived infiltration and inflow (RDII). Rainfall data with the highest cumulative rainfalls during the period of flow monitoring was utilized to calibrate wet weather flows (February 28 through March 4th with 3.0 inches). The storm event rainfall was entered into InfoSWMM, and RTK parameters were then adjusted to calibrate the model with flow monitoring data. Again, total modeled flows at the WWTP were compared to the targeted average daily flow and WWTP influent flow data, in addition to calibrating the model at various locations within the collection system. An example of wet weather flow calibration results is shown below in Figure 3-17. RTK values were adjusted to calibrate the model to meet the highest peaks observed.

In general, the modeled peaks could be matched within 10% of the observed peaks; however, some total volumes were less in the model than were observed. This indicates the presence of infiltration, which rises and falls with rainfall events. While the total volume is less in the model, the critical measure is to match the peak flows.

FIGURE 3-17: SAMPLE WET WEATHER CALIBRATION SITE 3, JAN 2ND - 4TH



- Design Storm

The design storm used for model evaluation was the 5-year, 24-hour storm event. A standard 24-hour Natural Resources Conservation Service rainfall distribution for a Type 1A storm was used. The rainfall for the 5-year, 24-hour storm event from National Oceanic and Atmospheric Administration isopluvial maps is 3.4 inches. This was used as the multiplier for the Type 1A storm hyetograph. The calibrated model was run with the design storm event.

The modeled PIF_5 and $PDAF_5$ at the WWTP were compared to the modified PIF_5 and $PDAF_5$ planning criteria in Table 3-16. The modeled PIF and $PDAF_5$ at the plant were lower than the planning criteria when run with the existing pipe diameter. The lower flows result from constrained flow due to undersized pipes that cause surcharging and flooding within the system. The model was also run in an unconstrained condition where the pipe sizes were increased to eliminate surcharging and bottlenecks. Under this scenario, the flows were about 7% lower for the peak day volume but only 1% lower for the peak instantaneous flow. Additional discussion and details of existing system capacity limitations are summarized in the following section.

TABLE 3-16: PLANNING CRITERIA VS. MODELED PEAK FLOWS

Flow Criteria	Planning	Constrained Model Outflow	Unconstrained Model Outflow	Difference
Peak Day Volume (MG)	3.35	2.6	3.12	-6.8%
PIF (gpm)	3,326	2,407	3,289	-1.1%

- Existing System Evaluation

The calibrated model was used to assess the existing system capacity during a 5-year, 24-hour design storm event.

Pump Station	Reported Firm Capacity (gpm)	Field Measured Firm Capacity (gpm)[1]	Constrained PIF (gpm)	Uncsontrained PIF (gpm)	Sufficient Capacity (yes/no)[2]
E Street PS	700	500	1,250	1,400	**No**
Washington St PS	770	820	1,155	1,890	**No**

1) Field measured firm capacity based on average measurement from the two pumps.

2) Sufficient capacity based on field measured firm capacity comapred with unconstrained PIF

Figure 3-18 below and Figure 12a in Appendix A illustrate the potential overflow sites and pipe capacity limitations identified during the existing system peak instantaneous flow model evaluation. The figure shows the maximum depth divided by the system's full depth (d/D) during a $PDAF_5$ event. The d/D is a ratio of how full the pipe is during the highest flow period. For example, an 8-inch pipe with 6 inches of water at its max depth would have a 75% d/D. The figure is color-coded to show a gradation of pipes based on utilized capacity (e.g., red = flowing at >100% of its depth, orange = flowing at 85-99% of its depth, yellow = flowing at 75-84% of its depth, etc.). The planning criteria for undersized pipelines is if the flow is equal to or greater than 85% of full capacity based on maximum flow depth (d/D). As stated in Chapter 2, the Department of Environmental Quality prohibits sanitary sewer overflows, and surcharging in wastewater systems is generally discouraged.

Figure 12b in Appendix A illustrates the maximum flow divided by full flow (q/Q) during the existing system peak instantaneous flow model evaluation. q/Q shows the maximum flow experienced by the pipe segment divided by the theoretical maximum flow based on Manning's equation. 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.). This study does not establish a trigger for maximum q/Q in the collection system, but the figure does illustrate which pipes may cause bottlenecks in the future.

The existing system evaluation shows a significant portion of the modeled pipelines operating at or above capacity. This primarily observed pipelines lower in the sewer basins near the lift stations. Bottlenecks in this part of the system cause surcharging and flooding in the upstream manholes. In summary, the main trunkline conveying flows to the lift stations is undersized and will require improvements to reduce the risk of overflows and allow for growth within the City. Additional discussion of each deficiency location and alternatives to address the issue are discussed in Chapter 4.

Table 3-17 compares the PIF to the pump station measured firm capacities. As noted previously, the E Street Pump Station's measured capacity is less than the reported capacity; however, the unconstrained PIF is greater than both the measured and reported capacity. The City has experienced overflows at this lift station, indicating the pumps are insufficient. The Washington Pump Station had a slightly higher measured flow than was reported, but again, the unconstrained PIF is greater than both recorded flow rates. It is recommended that the City complete improvements at both of these lift stations to be able to handle the PIF and reduce the risk of overflows.

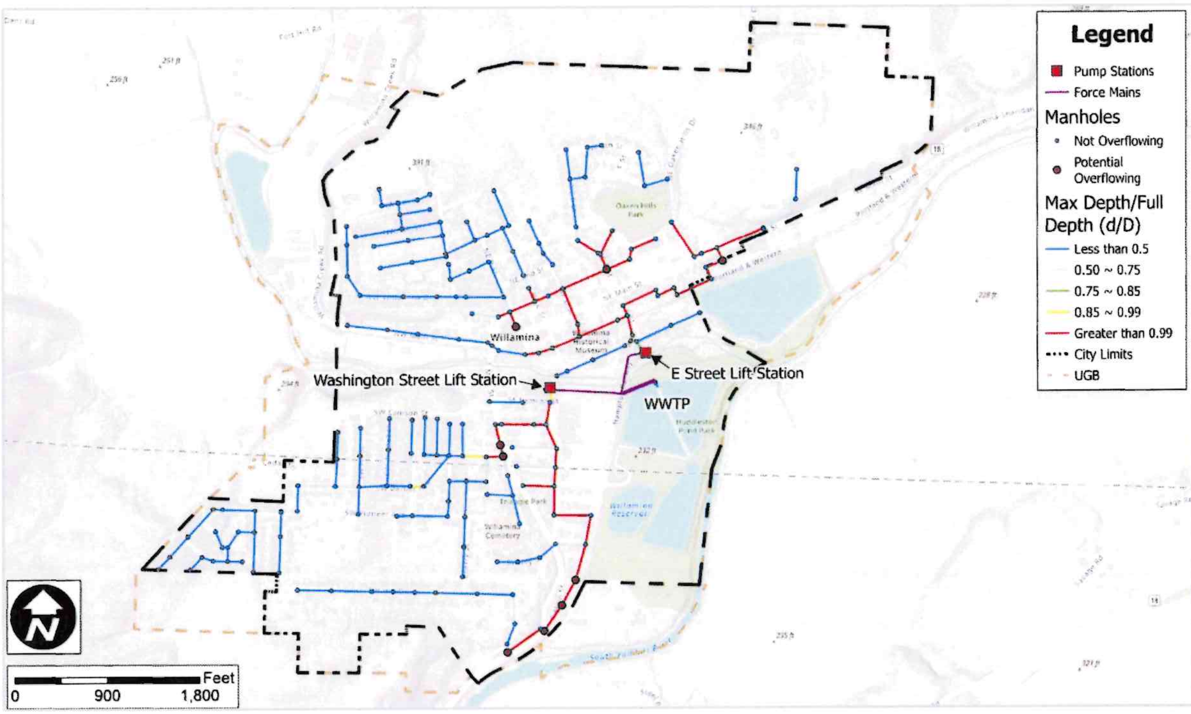
TABLE 3-17: EXISTING PIF VERSUS PUMP STATION CAPACITY

Pump Station	Reported Firm Capacity (gpm)	Field Measured Firm Capacity (gpm)[1]	Constrained PIF (gpm)	Uncsontrained PIF (gpm)	Sufficient Capacity (yes/no)[2]
E Street PS	700	500	1,250	1,400	No
Washington St PS	770	820	1,155	1,890	No

1) Field measured firm capacity based on average measurement from the two pumps.

2) Sufficient capacity based on field measured firm capacity comapred with unconstrained PIF

FIGURE 3-18: EXISTING PIF D/D



- Pipeline Conditions

In-field pipeline material condition inspection and review were not included as part of this report. However, it is important to note that one of the basic assumptions of the hydraulic model is that all lines 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 modeled capacities discussed in this chapter represent the capacity assuming the sewer lines are in good working order.

- Future Flow Projections and Model Scenarios

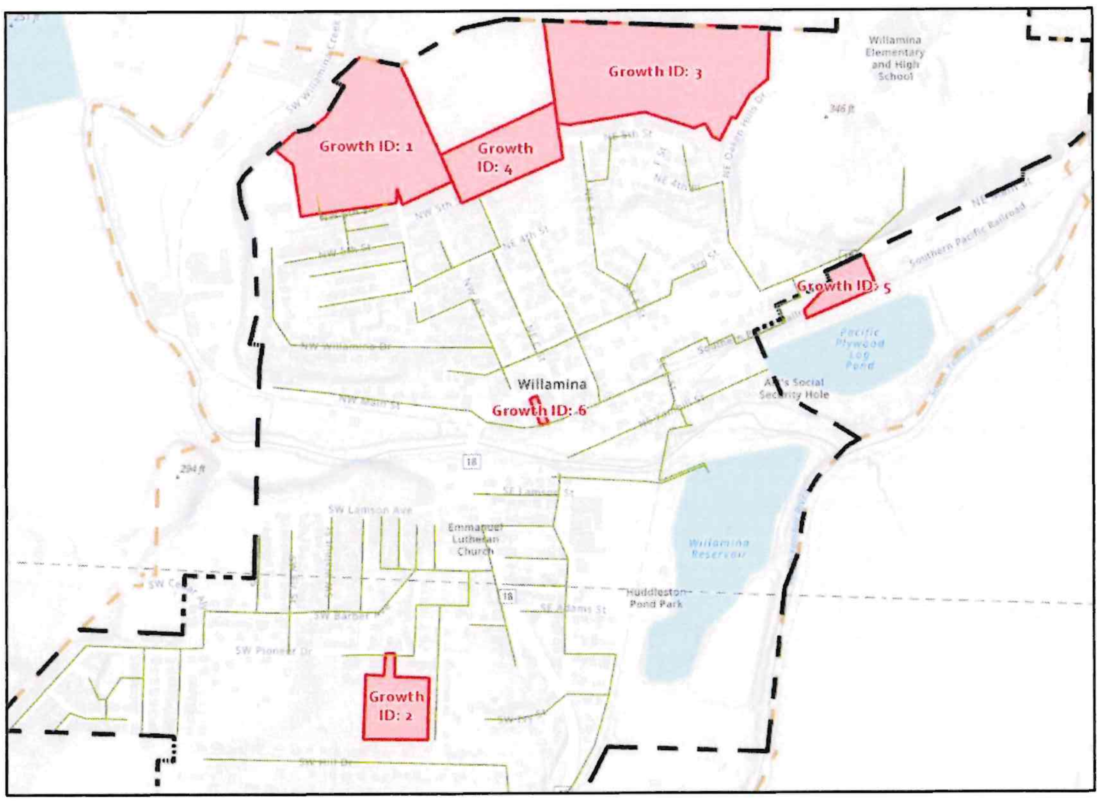
Future flows were distributed based on the EDUs assigned during the Future Growth Meeting. The topography within each growth area was reviewed, and the associated flows were assigned to the nearest existing manhole where the growth area could gravity flow. The total number of EDUs corresponds to the total projected population increase (Chapter 2). After identifying areas where growth is anticipated, 268 EDUs (545 person population increase) and the flows based on the density of the areas were distributed to the model accordingly. The City anticipates some commercial and industrial growth by adding one winery and one plywood mill within the E Street Basin. Flows per capita for projected population growth were assumed to be like existing flows per capita. Projected flows per zoning designation for the 20-year planning period are presented in Table 3-18.

TABLE 3-18: 20-YEAR PROJECTED FLOWS BY ZONE

Growth Area ID	Area (ac)	Type	#EDUs[1]	People	Flow (GPD)	Flow (GPM)	Model Load Junction
1	18.6	Single Family Residential	96	195	286,474	199	1/2 flow to J3, 1/2 flow to J7
2	4.7	Single Family Residential	26	53	77,587	54	C4-2
3	22.1	Single Family Residential	88.44	180	263,914	183	1/3 KA4, 1/3 KA12, 1/3 KA10
4	7.3	Multi-Family Residential	58	118	173,078	120	B5-4
5	2.3	Commercial (Plywood Mill)	n/a	-	5,750	4	M4
6	0.3	Commercial (Winery / Cidery)	n/a	-	2,572	2	B2-2
Total			268	545	809,374	562	-

1) EDUs from growth areas 1 and 2 are based on reported EDUs by the City. Growth Area 3 is based on 4 EDUs/gross acre. Growth Area 4 is based on 8 EDUs/gross acre and the total number of EDUs was adjusted to match the projected populations in 2045. Growth Area 5 is based on 2,500 PGD/acre. Growth Area 6 flows are based on the maximum month water consumption from Coyote Joes Restaurant in 2020.

FIGURE 3-19: GROWTH AREAS



During the future planning meeting with the City, they provided the number of EDUs allowed within the first two Growth Area ID Items. This correlated to an increase of 245 people or 122 EDU's. These growths can be seen in Figure 3-19 above.

➢ 20-Year Capacity Limitations

The model was run to evaluate the effects of a 2045 peak day flow event on the existing system. Like the Existing Capacity Limitations section, Figures 13A and 13B (Appendix A) are color-coded to show a gradation of pipes based on d/D_{full} and q/Q_{max}, respectively, and potential overflow manholes.

The capacity limitations in the 20-year projected scenario are similar to the deficiencies described in the existing system evaluation. Alternatives to alleviate these deficiencies are described in the next chapter.

3.3.7. WWTP Capacity Evaluation

Capacities of each treatment process were evaluated to assess WWTP limitations. Evaluations assume all components were online and functioning. For each process, hydraulic or biological limitations are identified.

- Headworks

The capacity limitation of the headworks process is based on its ability to pass the peak instantaneous flow. The current headworks screen provided by the City was rated for the capacity that the City required; however, the screen malfunctioned and is unusable. The City has been using a backup screen and manually raking its surface. The capacity of the original automatic mechanical influent screen (according to the screen manufacturer) is 4.32 MGD (3,000 gpm). The capacity of the City's magnetic influent flow meter is approximately 4.06 MGD (2,820 gpm). The headworks is insufficient for the existing and future 2045 instantaneous flow rates of 4.79 and 5.77 MGD, respectively.

The operator has reported that the pipe into Lagoon #1 from the diversion structure is hydraulically limited and experiences surcharging during high-flow events, causing the flow to be distributed to Lagoon #1 and Lagoon #2 concurrently.

- Aerated Lagoons #1 and #2

The capacity limitation of the aerated lagoons is based on their ability to biologically treat the organic loading into the WWTP. A minimum of two lagoons is required for reliability. The lagoons need to be able to treat the maximum month loading during the coldest influent temperatures (where the microbiological activity is slowest). While there is no current ammonia discharge limit, it is important to maintain low or non-detectable readings. For this reason, the ability of the WWTP to continually achieve nitrification was evaluated.

The lagoons were designed for a residence time of 18.2 days within Lagoon #1 and 17.9 days within Lagoon #2 at a MMWWF of 0.688 MGD. Based on the future loadings, the estimated treatment capacity of the aerated lagoons, assuming a working aeration system, is 1.3 MGD to meet permit requirements for BOD_5 removal of 85% between the two lagoons.

The firm capacity of the lagoons is sufficient for existing and future flow conditions. This capacity is based on Lagoon #1 and #2 operating at average water elevations. When temperatures are higher, operating the lagoons at maximum water depth may not be necessary. Additional detention time and facultative treatment can be obtained through Lagoons #3 and #4.

The 10-inch transfer pipe capacity is estimated to be 1.8 MGD, which is inadequate for the existing and future conditions. The limitation of this piping is based on hydraulic capacity to pass the peak day flow (it is assumed that peak instantaneous flows are buffered out in Lagoon #1). No redundancy is required in this piping. The maximum flow through the piping assumes average water depth in the lagoons.

- Aeration System

The capacity of the blowers is based on their ability to provide adequate oxygen to support the growth of the microorganisms responsible for wastewater treatment. Redundancy is required in aeration blowers and is provided with two units (one duty, one standby). While the treatment objective is to reduce BOD_5, both autotrophic and heterotrophic bacteria exist in the lagoons, and it is assumed that sufficient oxygen must be provided for both carbon and nitrogen oxidation.

The blowers can deliver 480 SCFM each and have a combined firm capacity (with one of the 40 HP blowers out of service) of approximately 1,270 lbs. oxygen per day. The aeration system was designed for an influent flow of 0.3 MGD and a BOD_5 loading of 488 lbs/day. The original aeration system was not designed with additional oxygen requirements for total Kjeldahl nitrogen (TKN). Based on the BOD and TKN loading criteria discussed in this chapter, the required oxygen requirements for treating the 2045 MMWWF of 1.49 MGD is 3,240 SCFM. The current aeration system is undersized to provide sufficient oxygen for oxidation, which limits the treatment that can occur.

- Effluent Storage Lagoons #3 and #4

Two processes are evaluated for capacity. First, the capacity of the transfer pumps to move water from Lagoon #3 to Lagoon #4, and second, the volume of these lagoons. As noted above, the two lagoons are hydraulically connected via a pump station. The capacity limitation of the transfer pumps is based on their hydraulic capacity to move water into Lagoon #4. Redundancy is required with two pumps (one duty, one standby). The pump has a firm capacity of 1.30 MGD (900 gpm). The as-built drawings show a 6-inch discharge pipe to Lagoon #4 from the transfer pump. Based on a pumping rate of 900 gpm, a velocity of 10 ft/s is developed in the discharge pipe. Typical designs intend to keep velocities in piping between 2-8 ft/s. The transfer pump does not have firm capacity for the existing and future peak day average flow.

A theoretical water balance was used to evaluate the adequacy of the storage volume. The water balance shows how much water must be held in the lagoons during the non-disposal seasons. The water balance is calculated using the influent flow to the lagoons plus precipitation and less evaporation. To be conservative, losses from spray gun evaporation or seepage through the liners were not considered. The precipitation data is taken from City rain gauges. Evaporation data is taken from local evaporation station averages (Agricultural Waste Management Field Handbook, 1998). The net loss in volume in the existing lagoons is 3.96 MG during the non-discharge period. The City should consider performing a seepage test of the lagoons and factoring the seepage volume into the water balance.

The volume of water contained in Lagoons #3 and #4 is 25 MG. This assumes Lagoon #3 operates at the maximum water depth of 8 feet which is the difference between the maximum water depth of 11 feet and the minimum water depth of 3 feet, and all 12 feet of available storage in Lagoon #4 can be utilized. Storage within the aerated lagoons were also incorporated into the water balance. It is assumed the lagoons will operate at the minimum water level which provides 2 feet of available storage between the maximum water depth of 8 feet and minimum water depth of 6 feet. This results in a storage volume of 7.9 MG. Based on the water balance, the total storage volume available within all lagoons is approximately 33 MG. This storage volume is insufficient for the current and future wastewater flows during the non-discharge season. While some excess water is likely lost due to seepage and utilization of the spray guns, the WWTP has had occasions of nearly exceeding available storage in the lagoons, and additional storage is necessary.

- Disinfection

The capacity limitation of the disinfection process is associated with the contact time of chlorine with the wastewater, as well as the capacity of the chlorine pumps. The capacity limitation of the dechlorination system is based on the capacity of the dechlorination pumps. Redundancy in the chlorine contact basin or with the chlorine mixer is not required. However, redundancy in chemical pumping is needed to ensure an identical piece of equipment is available for replacement.

The estimated chlorine contact basin volume is approximately 26,600 gallons. The basin was originally designed for a contact time for MMWF and PDWWF of 56 minutes and 28 minutes, respectively. With a volume of 26,600 gallons, the chlorine contact chamber has a capacity of up to 0.64 MGD AADWF. The required contact times by Oregon guidelines are 20 minutes at the peak daily flow, 15 minutes at peak hourly flow, or 60 minutes at average dry-weather flow, whichever results in the largest basin. The lagoons have a significant amount of storage, and it is assumed high flow events can be equalized between the lagoons to maintain contact time through the chlorine contact basin. Therefore, a contact time of 30 minutes at MMWWF was chosen for the design criteria as to not oversize the basin. The 2045 peak daily flow rate is 4.04 MGD, the peak instantaneous flow rate is 5.77 MGD, and the average dry-weather flow is 0.56 MGD. At these future design flows, the chlorine contact basin can meet the 60-minute contact time at average dry-weather flow but cannot meet the 20-minute or 15-minute requirement.

The maximum generation of sodium hypochlorite is 36 pounds per day, and the existing system was designed for a solution flow rate of 22 gph (540 gpd). The capacity of the sodium bisulfite generation is 26 pounds per day, and the existing system was designed for 0.24 gph (5.76 gpd).

Since being replaced, both the existing sodium hypochlorite chemical feed pump and sodium bisulfite chemical feed pump are rated to a maximum pump rate of approximately 5.60 gph (134 gpd). The operator reports using an average of 10 pounds of chlorine per day during the discharge period. Some issues may limit the disinfection capacity as the flows increase and detention time within the lagoons and chlorine contact basin decreases. Baffles or mixer modifications may also be recommended for future flows.

- Outfall Discharge

The capacity limitation of the outfall is based on the hydraulic capacity of the outfall piping. There are no redundancy requirements. The flow is discharged from the chlorine contact basin to a vault containing a gooseneck flow meter. The discharge from the flow meter to the river is a single port 8-inch HDPE pipe with a diffuser. Given the size and slope of the outfall, the estimated capacity is 1.39 MGD, which is insufficient.

- Summary

A summary of the existing hydraulic and treatment capacity for the unit processes at the plant is provided in Table 3-19. Alternatives to address the lagoon deficiencies and capacity limitations are discussed in Chapter 4.

TABLE 3-19: WWTP CAPACITY SUMMARY

Equipment	Governing Flow	Firm Capacity Provided (MGD)	Current Capacity Needed (MGD)	2045 Capacity Needed (MGD)	Limiting Factor
Headworks Screen/Compactor	PIF_5	4.32	4.58	5.77	Hydraulic
Aerated Lagoons (#1 and #2)	$MMWWF_5$	1.30	1.18	1.49	Detention Time, Treatment (BOD_5 removal)
Aeration System	$MMWWF_5$	0.22	1.18	1.49	Aeration requirements for treatment
Pipe between Lagoons #1 and 2	$PDAF_5$	1.80	3.21	4.04	Hydraulic
Transfer Pumps	$PDAF_5$	1.30	3.21	4.04	Hydraulic
Effluent Storage	ADWF	19.3	40.1	49.1	Non-discharge period (May 1 - Oct 30)
Chlorine Disinfection	PIF_5	2.55	4.58	5.77	Hydraulic retention time of 15 minutes

CHAPTER 4 - ALTERNATIVES CONSIDERED

There are many different alternatives to meet the wastewater facility deficiencies discussed in this facility planning study. Keller and the City discussed several options, and the alternatives evaluated are discussed in this chapter.

4.1. COLLECTION SYSTEM ALTERNATIVES

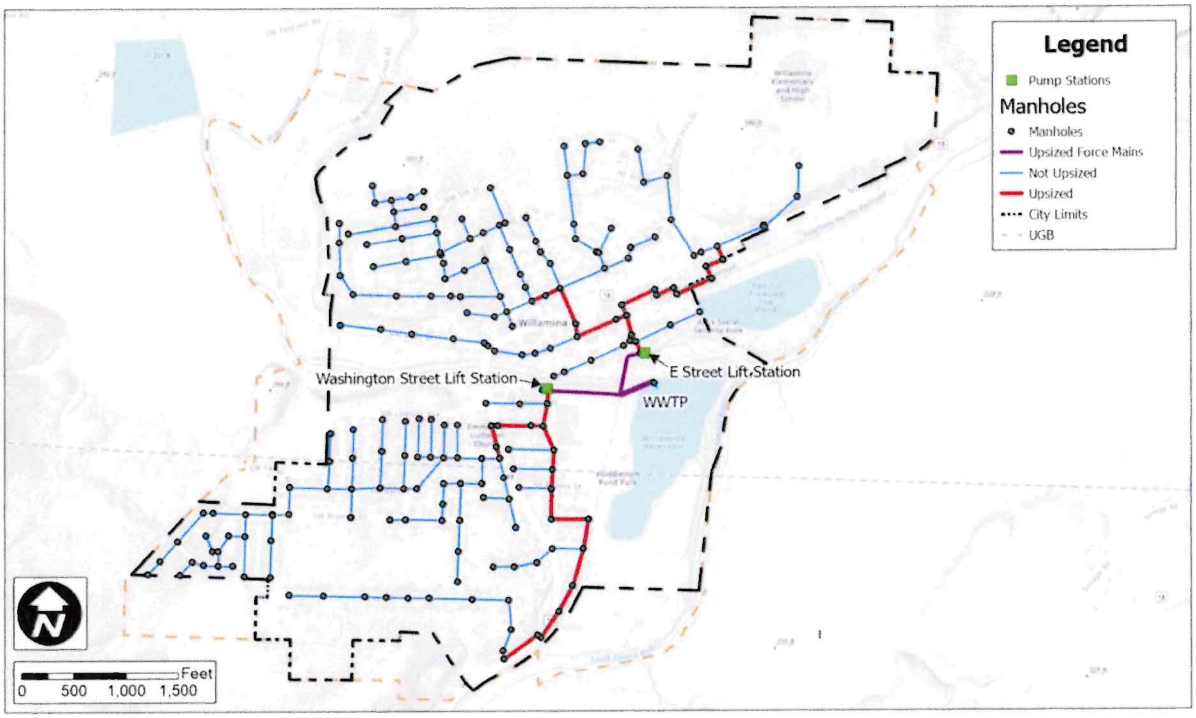
As documented in Chapters 2 and 3, there are capacity deficiencies within the collection system during the current design flow events. Improvements to correct the identified deficiencies could be addressed by implementing several improvement alternatives. This section describes alternatives which could be implemented to eliminate surcharging in the existing gravity pipes, upsize the capacity of the lift stations to meet the peak instantaneous flow, and reduce the likelihood of sanitary sewer overflows. Environmental impacts, land requirements, sustainability considerations, water and energy efficiency, green infrastructures, or other impacts are documented in this chapter. Cost comparisons and a recommended alternative selection are detailed in Chapter 5. A brief description of the alternatives is provided below.

- Alternative 1 – Upsize existing pipes in their current alignment and upgrade the two lift stations and force mains to pass the future peak instantaneous events without surcharging or overflows.
- Alternative 2 – Divert flows from SW Hill Drive to SW Cherry Street to reduce flows in the trunkline along S Main Street. This alternative does still require upsizing existing pipes in their current alignment, upsizing the force mains, and upgrading the two lift stations.
- Alternative 3 – Implement I/I reduction efforts consisting of CCTV and smoke testing, cured-in-place pipe lining (CIPP) or mainline replacement, manhole rehabilitation, and service lateral replacements. The I/I reduction efforts will reduce the length and size of pipe required to be upsized to pass the peak instantaneous events. This alternative considers two I/I target reduction scenarios:
    - Alternative 3.1 – Assumes CCTV and smoke testing to identify and repair sources of direct inflow. Install CIPP for all pipes installed in the original collection system in 1966 and lining the connected manholes. This alternative assumes a 20% reduction in PDF_5 and PIF_5 flows due to the reduced I/I achieved by the improvements.
    - Alternative 3.2 – Assumes the same improvements as Alternative 3.1; as well as replacement of all service laterals from the sewer main to the property boundary. This alternative assumes a 50% reduction in PDF_5 and PIF_5 flows due to the reduced I/I achieved by the improvements.

4.1.1. Alternative 1 – Upsize Existing Infrastructure

The hydraulic model was used to identify the required pipe upsizing required to pass the design flow event without surcharging. The pipes requiring upsizing are summarized in Figure 4-1. In general, it consists of approximately 7,300 LF of upsized pipe within the system.

FIGURE 4-1: ALTERNATIVE 1 IMPROVEMENTS



The design criteria for the lift stations under Alternatives 1 and 2 are provided in Table 4-1. These flows assume no reduction in flows from the projections in Chapter 3.

TABLE 4-1: ALTERNATIVES 1 AND 2 DESIGN FLOWS (2045)

Scenario (gpm)	E Street	Washington Street
AADF	210	170
ADWF	100	80
AWWF	340	280
PDF_5	1,510	1,220
PIF_5	2,060	1,960

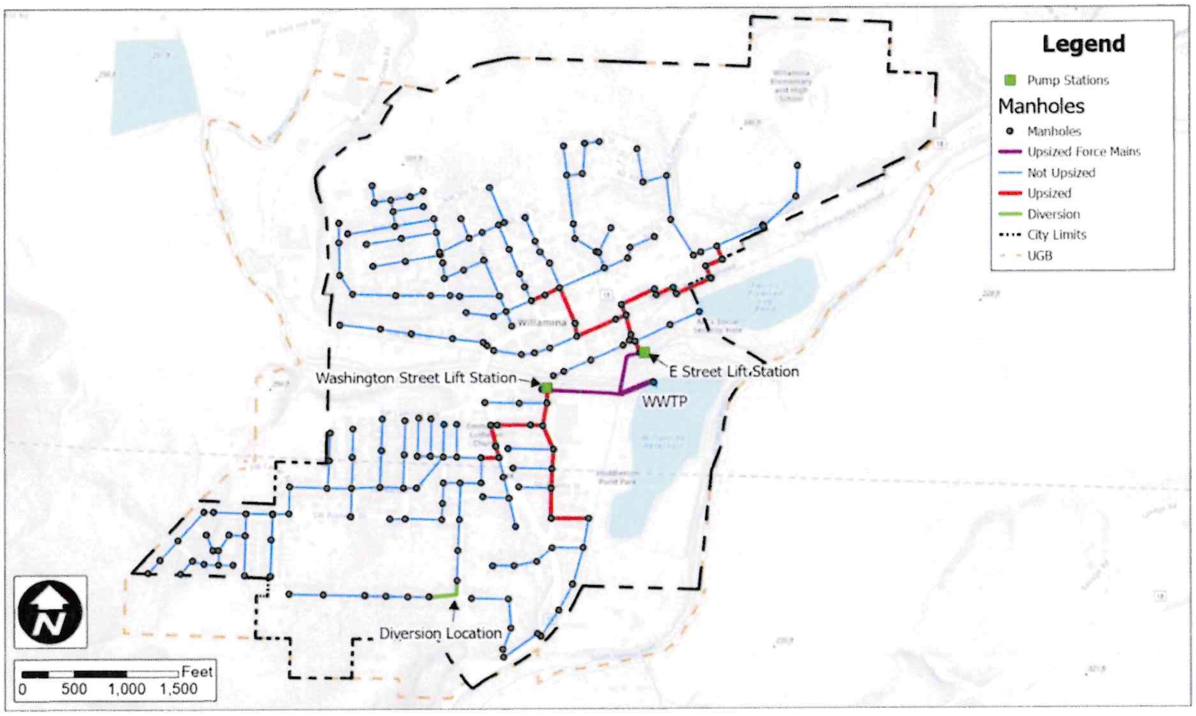
The lift stations and their force mains require additional capacity to convey the future peak flows. The current capacity of the E Street and Washington Street lift stations is 700 gpm and 770 gpm respectively. The anticipated peak flows in each basin are approximately two times greater than the existing capacity. Due to the large increase in capacity and for budgeting purposes, it was assumed both lift stations will require a complete replacement consisting of new wetwells, valve vaults, pumps, electrical components, controls, valves, and piping. For the force main upgrades, it was assumed a parallel 12-inch pipe would be constructed to the WWTP from each lift station. Due to the large fluctuation in seasonal flows, a single upsized pipe could have issues with achieving scour velocity during low flow periods. A parallel pipe would allow the City to use the existing 8-inch during low flow periods and the 12-inch during high flows. A parallel line also provides resiliency to the system and would allow the lift station to still operate in the event one of the force mains is damaged. The specific force main sizing and configuration should be evaluated further during the pre-design stage.

4.1.2. Alternative 2 – Divert Flows to SW Hill Drive

The design flows for this alternative are the same as presented in Table 4-1. The hydraulic model was used to identify the required pipe upsizing under Alternative 2 which diverts flows from SW Hill Drive to SW Cherry Street. The required pipe upsizing associated with this alternative is illustrated in Figure 4-2. In general, it consists of approximately 6,400 LF of upsized pipe within the system.

This alternative does not reduce the system flows and therefore the same improvements to the lift station and force mains are required.

FIGURE 4-2: ALTERNATIVE 2 IMPROVEMENTS



4.1.3. Alternative 3.1 – Moderate I/I Reduction

The design criteria for the lift stations under Alternative 3.1 are provided in Table 4-2. These flows assume a 20% reduction in flows from the projections included in Chapter 3. This alternative assumes a reduction in systemwide flows as a result of moderate I/I reduction activities. The assumed 20% I/I reduction was established based on studies for other collection systems in Oregon which evaluated the effectiveness of I/I reduction activities.

TABLE 4-2: ALTERNATIVE 3.1 DESIGN FLOWS

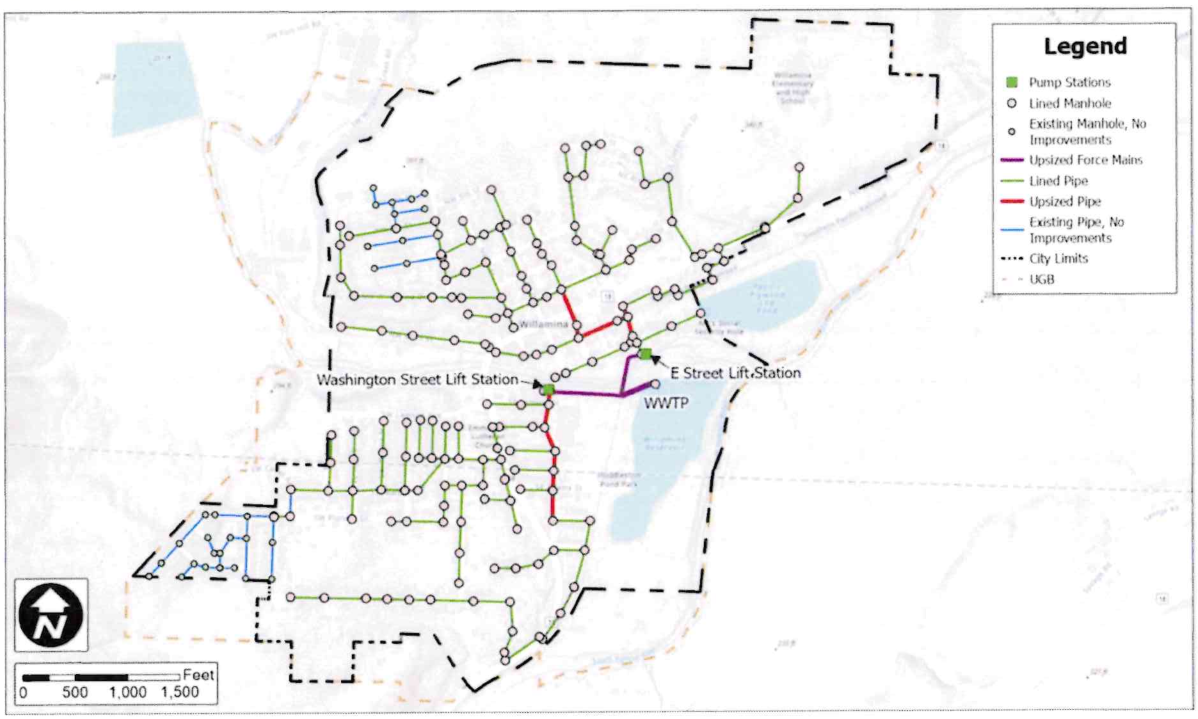
Scenario (gpm)	E Street	Washington Street
AADF	186	152
ADWF	100	80
AWWF	272	224
PDF_5	1,208	976
PIF_5	1,648	1,568

The I/I reduction efforts to achieve a 20% reduction were assumed to consist of completing CCTV and smoke testing of the entire collection system to identify sources of I/I within the collection system. Sources of I/I may include root intrusion, cracked pipes, gaps between pipes and manholes, or other defects that would allow I/I to enter the collection system. While specific improvements to reduce I/I should be identified by reviewing CCTV results and repairing primarily pipes identified with defects, the CCTV evaluation was not completed as part of this facility plan. For this study, it was assumed that the pipe installed during the construction of the original collection system in 1966 will require lining with cured-in-place pipe and lining the connected manholes. This consists of approximately 87% of the total collection system by length. It was assumed sewer pipes constructed more recently have less I/I due to the newer materials and more watertight construction. I/I reduction efforts in the new areas would have less of an impact compared to the other parts of the system and are not likely worth the effort.

The hydraulic model was used to identify the required pipe upsizing under the 20% flow reduction scenario. The extent of the I/I improvements and pipe upsizing are illustrated in Figure 4-3. In general, this alternative consists of 37,000 LF of cured-in-place pipe, 180 lined manholes, and 6,000 LF of upsized pipe.

While flows are reduced in this alternative, lift station and force main upgrades are still required in this alternative.

FIGURE 4-3: ALTERNATIVE 3.1 IMPROVEMENTS



4.1.4. Alternative 3.2 – Aggressive I/I Reduction

The design criteria for the lift stations under Alternative 3.2 are provided in Table 4-2. These assume a 50% reduction in I/I flows from the projections included in Chapter 3 as a result of aggressive I/I reduction activities. The assumed 50% I/I reduction was established based on studies for other collection systems in Oregon which evaluated the effectiveness of I/I reduction activities.

TABLE 4-3: ALTERNATIVE 3.2 DESIGN FLOWS

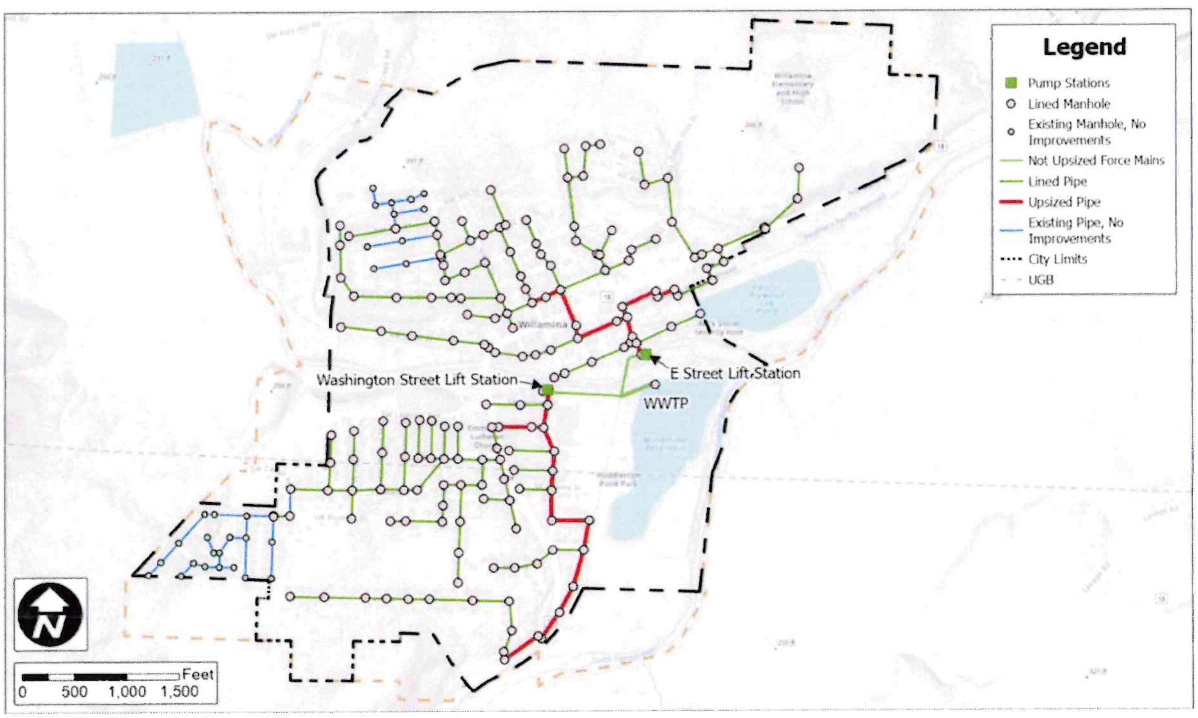
Scenario (gpm)	E Street	Washington Street
AADF	135	110
ADWF	100	80
AWWF	170	140
PDF_5	755	610
PIF_5	1,030	980

The I/I reduction efforts were assumed to consist of the same activities described in Alternative 3.1, but also include the replacement of sewer service laterals from the sewer main to the property boundary for areas included in the original collection system.

The hydraulic model was used to identify the required pipe upsizing under the 50% flow reduction scenario. The extent of the I/I improvements and pipe upsizing is illustrated in Figure 4-4. In general, this alternative consists of 41,000 LF of cured-in-place pipe, 180 lined manholes, and 2,500 LF of upsized pipe.

The flows are significantly reduced under this alternative and the extent of the lift station upgrades is less than the other alternatives. This alternative assumes the existing wetwells, vaults, valves, and piping can be used with the upgrades consisting of new pumps and electrical equipment. The existing 8-inch force mains are sufficient due to the decreased flows and therefore this alternative does not include any force main improvements.

FIGURE 4-4: ALTERNATIVE 3.2 IMPROVEMENTS



4.2. COLLECTION SYSTEM ENVIRONMENTAL IMPACTS

The potential environmental impacts of the alternatives are summarized in the following sections. A summary of the impacts is presented in Table 4-4.

TABLE 4-4: COLLECTION ALTERNATIVES GENERAL IMPACT SUMMARY

Impact Criteria	Alt. 1: Upsize Existing Infrastructure	Alt. 2: Divert Flows and Upsize Existing Infrastructure	Alt. 3.1: Moderate I/I Reduction	Alt 3.2: Aggressive I/I Reduction
Land Use/ Important Farmland/ Formally Classified Lands	No impact	No impact	No impact	No impact
Floodplains/ Wetlands	Lift stations are in the 500-year floodplain. If moved, the lift station elevations will be higher than the floodplain.	Lift stations are in the 500-year floodplain. If moved, the lift station elevations will be higher than the floodplain.	Lift stations are in the 500-year floodplain. If moved, the lift station elevations will be higher than the floodplain.	No impact
Cultural, Biological, and Water Resources	No impact	No impact	No impact	No impact
Socio-Economic/ Environmental Justice Issues	Economic Costs	Economic Costs	Economic Costs	Economic Costs

4.2.1. Land Use / Prime Farmland / Formally Classified Lands

None of the alternatives are anticipated to change land use, impact prime farmland, or disturbed significant new land. The improvements are expected to occur within the existing pipe alignment and right-of-way.

4.2.2. Floodplains / Wetlands

The two existing lift stations are located within the mapped Willamina Creek 500-year floodplain. Improvements to the lift stations will include provisions for flood protection if they are moved into the floodplain. Some of the existing pipe alignments cross wetlands but construction techniques will be used to avoid impacting the wetlands and no new obstructions to the flood plain or wetland areas are anticipated.

4.2.3. Cultural, Biological, and Water Resources

The improvements being evaluated are on previously disturbed lands and it is not anticipated that any of the alternatives will interfere with cultural, biological, or water resources.

4.2.4. Socio-Economic Conditions

Alternatives are not anticipated to have a disproportionate effect on any segment of the population (economic, social, or cultural status). The main economic impact is the cost of the alternatives.

4.3. COLLECTION SYSTEM LAND REQUIREMENTS

The alternatives do not likely require the purchase of additional land. The lift station upgrades were assumed to be completed within the existing City-owned property. The pipeline improvements are assumed to be within the right-of-way.

4.4. COLLECTION SYSTEM POTENTIAL CONSTRUCTION PROBLEMS

Potential construction problems associated with the alternatives are summarized below.

- Subsurface Bedrock – Unforeseen ground conditions such as rocky terrain, unstable soil, or groundwater can complicate excavation and trenching processes, leading to delays and increased construction costs. Specifically, if the existing trenches are not large enough for the upsized gravity pipes.

- High Groundwater – High groundwater during construction would require dewatering.
- Crossing Willamina Creek – Force main improvements from the E Street Lift Station would require crossing Willamina Creek. Construction would likely consist of boring or directional drilling underneath the creek.
- Conflicting Utilities – One of the most common issues is encountering existing utility lines like water pipes, gas lines, or electrical cables. Accidentally damaging these lines can cause disruptions to essential services and pose safety hazards.
- Traffic Disruption – Construction activities may disrupt traffic flow in urban areas, leading to congestion, detours, and safety hazards for motorists and pedestrians.

4.5. COLLECTION SYSTEM SUSTAINABILITY CONSIDERATIONS

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

4.5.1. Water and Energy Efficiency

Reducing I/I leads to smaller pumps in the lift stations and lower power usage. Additionally, there may be less power consumption at the WWTP due to the decreased flows. The lift station upgrades could include soft starts, VFDs, and energy efficient pumps.

4.5.2. Green Infrastructure

Pipeline improvements could be coordinated with local stormwater improvements. Reducing I/I flows leads to less volume discharged from the WWTP.

4.5.3. Other

System resiliency and simplicity will be optimized with the updated SCADA required to complement the alternatives.

4.6. COLLECTION SYSTEM COST ESTIMATES

The advantages, disadvantages, and comparative costs of the alternatives are presented in Chapter 5. The cost estimates are a Class 5 cost opinion, as defined by the Association for the Advancement of Cost Engineering (AACE). The costs are provided in Appendix I. Lift-cycle cost estimates were not completed for the collection system alternatives because each of the alternatives will have similar costs for O&M, useful life, and labor. There will be less power usage for the I/I reduction alternatives; however, the capital costs are the most influential factor when evaluating the costs.

4.7. TREATMENT AND STORAGE ALTERNATIVES

Alternative solutions to address the insufficient storage capacity within the lagoons are discussed below. Each alternative would also require specific treatment improvements to address the additional deficiencies discussed in Chapter 3. I/I reduction from collection system improvements would mainly improve the treatment in the lagoons by increasing the retention time during wet weather. The impact on the storage requirements would not be as dramatic since the storage period (non-discharge) is from late spring to early fall, and I/I is already reduced during this period.

4.7.1. Design Criteria

The characteristics of the wastewater that form the basis for sizing the wastewater facilities are summarized in Chapter 3. Design criteria that will be used for sizing various potential components are summarized in several parts in the following sections.

4.7.2. Regionalization

Due to the political complexity, physical distance, and pipeline cost between Willamina and a city with larger wastewater facilities, developing a partnership with another community to share wastewater facilities is not currently of interest to the City.

4.7.3. No Action Alternative

This option is to continue to dispose of the water as is currently done without incorporating changes. As mentioned in Chapter 3, there is inadequate lagoon storage and treatment deficiencies with the headworks, aeration, and disinfection system. Water would continue to be stored in the effluent storage lagoons during the summer until it can be discharged to the South Yamhill River. However, as flows increase, there will not be enough storage available within the lagoons for the entire non-discharge period, and the City could face overflow events or non-permitted discharge, violating the current NPDES permit.

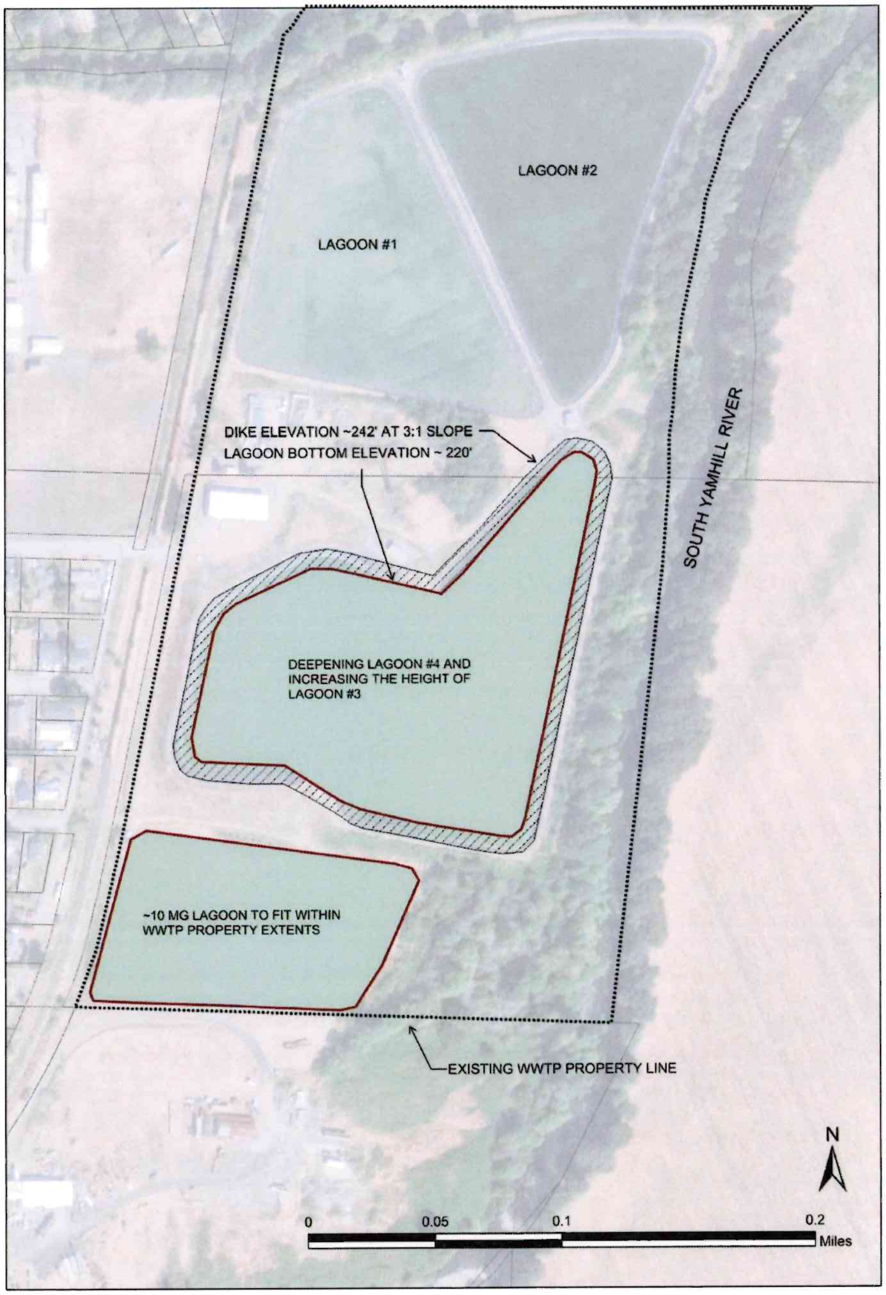
4.7.4. Reuse

To decrease the storage needed during the non-discharge period, the City previously evaluated reuse of the water on poplar trees within the available land at the WWTP. However, the reuse design was never executed due to the difficulty in operating the reuse system and removing the trees. For similar reasons, the City is not currently interested in evaluating reuse further.

4.7.5. New Storage Lagoon

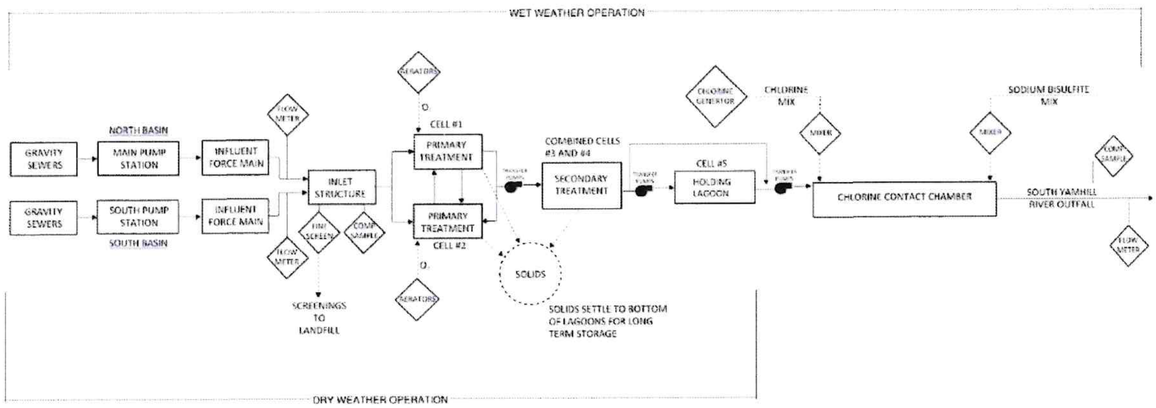
This option would add storage capacity with a new storage lagoon to supplement the storage provided in Lagoons #3 and #4. This alternative would allow the City to maintain the current permit requirements and have sufficient storage during the non-discharge period. A map showing the proposed location of the new lagoon is shown in Figure 4-5.

FIGURE 4-5: NEW STORAGE LAGOON #5



Based on the future conditions, it is expected that up to a total of 49 MG of wastewater flow must be stored during the 20-year planning period. With the existing lagoons having a volume of 33 MG, an additional 17 MG of storage volume is required. This alternative would construct a new Lagoon #5 with approximately 10 MG capacity south of Lagoons #3 and #4. To meet future storage requirements, Lagoons #3 and #4 would also need to be combined. Additional WWTP improvements included as part of this alternative are headworks improvements (including a new mechanical screen, flow measurement, and larger diversion box and piping), new aeration system and blowers in Lagoons #1 and #2, disinfection system improvement (including a larger chlorine contact basin and increased capacity chlorine disinfection system), a new SCADA system and backup power, the combination of Lagoon #3 and Lagoon #4, and larger piping between the lagoons and from the chlorine contact basin to the outfall. The pump station would be modified to transfer water between Lagoon #3 and the new storage lagoon (Lagoon #5). The projects as part of this alternative are discussed in more detail in Chapter 6. The schematic for this alternative is shown in Figure 4-6.

FIGURE 4-6: WWTP SCHEMATIC WITH STORAGE LAGOON #5

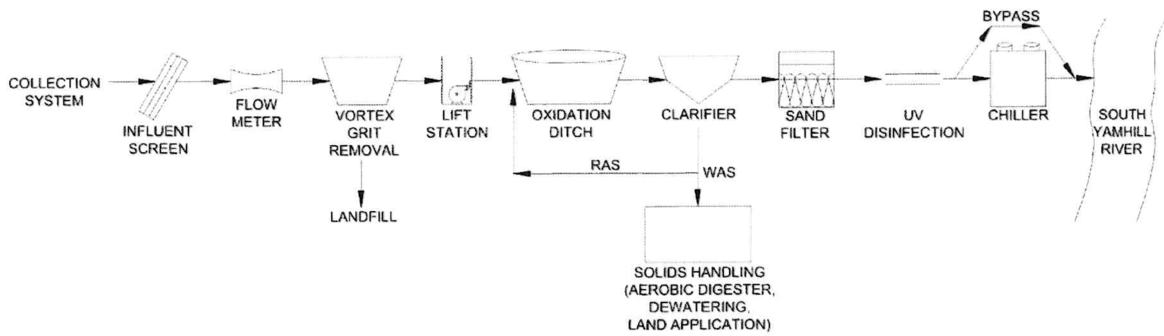


4.7.6. Mechanical Treatment (Year-Round Discharge)

Year-round discharge to the South Yamhill River would eliminate the need to increase the storage; however, more stringent permit limits may be required to protect the river during the dry season (currently the non-discharge season). These permit limits might include ammonia, phosphorus, and temperature. The cost for the additional treatment facilities to achieve ammonia, phosphorus, and temperature limits would be very significant. A sophisticated mechanical plant was chosen for this alternative to meet the required treatment levels consistently, including tertiary treatment and cooling.

For this alternative, we have included a new headworks with screening and grit removal to protect the downstream treatment equipment, oxidation ditches with secondary clarifiers (mechanical secondary treatment system) for ammonia, BOD_5, and TSS removal, followed by sand filters for phosphorus removal, ultraviolet light (UV) disinfection to meet the total residual chlorine and E. coli requirements, and mechanical cooling (cooling towers or chillers) for temperature reduction. With this alternative, there would be more biosolids production than with a lagoon system. The biosolids would be aerobically digested, dewatered, and land applied by farmers. It was assumed that the new treatment facilities could be constructed within the footprint of Lagoon #1. A schematic of the alternative is shown in Figure 4-7.

FIGURE 4-7: MECHANICAL TREATMENT SCHEMATIC



Due to the expected hydraulic loss through the treatment process, a lift station is anticipated to be required before the oxidation ditch. The oxidation ditch process uses activated sludge (a concentrated mixture of microorganisms) to remove biodegradable organics and ammonia to comply with effluent requirements. The incoming wastewater is mixed with activated sludge that is settled and returned from secondary clarifiers (return activated sludge (RAS)). Aeration is added to some sections of the oxidation ditch and DO probes would be used to ensure adequate DO is present and initiate automatic process control. An example of oxidation ditches and secondary clarifiers is shown in Figure 4-8.

FIGURE 4-8: OXIDATION DITCHES AND SECONDARY CLARIFIERS

Chemical addition and filtration would be used downstream of the secondary clarifiers to polish the water further by removing small particles and phosphorus that did not settle in the clarifiers. Upflow sand filters are included in this alternative. The water enters near the bottom of the filter tanks and flows up through layers of granular media, filtering out solids in the water. When the water reaches the top of the filter, it passes over the effluent weir. A backwash system operates to remove the solids that are collected on the sand. The backwash water is recycled back to the WWTP influent lift station. An example of sand filters is shown in Figure 4-9.

FIGURE 4-9: SAND FILTERS

Following filtration, the treated water would next be disinfected. For this alternative, UV disinfection was included. UV can be very effective with the high quality of treated water coming from the filters. Mechanical cooling (cooling towers or chillers) would be used for temperature reduction.

Mechanical secondary treatment requires routine disposal of biosolids. For this alternative, it was assumed that the biosolids would be stabilized in aerobic digesters and mechanically dewatered at the WWTP. The treated biosolids would then be hauled to farmers' fields for land application. The sludge is pumped from the secondary clarifiers to the dewatering equipment, mixed with a polymer and dewatered. The filtrate (water that is removed from the biosolids) is sent to the new lift station.

4.8. WWTP ENVIRONMENTAL IMPACTS

The potential environmental impacts of the alternatives are summarized in the following section. A summary of the impacts is shown in Table 4-5.

4.8.1. Land Use / Prime Farmland / Formally Classified Lands

The improvements would be on already disturbed land at the WWTP.

4.8.2. Floodplains / Wetlands

None of the alternatives would create new obstructions to the flood plain or be located in wetland areas.

4.8.3. Cultural, Biological, and Water Resources

The improvements being evaluated are on previously disturbed lands and it is not anticipated that any of the alternatives will interfere with cultural, biological, or water resources.

4.8.4. Socio-Economic Conditions

Alternatives are not anticipated to have a disproportionate effect on any segment of the population (economic, social, or cultural status). The main economic effect is the cost of the alternatives.

TABLE 4-5: WWTP ALTERNATIVES GENERAL IMPACT SUMMARY

Impact Criteria	Alt. 1: New Storage Lagoon	Alt. 2: Mechanical Treatment
Land Use/ Important Farmland/ Formally Classified Lands	No impact. City owns the land required for the lagoon.	No impact. City would construct mechanical treatment at existing treatment plant site.
Floodplains/ Wetlands	No Impact	No Impact
Cultural, Biological, and Water Resources	No Impact	Undetermined - changing to discharge during current non-discharge period.
Socio-Economic/ Environmental Justice Issues	Economic Costs for Construction and O&M	Economic Costs for Construction and O&M

4.9. WWTP LAND REQUIREMENTS

The alternatives can be completed on the remaining available land that the City owns. Potential expandability is also available in land to the west of the current WWTP for storage requirements beyond the 20-year planning period. This land has the potential to be converted into reuse land application or stormwater improvements.

4.10. WWTP 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. 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. Construction plans for any of the alternatives would also include provisions to control dust and runoff.

4.11. WWTP SUSTAINABILITY CONSIDERATIONS

Sustainable utility management practices include environmental, social, and economic benefits that aid in creating a resilient utility. Additional treatment at the WWTP would require additional energy but improve the effluent water quality.

4.11.1. Water and Energy Efficiency

A mechanical treatment system is more efficient and has less footprint than the current aerated lagoon system. However, the additional treatment needed for discharge during the current non-discharge period results in significantly more energy usage. Also, the mechanical treatment will generate additional biosolids that must be removed. Biosolids treatment with land application, because of the nutrients, would be beneficial to the farmland.

4.11.2. Green Infrastructure

Any pump station and blowers will consider VFDs and energy-efficient pumps.

4.11.3. Other

System resiliency and simplicity will be optimized with the updated SCADA required to complement the alternatives.

4.12. COST ESTIMATES

The advantages, disadvantages, and comparative costs of the WWTP alternatives are presented in Chapter 5. The cost estimates are a Class 5 cost opinion, as defined by the AACE. In addition to project capital costs, annual 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 2.5%. The equipment (unless a short-lived asset) is assumed to have a 20-year useful life, so no depreciation or salvage value is included when comparing the alternatives. An average rate of $0.09 per kWh was used to estimate power costs, and an average labor cost of $50 per hour was used to estimate maintenance costs.

CHAPTER 5 - SELECTION OF AN ALTERNATIVE

This chapter evaluates the alternatives from Chapter 4. The advantages, disadvantages, and comparative costs (where applicable) are presented.

5.1. COLLECTION SYSTEM ALTERNATIVES

5.1.1. Collection System Cost Estimates

The capital costs for each of the alternatives is summarized in Table 5-1 and the detailed costs are provided in Appendix I. The costs for Alternative 1 consist of the upsized pipe in the system to convey the projected 20-year peak flows. The costs also also include two new lift stations, service line reconnections, new force main piping, and replacement manholes. Alternative 2 consists of similar elements included in Alternative 1 but reflect the different lengths of pipe to be upsized due to the diverted flows.

The I/I reduction alternative costs include flow monitoring, CCTV, and lining sewer mains and manholes. Alternative 3.1 also still includes upsizing pipes, lift stations, and force mains to pass the 20-year peak flow event assuming a 20% reduction in I/I flows. Alternative 3.2 includes similar elements to Alternative 3.1, but also the service laterals would be replaced. However, due to the reduction in I/I flows, the existing lift stations and force mains can continue to be used and costs only include upsized pumps and electrical equipment.

TABLE 5-1: COLLECTION SYSTEM ALTERNATIVE COSTS

Item	Alt 1	Alt 2	Alt 3.1	Alt 3.2
Lift Station Replacement	$ 2,552,000	$ 2,552,000	$ 2,552,000	$ 1,640,000
Gravity Pipe Upsizing	$ 4,005,000	$ 3,372,000	$ 3,212,000	$ 1,328,000
I/I Reduction Efforts	$ -	$ -	$ 4,952,000	$ 6,623,000
Force Main Capacity	$ 1,000,000	$ 1,000,000	$ 1,000,000	$ -
Improvements Subtotal	**$ 7,557,000**	**$ 6,924,000**	**$ 11,716,000**	**$ 9,591,000**
Mobilization and General Conditions	$ 1,520,000	$ 1,410,000	$ 2,370,000	$ 1,920,000
Subtotal	**$ 9,077,000**	**$ 8,334,000**	**$ 14,086,000**	**$ 11,511,000**
Construction Contingency	$ 1,589,000	$ 1,456,000	$ 2,464,000	$ 2,016,000
Subtotal	**$ 10,666,000**	**$ 9,790,000**	**$ 16,550,000**	**$ 13,527,000**
Market Contingency	$ 681,000	$ 624,000	$ 1,056,000	$ 864,000
Subtotal	**$ 11,347,000**	**$ 10,414,000**	**$ 17,606,000**	**$ 14,391,000**
Contractor Overhead & Profit	$ 760,000	$ 700,000	$ 1,180,000	$ 960,000
Total Construction Cost	**$ 12,107,000**	**$ 11,114,000**	**$ 18,786,000**	**$ 15,351,000**
Engineering, Legal, and Administative	$ 3,412,675	$ 3,137,850	$ 5,259,650	$ 4,303,775
Total Project Cost	**$ 15,520,000**	**$ 14,260,000**	**$ 24,050,000**	**$ 19,660,000**

The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. 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 costs presented herein.

5.1.2. Collection System Non-Monetary Factors

In the section below, Table 5-2 shows the overall advantages and disadvantages of each alternative. Additional information for these alternatives is provided below.

TABLE 5-2: CONVAYENCE SYSTEM ADVANTAGES AND DISADVANTAGES

Alternatives	Advantages	Disadvantages
Alternative 1: Upsize Existing Infrastructure	• Sizes facilities for worst-case scenario • Low risk for future sanitary sewer overflows (SSOs)	• Does not address current I/I issues • Higher capital costs
Alternative 2: Divert Flows and Upsize Existing Infrastructure	• Less pipe to be upsized than Alternative 1 • Lower costs than Alternative 1 • Utilizes additional capacity in other trunklines	• Does not address current I/I issues
Alternative 3.1: Moderate I/I Reduction	• Requires less pipe to be upsized • Reduces I/I flows • Benefits at the WWTP due to lowered flows	• Does not fully relieve the system of I/I • Complex implementation • Higher risk for SSOs • Reduction in flows is not guaranteed • Requires active I/I reduction program
Alternative 3.2: Aggressive I/I Reduction	• Requires the least amount of pipe to be upsized. • Reduces I/I flows • Benefits at the WWTP due to lowered flows	• Complex implementation • Higher risk for SSOs • Reduction in flows is not guaranteed • Requires active I/I reduction program

Alternatives 1 and 2 are the most straight-forward approaches and sizes the infrastructure for the worst-case flow scenario. There is less uncertainty with this alternative and the upsized infrastructure results in the lowest potential for future sanitary sewer overflows (SSOs). Alternatives 3.1 and 3.2 have more risk involved because I/I mitigation efforts do not have guaranteed reduction in flows. The effectiveness of I/I mitigation varies significantly from system to system and the actual reduction in flows may vary from the values assumed in this study. Additionally, Implementation of Alternatives 3.1 and 3.2 would be complex and would require a detailed phased approach to balance the need to upsize infrastructure today and plan for reduction in flows in the future. The first phased item would consist of data collection including CCTV, smoke testing, and flow monitoring. Sizing pipes and pumps for the improvements to reduce the risk of SSOs would be difficult because the impact of the I/I reduction efforts cannot be observed until the efforts are complete. Whether Alternatives 3.1 or 3.2 are selected or not, an ongoing I/I reduction program should be implemented in the City. Typically, I/I reduction projects see the highest reduction in flows during the first couple years following the improvements; however, if an active program to continue with the efforts is not implemented, the flows would increase over time (as the collection system continues to age) and could exceed the infrastructure capacity.

5.1.3. Collection System Recommendation

A combination of the alternatives described above is recommended and should be implemented in a phased approach. The capital improvement plan in chapter 6 describes the recommended phasing and costs in further detail. The City's recent SSOs have occurred at the lift stations. For this reason, the highest priority should consist of upgrading the lift stations and force mains with the criteria described in Alternatives 1 and 2. While these projects are occurring, a I/I reduction program consisting of CCTV and pipeline repair of observed sources of inflow should be done. If SSOs are still occurring after the lift station and force main upgrades, upsizing gravity pipelines should be completed either concurrently or prior to the I/I reduction efforts. As a part of the I/I reduction effort a subsequent study to review the updated flows should be completed. Additional flow monitoring and modeling should be a part of this study to determine if there is a significant reduction in flows. This analysis should include a re-evaluation of the pipe sizes required to pass the peak flows without surcharging. Unless long-term decreases in the flows are observed, it is not recommended that the upgrades be sized assuming a significant reduction in flows. Following the lift station improvements, the next highest priority capital improvement project should consist of upsizing the gravity pipelines as described in Alternative 2 or as updated in the I/I reduction study. The last recommended improvement should include the upsizing of the additional gravity sewer piping to allow for unconstrained flow in the system if the I/I reduction does not cover any reduction in flows.

5.2. TREATMENT AND STORAGE ALTERNATIVES

The alternatives for effluent disposal presented in Chapter 4 include continued winter discharge with a new storage lagoon and year-round discharge with upgrading to a mechanical treatment plant.

5.2.1. Life Cycle Cost Analysis

Cost estimates for the Chapter 4 treatment and storage alternatives are presented in Table 5-3.

TABLE 5-3: TREATMENT AND STORAGE ALTERNATIVES COST COMPARISON

Item	Alt 1: New Storage Lagoon	Alt 2: Mechanical Treatment
Headworks	$ 651,000	$ 3,100,000
Lagoon Improvements	$ 2,294,000	$ -
New Storage Lagoon	$ 1,358,000	$ -
Combining Lagoon #3 and #4	$ 1,852,000	
Secondary Treatment	$ -	$ 9,200,000
Filtration	$ -	$ 3,700,000
Disinfection Systems	$ 656,000	$ 2,500,000
Effluent Cooling	$ -	$ 5,000,000
Solids Handling	$ -	$ 7,200,000
Discharge	$ 161,000	$ 161,000
Improvements Subtotal	**$ 6,972,000**	**$ 30,861,000**
Mobilization & General Conditions	$ 697,000	$ 3,086,000
Subtotal	***$ 7,669,000***	***$ 33,947,000***
Market Contingency	$ 767,000	$ 3,390,000
Subtotal	***$ 8,436,000***	***$ 37,337,000***
Construction Contingency	$ 2,531,000	$ 11,200,000
Subtotal	***$ 10,967,000***	***$ 48,537,000***
Contractor Overhead & Profit	$ 1,097,000	$ 4,850,000
Total Construction Cost	**$ 12,064,000**	**$ 53,387,000**
Engineering, Legal, and Administrative	$ 3,020,000	$ 13,350,000
Total Project Cost	**$ 15,084,000**	**$ 66,737,000**
Electricity and Fuel	$ 50,000	$ 140,000
Chemicals	$ 15,000	$ 40,000
Disposal	$ 1,000	$ 30,000
Parts	$ 17,000	$ 40,000
Personnel	$ 20,000	$ 130,000
Estimated Annual O&M	$ 103,000	$ 380,000
20-Year Life Cycle Cost	**$ 16,770,000**	**$ 72,670,000**

The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. 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 costs presented herein.

5.2.2. Non-Monetary Factors

Table 5-4 presents an evaluation of the effluent disposal alternatives.

TABLE 5-4: TREATMENT AND STORAGE ALTERNATIVES EVALUATION

Alternative	Advantages	Disadvantages
Alt. 1: New Storage Lagoon	• Lowest capital cost. • Lowest life-cycle cost. • Less sludge production. • Same discharge permit requirements. • Same operator license.	• Large storage required. • Additional operation considerations. • Larger construction footprint than other alternative. • Future expansion may be more difficult. • Chlorine safety hazards.
Alt. 2: Mechanical Treatment (Year-Round Discharge)	• Higher quality water. • Provides flexibility for water reuse. • Easier to add advanced treatment to remove emerging contaminants of concern (e.g., PFAS). • Eliminates the need for increasing the effluent storage.	• Higher capital and O&M than other alternative. • Likely more stringent effluent limits. • More frequent biosolids removal. • Entirely new treatment process to learn. • Higher operator license (estimate Class II).

5.2.3. Treatment and Storage Recommendation

The recommended alternative is Alternative 1 - New Storage Lagoon. Alternative 1 upgrades the WWTP treatment while requiring the lowest capital and O&M costs. The treatment facility would remain Class I, and additional permitting would not be necessary. The new storage lagoon is a large capital investment. However, this alternative avoids the complications that come with upgrading to a mechanical treatment process, such as frequent biosolids handling and higher electricity costs.

5.2.4. I/I Mitigation Impacts on WWTP

Collection system improvement alternatives to reduce I/I have potential impacts on capital improvement costs associated with the WWTP. Both 20% and 50% flow reductions from I/I improvements were compared (Collection System Alternatives 3.1 and 3.2, respectively); however, actual results will vary. The potential impacts to the WWTP Alternative 1 improvement costs from I/I reductions are discussed below.

➢ Headworks

The governing flow condition for headworks equipment is the peak flow into the plant. I/I improvements would reduce the 2045 peak instantaneous flow from 5.77 MGD to 4.63 MGD at 20% reduction and down to 2.89 MGD at 50% reduction.

At 20% flow reduction, transfer piping size to the lagoons can be reduced. This would result in a reduction of the total project costs by approximately $113,000.

At 50% flow reduction, a slightly smaller screen and channel could be installed as well as reduced transfer piping size to the lagoons. This would result in a reduction of the total project costs by approximately $188,000.

➢ Lagoon Improvements

I/I improvements would also reduce the 2045 maximum month wet weather flow from 1.49 MGD to 1.24 MGD at 20% reduction and 0.88 MGD at 50% reduction. These reductions would impact both transfer piping and transfer pumping station capacity as well as treatment capacity. New transfer piping and a new pump station would still be required.

At 20% flow reduction, the transfer piping between Lagoons #1 and #2, as well as the discharge to Lagoon #3 can be reduced, and smaller pumps can be used for the transfer pump station. This would result in a reduction of the total project costs by approximately $64,000.

At 50% flow reduction, the transfer piping between Lagoons #1 and #2, as well as the discharge to Lagoon #3 can be reduced, and smaller pumps can be used for the transfer pump station. This would result in a reduction of the total project costs by approximately $195,000.

The reduced flows would not result in any capital savings for the aeration system as the loads would not change, but the reduced flows would provide greater flexibility in operations for the water levels within the lagoons. Reducing the I/I would increase the concentration of BOD and TSS into the treatment plant, which would in turn make it easier to achieve the target percent removal of these constituents required in the permit (i.e., not as low of an effluent concentration would be required to meet the percent removal requirement).

- Disinfection Improvements

I/I improvements would make it easier to meet the required chlorine contact times. The reduction in flow would minimize the size needed for the contact basin.

A 20% reduction in flows would require a smaller chlorine contact basin. Upsizing the contact basin would still be required but could be reduced from 100 feet to 80 feet. This would result in cost savings of approximately $50,000.

A 50% flow reduction would reduce the length of the basin to 60 feet. This would result in cost savings of approximately $91,000.

- Outfall Improvements

I/I improvements would impact the piping from the chlorine contact basin to the discharge sampling manhole. A 20% reduction in flows would not have an impact on the costs. New transfer piping would still be required but could be reduced in diameter. A 50% flow reduction would result in a reduction of the total project costs by approximately $32,000.

5.3. COMBINED I/I ALTERNATIVE RECOMMENDATIONS

While I/I reduction activities result in reducing the need for upsized infrastructure in the collection system as well as cost savings at the WWTP, the savings at the WWTP are minimal in comparison to the collection system capital costs as shown in Table 5-5 below. Additionally, the potential flow reduction the WWTP will experience as part of I/I mitigation efforts could vary significantly. As mentioned previously, the City should begin implementing an I/I reduction program that targets the largest sources of I/I. These can be addressed more economically than lining and replacing services as summarized in the descriptions of Alternatives 3.1 and 3.2.

TABLE 5-5: COLLECTION AND TREATMENT I/I SUMMARY

Item	Alternative 2	Alternative 3.1	Alternative 3.2
Collection System Improvements	$ 14,260,000	$ 24,050,000	$ 19,660,000
WWTP Savings	$ -	$ 227,000	$ 506,000
Total Capital Costs	$ 14,260,000	$ 23,830,000	$ 19,160,000

CHAPTER 6 - PROPOSED PROJECT (RECOMMENDED ALTERNATIVE)

This chapter consists of the recommended improvements to address the wastewater treatment and collection system deficiencies. The chapter also includes a description of the preliminary project design and schedule, anticipated permit requirements, sustainability considerations, cost estimates, and annual operating budgets. A location map showing the changes to the wastewater treatment plant is included in Figure 15 (Appendix A).

6.1. Preliminary Project Design

Detailed project summary sheets are included in Appendix K. Each project summary sheet provides the objective, key issues, cost estimate, and project location map. The project improvements are summarized in the sections below.

6.1.1. Collection System Preliminary Project Design

This section discusses the preliminary project design of the recommended alternative.

- CS.1.1 – Lift Station and Force Main Improvements: Capacity improvements in a collection system should begin at the most downstream location in the system. For the City's system, this includes the two existing lift stations and forcemain. This project will consist of replacing the Washington Street and E Street Lift Stations as well as installing a parallel force main from each lift station to the WWTP. The project assumes complete replacement of the lift station including the wetwell, mechanical equipment, electrical equipment, and controls. Additionally, a parallel 12-inch force main should be constructed from each lift station to the WWTP. The lift station design flows are summarized in Table 6-1.

  The pump selection should consider both high and low flow pumping scenarios. This may result in a 3 to 4 pump configuration with one smaller pump for low flow and larger pumps to convey the peak flows. The improvements should also consider only installing pumps to convey existing peak instantaneous flows with provisions for additional pumping capacity if flows increase with new development. A phased approach provides the City with flexibility if their I/I reduction efforts reduce flows and future PIF is not ever reached. The specific pumping selection and operation should be assessed during the predesign phase of the project.

TABLE 6-1: CS1.1 PRELIMINARY DESIGN FLOWS

Lift Station Name	Existing PIF (gpm)[1]	Future PIF (gpm)[1]
E Street	1,400	2,060
Washington Street	1,890	1,960

1) Based on unconstrained flows (i.e., no upstream surcharging or flooding).

- I/I Reduction Program: This project does not consist of a single capital project, but is a recommended program to target I/I reduction within the system. The general goals of the program are outline below in Table 6-2. In general, it is recommended that the City set aside $159,000/year for I/I mitigation efforts.

The first general step in this program is to begin an active CCTV program to document the condition of the gravity pipes and identify large sources of inflow. The City could complete this in-house or contract a third party to complete the inspections. The City could consider implementing a CCTV program that rates the condition of pipe based on the National Association of Sewer Service Companies (NASSCO) Pipe Assessment and Certification Program (PACP) which is a standardized approach that scores pipes based on defects and conditions such as cracks, corrosion, deformation, and structural integrity. This requires a certified inspector to review the CCTV footage. Results from the CCTV analysis can be used to develop a pipeline and manhole lining program. It was assumed that all AC pipes would be lined within the next 50 years because the CCTV analysis was not completed at the time of this study. In addition to I/I reduction efforts on the mainlines, it is also recommended to complete smoke testing in the systems every 10 years. Smoke testing was completed on the majority of the existing system as a part of this facility plan and identified several items which could be addressed to reduce I/I. These items include notifying property owners regarding open/broken cleanouts and leaking sewer laterals. Also, two cross connections with the storm system were identified and should be removed so stormwater does not enter the sewer system. Flow monitoring should be completed as needed to track the progress of I/I reduction activities. Flow monitors can be placed in the system during the wet season to isolate basins and locate which sewer lines have more I/I.

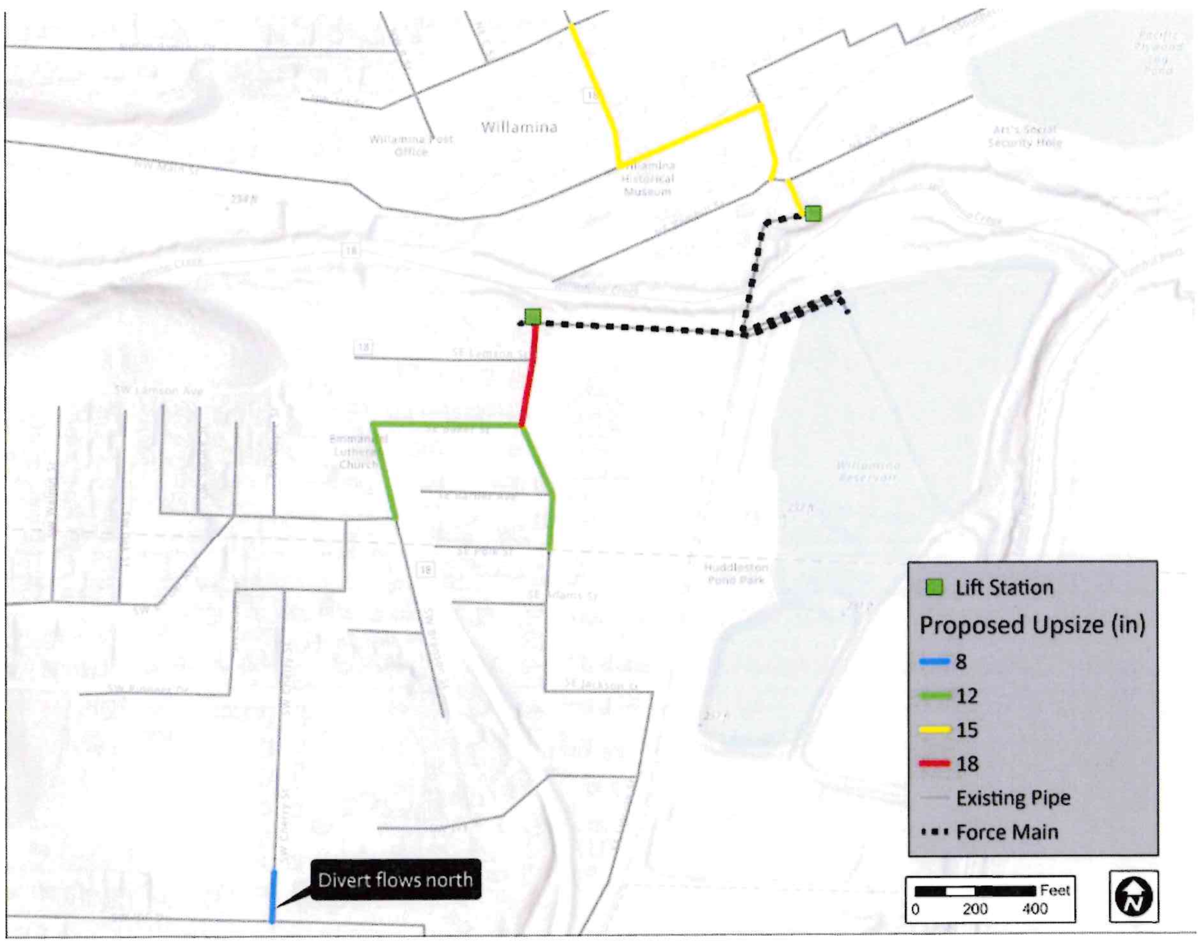
TABLE 6-2: I/I REDUCTION PROGRAM OUTLINE

	Frequency	Target Quantity	Cost
CCTV	Every year (20% annually)	10,000 LF / Year	$30,000
Smoke Testing	Every 10 years (10% annually)	20 manholes / year or entire system every 10 years	$2,000
Flow Monitoring	As needed; After completion of CIPs	Assumed every 5 years	$2,000
Cured-in-Place Pipe Lining or Replace	As needed based on CCTV inspection	Line or replace all AC Pipe within 50-years as needed, (1000 LF per year)	$100,000
Manhole Lining	As needed based on CCTV inspection	Line or replace all AC Manholes within 50-years as needed (5 Manholes per year)	$25,000
Spot Repairs	As needed based on CCTV inspection	As needed	-
		Target Annual I/I Program Funding	**$159,000**

- CS.2.1 – Upsize Gravity Trunklines: This project consists of upsizing the gravity sewer pipes so that no flooding is shown in the model during the existing peak day flows. Note, the proposed pipes are sized for future flow conditions. This project consists of 400 LF of 8-inch pipe, 1,200 LF of 12-inch pipe, 1,300 LF of 15-inch pipe, and 400 LF of 18-inch pipe, and is illustrated in Figure 6-1.

  The extent of the gravity pipe upgrades should be re-assessed as a part of the predesign phase of the project. Additional flow data from the lift stations and WWTP should be available with the improvements which can be used to confirm the recommended pipe upsizing in this facility plan. The City's future I/I reduction efforts are anticipated to reduce overall flows and may result in less pipe upsizing being required. For the sake of this capital improvement plan, it was assumed there was no reduction in the flows.

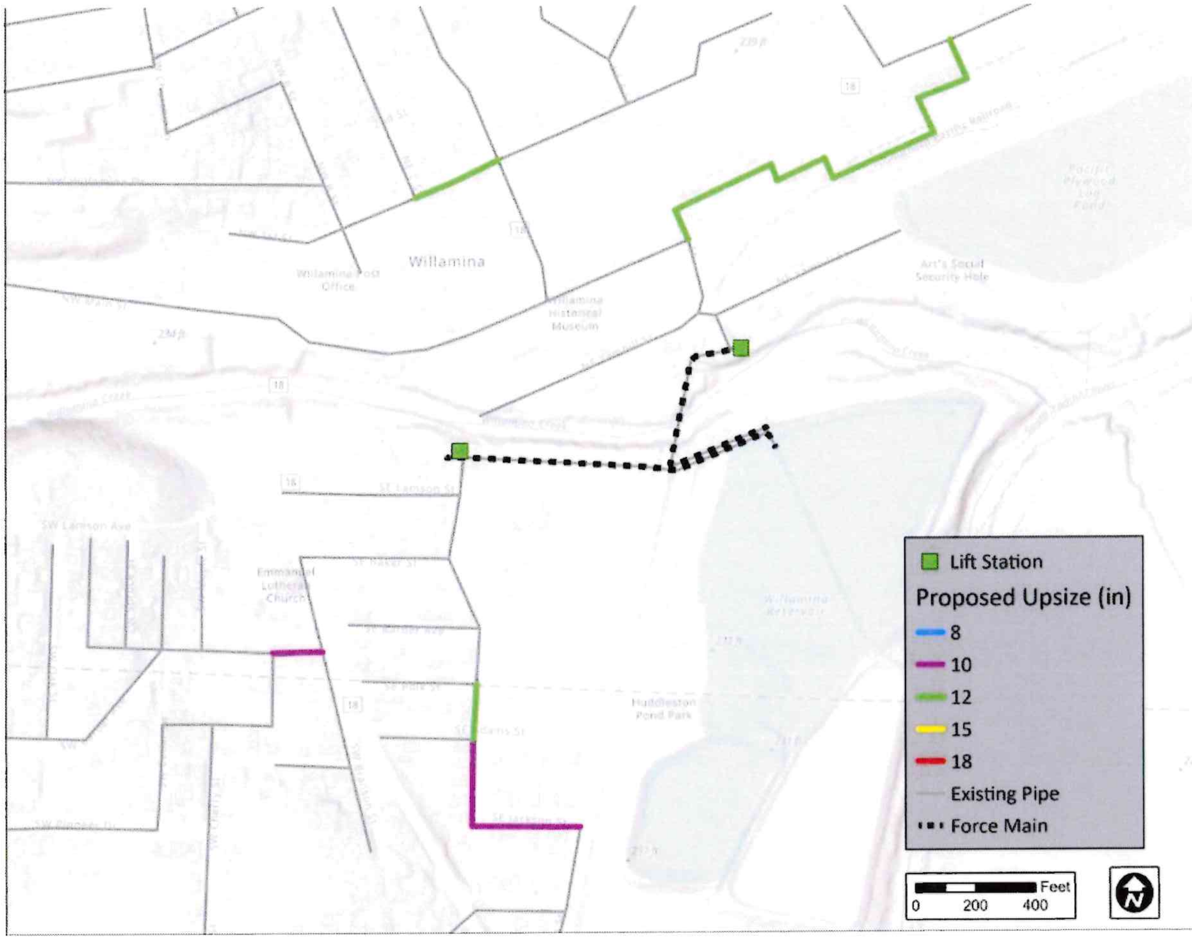
FIGURE 6-1: CS.2.1 PROJECT EXTENT



- CS.3.1 – Additional Gravity Pipe Upsizing: This project consists of additional gravity pipe upsizing to eliminate surcharging in the system under future flows. This project was assumed to consist of 1,200 LF of 10-inch pipe and 1663 LF of 12-inch pipe, and is illustrated in Figure 6-2.

  Similar to CS.2.1, the extent of the upsizing should be re-evaluated after the lift stations are replaced and after I/I reduction activities have been implemented. For the sake of this capital improvement plan, it was assumed there was no reduction in the flows.

FIGURE 6-2: CS.3.1 PROJECT EXTENT



6.1.2. WWTP Preliminary Project Design

In summary, the following modifications are recommended to address the WWTP deficiencies:

- T.1.1 – Headworks Improvements: This project consists of replacing the existing headworks to provide a new mechanical and manual bar screen, increasing the size of the headworks channel and diversion structure, and upsizing the transfer piping from the headworks to the lagoons to increase pipe capacity and allow for proper lagoon operation in series during high flow events. It is recommended the influent flow meters be replaced and properly calibrated.
- T.1.2 – New Storage Lagoon and Pump Station: This project focuses on increasing the available storage during the non-discharge period by constructing a fifth lagoon and pump station. As discussed in Chapter 4, the footprint of the lagoon will be within the available land at the WWTP providing approximately 10 MG of storage. The existing pump station used to transfer water from Lagoon #3 to Lagoon #4 will be reconstructed to have appropriate pumps and valving to transport water to the new lagoon as well.
- T.1.3 – Aeration System and Blowers: To continue to meet permit requirements with the increased flows and loadings, additional aeration is needed within the two existing aerated lagoons. To achieve this, the existing aeration system, including the blowers and mechanical components, will be replaced and additional diffusers will be added.

- <u>T.1.4 – Chlorine Disinfection and Contact Basin:</u> The purpose of this project is to increase the capacity of the disinfection system by constructing a new basin to achieve longer contact times during higher flow events and resolve deficiencies related to the existing sodium hypochlorite generator. The new chlorine contact basin will be a buried concrete vault in the area north of Lagoon #4. A new pump station will also be constructed to pump up to the contact basin. This allows for the contact basin to be higher in elevation than it is currently to achieve greater separation from the 100-year South Yamhill River flood elevation.
- <u>T.1.5 – Discharge Piping to the Outfall:</u> This project is to address deficiencies identified in Chapter 3 by replacing the existing effluent piping to the outfall to increase the hydraulic capacity and replace the existing effluent flow meter.
- <u>T.1.6 – Miscellaneous WWTP Improvements:</u> The purpose of this project is to address components of the WWTP that have deficiencies to improve overall operations. This includes upsizing the transfer piping between lagoons for adequate capacity, SCADA improvements, backup power at the WWTP, lagoon depth measurement devices, irrigation spray guns, and effluent flow meter.
- <u>T.2.1 – Combine Lagoons #3 and #4:</u> The water balance and storage analysis resulted in a storage deficiency of approximately 17 MG. Based on the evaluation, the construction of Lagoon #5 will not result in sufficient capacity. Combining the two existing lagoons was determined to be a Priority 2 project to allow for the City to monitor flows and water levels within the lagoons during the non-discharge period to assess the additional storage required. If additional storage is required, Lagoons #3 and #4 can be combined to achieve approximately 10 MG of additional storage and meet the 2045 storage requirements.
- <u>T.2.2 – Lagoon Liner Replacement:</u> This project addresses the deficiencies identified with the existing lagoon liners. In addition to the deficiencies with the liner in Lagoons #1 and #3, lagoon liners should be replaced after 20-30 years.
- <u>T.3.1 – Facility Planning Study Update:</u> The purpose of this project is to re-evaluate the WWTP and collection system after completing the Priority 1 and 2 improvements for the impact on I/I.

6.2. PROJECT SCHEDULE

The specific schedule for each project will be determined by the City during the predesign phase of each proposed improvement. Table 6-3 presents an estimated schedule for CIP projects for the next six years. Actual costs may vary depending on market conditions and shall be updated as projects are further refined in the pre-design and design phases.

TABLE 6-3: 6-YEAR CAPITAL IMPROVEMENT PLAN

ID#	ITEM	COST	Opinion of Probable Costs (2024 Dollars)					
			2025	2026	2027	2028	2029	2030
CS.1.1	Lift Station Improvements \ Forcemain	$ 7,355,000		$ 1,103,250	$ 3,125,875	$ 3,125,875		
T.1.1	Headworks Improvements	$ 1,448,000				$ 724,000	$ 724,000	
T.1.2	Storage Lagoon	$ 2,883,000			$ 1,441,500	$ 1,441,500		
T.1.3	Aeration System and Blowers	$ 3,767,000				$ 565,050	$ 1,600,975	$ 1,600,975
T.1.4	Disinfection System and Chlorine Contact Basin	$ 1,453,000		$ 726,500	$ 726,500			
T.1.5	Discharge Piping to Outfall	$ 449,000		$ 449,000				
T.1.6	Micellaneous Plant Priority 1 Improvements	$ 1,131,000						$ 1,131,000
	Total Capital Cost	*$ 18,486,000*	*$ -*	*$ 2,279,000*	*$ 5,294,000*	*$ 5,857,000*	*$ 2,325,000*	*$ 2,732,000*
	I/I Reduction Program	-	$159,000	$159,000	$159,000	$159,000	$159,000	$159,000
	Short-Lived Asset Replacement	-	$124,000	$124,000	$124,000	$124,000	$124,000	$124,000
	Total FY Cost	-	*$ 283,000*	*$ 2,562,000*	*$ 5,577,000*	*$ 6,140,000*	*$ 2,608,000*	*$ 3,015,000*

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 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 2024 dollars and does not include escalation to time of actual construction.

6.3. PERMIT REQUIREMENTS

The City's NPDES discharge permit was recently renewed (went into effect on November 1st, 2021) without many changes. A permit modification to the NPDES permit is not anticipated for the improvements noted in this facility planning study.

The City's NPDES permit, (in addition to the Influent, Effluent, and Recycled Water Monitoring Reports), included details on the following items:

- *Annual Inflow and Infiltration Report* – Details of activities performed during the past year and activities planned for the coming year.
- *Sludge Depth Survey* – Determine the levels of sludge in the lagoons and compare the design sludge depth to the actual sludge depth.
- *Industrial User Survey* – Determine the presence of any industrial users that are subject to pretreatment.
- *Chlorine Study Plan* – Develop and submit an implementation plan for a total chlorine residual chlorine study to determine the relationship between the chlorine concentrations in the samples collected at the monitoring point and the concentrations at (or near) the point of discharge to the South Yamhill River.
- *Emergency Response and Public Notification Plan* – Ensures the contact information for the applicable public agencies is accessible and up to date.
- *Quality Assurance and Quality Control (QA/QC) Program* – If not already developed, the City must create a QA/QC program to verify the accuracy of the sample analysis.
- *Hauled Waste Control Plan* – Describes the waste received by the publicly owned treatment works, if applicable.
- *Biosolids Management Plan* – Before distributing biosolids to the public, the permittee must develop and maintain a Biosolids Management Plan and Land Application Plan

Refer to the NPDES Permit for additional information on these items.

Collection system improvements will also likely require some permits to be acquired prior to construction. Primarily CS.1.1 which includes the force main crossing Willamina Creek and the railroad. Permits from Army Corps of Engineer (404 Permit), Oregon Department of State and Land, and local city and county permits should be acquired as part of the pre-design process.

6.4. SUSTAINABILITY CONSIDERATIONS

6.4.1. Water and Energy Efficiency

Lagoon improvements and the construction of additional storage will result in more balanced flows and improve treatment reliability and quality. The proposed alternatives also include incorporation of a SCADA system. This allows for better system resiliency and operation simplicity, as well as improved system optimization. Overall, the proposed projects seek to be environmentally conscious, economically feasible, and socially beneficial. Replacing and relining the collection system reduces the risk of leaks or seepage, which protects groundwater sources. The new lift stations will also utilize energy-efficient pumps and motors with VFDs.

6.4.2. Green Infrastructure

The following were considered and recommended for implementing green infrastructure.

- Construct upgrades at the existing lift stations to include pervious surfaces such as gravel, native plants, or pervious pavers.

6.5. TOTAL PROJECT COST ESTIMATE (ENGINEER'S OPINION OF PROBABLE COST)

Costs shown are planning-level estimates and can vary depending on market conditions; they shall be updated as the project is further refined in the pre-design and design phases. Appendix K contains separate summary sheets for each capital improvement. Table 6-4 shows costs with the percent SDC eligibility, which represents the increased capacity required for future connections. Treatment project SDC eligibility is estimated as a function of existing and future design flows. For the SCADA improvements, which are not associated with an increase in flows, the percent SDC eligible is derived from the percent growth in population over the 20-year planning period. Lift station growth was estimated based on current and projected EDUs in the sewer basin of interest. Existing SDCs are summarized in Chapter 2.

TABLE 6-4: 20-YEAR CAPITAL IMPROVEMENT PLAN

Project ID#	Project Name	Total Estimated Cost (2024)	SDC Growth Apportionment		City's Estimated Portion
Total Priority 1 Improvements (0-6 years)					
CS.1.1	Lift Station Improvement \ Forcemain	$7,355,000	18%	$1,336,000	$6,019,000
T.1.1	Headworks Improvements	$1,448,000	21%	$299,000	$1,149,000
T.1.2	Lagoon 5	$2,883,000	18%	$528,000	$2,355,000
T.1.3	Aeration System and Blowers	$3,767,000	21%	$784,000	$2,983,000
T.1.4	Disinfection System and Chlorine Contact Basin	$1,453,000	21%	$300,000	$1,153,000
T.1.5	Discharge Piping to Outfall	$449,000	18%	$82,000	$367,000
T.1.6	Miscellaneous Plant Priority 1 Improvements	$1,131,000	21%	$232,000	$899,000
	Total Priority 1 Improvements (rounded)	**$18,486,000**	**-**	**$3,561,000**	**$14,925,000**
Total Priority 2 (6-13 years)					
CS.2.1	Upsizing Gravity Trunklines	$4,447,000	21%	$914,000	$3,533,000
T.2.1	Combine Lagoon 3 & 4	$3,887,000	21%	$799,000	$3,088,000
T.2.2	Lagoon Liner Improvements	$3,460,000	21%	$711,000	$2,749,000
	Total Priority 2 Improvements (rounded)	**$11,794,000**	**-**	**$2,424,000**	**$9,370,000**
Total Priority 3 Improvements (13-20 years)					
CS.3.1	Upsizing Gravity Mains	$2,704,000	21%	$556,000	$2,148,000
T.3.1	Facility Planning Study Update	$150,000	21%	$31,000	$119,000
	Total Priority 3 Improvements (rounded)	**$2,854,000**	**-**	**$587,000**	**$2,267,000**
	TOTAL SYSTEM IMPROVEMENTS COSTS (ROUNDED)	**$33,134,000**	**-**	**$6,572,000**	**$26,562,000**

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 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 2024 dollars and does not include escalation to time of actual construction.

6.6. ANNUAL OPERATING BUDGET

An itemized annual operating budget for the fiscal year 2020-2021 is provided in Appendix J. Additional information on budget specifics can be found in the following sections.

6.7. INCOME

6.7.1. Revenue

- Potential User Rate Impacts

The existing sewer rate schedule consists of a monthly flat rate fee of $70.46 per equivalent dwelling unit (EDU). The Sewer Operating Fund generates approximately $1,565,000 in revenue for use to offset short-term asset replacement and O&M costs. The portion of the existing budget that can be used for capital improvement projects varies from year to year. With this in mind, the rate impacts assume that none of the existing revenue/budget can be used annually to offset future capital improvements.

Table 6-5 shows the existing and potential charges for sewer services every month for one EDU. The user rate impacts can vary depending on the amount of SDC funds available, as shown in the table. City staff periodically update a Capital Improvement Plan (CIP) for wastewater treatment, pump stations, and collection systems.

Funding for the recommended system improvements may come from any number of sources. This section presents potential user rate impacts if priority improvements are funded only through a low interest loan with debt service payments (30 year, 1.5%) made through a user rate increase. The amounts shown in the table also assume that there is no surplus in the annual budget contributing to the annual debt service payment. Grant funds, lower interest loans, or principal forgiveness may also be available which could further lessen the user rate impacts shown in Table 6-5. Keller Associates recommends that the City actively pursue these opportunities that would mitigate user rate impacts. A separate user rate study is recommended to complete a more detailed evaluation of potential user rate impacts.

TABLE 6-5: USER RATE IMPACT

	Annual Payment (30 year, 1.5%)	Monthly User Rate without SDCs	Monthly User Rate Including SDCs
Existing User Rates (2023)	-	$81.03	$81.03
Priority 1 Improvements[1]	$769,742	$159.26	$101.87
1) Assumes $10,000,000 in grants are secured.			

It should be noted that all costs are in 2024 dollars, and that the City should plan on annual increases in user rates of 2-5% to account for cost-of-living adjustments. This table assumes $10,000,000 of grant funds are secured to offset the amount of costs that the city is expected to have to make to account for the current deficits in the system.

- System Development Charge

The scope of this study included estimating the SDC eligibility for each identified capital improvement. The estimated SDC eligibility for each identified capital improvement is shown in Table 6-5.

6.8. ANNUAL OPERATIONS AND MAINTENANCE COSTS

The City of Willamina adopted budget document for fiscal year 2023 allocates an operations and maintenance (O&M) budget of $791,500 for the sewer fund.

6.9. DEBT REPAYMENTS

The Wastewater Fund pays for expenditures for the City's sewer collection, treatment, and disposal system, including daily operations, maintenance, regulatory compliance, facility expansion, replacement, and capital reserves. The primary revenue source for the Wastewater Fund is user's fees. The 2022 fiscal year adopted sewer budget is presented in Table 2-5.

6.10. RESERVES

6.10.1. Debt Service Reserve

There is currently no outstanding debt for the City of Willamina wastewater department.

6.10.2. Short-Lived Asset Reserve

A table of short-lived assets is shown in Table 6-6 and long-lived assets are shown in Table 6-7. This table shows an idealized replacement budget which accounts for replacement of existing infrastructure based on its assumed useful life. Fully funding this replacement budget would provide the City with the necessary funding to replace infrastructure as it reaches the end of its useful life. Fully funding an asset replacement program may not be feasible for the City. It is recommended, at a minimum, that the City begin by funding the short-lived assets at the lift stations and WWTP to have adequate funds to replace lift station and WWTP assets. Asset replacement for equipment with a longer life can be ramped up over time as the system ages, and replacement is more needed. CIP lining of pipes associated with the I/I reduction activities will extend the useful life of the AC piping, essentially acting as a pipeline replacement program.

TABLE 6-6: SHORT-LIVED ASSET REPLACEMENT

Short Lived Assets	Quantity	Unit	Unit Cost	Total Replacement Cost	Typical Useful Life / Frequency (years)	Annualized Replacement Cost
Washington Lift Station						
Submersible Pump & Motor (200hp)	3	EA	$115,000	$350,000	20	$17,500
Routine Pump Inspection	1	LS	$3,000	$3,000	5	$600
Impeller Replacement	3	EA	$10,000	$30,000	10	$3,000
Instrumentation / HVAC	1	LS	$100,000	$100,000	15	$6,700
Control Panel & Electrical	1	LS	$100,000	$100,000	15	$6,700
			Total River Lift Station	**$583,000**	-	**$34,500**
E Street Lift Station						
Submersible Pump & Motor (200 hp)	3	EA	$115,000	$350,000	20	$17,500
Routine Pump Inspection	1	LS	$3,000	$3,000	5	$600
Impeller Replacement	3	EA	$10,000	$30,000	10	$3,000
Instrumentation / HVAC	1	LS	$100,000	$100,000	15	$6,700
Control Panel & Electrical	1	LS	$100,000	$100,000	15	$6,700
			Total Spring Street Lift Station	**$583,000**	-	**$34,500**
Wastewater Treatment Plant						
Headworks Screens	2	EA	$50,000	$100,000	10	$10,000
Lagoons Blowers	2	EA	$75,000	$150,000	20	$7,500
Transfer Pumps	2	EA	$10,000	$20,000	15	$1,400
Chlorination/Dechlorination pumps	2	EA	$10,000	$20,000	10	$2,000
Instrumentation / HVAC	1	LS	$100,000	$100,000	15	$6,700
Control Panel & Electrical	1	LS	$100,000	$100,000	15	$6,700
			Total Wastewater Treatment Plant	**$490,000**	-	**$34,300**
			Total Material Costs	**$1,166,000**	-	**$103,300**
				Subtotal	-	**$103,300**
				Contingency	20%	$20,700
				Total Construction Cost	-	**$124,000**
				Total Short-Lived Asset Replacement (rounded)	-	$124,000

TABLE 6-7: LONG LIFE ASSET REPLACEMENT

Long Lived Assets	Quantity	Unit	Unit Cost	Total Replacement Cost	Typical Useful Life (years)	Annualized Replacement Cost
Pipelines / Cleanouts						
8" Diameter Gravity Sewer Pipe Fully Installed (PVC)	42,521	LF	$330	$14,030,000	75	$188,000
10" Diameter Gravity Sewer Pipe Fully Installed (PVC)	2,115	LF	$350	$740,000	75	$10,000
12" Diameter Gravity Sewer Pipe Fully Installed (PVC)	2,872	LF	$390	$1,120,000	75	$15,000
15" Diameter Gravity Sewer Pipe Fully Installed (PVC)	1,396	LF	$410	$570,000	75	$8,000
18" Diameter Gravity Sewer Pipe Fully Installed (PVC)	377	LF	$450	$170,000	75	$3,000
36" Diameter Gravity Sewer Pipe Fully Installed (PVC)	47	LF	$480	$20,000	75	$1,000
12" Diameter Force Main Sewer Pipe Fully Installed (PVC)	2,000	LF	$350	$700,000	75	$10,000
Manhole	209	EA	$12,000	$2,510,000	50	$51,000
Total Pipelines / Cleanouts				**$19,860,000**	-	**$286,000**
Washington Lift Station						
Valves / Meters	1	LS	$140,000	$48,000	30	$1,600
Onsite Diesel Generator (70kW)	1	EA	$70,000	$50,000	30	$1,700
Site paving, fencing, landscaping, etc.	1	LS	$100,000	$10,000	30	$400
Building	1	LS	$384,000	$380,000	40	$9,500
Total River Lift Station				**$488,000**	-	**$13,200**
E Street Lift Station						
Valves / Meters	1	LS	$140,000	$140,000	30	$4,700
Onsite Diesel Generator (70kW)	1	EA	$70,000	$70,000	30	$2,400
Site paving, fencing, landscaping, etc.	1	LS	$100,000	$100,000	30	$3,400
Building	1	LS	$384,000	$380,000	40	$9,500
Total Spring Street Lift Station				**$690,000**	-	**$20,000**
Total Collection System Replacement Costs				**$21,038,000**		**$319,200**
Mobilization					10%	$31,920
Subtotal					-	**$351,120**
Contingency					20%	$70,200
Total Construction Cost					-	**$421,300**
Engineering					20%	$84,300
Total Long-Lived Asset Replacement (rounded)					-	**$506,000**

6.10.3. Financing Options

Financing and incentive options that may assist with offsetting costs associated with implementing the CIP include, but are not limited to user rate increases, SDCs, DEQ State Revolving Fund Loan Program, Oregon Infrastructure Finance Authority grants and loans, USDA Rural Utilities Services loans and grants, direct state loans or appropriations, revenue bonds, general obligation bonds, US Economic Development Administration grants, and Energy Trust of Oregon.

A "One-Stop" funding meeting is recommended for the City where funding packages can be developed using the various funding sources described below:

- Oregon Department of Environmental Quality (Clean Water State Revolving Fund).
- Oregon Economics and Community Development Department (Community Development Block Grant Program). Availability is dependent on the median household income and user rates. Priority given to cities with compliance infractions.
- U.S. Department of Agriculture (Rural Development Program). Grant and loans available to communities with less than 10,000 people. Eligibility based on user rates, average household income, and compliance issues.
- U.S. Economic Development Administration. Grant and loan funds available based on economic development potential.
- Oregon Economics and Community Development Department (Water/Wastewater Financing Program). State funded program (Oregon Lottery). Grant and loan funds are generally provided on a 50/50 basis. Eligibility based on average household income and compliance issues.

- Oregon Economics and Community Development Department (Special Public Works Program). State funded program (Oregon Lottery). Loan funds only. Eligibility based on average household income and compliance issues.

CHAPTER 7 - CONCLUSIONS AND RECOMMENDATIONS

The City Council adopted the Willamina WWFPS on July 9, 2024. The Capital Improvement Plan presented in Chapter 6 outlines the recommended improvements for the wastewater system.

7.1. OTHER CONSIDERATIONS

A geotechnical analysis may be required in certain areas when construction begins due to unknown field conditions. Depending on project costs, a value engineering study may also be needed to improve the project during the predesign phase. It is also recommended that the City updates its mixing zone study when any projects may impact the flow out of the outfall.

Further flow monitoring should occur at the WWTP. This will assist in the design of the proposed Lagoon #5 and evaluate the success of the I/I reduction program.