Scappoose Oregon

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Collection System Facilities Planning Study

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City Ordinance #895

KELLER ASSOCIATES

CITY OF SCAPPOOSE COLLECTION SYSTEM FACILITIES PLANNING STUDY UPDATE

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PROJECT NO. 219123

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ACRONYMS, ABBREVIATIONS, AND SELECTED DEFINITIONS

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

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

EXECUTIVE SUMMARY

In 2019, the City of Scappoose, Oregon, contracted with Keller Associates, Inc. to complete a collection system facilities planning study update (CSFPS) for the City's sanitary sewer collection system. A facilities plan update for the wastewater treatment plant (WWTP) was completed in 2018 (2018 FPU, Carollo). This section summarizes the major findings of the CSPFS, including brief discussions of alternatives considered and final recommendations.

ES.1 PLANNING CRITERIA

Regulatory requirements, engineering best practices, and City-defined goals and objective form the basis for planning and design. The City's conveyance system will be sized for the projected 20-year peak instantaneous flow rates associated with the 5-year, 24-hour storm event. Pipes will be considered full and trigger improvements when the maximum depth exceeds two-thirds (2/3) of the full depth of the pipe in accordance with the City Design Standards. When sizing gravity collection systems, pipelines will be sized according to the City standards; pipe size shall be determined by using two-thirds (2/3) of the full depth of the pipe. Sewage pump stations will be designed to handle these flows with the largest pump out of service (defined as firm capacity).

ES.2 PLANNING CONDITIONS

ES.2.1 Study Area and Land Use

The study area coincides with the updated urban growth boundary (UGB) and is shown in Figure 1 (Appendix A). The wastewater system currently serves only those areas within the UGB. Build-out of the vacant properties within the UGB is not expected to occur during the current 20-year planning period. Keller Associates recommends that future development within the UGB provide adequate conveyance for connection of the upstream sewer basins.

ES.2.2 Demographics

The City's population has been increasing at an unsteady rate over the past few decades. Historical populations were obtained from records of the County and Portland State University (PSU). This study will use the population projections developed as part of the 2018 FPU, which are based on the population data and projections completed by collaborative efforts of the County and Portland State University (PSU). Table ES.1 shows the 2017 and 2035 populations from the FPU. The 2040 population projection used the 2035 population as a starting point for projections. The 2040 projection was calculated using an average annual growth rate (AAGR) of 1.2% from the 2017 PSU Coordinated Population Forecast document.

TABLE ES.1: POPULATION AND PROJECTIONS

Year	Population	Source
2017	7,610	Facilities Plan Update (Carollo, Mar 2018)
2035	10,461	Facilities Plan Update (Carollo, Mar 2018)
2040	11,104	Projected Using AAGR of 1.2%

Note: Coordinated Growth Rates (AAGR) from PSU Coordinated Population Forecast 2017-2067 Columbia County.

Future land use and flow projections for the 20-year planning period were developed using criteria and methodology based in the 2018 FPU (Chapter 2.6-2.8). The City's Community Development Center (CDC), which includes the planning and engineering departments, provided locations within the City UGB that are anticipated to be developed during the 20-year planning period. These future growth areas, with respective City zoning designation, are presented in Figure 6. Existing, buildout, 20-yr growth, and 2040 acreages by land use type are summarized in Table ES.2

TABLE ES.2: EXISTING, BUILDOUT AND 2040 LAND USE

Land Use Designation[1]	Existing Acreage (ac)[1]	Buildout Acreage (ac)[1]	20-Yr Growth Acreage (ac)	2040 Acreage[2]
Airport Employment	201	518	158	359
Commercial	171	179	4	175
General Residential	323	176	-	323
Industrial	23	267	122	145
Manufactured Home	72	86	7	79
Public Lands	18	262	122	140
Suburban Residential	290	868	289	579
Total	***1098***	***2356***	***703***	***1,800***

[1]2018 FPU (Table 2.5)

[2]Adjusted to reflect unconstrained acreages from 2017 Housing Needs Analysis

The 2017 Housing Needs Analysis (HNA) was used to adjust the buildout acreages from the FPU to more accurately represent the acreage estimated with associated 20-year growth as these estimates remove constrained land due to wetlands, floodplain, and steep slopes. Flow factors (estimated average wastewater generated from land use types) from the FPU were used as a basis and adjusted to account for the HNA land constraint adjustments. Further discussion of the revised projections is provided in Section 1.3.

ES.2.3 Wastewater Flows

The wastewater flow analysis from the 2018 FPU was used to provide design flows and flow projections for the planning period. Flows for 2040 were projected from the 2035 flows using the population growth summarized in the previous section and unit flows based on the 2018 FPU. The FPU used the methods recommended by the Oregon DEQ in "Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon" and historical data to develop design flows in the City's system (summarized in Section 1.4).

As mentioned previously, flow projections for the 20-year planning period were developed using criteria and methodology based in the 2018 FPU (Chapter 2.6-2.8). Design flows and projected 2035 flows from the FPU were used as the basis for projected 2040 flows. Population growth, 20-year growth acreages, and large customer projections (as described in Section 1.3) were used to develop 2040 projected system flows. Table ES.3 shows the design and flow projections for the 20-year planning period.

TABLE ES.3: DESIGN AND PROJECTED FLOWS (MGD)

	Design Flow (MGD)[1]	Projected Unit Flow (gpcd)	Projected Flows (MGD)[2]	
Year	2017	2017	2035	2040
Population	7,610	7,610	10,461	11,104
ADWF	0.646	85	0.956	1.01
AWWF	0.878	115	1.30	1.37
$MMDWF_{10}$	0.871	114	1.22	1.29
$MMWWF_5$	1.200	158	1.68	1.78
$PDAF_5$	2.770	364	3.88	4.11
PIF_5	3.96	520	5.56	5.89

1. Design flows and 2035 projected flows are from 2018 Scappoose Facilities Plan Update.

2. 2035 projected flows are from 2018 FPU and 2040 projected flows based on design flows and population projections.

ES.3 COLLECTION SYSTEM EVALUATION

ES.3.1 Pump Station Evaluation

There are seven pump stations in the Scappoose collection system and one that services the Miller WTP (see Figure 7 for locations). Smith Road Pump Station is the largest; with a triplex system and an existing capacity of 1,100 gpm per pump. It pumps through approximately 1,600 feet of 12-inch force main. The other six pump stations are each similarly-sized, smaller, duplex systems. Spring Lake has a firm capacity of 160 gpm; Keys Landing has a firm capacity of 120 gpm; both Highway 30 and Seven Oaks have firm capacities of 150 gpm; Charles T Parker has a firm capacity of 398 gpm; and East Airport has a firm capacity of 336 gpm. A general inventory of the pumping facilities was completed and is summarized in Chapter 2. Several issues were identified at each of the lift stations; recommendations to correct these issues are summarized in Chapter 2.

ES.3.2 Pipeline Condition and Capacity Evaluation

Scappoose's gravity collection system includes approximately 32 miles of gravity pipelines ranging from 4- to 30-inches in diameter and approx. 1.4 miles of force mains (Figure 7). Currently, approximately half of all flow goes through the Smith Road Pump Station to reach the WWTP.

Approximately half of the collection system pipes are concrete, and half are PVC. These material types provide a good indication of the age of the sewer lines; with concrete pipes generally installed from the 1930s into the 1980s, and PVC pipe installed thereafter. There are also a couple of ductile iron, cast iron, and unknown material sections throughout town. Improvements and additions to the original wastewater collection system have been completed throughout its lifespan. The City's current system includes approximately 700 manholes.

Portions of the system were smoke tested to identify sources of infiltration and inflow (I/I) in 2016. These portions tested, as well as problem locations found, are summarized in Appendix C.

A GIS-based computer model (InfoSWMM Suite 14.7, Update #1) of the collection system was built and exercised to evaluate capacities of the system's trunk lines. Model results (Figures 11 and 12) were used to determine capacity-related problems that exist in the system for the design criteria and flow scenarios analyzed. Several areas of town experience flows that exceed the existing capacity of the pipelines during storm events.

ES.4 COLLECTION SYSTEM IMPROVEMENTS

The following summarizes the alternatives and recommended improvement projects to the City's collection system:

ES.4.1 Collection System Alternatives

Three collection system alternatives were considered to address capacity deficiencies in the Smith Road Pump Station Basin: Alternative 1, which includes continuing Keys WTP operations during daytime and upsize existing trunk lines; and Alternative 2, which includes shifting Keys WTP operations to nighttime, low sewer flows and upsize existing trunk lines necessary; Alternative 3, which includes installing a new trunk line across town along SW Maple Street and rehabilitating existing lines parallel to South Scappoose Creek. All of these improvements also include a new pipeline north of Veterans Park that would allow the City to abandon the South Scappoose Creek crossing on the north side of the park (referred to as the Kucera line) as well as upsizing existing pipes on Smith Road.

In addition to the alternatives described above to address capacity limitations in the Smith Road Pump Station Basin, improvements to address the remaining capacity limitations in the system were evaluated. Other alternatives reviewed for these limitations were found to be not feasible or cost prohibitive. Improvements to address deficiencies include upsizing the existing pipelines in the following locations.

- High School Way Basin trunk line
- Laurel and 3rd Streets
- Tyler Street
- Wagner Court
- Miller WTP Improvements
- Columbia Avenue at the WWTP

ES.4.3 Recommended Collection System Improvements

Alternative 2 was evaluated to have the lowest cost of the alternatives and was selected as the preferred alternative to address deficiencies in the Smith Road Pump Station Basin. The Columbia Avenue pipe at the WWTP was excluded from the CIP because the project is included in the Priority 2 projects of the 2018 FPU. The location and extents of all recommended improvements are summarized in the collection system Capital Improvement Plan (CIP) (Figure 15). Priority 1 improvements are intended to correct existing deficiencies and are recommended to be completed as soon as funding allows; Priority 2 improvements address future growth.

ES.5 CAPITAL IMPROVEMENT PLAN

ES.5.1 Summary of Opinion of Probable Costs

Table ES.4 presents the 20-year CIP. Projects are organized by priority. Opinion of probable costs reflect planning-level estimates and should be refined in the subsequent pre-design and design phases of implementation. Priority 1 improvement expenses are anticipated to occur over the 4-5 years. Priority 2 are primarily triggered by growth.

TABLE ES.4: 20-YEAR CAPITAL IMPROVEMENT PLAN[1]

ID #	Item	Total Estimated Cost (2020)	SDC Growth Portion %	SDC Growth Portion Cost	City's Estimated Portion
Priority 1 Improvements					
1.1	Smith Road Pump Station	$ 883,000	24%	$ 215,000	$ 668,000
1.2	WTP Improvements and Smith Rd	$ 908,000	40%	$ 364,000	$ 544,000
1.3	Laurel and 3rd St	$ 594,000	18%	$ 109,000	$ 485,000
1.4	Old Portland Rd	$ 1,398,000	10%	$ 145,000	$ 1,253,000
1.5	High School Wy to Elm St	$ 1,412,000	12%	$ 165,000	$ 1,247,000
1.6	Dutch Canyon Rd	$ 285,000	2%	$ 5,000	$ 280,000
1.7	Pump Station Improvements (Springlake and HWY 30 PS)	$ 84,000	36%	$ 30,000	$ 54,000
Total Priority 1 Improvements (rounded)		***$ 5,564,000***		***$ 1,033,000***	***$ 4,531,000***
Rate Impact (20 yr, 1.6%)					***$ 8.70***
Priority 2 Improvements					
2.1	4th St to Smith Rd Pump Station	$ 958,000	13%	$ 126,000	$ 832,000
2.2	Tyler St	$ 303,000	13%	$ 40,000	$ 263,000
2.3	Wagner Ct	$ 126,000	100%	$ 126,000	$ -
2.4	Pump Station Improvements (Keys Landing and Seven Oaks PS)	$ 149,000	36%	$ 53,000	$ 96,000
2.5	Miller WTP Improvements	$ 49,000	36%	$ 17,000	$ 32,000
Total Priority 2 Improvements (rounded)		***$ 1,585,000***		***$ 362,000***	***$ 1,223,000***
Rate Impact (20 yr, 1.6%)					***$ 2.35***
TOTAL COLLECTION SYS. IMPROVEMENTS COSTS (rounded)		**$ 7,149,000**			**$ 5,754,000**

Notes:

1. The opinion of probable cost herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2020 dollars and does not include escalation to time of actual construction. Keller Associates has no control over variances in the cost of labor, materials, equipment, services provided by others, contractor's methods of determining prices, competitive bidding or market conditions, practices or bidding strategies. Keller Associates cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the cost presented herein.
2. All costs in 2020 Dollars. Costs include mobilization (5%), contractor overhead and profit (OHP; 15%), contingency (30%), engineering and construction management services (CMS; 20-30%), and legal, administrative, and permitting services (2%).
3. The Capital Improvement Plan does not include annual pipeline replacement, pipeline cleaning and inspection, and lift station maintenance budgets. These budgets are discussed in Section 6.6.

ES.5.2 Budget and Rate Impacts

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 through a low-interest loan with debt service payments (20 year, 1.6%) made through a user rate increase. Table ES.5 outlines the potential residential user rate impacts and assumes a flat rate increase to all 2,553 sewer EDUs. Actual rate impacts will vary depending on the City's rate structure, available system development charge (SDC) funds, existing budget surplus, funding source(s), potential grants, project phasing, and terms of the loan. A separate user rate study may be warranted to complete a more detailed evaluation of potential user rate impacts.

TABLE ES.5: POTENTIAL USER RATE IMPACT

	Annual Payment (20 years, 1.6%)	User Rate Increase	User Rate Total
Existing User Rate	$ -	$ -	$ 55.96
Prioritiy 1 Improvements	$ 266,520	$ 8.70	$ 64.66

ES.5.3 Other Annual Costs

In addition to the capital improvement costs presented in the CIP, Keller Associates recommends the following for consideration in setting annual budgets:

- Collection system replacement/rehabilitation needs: City should eventually budget an additional $717,500/year (to be either contracted out or completed using City crews).
- Collection system cleaning and CCTV needs: following the timeline described in Section 6.1.2, the City should budget approximately $112,000/year.

ES.5.4 SDCs

The City's current sewer System Development Charge (SDC) for a single-family home is $4,276.04. The sewer SDC is typically divided into two components: reimbursement and growth. The scope of this study included estimating the SDC eligibility for each identified capital improvement. It is the intent that this information will be utilized by the City's financial consultant to update the City's SDCs. The estimated SDC eligibility for each identified capital improvement is shown in Table ES.4.

ES.5.5 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, revenue bonds, general obligation bonds, US Economic Development Administration grants, and Energy Trust of Oregon.

CHAPTER 1 – PROJECT PLANNING

The City of Scappoose owns and operates a municipal sewage collection system and a wastewater treatment plant (WWTP). The City had a Facilities Plan Update (FPU) for the treatment process and the WWTP completed in 2018. The purpose of this collection system facilities planning study (CSFPS) is to assess the needs of the City for the wastewater collection system, evaluate if the existing pipe network and lift stations can meet those needs, and provide a long-term plan to implement improvements to the collection system so the needs of the City can be met. The CSFPS will coordinate relevant planning criteria with the FPU, including population and flow projections, to maintain consistency in the City's planning for the entire wastewater system.

This CSFPS describes the conditions, flows, and problems in the existing collection system; analyzes the hydraulic flow data; and provides recommendations for improvements to the collection system over the 20-year planning period.

1.1 LOCATION

The study area consists of all areas within the City of Scappoose Urban Growth Boundary (UGB). Figures 1 and 2 in Appendix A show the land use, the existing service areas, the topography, and the floodplains in the study area. The study area slopes generally to the east toward the WWTP and eventually the Multnomah Channel. Sections of the western side of the City slope toward the South Scappoose Creek, which is a tributary of the Multnomah Channel.

1.2 ENVIRONMENTAL RESOURCES PRESENT

An inventory of the existing environmental resources is summarized below, which will be used to consider the environmental impacts of alternatives. The factors analyzed in this section include land use, prime farmland, and formally classified lands; earthquake hazards; floodplains; wetlands; cultural, biological, water, and coastal resources; and socio-economic conditions.

1.2.1 Land Use/Important Farmland/Formally Classified Land

Land use in the study area is shown in Figure 1 (Appendix A). Figure 3 in Appendix A shows the County soils shapefile farmland designation in the area. The majority of the city is designated by the National Resources Conservation Service (NRCS) as Prime Farmland, although it is currently zoned and used for other purposes.

1.2.2 Earthquake Hazards

Figure 4 (Appendix A) illustrates the relative earthquake hazard map produced by the Oregon Department of Geology and Mineral Industries for the Saint Helens-Columbia City-Scappoose Urban Area. This map shows one Zone A (highest hazard) area on the west side of Scappoose. There are areas of Zone B (intermediate to high hazard) and Zone C (low to intermediate hazard) determinations surrounding the Zone A area.

1.2.3 Floodplains

The Federal Emergency Management Agency (FEMA) publishes flood insurance studies that classify land into different flood zone designations. As illustrated in Figure 2 (Appendix A), some portions of the study area are located inside the 100-year and 500-year floodplains of Scappoose Creek and Jackson Creek. The WWTP is just outside the FEMA mapped 100-year floodplain.

1.2.4 Wetlands

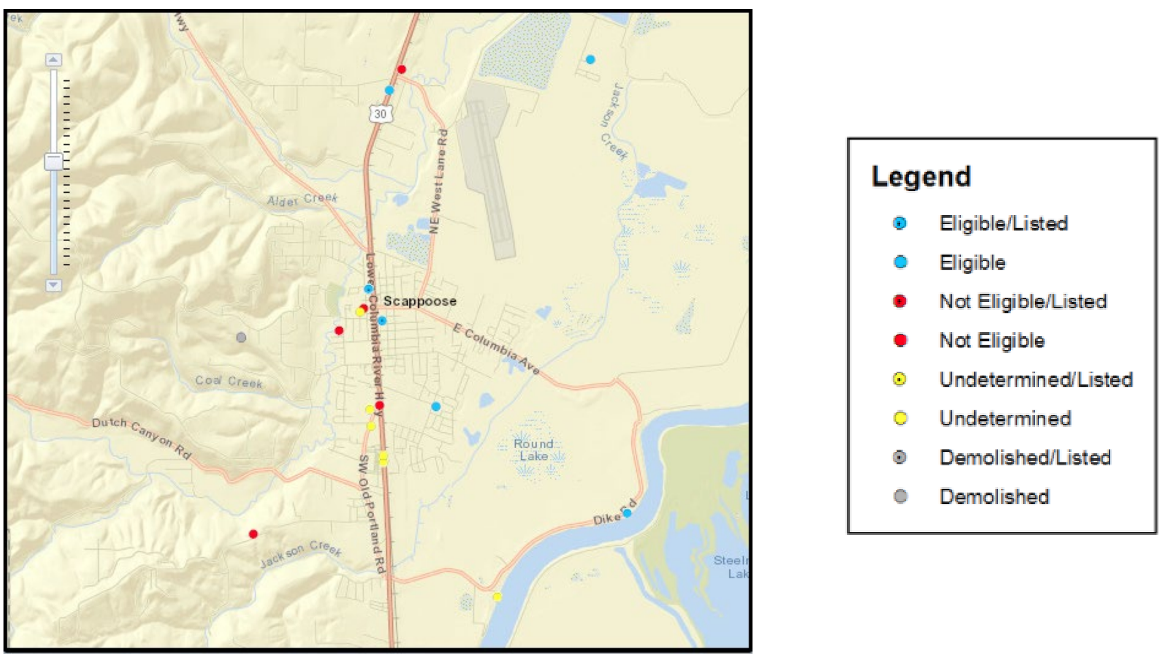
The Oregon Department of State Lands (ODSL) keeps an inventory of local wetland areas in Oregon. The City of Scappoose has a completed Local Wetland Inventory from December 1998. Wetland delineation was not within the scope of this project, so the ODLS Local Wetland Inventory and the U.S. Fish and Wildlife National Wetlands Inventory (Figure 5, Appendix A) were used to determine wetland areas that could potentially be impacted. There are a number of small wetland areas around Scappoose Creek and the eastern edge of the City.

1.2.5 Cultural Resources

The State Historic Preservation Office (SHPO) maps above-ground cultural resources on their website. According to the SHPO website, there are eleven structures listed as “eligible”, and five structures listed as “undetermined”, cultural resources within the UGB. The map from the SHPO website is shown as Chart 1-1.

The SHPO also keeps track of underground cultural resources. They only provide information from their database to professional archaeologists, with one exception: They will provide information for small project areas if given the complete legal description of the project location, a United States Geological Survey (USGS) map of the project area, and a description of the project and ground disturbance. The SHPO should be consulted as part of the design process of any proposed recommendation.

CHART 1.1: ABOVE-GROUND CULTURAL RESOURCES



1.2.6 Biological Resources

The Pacific Northwest Interagency Special Status / Sensitive Species Program lists the endangered, threatened, and sensitive species for the state and county by Bureau of Land Management (BLM) district. The City of Scappoose lies within the Salem BLM District. Endangered species in the district include the fender’s blue butterfly, Columbian white-tailed deer, Bradshaw’s desert parsley, and Willamette Valley daisy. The fish in the Salem district that are listed as sensitive or threatened include the cutthroat trout (vulnerable), chum salmon (critical), coho salmon (endangered), steelhead (critical), and chinook salmon (critical).

1.2.6 Water Resources

North and South Scappoose Creeks flow together into Scappoose Creek in the study area. The creek ends in Scappoose Bay, which subsequently joins the Multnomah Channel north of the City. The WWTP outfalls into the Multnomah Channel. As of the most recent listing in 2012, the Multnomah Channel is 303(d) listed by DEQ for dissolved oxygen, mercury, and temperature. There are no wild or scenic rivers in the study area.

1.2.7 Coastal Resources

There are no coastal areas within the study area.

1.2.8 Socio-Economic Conditions

The population in the area is primarily (91%) Caucasian, according to the 2010 census. Hispanics make up 5% of the population. The median household income is $79,401 (in 2018 dollars), which is 33% higher than the state average. The improvements will provide mutual benefit to all sanitary sewer customers and improve the overall economic vitality of the area.

1.2.9 Miscellaneous Issues

Other environmental resources considered were air quality and soils. Scappoose is not located in an area designated as an air maintenance or nonattainment area by DEQ. A soils map is provided in Figure 3 (Appendix A); soils in the area are generally silt loam.

1.3 POPULATION TRENDS AND 20-YEAR GROWTH

This study will use the population projections developed as part of the 2018 FPU, which are based on the population data and projections completed by collaborative efforts of the County and Portland State University (PSU). Table 1-1 shows the 2017 and 2035 populations from the FPU. The 2040 population projection used the 2035 population as a starting point for projections. The 2040 projection was calculated using an average annual growth rate (AAGR) of 1.2% from the 2017 PSU Coordinated Population Forecast document.

TABLE 1.1: POPULATION AND PROJECTIONS

Year	Population	Source
2017	7,610	Facilities Plan Update (Carollo, Mar 2018)
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2040	11,104	Projected Using AAGR of 1.2%

Note: Coordinated Growth Rates (AAGR) from PSU Coordinated Population Forecast 2017-2067 Columbia County.

Future land use and flow projections for the 20-year planning period were developed using criteria and methodology based in the 2018 FPU (Chapter 2.6-2.8). The FPU estimates the City will reach buildout conditions in 2067 and provides land use acreages for buildout. Linear interpolation was used to estimate the land use acreages for 2040. The 2017 Housing Needs Analysis (HNA) was used to adjust the buildout acreages from the FPU to more accurately represent the acreage estimated with associated 20-year growth as these estimates remove constrained land due to wetlands, floodplain, and steep slopes. Flow factors (estimated average wastewater generated from land use types) from the FPU were used as a basis and adjusted to account for the HNA land constraint adjustments. Existing, buildout, 20-yr growth, and 2040 acreages by land use type are summarized in Table 1.2. The associated flow factors, flow from 20-year growth, and 2040 flow projections by land use type are summarized in Table 1.3. Estimated locations of 20-year growth areas were provided by the City Planning Department and are shown on Figure 6.

TABLE 1.2 EXISTING, BUILDOUT AND 2040 LAND USE

Land Use Designation[1]	Existing Acreage (ac)[1]	Buildout Acreage (ac)[1]	20-Yr Growth Acreage (ac)	2040 Acreage[2]
Airport Employment	201	518	158	359
Commercial	171	179	4	175
General Residential	323	176	-	323
Industrial	23	267	122	145
Manufactured Home	72	86	7	79
Public Lands	18	262	122	140
Suburban Residential	290	868	289	579
Total	***1098***	***2356***	***703***	***1,800***

[1]2018 FPU (Table 2.5)

[2]Adjusted to reflect unconstrained acreages from 2017 Housing Needs Analysis

TABLE 1.3 20-YEAR GROWTH FLOW PROJECTIONS

Land Use Designation[1]	20-Yr Growth Acreage (ac)	2040 Acreage[2]	Flow Factor (gpad)[2]	Flow from Growth (mgd)	Flow from Growth (gpm)	2040 System Flow (mgd)
Airport Employment	158	359	90	0.014	10	0.03
Commercial	4	175	950	0.004	3	0.17
General Residential	-	323	810	0.010	7	0.27
Industrial	122	145	1,200	0.147	102	0.17
Manufactured Home	7	79	780	0.010	7	0.07
Public Lands	122	140	-	-	-	-
Suburban Residential	289	579	727	0.130	90	0.26
Total	***703***	***1,800***		***0.314***	***218***	***0.97***

[1]2018 FPU (Table 2.5)

[2]Adjusted to reflect unconstrained acreages from 2017 Housing Needs Analysis

In addition to the 20-year growth acreages described above, large customer projections were identified in the 2018 FPU. Projections for these customers were developed individually based on the type of development. The total ADWF from these customers is estimated at 0.14 MGD. Customers and estimated flows are summarized in Table 1.4. The City identified the locations of these customers and are reflected in Figure 6 growth areas.

TABLE 1.4 LARGE CUSTOMER PROJECTIONS

Large Customer	# of students/ employee/...	Flow Unit	ADWF (gpd)	ADWF (gpm)
Community College	500 students (next 2 years)	15 gpd/student	7,500	5
	1500 students (by 2035)	15 gpd/student	22,500	16
50 acres of Hotels	200 rooms	150 gpd/room	30,000	21
Metallurgy R&D - 10 acres	50 employees	30 gpd/employee	1,500	1
42 acres of light industrial	-	900 gpad	37,800	26
50 acres of light/heavy industrial	-	900 gpad	45,000	31

Source: 2018 FPU, Table 2.6

1.4 WASTEWATER FLOWS

The wastewater flow analysis from the 2018 FPU was used to provide design flows and flow projections for the planning period. Flows for 2040 were projected from the 2035 flows using the population growth summarized in the previous section and unit flows based on the 2018 FPU. The FPU used the methods recommended by the Oregon DEQ in “Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon” and historical data to develop design flows in the City’s system (summarized in Table 1-6). The design flows used in this study are defined in the following sections. Additional details of the flow analysis can be found in Chapter 1.5 of the FPU.

1.4.1 Average Dry-Weather Flow (ADWF)

The average dry-weather flow (ADWF) is the average daily flow for the period of May through October.

1.4.2 Average Wet-Weather Flow (AWWF)

The AWWF is the average daily flow for the period encompassing November through May.

1.4.3 Maximum Month Average Dry-Weather Flow ($MMDWF_{10}$)

The DEQ defines the $MMDWF_{10}$ as the maximum monthly average dry weather flow with a 10% probability of occurrence. This typically represents the rainiest summer month of high groundwater. Since Oregon DEQ states that May is typically the maximum month for the dry-weather period of May through October, selecting the May 90% precipitation exceedance most likely corresponds to the maximum month during the dry-weather period for a 10-year event.

1.4.4 Maximum Monthly Average Wet-Weather Flow ($MMWWF_5$)

The $MMWWF_5$ represents the highest monthly average during the winter period of high groundwater. Since Oregon DEQ states that January is typically the maximum month for the wet-weather period of January through April, selecting the January 80% precipitation exceedance most likely corresponds to the maximum month during the wet-weather period for a 5-year event.

1.4.5 Peak Day Flow ($PDAF_5$)

The peak day flow ($PDAF_5$) represents the maximum daily average flow associated with a 5-year storm event.

1.4.6 Peak Instantaneous Flow (PIF_5)

The peak instantaneous flow (PIF_5) is defined by the DEQ as the peak hour flow sustained for one-hour during a 5-year storm event.

1.4.7 Infiltration and Inflow (I/I)

Infiltration and inflow (I/I) is storm water or groundwater that enters the sanitary sewer system. I/I can come from a variety of sources, such as storm sewers connected to the sanitary sewer, storm inflow through manhole lids, and groundwater infiltration into cracked/broken pipelines and services. The peak hour flow compared to the base flow (average dry weather flow) is an indication of I/I influence in the system. The system-wide peaking factor for peak day flow to base flows is 4.56 (2018 FPU). I/I exists in the system and contributes to high peak hour flows. Some communities in western Oregon experience peak flows in excess of 10 times the base flow.

The City does not have a formal program to remove I/I; instead, pipeline lining and replacement projects are completed as budget allows. Large defects discovered during CCTV inspection are given priority. New, future construction should experience less I/I due to the use of newer and more watertight sewer components and improved construction practices. It is recommended the City continue to monitor, identify, and seek to reduce/eliminate sources of I/I in the system.

1.4.8 Projected Flows

As mentioned previously, flow projections for the 20-year planning period were developed using criteria and methodology based in the 2018 FPU (Chapter 2.6-2.8). Design flows and projected 2035 flows from the FPU were used as the basis for projected 2040 flows. Population growth, 20-year growth acreages, and large customer projections (as described in section 1.3) were used to develop 2040 projected system flows. These flows are summarized in Table 1.5.

TABLE 1.5: DESIGN AND PROJECTED FLOWS (MGD)

	Design Flow (MGD)[1]	Projected Unit Flow (gpcd)	Projected Flows (MGD)[2]	
Year	2017	2017	2035	2040
Population	7,610	7,610	10,461	11,104
ADWF	0.646	85	0.956	1.01
AWWF	0.878	115	1.30	1.37
$MMDWF_{10}$	0.871	114	1.22	1.29
$MMWWF_5$	1.200	158	1.68	1.78
$PDAF_5$	2.770	364	3.88	4.11
PIF_5	3.96	520	5.56	5.89

1. Design flows and 2035 projected flows are from 2018 Scappoose Facilities Plan Update.
2. 2035 projected flows are from 2018 FPU and 2040 projected flows based on design flows and population projections.

1.5 PLANNING CRITERIA

The City's conveyance system will be sized for the projected 20-year peak instantaneous flow rates associated with the 5-year, 24-hour storm event. It should be noted, efforts to reduce I/I in the collection system could further extend the service population. Pipes will be considered full and trigger improvements when the maximum depth exceeds two-thirds (2/3) of the full depth of the pipe in accordance with the City Design Standards. When sizing gravity collection systems, pipelines will be sized according to the City standards; pipe size shall be determined by using two-thirds (2/3) of the full depth of the pipe. Sewage pump stations will be designed to handle these flows with the largest pump out of service (defined as firm capacity).

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

1.6 COMMUNITY ENGAGEMENT

The Scappoose community had the opportunity to engage in the planning process by participating in City Council meetings. The final CSFPS was presented to the City Council for approval and adoption.

CHAPTER 2 – EXISTING COLLECTION SYSTEM FACILITIES

This chapter contains a description and evaluation of the existing wastewater collection system, including pump stations and pipelines, for the City of Scappoose.

2.1 LOCATION MAP

Maps of the existing collection system are included in Figures 7 and 8 (Appendix A). The wastewater facilities are all located within the City limits.

2.2 HISTORY

The majority of the collection system was constructed in 1972 of concrete pipe. Most of the pipe installed since that time has been polyvinyl chloride (PVC). There are small segments of ductile iron and cast iron pipes in the system.

2.3 SYSTEM DESCRIPTION

The wastewater collection system consists of approximately 32 miles of gravity mains, 1.4 miles of force mains, and seven pump stations that discharge to the wastewater treatment plant (WWTP). The pipelines range from 4 to 30 inches in diameter. There are approximately 700 manholes in the City's collection system. Figure 7 (Appendix A) illustrates the pipe diameters and Figure 8 illustrates the pipe material in the City's collection system. Currently, over half of the City's wastewater is pumped at least once prior to the WWTP headworks.

2.4 PUMP STATIONS

There are seven pump stations in the Scappoose collection system. See Figure 7 for locations of the pump stations. The Smith Road Pump Station is the largest of the pump stations. The remaining six pump stations are smaller. Keller Associates visited five of the pump station sites in 2015 and reviewed record drawings of all pump stations to complete a general inventory of facilities. Pump station inventories are summarized in Table 2-1. Appendix B includes available data such as pump curves, design parameters, and other data resources. More detailed discussions of each pump station follow.

TABLE 2.1 – PUMP STATION INVENTORY

	Smith Road	Spring Lake	Keys Landing	Highway 30	Seven Oaks	Charles T Parker	East Airport
PUMP STATION							
Type	Wet-well, dry-well, triplex pump system	Wet-well, submersile, duplex pump system	Wet-well, surface-mounted, duplex pump system	Wet-well, surface-mounted, duplex pump system	Wet-well, surface-mounted, duplex pump system	Wet-well, submersible, duplex pump system	Wet-well, submersible, duplex pump system
Pump Type	Vertical, variable speed, non-clog centrifugal (Crane Deming model 7196-4x4x12x3)	Submersible, constant speed, centrifugal (Wilo Model FK17.1-6/8KEx)	Horizontal, constant speed, self-priming centrifugal (Hydromatic model #183)	Horizontal, constant speed, self-priming centrifugal (Hydromatic model #183)	Horizontal, constant speed, self-priming centrifugal (Hydromatic model #183)	Submersible, constant speed, centrifugal (Flygt Model NP3127HT)	Submersible, constant speed, centrifugal (Flygt NP3127HT)
Capacity* (gpm)	Each pump: 1,100 gpm @ approx. 28 ft. TDH	Each pump: 160 gpm @ approx. 26 ft. TDH	Each pump: 120 gpm @ approx. 46 ft. TDH (with 13 ft. suction lift)	Each pump: 150 gpm @ approx. 42 ft. TDH (with 19 ft. suction lift)	Each pump: 150 gpm @ approx. 45 ft. TDH (with 16 ft. suction lift)	Each pump: 398 gpm @ approx. 55 ft. TDH	Each pump: 336 gpm @ approx. 59 ft. TDH
Pump HP (each)	15hp @ 1200 rpm (460V, 60 Hz, 3 ph)	2.85hp @ 1140 rpm (230V, 60 Hz, 1 ph)	5hp @ 1750 rpm (230V, 60 Hz, 3 ph)	7.5hp @ 1800 rpm (230V, 60 Hz, 3 ph)	7.5hp @ 1750 rpm (230V, 60 Hz, 3 ph)	11hp @ 1720 rpm (480V, 60 Hz, 3 ph)	10hp @ 1750 rpm (480V, 60 Hz, 3 ph)
Level Control Type	Pressure level transducer in wet well	Pressure transducer with backup high level float	Four (4) Mercury level sensors	Ultrasonic transducer, backup floats	Four (4) Mercury level sensors	Pressure transducer, backup floats	Ultrasonic transducer, backup floats
Overflow Point	Existing manhole just west of pump station	At pump station	At pump station	At pump station	At MH just north of pump station	At pump station	At pump station
Overflow Discharge	To creek west of pump station	To Spring Lake	To residences near pump station	To storm system at HWY 30 and then into Jackson Creek	To residences near pump station	To road	To road
Auxiliary Power Type	Permanent diesel generator	Portable generator	Portable generator	Portable generator	Portable generator	Permanent diesel generator	Permanent diesel generator
Location	inside pump station	At WWTP	At WWTP	At WWTP	At WWTP	at pump station	at pump station
Output	60 kW	17 kW	17 kW	17 kW	17 kW	50 kW	50 kW
Fuel Tank Capacity (gal)	120	100	100	100	100	260	260
Transfer Switch	Automatic	Manual	Manual	Manual	Manual	Automatic	Automatic
Alarm Telemetry Type	Autodialer and red light outside	Autodialer	Autodialer and red light outside	Autodialer and red light outside	Autodialer and red light outside	Autodialer	Autodialer
Originally Constructed	1972	1993	1996	1996	1996	2019	2019
Year(s) Upgraded	2001	2001 and 2003, 2020	NA	NA	NA	NA	NA
Wet Well Diameter (ft)	6	7	6	8	7	6	8
Wet Well Net Storage (gal)	634	432	528	2,237	1,315	5,075	8,600
FORCE MAIN							
Length, Type	Approx. 1,600 ft. of 12-inch Ductile Iron	Approx. 1,726 ft of 4-inch PVC	Approx. 550 ft of 4-inch PVC	Approx. 250 ft of 6-inch C900	Approx. 1,565 ft. of 4-inch C900 PVC	Approx. 1,915 ft. of 4- & 6-inch C900 PVC	Approx. 3,955 ft. of 6- & 10-inch C900 PVC
Profile, Continuously Ascending (Yes/No)	Yes	Yes	Yes	Yes	No	No	No
Discharge Location	MH at intersection of Burlington Northern RR and Laurel Street	MH at 6th Street and Seven Oaks Drive	MH on SW Keys Road at end of SW Keys Landing Way	MH on SW Dutch Canyon Road off Columbia River HWY	MH at 6th Street and Seven Oaks Drive	MH at Wagner Court	MH at Crown Zellerbach Rd and Bird Road
Air Release Valves	None	None	None	Yes	Yes	Yes	Yes
Vacuum Release Valves	None	None	None	Yes	Yes	Yes	Yes
Sulfide Control System	None	None	Yes	Yes	Yes	None	None

*Capacity as reported in O&M Manuals

Smith Road Pump Station

The Smith Road Pump Station is the largest pump station in the system and is located near the intersection of NW 1st Street and EJ Smith Road. The pump station is fully fenced and consists of a 6-foot diameter wet well with a pressure level sensor and a high level float switch, a dry well with three 15 HP vertical mounted centrifugal pumps, a float switch to alarm if there is flooding in the dry well, and a sump pump to remove any water that collects in this area. Confined space entry is required to enter the dry well. In a separate masonry structure is a pump station control panel and a permanent standby generator with automatic transfer switch. The station was upgraded from a duplex to a triplex pumping station in 2001. Three new pumps, a level sensing device, control panels, a permanent stand-by power generator and a concrete structure to house the generator were installed during the upgrade. The SCADA system, Mission Controls, sends alarms to the operators. Also, a red-light alarm is located on the outside of the dry well (though this has not been tested and City staff do not know if it is operational).

Smith Road Pump Station

The pumps operate in a duty-duty-standby configuration and discharge through a 12-inch force main. The pumps rotate through duty-duty-standby operation so all three pumps are run regularly and equally. The pump motors are soft start and controlled by the level in the wet well as measured by the pressure level sensor. The operator can adjust the level settings in the pump station programmable logic controller (PLC) for the pump on and off settings. Alarms are activated by the pressure level sensor readings for low and high wet well levels. A float switch activates an overflow alarm. The City has tested the level sensor, floats, and alarms and they are all functioning properly. The pumps are nearly 20 years old and are in need of replacement. The City plans to replace the pumps in the next year.

Keller Associates analyzed the pump run time data from the past three years to assess capacity. If the pump station runs two pumps for more than 20 hours per day, or if it cannot pass the peak instantaneous flow with one pump offline, the pump station is considered to be undersized. During the last three years, the maximum individual pump run time was 19.6 hours/day. Based on model results, the current peak flow through Smith Road PS during a 5-year, 24-hour storm is 1,500 gpm. The firm capacity of the pump station should be able to handle the peak flow. Each pump in the pump station is rated for 1,100 gpm, the firm capacity of the Smith Road PS is stated as 2,200 gpm in the Operation and Maintenance (O&M) manual. Pump tests should be completed to verify the firm capacity of the pump station. Based on the information provided, the pump station is nearly at capacity for current peak design flows. Pump run time should continue to be watched and the City may consider beginning design of capacity upgrades now, so they are complete for pump replacement in the next year or two.

The operational volume of the pump station is approximately 425 gallons. The duty pump runs for approximately 1-2 minutes under normal flow conditions to empty the operational volume. During peak flow days, the pumps have recorded a total of 200 starts per day. This number of starts can cause the motors to overheat and indicates that the wet well operating volume is inadequate. The City should evaluate options to increase operating volume or decrease the starts of pumps with variable frequency drives (VFD) to alleviate this strain on the pumps. The pump runs approximately once an hour during the lowest flows of the day.

The pump station is located within the mapped 100-year flood plain. However, record drawings show the elevation of the top of the dry well as approximately 0.3 feet above the 100-year flood elevation (recorded as 46.66 feet in the record drawings). The elevation of the wet well rim is not shown on the record drawings. A survey should be completed to confirm the existing pump station access is indeed above the 100-year base flood elevation.

According to City Staff, the pump station has overflowed in the past due to grease buildup. The pump station overflows to a nearby manhole at the east end of Laurel Street, which consequently overflows out the top of the manhole and into the street. Since the manhole is near Scappoose Creek, an overflow could negatively impact the creek's water quality.

A number of deficiencies were noticed with the Smith Road Pump Station. These deficiencies and recommendations to correct them are provided below.

Deficiencies

- Pumps are nearing end of useful life.
- Small operating volume. High number of pump starts per day.
- Pump station is nearing capacity.
- Access hatch is heavily corroded.
- Confined space entry required to access and maintain pump and motor equipment.
- Grease builds up.
    - Heavy grease ring in wet well.
- No pre-screening.
- Exterior red light and audible alarms have not been tested.
- One of the blowers for the fan in the dry well does not work.
- Bypass is missing cap for cam fitting.

Recommendations

With all the deficiencies listed above, it is recommended the pump station be completely replaced or have major upgrades including converting to a submersible pump station.

Spring Lake Pump Station

The Spring Lake Pump Station is located near the intersection of Westlake Drive and SE 6th Street. The pump station was recently upgraded in 2020. The pump station consists of the wet well with pressure transducer level control, backup high-level float switch, two submersible pumps, and a control panel. In 2020, pumps, power transfer switch, and controls were replaced as well as a new valve vault.

The gravity lines draining to the pump station are all private lines except one 8-inch diameter line. The pumps are controlled by the pressure transducer using a lead on, lag on, and pump off operational strategy. There is a high-level float as a backup alarm. In the event of a power outage, an alarm is sent to the operator via Mission Controls. The pump station has a manual power transfer switch with receptacle. In the event of a power failure, a 17 kW portable generator, normally housed at the WWTP, can be transported to the site and connected to the receptacle for backup power.

Spring Lake Pump Station

The O&M manual states that the pump capacity is 160 gpm. In the three-year pump run time history analyzed (2012-2015), the maximum pump run time in one day was 13.8 hours. This indicates the pump station is likely not undersized for the current demand. There have been no known issues with the pump station overflowing or with pumps running continuously for an extended period of time. The pump station overflow discharges directly into Spring Lake. Spring Lake and any in-stream activities immediately downstream would be negatively affected, if an overflow occurred.

The pump station has historically not operated well and alarms are consistently sent to City staff. Since the new pumps have been installed the number of alarms sent to operators has decreased. Grease buildup and rag issues in the wet well still occur. City staff have the grease removed from the surface of the wet well every three months. City staff also report heavy paper products in the wet well. City believes this comes from the east inlet line (private line) that may have bellies or sags, which cause buildups and can overwhelm the wet well when released to pump station. The new pumps have been handling the flow fine without issue since installation.

During this study, City staff noted that the force main has a high point prior to discharging to the gravity collection system. Keller Associates staff recommended testing the pump station for a potential air lock scenario occurring in the force main, which could cause the pumps to operate outside of their normal efficiencies. Keller Associates recommends performing these tests and installing an air release/vacuum valve if air locks are a problem. If future work is completed on the force main, changing the discharge location to 6th and Fredrick Streets to shorten the force main length and reduce TDH should be evaluated.

A number of deficiencies were noticed with the Spring Lake Pump Station. These deficiencies and recommendations to correct them are provided below.

<u>Deficiencies</u>

- Possibility for air lock in force main.
- Grease builds up in the wet well.
- Heavy paper products in influent to pump station.
- No, permanent backup power.

<u>Recommendations</u>

- Perform test(s) to assess whether there is air lock in the pump station force main. Install an air release valve at the high point of the force main profile.
- Establish pre-treatment program to prevent grease from reaching the collection system.
- Perform a review of the electrical system and check that all equipment meets compliance standards.
- Inspect the east inlet line for bellies or sags and recommend rehabilitation as necessary.

Keys Landing Pump Station

Keys Landing Pump Station

The Keys Landing Pump Station is located near the intersection of Keys Landing Way and SW Keys Road. The pump station is a package unit and was constructed in 1996. The pump station consists of the wet well with four float switches; two surface-mounted, 5 HP, horizontal centrifugal pumps; and a control panel. The station has a pinch valve that drains the discharge line after it is done pumping. The wet well is wider at the base than the top. Both pumps were rebuilt in 2014.

The pumps are controlled by the float switches using a lead on, lag on, and pump off operational strategy. The top-most float switch is used to trigger a high-level alarm. Mission Controls sends alarms to the operator. A red alarm light is also located on the outside of the pump station, though it has not been tested recently. In the event of a power outage, an alarm is sent via Mission Controls.. The pump station has a manual power switch with a receptacle. In the event of a power failure, a 17 kW portable generator, normally housed at the WWTP, can be transported to the site and connected to the receptacle for backup power. City staff has indicated that they have ample time to respond during power outages, and the system model results suggest it would take hours before an overflow would occur. There have been no known issues with the pump station overflowing or with pumps running continually for an extended period of time. In the three-year pump run time history analyzed (2012-2015), the maximum run time was 2 hours a day. This indicates the pump station has adequate capacity.

City exercises the pumps every other week and this has reduced the number of alarms the pump station sends. The second pump has had a history of losing prime. The O&M manual states that the pump capacity is 120 gpm. There have been some odor complaints for this pump station. The pump station overflow, at the nearest gravity manhole, is currently plugged to prevent odors. City staff report that due to site conditions (icy roads, etc.), it can be difficult to get access to the pump station in the winter to hook up the portable generator in the case of a power outage. The pump station is located relatively close to the Keys Landing Water Treatment Plant (WTP). City staff have discussed using stand-by power available at the WTP for this pump station.

A number of deficiencies were noticed with the Keys Landing Pump Station. These deficiencies and recommendations to correct them are summarized below.

Deficiencies

- Occasional odor concerns in summer.
- Overflow is plugged to alleviate odor concerns.
- Heavy corrosion on wet well piping.
- The second pump does not stay primed.
- Difficult to bring portable generator to site, particularly in winter.
- Clamshell door does not stay open on its own; needs to be propped up.

Recommendations

- Address odor control to allow for opening overflow.
- Clean and coat piping in wet well and valve vaults as necessary.
- Change operation to alternate between pumps more frequently to balance run times.
- Assess feasibility to coordinate with WTP to get stand-by power for the pump station.
- Repair clamshell door so that it remains open without support.
- Perform a review of the electrical system and check that all equipment meets compliance standards.

Highway 30 Pump Station

The Highway 30 Pump Station is located near the intersection of Highway 30 and Dutch Canyon Road. The pump station is a package unit constructed in 1996. The pump station consists of the wet well with a ultrasonic level transducer and backup float switches; two surface-mounted, 7.5 HP, horizontal centrifugal pumps; a pinch valve in a separate vault; and a control panel. Both pumps were rebuilt in 2014. The station had a new flow sensor installed and wash water plumbed to a yard hydrant in 2015. There is also a sulfide control system including a 1/4HP air compressor at the pump station.

Highway 30 Pump Station

The operation and alarms for the pump station are similar to the Keys Landing Pump Station. The pump station has a manual power transfer switch and receptacle that can be connected to a portable generator, similar to the Spring Lake and Keys Landing Pump Stations. If the pump station were to overflow, it would occur at the first manhole upstream. The discharge would flow into the storm system on Highway 30, which then flows to Jackson Creek. There have been no known issues with the pump station overflowing or with pumps running continuously for an extended period of time. The O&M manual states that the pump capacity is 150 gpm. In the three-year pump run time history analyzed (2012-2015), the maximum run time was 7.2 hours a day. The pump station has adequate capacity.

The Highway 30 Pump Station collects wastewater from a number of commercial sources. The wet well consistently has grease buildup issues. The station is also susceptible to damage from cars because of its proximity to Highway 30. Minor corrosion was noted on pipes on the suction side of the system during the site visit.

A few deficiencies were noted with the Highway 30 Pump Station. These deficiencies and recommendations to correct them are summarized below.

Deficiencies

- Heavy grease in wet well.
- Minor corrosion on pipes.
- Vulnerable to traffic collisions from highway.

Recommendations

- Use pretreatment program to address grease buildup.
- Clean and coat piping in wet well and valve vaults as necessary.
- Install approved ODOT traffic barriers to protect station from traffic collisions.
- Perform a review of the electrical system and check that all equipment meets compliance standards.

Seven Oaks Pump Station

The Seven Oaks Pump Station is located near the intersection of Seven Oaks Drive and SE 9th Street. The pump station is a package unit constructed in 1996. The pump station consists of the wet well with four mercury float switches; two surface-mounted, 7.5 HP, horizontal centrifugal pumps; and a control panel.

The operation and alarms for the pump station are similar to the other two, smaller pump stations (Keys Landing and Highway 30). There have been no known issues with the pump station overflowing or with pumps running continuously for an extended period of time. The O&M manual states that the pump capacity is 150 gpm. In the three-year pump run time history analyzed (2012-2015), the maximum runtime was 3.7 hours a day, thus the pump station has adequate capacity. Should there be an overflow, the pump station would overflow first at the manhole located north of the pump station at the intersection of Seven Oaks Drive and SE 9th Street.

Seven Oaks Pump Station

According to City staff, the pump station operates well and there are fewer issues than with other pump stations. A few deficiencies were noted with the Seven Oaks Pump Station. These deficiencies and recommendations to correct them are summarized below.

Deficiencies

- Foam insulation around bottom of clam shell is coming unglued.
- Minor corrosion on control panel and pumps.
- Heavy corrosion on pipes in wet well (except discharge pipe).
- Heavy grease on top layer of pipe.

Recommendations

- Repair insulation on bottom of clam shell covering.
- Clean and coat piping in wet well and valve vaults as necessary.
- Use pretreatment program to address grease buildup.

- Perform a review of the electrical system and check that all equipment meets compliance standards.

Charles T Parker and East Airport Pump Stations

The Charles T Parker (CTP) and East Airport Pump Stations have recently been installed. They are operable, but currently have little to no flow to the stations. The CTP Pump Station is located at the end of Charles T Parker Way. The East Airport Pump Station is located east of the airport.. Both stations are anticipated to service the industrial growth in the airport industrial areas. The pump stations were both designed and constructed by Romtec Utilities. They have similar equipment and operations. The pump stations consist of a wet well, two submersible 10 HP centrifugal pumps, valve vault, permanent onsite generator, and control building. CTP Pump Station also has a pig launch/force main vault. East Airport Pump Station includes a flow meter in a separate vault. Both stations have two force mains installed for future expansion. East Airport wet well is sized for a third, future pump for expansion. Additional equipment specifications for the pump stations are listed in Table 2.1 Pump Station Inventory.

The operation and alarms for the pump station are controlled by Allen-Bradley Micrologixs systems. There have been no known issues with the pump station as they are not operating regularly at this time. Infiltration and groundwater cause the CTP Pump Station to run occasionally. The O&M manuals state that the pump design flow for CTP is 398 gpm and for East Airport is 336 gpm. Should there be an overflow, the pump stations would overflow at the pump station sites.

There are currently no deficiencies at these pump stations.

2.5 PIPELINES AND MANHOLES

Figures 7 and 8 summarize the size and material of each type of gravity sewer line. The type of sewer pipe is a good indicator of the age of the sewer lines. Most of the concrete pipes in the City were installed in the 1970’s and the PVC pipes were installed in the 1990’s and more recently. Approximately 48% of Scappoose’s wastewater pipelines are concrete, with the remainder PVC. Concrete pipe is susceptible to corrosion, while PVC is inert. Eventually, all concrete pipe should be replaced or rehabilitated.

Inflow and infiltration

Inflow (direct entry of runoff into the sanitary sewer system through openings, illicit connections, or storm sewer interconnections) and infiltration (entry of groundwater into the sanitary sewer through cracks in pipes and unsealed manholes) are common in Scappoose. This is evidenced by the flow increases in the wet, high groundwater season and the discrepancy between wet weather and dry weather flows entering the treatment plant.

As part of the initial data collection of this project in 2016, steps were taken to begin to identify sources of I/I using flow monitoring data and smoke testing. Continued use of this and other methods, such as CCTV and night-time monitoring, will assist the City in pinpointing sources of I/I and prioritizing pipeline rehabilitation/replacement activities. The following section describes smoke testing methods and the findings in more detail.

Smoke Testing

Smoke testing had not previously been performed, so the City elected to have portions of the gravity collection system smoke tested as part of the planning effort in 2016. Flow monitoring data, age and material of pipes, and knowledge from City staff were used to prioritize areas of the City for smoke testing.

Keller Associates worked with City staff to smoke test sections of the gravity sanitary sewer pipelines in the City between July 20th - 27th, 2016. Areas of the City that were smoke tested are shown on in Appendix C and include 96,300 linear feet of pipelines.

Smoke introduced into the sanitary sewer system should only be released from nearby manhole and cleanout pick holes and plumbing vents on buildings. Smoke found to be emitting from other locations is a sign that water may be entering the sanitary sewer system in these locations. Overall, there were not a lot of unexpected locations found to be releasing smoke. The problem areas located during smoke testing are summarized in Appendix C, along with photographs for each identified problem area. Roughly half of the problems found while smoke testing were related to open or broken cleanout caps. There were a few cross connections identified, including roof and driveway drains. In addition, a few manholes were smoking around the rims or elsewhere on the manhole collar. There were a handful of cases where smoke was found inside houses; most of these were likely a result of dry P-traps. Recommended action to correct identified problems are also summarized in Appendix C.

Steinfeld Plant's Gravity Line

The 8-inch concrete gravity main running along SE Elm Street from SE 1st Street to SE 4th Street previously serviced the Steinfeld plant. The plant closed by 2001 and the property was developed into residential property which is now the only user type on this gravity pipeline. The 1,350-foot section of pipe was CCTV inspected in August 2011 and these video inspections were reviewed as part of this study.

The CCTV logs reveal that there is some surface deterioration on the bottom third, but not significant scouring. The scouring appears similar to scouring on other CCTV inspections completed in the area. The CCTV operator identifies a number of small to medium offset joints throughout the length, which create sags identified by the operator. The inspection logs indicate the line is in good condition. There are no major defects seen in the CCTV inspections. The scope of this study did not include any other CCTV inspection video reviews.

2.6 COLLECTION SYSTEM COMPUTER MODEL

This section summarizes the wastewater collection system model development process and existing collection system hydraulic analysis. It outlines the model construction and model calibration process, and documents existing deficiencies. Improvements to address these deficiencies are presented in Chapter 4.

2.6.1 Model Selection

InfoSWMM Suite 14.7, Update #1, was selected as the modeling software for this project. InfoSWMM is a fully dynamic model which allows for evaluation of complex hydraulic flow patterns.

2.6.2 Model Creation

The model built in 2016 was used as the basis for this model update. GeoSolve, Inc. is a GIS consulting firmed hired by the City to maintain the Scappoose GIS database. Information from this database was used to update pipe diameter and invert elevation data that has changed since the 2016 model creation.

Trunk lines with diameters of 10-inches and larger were modeled. Any pipes that connect the trunk lines together were also modeled regardless of their diameter. Figure 10 in Appendix A shows the modeled lines in the system. Several queries were conducted to reveal anomalies in the data. These included reverse slope pipes, changes in pipe size, and uncommon configurations in the pipe network. Anomalies were discussed with City personnel, additional field work was completed, and appropriate changes were made to the model. The City reviewed existing pipe size and added infrastructure to the model. The primary additional infrastructure added since 2016 includes pipe upsizing and extensions in the airport industrial area, the CTP and East Airport pump stations, and upsizing of the 21-inch line on Columbia Avenue to 30-inch diameter.

Pump Stations

The Smith Road, Highway 30, Spring Lake, Seven Oaks, CTP, and East Airport pump stations were included in the model. Keys Landings pump station was not included because it handles a small amount of flow and does not connect to a 10-inch or larger line. The pump station wet well dimensions and set points were taken from the operations and maintenance (O&M) manuals, then checked by system operators. The O&M manual pump curves and duty points were used in the model to characterize the pump station pumps.

2.6.3 Model Calibration

Model loads refer to the wastewater flows that enter the sewer collection system. These loads are comprised of wastewater collected from individual services (base flows), plus groundwater infiltration and storm water inflows (I/I). As part of this study, flow monitoring was completed during the wet weather period from December 2019 into January 2020. Flow monitoring data was collected at various manholes throughout the system to help calibrate the model. Six monitoring sites were selected, dividing the system into basins. Figure 9 in Appendix A shows a map of flow meter locations and basins. The basins were used to characterize flows throughout the system. The collected data was analyzed along with continuous precipitation data to establish typical 24-hour patterns, average flows at each site, and gauge rainfall influence in the system. Both dry weather and wet weather periods were used for loading and calibration efforts. Loads for the model were developed and calibrated in several stages as described below.

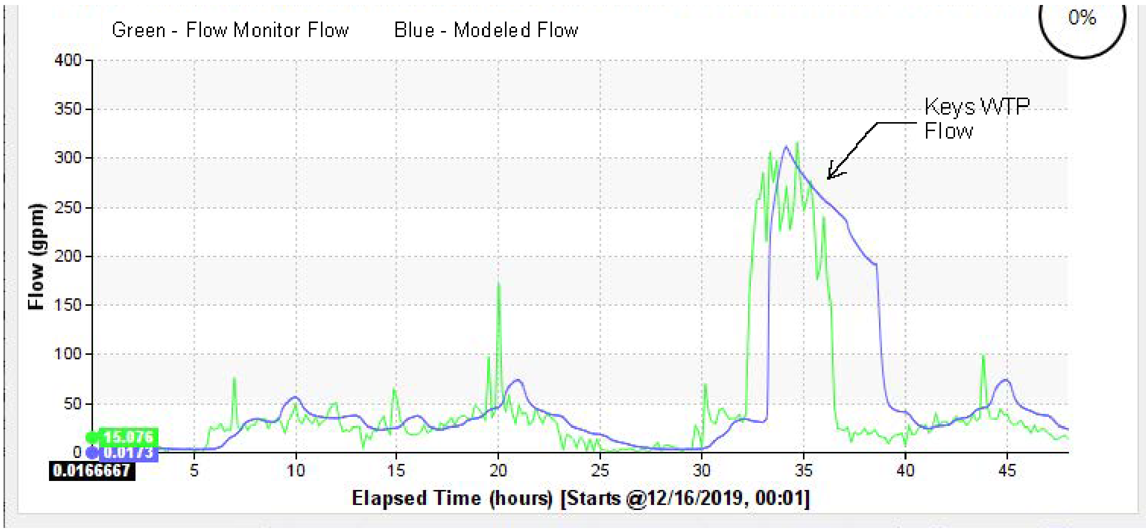
Dry Weather Calibration

For a starting point, base flows were estimated using GIS address data to provide spatial distribution of equivalent dwelling units (EDU) and dry weather flow per capita criteria. An average dry weather flow was assigned to each modeled manhole based on the EDU contributing to the manhole.

A period of five dry days (none or trace amounts of rainfall) was analyzed from the flow monitoring data. A typical day was selected for each site, which was utilized to develop a diurnal flow pattern for the basin. These typical patterns were assigned to all existing flows within the basin corresponding to the monitoring site.

In addition to calibrating the model at various locations within the collection system, total modeled influent flows at the Wastewater Treatment Plant (WWTP) were also compared to the targeted design average daily flow. An example of calibration results is shown in Chart 2-1. The blue line shows the model results and the green line show flow monitoring data collected. The large increase in flow around hour 32 is the Keys WTP backwash contribution to the sewer system. The maximum WTP load conditions seen in the system were discussed with operators and modeled to reflect these conditions.

CHART 2.1 – SAMPLE DRY CALIBRATION SITE 6

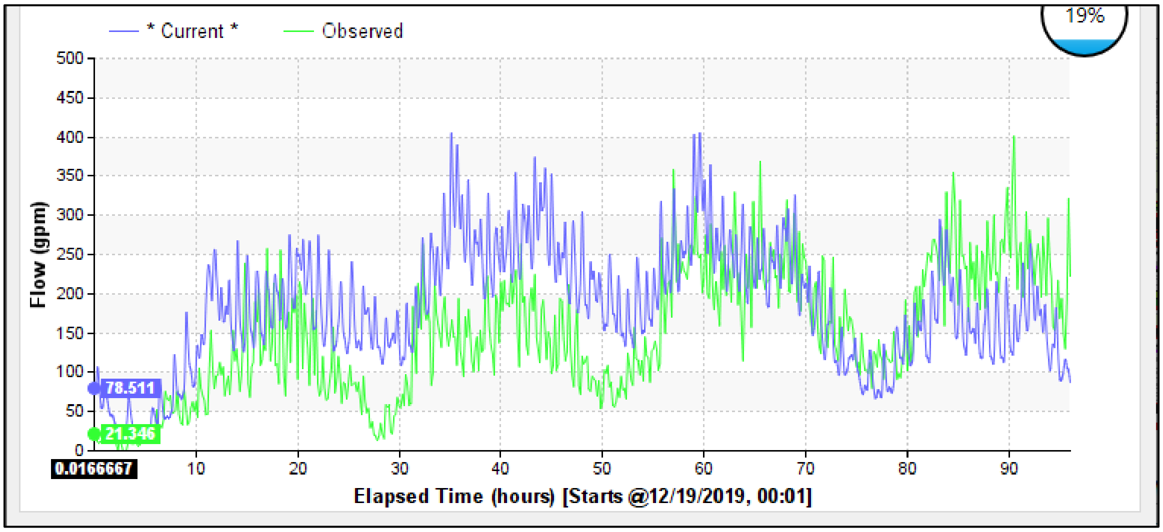


A secondary set of flow monitoring was completed in April as development of the existing system model showed lower flows from monitoring data at Sites 4 and 5 than the WWTP discharge monitoring reports (DMR). The flow monitoring completed in 2016 also showed significant flow in the system downstream of Sites 4 and 5 in comparison with the DMR records. Monitoring was completed at three additional locations on Columbia Avenue for 1-3 days to assess contributions from the connecting lines between Sites 4 and 5 and the WWTP. The monitoring indicated additional flows from the Bird Road and the Miller Subdivision with portions of I/I contributing. It is recommended the City continue to investigate I/I in this lower portion of the collection system as part of the City-wide I/I reduction efforts.

Wet Weather Calibration

The RTK method was used for rainfall-derived infiltration and inflow (RDII) prediction. Rainfall data with the 24-hour period with the highest cumulative and highest intensity rainfall during the period of flow monitoring was utilized to calibrate wet weather flows. The storm event was entered into InfoSWMM. RTK parameters were then adjusted to match model results with flow monitoring data. Total modeled influent flows at the WWTP were compared to the targeted design average daily flow in addition to calibrating the model at various locations within the collection system. Example calibration is shown in Chart 2-2.

CHART 2.2 – SAMPLE WET CALIBRATION SITE 6



Design Storm

The design storm for model evaluation was the 5-year, 24-hour storm event. A standard 24-hour NRCS rainfall distribution for a Type 1A storm was used. The rainfall for the 5-year, 24-hour storm event from NOAA isopluvial maps is 2.8 inches (matches the rainfall event used in the FPU). This was used as the multiplier for the Type 1A storm hyetograph. The existing system, calibrated model was run with the design storm event and daily and peak flows at the WWTP were compared to the planning criteria $PDAF_5$ and PIF_5 (Table 1.4). The maximum day and peak flows were lower than the planning criteria. Reviewing flow monitoring and DMR data, the wet weather storm event used for calibration had very dry antecedent conditions and the RDII response matched these conditions. The planning criteria and 5-year storm event account for wet antecedent conditions and high groundwater as are typical for large storm events in the area. Additional infiltration was added to the overall system to represent wet antecedent conditions and further calibrate the 5-year design storm response. Appendix D contains a summary of the data and results used for modeling and evaluation purposes.

2.6.4 Existing System Capacity Limitations

The calibrated model was exercised to determine the effects of a 2020 peak day flow event on the system. Figure 11 in Appendix A illustrates the utilized capacity of the existing system. 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 67-84% capacity, etc.). As stated in Chapter 1, the planning criteria for undersized pipelines is 67% capacity or more. The manholes shown in orange experience surcharging and represent the greatest risk for backing up services and possible overflows. The majority of pipes at higher capacities are located on the Smith Road trunk line, downstream of the Smith Road PS force main discharge, and the High School Way basin (trunk line up along Old Portland Road and down to Elm Street).

None of the manholes in the model show potential sanitary sewer overflows, but there is surcharging present at some locations (Figure 11). Surcharging in these locations has been noted by City staff. There are three locations on SW Old Portland Road where pipes surcharge a few inches above pipe crown. The most recent CCTV inspection of this line show there are a number of sags along the line that are likely causing the surcharged conditions and should be resolved when the pipeline is upsized.

2.6.5 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 of the 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.

2.7 20-YEAR COLLECTION SYSTEM PERFORMANCE

This section summarizes 20-year (2040) flow projections and the model evaluation of 2040 future system expansion, and documents anticipated future deficiencies. Improvements to address these deficiencies are presented in Chapter 4.

2.7.1 Future Flow Rate Projections & Model Scenarios

Future loads were distributed based on City projected residential, commercial, and industrial growth over the 20-year planning period and in alignment with the Housing Needs Analysis for the City. The projected growth areas are shown in Figure 6. Flows per land use for projected growth areas were aligned with the flow factors used in the FPU. Future flow contributions from growth areas were projected using area and land use factors. Additional flow was allocated for future large industrial users as directed by City staff and in alignment with the FPU projected flows. WTP flows (for both Miller Road Plant and Keys Landing Plant) were assumed to remain the same in the future. Projected system flows are presented in Table 1-4 and match the FPU.

Flow for each growth area identified in Figure 6 was added to the closest modeled manhole to allocate future flows. Additional gravity mains were added to the model and evaluated for carrying increased flows from these growth areas to the system. These are identified in Figure 6. In addition, City-provided contour data was used to assess potential pump stations required to provide sewer service to areas within the UGB. These general locations are shown in Figure 6. Buildout areas and the most current Buildable Lands Study available, should be taken into account during the design of any of these proposed pump stations. The future model was run to analyze the effects of future growth on the system in 2040.

2.7.2 Future System Capacity Limitations

Modeling results (Figure 12) show that most of the pipelines at or above capacity are in similar areas as the existing system evaluation. Additional pipes along 4th Street and Em Watts Road are undersized for future flows and surcharging occurs on the line. The High School Way basin has slightly higher utilized capacities, but the general areas undersized are largely unchanged between 2020 and 2040. The pipeline along Tyler Street has sufficient capacity for current flows, but needs to be upsized for 20-year flows. The pipeline downstream of the CTP Pump Station force main discharge is undersized for future flows.

2.7.3 WTP Impacts

WTP flows (for both Miller Road Plant and Keys Landing Plant) were assumed to be the same for existing and future evaluations. The flows used in the existing system are conservative and represent the largest volumes of backwash and filter-to-waste as provided by City operators. The Keys WTP significantly impacts the capacity of the western line to handle system flows. Both existing and future models were evaluated if WTP flows were automated and released to the sewer system over an extended period during the night when system flows are lowest. Figures 13 and 14 show the existing and future capacity evaluation with nighttime WTP flows. As illustrated in both figures, removing WTP flows from the sewer or shifting them to periods of low flow, would significantly increase the system capacity for sewer base flows. It is recommended that the water treatment plants develop a plan to recycle the wash water to reduce water use, as well as evaluate automated systems to allow for backwash flows to be released during nighttime flows. These recommendations will be included in the alternatives presented in Chapter 4.

2.8 CITY ORDINANCES & PRETREATMENT

Chapter 13.12 of the Municipal Code outlines City regulation of the sewer service system. Fats, oils, and grease (FOG) currently cause problems in the system by clogging and backing up pipes and pump stations. Many of the pump stations accumulate FOG and require maintenance to remove FOG build up. FOG can build up and clog pipelines as well. City staff are aware of a number of food establishments that do not have FOG interceptors and there are no regular inspections of those that are installed in the City. Section 13.12.050 D states that grease, oil, and sand interceptors shall be provided when, in the opinion of the City Manager or designee, they are necessary for the proper handling of liquid wastes. This ordinance can be used to require that all food establishments, new or existing, in the City install FOG interceptors. The City could add language to the ordinance to require as a minimum, all food establishments and any other type of commercial or small industrial establishments, have and maintain an interceptor.

FOG interceptors will not be effective if they are not properly cleaned and maintained. The owner is responsible for maintaining "continuously efficient operation at all times" according to City code. In order to ensure that this happens, the City should consider implementing some type of inspection program to monitor and enforce cleaning and maintenance of interceptors. The City could revise this section of City code to require cleaning and maintenance on a specific time schedule. Part of the inspection program could involve establishments to submit regular cleaning and service reports. The City may also want to encourage the use of a pumping company that is part of the Preferred Pumper Program, which ensures that cleaning and maintenance meet regional standards. Companies registered with the Preferred Pumper Program certify that they will follow pump-out criteria. More information on preferred pumpers can be found at http://preferredpumper.org/.

Section 13.12.120 outlines the violation and penalty procedures. When a violation of the code occurs, a written notification stating the nature of the violation and a reasonable time limit to remedy the violation is sent to the user. If the violation is not corrected within the stated time limit, the person is guilty of a misdemeanor and upon conviction can be fined a maximum of $200 per violation. Each day the violation continues is considered a separate violation. As comparison, the City of Sheridan also has a $200 maximum for each violation, while the City of St. Helens has a maximum of $2,000 for each violation.

Industrial users are addressed separately in Chapter 13.16 of the municipal code. Currently, the City does not have any identified industrial users. The projected growth of the City is anticipated to bring a number of potential industrial users to the City in the near future. Currently, the ordinance allows the City to require pretreatment facilities as deemed necessary by the City Manager to comply with the requirements of the chapter. The violation penalty is a fee up to $500 per day as the violation continues. The City Manager can issue a cease and desist order with a time schedule and preventative or remedial actions that must be followed. If the user fails to comply, the City can take actions deemed necessary including immediate severance of the sewer connection. The ordinance also covers minimum compliance with state and federal regulations; substance limitation revisions; general discharge prohibitions; dilutions; accidental and unlawful discharges; City development of a fee structure; general administration of permits; and analysis, reporting, and monitoring requirements. The City should review the sewer system industrial user ordinance and make any desired adjustments in anticipation of new industrial users moving into the City.

2.9 FINANCIAL STATUS OF EXISTING FACILITIES

See Chapter 6 for the financial status of existing facilities.

2.10 WATER/ENERGY/WASTE AUDITS

The City completed a baseline energy evaluation with support from Columbia River Public Utility District (PUD) and Energy Smart Industrial (ESI) in 2019 (Appendix E). The evaluation was part of a multi-year Strategic Energy Management (SEM) engagement to address operations and maintenance opportunities and improve the effectiveness of previously implemented action items. The City continues to work on the SEM and track energy performance and savings.

The City has not completed any recent water or waste audits.

CHAPTER 3 – NEED FOR PROJECT

3.1 HEALTH, SANITATION, AND SECURITY

The Clean Water Act of 1972 provides the primary regulations for water quality in the waters of the United States. It requires that point source contributions to surface waters obtain a discharge permit (currently permits are issued from Oregon DEQ as NPDES permits). These permits determine the conditions for discharge into surface waters. Oregon DEQ provides information about other Clean Water Act items, including the status of receiving streams, beneficial uses, and waste load allocations from the TMDL in the NPDES Fact Sheet for Scappoose. DEQ will update the fact sheet during the process of renewing the discharge permit. Additional information on compliance with the NPDES permit for Scappoose can be found in the 2018 FPU.

Other issues regarding public health, sanitation and security involve events when untreated or undertreated effluent overflows onto the ground or is discharged to surface water. There have not been any recent overflows in the Scappoose wastewater system.

All seven pump stations in the collection system are secured by either clam shell covers or a locked gate and barbed wire fence. The WWTP is also secured by a locked gate and barbed wire fence.

3.2 AGING INFRASTRUCTURE

Aging infrastructure is an issue for Scappoose. Approximately half of the collection system is concrete pipe, which is often susceptible to wear damage, cracking, root intrusion, and other problems associated with older, brittle materials. Infiltration will increase due to aging pipes, and is already an issue. Details of the system deficiencies are discussed in Chapter 2 of this report.

3.3 SYSTEM DEFICENCIES

Collection system deficiencies, including pump stations, are discussed in detail in Chapter 2. A summary of system-wide deficiencies is included below.

Smith Pump Station

Smith Pump Station has a small operating volume causing flow to back up into the gravity mains and pumps to have a high number of starts per day. The pump station is near or at capacity based on pump run time for 2019. The pumps are nearing the end of their useful life. The access hatch is heavily corroded, there is grease build up in the wet well, and one of the blowers for the fan in the dry well does not work.

Spring Lake Pump Station

There is potentially an air lock in the force main. Grease and paper products in influent have been an issue in the past.

Keys Landing Pump Station

One pump does not stay primed; however, the City has changed to exercising the pumps weekly and reduced the number of alarms received from the pump station. There have been odor problems at the station. The wet well piping has heavy corrosion. It is difficult to get the portable generator on site; particularly in winter. The clam shell door does not stay open on its own.

Highway 30 Pump Station

This station is vulnerable to traffic collisions from the highway. There is heavy grease in the wet well and minor corrosion on the pipes.

Seven Oaks Pump Station

There is heavy grease on top layer of the pipe, heavy corrosion on pipes in the wet well, and minor corrosion on the control panel and pumps. There is a small section of foam insulation on the bottom of the clam shell that is coming unglued.

Charles T Parker and East Airport Pump Stations

Theses pump stations were recently constructed and there are currently no deficiencies at the stations.

Collection System

The Smith Road Pump Station basin trunk lines and High School Way basin trunk line are undersized for current and future planning flows and cause surcharge sites. Laurel and 3rd Streets downstream of the Smith Road Pump Station force main discharge are undersized for current and future flows. The last pipe segment from Columbia Avenue into the WWTP headworks is undersized. The trunk line on SE Tyler Street is at or above capacity for 2040 flows. The line on Wagner Court downstream of the Charles T Parker Pump Station force main discharge is undersized for 2040 flows.

3.4 REASONABLE GROWTH

Wastewater facility improvements are needed to stay ahead of growth due to potential increased population and new construction. Chapter 1 of this report discussed population growth projections including customers served and the wastewater flows associated with this growth. The collection system will have to be expanded to accommodate the potential growth in the planning period. These improvements, where possible, will be proportionately funded by the new growth through the use of system development charges (SDCs).

CHAPTER 4 – ALTERNATIVES CONSIDERED

This chapter discusses alternatives that were considered to address the collection system deficiencies mentioned in Chapter 2. It also includes planning criteria and environmental and constructability considerations.

4.1 PLANNING CRITERIA

The planning criteria used for this collection system facilities planning effort are summarized as follows. The City's conveyance system will be sized for the projected 2040 peak instantaneous flow rates associated with the 5-year, 24-hour storm event, which is 5.9 MGD (2040 PIF_5 in Table 1.4). The City decided that the criteria for requiring improvements is when pipes have less than 33% remaining capacity (max flow depth/full depth (d/D) greater than 67%). Recommended improvements will be sized to flow at 67% capacity or less for the 2040 peak flows to allow for additional growth for the life of the pipe and in accordance with the City design standards. Additionally, it should be noted that efforts to reduce I/I in the collection system could further extend the lifetime of the pipeline with regards to capacity.

4.2 ALTERNATIVES DESCRIPTION

Alternatives are described and discussed below. Maps of the alternatives are included below. Overall planning level project cost estimates for alternatives are presented in Chapter 5. For each set of alternatives, there is also an unstated option to do nothing and make no changes. This option perpetuates existing deficiencies and increases the risk of surcharging, overflows, environmental damages, DEQ violations, and subsequent fines.

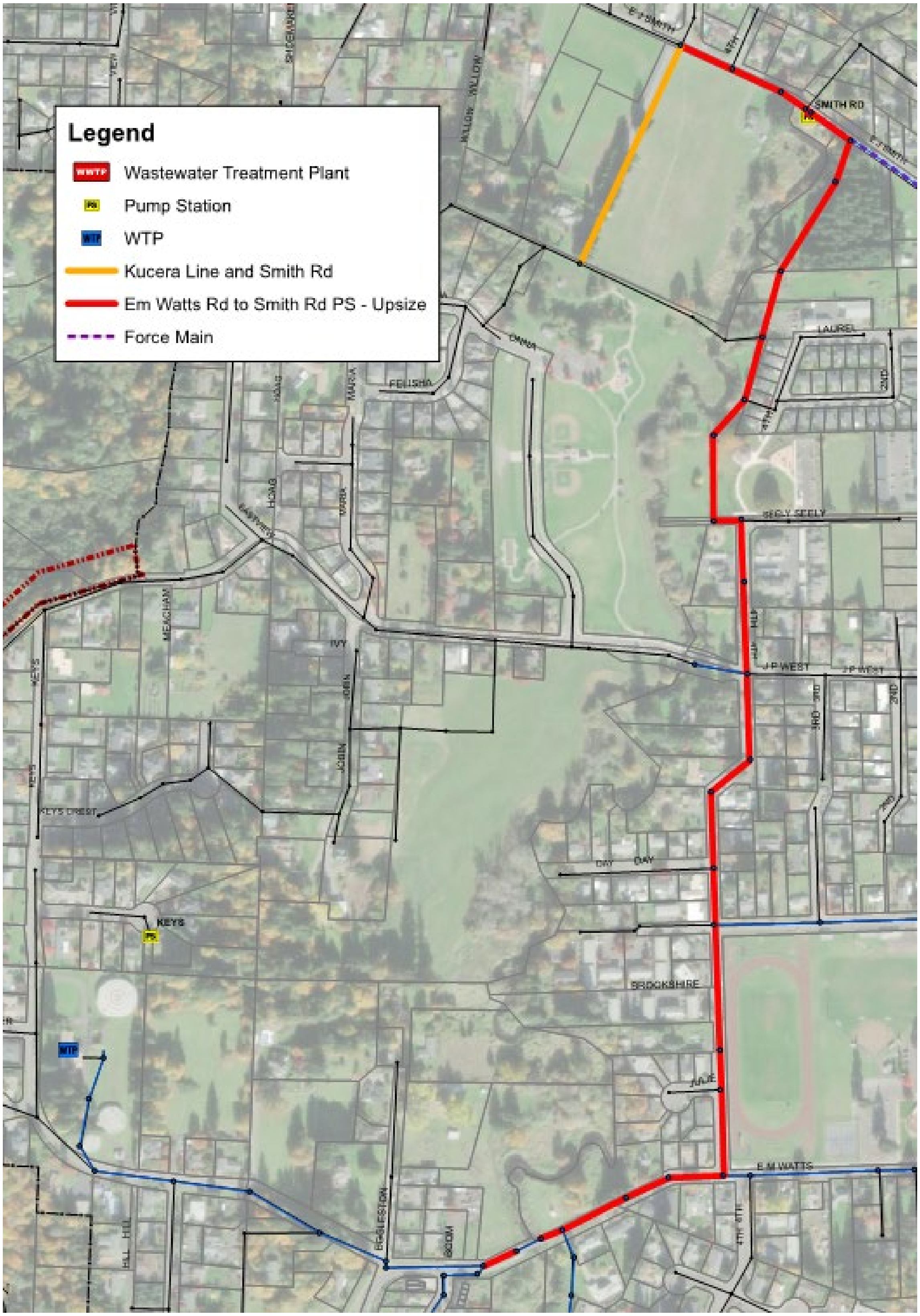
Smith Road Pump Station Basin

The City has identified the potential to abandon the creek crossing on the north side of Veterans Park by running a pipeline north across the open field from MH0418 to MH0400 on Smith Road and 5th Street (referred to as the Kucera line). The City has already completed a survey for this alignment. Alternatives 1 and 2 discussed below, include the new Kucera line, abandoning the creek crossing, and upsizing the line along Smith Road from 5th Street to the Smith Road Pump Station to a 15-inch diameter pipe with the last section to the pump station being an 18-inch diameter pipe. It should be noted that this last section extends under the creek and will require a boring. Alternative 3 discussed below, includes the new Kucera line, abandoning the creek crossing, and upsizing the line along Smith Road from 5th Street to the manhole east of the creek crossing to a 12-inch diameter pipe. The last sections to the pump station (including the creek crossing) would not be upsized from existing size. However, this would result in the creek crossing (existing 10-inch line) being smaller than the upstream pipe diameter (upsized to 12-inch).

Alternative 1 – Continue WTP Operations and Upsize Existing

This alternative requires upsizing of the 12-inch pipeline parallel to Scappoose Creek and along SW 4th Street from SW EM Watts Road to the Smith Road Pump Station if the Keys WTP continues to operate with backwash flows during high system flows in the morning. The existing undersized pipes are highlighted in Figures 11 and 12 (Appendix A). Alternative 1 includes upsizing the red colored pipes shown in Figure 4.1 (next page), extending from EM Watts Road to Smith Road Pump Station as well as NW up Smith Road from the pump station. A portion of the line is located in close proximity to the South Scappoose Creek. Replacement of this pipeline may require additional bank stabilization work. The new, Kucera line is shown in orange in Figure 4.1. This alternative includes approximately 4,800 LF of pipe being upsized to 10-, 15-, and 18-inch diameter pipe.

FIGURE 4.1 – ALTERNATIVE 1



Alternative 2 – Shift WTP Operations to Night and Upsize Existing

This alternative evaluates required upsizing on the 12-inch pipeline parallel to Scappoose Creek and along SW 4th Street if the Keys WTP automates operation to allow for backwash flows to be released during nightime, low flow periods. WTP flows could be adjusted to release at various rates between approximately 10 pm - 7 am to minimize peak flows to the collection system. The undersized pipes with the nighttime Keys WTP flows are illustrated in Figures 13 and 14 (Appendix A).

Alternative 2 includes the colored lines in Figure 4.2 (next page). This alternative could be split into two projects – the new Kucera line and improvements on Smith Road (orange lines in Figure 4.2), followed by improvements from 4th St to the Smith Road Pump Station (light and dark blue lines in Figure 4.2). The improvements from 4th Street to the Smith Road Pump Station approximately 1,100 LF of pipe being upsized to 15- and 18-inch diameter pipe (dark blue Figure 4.2) as well as cured-in-place-pipe rehabilitation of the remaining line (light blue Figure 4.2). Trenchless technology along this portion could provide cost savings to open trench replacement because of proximity to the creek bank.

Alternative 3 – Construct New HWY 30 Crossing at Maple Street and Shift WTP Operations to Night

This alternative evaluates the option to construct a new crossing under HWY 30 and the railroad at Maple Street. Flow from 4th Street south of Maple Street would be directed through the new crossing and continue by gravity to Columbia Ave at Jay Davis Lane. An overflow would be provided at 4th and Maple Street for flow to go to the Smith Road pump station if the manhole surcharges sufficiently. This alternative includes approximately 5,000 LF of new 12-inch pipe from 4th and Maple Streets to the connection on Columbia Ave at Jay Davis Lane. The improvements would bore under the highway and railroad. The extents of the alternative are shown in Figure 4.3 (two pages down). This alternative may include easement acquisition on the Jay Davis property to connect the new pipeline from the end of Maple Street to Jay Davis Lane.

This alternative was reviewed with both daytime and nighttime WTP flows. The daytime WTP flows require pipe upsizing along 4th Street and EM Watts south of the new pipeline at Maple Street. This additional pipeline is anticipated to be more costly than improvements at the Keys WTP to allow for automated flow metering during nighttime flows. The improvement extents (Figure 4.3) include Keys WTP improvements. Alternative 3 provides an opportunity for the City to re-route flows by gravity to the WWTP. This would reduce the flow going through the Smith Road Pump Station, which would reduce O&M costs and could provide cost reductions to Smith Road Pump Station capital improvements.

This alternative also includes rehabilitation of the trunk line parallel to the creek by CIPP or similar trenchless methods to reduce creek impacts as shown by light green pipes in Figure 4.3.

FIGURE 4.2 – ALTERNATIVE 2

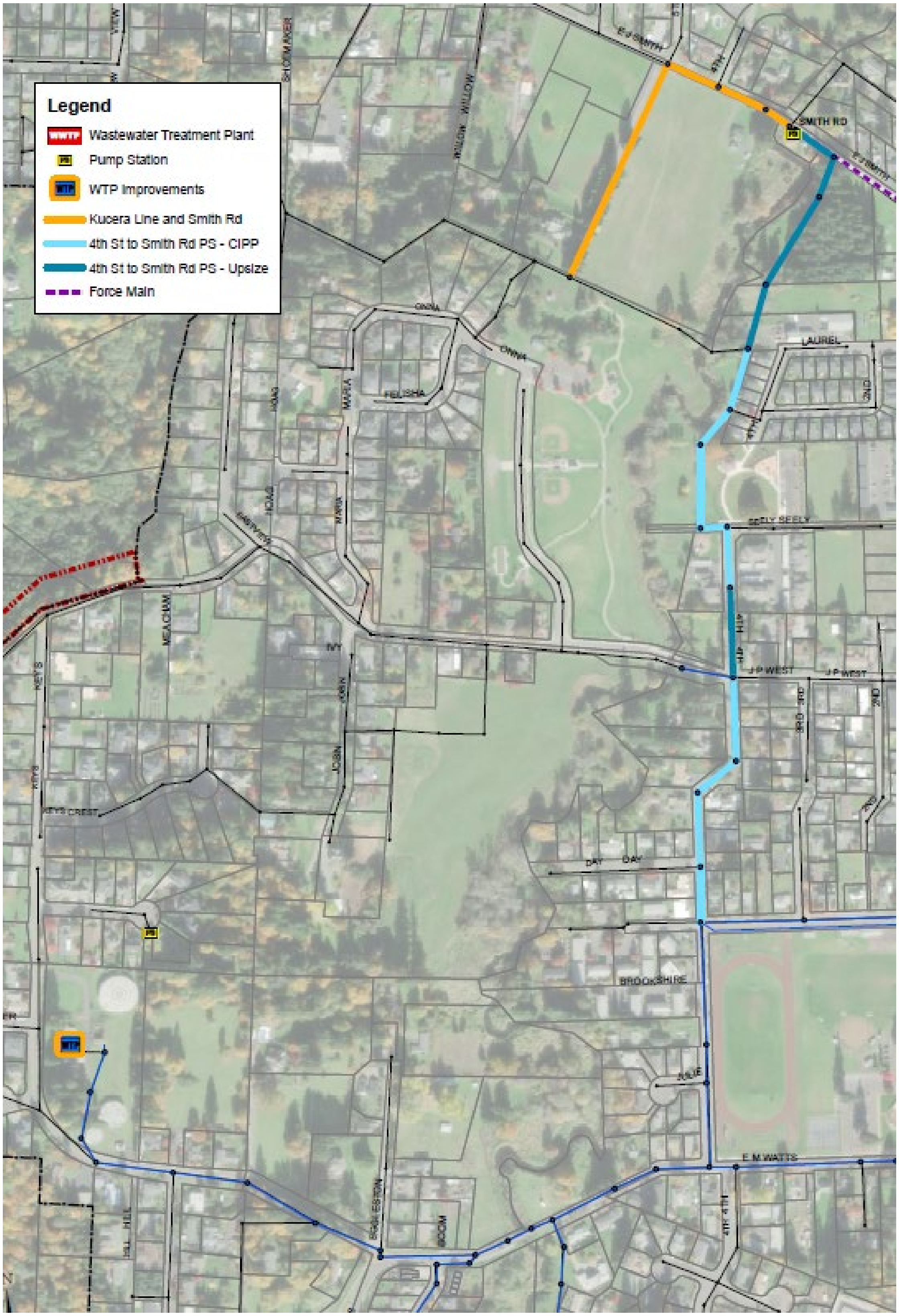
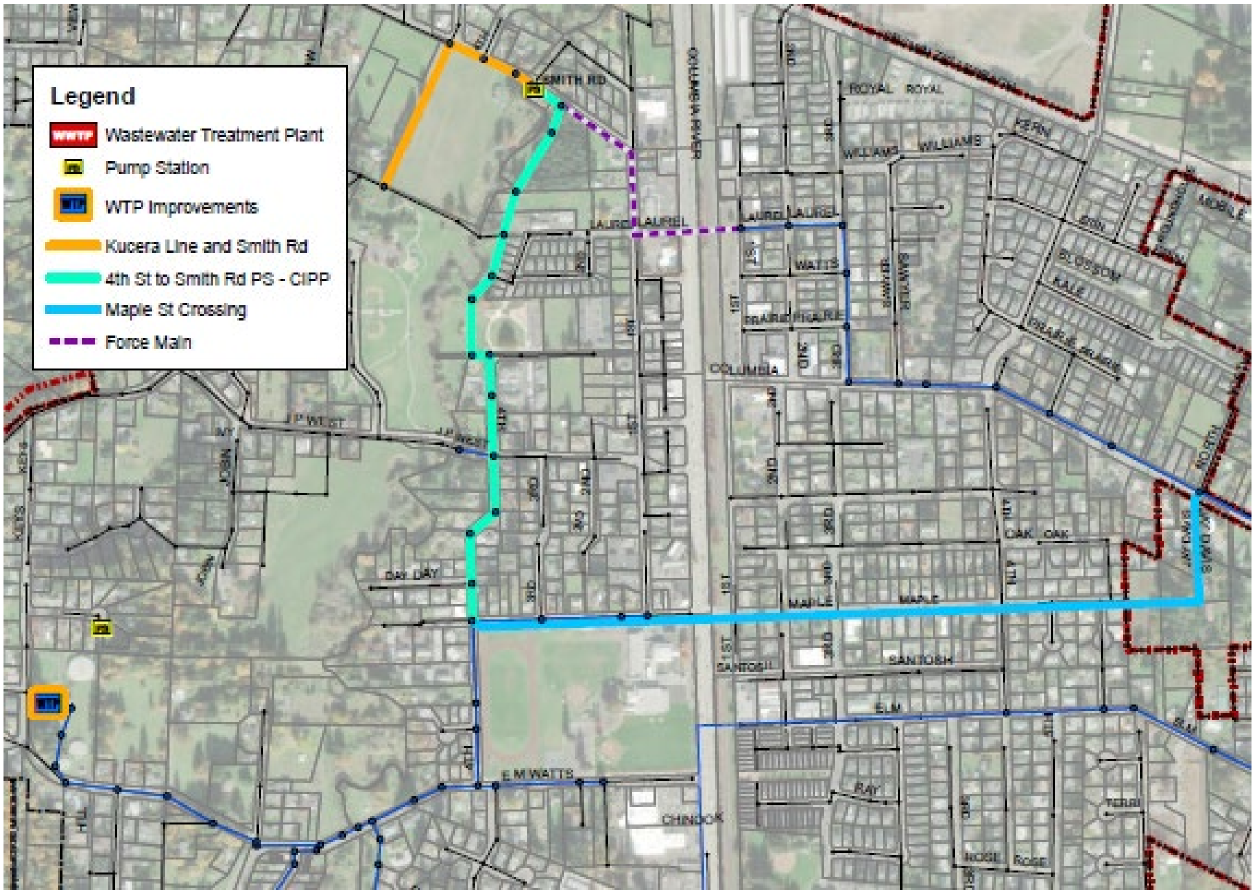


FIGURE 4.3 – ALTERNATIVE 3



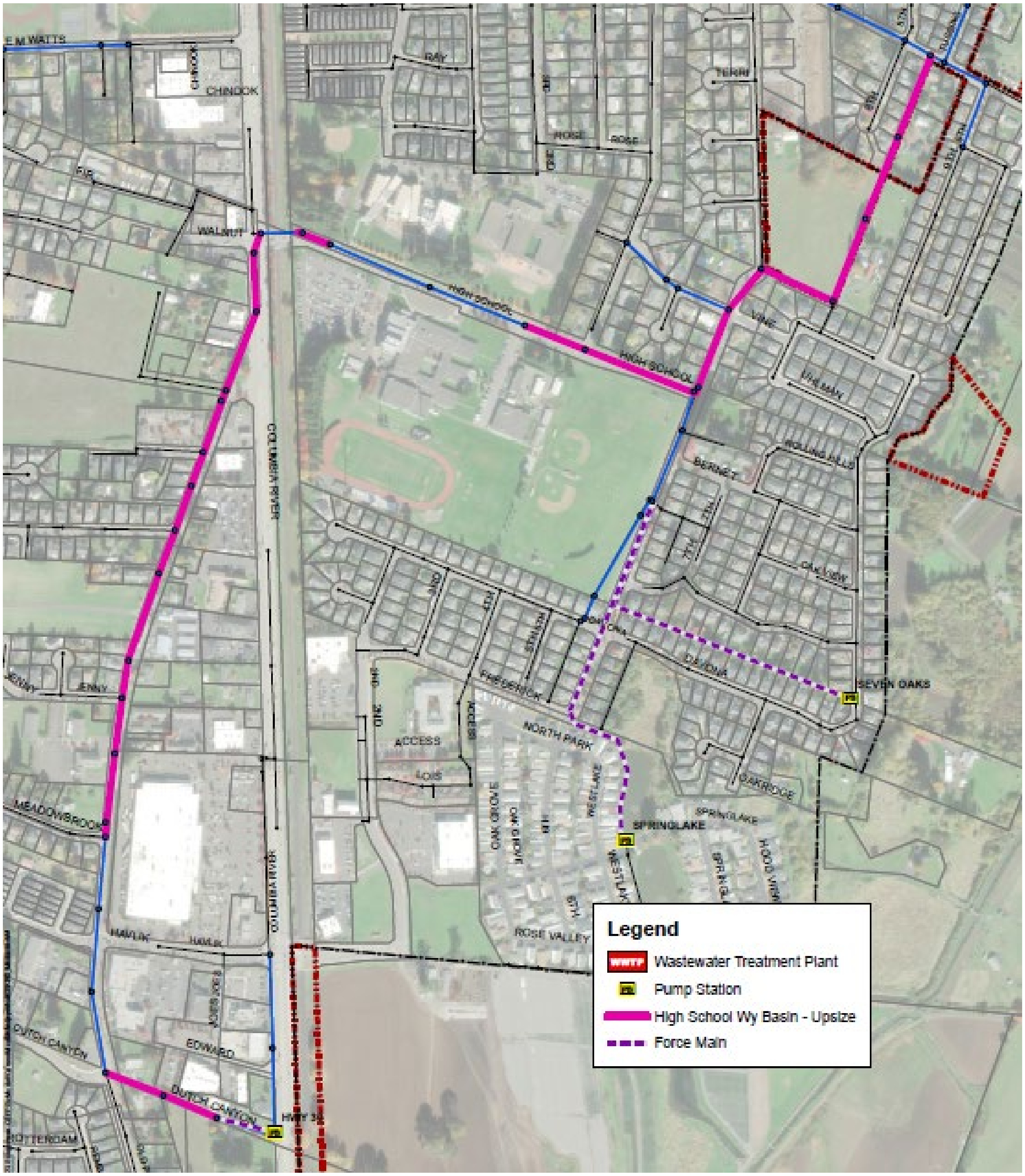
4.3 ADDITIONAL IMPROVEMENT DESCRIPTION

In addition to the alternatives described above to address capacity limitations in the Smith Road Pump Station Basin, improvements to address the remaining capacity limitations in the system were evaluated. Other alternatives reviewed for these limitations were found to be not feasible or cost prohibitive. For each set of improvements, there is also an unstated option to do nothing and make no changes. This option perpetuates existing deficiencies and increases the risk of surcharging, overflows, environmental damages, DEQ violations, and subsequent fines. The additional improvements are described in the following sections.

High School Way Basin - Upsize Existing

These improvements address required upsizing of the existing line to alleviate capacity issues along the High School Way Basin trunk line. Improvements would include approximately 4,200 LF of 10-inch pipe and 2,100 LF of 15-inch pipe. This option includes approximately 500 LF of work along HWY 30. The extent of improvements is shown on Figure 4.4 (next page). A second highway/railroad crossing was reviewed as an option for improvements. However, discussions with the City revealed that construction along the highway and existing utilities adjacent to the highway make this option not feasible and cost prohibitive.

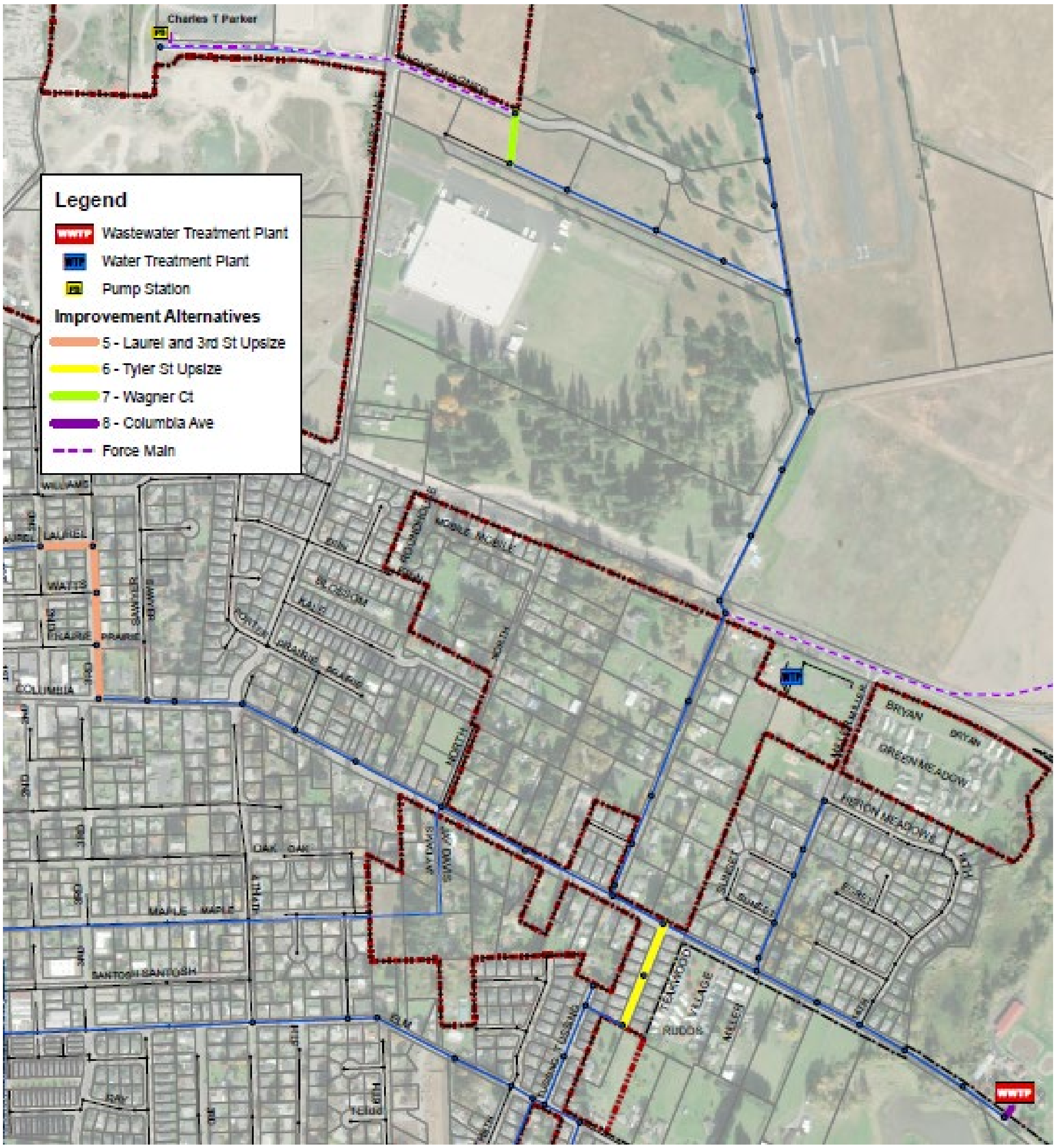
FIGURE 4.4 – HIGH SCHOOL WAY BASIN IMPROVEMENTS



Miscellaneous Improvements

There are four additional locations that require improvements to address system capacity limitations – Laurel and 3rd Streets, Tyler Street, Wagner Court, and Columbia Avenue at the WWTP. The improvement locations are shown in Figure 4.5 (next page). Parallel pipelines and a force main extension were considered for Laurel and 3rd Streets improvements, but ultimately these options were assessed to be unfeasible or cost prohibitive. When these improvements are designed, review of the Smith Road Pump Station should be included to determine if there could be any cost savings to the City by combining projects. If force main improvements are needed for Smith Road Pump Station, a force main extension to 3rd Street and Columbia Avenue could provide cost savings. At this stage of planning, the problem area on Laurel and 3rd Streets is best suited to upsize existing pipes. The Laurel and 3rd Street improvements include approx. 1,100 LF of upsizing.

FIGURE 4.5 – ADDITIONAL IMPROVEMENTS



The capacity limitations on Tyler Street and Wagner Court are best suited for upsizing existing pipes. The Tyler Street Improvements include approx. 600 LF of upsizing. The Wagner Court improvement is the 300 LF pipeline downstream of the Charles T Parker Pump Station force main discharge manhole.

The Columbia Avenue improvement is the final, 70-foot pipe segment into the WWTP headworks. This improvement alleviates capacity issues in the segment, but it has minimal impact on the capacity of the rest of the Columbia Avenue trunk line. This project is included in the Phase 2 improvements in the 2018 FPU with headworks improvements. The cost to upsize this section of pipe is not included in the collection system capital improvement projects (CIP) as it is included in the 2018 FPU cost estimates.

For all pipeline upsize projects, the existing line can be upsized or a parallel line can be constructed. The pipe age and condition of existing pipes should be evaluated before the option of a parallel line is considered as it may be more cost effective to replace the line to extent useful life. There are also options to use trenchless technologies such as pipe bursting, cured-in-place-pipe installation, or slip lining. Under the right circumstances, these approaches can be less costly than the open cut construction approach. Keller Associates recommends that each pipeline segment be evaluated during the preliminary design phase to determine the optimum replacement strategy. This effort includes a careful review of CCTV conditions and other site constraints, and should be completed as part of the concept or pre-design phase of pipeline rehabilitation / replacement projects.

4.4 MAP

Maps of the existing collection system are provided in Figures 7 and 8 (Appendix A). The various alternatives are shown in the figures presented above.

4.5 ENVIRONMENTAL IMPACTS

Potential impacts of the alternatives to the environmental resources presented in Chapter 1 are described below.

Land Use / Prime Farmland / Formally Classified Lands

No proposed projects will occur on prime farmland that is not already used for other purposes.

Floodplains

As shown in Figure 2, some portions of the study area (including the wastewater treatment plant) are located inside the 100- and 500-year floodplains of the Multnomah Channel. None of the alternatives would create new obstructions to the floodplain.

Wetlands

None of the alternatives are located in wetland areas (Figure 5).

Cultural Resources

None of the alternatives would interfere with the above-ground cultural resources identified by the State Historic Preservation Office.

Biological Resources

Several fish in the Salem BLM District are listed as sensitive or threatened; however, no in-stream work is anticipated with any of the alternatives, so no fish species would be disturbed. Endangered species in the district include the fender's blue butterfly, Columbian white-tailed deer, Bradshaw's desert parsley, and Willamette Valley daisy. It is unlikely that any of the plants exist on the proposed project sites since the areas have been previously disturbed and paved or landscaped. If the butterfly or deer is found, further investigation would be undertaken to determine the necessary mitigation measures.

Water Resources

Modifications to the collection system would reduce the risk of overflows and potential to spill into waterways. Alternatives 1 and 2 both include the abandonment of a creek crossing, which reduces the risk of leaks or breaks that would discharge wastewater into the creek directly. The upsize at Smith Road Pump Station is recommended to be bored so impacts to the creek are minimized. There are no other alternatives that involve stream crossings.

Socio-Economic Conditions

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

4.6 LAND REQUIREMENTS

The City would need to purchase easements for Alternative 3 for the new HWY 30 crossing at Maple Street and along the Jay Davis property near Columbia Ave. The other alternatives and improvements presented in this chapter do not require the City to purchase new easements.

4.7 POTENTIAL CONSTRUCTION PROBLEMS

The depth of the water table and subsurface rock may affect 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 in Columbia County. Construction plans for any of the alternatives would also include provisions to control dust and runoff.

4.8 SUSTAINABILITY CONSIDERATIONS

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

Water and Energy Efficiency

Alternative 2 could potentially decrease water and energy usage depending on the changes made at the WTP. If the WTP can recycle some of their backwash flows, the water usage would decrease. If the flows are recycled or treated and divert away from the sewer, the energy usage at Smith Road Pump Station could decrease with the decreased flows.

Alternative 3 could potentially decrease energy usage at Smith Road Pump Station as some of the flow from the trunk line will flow by gravity in the new Maple Street crossing. This flow would not be pumped through the Smith Road Pump Station.

Green Infrastructure

Reducing the WTP flows to the sewer has potential to decrease usage at the WTP through recycling or use for other beneficial uses.

Other

The line from 4th Street to the Smith Road Pump Station runs parallel to the South Scappoose Creek. Constructability, cost, and impacts to the creek should be evaluated during the predesign phase to assess the optimal replacement strategy and impacts to the creek during construction.

CHAPTER 5 – SELECTION OF AN ALTERNATIVE

Alternatives were considered to address deficiencies noted in the previous sections. Detailed descriptions of each alternative are included in Chapter 4. Advantages, disadvantages, and comparative costs (where applicable) are presented for evaluating each alternative. The planning level cost estimates presented include common improvements between multiple alternatives. Detailed opinion of probable costs for each alternative are presented in Appendix F.

5.1 COMPARATIVE ANALYSIS (COSTS AND NON-MONETARY FACTORS)

5.1.1 Alternative 1 – Continue WTP Operations and Upsize Existing

Alternative 1 includes maintaining WTP operations as existing and upsizing existing trunk line from EM Watts to the Smith Road Pump Station. This alternative also includes improvements on Smith Road and the new Kucera line to eliminate the creek crossing on the north side of Veteran's Park. Figure 4.1 in Chapter 4 highlights improvements included in this alternative. The pipeline improvements include the section of pipeline that was exposed in the South Scappoose Creek bank in previous years due to flooding events. There could be additional creek bank stabilization required with this alternative. A preliminary opinion of probable cost is summarized in Table 5.1. This alternative is anticipated to include the highest peak flows going into the Smith Road Pump Station (in comparison to Alternatives 2 and 3) and thus is expected to have higher O&M costs associated with these higher flows.

5.1.2 Alternative 2 – Shift WTP Operations to Night and Upsize Existing

Alternative 2 includes modifying WTP operations to automatically meter WTP flows to the sewer system at night, during periods of low flow. This alternative would include upsizing the existing trunk line from SW 4th and Maple Streets to the Smith Road Pump Station, approximately 3,500 linear feet less upsizing compared to Alternative 1 due to shifting WTP operations to night-time. This alternative also includes improvements on Smith Road and the new Kucera line to eliminate the creek crossing on the north side of Veteran's Park similarly to Alternatives 1 and 3. Figure 4.2 in Chapter 4 highlights improvements included in this alternative. A preliminary opinion of probable cost is summarized in Table 5.1 with the costs of splitting the project as discussed in Chapter 4 included. The Keys WTP improvements to automate and control backwash flows at night are included in the Alternative 2 estimate (in the Keys WTP & Smith Road Improvements phase). This alternative anticipates higher peak flows going into the Smith Road Pump Station when compared with Alternative 3, but less than Alternative 1. Thus, it is anticipated that O&M costs for Alternative 2 will fall somewhere between Alternatives 1 and 3.

5.1.3 Alternative 3 – Construct New HWY 30 Crossing at Maple Street and Shift WTP Operations to Night

Alternative 3 includes improvements to the Keys WTP to meter flows to the sewer during nighttime, low flows and constructing a new gravity pipeline on Maple Street crossing under the highway and railroad. This alternative also includes improvements on Smith Road and the new Kucera line to eliminate the creek crossing on the north side of Veteran's Park. Alternative 3 includes trenchless lining of the existing 12-inch line from SW 4th and Maple Streets to the Smith Road Pump Station as this pipeline is nearing the end of its useful life. The extents of the alternative are shown in Figure 4.3 in Chapter 4. A preliminary opinion of probable cost is summarized in Table 5.1. This alternative has the lowest flow to the Smith Road Pump Station and thus lowest O&M costs for operating the pump station. The new gravity line would add additional linear feet of pipeline to the system that would require maintenance. If existing services are reconnected to the new line and not run parallel, the additional linear feet of pipeline would be minimized.

TABLE 5.1 – SMITH ROAD PUMP STATION BASIN ALT. COST ESTIMATES

Improvement	Project Estimate (Rounded)
Alternative 1	$ 3,261,000
Alternative 2	$ 1,866,000
Alternative 3	$ 4,014,000

The recommended alternative to address capacity limitations in the Smith Road Pump Station basin is Alternative 2, shift Keys WTP flows to nighttime, low flow periods and upsize the existing line. This alternative has the lowest capital cost of the alternatives and impacts the shortest length of pipeline. The improvements can be separated into two phases. The first including WTP, Kucera line, and Smith Road improvements. The second phase would include upsizing and rehabilitation from SW 4th and Maple Streets to the Smith Road Pump Station.

5.1.4 High School Way Basin Improvements – Upsize Existing

As presented in Chapter 4, multiple alternatives to address capacity limitations in the High School Way Basin were not evaluated because of feasibility or costs. Improvements to the basins include upsizing the existing line on Old Portland Road, High School Way, and 6th Street to Tussing Way along the east trunk line. Improvements would include approximately 4,200 LF of 10-inch pipe and 2,100 LF of 15-inch pipe. This alternative includes approximately 500 LF of work along HWY 30. The extents of the alternative are shown in Figure 4.4 in Chapter 4. The project has been separated into three main sections – Old Portland Road, High School Way to Elm St, and Dutch Canyon Rd. The associated opinion of probable costs of these improvements are shown in Table 6.3. As the City begins projects in these areas, projects could be combined to provide potential cost savings to the City. Detailed cost estimates are in Appendix F.

5.1.5 Miscellaneous Improvements

Additional improvements include Laurel and 3rd Streets, Tyler Street, and Wagner Court as described in Chapter 4 and improvements at Miller WTP. The improvements at Miller WTP include adding automation and controls to the backwash pumps to facilitate nighttime discharge of the backwash water to the sewer system to reduce peak flows in the sewer line. The associated opinion of probable costs of these improvements are shown in Table 6.3. As the City begins projects in these areas, projects could be combined to provide potential cost savings to the City. Detailed cost estimates are in Appendix F.

CHAPTER 6 – PROPOSED PROJECT (RECOMMENDED ALTERNATIVES)

This section consists of the recommended plan to address the wastewater collection system deficiencies identified in previous chapters. A location map showing the improvements to the collection system is shown in Figure 15 (Appendix A).

6.1 PRELIMINARY PROJECT DESIGN

6.1.1 Collection System

The proposed collection system projects are summarized below.

- Smith Road Pump Station improvements
- WTP nighttime flows, Kucera new pipeline, Smith Road pipeline upsizing
- 4th Street to Smith Road Pump Station pipeline upsizing and rehabilitation
- High School Way basin pipeline upsizing
    - Old Portland Road pipeline
    - High School Wy to Elm St pipeline
    - Dutch Canyon Road pipeline
- Laurel and 3rd Streets pipeline upsizing
- Tyler Street pipeline upsizing
- Wagner Court pipeline upsizing
- Additional Pump Station improvements

Detailed project descriptions are included in Chapter 5. Detailed opinion of probable cost for each project can be found in Appendix F.

6.1.2 Pipeline Cleaning and CCTV

Cleaning and CCTV inspection work has been subcontracted out in the past. The City currently cleans and inspects 20-25% of the collection system each year. The City has not experienced challenges with debris buildup or conditions that require more frequent cleaning and inspection. If areas are identified as problematic, the City may want to clean and inspect these areas every year or two, or more regularly if required. Areas with adverse grades or large sags may require more frequent attention.

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

6.1.3 Service Lines

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

6.1.4 Flow Monitoring

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

6.1.5 Pipeline Replacement Program

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

Keller Associates recommends the City begin budgeting for replacement/rehabilitation of an average of 2,300 feet of the collection pipeline system each year. This amount would allow replacement of the entire system within approx. 75 years, the estimated useful life of pipelines. Concrete pipes in the system should be replaced first. The linear feet of pipeline and number of manholes replaced annually is an average and should be adjusted based on future CCTV and other maintenance records. The costs associated with funding an on-going replacement and rehabilitation program are summarized in Table 6.1 and discussed further in Section 6.6.

TABLE 6.1: REPLACEMENT BUDGETS

Item	Lifespan	Cost/Year
Pipelines	75 year	$ 519,000
Manholes	50 year	$ 109,000
Cleanouts	50 year	$ 4,000
Laterals/Cleanouts	50 year	$ 85,500
	Total	**$ 717,500**

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

6.1.6 Pump Station Improvements

Pump station improvements are included in the detailed opinion of probable costs (Appendix F). Smith Road, Springlake and Highway 30 pump station improvements are in Priority 1. The Keys Landing and Seven Oaks pump station modifications are part of Priority 2. The opinion of probable costs are summarized in Table 6.2 below. It is recommended that at the end of useful life for each Highway 30, Seven Oaks, and Keys Landing pump stations, evaluations be completed to assess the benefits and cost of converting the stations to submersible pump stations. This could improve future maintenance access to pumps and associated piping and valving.

TABLE 6.2: PUMP STATION IMPROVEMENT ESTIMATES

Pump Station Improvements	Project Estimate (Rounded)
Smith Road Pump Station	$ 883,000
Springlake	$ 46,000
Highway 30	$ 38,000
Seven Oaks	$ 44,000
Keys Landing	$ 105,000

6.2 SUSTAINABILITY CONSIDERATIONS

6.2.1 Water and Energy Efficiency

The proposed Priority 2 improvements along 4th St to the Smith Road Pump Station is anticipated to reduce I/I flows to the pump station, which would provide energy savings at the pump station. Other pipe replacements may also reduce I/I flows in the system and could provide energy savings at the WWTP.

6.2.2 Green Infrastructure

Recommendations of this report include reducing I/I. This diverts storm water to its natural course instead of it ending up in the sanitary sewer, and eventually the WWTP. Reducing I/I also decreases associated energy required to carry and treat the water.

6.2.3 Other

The proposed alternatives reduce peak flows through the collection system by removing WTP sewer contributions to periods of low flow. This results in lower capacity demands on pipelines, pump stations, and the treatment plant.

6.3 ENGINEER'S OPINION OF PROBABLE COST

The summary of the Scappoose collection system improvement costs is in Table 6.3 (Capital Improvement Plan (CIP)). The percent SDC eligibility factored in the existing design flow, existing capacity, and capacity after the improvements are completed. The amount of capacity that can be utilized for future connections is divided by the future capacity in 2040. For projects that did not have an increase in flows, the percent SDC eligible is derived from the percent growth in population over the 20-year planning period. 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 project development, pre-design, and design phases. Individual project sheets for Priority 1 projects are included in Appendix H. Each project sheet consists of a project objective, description, location map, and cost estimate.

TABLE 6.3: 20-YEAR CAPITAL IMPROVEMENT PLAN

ID #	Item	Total Estimated Cost (2020)	SDC Growth Portion %	SDC Growth Portion Cost	City's Estimated Portion
Priority 1 Improvements					
1.1	Smith Road Pump Station	$ 883,000	24%	$ 215,000	$ 668,000
1.2	WTP Improvements and Smith Rd	$ 908,000	40%	$ 364,000	$ 544,000
1.3	Laurel and 3rd St	$ 594,000	18%	$ 109,000	$ 485,000
1.4	Old Portland Rd	$ 1,398,000	10%	$ 145,000	$ 1,253,000
1.5	High School Wy to Elm St	$ 1,412,000	12%	$ 165,000	$ 1,247,000
1.6	Dutch Canyon Rd	$ 285,000	2%	$ 5,000	$ 280,000
1.7	Pump Station Improvements (Springlake and HWY 30 PS)	$ 84,000	36%	$ 30,000	$ 54,000
Total Priority 1 Improvements (rounded)		***$ 5,564,000***		***$ 1,033,000***	***$ 4,531,000***
Rate Impact (20 yr, 1.6%)					***$ 8.70***
Priority 2 Improvements					
2.1	4th St to Smith Rd Pump Station	$ 958,000	13%	$ 126,000	$ 832,000
2.2	Tyler St	$ 303,000	13%	$ 40,000	$ 263,000
2.3	Wagner Ct	$ 126,000	100%	$ 126,000	$ -
2.4	Pump Station Improvements (Keys Landing and Seven Oaks PS)	$ 149,000	36%	$ 53,000	$ 96,000
2.5	Miller WTP Improvements	$ 49,000	36%	$ 17,000	$ 32,000
Total Priority 2 Improvements (rounded)		***$ 1,585,000***		***$ 362,000***	***$ 1,223,000***
Rate Impact (20 yr, 1.6%)					***$ 2.35***
TOTAL COLLECTION SYS. IMPROVEMENTS COSTS (rounded)		**$ 7,149,000**			**$ 5,754,000**

Notes:

1. The opinion of probable cost herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2020 dollars and does not include escalation to time of actual construction. Keller Associates has no control over variances in the cost of labor, materials, equipment, services provided by others, contractor's methods of determining prices, competitive bidding or market conditions, practices or bidding strategies. Keller Associates cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the cost presented herein.
2. All costs in 2020 Dollars. Costs include mobilization (5%), contractor overhead and profit (OHP; 15%), contingency (30%), engineering and construction management services (CMS; 20-30%), and legal, administrative, and permitting services (2%).
3. The Capital Improvement Plan does not include annual pipeline replacement, pipeline cleaning and inspection, and lift station maintenance budgets. These budgets are discussed in Section 6.6.

6.4 PROJECT SCHEDULE

An estimated schedule for Priority 1 improvements is shown in Table 6.4. Individual schedules for each project will be further refined at a later date by the City during the pre-design phase for each proposed improvement. Costs presented here are planning-level estimates. Actual costs may vary depending on market conditions and must be updated as projects are further refined in the project development, pre-design, and design phases.

TABLE 6.4: PRIORITY 1 CAPITAL IMPROVEMENT PLAN

ID #	Item	Total Estimated Cost (2020)	Opinion of Probable Costs (2020) 2021	2022	2023	2024
Priority 1 Improvements						
1.1	Smith Road Pump Station	$ 883,000	$ -	$ 883,000	$ -	$ -
1.2	WTP Improvements and Smith Rd	$ 908,000	$ -	$ 181,600	$ 726,400	$ -
1.3	Laurel and 3rd St	$ 594,000	$ -	$ 118,800	$ 475,200	$ -
1.4	Old Portland Rd	$ 1,398,000	$ -	$ -	$ 279,600	$ 1,118,400
1.5	High School Wy to Elm St	$ 1,412,000	$ -	$ -	$ 282,400	$ 1,129,600
1.6	Dutch Canyon Rd	$ 285,000	$ -	$ -	$ 57,000	$ 228,000
1.7	Pump Station Improvements	$ 84,000	$ -	$ 84,000	$ -	$ -
Total Priority 1 Improvements (rounded)		***$ 5,564,000***	***$ -***	***$ 1,267,400***	***$ 1,820,600***	***$ 2,476,000***

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

6.5 FINANCIAL STATUS OF EXISTING FACILITIES

The financial information statement for the City of Scappoose sewer utility is located in Appendix G. This information includes the latest two resolutions regarding sewer rates. The current basic residential rate is $55.96 plus commodity per month. Estimated total wastewater fund resources for the 2020-2021 fiscal year are approximately $5,056,130. The annual charges for services are estimated to be $2,171,125 of the $5,056,130.

Annual O&M costs for the wastewater system, separated by type of expense, are also shown in Appendix G. For the 2020-2021 fiscal year, the estimated total expenditures for the wastewater fund is approximately $4,265,130.

The City took out construction loans for the sewer utility in 2011 and 2014. The 2011 loan is through DEQ with 0.00% interest that will mature in 2031. Its balance as of July 1, 2020, was $185,231. The 2014 loan is a $700,000 loan from US Bank, with an interest rate of 2.47%. That loan matures in 2021 and has a balance (as of July 1, 2020) of $100,000. The annual debt service of both loans is $119,803. The DEQ loan has a holding or reserve requirement of one half of the average annual debt service – $8,821. The US Bank loan does not have any holding requirements.

The City plans to take out a loan from the DEQ Clean Water State Revolving Fund (CWSRF) for $6,430,00 to cover Phase I of the WWTP improvements. The loan is anticipated to be for 30 years at 2.01% interest amortized over a 15-year period with an annual fee of 0.5% of the outstanding loan amount. There is the opportunity for $500,000 of principal forgiveness at the completion of the project. The City anticipates the first payment for this loan will be due in spring of 2021 of an estimated $180,000. The required loan reserve is estimated to be around $250,000.

6.6 ANNUAL OPERATING BUDGET

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

6.6.1 Potential User Rate Impacts

The existing sewer rate schedule consists of a monthly flat rate fee of $55.96 per EDU plus commodity. Operators indicate that commodity fees typically constitute 4% of system fees. The current number of EDUs is 2,553.

The rate impacts assume that none of the existing revenue/budget can be used annually to offset future capital improvements or sewer rehabilitation/replacement budgets. The City will continue to evaluate this as individual projects progress.

Table 6.4 shows the existing and potential monthly charge for sewer services for one EDU. 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 (20-year, 1.6%) made through a user rate increase. Calculations for the user rate impact are referenced in Appendix G. The user rate impacts assume that SDC funds or commercial/industrial development contributions will be available for Priority 1 (urgent need). The user rate impacts also assume, as directed by City staff, there is no surplus in the annual budget contributing to the annual debt service payment.

In the event that grant funds, low-interest loans, or principal forgiveness could be obtained, then the user rate impacts would be less than those illustrated in Table 6.5. Keller Associates recommends that the City actively pursue these opportunities in order to mitigate user rate impacts.

TABLE 6.5: USER RATE IMPACT

	Annual Payment (20 years, 1.6%)	User Rate Increase	User Rate Total
Existing User Rate	$ -	$ -	$ 55.96
Prioritiy 1 Improvements	$ 266,520	$ 8.70	$ 64.66

The anticipated user rate impacts presented in Table 6.5 do not account for recommended replacement/rehabilitation or short-lived asset replacement budgets. These budget items will require further analyzing by the City's financial consultant as part of a full-rate study. The rate impacts for pipeline replacement/rehabilitation are usually delayed until after the capital improvement plan for the collection system is completed. This is due to the fact that the CIP should be replacing more than the average linear footage recommended for the annual replacement/rehabilitation. Keller Associates does not anticipate that future collection system improvements will increase the City's O&M requirements. In fact, with new facilities and pipelines, it may be the case that maintenance demands for the collection system could decrease slightly. It should be noted that all costs are in 2020 dollars. The City currently increases user rates by the Engineering News-Record (ENR) Construction Cost Index (CCI) for the 20-City Annual Average, which historically has been approximately 3%.

6.6.2 System Development Charges

The City's current sewer System Development Charge (SDC) for a single-family home is $4,276.04. The sewer SDC is typically divided into various components: reimbursement, growth, and administrative. The scope of this study included estimating the SDC eligibility for each identified capital improvement. It is the intent that this information will be utilized by the City's financial consultant to update the City's SDCs. The estimated SDC eligibility for each identified capital improvement is shown in Table 6.3.

6.6.3 Annual O&M Costs

In addition to the capital improvement costs presented in Table 6.3, the following expected annual operating costs are recommended for consideration in setting annual budgets:

- Additional collection system replacement/rehabilitation needs: City should eventually budget an additional $717,500/year (to be either contracted out or completed using City crews).
- Collection system cleaning and CCTV needs: following the timeline described in Section 6.1.2, the City should budget approximately $112,000/year.