CITY OF ST. HELENS, OR
WASTEWATER MASTER PLAN

REGISTERED PROFESSIONAL ENGINEER
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OREGON
NOV. 13, 2018
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EXPIRES: 12/31/2023

NOVEMBER 2021

KA PROJECT NO. 220060-002 | CITY PROJECT NO. P-511

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City of St. Helens

RESOLUTION NO. 1940

A RESOLUTION ADOPTING THE ST. HELENS WASTEWATER MASTER PLAN

WHEREAS, the last complete update to the City's Wastewater Collection System Master Plan was in April 1989; and

WHEREAS, ORS 197.712(2)(e) requires a city to develop and adopt public facility plans for areas within their urban growth boundary containing a population greater than 2,500 persons; and

WHEREAS, the City of St. Helens Municipal Code 19.08.030 Public Services And Facilities Goals promote the development of an orderly arrangement of public facilities and services to serve as a framework for urban development, and the designing and locating public facilities so that capacities are related to future as well as present demands, that ample land is available for building and plant expansion, and that public works plants and utility structures reflect due regard for their environmental impact; and

WHEREAS, an updated Wastewater Collection System Master Plan is needed to provide for growth and planning for future development; and

WHEREAS, Engineering consultant, Keller Associates, has prepared an updated Wastewater Collection System Master Plan, attached as Exhibit A, and has presented said plan to the Planning Commission on October 12, 2021 and to the City Council at the November 3, 2021 Work Session; and

WHEREAS, consultant has prepared the St. Helens Wastewater Collection System Master Plan after extensive review and analysis of existing plans, policies, studies and other information, and has afforded all interested parties opportunity to review the plan.

NOW, THEREFORE, THE CITY OF ST. HELENS RESOLVES that the St. Helens Wastewater Collection System Master Plan, attached as Exhibit A, is adopted and shall be used as a guide for the development and implementation of a complete, wastewater collection system.

APPROVED AND ADOPTED by the City Council on November 17, 2021 by the following vote:

Ayes: Morten, Topaz, Chilton, Birkle, Scholl

Nays: None

Rick Scholl, Mayor

ATTEST:

Kathy Payne

Kathy Payne, City Recorder

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ACRONYMS AND ABBREVIATIONS

AACE	Associate for the Advancement of Cost Engineering
AADF	Average Annual Daily Flow
ac	Acre
AC	Asbestos Cement
ADWF	Average Dry Weather Flow
AWWF	Average Wet Weather Flow
BLM	Bureau of Land Management
CCTV	Closed-Circuit-Television
CFS	Cubic Feet per Second
CIP	Capital Improvement Plan
CIPP	Cured-in-Place Pipe
CMOM	Capacity Management, Operation, and Maintenance
CMP	Corrugated Metal Pipe
C/O	Cleanouts
CWSRF	Clean Water State Revolving Fund
DEQ	Department of Environmental Quality
DI	Ductile Iron
DOGAMI	Department of Geology and Mineral Industries
DSL	Department of State Lands
DWF	Dry Weather Flow
d/D	Maximum Depth Divided by Full Depth
EDU	Equivalent Dwelling Unit
EPA	Environmental Protection Agency
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
FOG	Fats, Oils and Grease
fps	Feet per Second
FRP	Fiberglass Pipe
FTE	Full Time Equivalent
GIS	Geographical Information System
gpad	Gallons per Acre per Day
gpcd	Gallons per Capita per Day
gpd	Gallons per Day
GPM	Gallons per Minute
GW	Greenway Basin
HDPE	High-Density Polyethylene

HGL	Hydraulic Grade Line
HOA	Hand/Off/Auto
HVAC	Heating, Ventilation and Air Conditioning
I/I	Infiltration and Inflow
LF	Linear Feet
LID	Low Impact Development
LOS	Level of Service
LWI	Local Wetlands Inventory
MG	Million Gallons
MGD	Million Gallons per Day
MGY	Million Gallons per Year
MH	Manhole
MMDWF	Maximum Monthly Dry Weather Flow
MMWWF	Maximum Monthly Wet Weather Flow
MMF	Maximum Month Flow
MS4	Municipal Separate Storm Sewer System
NAVD88	North American Vertical Datum of 1988
NGVD29	National Geodetic Vertical Datum of 1929
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollution Discharge Elimination System
NRCS	Natural Resources Conservation Service
OAR	Oregon Administrative Rules
ODOT	Oregon Department of Transportation
ODSL	Oregon Department of State Lands
O&M	Operations and Maintenance
OH&P	Overhead and Profit
ORS	Oregon Revised Statutes
PACP	Pipeline Assessment Certification Program
PDAF	Peak Daily Average Flow
PDF	Peak Day Flow
PF	Peak Factors
PHF	Peak Hour Flow
PIF	Peak Instant Flow
PLC	Programmable Logic Controller
PVC	Polyvinyl Chloride
PW	Public Works
PWDS	Public Works Design Standards
PWkF	Peak Week Flow
RCP	Reinforced Concrete Pipe

RDII	Rainfall-Derived Infiltration and Inflow
ROW	Right-of-Way
SBUH	Santa Barbara Unit Hydrograph Method
SCADA	Supervisory Control and Data Acquisition
SCS	Soil Conservation Service
SDC	System Development Charge
SHMC	St. Helens Municipal Code
SHPO	State Historic Preservation Office
SRF	State Revolving Fund
SWMM	Stormwater Management Model
TDH	Total Dynamic Head
TMDL	Total Maximum Daily Load
UGB	Urban Growth Boundary
USACE	United States Army Corp of Engineers
USFWS	United States Fish and Wildlife Service
USGS	United States Geological Survey
VCP	Vitrified Clay Pipe
VFD	Variable Frequency Drive
WQMP	Water Quality Management Plan
WWF	Wet Weather Flow
WWTP	Wastewater Treatment Plant

SECTION 1 - EXECUTIVE SUMMARY

In 2020, the City of St. Helens, Oregon (City), contracted with Keller Associates, Inc. (Keller) to complete a wastewater master plan (WWMP) for the City's wastewater collection system. The study area consists of all areas within the City's Urban Growth Boundary (UGB). This section summarizes the major findings of the wastewater master plan, including brief discussions of alternatives considered and final recommendations.

1.1 PLANNING CRITERIA

City-defined goals and objectives, Public Works Design Standards (PWDS), engineering best practices, and regulatory requirements form the basis for evaluation and planning within this study. Applicable regulatory requirements include the Oregon Department of Environmental Quality (DEQ) Pump Station Regulatory Requirements, Capacity Management, Operation and Maintenance (CMOM) Guidance, Land Use and Comprehensive Plan Requirements, and City Municipal Code.

The capacity of the City's conveyance system is based on the ability of the system to convey projected 20-year peak instantaneous flow rates associated with the 5-year, 24-hour storm event. For the collection system model evaluation, pipes are considered at capacity when peak flows exceed 85% of full depth in accordance with industry standards. When sizing gravity collection systems, pipelines shall be sized to convey 20-year, projected peak flows at 85% or less depth to diameter ratio (d/D). Pump stations will be evaluated and sized (if necessary) to handle these peak flows with the largest pump out of service (defined as firm capacity).

1.2 PLANNING CONDITIONS

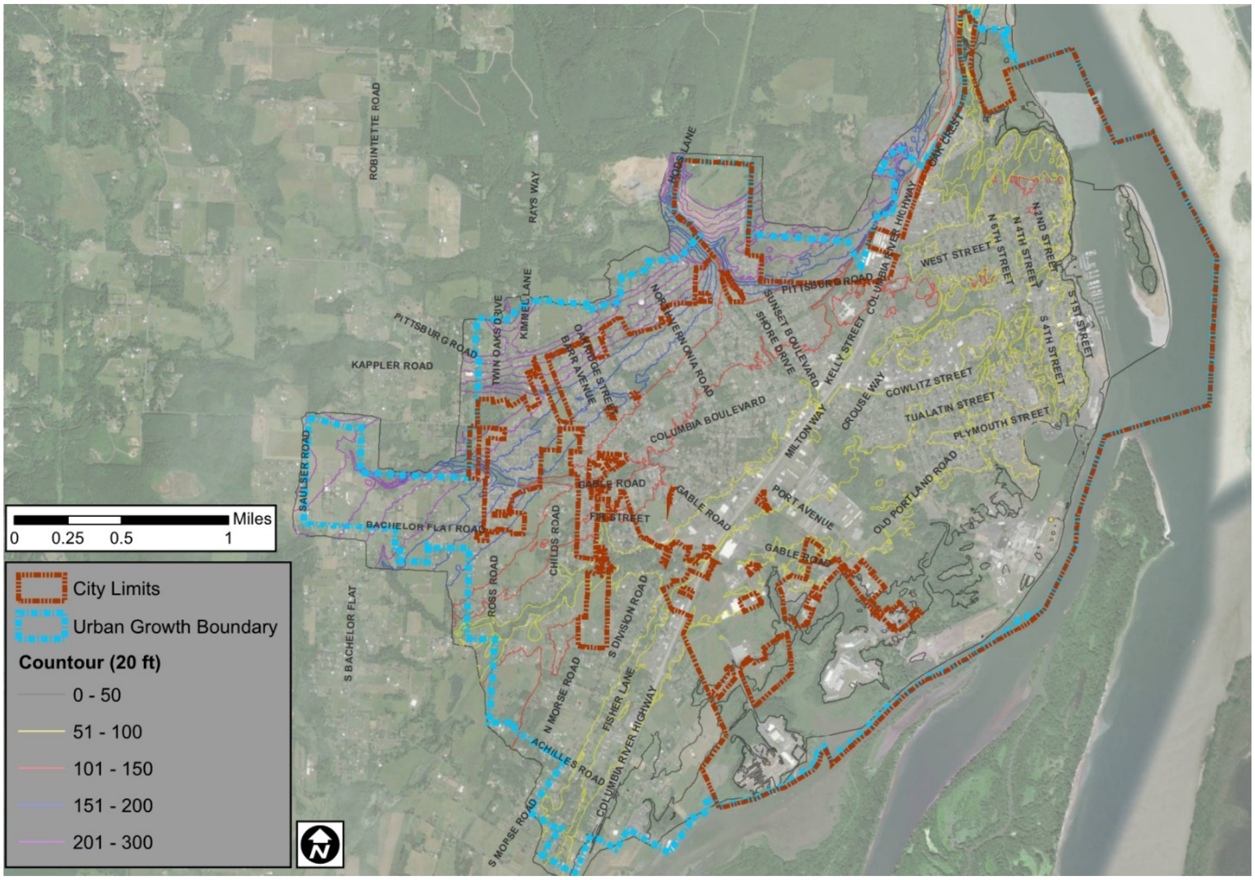
1.2.1 STUDY AREA AND LAND USE

The study area, consisting of the City's UGB and general topography, are shown in Figure 1-1. The study area slopes to the south and east toward the Columbia River. The City of St. Helens owns and operates a wastewater collection system within its UGB. Columbia City's wastewater collection system discharges to and flows through the St. Helens collection system to the City's Wastewater Treatment Plant (WWTP) for treatment. Evaluation of the Columbia City system, aside from the impacts of population growth and infiltration and inflow (I/I) on the St. Helens system, is not included in the scope of this study. The wastewater system currently serves only areas within the St. Helens and Columbia City UGBs. Further expansion of the UGB was not considered in this report.

1.2.2 DEMOGRAPHICS

The City's population has been increasing at a steady rate over the past few decades but has leveled out in recent years. Historical populations for the City of St. Helens and Columbia City were obtained from the U.S. Census and Columbia County in cooperation with Portland State University (PSU). PSU analyzes historical trends and anticipates growth patterns to develop growth rates for 5-year increments. The most current population estimate provided by PSU for the combined area of St. Helens and Columbia City was 15,895 in 2020. The PSU coordinated growth rates provide a population projection for 2040 to be 19,506, which is St. Helens and Columbia City combined. These growth rates were reviewed and approved by the technical advisory committee (TAC) for this planning study. The estimated average annual growth rate from 2019 to 2040 is approximately 1.1% for St. Helens and 0.5% for Columbia City.

FIGURE 1-1: CITY LIMITS, UGB, AND TOPOGRAPHY



1.2.3 WASTEWATER FLOWS

Historical wastewater flows were evaluated using statistical methods following DEQ guidance to develop planning flows and provide flow projections for the planning period. Observed flows for each year from 2015–2019 and planning flows are summarized in Table 1-1 below. During the system flow evaluation process, it was discovered that the current influent flow measurement at the WWTP may not reliably measure peak influent flows during high flow events. The City provided direction to review available data, use engineering judgement, and estimate system flow planning criteria values to reflect the current system demand. Modified planning criteria was established and is presented in Table 1-1.

TABLE 1-1: OBSERVED HISTORICAL FLOWS & PLANNING FLOWS

St. Helens Historical Flows (MGD[1])								
Year	2015	2016	2017	2018	2019	5-Year Avg	Planning	Modified Planning
Population	15,050	15,085	15,225	15,225	15,395		15,895	15,895
ADWF	0.98	1.31	1.25	0.95	1.09	1.11	1.11	1.11
$MMDWF_{10}$	2.71	2.56	2.87	3.03	2.79	2.79	3.03	3.03
AADF	2.35	2.43	2.64	1.92	1.85	2.24	2.24	2.24
AWWF	3.73	3.56	4.01	2.90	2.59	3.36	3.36	3.36
$MMWWF_5$	7.88	7.81	5.84	4.46	3.99	5.99	7.88	7.88
PWkF	14.19	7.54	8.93	5.90	8.86	9.08	14.19	14.19
$PDAF_5$	21.19	13.08	17.76	9.60	21.90	16.71	21.90	19.90
PIF_5	31.4	27.4	24.6	13.9	32.2	25.90	33.98	26.00
Yearly Total (MG[1])	856	889	955	700	669			
Total Rainfall (in/yr)	47	48	51	31	33			

1) MGD = million gallons per day; MG = million gallons

ADWF = Average Dry-Weather Flow
$MMDWF_{10}$ = Maximum Monthly Dry-Weather Flow
AADF = Average Annual Daily Flow
AWWF = Average Wet-Weather Flow
$MMWWF_5$ = Maximum Monthly Wet-Weather Flow
PWkF = Peak Week Flow
$PDAF_5$ = Peak Daily Average Flow
PIF_5 = Peak Instantaneous Flow

Comparison of the dry weather and wet weather system flows in Table 1-1 shows that the City of St. Helens experiences large increases in flow during wet weather events. The high wet weather flows are associated with large inflow and infiltration (I/I) influence in the system.

To project the planning flows derived from the analysis, a projected flow per capita (reported in gallons per capita per day, [gpcd]) was developed. Projected planning system flows (millions of gallons per day [MGD]) are based on 2019 modified planning flows with the addition of the product of projected unit flows (gpcd) and projected population increase shown in Table 1-2. Actual future flows will depend on several variables and could potentially be decreased through aggressive I/I reduction efforts.

TABLE 1-2: PROJECTED PLANNING FLOWS

	Planning Flow (MGD)	Planning Unit Flow (gpcd)	Projected Unit Flow (gpcd)	Projected Planning Flow (MGD)				
Year	2019	2019	2019	2020	2025	2030	2035	2040
Population	15,395	15,395	15,395	15,895	16,727	17,605	18,530	19,506
ADWF	1.11	72	72	1.15	1.21	1.28	1.34	1.41
$MMDWF_{10}$	3.03	197	197	3.12	3.29	3.46	3.64	3.83
AADF	2.24	145	145	2.31	2.43	2.56	2.69	2.83
AWWF	3.36	218	218	3.47	3.65	3.84	4.04	4.25
$MMWWF_5$	7.88	512	300	8.03	8.28	8.54	8.82	9.11
PWkF	14.19	922	325	14.35	14.62	14.91	15.21	15.53
$PDAF_5$	19.90	1293	375	20.09	20.40	20.73	21.08	21.44
PIF_5	26.00	1689	525	26.26	26.70	27.16	27.65	28.16

1.3 COLLECTION SYSTEM EVALUATION

The existing wastewater collection system consists of approximately 60 miles of gravity sewer mains, 2.5 miles of force main, and nine pump stations.

1.3.1 PUMP STATION EVALUATION

High level facility evaluations were completed in October of 2020 with City operations personnel to review conditions of the pump station facilities, current maintenance activities, and known operational problems encountered by City staff.

Each pump station is a duplex pump station with submersible pumps located in the wetwell, with the exception of Pump Station 2 (PS#2). PS#2 is a duplex self-priming pump station that operates on a variable frequency drive (VFD) with a high and low setting. Table 1-3 below provides a summary for the pump stations evaluated.

TABLE 1-3: PUMP STATION INVENTORY

Name	PS#1	PS#2	PS#3	PS#4	PS#5	PS#7	PS#8	PS#9	PS#11
Type	Duplex, Submersible	Duplex, Self-Priming	Duplex, Submersible	Duplex Submersible	Duplex, Submersible	Duplex, Submersible	Duplex, Submersible	Duplex, Submersible	Duplex, Submersible
Year Constructed	1950s	1990	1997	1995	1994	1986	1991	1994	1996
Pump Type	Paco / Hydromatic Submersible	Gorman Rupps VSP (High / Low)	Wilo Type FA 10.51A Submersible	FLYGT NP - 3085	ABS AFP AFP(K) 1049.1-M105/4FM	Wilo Submersible	ABS SJS10W	Barns 4SE3724L	Hydromatic S4HVX-1500JD
Pump hp	36 / 30	40 / 22.5	6.2	3	14	15.5	1	3.7	15
Design Flow (gpm)	550	700 / 250	500	130	145	390	Unknown	200	143
Design Head (ft)	110	82 / 52	10.7	22	98	83	4	24	74
Low Level Alarm (ft)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.42	N/A
Pump Off Level (ft)	1.33	1.50	2	6.2	2.00	3.83	2.83	0.58	0.75
Lead On Level (ft)	2	3	3.5	8.9	4.00	10.00	4.93	1.167	1.65
Lag On Level (ft)	2.5	3.5	4.33	10.0	5.00	10.5	Unknown	2.75	2
High Level Alarm (ft)	6	7.5	5.83	11.8	5.00	11	5.45	3.75	3.1
Level Control Type	Ultrasonic Level Sensor	Ultrasonic Level Sensor	Ultrasonic Level Sensor	Float Relays	Ultrasonic Level Senor	Ultrasonic Level Sensor	Float Relays	Float Relays	Float Relays
Flow Meter	No	No	No	No	No	No	No	No	No
Pressure Gauge	Yes	No	No	No	No	No	No	No	No
Auxiliary Power Type	Portable Generator	On-Site Generator	Portable Generator	Portable Generator	On-site Generator	On-site Generator	Portable Generator	Portable Generator	Portable Generator
Transfer Switch	MTS	ATS	MTS	MTS	ATS	ATS	MTS	MTS	MTS
Bypass Piping	No	No	No	Yes	No	No	No	No	No
Oder Control	None	None	None	None	None	None	None	None	None
Wet Well Depth (ft)	18	9	15.5	20.6	10.5	16	4	13	6.15
Wet Well Diameter (ft)*	12.67	8	7	6	6	6	3	5	5
Force main Diameter (in)	6	6	6	4	4	6 / 8	3	6	4
Force Main Length (ft)**	1,010	1,050	20	610	1,700	2,620	260	70	2,500

*Pump Station 1 has a rectangular wetwell
**Estimated using City GIS data

The pump station evaluation presents general observations and recommendations, along with specific recommendations for individual pump station sites. The general recommendations are provided as a guideline to allow the City to maintain the pump stations for the 20-year planning period. Overall, the pump stations are in good condition and are well maintained with minor housekeeping items such as partial installation of redundant high-level alarms, lack of fall protection, and lack of up-to-date accurate pump station drawings and pump information. These housekeeping items were identified during observations and discussions with City staff. No significant deficiencies were identified in the overall pump station condition evaluation.

1.3.2 INFILTRATION & INFLOW

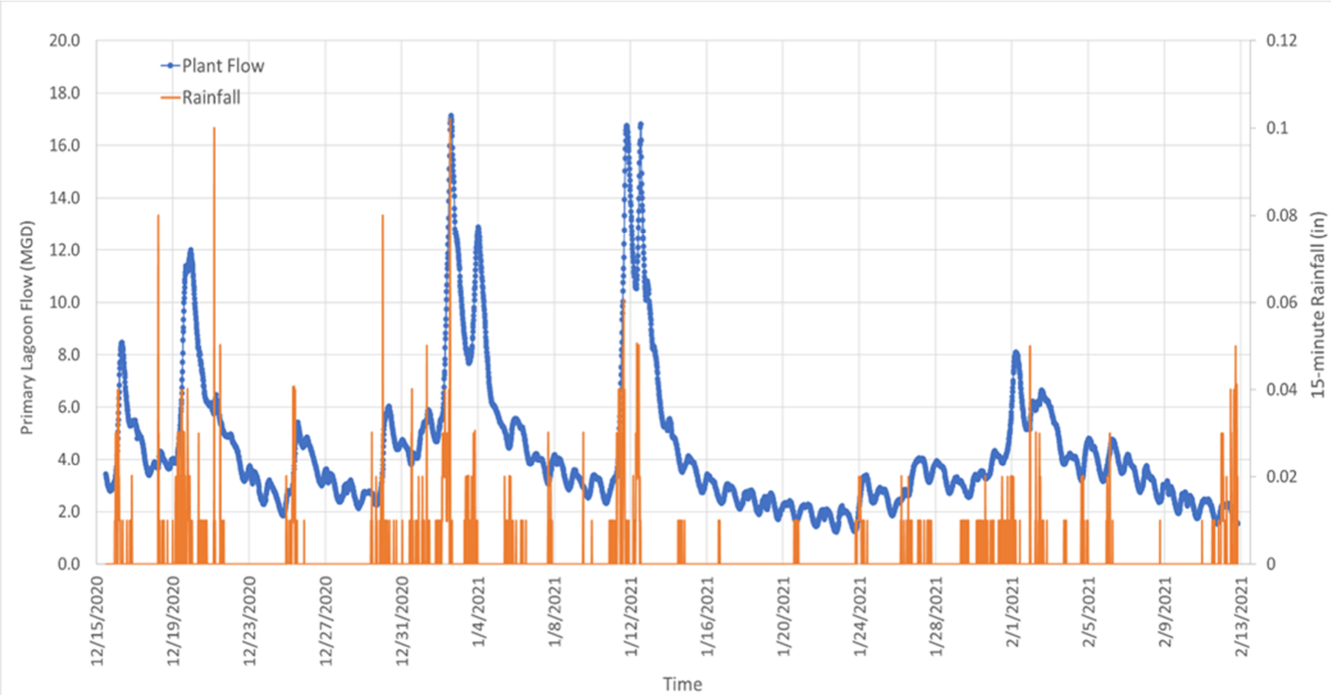
Infiltration and Inflow (I/I) is a concern in the St. Helens collection system. The rapid response between precipitation events and increased flows suggests that a significant component of peak flow is from storm water inflow. Estimated peak flows in the collection system are 20-25 times higher than annual dry weather flows. The sustained increase in flow over several days following a large storm event suggests that groundwater is also infiltrating into the City's wastewater collection system. Visual evidence of I/I influence in the system can be seen in Chart 1-1, which displays WWTP primary lagoon flow vs. 15-minute rainfall data for December 2020 through February 2021. The data is representative of typical wet weather seasonal response in the collection system.

Since the completion of the 2008 Wet Weather Capacity Evaluation, which documented I/I in St. Helens, the City has performed smoke testing and closed-circuit television (CCTV) inspections on the collection system. The City has also taken steps to address I/I in the system via pipeline replacement, pipe repair (including cure-in-place-pipe [CIPP] lining and spot repairs), and manhole rehabilitation and replacement. City staff have reported that the effort has produced noticeable I/I reduction (annual reported overflows have been reduced), but I/I still persists in the system.

This study included a high-level evaluation of I/I in the system. A preliminary evaluation to identify areas likely to experience the highest I/I was completed using available data. Pipeline age and material data, areas of suspected sump pump connections, City reported issues, and priority pipelines from the 2008 evaluation not addressed in the I/I reduction projects were compared to identify areas anticipated to have the highest I/I influence. The pipelines identified as highest risk for I/I should be considered as high priority for CCTV inspection and subsequent repair and/or replacement as needed. Overall, the evaluation identified approximately 8,000 feet of Priority 1 pipelines; 15,200 feet of Priority 2 pipelines; and 18,250 feet of Priority 3 pipelines for CCTV inspection. The primary area identified by City staff as likely to have improper stormwater sump pump connections was marked for additional investigations in order to locate and disconnect any stormwater sump pumps.

I/I prioritization and identification is an ongoing, evolving process. As the City collects more data, the prioritization evaluation needs to be updated to reflect the most recent data available. It is recommended the City work towards regular inspection of all system pipes and include this information in their ongoing I/I prioritization process.

CHART 1-1: DAILY FLOW AND PRECIPITATION DURING WET WEATHER



1.3.3 STAFFING EVALUATION

A high-level evaluation of existing wastewater staffing levels, deficiencies in existing staffing levels, and staffing recommendations was completed as part of this study. The City Public Works (PW) Operations staff, who are responsible for the operations and maintenance (O&M) of the wastewater collection system, and the WWTP staff, who are responsible for the O&M of the City's nine pump stations, were interviewed to collect information on existing staffing levels, annual O&M activities, and level of service (LOS) goals for the City wastewater infrastructure. In general, St. Helens' public works staff provide support for many City activities that are not directly related to public utility O&M (i.e. building maintenance, building remodels, City events, etc.), which reduces time and O&M activities they can spend and complete on utility infrastructure. It is recommended that either additional Full Time Employee (FTE) be budgeted for the PW Operations staff to complete the existing workload requested, or the responsibilities of the PW Operations staff be reduced to focus solely on utility O&M. Additionally, it is advised that staffing needs be re-evaluated every two to three years.

1.3.4 PIPELINE CAPACITY EVALUATION

A wastewater collection system model was developed using InfoSWMM software (Suite 14.7 Update #2) to evaluate existing and 20-year collection system capacity. Wastewater trunklines (10-inch diameter and larger) were included in the model as well as five pump stations. Some 8-inch pipelines were modeled to connect disparate areas that were served by 10-inch pipelines. Continuous flow monitoring was completed at six locations during the wet weather period between December of 2020 and January of 2021. The six flow monitoring locations divided the system into six monitoring basins, shown in Figure 1-2. The collected data was analyzed along with continuous precipitation data to establish typical 24-hour patterns, average base flows at each site, and gauge rainfall influence in the system. Both dry weather (minimal to no rain in days prior) and wet weather periods were used for base flows and calibration efforts.

Gravity pipelines were evaluated according to the City's Public Works Design Standards. Pipe capacity was assessed by evaluating the ratio of the depth of maximum flow to the diameter of the pipe (d/D), with pipes considered undersized if they exceed a ratio of 0.85. This planning criteria was established in meetings with City staff. Pump stations were evaluated based on the capacity to handle peak flows with the largest pump out of service (defined as firm capacity).

The calibrated model was used to assess the effects of a 5-year, 24-hour design storm event on the existing system. The existing system evaluation showed a significant portion of the modeled trunk lines operating at or above capacity. There are pipelines operating at or above capacity in each of the six monitoring basins, and almost all have manholes with the potential to overflow. The deficiencies found in the evaluation are caused by high peak flows and undersized trunklines. Figure 1-3 shows locations of over-capacity pipes in the existing system model, displayed in orange and red, with potential overflow locations marked with a red circle.

FIGURE 1-2: FLOW METER LOCATIONS AND MONITORING BASINS

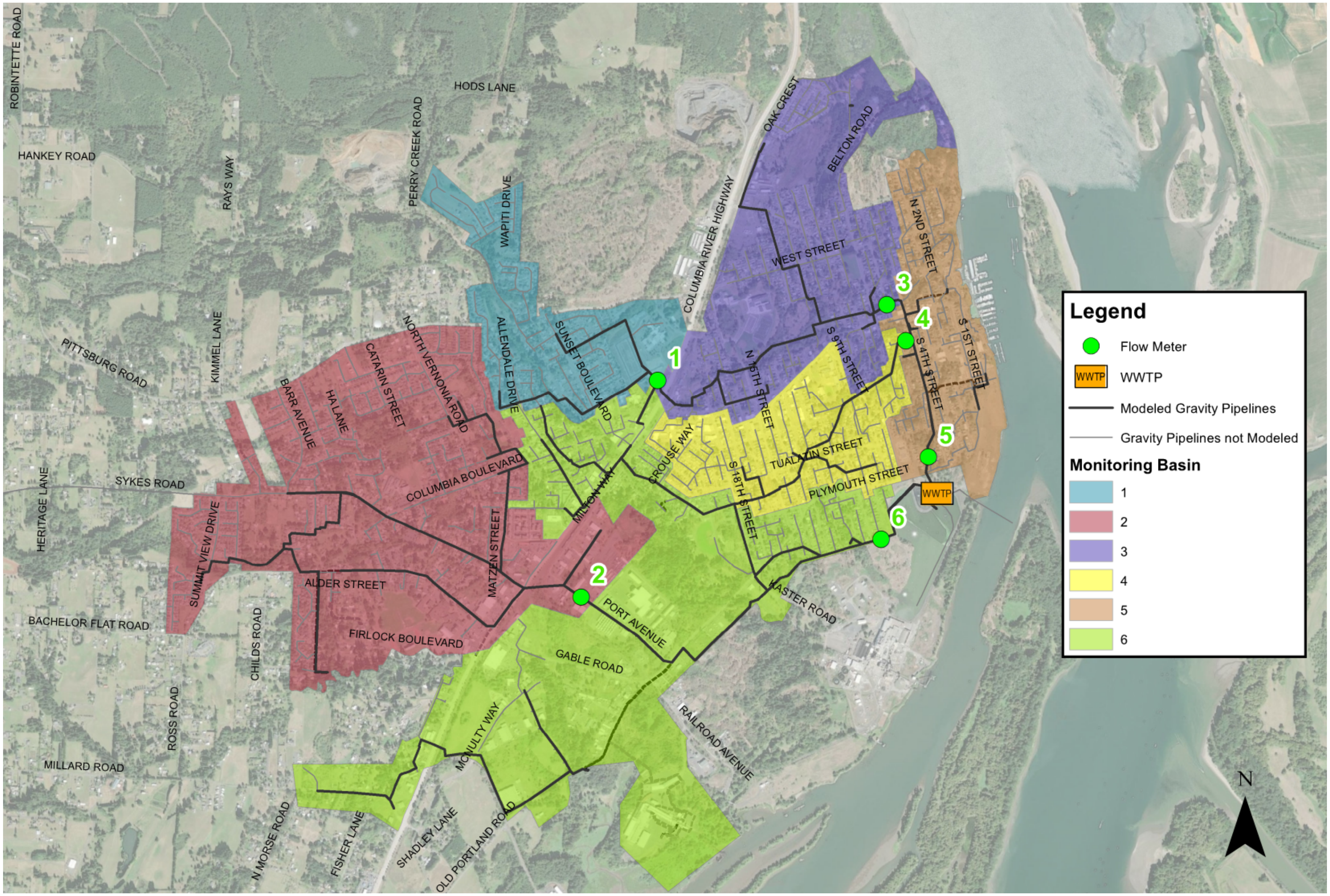
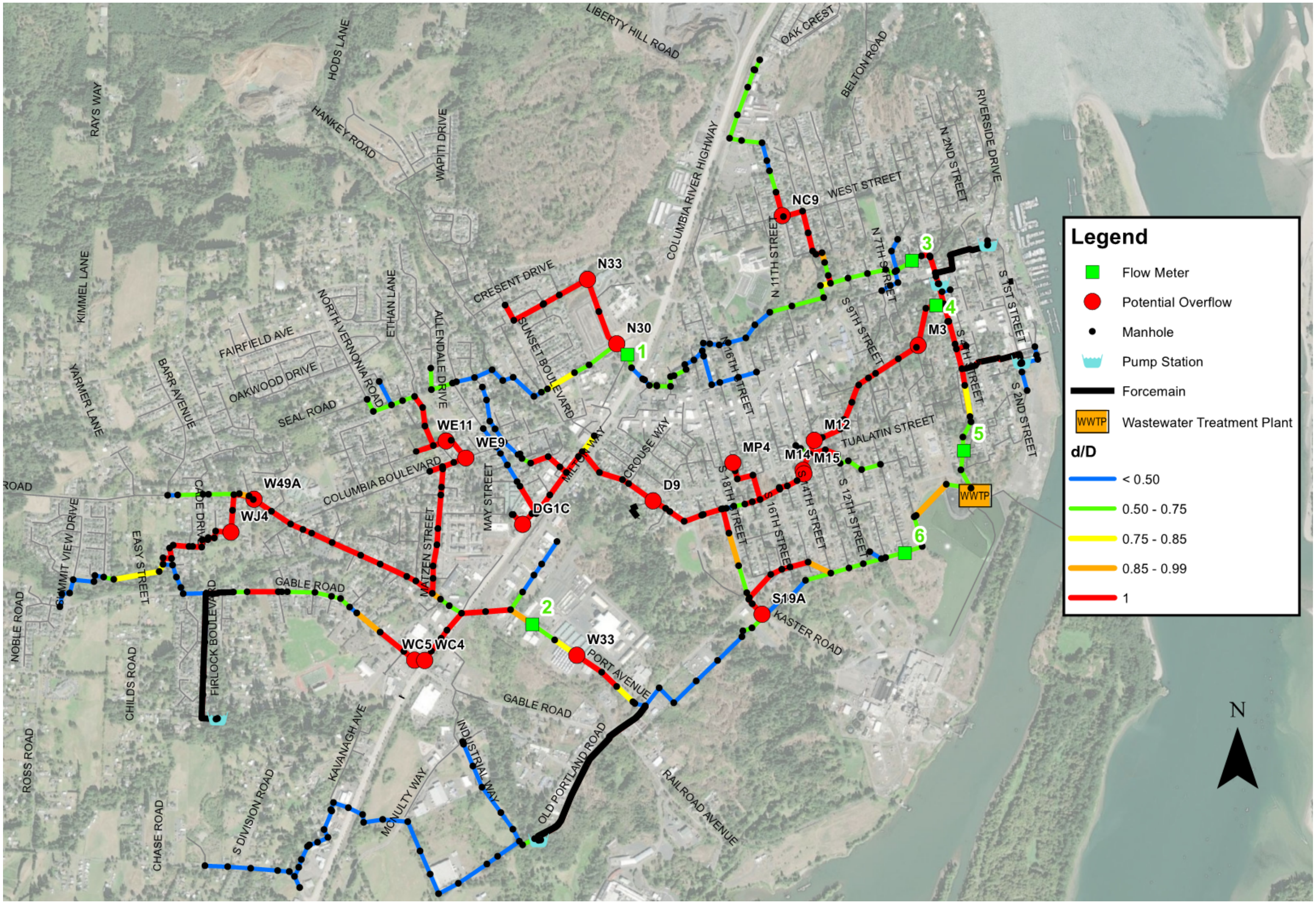


FIGURE 1-3: EXISTING SYSTEM EVALUATION – D/D AND POTENTIAL OVERFLOW LOCATIONS



For the 20-year capacity evaluation, future loads were distributed based on PSU population projections and City anticipated future residential, commercial, and industrial growth areas, shown in Figure 1-4. A majority of the areas anticipated to develop have topography that would allow for gravity flow to the existing collection system, while four growth areas may require additional infrastructure. These four identified areas are the Riverfront District (Growth Area #2), the Business Industrial Park (Growth Area #17), and Growth Areas #1 and #9 located near Pump Station 11 (PS#11).

The City is currently evaluating development options for the Riverfront District, which includes the relocation of Pump Station 1 (PS#1). A 10-inch pipeline at minimum slope would have the capacity to convey the projected 20-year flows through the Riverfront District. The proposed pipeline would be routed underneath the proposed roadways depicted in the current City planning documents.

The City is seeking new opportunities for the Industrial Business Park and completed a parcellation framework report for the site. To provide sewer service for the future development, a pump station will be required. The pump station will likely need to be located near the waterfront to follow existing topography. The gravity sewer piping will follow the proposed roadway alignments and drain to the proposed pump station location. The force main can be routed along existing and/or proposed roadways and discharge to the existing trunkline on Kaster Road. The existing gravity trunkline downstream on Old Portland Road has a section of parallel pipes which are capacity limited and should be included as part of the development process and project.

The City has expressed interest in relocating PS#11 further north, to the intersection of Firlok Park Street and Hazel Street. If relocated, the depth of the wetwell could be sized at predesign to receive flow via a gravity line from the northern portions of Growth Areas #1 and #9, which would involve a bore under McNulty Creek to serve Growth Area #1. These upgrades would include a new force main. The southern portion of both growth areas could be served by 8-inch pipelines conveyed to existing gravity trunklines. Grinder pumps might need to be installed at residences adjacent to McNulty Creek, as the relative elevation of these locations may make serving them via gravity pipeline not feasible.

Overall, problem areas identified in the 20-year evaluation reflect the same areas identified in the existing system analysis, with many of the deficiencies being caused by high peak flows and undersized trunklines exacerbated in the 20-year model. PS #7 is capacity limited for future growth and will require upsizing. Figure 1-5 shows locations of over-capacity pipes in the 20-year model, displayed in orange and red, with potential overflow locations marked with a red circle.

FIGURE 1-4: ANTICIPATED 20-YEAR GROWTH LOCATIONS

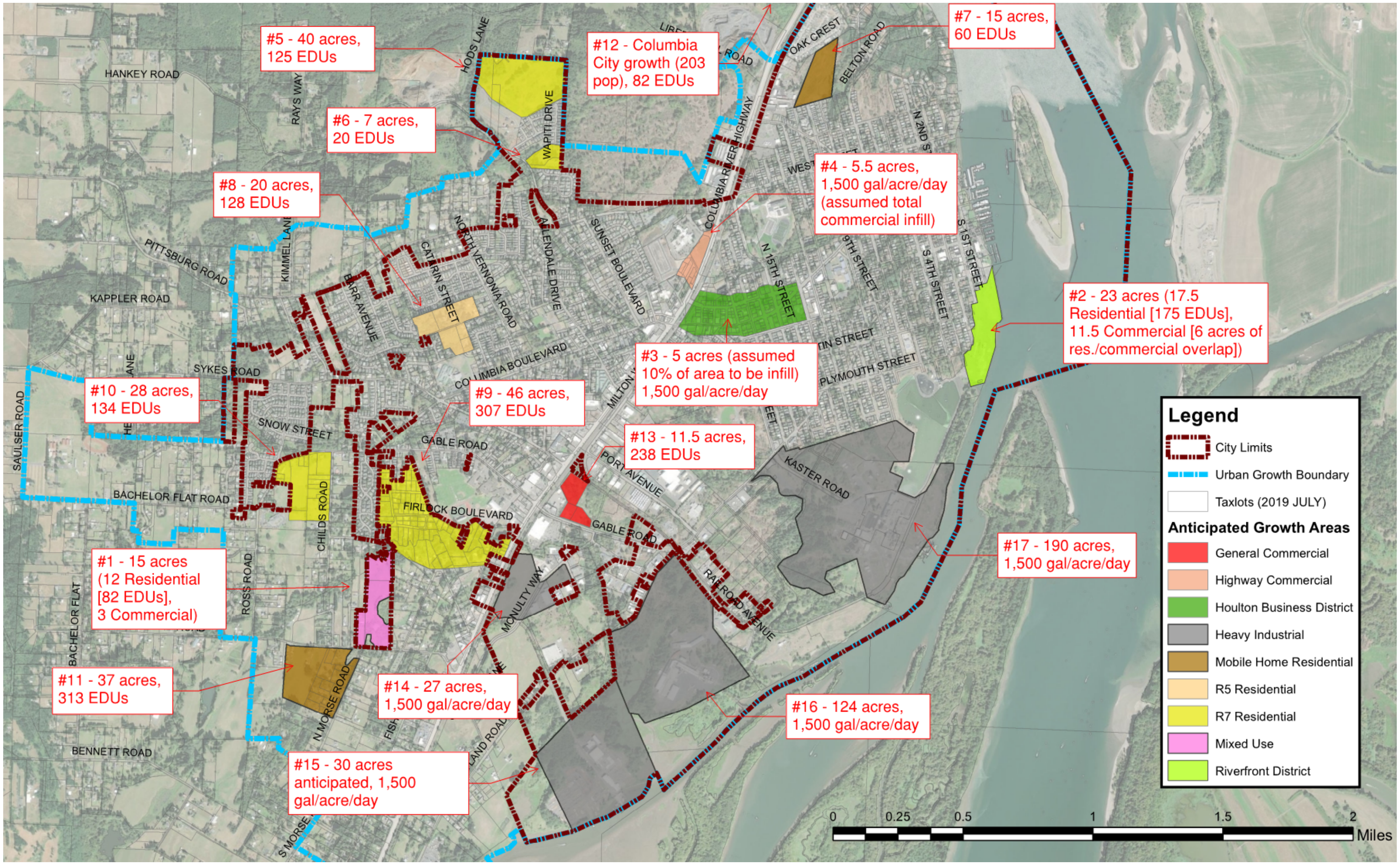
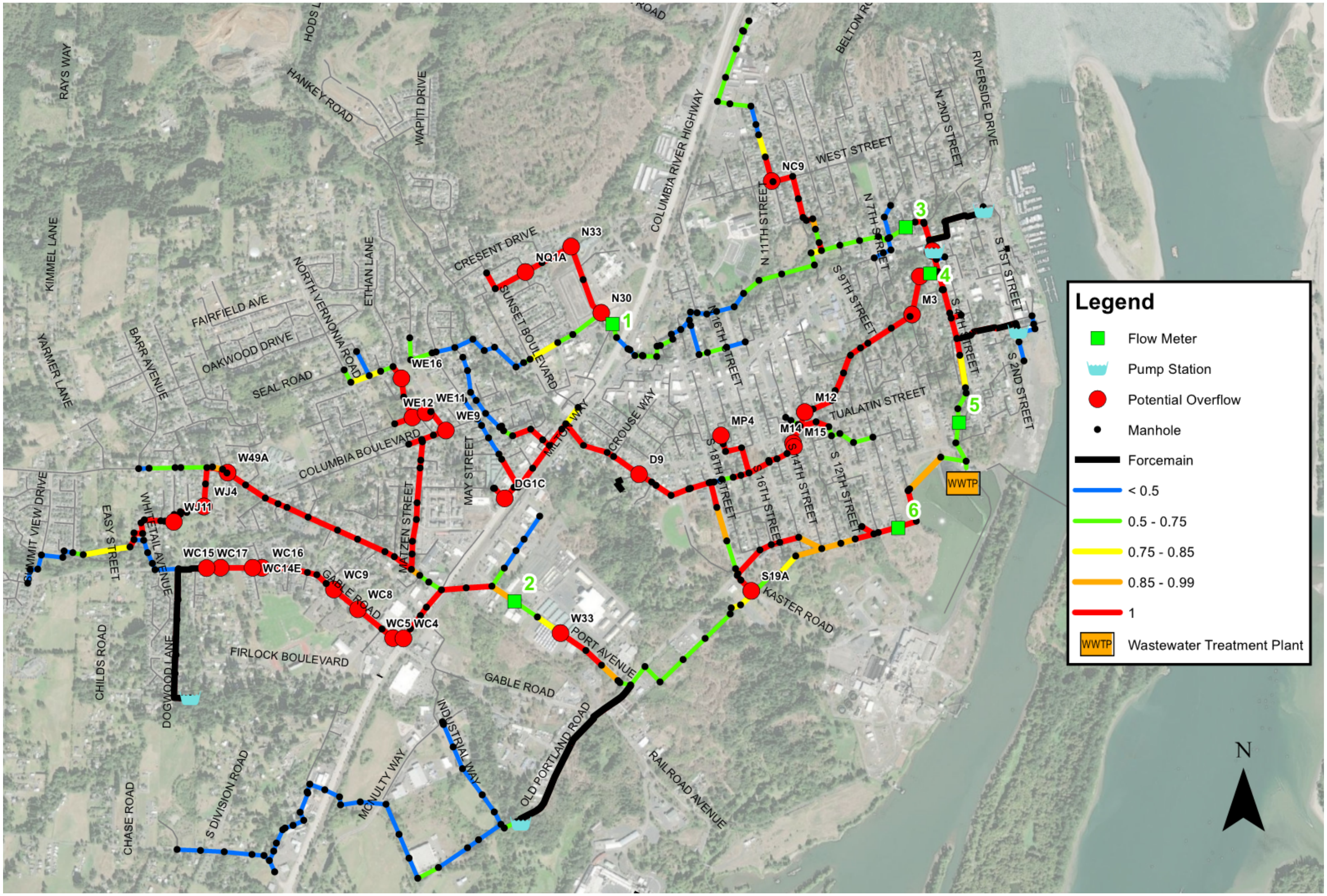


FIGURE 1-5: 20-YEAR SYSTEM EVALUATION – D/D AND POTENTIAL OVERFLOW LOCATIONS



1.3.5 PUMP STATION RESILIENCY

The compilation of this master plan included an assessment of pump station resiliency using a risk of failure evaluation. The risk of failure of an asset is a combination of the likelihood of failure and consequence of failure. Likelihood of failure is a measure of how likely an asset is to fail. An evaluation of the risks of failure can provide an importance, urgency, or priority to assets and provide guidance on the order in which asset deficiencies should be addressed. Assets with the highest risk of failure (product of likelihood of failure and consequence of failure) should be repaired or replaced first as they pose the largest threat to a system and community.

The analysis shows that PS#1 and PS#2 have the highest risks of failure. A failure at one of these pump stations would have the largest impact on the community and is most likely to happen based on the factors evaluated, indicating that deficiencies at these pump stations should be addressed soon after identified.

1.4 COLLECTION SYSTEM ALTERNATIVES

Alternatives to address collection system deficiencies discussed are summarized in the sections below. A few of the deficiencies identified do not have multiple, feasible, or cost-effective alternatives for improvements. Recommended improvements for these deficiencies are also included below.

1.4.1 SUMP PUMPS

Six alternatives were identified to address the presence of private sump pumps discharging into the collection system. The alternatives included: targeted distribution of educational material, smoke testing, dye testing and CCTV, visual inspection, point-of-sale inspection, and a reward-based disconnection incentive program. These alternatives were not considered mutually exclusive and could be performed in conjunction if the City chose to perform multiple projects at a time.

1.4.2 CONVEYANCE SYSTEM

Alternatives for conveyance were established for each flow metering basin. While some of the conveyance system deficiencies do not have multiple feasible alternatives, construction of new trunklines to redirect flow away from undersized pipelines or suspected points of overflow was considered by the City. The redirection of the conveyance system was considered a feasible alternative for Basins 2, 4, and 6. Upsizing the existing undersized trunklines to handle 20-year peak flows was considered a feasible alternative for each basin.

Additionally, the installation of parallel facilities or taking no action was presented to the City. The City could choose to construct parallel facilities in areas with limited remaining capacity, however this alternative was ultimately dismissed. Taking no action is not a viable option because surcharging and the potential for overflows would only worsen, which could result in negative impacts to human health and the environment, in addition to the increased risk of fines from the DEQ.

1.5 RECOMMENDED COLLECTION SYSTEM IMPROVEMENTS

To address the identified system deficiencies, the following improvements are recommended. Cost estimates for each of the recommended improvements are included in the section and incorporated in the Capital Improvement Plan (CIP).

1.5.11 WWTP INFLUENT FLOW METER

Priority 1 WWTP influent flow meter improvements address the suspected inaccurate influent peak flow measurement at the WWTP and would provide accurate measurement of influent peak flows during wet weather events. The total estimated cost for this improvement is $68,000.

1.5.2 PUMP STATIONS

Priority 1 pump station improvements address the continuation of upgrades the City of St. Helens is currently performing as well as the operations improvements, which include the installation of overflow alarms and adding a SCADA alarm to sound when both pumps in a pump station turn on. It is recommended that pump station runtimes continue to be recorded and reviewed by staff in conjunction with the recommended alarm data if both pumps are running to track as pump stations may be nearing firm capacity. Additionally, it is recommended that Pump Station 3 be equipped with an on-site generator to address its backup power deficiency and simplify portable generator operations during outages. The total estimated cost for these improvements is $100,000.

Priority 2 pump station improvements assume that the Riverfront District and Growth Areas #1 and #9 require the relocation of Pump Stations 1 and 11. Additionally, Priority 2 improvements address the general deficiencies, such as under-capacity pumps, fall protection provisions, level sensor redundancy, as well as flow and pressure monitoring. The total estimated costs for these improvements is $6,200,000.

Priority 3 pump station improvements include firm capacity increase of PS#7 as growth areas develop in the basin. The total estimated costs for these improvements is $2,200,000.

1.5.3 INFLOW AND INFILTRATION (I/I)

The City is advised to create an annual budget to fund an ongoing I/I reduction program, which would promote annual I/I improvement projects throughout the City. This type of work is anticipated to be a combination of sump pump identification and removal, lateral replacement, and mainline and manhole inspections and rehabilitation/replacement. System I/I reductions could reduce, delay, or eliminate the need for capacity-related pipeline upsizing projects and provide cost savings to the City over the planning period. Rather than have a separate replacement budget and I/I improvement budget, it is recommended the City adopt a combined fund of $500,000 annually for the 20-year planning period. This dollar amount is reflective of the estimated annual pipeline replacement cost, presented in Table 1-4.

1.5.4 SUMP PUMPS

It is recommended the City pursue a combination of educational material distribution, point-of-sale inspection, and a reward-based incentive program. A portion of the recommended I/I annual budget should be reserved for the printing and distribution of educational materials and to support a sump pump disconnection incentive program. Additionally, the City ought to update its code to include language requiring the seller to evaluate and disconnect any sump pumps from the sanitary sewer during inspection and before the property transfers ownership.

1.5.5 CONVEYANCE SYSTEM

Priority 1 improvements address potential overflows near the downtown and “tunnel” pipelines for the City (Basin 5), as well as deficiencies in Basin 4. Improvements include rerouting Basin 4’s trunkline along Tualatin Street to Basin 6, and upsizing gravity mains on S 4th Street, S 16th Street and S 17th Street. The annual I/I reduction projects could have significant impacts to the peak flows in Basin 5. It is recommended that flow monitoring be included in the concept design phase of this project to further define existing flows and compare the peak flows in Basin 5 following the I/I reduction work and Basin 4 improvements. The total estimated cost for these improvements is $8,100,000.

Priority 3 improvement projects will alleviate remaining existing and future capacity limitations in the collection system, but an intentional, ongoing I/I reduction program could reduce, delay, or eliminate the need for some of these improvements. These improvements include upsizing of existing undersized pipelines in Basins 1, 2, 3, and 6, and also involve construction of a new pipeline to reroute flow from Gable Road to Sykes Road, and reroute flow near Old Portland

Road and Kaster Road in Basin 6. The total estimated cost for these improvements is $22,700,000.

1.5.6 FUTURE INFRASTRUCTURE

There are four anticipated growth areas in the 20-year planning period that may require additional infrastructure to connect with the existing system, which include the Riverfront District (Growth Area #2), the Business Industrial Park (Growth Area #17), and Growth Areas #1 and #9 located near PS#11. Priority 2 improvements address the required infrastructure needed to serve the Riverfront District, Business Industrial Park, and Growth Areas #1 and #9. The costs for the proposed infrastructure at the Riverfront District are tied into the cost of the PS#1 relocation. The estimated cost of the proposed Riverfront District and Business Industrial Park infrastructure is $15,600,000. The proposed infrastructure for Growth Areas #1 and #9 is tied into the cost to relocate PS#11 and is estimated at $3,100,000.

1.5.7 OPERATIONS AND MAINTENANCE

In addition to regular maintenance, it is recommended that an annual pipeline replacement program be established. Typically, a budget for replacing the system components is based on average useful life. Average useful life of manholes and cleanouts are shown in Table 1-4.

It is recommended that the $500,000 amount presented in the I/I section above serve as a combined I/I reduction program budget and annual replacement budget. It should be noted that this is an interim amount presented for City budgeting purposes, with the purpose of increasing over time to the recommended $790,000 annual replacement budget for the system. Even after I/I improvements have significantly reduced peak flows in the system, the City should continue to maintain an annual replacement budget to fund ongoing O&M and meet the City's LOS goals.

Pipelines should be cleaned approximately every three to five years (frequency can be adjusted based on pipe material plus scour conditions and observations by City staff). Manhole rehabilitation and service line repairs should be coordinated with pipeline rehabilitation work. Emphasis should be placed on areas where pipe conditions pose the largest threat of sanitary sewer surcharging or more immediate threat of collapse.

TABLE 1-4: ANNUAL REPLACEMENT BUDGET

Item	Lifespan	Cost/Year
Pipelines	75 Years	$ 570,000
Manholes	50 Years	$ 210,000
Cleanouts	50 Years	$ 5,000
Total (rounded)		***$ 790,000***

1.5.8 PLANNING RECOMMENDATIONS

The City is recommended to update their planning documents every 5 years. Updates to the planning documents and models allow the City to re-assess needs and properly allocate budgets to address system deficiencies. The next update should include an evaluation of both the wastewater collection system and WWTP. A Master Plan Update for both the wastewater collection system and the treatment plant was included as a Priority 2 improvement, with an estimated cost of $300,000.

1.5.9 ENGINEERING DESIGN STANDARDS, CODE, AND COMPREHENSIVE PLAN REVIEW

The City's existing development code (Title 17), engineering design standards (Title 18), and comprehensive plan (Title 19) were reviewed for new development, as they pertain to wastewater conveyance, to identify potential deficiencies and provide recommendations for updates. The primary recommendations for review, updates, and additions include the following:

- Scheduling requirements
- Matching references to the Oregon Department of Transportation (ODOT)/ American Public Works Association (APWA) Oregon Standard Specifications for Construction (OSSC).
- Pipeline sizing, slope, cover, and utility spacing requirements
- Manhole design requirements
- Stream and creek crossing requirements

The City is advised to review and assess these recommended changes to these sections to City code, standards, and comprehensive plans to match current best practices in the industry. The City should then initiate the process of proposing changes to associated City documents to maintain consistency.

1.6 CAPITAL IMPROVEMENT PLAN

This section outlines the recommended plan to address the wastewater collection system deficiencies identified in previous sections. The alternative evaluation and recommended projects, with input from City staff, are the basis for the CIP for the wastewater collection system presented in this section.

1.6.1 SUMMARY OF COSTS

The cost summary of the 20-year CIP is listed in Table 1-5. Capital costs developed for the recommended improvements are Class 4 estimates as defined by the Association for the Advancement of Cost Engineering (AACE). Actual construction costs may differ from the estimates presented depending on specific design requirements and the economic climate when a project is at bid. An AACE Class 4 estimate is normally expected to be within -50 and +100 percent of the actual construction cost, which is typical for planning documents. As a result, the final project costs will vary from the estimated costs presented in this document. The costs are based on experience with similar recent collection system and WWTP upgrade projects. Equipment pricing from manufactures of the large equipment items was also used to develop the estimates. The total estimated probable project costs include contractor markups and 30% contingencies, which is typical of a planning-level estimate. Overall project costs include total construction costs, costs for engineering design, construction management services, inspection, as well as administrative costs. For the collection system projects, the contractor's overhead and profit are worked into the line items. Priorities are set for today and will be re-evaluated when there is a need for re-assessment. The CIP is based on modeling data that was available during the completion of this facilities plan. When projects are carried forward, the model, data, assumptions, etc., should be re-evaluated to make any necessary adjustments to the basis of the project. An estimated schedule for the next six years is shown in Table 1-6. Locations of the CIP projects can be found in Figure 1-6.

FIGURE 1-6: 20-YEAR CAPITAL IMPROVEMENT PLAN

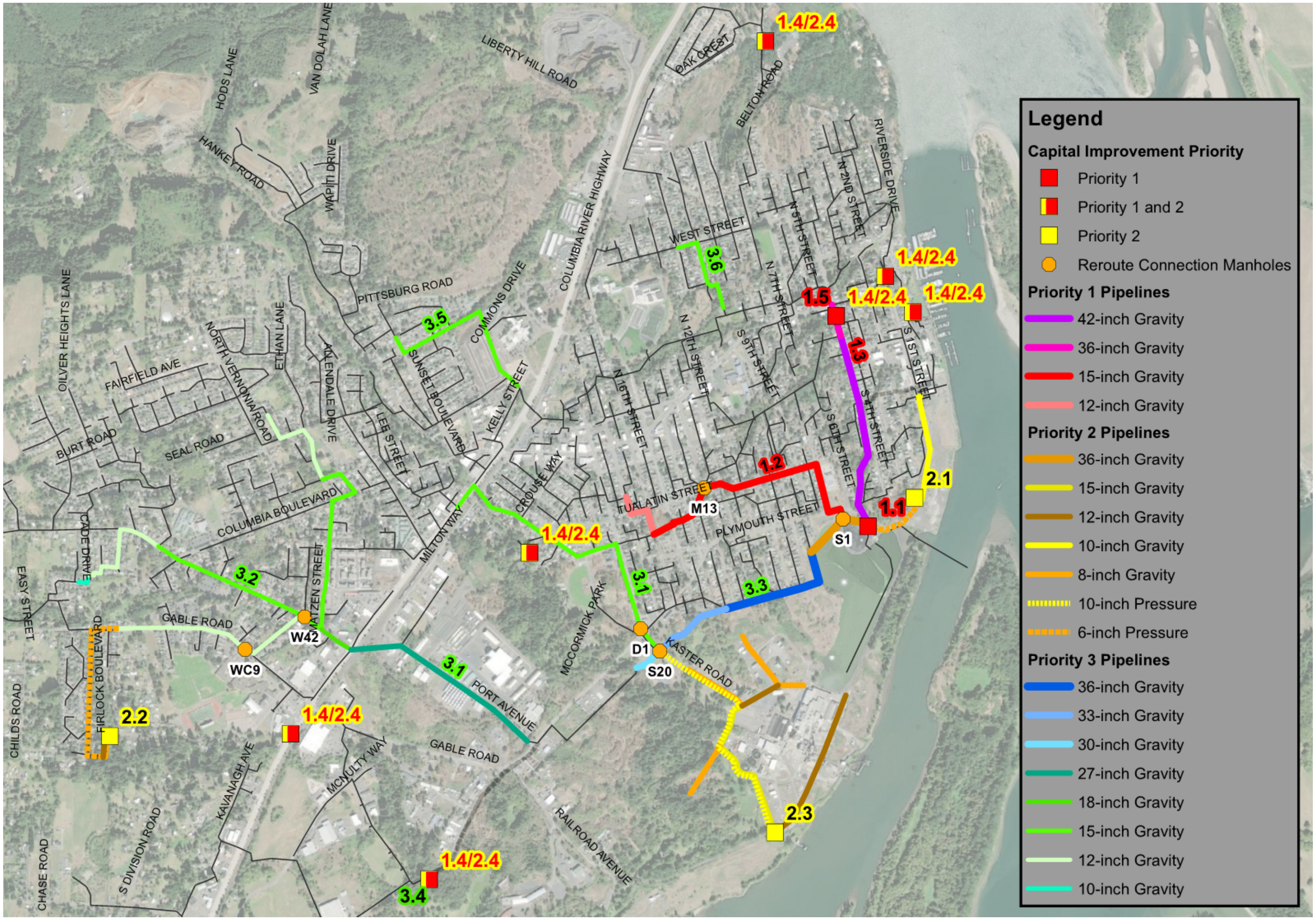


TABLE 1-5: 20-YEAR CAPITAL IMPROVEMENT PLAN (CIP)

Project No.	Project Name	Primary Purpose	Total Estimated Cost (2021)	SDC Growth Apportionment %	SDC Growth Apportionment Cost	City's Estimated Portion
Priority 1 Improvements						
1.1	WWTP Influent Flow Meter	Operations	$ 68,000	10%	$ 7,000	$ 61,000
1.2	Basin 4 Pipeline Upsize and Reroute	Capacity	$ 3,600,000	0%	$ -	$ 3,600,000
1.3	Basin 5 Pipeline Upsize	Capacity	$ 4,500,000	3%	$ 150,000	$ 4,350,000
1.4	Install Overflow Alarms	Operations	$ 9,000	20%	$ 2,000	$ 7,000
1.5	Pump Station 3 On-site Generator	Operations	$ 90,000	0%	$ -	$ 90,000
1.6	Annual I/I Reduction Program (6-Year)	Capacity	$ 3,000,000	20%	$ 590,000	$ 2,410,000
Total Priority 1 Improvement Cost (rounded)			**$ 11,300,000**			**$ 10,500,000**
Priority 2 Improvements						
2.1	Riverfront District Trunkline and Pump Station 1 Relocation	Capacity, Operations	$ 2,400,000	18%	$ 440,000	$ 1,960,000
2.2	Relocate Pump Station 11	Capacity, Operations	$ 3,100,000	68%	$ 2,110,000	$ 990,000
2.3	Industrial Business Park Trunklines and Pump Station	Capacity, Operations	$ 13,200,000	100%	$ 13,200,000	$ -
2.4	Pump Station Upgrades	Operations, Safety	$ 700,000	20%	$ 140,000	$ 560,000
2.5	Master Plan Update	Operations	$ 300,000	100%	$ 300,000	$ -
2.6	Annual I/I Reduction Program (8-Year)	Capacity	$ 4,000,000	20%	$ 790,000	$ 3,210,000
Total Priority 2 Improvement Cost (rounded)			**$ 23,700,000**			**$ 6,700,000**
Priority 3 Improvements						
3.1	Basin 6 Pipeline Upsize and Reroute	Capacity	$ 6,300,000	7%	$ 460,000	$ 5,840,000
3.2	Basin 2 Pipeline Upsize and Reroute	Capacity	$ 9,400,000	12%	$ 1,140,000	$ 8,260,000
3.3	Southern Trunkline Upsize	Capacity	$ 3,900,000	26%	$ 1,010,000	$ 2,890,000
3.4	Pump Station 7 Upgrades	Capacity	$ 2,200,000	65%	$ 1,430,000	$ 770,000
3.5	Basin 1 Pipeline Upsize	Capacity	$ 1,800,000	9%	$ 150,000	$ 1,650,000
3.6	Basin 3 Pipeline Upsize	Capacity	$ 1,200,000	3%	$ 40,000	$ 1,160,000
3.7	Annual I/I Reduction Program (6-year)	Capacity	$ 3,000,000	20%	$ 590,000	$ 2,410,000
Total Priority 3 Improvement Cost (rounded)			**$ 27,900,000**			**$ 23,000,000**
Total Collection System Improvement Costs (rounded)			**$ 62,900,000**			**$ 40,200,000**

Note:

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2021 dollars and does not include escalation to time of actual construction. Keller Associates has no control over variances in the cost of labor, materials, equipment, services provided by others, contractor's methods of determining prices, competitive bidding or market conditions, practices or bidding strategies. Keller Associates cannot and does not warrant or guarantee that proposals, bids, or actual construction costs will not vary from the cost presented herein.

TABLE 1-6: PRIORITY 1 CIP SCHEDULE

Project No.	Item	Cost (2021)	Opinion of Probable Costs 2022	2023	2024	2025	2026	2027
Priority 1 Improvements								
1.1	WWTP Influent Flow Meter	$ 68,000	$ 68,000					
1.2	Basin 4 Pipeline Upsize and Reroute	$ 3,600,000		$ 400,000	$ 3,200,000			
1.3	Basin 5 Pipeline Upsize	$ 4,500,000				$ 500,000	$ 4,000,000	
1.4	Install Overflow Alarms	$ 9,000	$ 9,000					
1.5	Pump Station 3 On-site Generator	$ 90,000	$ 90,000					
1.6	Annual I/I Reduction Program (6-Year)	$ 3,000,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000
Total (Rounded)		$ 11,300,000	$ 700,000	$ 900,000	$ 3,700,000	$1,000,000	$ 4,500,000	$ 500,000

Note:

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2021 dollars and does not include any escalation. 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.

1.6.2 OTHER ANNUAL COSTS

In addition to the capital improvement costs presented in Table 1-5 and Table 1-6, the following expected annual operating costs are recommended for consideration in setting annual budgets for the collection system:

Additional collection system replacement/rehabilitation needs: Based on linear feet of pipeline, and number of manholes and cleanouts, the City should ideally budget a total of $790,000/year for pipeline replacement/rehabilitation. Currently, it is recommended the City should establish a $500,000 annual fund for system replacement/rehabilitation. I/I replacement and rehabilitation projects performed as part of the Annual I/I Reduction Program may offset a portion or majority of these recommended costs, as pipeline rehabilitation addresses defects and extends pipeline lifespan.

The City should target the infiltration and inflow (I/I) projects as a part of the annual pipeline replacement/rehabilitation budget. Prioritizing these projects should help to reduce I/I flows into the system and potentially delay capital improvements triggered by increased system flows.

It is recommended that the City maintenance staff develop a program to clean the entire collection system every three years, and CCTV the entire collection system every six years.

Annual O&M costs for the collection system may increase slightly if Priority 3 improvements are made, as they increase the total linear feet of pipeline in the system.

It is estimated that approximately 3.5-4.0 FTE are needed to meet the recommended level of O&M for the City's LOS goals. As budgeted, the existing wastewater collections FTE staff appears to be adequate. However, the additional projects and work the PW Operations staff are currently requested to complete significantly decreases the budgeted FTE hours that can be spent on wastewater collections O&M. It is recommended that either additional FTE be budgeted for the PW Operations staff to complete the existing workload requested, or the responsibilities of the PW Operations staff be reduced to focus solely on utility O&M. In addition, the recommended CIP projects would increase workload of the engineering division. The engineering division may need additional staff to manage any sump pump identification and removal program, update and maintain the GIS database, coordinate CCTV inspection and resulting work orders, and manage capital improvements. Additional workload on the engineering and PW operations divisions should be included in planning for any of the recommended improvements and projects. It is recommended that staffing needs be reevaluated every two to three years.

1.6.3 OTHER FINANCIAL CONSIDERATIONS

The City previously had several wastewater debts that were refinanced into a single debt service in 2020. The yearly transfer for this payment is $600,000 and is set to mature in 2034. The City is currently exploring options for paying off the sewer debt sooner, potentially between 2026 and 2031.

The City should complete a full-rate study for the wastewater utility in order to evaluate potential user rate and system development charge (SDC) impacts of the recommended CIP. Estimated SDC eligibility for each identified capital improvement is included in Table 1-5 for use in completing a full rate study. It is recommended the City actively pursue opportunities for grant funds, low-interest loans, or principal forgiveness funding sources to mitigate user rate impacts. As the City prepares to proceed on CIP projects, if outside funding is desired, it is recommended the City setup a one-stop meeting with Business Oregon to identify and assess potential funding sources for the sewer projects.

SECTION 2 - PROJECT PLANNING

The City of St. Helens (City) owns and operates a municipal wastewater collection system and wastewater treatment plant (WWTP). The purpose of this study is to assess the City's wastewater collection system needs, evaluate if the City's existing collection system can meet those needs, and provide a long-term plan to implement improvements so the needs of the City can be met. This study describes the conditions, flows, and problems in the existing system, analyzes the hydraulic flow data, and provides recommendations for improvements to the collection system over the 20-year planning period.

2.1 LOCATION AND STUDY AREA

The City of St. Helens, Oregon is located adjacent to the Columbia River, approximately 25 miles northwest of Portland on US Highway 30. The City of St. Helens owns and operates a wastewater collection system within its Urban Growth Boundary (UGB). Figure 1 in Appendix A illustrates the study area and UGB for reference. Figure 1 also displays the topography within the City's UGB.

The City of Columbia City also owns and operates a wastewater collection system within its UGB. The Columbia City collection system discharges to and flows through the collection system in St. Helens to the St. Helens WWTP for treatment. No evaluation of the Columbia City system, aside from the impacts of population growth and existing flows on the St. Helens system, are included in the scope of this study.

2.2 ENVIRONMENTAL RESOURCES PRESENT

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

2.2.1 LAND USE

The City of St. Helens zoning includes residential, commercial, industrial, and public zoning within the city limits. A zoning map for the study area is in Figure 2 in Appendix A. Approximately half of the zoning within the city limits is residential. Heavy and light industrial zones are concentrated in the southern portion of the City, while most commercial areas surround the highway or are located in the Houlton Business District or Riverfront District.

2.2.2 FLOODPLAINS

Information on the floodplains in the study area is available from the Federal Emergency Management Agency (FEMA) Map Service Center. These maps show portions of the planning area which lie within the 100-year floodplain adjacent to the floodway of the Columbia River and several other small drainages. Figure 3 in Appendix A shows the flood areas within the study area obtained from the FEMA website. This figure is for display purposes only. For specific projects in these areas, the individual FEMA Flood Insurance Rate Map (FIRM) Panels should be referenced.

2.2.3 WETLANDS

St. Helens completed a Local Wetlands Inventory (LWI) in 1999 that was accepted by the Department of State Lands (DSL) and is referenced in the City's Comprehensive Plan as of May 2020. In the Comprehensive Plan, the City takes inventory and maps their wetlands to assess their functions in order to determine "Locally Significant Wetlands" that contribute to wildlife habitat, fish habitat, water quality, floodwater retention, recreational opportunities, and/or educational opportunities. The Comprehensive Plan lists the following wetlands as Locally Significant Wetlands: Dalton Lake, McNulty Creek, Frogmore Slough, Jackass Canyon, Milton Creek, Unnamed Creek A, and Unnamed Creek B.

Approximately 443 acres of wetlands were identified within the study area, and were classified into the following wetland types, also shown in Figure 4 in Appendix A:

- Forested Wetland - A wetland with soil that is saturated and often inundated, and is dominated by woody plants taller than 20 feet. Water-tolerant shrubs and herbaceous plants are often beneath the forest canopy.
- Scrub/Shrub Wetland - A wetland dominated by shrubs and woody plants less than 20 feet. Water levels can range from permanent to intermittent flooding.
- Emergent Wetland - Wetlands dominated by erect, rooted herbaceous plants that can tolerate flooded soil conditions, but cannot tolerate being submerged for extended periods, e.g. cattails, reeds, and pickerelweeds.
- Rock Bottom Wetland - Wetlands with substrates having an areal cover of stones, boulders, or bedrock 75% or greater and vegetative cover less than 30%. Water regimes are restricted to subtidal, permanently flooded, interment exposed, and semipermanent flooded.
- Littoral Wetland - Wetlands situated in a topographic depression or a dammed river channel and lack trees and shrubs. Wetlands are permanently flooded with extensive areas of deep water.
- Upper Perennial Wetland - Water is flowing throughout the year and includes wetlands contained within a channel unless the wetland is dominated by trees, shrubs, and emergent, or habitats with water containing ocean derived alts in excess of 0.5%. The gradient of the channel is high, and velocity is fast.
- Intermittent Wetland - Similar to Riverine Upper Perennial Wetland, except water only flows for parts of the year.

Additionally, to protect the riparian areas and locally significant wetlands, including McNulty and Milton Creek, designated upland protection zones have been established where construction is limited or prohibited. Additional details on upland protection zones near recommended improvements are discussed in section 7.8.3.

2.2.4 HISTORIC SITES, STRUCTURES, AND LANDMARKS

The National Register of Historic Places lists one historic site for St. Helens: the St. Helens Downtown Historic District, which is composed of approximately 101 buildings. Additionally, 23 areas and structures within city limits which hold local significance were identified as “designated landmarks” by City Ordinance Number 3250. Many of these landmarks are located within the St. Helens Downtown Historic District. A map of the Downtown Historic District and the designated landmarks can be found in Figure 5 in Appendix A.

2.2.5 BIOLOGICAL RESOURCES

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

2.2.6 WATER RESOURCES

The Columbia River, Jackass Canyon, Milton Creek, McNulty Creek, the Frogmore Slough, and two unnamed creeks flow through the study area. The WWTP outfalls to the Columbia River. Section 303(d) of the Clean Water Act establishes a list of impaired waters and total maximum daily loads (TMDL) for pollutants in each water body. Jackass Canyon is 303(d) listed for sedimentation and has a TMDL for temperature. McNulty Creek is 303(d) listed for biological criteria. The Lower Columbia River is 303(d) listed for arsenic, DDE 4,4, fecal coliforms, and PCBs, and has a TMDL for dioxins and temperature.

2.2.7 COASTAL RESOURCES

There are no coastal areas within the study area.

2.2.8 SOCIO-ECONOMIC CONDITIONS

According to the City's Housing Needs Assessment, completed in May of 2019, the City has been experiencing a steady growth and anticipates to experience more steady growth in the future. The median household income is $45,789, which is 33% less than the 2019 national average according to census.gov. 31.7% of the City is considered to be low-income, or earning less than $30,000 per year. The assessment states that approximately 25% of households are "severely rent burdened", meaning they spend more than 50% of income on rent and utilities. Higher rates can be a challenge for economic growth.

All areas in the City have access to the City collection system, which delivers the City's designated level of service to all users. Recommended improvements in this plan will help achieve the same level of service throughout the collection system for all users. City Council holds a public meeting to review and adopt the Wastewater Master Plan.

2.2.9 CLIMATE, GEOLOGIC HAZARDS, AND SOILS

Climate

The climate in St. Helens is characterized by dry and temperate summers and cool and wet winters. Table 2-1 summarizes the climate data for St. Helens. The National Oceanic and Atmosphere Administration (NOAA) Monthly Normals for St. Helens were used for the mean temperatures. NOAA data for precipitation was not available for St. Helens, as such, climate normals were taken from the nearby weather station in Scappoose, OR.

TABLE 2-1 CLIMATOLOGICAL DATA (2006-2020)

	Jan	Feb	Mar	Apr	May	Jun	July
Precipitation (in)	6.04	4.27	4.81	2.95	2.23	1.41	0.3
Mean Temp (F)	40.2	42.2	46.1	50.3	57.6	62.2	68.2
	Aug	**Sep**	**Oct**	**Nov**	**Dec**	**Sum / Average**	
Precipitation (in)	0.43	1.78	3.84	6.28	6.7	41.04	
Mean Temp (F)	68.6	63.1	53.3	45.1	39.2	53	

Geologic Hazards

Potential geologic hazards in the St. Helens area include landslides and earthquakes. There are no known volcanoes in the direct vicinity that would cause a volcanic hazard. The Oregon Department of Geology and Mineral Industries (DOGMI) categorizes St. Helens in the low-to-high susceptibility range for landslides, and this is corroborated by the Multi-Hazard Mitigation Plan for Columbia County. Additionally, the City provided GIS shapefiles which reflect the DOGAMI findings on landslide susceptibility; only a small area bordering the northern City limits are considered high susceptibility for landslides. Figure 6 in Appendix A depicts the landslide hazard zones. The Multi-Hazard Mitigation Plan also reveals that in the past, seismic activity was fairly low, but because of more recent earthquakes, awareness of a potential problem has increased. The Multi-Hazard Mitigation Plan simulated earthquake damage produced by a magnitude 9 Cascadia Earthquake, and St. Helens fell into the light to moderate damage category. Local hazard maps show the area within City limits fall within zones A through D, with zone A indicating a very small probability of experiencing damaging earthquake effects and zone D indicating the possibility of very strong shaking that can cause considerable damage in structures lacking special design. Figure 7 in Appendix A depicts a hazard map for seismic activity/earthquake hazards. Additional details and discussion of geologic hazards is included in the Geotechnical Planning Report (Shannon & Wilson, 2021) in Appendix B.

Soils

In general, the soils within the St. Helens area are either rock complex or silty loam, and the slopes vary from zero to thirty percent, according to the NRCS website. Typically, surface soil is very shallow in St. Helens, and sits on top of unfractured basalt rock. This is often a challenge for utility construction and can be a significant cost factor, particularly in pipeline projects. Figure 8 in Appendix A shows the soil map for the study area. See Appendix B for more details on the study area geology and geologic hazards completed by Shannon & Wilson Geologic Investigation.

2.2.10 AIR QUALITY

Currently, the City does not lie within an Environmental Protection Agency (EPA) non-attainment area. No permanent impacts to air quality are anticipated from the recommended improvements. Best management construction practices are advised to be employed during construction to minimize dust.

2.3 POPULATION TRENDS

The official population projections for the City of St. Helens and the City of Columbia City reflect the collaborative efforts of Columbia County and Portland State University (PSU). These agencies published a document in June 2020, establishing the official coordinated population rates for all the cities in Columbia County. The document is titled “Coordinated Population Forecast for Columbia County, its Urban Growth Boundaries (UGB), and Area Outside UGBs 2020-2070”, and includes a summary of historical populations from the U.S. Census. Table 2-2 presents the historical populations from the referenced document.

Each year, PSU establishes a preliminary population estimate in November, which is sent to state and local jurisdictions and community partners. PSU then sends a certified population estimate in December. For this wastewater master plan, the base starting point for population projections is the July 2019 certified population estimate. The average annual growth rate (AAGR) from the PSU referenced document provided the future population estimates in this report. The overall estimated population growth from 2019 to 2040 for the City of St. Helens (from 13,464 to 17,318) reflects an AAGR of 1.1%. This percentage closely resembles the 1.0% growth rate reported in the Housing Needs Assessment. The estimated growth from 2019 to 2040 for the City of Columbia City (1,985 to 2,188) reflects an AAGR of 0.5%. As a result, the total population for the two cities is anticipated to be 19,506 in 2040.

TABLE 2-2 POPULATION HISTORY AND PROJECTIONS

Year	St. Helens	Columbia City	Sum	Source
1990	7,535	1,003	8,538	US Census Bureau
2000	11,857	1,571	13,428	2020-2070 PSU Coordinate Population Forecast: US Census Bureau
2010	14,839	1,946	16,785	2020-2070 PSU Coordinate Population Forecast: US Census Bureau
2015	13,095	1,955	15,050	PSU Certified July 1, 2015
2019	13,410	1,985	15,395	PSU Certified July 1, 2019
2020	13,915	1,980	15,895	PSU Certified July 1, 2020
2025	14,697	2,030	16,727	Projected Using AAGR of 1.1% for St. Helens, 0.5% for Columbia
2030	15,524	2,081	17,605	Projected Using AAGR of 1.1% for St. Helens, 0.5% for Columbia
2035	16,396	2,134	18,530	Projected Using AAGR of 1.1% for St. Helens, 0.5% for Columbia
2040	17,318	2,188	19,506	Projected Using AAGR of 1.1% for St. Helens, 0.5% for Columbia

Note: Coordinated Growth Rates (AAGR) from PSU Coordinated Population Forecast 2020-2070 Marion County

2.4 FLOWS

The wastewater flows analysis reviews historical wastewater flows and provides projected flows for the planning period. This section summarizes the results of the analysis. The City's projected flows were estimated using the methods recommended by the Oregon Department of Environmental Quality (DEQ) in "Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon." A few of the values developed from the DEQ methods were adjusted based on observed flow events at the WWTP. Adjustments are noted in the individual sections below.

2.4.1 AVERAGE ANNUAL DAILY FLOW (AADF)

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

2.4.2 AVERAGE DRY-WEATHER FLOW (ADWF)

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

2.4.3 AVERAGE WET-WEATHER FLOW (AWWF)

The average wet-weather flow (AWWF) is the average daily flow for the periods encompassing January 1 through April 30 and November 1 through December 31 of the calendar year. An AWWF was calculated for each year of data. Years with a complete data set (2015 – 2019) were averaged to obtain the AWWF.

2.4.4 MAXIMUM MONTHLY DRY-WEATHER FLOW ($MMDWF_{10}$)

The maximum monthly dry-weather flow ($MMDWF_{10}$) represents the month with the highest flow during the summer months. DEQ's method for calculating the $MMDWF_{10}$ is to graph the January through May monthly average flows for the most recent years against the total precipitation for each month. DEQ states that May is typically the maximum monthly flow for the dry-weather period (May through October). Selecting the May 90% precipitation exceedance most likely corresponds to the maximum monthly flow during the dry-weather period for a 10-year event. The May 90% precipitation exceedance value (3.90 inches for Scappoose, as no data was available for St. Helens) is extrapolated from the NOAA Summary of Monthly Normals from 2006-2020.

Data from 2015–2019 was used according to the DEQ guidance to produce Chart 2-1. Table 2-3 summarizes the data points illustrated in the chart.

2.4.5 MAXIMUM MONTHLY WET-WEATHER FLOW ($MMWWF_5$)

The maximum monthly wet-weather flow ($MMWWF_5$) represents the highest monthly average during the winter period. DEQ's method for calculating the $MMWWF_5$ is to graph the January through May average daily flows against the monthly precipitation. DEQ states that January is typically the maximum monthly flow for wet weather (November through April). Selecting the January 80% precipitation exceedance value (7.73 inches as obtained from the NOAA Summary of Monthly Normals for Scappoose as data was not available for St. Helens) most likely corresponds to the maximum monthly flow during the wet-weather period for a 5-year event. The DEQ method and $MMWWF_5$ result are illustrated in Chart 2-1 and summarized in Table 2-3.

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

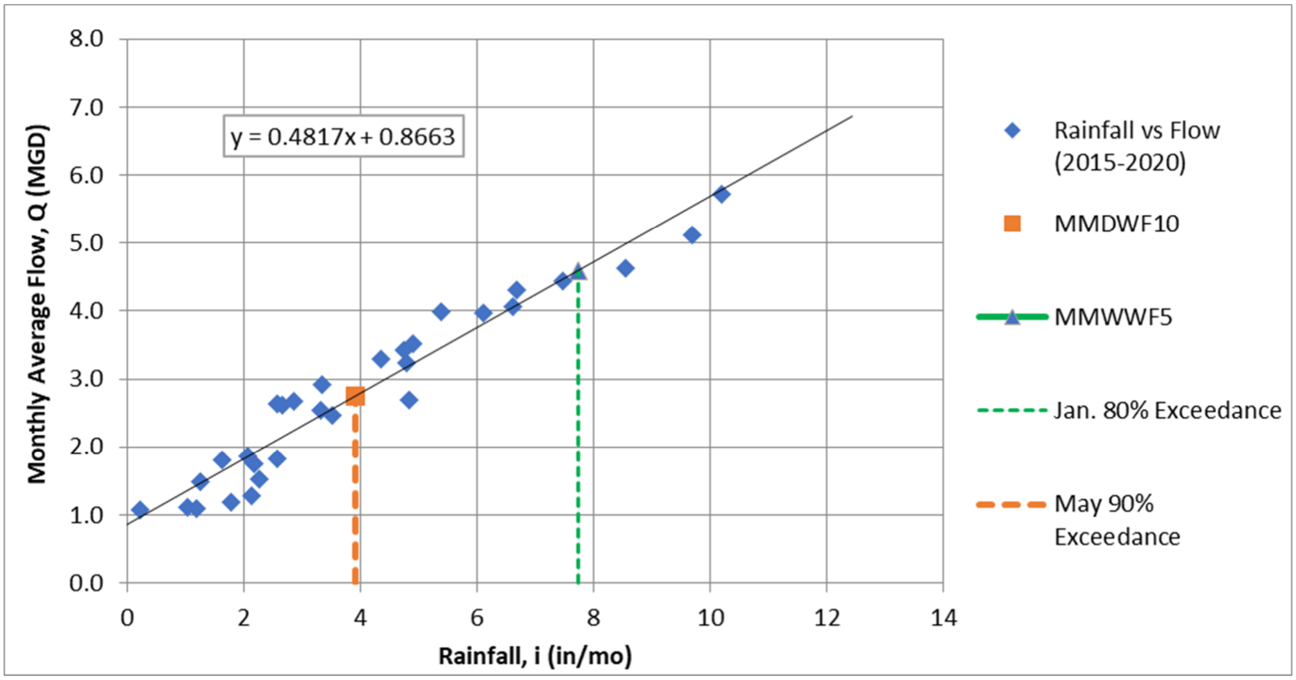


TABLE 2-3: MONTHLY AVLERAGE FLOW VS. RAINFALL ($MMDWF_{10}$ AND $MMWWF_5$)

Month	Monthly Average Flow (MGD)						Rainfall (in/mo)					
	2015	2016	2017	2018	2019	2020	2015	2016	2017	2018	2019	2020
January	3.29	4.44	3.99	4.31	2.67	5.12	4.36	7.47	5.39	6.69	2.85	9.70
February	3.51	3.42	5.72	2.92	4.07	2.62	4.91	4.74	10.19	3.34	6.62	2.66
March	2.68	3.96	4.63	2.64	1.81	1.83	4.83	6.10	8.55	2.56	1.62	2.56
April	1.76	1.52	3.23	2.54	2.47	1.49	2.17	2.27	4.80	3.32	3.51	1.26
May	1.10	1.18	1.87	1.06	1.09	1.28	1.04	1.78	2.06	0.22	1.19	2.12
$MMDWF_{10}$	2.75						3.90					
$MMWWF_5$	4.59						7.73					

To confirm the validity of the DEQ method, a 30-day rolling average of the available flow data (January 1, 2015, through December 31, 2019) was evaluated. The maximum observed 30-day rolling average flow was 7.88 MGD and occurred from December 1, 2015 through December 30, 2015. An $MMWWF_5$ of 7.88 MGD was used because the observed flow was higher than the DEQ estimated flow.

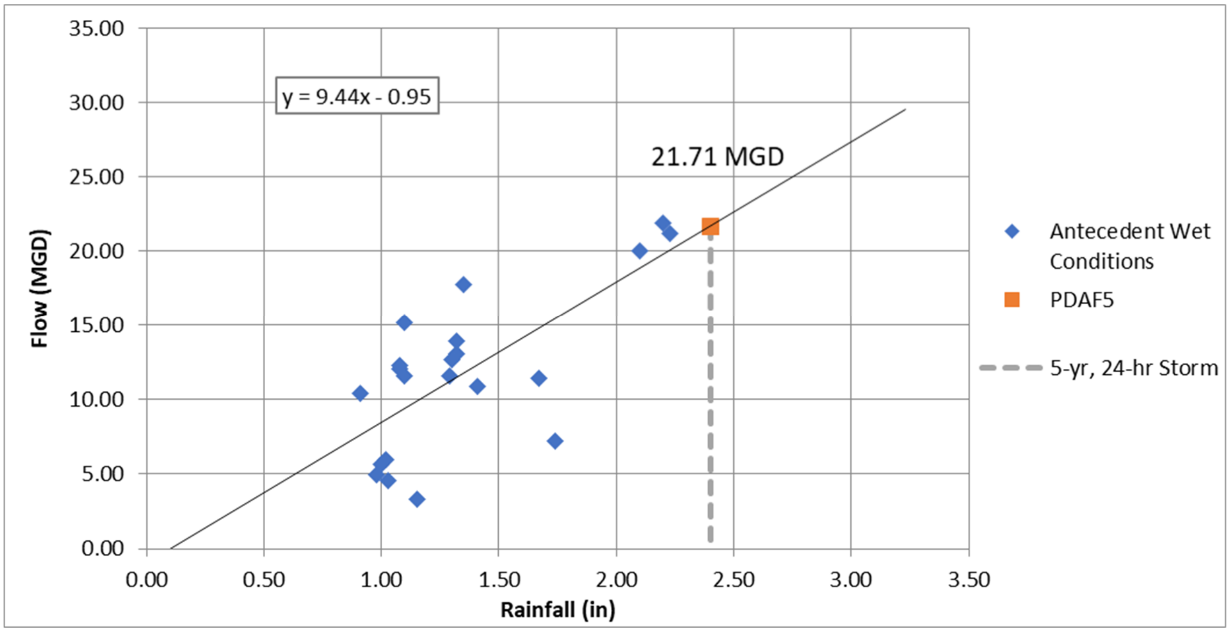
2.4.6 PEAK WEEK FLOW (PWKF)

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

2.4.7 PEAK DAILY AVERAGE FLOW ($PDAF_5$)

As outlined by the DEQ, the peak daily average flow ($PDAF_5$) corresponds with a 5-year storm event. The DEQ's method for determining $PDAF_5$ involves plotting daily plant flow against daily precipitation for significant storm events, while only using data for wet-weather seasons when groundwater is high. For this method, only significant storm events with antecedent wet conditions were plotted. A trendline was fitted to the data; the $PDAF_5$ was the resultant flowrate associated with the rainfall produced by the 5-year, 24-hour storm event (2.4 inches per the NOAA isopluvial maps for Oregon). A significant storm event was considered more than 1-inch of rainfall in 24-hours. Antecedent conditions were evaluated on a case-by-case basis, and wet conditions were assumed if any day in the preceding three had a storm event of 0.5-inches or larger. Data was also considered based on cumulative rainfall for 30 days before the storm event. No consistent, observable pattern between 30-day prior rainfall and flow conditions was discovered. As such, no cutoff for 30-day cumulative rainfall was used for purposes of this analysis. Chart 2-2 below shows the results of the DEQ analysis.

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



In analyzing the data, peak flows at the WWTP occurred on the same day or the following day as storm events. The $PDAF_5$ developed, using DEQ's method, was compared with the top five peak day flow events from 2015-2019 with antecedent wet conditions (see Table 2-4 below). The $PDAF_5$ observed in 2019 was selected as the planning value for this study because it is higher than the $PDAF_5$ flow developed using DEQ's method and is a more conservative planning value.

TABLE 2-4: TOP FIVE FLOW EVENTS

Date	DMR Flow (MGD)	Rain (in/day)	Peak Inst. Flow (MGD)	60 day rainfall (in)
12-Feb-19	21.90	2.20	32.2	12.56
8-Dec-15	21.19	2.23	31.4	17.75
7-Dec-15	20.06	2.10	29.3	15.52
18-Jan-17	17.76	1.35	24.6	13.96
16-Feb-17	13.94	1.32	19.1	13.16

2.4.8 PEAK INSTANTANEOUS FLOW (PIF_5)

The peak instantaneous flow (PIF_5) represents the peak flow recorded at the WWTP. The DEQ recommends evaluating hourly or instantaneous flow data for high-flow days if available. The peaking factor (peak instantaneous to average daily ratio) is often less during heavy flows than during normal flow rates because of infiltration influence from high groundwater. The City provided continuous flow data for high-flow days in the last five years to evaluate this peaking factor. The average peaking factor was 1.55 (data summarized in Appendix B). Using a peaking factor of 1.55 and the $PDAF_5$, a PIF_5 of 33.98 MGD was selected.

2.4.9 INFILTRATION AND INFLOW (I/I)

I/I is an issue in the collection system, and results in the high peak flows experienced at the WWTP during wet weather (Appendix B). The City has been working to characterize and evaluate I/I throughout the collection system. The I/I work completed previously, and for this study, is discussed in Section 3. The City's ongoing efforts to reduce I/I in its collection system will reduce flows to the treatment plant.

2.4.10 OBSERVED HISTORICAL FLOWS AND PROJECTED PLANNING FLOWS

Table 2-5 summarizes the observed flows for each year from 2015-2019. The historical flows were derived as described in the preceding paragraphs.

During the system evaluation process, it was discovered that the City's method of flow measurement at the WWTP may not reliably measure peak influent flows during high flow events. The City's WWTP influent flow is measured at the primary lagoon effluent weir with an ultrasonic level sensor. From the primary lagoon weir, effluent flows through a 36-inch pipe to the chlorine contact basins (CCB). During high flows, operators open the headworks bypass channel, which allow flow to bypass the headworks screens and the primary lagoon. The bypass channel flows directly into the CCB. The CCB has a similar flow measurement setup as the primary lagoon. Flow is measured at the effluent weirs with ultrasonic level sensors. When the bypass channel is open, operators record the CCB effluent flow as the plant influent flow. This flow is recorded because the bypass channel flow is not accounted for in the primary lagoon effluent flow measurement. Operators report that the primary lagoon depth fluctuates more than one foot during higher flow events. Review of the recorded plant data indicates that the WWTP influent flow measurements do not reflect peak flows from the collection system. Historical influent trends were reviewed for the highest recorded WWTP daily flows, which show both the recorded primary lagoon effluent and the CCB effluent. The trends show a sharp increase in the CCB flow, which corresponds to the bypass channel being opened. When the bypass channel is opened, the depth of the primary lagoon begins to equalize (decrease) and results in primary lagoon effluent flows that continue to discharge to the CCB. The lagoon effluent also results in CCB flow measurements that are higher than the headworks influent. This is due to the continued discharge from the primary lagoon adding to the bypass flows flowing directly to the CCB. There is not evidence that the weir measurements are inaccurate, but that they do not accurately reflect the peak flows at the headworks due to attenuation and compounding flows.

The hydraulic model of the collection system further confirms this assessment as the hydraulic capacity of the collection system is lower than historical WWTP discharge monitoring report (DMR) flows. The City completed an I/I Reduction Program project in 2008. The technical memorandum from this project (2008, Brown and Caldwell) summarizes the hydraulic evaluation of the collection system and supports that the collection system capacity is lower than the peak influent flow criteria developed at that time. City staff have indicated that no improvements to increase pipeline capacity in the collection system, except for projects addressing inflow and infiltration, have been completed since the 2008 study. These two evaluations were completed independently. Both evaluations of the collection system capacity support the assessment that the WWTP CCB effluent flows do not reflect the influent peak flows at the WWTP headworks. Additional discussion on the development and calibration of the hydraulic model is included in Section 4.

These findings and assessment were discussed with City staff. The City directed Keller to review available data, use engineering judgement, and estimate system flow planning criteria values to reflect the current system demand. These values are estimates due to the unknowns and limited data available. The PIF_5 and $PDAF_5$ planning criteria were modified. These two criteria are most likely to be impacted by the flow measurement process at the existing WWTP. The PIF_5 was reduced to 26 MGD to reflect the estimated flow influence from a 5-year storm event based on review of treatment plant flow trends, collection system capacities, and model responses. The $PDAF_5$ was reduced by 2 MGD to 19.9 MGD. This reduction was estimated from the daily WWTP trend data of historical peak events where the trends indicate the bypass channel was opened (sharp increase in the CCB flow data). Comparison of the primary lagoon effluent data and CCB data provided an estimate for peak day flows during the high events. Table 2-5 summarizes the observed, historical flows and planning criteria as described in previous sections, as well as the modified planning criteria described in this section.

It is recommended the City add influent flow measurement to the headworks facilities to more accurately track system flows and I/I over time. This planning criteria should be reviewed and updated as additional flow data is collected. Additional discussion on WWTP flow measurement improvements is included in the alternatives discussion in Section 5.

TABLE 2-5: OBSERVED HISTORICAL FLOWS & PLANNING CRITERIA

St. Helens Historical Flows (MGD[1])								
Year	**2015**	**2016**	**2017**	**2018**	**2019**	**5-Year Avg**	**Planning**	**Modified Planning**
Population	**15,050**	**15,085**	**15,225**	**15,225**	**15,395**		**15,895**	**15,895**
ADWF	0.98	1.31	1.25	0.95	1.09	1.11	1.11	1.11
$MMDWF_{10}$	2.71	2.56	2.87	3.03	2.79	2.79	3.03	3.03
AADF	2.35	2.43	2.64	1.92	1.85	2.24	2.24	2.24
AWWF	3.73	3.56	4.01	2.90	2.59	3.36	3.36	3.36
$MMWWF_5$	7.88	7.81	5.84	4.46	3.99	5.99	7.88	7.88
PWkF	14.19	7.54	8.93	5.90	8.86	9.08	14.19	14.19
$PDAF_5$	21.19	13.08	17.76	9.60	21.90	16.71	21.90	19.90
PIF_5	31.4	27.4	24.6	13.9	32.2	25.90	33.98	26.00
Yearly Total (MG[1])	856	889	955	700	669			
Total Rainfall (in/yr)	47	48	51	31	33			
1) MGD = million gallons per day; MG = million gallons								

To project the planning flows for future populations, projected flow per capita (reported in gallons per capita per day, gpcd) was developed. As shown in Table 2-6, projected unit flows are lower than the planning unit flows of the existing system. Projected unit flows were developed to recognize the existing effects of I/I on the current system, and assume reduced I/I influence on wet-weather flows in the future as new construction with better construction methods and materials are built. Projected future flows using the projected unit flows are shown in Table 2-6. Actual future flows will depend on several factors and could potentially decrease through aggressive I/I reduction efforts. It is recommended that flows be reviewed periodically, and future capital projects phased where practical.

TABLE 2-6: PROJECTED FLOWS WITH I/I REDUCTION

	Planning Flow (MGD)	Planning Unit Flow (gpcd)	Projected Unit Flow (gpcd)	Projected Planning Flow (MGD)				
Year	2019	2019	2019	2020	2025	2030	2035	2040
Population	15,395	15,395	15,395	15,895	16,727	17,605	18,530	19,506
ADWF	1.11	72	72	1.15	1.21	1.28	1.34	1.41
$MMDWF_{10}$	3.03	197	197	3.12	3.29	3.46	3.64	3.83
AADF	2.24	145	145	2.31	2.43	2.56	2.69	2.83
AWWF	3.36	218	218	3.47	3.65	3.84	4.04	4.25
$MMWWF_5$	7.88	512	300	8.03	8.28	8.54	8.82	9.11
PWkF	14.19	922	325	14.35	14.62	14.91	15.21	15.53
$PDAF_5$	19.90	1293	375	20.09	20.40	20.73	21.08	21.44
PIF_5	26.00	1689	525	26.26	26.70	27.16	27.65	28.16

2.4.11 FUTURE FLOW PROJECTIONS & MODEL SCENARIOS

Future loads were distributed based on PSU population projections and City projected future residential, commercial, and industrial growth. Flows per capita for projected population growth were assumed to be similar to existing flows per capita. Flowrates anticipated in the 20-year planning period are identified in Table 2-6. Growth areas identified by the City can be found in Figure 9 in Appendix A. Residential flows were projected using future growth areas, City zoning, projected number of equivalent dwelling units, and ADWF per capita. Projected industrial and commercial development is anticipated to grow within the industrial and commercial areas identified by the City, with both zoning designations assumed to contribute 1,500 gallons per acre per day (gpad) to the wastewater system. Residential, commercial and industrial loading calculations for the growth areas can be found in Appendix B.

2.5 PLANNING CRITERIA

2.5.1 COLLECTION SYSTEM

The City's conveyance system will be sized for the projected 20-year peak instantaneous flow rates associated with the 5-year, 24-hour storm event. For the collection system model evaluation, pipes will be considered at capacity when peak flows exceed 85% of full depth in accordance with industry standards. When sizing gravity collection systems, pipelines will be sized according to planning criteria established in meetings with the City. Pipelines shall be sized to convey 20-year, peak flows at 85% or less depth to diameter ratio (d/D). Where appropriate, major trunklines and new lines may be sized one nominal pipe size larger than hydraulically required for areas that may not be at buildout by the end of the planning period. Additionally, it should be noted, efforts to reduce I/I in the collection system could further extend the service population. Sewage pump stations will be designed to handle these flows with the largest pump out of service (defined as firm capacity).

The City's existing sanitary sewer policies, design standards, and construction standards were reviewed as part of the master plan effort. Deficiencies identified and recommended updates are summarized in a technical memorandum, included in Appendix C for reference.

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).

2.6 REGULATORY REQUIREMENTS & GUIDANCE

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

2.6.1 COLLECTION SYSTEM

Pump Station Regulatory Requirements

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

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

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

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

Standby Power – Standby power is required for every pump station because extended power outages may lead to wastewater backing up into homes and sanitary sewer overflows. Mobile generators or portable trash pumps may be acceptable for pump stations, depending on the risk of overflow, available storage in the wet well and pipelines, alarms, and response time.

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

Pipeline Guidelines (CMOM Guidance)

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

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

In December 2009, the DEQ developed a Sanitary Sewer Overflow Enforcement Internal Management Directive that provides guidance for preventing, reporting, and responding to sanitary sewer overflows. The DEQ updated this document in November 2010.

Excessive Infiltration and Inflow

EPA defines excessive I/I as the quantity that can be economically eliminated from a sewer system by rehabilitation. Some guidelines for determining excessive I/I were developed in 1985 by EPA based on a survey of 270 standard metropolitan statistical area cities (EPA Infiltration/Inflow Analysis and Project Certification, 1985). Non-excessive numeric criteria for infiltration was defined as average daily dry-weather flows that are below 120 gallons per capita day (gpcd). Similarly, a guideline of 275 gpcd average wet-weather flow was established as an indicator below which is considered non-excessive storm water inflow. According to the flow evaluation completed as part of this study (Section 2.4), flows at the St. Helens treatment plan show excessive I/I in the collection system per these guidelines.

Pipeline Surcharging

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

Illicit Cross Connections

Cross-connections to the stormwater system are prohibited by City Code, Section 13.14.090. This prohibition includes discharges to the sewer system via connecting roof downspouts, exterior foundation drains, areaway drains, and sump pumps. Any illicit cross connections from the City's stormwater system should be removed. Based on the rapid and significant I/I response in the City collection system, City staff expect there are sump pumps connected to the sewer system in several areas. Further discussion on sump pumps can be found in Sections 3 and 5 of this report.

2.7 COMMUNITY ENGAGEMENT

The City provided several opportunities for community engagement with the wastewater master planning process through a City Council workshop, a Planning Commission meeting presentation, and City Council adoption process. These meetings provided members of the community spaces to engage in the planning process and a platform provide comments.

SECTION 3 - COLLECTION SYSTEM EXISTING FACILITIES

3.1 SYSTEM DESCRIPTION

The City of St. Helens owns and operates a wastewater collection system consisting of approximately 60 miles of gravity pipeline, 2.5 miles of force main pipeline, and nine pump stations. The pipelines range from 4-inch to 33-inch in diameter. Figure 10 (Appendix A) illustrates the pipe diameters, and Figure 11 (Appendix A) illustrates the pipe material in the City's collection system. The wastewater collection system contains more than 1,300 manholes. Pump station locations and their basins are shown in Figure 12 (Appendix A).

3.2 PUMP STATIONS

The City owns and operates nine pump stations throughout the wastewater collection system that are listed by number: Pump Station(s) #1, #2, #3, #4, #5, #7, #8, #9, and #11. The locations of the pump stations are shown in Figure 3-1. Each pump station is equipped with two submersible, constant speed pumps with the exception of PS#2, which has variable frequency drives (VFDs) for both pumps. Each of the pump stations are equipped with Mission Cellular that connects them to the City's supervisory control and data acquisition (SCADA) system. Three of the pump stations are equipped with an onsite generator and an automatic transfer switch, while the remainder are serviced via manual transfer switches and two portable generators kept onsite at the WWTP.

On October 6, 2020, Keller Associates visited each pump station with City staff to observe visual equipment condition and document any known issues. A comprehensive condition evaluation nor pump tests of the pump stations were included in the scope of this master plan. This section presents general observations and recommendations, along with specific recommendations for individual pump station sites. General observations and some recommendations are presented first for the pump station sites. General recommendations are provided as a guideline to allow the City to maintain the pump stations for the 20-year planning period. Any items of concern observed during the onsite evaluation are also noted. Pump station specific observations and recommendations follow. A summary of each pump station's equipment is presented in Table 3.1.

FIGURE 3-1 – EXISTING PUMP STATION BASINS

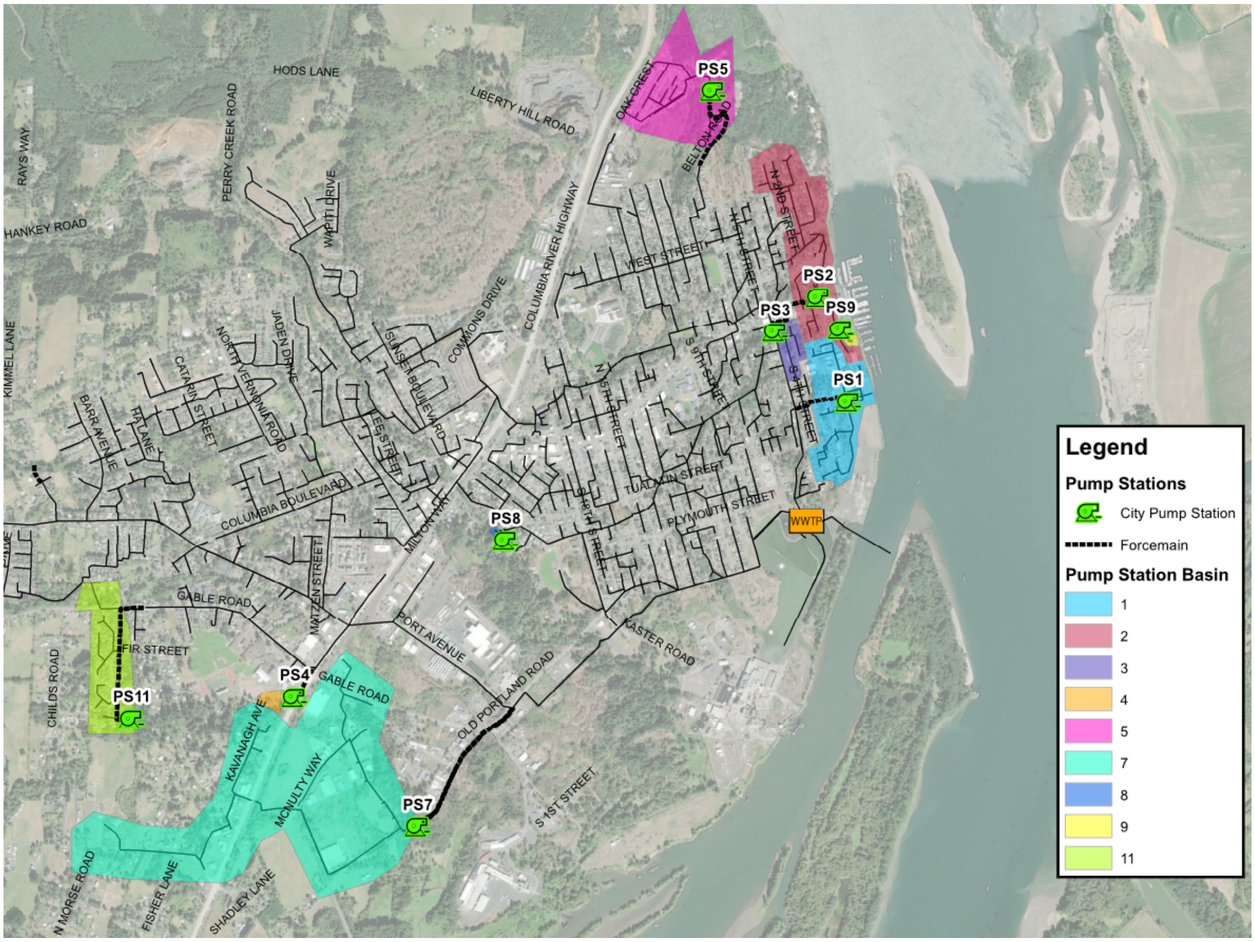


TABLE 3-1 – PUMP STATION SUMMARY

Name	PS#1	PS#2	PS#3	PS#4	PS#5	PS#7	PS#8	PS#9	PS#11
Type	Duplex, Submersible	Duplex, Self-Priming	Duplex, Submersible	Duplex Submersible	Duplex, Submersible	Duplex, Submersible	Duplex, Submersible	Duplex, Submersible	Duplex, Submersible
Year Constructed	1950s	1990	1997	1995	1994	1986	1991	1994	1996
Pump Type	Paco / Hydromatic Submersible	Gorman Rupps VSP (High / Low)	Wilo Type FA 10.51A Submersible	FLYGT NP - 3085	ABS AFP AFP(K) 1049.1-M105/4FM	Wilo Submersible	ABS SJS10W	Barns 4SE3724L	Hydromatic S4HVX-1500JD
Pump hp	36 / 30	40 / 22.5	6.2	3	14	15.5	1	3.7	15
Design Flow (gpm)	550	700 / 250	500	130	145	390	Unknown	200	143
Design Head (ft)	110	82 / 52	10.7	22	98	83	4	24	74
Low Level Alarm (ft)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.42	N/A
Pump Off Level (ft)	1.33	1.50	2	6.2	2.00	3.83	2.83	0.58	0.75
Lead On Level (ft)	2	3	3.5	8.9	4.00	10.00	4.93	1.167	1.65
Lag On Level (ft)	2.5	3.5	4.33	10.0	5.00	10.5	Unknown	2.75	2
High Level Alarm (ft)	6	7.5	5.83	11.8	5.00	11	5.45	3.75	3.1
Level Control Type	Ultrasonic Level Sensor	Ultrasonic Level Sensor	Ultrasonic Level Sensor	Float Relays	Ultrasonic Level Senor	Ultrasonic Level Sensor	Float Relays	Float Relays	Float Relays
Flow Meter	No	No	No	No	No	No	No	No	No
Pressure Gauge	Yes	No	No	No	No	No	No	No	No
Auxiliary Power Type	Portable Generator	On-Site Generator	Portable Generator	Portable Generator	On-site Generator	On-site Generator	Portable Generator	Portable Generator	Portable Generator
Transfer Switch	MTS	ATS	MTS	MTS	ATS	ATS	MTS	MTS	MTS
Bypass Piping	No	No	No	Yes	No	No	No	No	No
Oder Control	None	None	None	None	None	None	None	None	None
Wet Well Depth (ft)	18	9	15.5	20.6	10.5	16	4	13	6.15
Wet Well Diameter (ft)*	12.67	8	7	6	6	6	3	5	5
Force main Diameter (in)	6	6	6	4	4	6 / 8	3	6	4
Force Main Length (ft)**	1,010	1,050	20	610	1,700	2,620	260	70	2,500

*Pump Station 1 has a rectangular wetwell
**Estimated using City GIS data

3.2.1 GENERAL OBSERVATIONS

Sites and Security

The pump stations are easily accessible from streets throughout the City. At the time of the site visit, four of the pump stations were equipped with some type of security fence, building, or enclosure (i.e. clam shell). Generally, electrical panels and access hatches were locked, however, some manhole access to wetwells or valve vaults were not locked. No intrusion alarm system nor video equipment were observed at the sites. Use of video security provides a deterrent to vandalism, improved public safety, and a higher level of confidence in the reliability of the system. If the City experiences issues with vandalism or tampering, additional security barriers, such as fences or buildings, should be installed to prevent system tampering.

Telemetry

All pump stations are connected to the Mission cellular SCADA system. Operators receive pump station data (such as runtime, etc.) through Mission SCADA, and the City has not had problems with this system. During the most recent power outage, the City did not have any problems and continued to receive data, alarm notifications, etc. during the outage.

Operations

At the time of site visits, no odor control devices were reported on any of the pump stations and no odor issues were noted by staff at this time either. Although, if the City does receive odor complaints, it would be recommended to evaluate if odor control is needed at the pump stations.

The pump stations do not have flow meters or pressure gauges installed on the force main discharge piping. Pressure gauges on discharge piping can provide information to assess pump performance. Flow meters and pressure gauges on pump station discharge piping are not required but should be considered with each pump station upgrade and construction of new pump stations. Monitoring flow at pump stations is recommended for maintenance and operational benefits. A record of flow from a pump station can provide information on pump, sewer, and inflow conditions; unauthorized inflow; and future planning for expansion or replacement.

Housekeeping/Maintenance

Overall, the pump stations are kept in clean and orderly condition. Most of the pump stations have access to wash-down water onsite for regular maintenance. The City visually inspects pump stations approximately twice a week. Fats, oils, and grease (FOG) buildup in wetwells are cleaned out with the vactor truck twice a year and more regularly if needed.

The City does not have accurate/up-to-date record drawings or pump information for several of the pump stations. It is recommended that accurate/up-to-date record drawings and pump information be kept on-site as well as at City maintenance shop to aid in future facility upgrades and ongoing system maintenance. Available pump curves for the pump stations can be found in Appendix D.

Safety Equipment

At the time of the site visits, all but two of the pump stations (PS#7 and PS#9) lacked adequate fall protection for the wetwell and valve vaults. It is recommended the City install fall protection to protect the safety of its operators.

Emergency Generators and Backup Power

Three pump stations, PS#2, PS#5, and PS#7, have permanent, onsite generators with automatic transfer switches. The permanent generators are located outside in weatherproof enclosures and run on diesel fuel stored in an above-ground tank at each generator. The fuel tanks are located under the generator frame skid (referred to as a sub-base fuel tank with double wall containment) and fuel is pumped directly from the tank. The generators receive regular maintenance about once per year and are exercised weekly.

In the case of a power outage, the remaining pump stations have connections for portable generators that are stored at the WWTP. City staff report having two portable diesel generators, one that is sized for PS#1 and one sized for the remainder of the pump stations. In the event of a total power blackout, the City does not have the capacity to provide backup power to all of its pump stations at once. Lack of backup power could lead to sanitary sewer overflows, which are both a major environmental and public health issue.

Bypass Pumping Provisions

Only one of the pump stations, PS#4, was noted to have a bypass piping connection. Bypass piping allows for pump connection and conveyance of wastewater out of the wetwell during improvement work and is recommended to be installed for ease of maintenance. The City has one wastewater vactor trunk that can be used to pump out a wetwell if there is an equipment or pipe failure, power outage, or other issue preventing pump station operation. Lack of bypass piping complicates the operators' ability to pump out wetwells for maintenance or to prevent overflows.

Sensor and Alarm Redundancy

Currently, approximately half of the City's pump stations have level sensor redundancy; they are equipped with both ultrasonic level sensors and backup floats. Levels in PS#4, PS#8, PS#9, and PS#11 are only monitored via level floats. Lack of level measurement redundancy increases risk of overflows in the case of sensor malfunction, so level measurement redundancy is recommended on all pump stations. Each of the pump stations is equipped with a high-level alarm that is connected to the City's SCADA system, and as mentioned, City staff have reported no issues with receiving notifications or alarms during power outages.

The City is in the process of adding overflow alarms at each of their pump stations per DEQ guidance. Additional recommendations on alarms are discussed in Section 7 of this report.

Firm Capacity

Firm Capacity refers to the capacity of a pump station with its largest pump offline. An evaluation of the existing pump stations' firm capacities can be found in Section 4.

3.2.2 PUMP STATION #1

PS#1 is located on the east end of the City, within the sidewalk on S 1st Street near Cowlitz Street, and was constructed during the 1950s. Primarily serving the Riverfront district, wastewater is collected in a 9-foot x 14-foot rectangular, concrete wetwell. The pump station discharges to a 6-inch diameter forcemain that conveys water to the trunkline on S 4th Street.

The pump station has a drywell which contains the controls and manual transfer switch for the pump station. The drywell requires a confined space entry during power outages to transfer power. Additionally, the wetwell has an overflow pipe that is currently plugged but can be opened manually. The level is recorded via an ultrasonic level sensor with backup floats, however there is no fall protection installed at the pump station.

During the site visit, City staff reported some FOG buildup in the wetwell. Excessive FOG can cause blockages in pipelines and pumps, reducing conveyance capacity. The City experiences moderate

I/I influence at the pump station. In the future, this pump station may be abandoned and relocated as the City's waterfront property develops.

3.2.3 PUMP STATION #2

PS#2 was constructed in 1991 and is located on the east side of town, between N. River Street and N 2nd Street, north of Columbia Boulevard. The station is housed in a brick building and collects wastewater in a concrete, 8-foot diameter wetwell. PS #9 discharges into the PS #2 basin, and a manhole outside of the building provides access to the wetwell. The duplex, self-priming pumps deliver flow west through a 6-inch diameter forcemain, which is approximately 1,050 feet in length, to the trunkline on S 4th Street. There is no easy bypass connection on the discharge piping for maintenance. An onsite generator is located in the building. There is no fall protection installed at the pump station.

During the site visit, City staff reported that historically this station experienced significant I/I, which resulted in capacity issues. After the City's I/I Reduction Program from 2012 to 2014, the pump station has seen a significant decrease in flow and no capacity issues have been noted in the last few years. A single I-beam with a crane is available for pump removal, but there are no beams for pump motor removal. No other major issues were noted during the site visit; the pump station appears to be in good working order.

3.2.4 PUMP STATION #3

PS#3 pumps and wetwell are located within the drive lanes of S 4th Street, which is south of Columbia Boulevard. The electrical and controls box is located to the side of the road and protected from traffic by four bollards. Wastewater is collected in the 7-foot diameter wetwell under the road and pumped via a 4-inch forcemain to the trunkline on the opposite side of the road. Both the wetwell and valve vault are located in the drive lanes; traffic control is needed for pump station maintenance.

The wetwell is monitored with an ultrasonic level sensor and backup floats. City staff have reported some grease buildup, but not enough to require frequent maintenance. The upstream area is reported to have a moderate level of I/I. The inlet tee in the wetwell has to be removed to remove either pump for maintenance. There is no fall protection installed at the pump station.

An overflow pipe is located in the wetwell, which drains to the storm system upstream of Godfrey Park.

3.2.5 PUMP STATION #4

PS#4 is located on the southwest side of City limits, at the Firlock Boulevard and Columbia River Highway intersection. The pump station was constructed in 1991 and reconstructed in 2013. It is believed this pump station serves the local shopping center and portions of the high school. The pump station is located adjacent to a parking lot with no traffic protection. Wastewater is collected in a 6ft diameter wetwell and conveyed via a 4-inch forcemain to the trunkline at the intersection of Gable Road and the Columbia River Highway. There is no fall protection installed at the pump station.

The level in the wetwell is monitored via floats. A bypass connection is located within the valve vault. During the site visit, City staff said the pump station does not have FOG, I/I, or other major problems. The runtimes of this station are very low, as its collection area is believed to only be the local shopping center and portions of the high school.

3.2.6 PUMP STATION #5

PS#5 is located in the northeast corner of the City, on Madrona Court, and was constructed in 1994. Wastewater flows are collected in a 6-foot wetwell and pumped through a 4-inch forcemain to the gravity line on N 6th Street. The pump station is equipped with an onsite generator and an automatic transfer switch in case of power loss. There is no fall protection installed at the pump station.

Ultrasonic level sensors, with backup floats, monitor levels in the wetwell. If the pump station were to overflow, it would overflow at the wetwell lid and onto the site. The station is reported to have high I/I, with City staff confirming that it is normal to have an overflow event once every two years. Additionally, it was reported that a high amount of non-flushable items tend to accumulate in the wetwell, resulting in City staff needing to use a vacuum truck to empty the contents out of the wetwell approximately once every quarter.

The valve vault is equipped with a port for pipe pigging, an operation that clears the force main of excess debris. There is also an onsite 6,000-gallon storage tank. The onsite manhole has a gate valve which is used to backup flow into the tank during periods of high I/I. The tank can then discharge at a slower rate into the wetwell, which provides some mitigation of overflow events during smaller I/I events.

3.2.7 PUMP STATION #7

PS#7 is located adjacent to Old Portland Road in the southern portion of the City, and was originally constructed in 1986. In 2014/2015 the pump station was upgraded to a 6-foot wetwell with submersible pumps. Wastewater is pumped through a 6-inch forcemain to the trunkline at the intersection of Port Avenue and Old Portland Road. An 8-inch forcemain runs parallel to the 6-inch forcemain, which was used as an overflow from the Armstrong property to PS#7. The 8-inch forcemain is not currently in use.

The pump station has an on-site generator with an automatic transfer switch. City staff exercises the generator on a weekly basis. The wetwell is equipped with ultrasonic level sensors with backup level floats. There is no piped overflow, however, if there was an overflow, flooding would first occur at the wetwell lid. City staff reported that this pump station operates well with no major issues. A portion of the collection system upstream of this pump station reaches outside of City limits. There is an existing connection to a restaurant outside of City Limits that is currently closed, and there may be a few additional connections on properties that have yet to be annexed into City limits.

3.2.8 PUMP STATION #8

PS#8 is located on Clark Street and was constructed in 1991. Wastewater is collected into a 3-foot diameter wetwell and is pumped into a 4-inch diameter force main, which is 261 feet long, that discharges to the gravity sewer along Tualatin Street. The wetwell is equipped with level floats. There is no fall protection installed at the pump station.

During the site visit, it was noted that the pump station was in overall good condition, with no recurring problems reported by the operating staff. This is likely because the pump station currently only serves one home and has very low run times while the remaining houses in the area are served by septic tanks. According to staff, one of the pumps was replaced in 2005.

3.2.9 PUMP STATION #9

PS#9 is located on S River Street and serves a small area next to the marina. The pump station collects wastewater in a 5-foot diameter wetwell, and discharges across the street to a gravity line in S River Street, which flows to PS#2. The pump station was upgraded in 2018 and the electrical panel is protected from the parking lot with bollards.

The level within the wetwell is monitored via level floats. During the site visit, City staff noted that this pump station has had issues with rags and non-flushable items. The City is working with the local Homeowners' Association (HOA) to prevent this issue from occurring again in the future.

3.2.10 PUMP STATION #11

PS#11 was constructed in 1998, and is located in the western portion of the City on Maple Street. Wastewater is collected in the 5-foot wetwell and conveyed through a 4-inch force main to the trunkline on Gable Road. The pump station is enclosed with a Hydronix clam shell. This site has no on-site water available and no permanent light fixture. City staff have to use trunks, flashlights, etc. to illuminate this area during maintenance, and bring a water truck for cleaning. There is no fall protection installed at this pump station.

Currently, the City is considering moving the pump station north along Maple Street to collect additional wastewater from development to the east, which are currently on septic systems. These houses are located outside of City limits on County property, and with aging septic systems, these properties will likely require sewer connection in the future. PS#11 could serve the area if relocated north.

During the site visit, City staff reported that this pump station experiences a significant amount of FOG. Normally, the staff has to clear the FOG from the wetwell quarterly.

3.3 GRAVITY MAINS

Generally, the most efficient way to evaluate the condition of the wastewater collection system is through routine CCTV inspections. The City has not performed a significant length of CCTV inspection in the last 5 years. Without CCTV inspection data, the condition of the collection system is typically analyzed by reviewing pipeline age and material to identify pipe segments more likely to have potential defects. Section 3.4 provides additional discussion about pipeline age and material, in addition to other factors that are indicative of the collection system's condition. Section 4 includes a modeled system evaluation to identify system capacity limitations.

3.4 INFILTRATION AND INFLOW

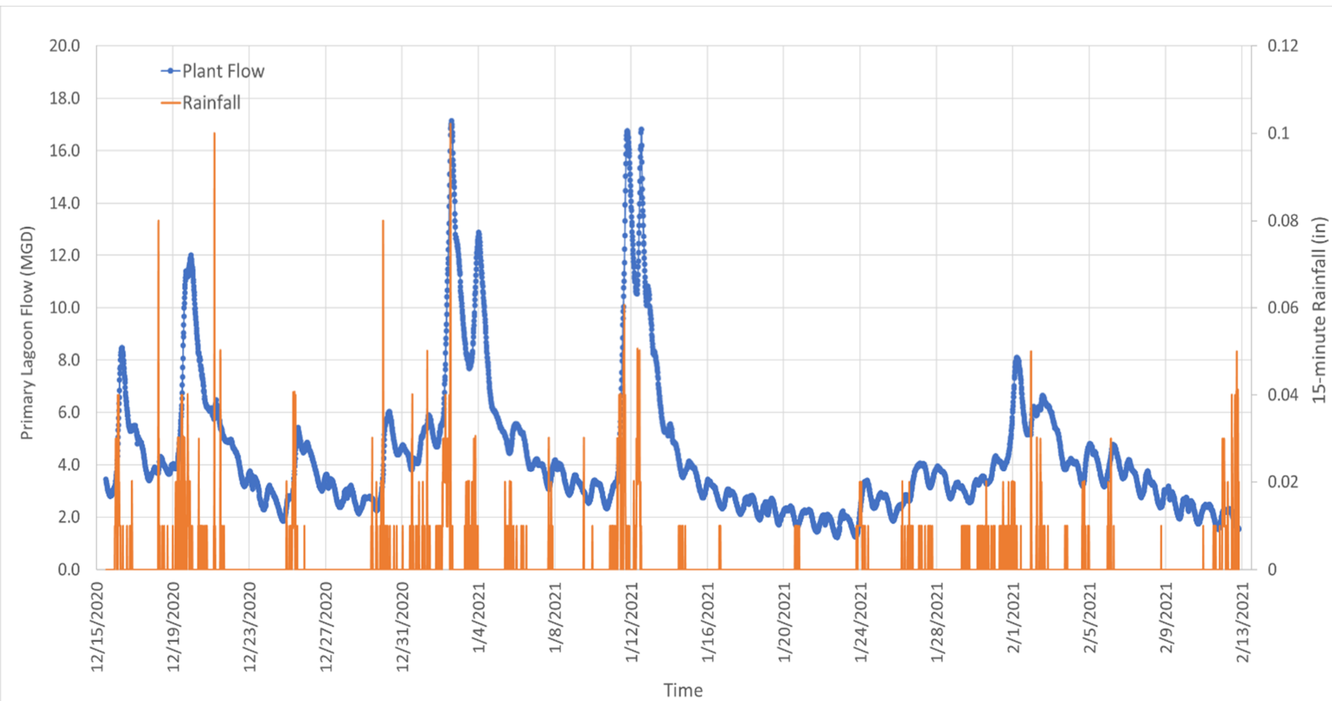
3.4.1 BACKGROUND

In 2008, Brown and Caldwell performed a Wet Weather Capacity Evaluation which documented infiltration and inflow (I/I) in St. Helens. The project included model creation and a capacity analysis. The results showed major I/I influence on peak system flows, for instance, peak hour flow events produced 25 MGD, 24 MG of which was I/I.

Since the completion of the study, the City has performed smoke testing and CCTV inspections on the collection system. The City has also taken steps to address I/I in the system via pipeline replacement, pipe repair (including CIPP lining and spot repairs), and manhole rehabilitation and replacement. City staff have reported that the effort has produced noticeable I/I reduction. For example, the City has confirmed that there have been fewer overflows at the pump stations, and has seen a significant decrease in the number of overflows that is reported to DEQ. While some reduction in I/I has been seen, there is still evidence of significant I/I influence in the system. This master plan included a high-level evaluation of I/I in the system.

Visual evidence of I/I influence in the system can be seen in Chart 3-1, which displays WWTP primary lagoon flow vs. 15-minute rainfall data for mid-December 2020 through mid-February 2021. The rapid response between precipitation events and high WWTP flows reinforces that a significant component of peak flow is from stormwater I/I. Flows for winter 2020/2021 are representative of previous years.

CHART 3-1 – WWTP FLOW VS. 15-MINUTE RAINFALL



A preliminary evaluation to identify areas likely to experience the highest I/I was completed using available data. Pipeline age and material data, areas of suspected sump pump connections, City reported issues, and priority pipelines from the 2008 evaluation not addressed in the I/I reduction projects were compared to identify areas anticipated to have the highest I/I influence. Additional details on each set of data are summarized in the following sections.

3.4.2 PIPE AGE

The City GIS database included pipeline installation date. According to this data, the City has pipes that were installed as early as 1911. The GIS installation data appears to have been updated as the City performed replacement and rehabilitation efforts. A breakdown of the pipelines by decade is shown in Table 3.2. Pipeline ages are also displayed in Figure 13 in Appendix A.

TABLE 3-2 – PIPELINE AGE BREAKDOWN BY DECADE

Decade Installed	Length of Pipe (ft)	% of Total
1910s	2,300	0.7%
1930s	7,700	2.4%
1940s	1,600	0.5%
1950s	6,800	2.2%
1960s	15,500	4.9%
1970s	37,500	11.9%
1980s	51,800	16.5%
1990s	64,500	20.5%
2000s	47,900	15.2%
2010s	58,300	18.5%
Unknown	20,400	6.5%
Total	314,300	100.0%

Typically, sanitary sewer pipelines have an expected service life of 50 to 100 years. The longer a pipe remains in the ground, the more likely the pipe is to experience cracks, root intrusion, breaks, and such defects that increase I/I into the system. As such, pipelines over 70 years old, those installed before the 1950s (about 3.7% of the City's pipelines), should be the highest priority to CCTV inspect. Those over 50 years old, installed prior to the 1970s (about 10.8% of the City's pipelines), should be the second priority. Pipelines of unknown installation date should be considered for secondary priority for inspection because they represent an unknown risk to the system and have the potential to be past their service life.

3.4.3 PIPE MATERIAL

The City GIS database includes pipeline material data. Pipeline material within the City consists of ductile Iron (DI), polyvinyl chloride (PVC), high-density polyethylene (HDPE), polyethylene (PE), concrete, cast iron, steel, and vitrified clay (VCP). The City has updated this data as they performed pipeline repair and rehabilitation efforts. The pipe material of pipes rehabilitated with cure-in-place-pipe (CIPP) lining has been updated within the GIS database to CIPP. Table 3.3 provides a full breakdown of pipelines by diameter and material. Figure 11 in Appendix A shows the locations of the pipelines by material.

TABLE 3-3 – PIPELINE SIZE AND MATERIAL BREAKDOWN (ALL LENGTHS IN FEET)

		Material								
		DI	PVC/ HDPE / PE	Concrete	Cast Iron/Steel	CIPP Restored	VCP	Unknown	Total	% of Total
Size	4" , 5"	0	5,500	200	50	0	0	0	5,750	1.8%
	6"	3,800	20,300	12,900	200	24,300	700	2,400	64,600	20.6%
	8"	2,600	93,900	34,800	0	16,500	100	10,300	158,200	50.3%
	10"	550	8,400	7,000	0	7,100	250	2,300	25,600	8.1%
	12"	450	8,000	10,600	0	2,800	0	0	21,850	7.0%
	15"	0	4,000	6,200	400	0	0	2,100	12,700	4.0%
	16"	0	2,800	0	650	0	0	0	3,450	1.1%
	18"	0	1,400	600	650	0	0	0	2,650	0.8%
	21"	0	1,400	450	0	0	0	0	1,850	0.6%
	24"	0	3,300	1,000	0	0	0	0	4,300	1.4%
	27"	0	0	1,200	0	0	0	350	1,550	0.5%
	30"	300	0	5,100	0	0	0	0	5,400	1.7%
	33"	0	0	1,900	0	0	0	0	1,900	0.6%
	Unknown	0	0	200	0	0	0	4,300	4,500	1.4%
	Total	7,700	149,000	82,150	1,950	50,700	1,050	21,750	314,300	100.0%
	% of Total	2.4%	47.4%	26.1%	0.6%	16.1%	0.3%	6.9%	100.0%	

Pipe material can be used as a rough estimation of pipeline age based on the historical materials of choice for sanitary sewer construction. For example, vitrified clay was the pipeline of choice around the turn of the 20th century. Cast iron and steel pipes are also often associated with older installations and are not widely used in recent sanitary sewer construction. As discussed in Section 3.3.2, older pipelines are at greater risk for deterioration or defects that allow I/I as well as increased risk of pipe failure. It is recommended these pipe materials be higher priority for CCTV inspections. As shown in Table 3.3, approximately 1,000 feet of the City's pipeline is vitrified clay, and about 2,000 feet is cast iron or steel.

Concrete pipes are still used for larger diameter pipelines but have the potential to be older installations. Concrete pipes as well as pipe with unknown material data should be considered as second priority. It is recommended that the City should update the GIS database with unknown pipes' material as CCTV inspection takes place.

3.4.4 CITY-IDENTIFIED SUMP PUMP AREAS

Sump pumps are used to remove water that has accumulated in a sump basin, most commonly found in the basements of homes. Generally, sump pumps handle stormwater and/or groundwater and are connected to the stormwater system. Sump pumps are not allowed to discharge to the sewer system per Section 13.14.090 of the City Municipal Code. The rapid and significant rainfall response observed by City staff in some of the major sewer trunklines suggests there may be stormwater sump pumps improperly connected to the sewer system. The City identified three areas of town which staff believed are likely to have active sump pumps improperly connected to the sewer.

The three areas are overlayed in Figure 14 in Appendix A. Recommendations on identifying and addressing sump pumps connected to the sewer are presented in Section 5.

3.4.5 REVIEW OF PREVIOUS STUDIES AND PROJECTS

As part of this planning effort, the previous Wet Weather Capacity Analysis (2008, Brown and Caldwell) was reviewed by Keller Associates. The study identified 62,300 feet of sanitary sewer pipelines as potential sources of high I/I. These priority pipelines and connected manholes were prioritized for CCTV inspection and rehabilitation/repair if necessary. The City subsequently performed CCTV on all identified pipelines and performed I/I rehabilitation and repair projects on the majority of the pipelines. These efforts were documented in the City's GIS database and record drawings.

Based on the City GIS database, 29 lengths of pipelines identified by the study were CCTV inspected, but did not have any repair or rehabilitation performed. Presumably, this is because no defects were found during inspections. As the most recent CCTV effort concluded in 2014, these pipes may have developed defects in the last 6-7 years. It is recommended that these 29 segments be considered a secondary priority for inspection and rehabilitation as necessary. These pipelines are shown in Figure 14 in Appendix A.

3.4.6 CITY-KNOWN PROBLEMS

The City provided Keller Associates with a list of known sewer problems that included historically reported capacity issues, sewer backups, and overflows. The full list with locations is shown in Appendix E, and the issues are also noted on Figure 14 in Appendix A. The areas with issues identified by the City are considered high priority for I/I identification as they have a known and significant effect on the populace of St. Helens.

3.4.7 I/I PRIORITIZATION AND SUMMARY

Each of these criteria were overlayed spatially using GIS data. Pipe segments which contained the intersection of multiple criteria were considered higher risk for I/I and high priority for CCTV inspection. For example, a vitrified clay pipe installed in the 1930s and in an identified sump pump area would be given high priority.

According to the City's GIS, several of the pipeline sections with City-identified issues have been replaced or repaired within the last 10 years. It is unlikely that the repaired or replaced pipe lengths contribute significant I/I to the system. If a pipe identified as a City-known problem was shown to have been repaired but the problems persisted, the collection system surrounding City-identified problem area was considered high priority for additional I/I investigation.

Figure 15 in Appendix A displays the prioritized pipes within the system. These pipelines should be considered as high priority for CCTV inspection and subsequent repair and/or replacement as needed. Overall, this evaluation identified 8,000 feet of Priority 1 pipelines; 15,200 feet of Priority 2 pipelines; and 18,250 feet of Priority 3 pipelines for CCTV inspection.

I/I prioritization and identification is an ongoing, evolving process. As the City collects more data, the prioritization evaluation should be updated to reflect the most recent data available. It should be noted that CCTV inspections are one of the most commonly used and telling methods to identify both structural and O&M (including I/I) defects in the system. The City does not currently maintain a regular CCTV inspection program, so it is recommended that the City work towards regular inspection of all system pipes and include this information in their ongoing I/I prioritization process. Additional discussion on recommended O&M is included in Section 5.

Future prioritization evaluation could incorporate additional criteria or information, such as consequence of failure. Risk is a function of both the likelihood of failure (pipeline condition) and the consequence of failure. Including consequence of failure to the prioritization process could involve adding criteria that characterizes the scale of impacts a pipeline failure would have. For example, a pipeline that services a small residential cul-de-sac would have a much smaller impact than a larger interceptor that services a business district or school/hospital. Adding consequence of failure or other criteria would allow the City to further prioritize sewer work to reduce risk within the collection system.

3.5 STAFFING EVALUATION

This section summarizes the City of St. Helens existing wastewater staffing levels, identifies deficiencies in existing staffing levels, and provides staffing recommendations.

3.5.1 GENERAL

Multiple divisions of the City Public Works (PW) Operations staff are responsible for the operations and maintenance (O&M) of the wastewater collection system. The PW Operations staff are responsible for the O&M of the gravity pipelines and associated structures (i.e. manholes and cleanouts). The WWTP staff are responsible for the O&M of the nine pump stations throughout the system. On February 25th, 2021, public works staff from both divisions were interviewed by Keller Associates to assess existing levels of wastewater staffing and annual O&M activities, to identify deficiencies in staffing and equipment, and provide recommendations to assist the City in meeting level of service (LOS) goals for the wastewater collection system. In general, the public works staff in St. Helens provide support for many City activities that are not directly related to public utility O&M (i.e. building maintenance, building remodels, City events, etc.). The sections below provide more detail regarding existing wastewater collection system staffing and recommendations.

3.5.2 EXISTING WASTEWATER COLLECTION SYSTEM STAFFING

During staff interviews, the general roles and responsibilities of the PW Operations staff and WWTP staff for wastewater collection system O&M was summarized. A list of O&M activities and approximate time, frequency, and size of crew was developed to evaluate the approximate annual labor hours spent on wastewater collection O&M. The primary O&M activities include cleaning and CCTV inspection of pipelines and manholes, I/I investigation and flooding mitigation, responding to problematic areas or reports, regular pump station cleaning and maintenance, and pump station mechanical repairs or replacements (including pump plugs, etc.). It is estimated that approximately 2.0 full time employee (FTE) is spent annually on wastewater collection O&M activities.

The current, budgeted FTE for wastewater collection systems O&M is approximately 4.5 FTE. This includes 0.5 FTE from the engineering department for construction inspection and permitting support. Additional discussions with the PW and engineering staff show that the PW Operations staff are requested to complete significant tasks and projects outside of utility O&M. Some of these tasks include, but are not limited to, building maintenance; building remodels and renovations; City events setup, takedown, and traffic control; park projects and maintenance; and groundwork for City projects. It is estimated that the PW Operations staff spend 50% or more of their time completing work that is not directly related to utility O&M. These additional tasks pull the PW Operations staff away from utility maintenance activities and prevent them from spending the allocated FTE on utility O&M. Of the four utilities that the PW Operations staff operate and maintain, staff reports being pulled off of wastewater collections work more frequently than stormwater or water O&M activities. Existing maintenance practices on the gravity collection system tend to be reactive because the additional projects the PW Operations staff complete minimizes the time they can spend on utilities O&M, and especially wastewater collections O&M.

3.5.3 RECOMMENDED COLLECTION SYSTEM O&M AND STAFFING

Level of service (LOS) goals were discussed with PW Operations staff for the wastewater collection system. The desired LOS goals are summarized below.

- Gravity collection system
  - No overflows
  - Address reported problems in a timely manner to prevent interruptions to service
  - Complete regular maintenance, repairs, and replacements to minimize interruptions and failures (perform proactive O&M in lieu of reactive O&M)

- Pump stations and forcemains
  - No overflows
  - Onsite generators turn on automatically and provide reliable backup power
  - Clear, safe access to pump stations
  - Trained for emergency preparedness
  - Complete regular maintenance, repairs, and replacements to minimize interruptions and failures (perform proactive O&M in lieu of reactive O&M)

A summary of general recommended O&M activities to achieve these LOS goals and follow industry good practice is listed below.

- Clean the collection system pipelines and structures once every three years (clean approximately 1/3 system annually)
- CCTV inspect the collection system pipelines and structures once every six years (inspect approximately 1/6 of system annually)
- Repair or replace defects as identified
- Investigate sources of I/I during the wet season
- Respond to problems as they are identified or reported
- Complete routine weekly, monthly, and quarterly cleaning and inspections of pump stations and equipment
- Repair/replace miscellaneous mechanical equipment as identified
- Respond to pump plugs as needed
- Complete annual staff training
- Facilitate public education and outreach
- Complete construction inspection and permitting

Using similar expected labor hours for O&M as the existing staffing evaluation, it is estimated that approximately 3.5-4.0 FTE are needed to meet the LOS goals and O&M activities described above.

As budgeted, the existing wastewater collections FTE staff appears to be adequate. However, the additional projects and work the PW Operations staff are currently requested to complete significantly decreases the budgeted FTE that can be spent on wastewater collection O&M. It is recommended that either additional FTE be budgeted for the PW Operations staff to complete the existing workload requested, or the responsibilities of the PW Operations staff be reduced to focus solely on utility O&M. This staffing evaluation is a high-level, initial estimate. It may be helpful for the City to track the number of hours the PW Operations staff spend on various activities and utilities throughout the year to assess how best to budget and allocate City resources and provide recommended O&M on the utilities. It is recommended that staffing needs be reevaluated every two to three years.

In addition to annual O&M discussed above, an annual replacement program should be maintained. Wastewater infrastructure replacement and rehabilitation needs will increase as the collection system ages. It is recommended that CCTV inspection reports be reviewed to prioritize rehabilitation and replacement efforts. An annual replacement program is an important part of proactively maintaining the wastewater collection system. Staffing FTE and construction cost for an annual replacement program were not included in the staffing evaluation, but construction costs are discussed and estimated in Section 8. If the PW Operations staff are asked to be responsible for and complete some of the rehabilitation or replacement work, this would increase the budgeted FTE for the PW Operations staff.

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SECTION 4 - COLLECTION SYSTEM HYDRAULIC EVALUATION

4.1 COLLECTION SYSTEM COMPUTER MODEL

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

4.1.1 MODEL CONSTRUCTION

InfoSWMM Suite 14.7 Update #2 was selected as the modeling software for this project. InfoSWMM is a fully dynamic model which operates in conjunction with Esri ArcGIS and allows for evaluation of complex hydraulic flow patterns.

The City maintains a GIS database of City wastewater infrastructure, and from this database, pipe diameter and invert elevation data were populated for the model. Available record drawings and input from City staff were also used to populate the model. As part of model construction, 27 spot elevation locations along trunklines were surveyed throughout the City to compare GIS database elevations with existing field elevations. In places where survey data was unable to be collected, record drawings were referenced.

During the survey process, it was discovered that the majority of the City's GIS was on the NGVD29 vertical datum, while the most recent survey data was collected in the NAV88 vertical datum. The surveyor recorded an average 3.34-foot elevation difference between the two vertical datums in the St. Helens area, and the model was built on the NAV88 vertical datum. City GIS and record drawing elevation data on NGVD29 datum was shifted to NAV88 datum for further model development.

Pipelines with diameters of 10-inches and larger were included in the model. Additionally, approximately 7,500 linear feet of 8-inch pipelines were modeled to connect disparate areas that were served by 10-inch pipelines. Figure 16 in Appendix A shows the modeled pipelines by size. After the manholes and pipes were created, and elevation data was populated in the model, several queries were conducted to reveal anomalies in the data. Anomalies included reverse slope pipes, unusual changes in pipe size, and uncommon configurations in the pipe network. Anomalies were also discussed with City personnel and appropriate changes were made to the model.

Five of the nine pump stations were included in the existing system model (PS#1, PS#2, PS#3, PS#7, and PS#11). Pump station wetwell dimensions and operational set points were provided by the system operators or taken from the operations and maintenance (O&M) manuals or record drawings. Pump station pumps were characterized by the O&M manual pump curves when available. Pump field tests were not performed as part of this planning effort. All pump stations were modeled as duplex pump stations. Pump station capacities were evaluated using firm capacities (capacity with largest pump offline).

It is important to note that one of the basic assumptions of the hydraulic model is that all pipelines are free from physical obstructions such as roots and accumulated debris. Such maintenance issues, which certainly exist, must be discovered and addressed through consistent maintenance efforts. The modeled capacities discussed in this chapter represent the capacities assuming the wastewater collection lines are in good working order.

4.1.2 MODEL CALIBRATION

Model loads refer to the wastewater flows that enter the wastewater collection system and are comprised of wastewater collected from individual services (base flows), plus groundwater infiltration (GWI) and stormwater infiltration and inflow (I/I). As part of this study, flow monitoring was completed during the wet weather period from December 29th, 2020 to January 20th, 2021. Flow monitoring data was collected at six manholes throughout the system for model calibration.

The six monitoring sites divided the system into six basins. Figure 17 in Appendix A shows flow monitoring locations and basins used for model calibration. The collected data was analyzed along with continuous precipitation data to establish typical diurnal patterns, average base flows and GWI, and gauge rainfall influence at each site. Both dry weather and wet weather periods were used for loading and calibration efforts. Loads for the model were developed and calibrated in several stages as described below.

Base Flow Calibration

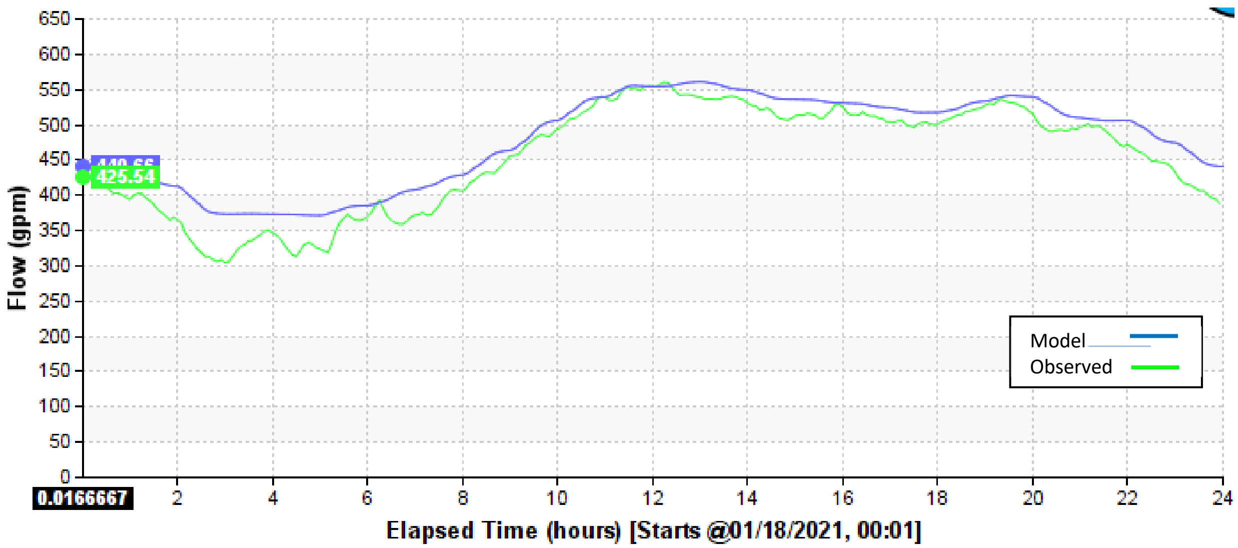
As a starting point, base flows were estimated using water consumption data from December 2019 to February 2020. Wintertime water consumption data was used to minimize any influence from irrigation usage. Total consumption for each user was provided in excel format by the City, and an average consumption for each user was calculated. Individual water meter locations for customers in St. Helens were linked to the wastewater model using GIS to provide a highly accurate distribution of wastewater loads. An average flow was assigned to each modeled manhole based on spatial allocation of the wastewater loads. Loads from pipelines not modeled were assigned to the first downstream, modeled manhole. Figure 4-1 depicts an example of load allocation from pipelines that were not modeled. Water consumption for the City of Columbia City is recorded by one meter in the St. Helens water consumption data. The average base flows for Columbia City were loaded as a single load on the manhole where the Columbia City collection system discharges to the St. Helens' system. The allocation process described yielded a total system base flow of 0.9 MGD.

FIGURE 4-1: LOAD ALLOCATION EXAMPLE

Diurnal patterns for each flow monitoring basin were developed from monitoring data of a representative dry day (day with trace amounts or no rainfall and antecedent dry conditions). Diurnal patterns for each monitoring basin were assigned to all base flows within the basin.

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

CHART 4-1: SAMPLE BASE FLOW CALIBRATION SITE 3



During the calibration process, flow monitor data from Sites #5 and #6 was found to be unreliable and did not match flows from upstream flow monitor locations. Alternative calibration methods for these two basins were developed. For location purposes, Site #5 is downstream of Sites #3 and #4 and the primary contributing flows to Basin 5 downstream of Basins 3 and 4 are flows from PS#1, PS#2, and PS#3. Historical pump runtime data was compared with WWTP discharge monitoring report (DMR) flow to estimate the percent of system flows conveyed through PS#1, PS#2, and PS#3. Base flow contributions from Basin 5 were estimated to be 5% of the system flows. Flows from Sites #5 and #6 combine downstream and enter the WWTP headworks, and there are very few base loads added to the system downstream of Sites #5 and #6. A modified calibration curve for Site #6 was developed based on the recorded flow at the WWTP minus the modified calibration curve for Site #5.

Modeled pump station flow and runtimes were reviewed and compared to pump station data provided by the City. Additional pump station information can be found in Section 3. Generally, modeled pump station flows were within 15% of the stations' reported capacities. PS#2 runs with high and low settings. A summary of modeled pump station flows can be found in Appendix F.

<u>Wet Weather Flow (WWF) Calibration</u>

The RTK method was used for rainfall-derived infiltration and inflow (RDII) prediction. Rainfall data for two 72-hour periods with the highest cumulative rainfalls during the period of flow monitoring was utilized to calibrate wet weather flows (January 2nd through 4th with 2.15 inches and January 11th through 13th with 2.30 inches). The storm event rainfall was entered into InfoSWMM and RTK parameters were then adjusted to calibrate the model with flow monitoring data. Again, total modeled flows at the WWTP were compared to the targeted average daily flow and WWTP influent flow data, in addition to calibrating the model at various locations within the collection system. An example of wet weather flow calibration results is shown below in Chart 4-2 and Chart 4-3. RTK values were adjusted to calibrate the model to meet the higher peaks between the two storm events. Generally, the first flow period of January 2nd through January 4th presented a larger response to rainfall than the second flow period, resulting in calibrated flows tending to be slightly higher than observed data for the second calibration period. Sites #1 and #3 had equipment issues overlapping a portion of the January 2nd-4th event and data was not recorded for a portion of the 4th at the sites. Data for the first rainfall event on the 3rd was still captured by both sites, so the calibration efforts for the Jan 2nd – 4th focused on matching the first rainfall response. Wet weather calibration curves for Basins 5 and 6 were developed using the same method as their base flow calibration counterparts. Calibration information on the remaining flow meters can be found in Appendix F. Pump runtime data was used to inform RTK values upstream of pump stations.

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

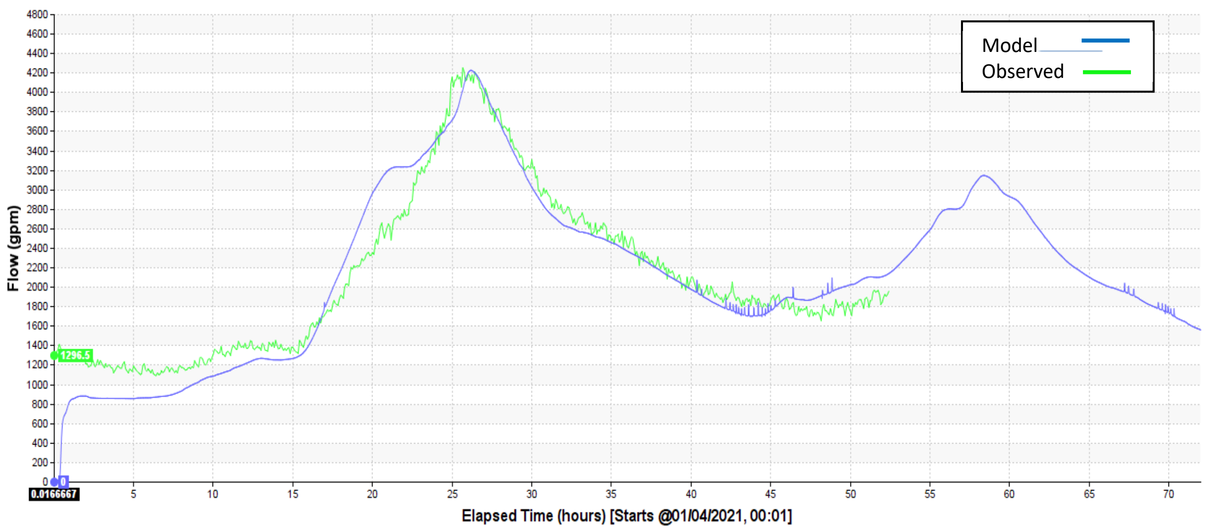
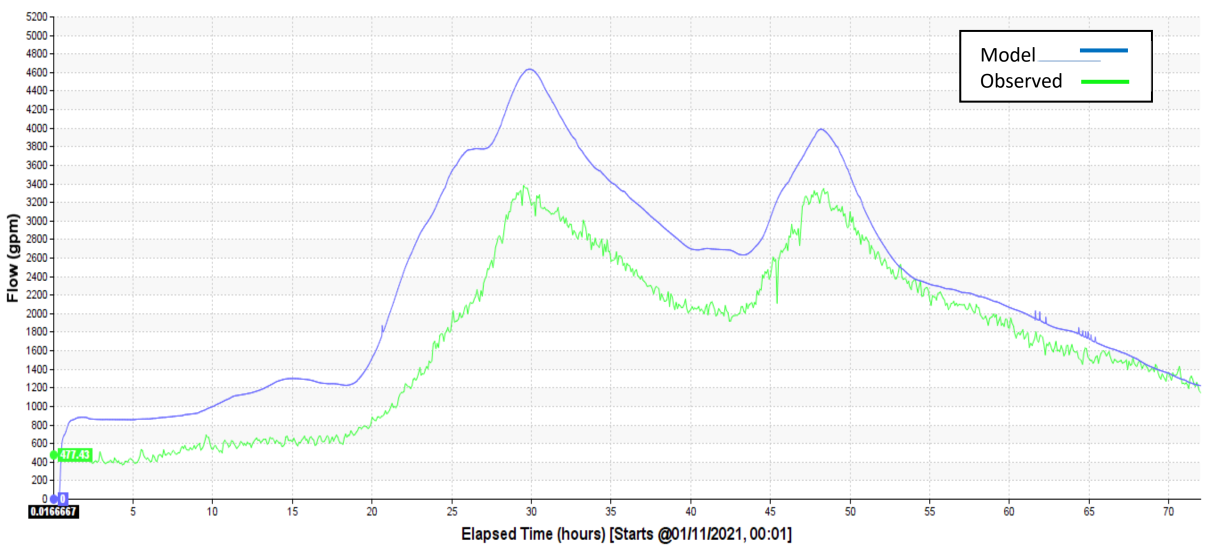


CHART 4-3: SAMPLE WET WEATHER CALIBRATION SITE 3, JAN 11TH - 13TH



Columbia City wastewater discharges to the collection system in St. Helens through a 6-inch forcemain. Two separate pump stations and the water treatment plant (WTP), also in St. Helens, discharge to the same forcemain. Modeling of Columbia City's pump stations was not included in the scope of this study. A maximum discharge estimate of 500 gpm from the Columbia City forcemain was taken from the 2013 Columbia City Master Plan. I/I contributions from Columbia City could result in an increase of pump starts and runtime but would not result in an increase to the peak pumping capacity. An assumed constant point load of 575 gpm (500 gpm plus a 15% safety factor to account for unknowns in pumping fluctuations) was used to model flows from Columbia City during wet weather.

Design Storm

The design storm used for model evaluation was the 5-year, 24-hour storm event. A standard 24-hour Natural Resources Conservation Service rainfall distribution for a Type 1A storm was used. The rainfall for the 5-year, 24-hour storm event from National Oceanic and Atmospheric Administration isopluvial maps is 2.4 inches. This was used as the multiplier for the Type 1A storm hyetograph. The existing system calibrated model was run with the design storm event.

The modeled peak instantaneous (PIF_5) and peak day ($PDAF_5$) flows at the WWTP were compared to the modified PIF_5 and $PDAF_5$ planning criteria (Table 4-1). The modeled peak instantaneous flows and peak day at the plant were lower than the planning criteria. These low peak flows were primarily due to surcharging and flooding throughout the system. The flow comparison is summarized in Table 4-1. The model was also ran with increased pipe capacities to review system flows if capacity limitations in the system were alleviated. These flows are summarized in Table 4-1 as Unconstrained Model Outflow. The calibrated model flow, with capacity limitations eliminated, is within 10% of the modified planning criteria flows. Additional discussion and details of existing system capacity limitations are summarized in the following section.

TABLE 4-1: PLANNING CRITERIA VS. MODELED PEAK FLOWS

Flow	Modified Planning Critieria (MGD)	Model Outflow (MGD)	Unconstrained Model Outflow (MGD)
$PDAF_5$	19.9	16.2	17.8
PIF_5	26.0	23.2	26.9

4.1.3 EXISTING SYSTEM EVALUATION

The calibrated model was used to assess the existing system capacity during a 5-year, 24-hour design storm event. Figure 18 in Appendix A illustrates the potential overflow sites and pipe capacity limitations identified during the existing system peak instantaneous flow model evaluation. The figure is color-coded to show a gradation of pipes based on utilized capacity (e.g., red = flowing at >100% capacity, orange = flowing at 85-99% of capacity, yellow = flowing at 75-84% capacity, etc.). As stated in Section 2, the planning criteria for undersized pipelines is if the flow is equal or greater than 85% of full capacity based on maximum depth of flow (d/D). The figure also displays manholes which experience surcharging and have the potential to overflow according to the model analysis. As stated in Section 2, the Department of Environmental Quality prohibits sanitary sewer overflows, and surcharging in wastewater systems is generally discouraged.

The existing system evaluation shows a significant portion of the modeled trunk lines operating at or above capacity. There are pipelines operating at or above capacity in each of the six basins, with most basins having manholes with the potential to overflow. Several of the deficiencies are caused by undersized trunklines. There are a few areas, where a downstream bottleneck is causing the upstream surcharging. Additional discussion of each deficiency location and alternatives to address the issue are discussed in Section 5.

Table 4-2 shows a list of modeled manholes that may experience potential overflows during peak flow conditions. Each of these locations experience surcharging due to downstream capacity constraints. A few of the listed manholes have abnormally shallow depths (under 4 feet). The elevation data is from the City's GIS database. The City may want to field measure the shallow manholes to assess accuracy of recorded depth data.

TABLE 4-2: POTENTIAL OVERFLOW LOCATIONS

Basin	Manhole Name	Manhole Depth (ft)
1	N30	2.5
1	N33	4.2
2	WC4	2.0
2	WC5	3.5
2	WE11	4.6
2	WE9	4.3
2	W49A	5.6
2	WJ4	4.6
3	NC9	6.0
4	M3	4.0
4	M12	3.8
4	M14	3.5
4	M15	3.4
4	MP4	4.4
6	DG1C	4.4
6	D9	6.3
6	S19A	4.9
6	W33	4.2

4.1.4 CRITICAL SLOPE AREAS

The City's 2003 Engineering Department Public Facilities Construction Standards Manual provides minimum pipe slopes for sanitary wastewater gravity mains (Table 4-3). Modeled gravity main slopes were compared with the recommended minimum slopes, and pipes that are less than their recommended minimum slope are highlighted with different colors based on pipe diameter in Figure 19 in Appendix A. Low slopes can cause capacity issues and require higher than normal O&M. These mains should be monitored for capacity, odor, and solids buildup problems. Pipes with low slopes may need to be cleaned more frequently to prevent solids buildup and flow disruption. The City currently cleans approximately 3% to 5% (10,000 to 15,000 ft) of the pipes in the collection system every year, with approximately 5% of the cleaned pipes CCTV inspected annually (~0.25% of the system). It is recommended the City perform a regular maintenance schedule of inspecting and cleaning approximately 17-20% of the pipes in the collection system per year. It should be noted if areas have consistent solids buildup or flow disruption issues, they may need to be cleaned more frequently.

Additionally, during review of the City's GIS, several areas through the City appeared to have trunklines beneath private property and potentially beneath private structures. While GIS map imagery may not be perfectly accurate, it provides reasonable proof of trunkline locations. Generally, it is advised that collection system pipelines, especially larger trunklines, do not cross under private structures, as it can cause additional liability in the case of pipe breaks or defects. Figure 19 in Appendix A displays the location of pipe segments whose location is suspected to be beneath established private structures. It is recommended these pipelines be relocated into the road right-of-way if improvements are completed.

TABLE 4-3: MINIMUM PIPE SLOPES

Pipe Size (inches)	Minimum Slope in Percent (feet per 100 feet)
8	0.40
10	0.28
12	0.22
15	0.15
18	0.12
21	0.10
24	0.08
27	0.07
30	0.06

Source: City of St. Helens Engineering Department Public Facilities Construction Standards Manual, 540.2.3

4.1.5 PUMP STATION RESILIENCY

The scope of work included assessing pump station resiliency via a comparison of peak hour inflows to firm capacity and a review of emergency power. The existing system's emergency power deficiencies are recorded in Section 3, and recommendations to resolve the deficiencies can be found in Section 7.

Concerning firm capacity, both the model and pump runtime data were reviewed for inadequate firm capacity. For the modeled pump stations, peak inflows to pump stations were estimated using the calibrated model. During the model evaluation, both pumps at PS#7 and PS#11 had to run during peak flows, indicating that peak flows had exceeded the pump stations' firm capacities.

Additionally, City-provided available pump runtime data from 2016 to 2020 was reviewed by Keller Associates. The date range of available data varied between pump stations, with PS#1, PS#2, PS#5, and PS#11 only having data as early as mid-2017. Data provided the number of starts per pump per hour and hourly runtime. The runtime data was analyzed to evaluate if the data indicated that all pumps had run at the same time (indication of nearing or exceeding firm capacity). A summary of the results is listed below.

- Data for PS#5 shows the station exceeding its firm capacity during large wet weather events, with the station having two or more days where the combined pump runtime was over 60 minutes per hour, which indicates both pumps were running together.
- PS#2 runs on a VFD with a high and low setting. The high setting VFD turns on after both pumps are running and the level exceeds the second high water setting. The pump station turns off one pump when the other pump operates in the high setting, which makes it difficult to assess potential exceedance of firm capacity. However, there were two instances during the largest rain event on 2/12/2019 where one pump ran on the high setting for 60 minutes on the hour, indicating that inflows may have exceeded firm capacity.
- PS#1 and PS#3 show that both pumps ran during the largest rain event on 2/12/2019. This rain event may have been larger than a 5-year storm event, as the City's anticipated 5-year storm is 2.4 inches and this rainfall event had two consecutive days of 1.8- and 2.2-inch rainfall.
- PS#4 shows day periods where one pump ran for 24 hours but the second did not turn on. This may be an indication of a malfunctioning pump or reporting software. The City should review this data to assess if a potential capacity deficiency is indicated.
- Due to the nature of the data received, it was not possible to decern if PS#4 and PS#7 ran over their firm capacities. However, they both displayed higher runtimes over 10 hours a day

during wet weather events, which may indicate both pumps running and/or that the stations are nearing firm capacity.

- It is recommended that the City continue to monitor runtimes for PS#1, PS#2, PS#3, PS#4, PS#5, and PS#7, and configure the SCADA system to alarm when both are running, which is indicative of a lack of firm capacity.

Generally, a lack of firm capacity presents potential risk to the system. Pump stations are evaluated at their firm capacity to build a level of redundancy into a system's pumping capacity. Firm capacity accounts for one pump to breakdown or be offline. Inadequate firm capacity increases risks of overflows in the system. It is recommended for the City to include an alarm at all pump stations to notify operators if all pumps turn on. This alerts operators to the potential of inadequate firm capacity at a station and can serve as a trigger for improvements. Pump station alternatives and recommendations can be found in Sections 5 and 7 of this report.

4.1.6 PUMP STATION RISK OF FAILURE

The risk of failure of an asset is a combination of the likelihood of failure and consequence of failure. Likelihood of failure is a measure of how likely an asset is to fail. Components of likelihood of failure for a pump station include items such as age, redundancy, alarms, condition, etc. Consequence of failure is a measure of the impacts a failure would have on the system and surrounding community. Components of consequence of failure for a pump station include items such as proximity to wetlands and waterways, number of homes served by pump station, industrial or commercial entities served by pump station, etc. An evaluation of the risks of failure can provide an importance, urgency, or priority to assets and provide guidance on the order in which asset deficiencies should be addressed. Assets with the highest risk of failure (product of likelihood of failure and consequence of failure) should be repaired or replaced first as they pose the largest threat to a system and community.

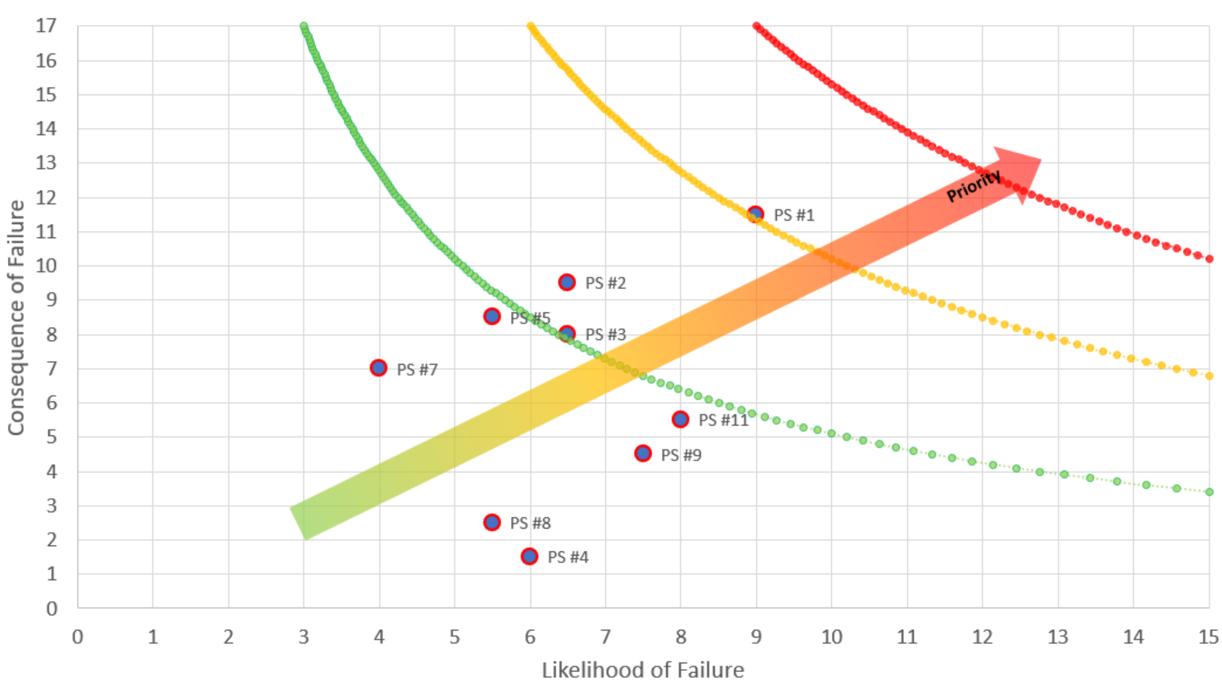
A high-level risk of failure evaluation was completed for the City-owned pump stations. A set of factors for likelihood of failure and consequence of failure were developed with input from City staff. These factors are summarized below.

- Likelihood of failure factors
    - Liquification hazard
    - Landslide susceptibility
    - Backup power
    - Capacity vs. demand
    - Wetwell and piping condition
    - Safety, security, and access
    - Age
    - Sensor and alarm redundancy
    - Influence from flooding (100-year floodplain)
- Consequence of failure factors
    - Capacity of pump station
    - Environmentally sensitive areas (proximity to wetlands/waterways or stormwater system)
    - Type of development served (i.e. hospitals, schools, emergency services, historical sites, industrial zone, or commercial zone)
    - Proximity for flooding private property
    - Portion of community served
    - Estimate of time to overflow

Each pump station was then assigned a score for each factor. For example, the consequence of failure factor "Portion of community served" was assigned a score of 0-3 for each pump station based on the number of EDUs served by the pump station. Pump stations serving less than 5 EDUs were given a score of 0. Those serving 5-50 EDUs were assigned a score of 1, 50-100 EDUs a score of 2, and over 100 EDUs a score of 3. The range of scores for each factor can be found in Appendix G.

After each pump station received a score for each factor, the likelihood of failure scores were totaled and the consequence of failure scores were totaled. The risk of failure for an asset is the product of its likelihood of failure and consequence of failure scores. This risk of failure can be represented graphically as shown in Figure 4-2. The arrow shows increasing risk of failure while the red, yellow, and green dotted lines are equipotential risk lines (all points on the line have equal risk of failure scores). The analysis shows that PS#1 and PS#2 have the highest risks of failure. A failure at one of these pump stations would have the largest impact on the community and is most likely to happen based on the factors evaluated. This analysis indicates that deficiencies at these pump stations should be addressed soon after identified. The risk of failure assessment can be used as a tool to prioritize recommended improvements described in Section 7, as well as provide guidance on importance, urgency, or priority to address any deficiencies identified in the future.

FIGURE 4-2: PUMP STATION RISK OF FAILURE ANALYSIS



4.2 FUTURE COLLECTION SYSTEM PERFORMANCE

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

4.2.1 FUTURE FLOW PROJECTIONS & MODEL SCENARIOS

Future loads were distributed based on PSU population projections and City projected future residential, commercial, and industrial growth (additional details in Section 2.4.11). Flows per capita for projected population growth were assumed to be similar to existing flows per capita. Flowrates anticipated in the 20-year planning period are identified in Table 2-6 in Section 2. Growth areas identified by the City can be found in Figure 9 in Appendix A. Residential flows were projected using future growth areas, City zoning, projected number of equivalent dwelling units, and ADWF per capita. Projected industrial and commercial development is anticipated to grow within the industrial and commercial areas identified by the City, with both zoning designations assumed to contribute 1,500 gallons per acre per day (gpad) to the wastewater system. Residential, commercial and industrial loading calculations for the growth areas can be found in Appendix B.

A 20-year $PDAF_5$ model was created, using the calibrated $PDAF_5$ existing system with the addition of the 20-year flows calculated for each growth area. The dry weather loads were applied to the trunkline manhole best fit to receive loads from each growth area. For the RDII loading on the 20-year growth areas, the RTK method was once again utilized. Based on direction from the City, Keller Associates assumed that the growth areas would have reduced RDII influence, as defects and I/I are less likely in new development. RDII flows were estimated to be equal to approximately 80% of the lowest existing RDII of the flow monitoring basins.

After applying the 20-year loads and RDII, the modeled peak instantaneous (PIF_5) and peak day ($PDAF_5$) flows at the WWTP were compared to the modified PIF_5 and $PDAF_5$ planning criteria (Table 4-4). Similar to the existing system, the 20-year modeled peak instantaneous flows and peak day at the plant were lower than the planning criteria, primarily due to surcharging and flooding throughout the system. The 20-year model was also ran with increased pipe capacities to review system flows if capacity limitations in the system were alleviated. These flows are summarized in Table 4-4 as Unconstrained 20-year Model Outflow. The calibrated model flow, with capacity limitations eliminated, is within 10% of the modified planning criteria flows.

TABLE 4-4: 20-YEAR PLANNING CRITERIA VS. MODELED PEAK FLOWS

Flow	Modified 2040 Planning Critieria (MGD)	20-Year Model Outflow (MGD)	Unconstrained 20-Year Model Outflow (MGD)
$PDAF_5$	21.4	18.3	21.0
PIF_5	28.2	25.5	31.7

4.2.2 20-YEAR SYSTEM EVALUATION

The 20-year model was used to assess the existing system capacity during a 5-year, 24-hour design storm event with 2040 flow projections. Peak 20-year flows exceed existing firm capacity of PS#7 and #11. PS#7 and #11 modeled capacities were increased to handle peak 20-year flows and assess potential downstream trunkline capacity limitations. Figure 20 in Appendix A illustrates the potential overflow sites and pipe capacity limitations identified during the 20-year system peak instantaneous flow model evaluation, using the same color-coded criteria established in the existing system evaluation. The same planning criteria as the existing system evaluation for pipelines and manholes was utilized in the analysis (d/D of 85% or higher indicates undersized pipelines, and no sanitary overflows allowed at manholes).

The 20-year system evaluation tells a similar story to the existing system evaluation: each of the six basins show a portion of the modeled trunk lines operating at or above capacity, with most basins having manholes with the potential to overflow. Problems exhibited in the existing system evaluation are exacerbated in the 20-year evaluation and many of the deficiencies are caused by undersized trunklines. The largest increases in additional surcharging and potential overflow locations in the 20-year evaluation occur on Gable Road and Old Portland Road from Kaster Road east. Additional discussion of each deficiency location and alternatives to address the issue are discussed in Section 5. The manholes that have the potential for overflow during peak conditions in the 20-year model overlap are presented in Table 4-5. It should be recognized that the potential overflow locations present in the existing system (Table 4-2) are still overflow locations in the 20-year model but have not been duplicated in Table 4-5.

TABLE 4-5: POTENTIAL OVERFLOW LOCATIONS IN THE 20-YEAR MODEL

Basin	Manhole Name	Manhole Depth (ft)
1	NQ1A	3.6
2	WC8	6.9
2	WJ11	4.1
2	WC15	5.7
2	WE12	4.8
2	WC17	6.6
2	WE16	4.4
2	WC14E	5.9
2	WC16	6.3
2	WC9	8.6
4	M2	8.0
5	I9A	7.6

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SECTION 5 - COLLECTION SYSTEM IMPROVEMENT ALTERNATIVES

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

5.1 PLANNING CRITERIA

The planning criteria used for this collection system facilities plan are outlined in Section 2 and summarized as follows for reference. The City's conveyance system will be evaluated for the projected 2040 peak instantaneous flow rates associated with the 5-year, 24-hour storm event (PIF_5 in Table 2-6). Criteria for requiring improvements is when the maximum flow depth/full depth (d/D) of a pipe is greater than 85%. Collection systems pipeline improvements will be sized to achieve d/D of less than 85% during the 2040 PIF_5 flow. Additionally, it should be noted that efforts to reduce I/I in the collection system could further extend the life of the pipeline with regards to capacity.

5.2 PUMP STATIONS

Pump station existing conditions were summarized in Section 3 and existing capacity limitations in Section 4. The deficiencies highlighted in Section 3 require relatively minor improvements to resolve. Capacity limitations identified in Section 4 show PS#7 and #11 are undersized for expected peak 20-year flows. No feasible alternatives were identified for pump station capacity improvements. Recommended short- and long-term pump station condition and capacity improvements are summarized in Section 7. The collection system alternatives below in Section 5.4 were evaluated with the assumption that PS#7 and #11 firm capacities were increased to meet expected peak 20-year flows.

5.3 SUMP PUMP ALTERNATIVES

As mentioned in Section 3, the rapid and significant rainfall response in certain sewer trunklines observed by City staff suggests that a number of areas within the City have illegal sump pump connections to the wastewater system. These areas are highlighted in Figure 14 in Appendix A. The City would like to identify and disconnect sump pumps in these areas to reduce I/I to the sewer system. The following alternatives have been identified to aid the City in this goal.

5.3.1 ALTERNATIVE SP1 – EDUCATIONAL MATERIAL

In other municipalities with illegal sump pump connections, targeted educational campaigns have been used to inform customers about sump pumps. This generally includes distribution of flyers or a page on the City's website providing information to customers. The information includes a description of what sump pumps are, visual aid on identifying them in the home, and information regarding the local law regarding sump pumps. In municipalities where sump pump connection to the wastewater system is against code, it is important to notify residents that the cross-connection is a code violation and should be disconnected from the wastewater system. Examples of flyers used in other municipalities with a similar ban on sump pump cross-connections can be found in Appendix H.

In addition to providing educational materials, some cities and municipalities offer assistance with disconnection of sump pumps. This generally involves including a phone number on the educational material that customers can call and receive aid from City staff on disconnecting their sump pump.

5.3.2 ALTERNATIVE SP2 – SMOKE TESTING

Smoke testing is a standard method used in I/I studies to identify defects in trunklines and service laterals, as well as illegal cross-connections. Smoke testing involves using smoker equipment to pump smoke into a collection system via a manhole, and then monitor the area served by the upstream system.

For identifying sump pump connections, houses with sump pumps or cross-connections may see smoke rising from around the foundation of the house. By visual inspection, houses are identified and the residents informed that they likely have an illegal sump pump connection. If the City decides to perform a more in-depth I/I study for the areas identified, then the City can perform smoke testing to both identify system defects in trunklines/laterals and the location of sump pumps simultaneously. Similar to alternative SP1, the City may offer staff support in helping customers disconnect their sump pump systems to ensure the disconnection is completed properly.

5.3.3 ALTERNATIVE SP3 – DYE TESTING AND CCTV

Dye testing and CCTV are also typical methods that can be used to detect cross-connections in a collection system. Dye testing involves dropping colored dye at or above a suspected cross-connection point (a basement drain, or area drain) and monitoring the collection system downstream, either through visual inspection in a manhole or cleanout, or via CCTV rover placed in the collection system. If dye is observed in the flow, it is indicative of a cross-connection.

The drawback of this alternative for identifying sump pump cross-connections, is the dye would have to be placed at the inlet of the sump pump. The location of the pumps is what is posing to be the biggest challenge for City staff. As such, this alternative is not recommended for identifying sump pump locations.

5.3.4 ALTERNATIVE SP4 – VISUAL INSPECTION

Another alternative is visual inspection. This involves City staff going to each property and inspecting the homes for potential cross connections. Primarily, storm drains and downspouts on the outside of the house that disappear into the ground and do not discharge to the yard are primary candidates for a cross connection.

The drawback of this method is that, in general, sump pumps are located within a basement or the foundation of a home and may not be visible from exterior inspection alone.

5.3.5 ALTERNATIVE SP5 – POINT-OF-SALE INSPECTION

The next alternative is Point-of-Sale Inspection. City staff can include a code requirement or ordinance to inspect each home for sump pump connections prior to sale. This type of inspection would require private homeowners/inspectors to identify and report to the City about which homes are equipped with sump pumps. From there, enforcement of disconnecting the pump can occur. The drawback to this method is that only homes going through inspection and sale will be affected.

5.3.6 ALTERNATIVE SP6 – REWARD-BASED DISCONNECT INCENTIVE

The City has also considered a reward-based incentive program, whereby owners of sump pumps would be incentivized to voluntarily disconnect their system from the sewer system. This reward could come in the form of direct monetary payment, or a credit on future sewer bills to the customer. The City currently has an annual budget directed to I/I projects, a portion of which City staff has expressed could be used for this incentive program.

Similar to Alternative SP1, the City could offer assistance in disconnecting the sump pumps. This would ensure a proper disconnect from the system, and staff could present the reward to the customer in a single trip. Alternative SP6 could be used in conjunction with Alternative SP1, as the educational material distributed can also serve as an advertisement for the incentive program.

See Table 5-1 below for a summary of the benefits and drawbacks of each alternative. A discussion on updates to the City’s code to address sump pumps can be found in Section 6.

TABLE 5-1: SUMMARY OF SUMP PUMP ALTERNATIVES

Alternative	Benefits	Drawbacks
SP1: Educational Material	• Cost efficient • Relatively easy to develop and distribute information	• No guarantee customers will disconnect sump pumps when informed.
SP2: Smoke Testing	• Effective at identifying cross connections, defects, and some sump pump locations • Can reduce overall cost by performing in conjunction with established I/I effort	• More expensive than alternative SP1 or SP4
SP3: Dye Testing and CCTV	• Effective at identifying system cross-connections	• Need to place dye at inlet of sump pumps, doesn't aid in identifying locations of pumps
SP4: Visual Inspection	• Can identify cross-connections to the collection system • Can be performed in conjunction with typical staff inspections/routine	• May be difficult to locate sump pumps on visual inspection alone (without entering the property or structure)
SP5: Point-of-Sale Inspection	• Puts responsibility on homeowner to identify and disconnect sump pump during home sales	• Only affects homes going through the selling process
SP6: Reward-Based Disconnect Incentive	• Provides additional incentive for users to disconnect sump pumps • Potential for more disconnects than SP1	• Increased cost to City for monetary payout or decreased revenue for billing credit

5.4 COLLECTION SYSTEM ALTERNATIVES

Collection system deficiencies discussed in Section 4 (Figure 20) reflect potential overflow locations and capacity issues. Alternatives for addressing system deficiencies in the following sections are organized by each of the six flow monitoring basins (Figure 16). Some of the deficiencies identified in Section 4 do not have multiple, feasible alternatives for improvements. These improvements are included in the following sections and are the recommended method to address the deficiency.

Preliminary cost estimates were evaluated for alternatives comparisons. Preliminary cost estimates are summarized in Table 5-2 at the end of this section. Advantages and disadvantages of alternatives, including capital cost and operations and maintenance (O&M) considerations, are also discussed below. Additional cost estimate details can be found in Appendix I. It should be noted that I/I reduction efforts undertaken by the City may decrease peak flows in the collection system, and could delay or eliminate the need for some of the capital improvements.

5.4.1 BASIN 1

1.a - Upsize Existing System:

Modeling depicts that most of the pipeline downstream and upstream of Kindre Street is undersized. The existing 10-inch pipeline should be upsized to a 15-inch pipeline and the pipeline segment between Kindre Street and Kelly Street should be upsized to an 18-inch pipeline to handle the projected 2040 PIF5 flows. Other methods of redirecting flow or adding additional parallel pipelines are not deemed cost effective for this area.

5.4.2 BASIN 2

The alternatives below were evaluated with the assumption that PS#11 firm capacity was increased to handle expected peak 20-year flows. Additional details on recommended pump station improvements are in Section 7.

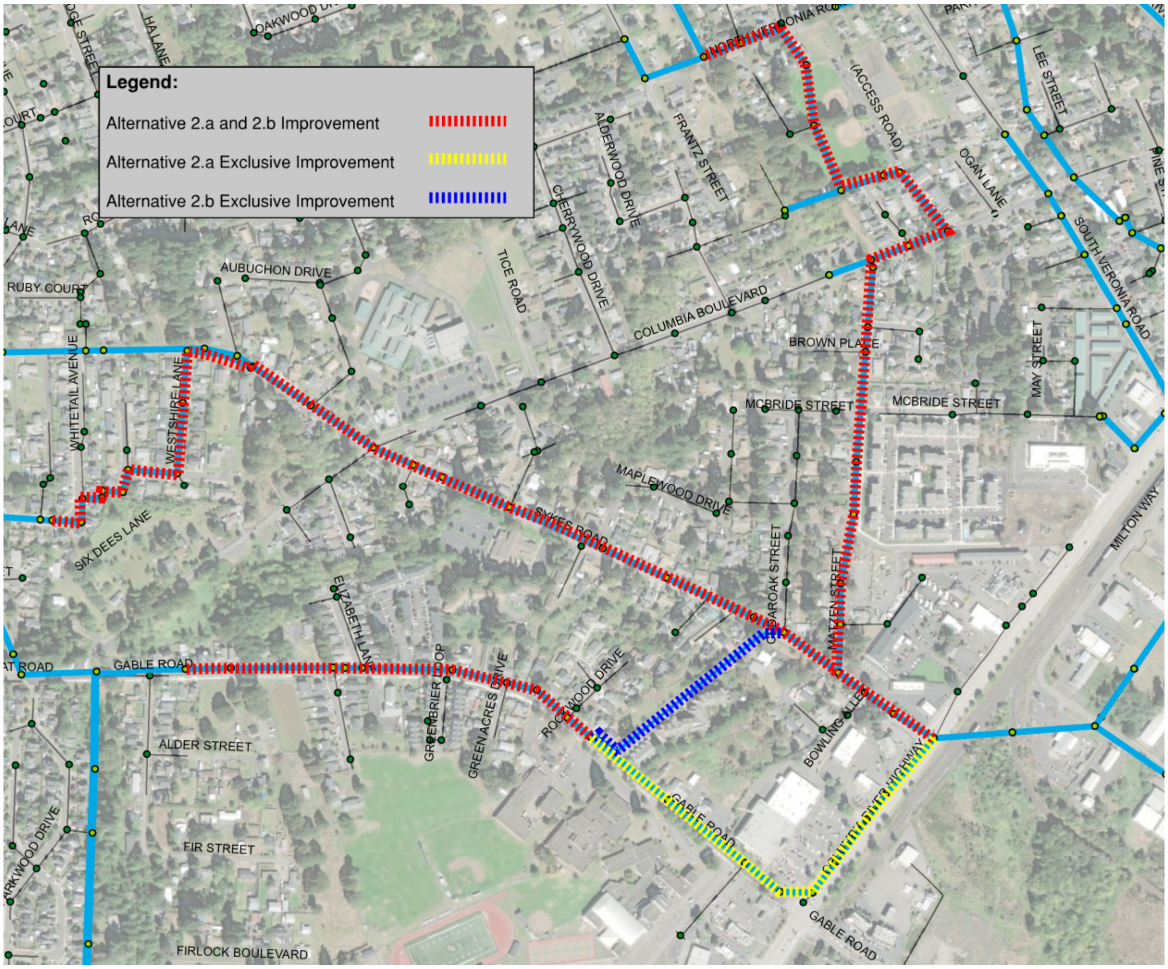
2.a - Upsize Existing System:

Many pipelines in Basin 2 are undersized for the projected flows. Pipeline size increases to handle 20-year PIF_5 flows include the trunkline along Gable Road, the trunkline along Sykes Road, the trunkline along Matzen Street, and the 8-inch line along Westshire Lane as shown in Figure 5-1. Typically, all these trunklines require two nominal pipe size increases to meet the 0.85 d/D criteria for the pipeline during PIF_5.

2.b - Upsize Existing System and Redirect flow from Gable Rd. to Sykes Rd.

Alternatively, flow down the Gable Road trunkline could be redirected to Skyes Road via a 12-inch pipeline from manhole WC9 to manhole W42. This would alleviate the need for improvements downstream on Gable Road. The rest of the pipeline upsizing outlined in Alternative 2.a would also be required for this alternative. The preliminary cost comparison between the two alternatives is depicted in Table 5-2 (located in Section 5.4.6), and no significant difference in O&M efforts could be distinguished when comparing these alternatives. The visual depiction of the two alternatives can be found in Figure 5-1.

FIGURE 5-1: BASIN 2 IMPROVEMENT ALTERNATIVE COMPARISON



5.4.3 BASIN 3

<u>3.a - Upsize Existing System:</u>

Only a few segments of the existing system in Basin 3 are considered to be undersized. If the pipe segment along N 10th Street to West Street is upsized from 12-inch to 15-inch in diameter, the pipeline will have adequate capacity to handle 20-year PIF_5 flows. Other methods of redirecting flow or adding additional pipelines are not deemed cost effective for this area.

5.4.4 BASIN 4

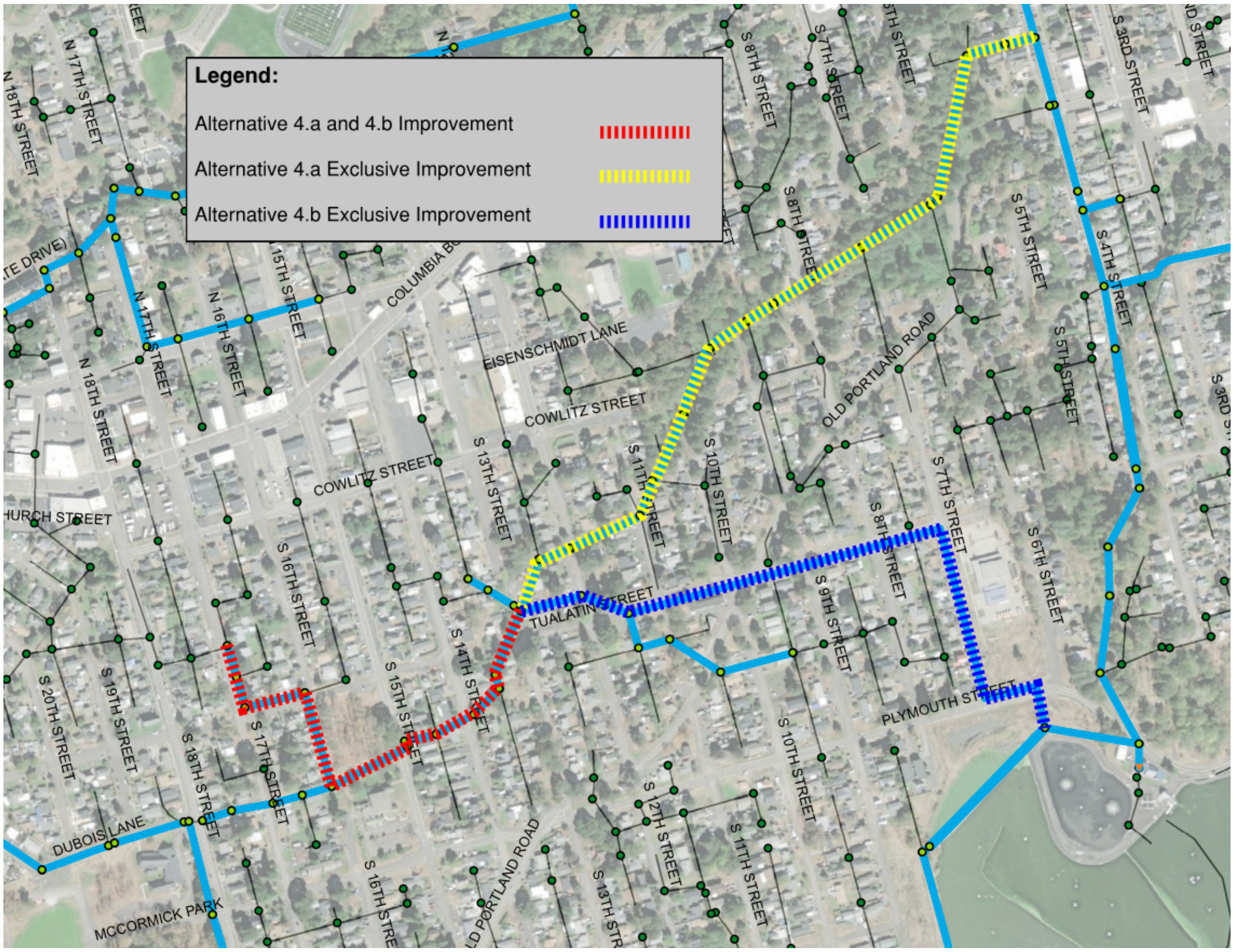
<u>4.a - Upsize Existing System:</u>

The majority of the 12-inch to 18-inch trunkline segments within Basin 4 are undersized for 20-year flows. To alleviate this, the majority of the pipeline segments from the Basin 5 trunkline to S 17th Street needs to be increased by one nominal pipe size, 15- to 21-inch segments.

<u>4.b – Upsize Existing System and Redirect flow from Tualatin Street to Basin 6:</u>

Alternatively, basin flow west of S 13th Street could be redirected down Tualatin Road and S 7th Street to alleviate the eastern portion of the basin and convey flow directly to manhole S1 in Basin 6, which has adequate capacity to handle 20-year flows from both Basin 6 and Basin 4 west of S 13th Street. This alternative would involve capping the existing pipe on S 13th Street, replacing the pipelines along Tualatin Street with a 15-inch trunkline sloped west to east, and construction of a new 15-inch trunkline from along Tualatin Street and S 7th Street to manhole S1 (south of S 6th Street). The main trunkline west of S 13th Street would still require upsizing from 10 and 12-inch to 12 and 15-inch (one nominal pipe diameter) to handle 20-year flows. No significant difference in O&M efforts could be distinguished when comparing these alternatives. Alternative 4.b opts to construct 2,760 feet of new pipe instead of upsizing the 3,220 feet of pipe east of S 13th Street. The cost comparison between the alternatives is presented in Table 5-2 (located in Section 5.4.6). A visual depiction of these alternatives is shown in Figure 5-2.

FIGURE 5-2: BASIN 4 IMPROVEMENT ALTERNATIVE COMPARISON



5.4.5 BASIN 5

5.a - Upsize Existing System:

The main 30-inch trunkline through Basin 5 is undersized for 20-year flows from Tualatin Street to Columbia Boulevard. An upsize to 36-inch pipelines north of manhole I9 (the inlet of basin 4) and 42-inch pipelines south of manhole I9 would be sufficient to handle 20-year PIF5 flows. The City's tunnel, adjacent to S 4th Street, consists of stacked 20 and 21-inch pipelines which are too undersized to handle peak flows. Upsizing each of the pipelines individually is not feasible due to their stacked nature. Thus, these pipelines should be replaced by a singular 42-inch pipeline. Open trenching may not be possible due to the nature of the tunnel; additional costs have been assumed to account for pipe removal and horizontal drilling.

Basin 5 also includes PS#1, which is expected to be relocated with the Riverfront development and will cause flows captured by this pump station to be discharged south of the tunnel near the WWTP, rather than north of the tunnel where the station currently discharges. This change does not re-direct enough flow to resolve capacity issues in the basin. Other methods of redirecting flow or adding additional pipelines were not deemed cost effective for this area.

5.4.6 BASIN 6

The alternatives below were evaluated with the assumption that PS#7 firm capacity and the southern trunkline capacity from west of Kaster Road to Plymouth Street were increased to handle expected peak 20-year flows upsized to 30-, 33-, and 36-inch pipeline. Additional details on recommended pump station and southern trunkline improvements are provided in Section 7. Cost estimate for the southern trunkline improvements is included in the Basin 6 alternatives cost estimates in Table 5-2.

<u>6.a – Upsize Existing System</u>

Basin 6 has several undersized pipelines, including trunklines along Port Avenue, Columbia River Highway, Dubois Lane, S 18th Street, Old Portland Road, and south of Umatilla Street. Pipe diameter increases are required ranging from one to three nominal sizes to convey the 20-year peak flows.

<u>6.b – Upsize Existing System and Redirect Flow from Old Portland Rd. to Kaster Rd.</u>

Rather than upsizing the length of pipeline between manhole S17 and S12 (along Old Portland Road and Umatilla Street), a new 15-inch pipeline can be constructed from manhole D1 (north of Portland Road) to manhole S20 on Kaster Road to convey flows directly to the 27-inch trunkline in Basin 6. The connection to the manhole on Portland Road can be capped, which would eliminate the need for upsizing the approximately 1,400 feet of pipe along Old Portland Road and Umatilla Street. The remainder of the pipeline upsizing presented in Alternative 6.a would still need to be completed in this alternative. A visual comparison of the alternatives can be found in Figure 5-3. The cost comparison between the two alternatives is presented in Table 5-2. No significant difference in O&M efforts could be distinguished when comparing these alternatives.

FIGURE 5-3: BASIN 6 IMPROVEMENT ALTERNATIVE COMPARISON

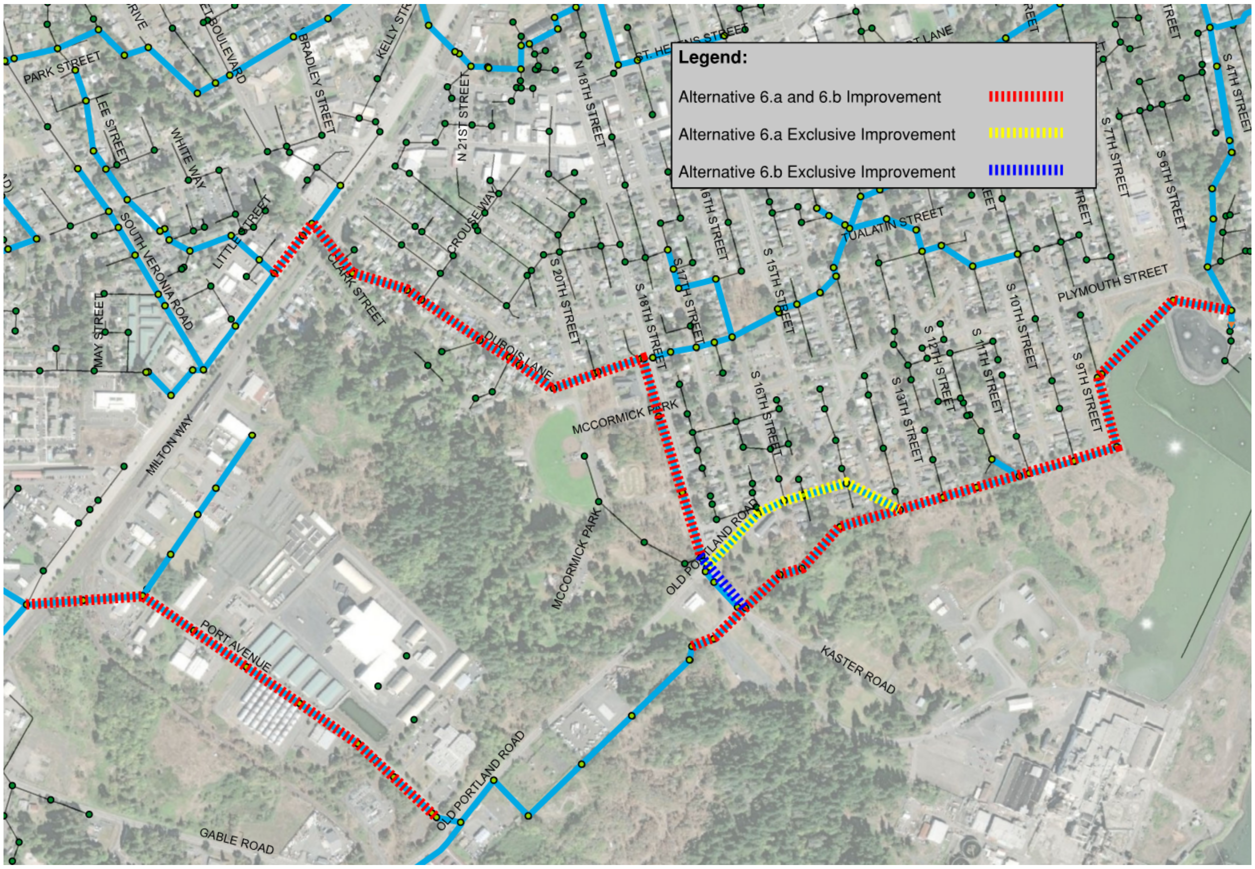


TABLE 5-2: SUMMARY OF COSTS FOR COLLECTION SYSTEM ALTERNATIVES

Alternative No.	Alternative	Estimated Total Project Cost (rounded)
1.a	Basin 1 - Pipeline Upsize	$1,800,000
2.a	Basin 2 - Pipeline Upsize	$9,500,000
2.b	Basin 2 - Pipeline Upsize and Redirect from Gable Rd. to Sykes Rd.	$9,400,000
3.a	Basin 3 - Pipeline Upsize	$1,200,000
4.a	Basin 4 - Pipeline Upsize	$3,700,000
4.b	Basin 4 - Pipeline Upsize and Redirect from Tualatin St. to Basin 6	$3,600,000
5.a	Basin 5 - Pipeline Upsize	$4,500,000
6.a	Basin 6 - Pipeline Upsize	$12,300,000
6.b	Basin 6 - Pipeline Upsize and Redirect from Old Portland Rd to Kaster Rd.	$11,500,000

In addition to these alternatives, installation of parallel facilities or taking no action could be considered. Parallel facilities could be constructed in areas with limited remaining capacity. This alternative would increase the system's capacity and generally costs less than full replacements. Another advantage of constructing parallel facilities is that existing infrastructure could be left in service while the parallel facilities are constructed. The disadvantages of this alternative include the long-term increase in maintenance costs associated with maintaining parallel facilities and the potential higher life-cycle costs associated with the eventual replacement or rehabilitation of the original pipeline. Additionally, the City has shallow bedrock throughout the majority of city limits, and the additional cost of rock excavation may make the prospect of parallel pipelines less desirable than upsizing pipelines within established trenches. City staff generally prefer to upsize existing gravity pipelines over the construction of parallel pipelines. This preference has been reflected in Table 5-2 above and in the recommended alternatives in Section 7.

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

I/I reduction improvements to the system may mitigate the need for large scale capital improvements. The City acknowledges that the I/I shown in the existing system flows is uniquely large compared to municipalities of similar size. Lowering peak flows decreases the likelihood of surcharged pipes or overflows to occur within the system. See Section 7 for additional discussion on recommended steps to reduce system I/I.

Section 7 summarizes the recommended alternatives to resolve the collection system deficiencies.

5.5 FUTURE INFRASTRUCTURE

5.5.1 RIVERFRONT DISTRICT

The City is currently evaluating development options for the Riverfront development, located adjacent to Columbia River and downtown. The development will need a pump station to provide sewer service to the area due to the topography. As part of this process, it is recommended the City relocate PS#1 to the south, adjacent to a planned S. 1st Street extension in the Riverfront District. This relocation would allow PS#1 to serve both the Riverfront development and its existing sewer basin. The existing sewer basin would be connected to a new trunkline in the Riverfront development and flow by gravity to the new PS#1. It is recommended that the firm capacity of the pump station be increased from 550 gpm to approximately 700 gpm to accommodate the anticipated 20-year flows from the existing sewer basin and the Riverfront development. Figure 21 in Appendix A depictions the proposed infrastructure overlayed with City planning figures. Additional information on the Riverfront Development can be found in the City's Riverfront Connector Plan, dated 2019, and the St. Helens Waterfront Framework Plan, dated December 2016. A copy of each is available on the City's website.

5.5.2 INDUSTRIAL BUSINESS PARK

The City's industrial business park is situated along the Columbia River and has historically been used by industries for wood products (formerly the Boise White Paper, LLC mill operations site) until the City acquired the 225-acre property. The City is seeking new opportunities for the business park and wastewater infrastructure should be planned for appropriately.

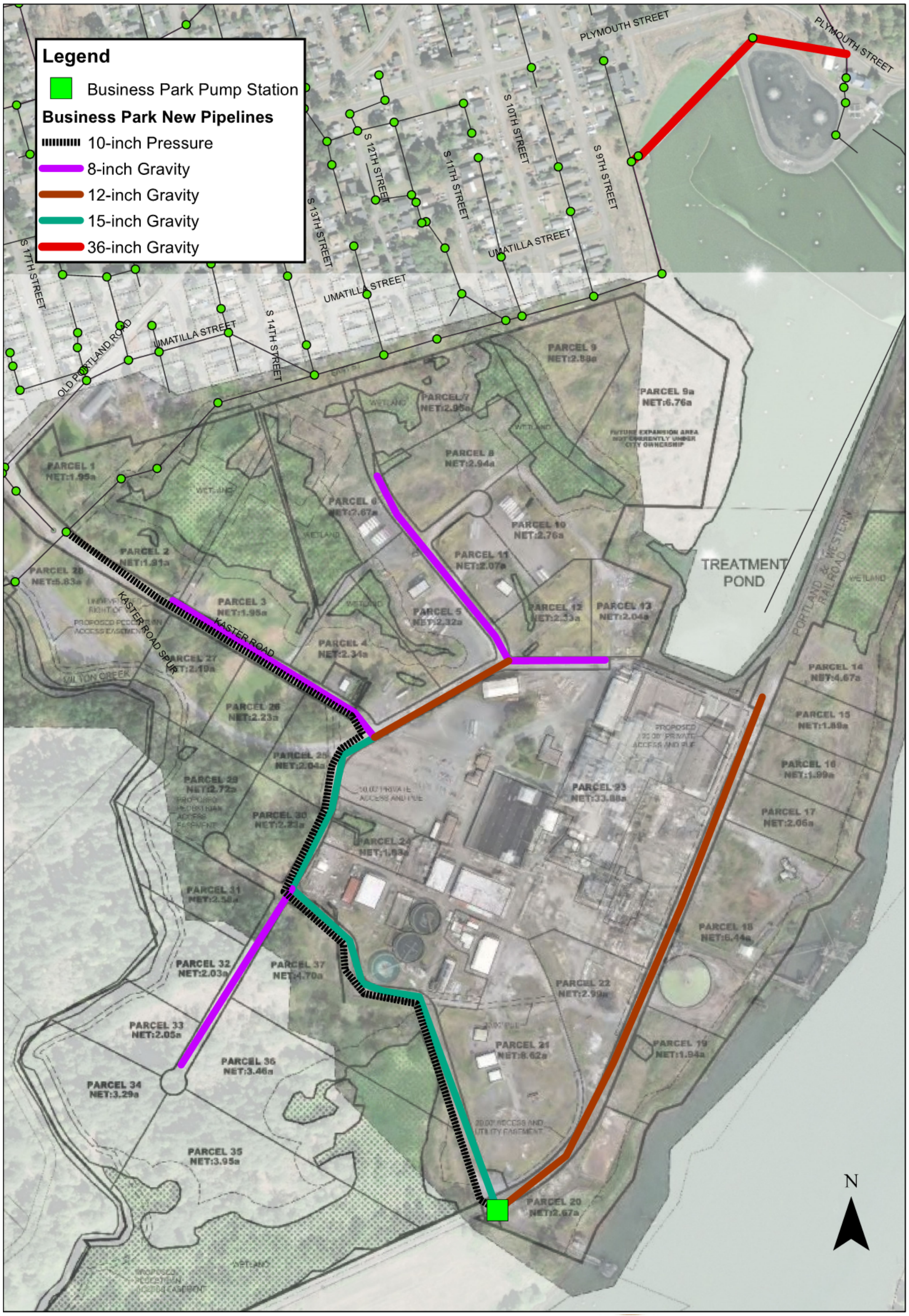
The City completed the St. Helens Industrial Business Park Parcellation Framework Report in July of 2020, which details the parcellation plan for the site and the existing infrastructure on the site (available on the City's website).

The topography of the site generally shows the ground elevation sloping down from northeast to southwest. The majority of the site cannot be served by gravity with the existing trunklines which border the north end of the property. To provide sewer service to most of the future development, a pump station will be needed. The pump station will likely need to be located near the waterfront to follow existing topography. The gravity sewer piping will follow the proposed roadway alignments and drain to the proposed pump station location. The force main can be routed along existing and/or proposed roadways and discharge to the existing trunkline on Kaster Road. The existing gravity trunkline downstream south of Umatilla Street and extending east has a section of parallel pipes which are capacity limited. The pipes exceed a d/D of 0.85, but do not surcharge above top of pipe during peak design flows.

The anticipated loading for the site matches the other projected industrial developments in the 20-year planning period. Flow was allocated to the property based on a 1,500 gpad base rate, which matches the allocations for the other industrial and commercial growth areas (details shown in Appendix B). The site is expected to flow by gravity to the proposed pump station. The pump station force main is proposed to discharge to the existing system in Kaster Road south of the intersection of Old Portland Road. The pump station firm capacity should be sized to handle the estimated 20-year peak flow for the development of approximately 1,300 gpm. Proposed pipelines are sized to handle peak flows at 85% full depth. The proposed wastewater pipe alignment, pump station, and force main are shown in Figure 5-4 (see Figure 22 in Appendix A for full sized figure). It is recommended that the existing parallel pipelines and pipeline segment downstream be upsized to 36-inch pipeline as part of the improvements to accommodate the additional flows from the Industrial Business Park (Figure 5-4). The flow rate assumptions made in this plan and subsequent infrastructure sizing should be re-evaluated once more information is known on the specific industries the development will serve and during the predesign phase.

Cost estimates for the proposed wastewater infrastructure for the business park can be found in Section 7.

FIGURE 5-4: INDUSTRIAL BUSINESS PARK PROPOSED WASTEWATER INFRASTRUCTURE



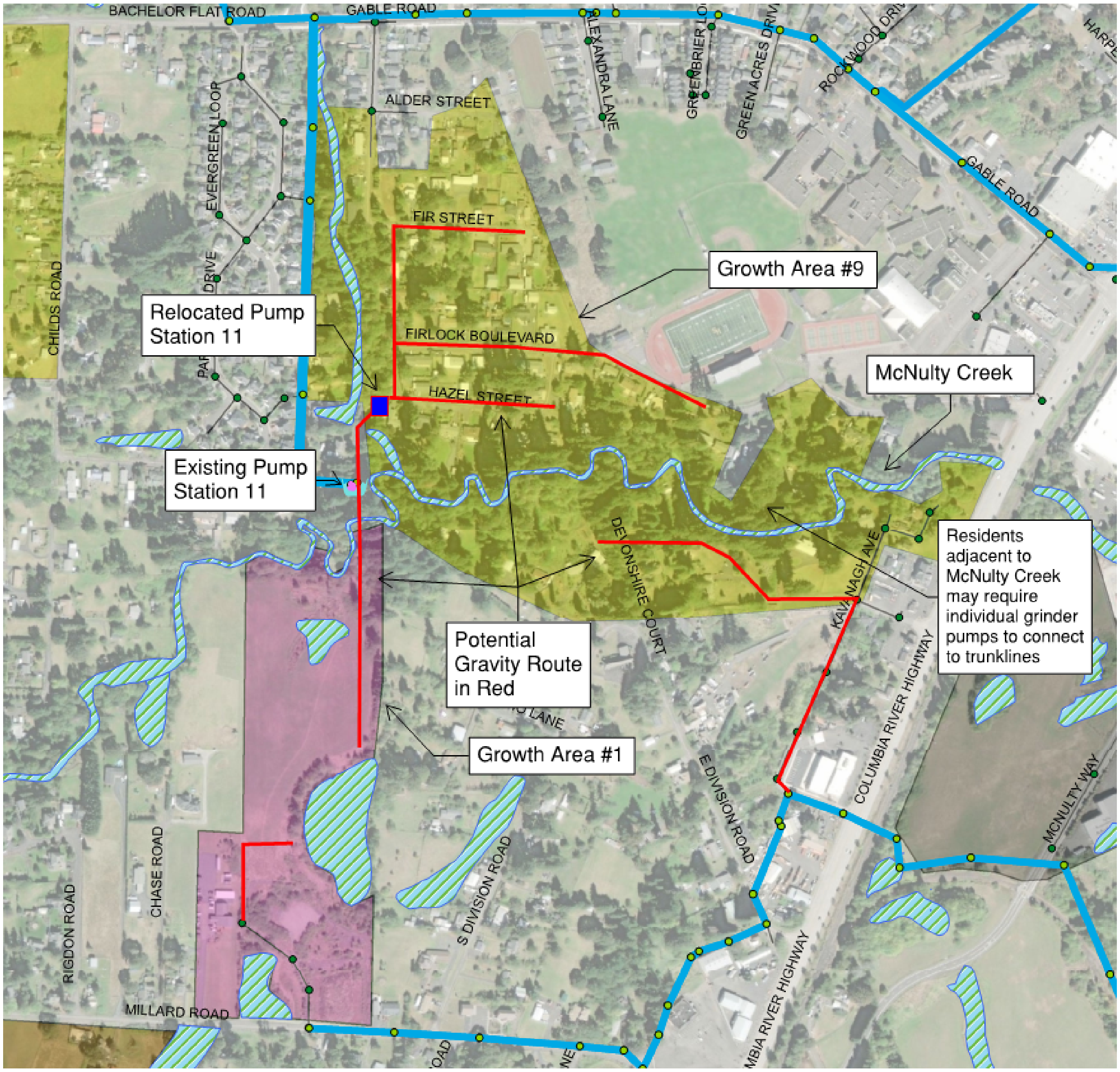
5.5.3 ADDITIONAL GROWTH AREA INFRASTRUCTURE

Within the 20-year period, the areas anticipated to take on residential, commercial, and industrial growth are documented in Figure 9 in Appendix A. Most of these areas have topography that allow for gravity flow into the existing collection system. There are some growth areas, however, that may require additional infrastructure. Growth Areas #1 and #9, highlighted in Figure 5-5, present challenging topography, primarily due to the wetlands in the area. Provided City GIS and topology information utilized in this study are accurate, it is feasible that southern portions of Growth Area #1, in pink, and of Growth Area #9, in yellow, can be served by 8-inch gravity lines from Basin 6 (upstream of PS#7). The northern portion of Growth Area #1 is anticipated to flow by gravity north to PS#11. This alignment assumes a boring under McNulty Creek.

The City has expressed interest in relocating PS#11 further north, to the intersection of Firlok Park Street and Hazel Street. If done, the depth of the wetwell can be sized at predesign to receive flow via gravity line from the northern portions of Growth Areas #1 and #9. Again, this would assume a bore under McNulty Creek to serve the portion of Growth Area #1. A potential layout for the pipelines is depicted in Figure 5-5. Grinder pumps may need to be installed at residences adjacent to McNulty Creek, as the relative elevation of these locations may make serving them via gravity pipeline not feasible.

The anticipated peak 20-year flows to PS#11 are approximately 550 gpm. This includes estimated flows from Growth Area #10, located to the northwest of the pump station, which is expected to flow by gravity to PS#11. PS#11 will require firm capacity improvements when it is relocated, in addition to increasing the depth of the wetwell. PS#7 is anticipated to need firm capacity improvements as additional growth areas develop in the basin. Cost estimates for the recommended infrastructure are summarized in Section 7.

FIGURE 5-5: GROWTH AREAS #1 AND #9 PROPOSED WASTEWATER INFRASTRUCTURE



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SECTION 6 - ENGINEERING STANDARDS & COMPREHENSIVE PLAN REVIEW

The City's existing development code (Title 17), engineering design standards (Title 18), and comprehensive plan (Title 19) were reviewed for new development as they pertain to wastewater conveyance to identify potential deficiencies and provide recommendations for updates.

6.1 ENGINEERING STANDARDS & COMPREHENSIVE PLAN REVIEW

The following documents were examined during this review effort.

- St. Helens Municipal Code (SHMC) Title 17 Community Development Code
- St. Helens Municipal Code (SHMC) Title 18 Engineering Standards Manual
- St. Helens Municipal Code (SHMC) Title 19 Comprehensive Plan

General observations and recommendations to update the City's policies and standards are summarized in the technical memorandum in Appendix C. The City should review the recommendations presented in the memo and assess if they agree with the proposed changes and additions to City Municipal Code, standards, and comprehensive plan. If the City agrees with some or all of the recommendations, the process to propose changes to the documents listed above should be initiated.

SECTION 7 - RECOMMENDED COLLECTION SYSTEM IMPROVEMENTS

This section consists of the recommended plan to address the wastewater collection system deficiencies. The recommended projects presented here have been incorporated into the St. Helens Capital Improvement Plan (CIP) in Section 8.

7.1 INFLUENT FLOW MONITORING IMPROVEMENTS

As discussed in Section 2, the current method of measuring wastewater influent flow may not reliably capture peak influent flows during high flow events, particularly when the headworks bypass is active. A Parshall flume, partially-full pipe electromagnetic flowmeter, and non-contact (above flow) sensor were considered for the application. Based on footprint, vertical drop available, and general capital costs, it is recommended that the City install a non-contact flow sensor in a new manhole along the 42" trunkline upstream of the City's headworks. One such sensor is the Hach Flo-Dar sensor that is mounted in a manhole just above the crown of the pipe and uses ultrasonic and radar technology to measure level, velocity, and calculate flow rate. The sensor could be connected to and recorded by the City's Supervisory Control and Data Acquisition (SCADA) system. Costs for the improvement are estimated below in Table 7-1, with additional details in Appendix J.

TABLE 7-1: PRIORITY 1 INFLUENT FLOW MONITORING IMPROVEMENTS

Project Name	Improvement Cost (rounded)
WWTP Influent Flowmeter	$68,000

7.2 RECOMMENDED PUMP STATION IMPROVEMENTS

Recommended pump station improvements summarized here address deficiencies summarized in Sections 3.2 and 4, including the relocation and improvements of PS#1 and PS#11. Costs presented in the following tables are planning level estimates and are in 2021 dollars. Actual costs may vary and should be refined further in the pre-design process. Engineering costs assume that multiple pump station projects will be grouped together for project administration efficiencies.

7.2.1 PRIORITY 1 – COMPLETE CURRENT AND URGENT UPGRADES

As stated in Section 3, the City is currently installing overflow alarms at each of its pump stations. This effort was undertaken as a proactive approach to anticipated DEQ guidance requiring installation of overflow alarms on new pump stations. As of this report, six stations have yet to receive the upgrade. Priority 1 pump station improvements address completion of this installation effort, including SCADA integration, and should be completed in the next six years. It is assumed that this effort for PS#1 and PS#11 will be completed with their Priority 2 upgrades, discussed in Section 7.2.2.

Additionally, it is recommended that the City add alarms on all pump stations that indicate when all pumps are running. The City should track when the alarm is triggered. If this alarm is frequent (more than once every 5 years), then it may indicate the pump station is running at or over its firm capacity and needs to be upgraded.

PS#2 is currently served by two pumps operating on VFDs. Both pumps operate with a high setting of 750 gpm and a low setting of 250 gpm. Currently, in the event of high inflow into the station, the station runs both pumps at low setting prior to switching one to the high setting. Generally, one pump switching to the high setting while the other pump continues to run indicates a lack of firm capacity. It is recommended the station be equipped with an alarm that indicates when one or both pumps switch into their high setting. The alarm should be integrated into SCADA, and a log should be kept of high setting incidents. Multiple alarms a year may be indication of a lack of firm capacity and a need for an upgrade.

Currently, during power outages, City staff alternates use of its portable generators at the multiple pump stations which lack on-site backup power. City staff have to prioritize which stations to supply emergency power to with the two available portable generators. It is recommended an on-site generator be installed at PS#3 to increase the City's backup power capabilities and simplify portable generator operations during outages.

It is assumed that adding firm capacity alarms for the pump stations incurs minimal cost to the City and can be completed in conjunction with installation of the overflow alarms. Improvement costs are summarized in Table 7-2. Cost estimate details can be found in Appendix J.

TABLE 7-2: PRIORITY 1 PUMP STATION IMPROVEMENTS

Project Name	Improvement Cost (rounded)
Install Overflow Alarms	$9,000
Install On-site Generator at Pump Station 3	$90,000
Total Project Costs (rounded)	***$100,000***

7.2.2 PRIORITY 2 – ADDRESS NOTED DEFICIENCIES AND ANTICIPATED GROWTH

Table 7-3 (at end of section) summarizes recommended Priority 2 improvements by pump station. These projects are identified as Priority 2 projects as they are not urgent to address significant deficiencies, but are recommended to address anticipated growth, as well as redundancy, safety, and O&M concerns reported in Sections 3 and 4. Relocation of both PS#1 and PS#11 accommodate anticipated future growth. General, minor improvements to remaining stations address redundancy, safety, and O&M concerns. The recommended pump station improvements include:

PS#1 Relocation

As discussed in Section 5.5.1, the City is currently evaluating development options for the Riverfront District. The proposed infrastructure includes relocation of PS#1, a new force main, and a new 10-inch gravity trunkline. The proposed infrastructure is shown in Figure 21 in Appendix A. The proposed alignment of the new, 10-inch gravity pipeline would start at the existing PS#1 location, run along the S 1st Street extension, and discharge at the anticipated new pump station location adjacent to the S 1st Street/Plymouth Street extension. The new pump station is recommended to include an on-site backup generator and have a firm capacity of approximately 700 gpm to accommodate the anticipated 20-year flows from the existing sewer basin and the Riverfront development.

Due to this project's proximity to the Columbia River, this project may encounter a high water table in the Riverfront development area. An estimate for dewatering groundwater has been included in the planning level costs. It was assumed that construction of the new roadway within the Riverfront development was not a part of this project.

PS#11 Relocation

As described in Section 5.5.3, PS#11 is proposed to be relocated north to serve homes in the Firlock area basin. Improvements are recommended to increase the firm capacity to approximately 550 gpm, including a new 6-inch force main, to handle anticipated peak flows in the 20-year planning period.

City staff also noted pump station safety and access concerns with the current pump station location in the middle of a bend in the road that does not have a wide shoulder or permanent lighting. City staff are currently using headlights and flashlights if servicing the station in the dark. Relocating and upgrading the pump station would address the access and safety concerns for this station while also providing the option to serve additional growth areas.

The proposed location of the new PS#11 is on the east side of McNulty Creek. The connection of the new pump station to the existing collection system (located on the west side of McNulty Creek) will require crossing over or under a McNulty Creek culvert. Open trench construction may disturb the existing culvert, which in turn may prompt environmental investigations into fish passage, additional permitting efforts, and additional construction costs. As such, it is recommended that a trenchless bore be utilized around the existing culvert for the pipeline extensions to minimize impact to the culvert. Due to the prevalence of bedrock in St. Helens, which may interfere with boring progress, a 40% contingency was assumed for this project.

General Pump Station Improvements

Additionally, safety, redundancy, capacity, and operations concerns at the remaining pump stations are recommended to be resolved via the following improvements:

- Based on the hydraulic evaluation and pump runtime analysis (Section 4.1.5), PIF_5 flows into PS#1, PS#2, PS#3, PS#4, PS#5, PS#7, and PS#11 may exceed the stations' firm capacities. It is recommended that pump station runtimes continue to be recorded and reviewed by staff in conjunction with the recommended alarm data if both pumps are running. If the runtimes depict a station running both pumps, and I/I improvements do not reduce flows into the pump stations, then the station firm capacity should be increased to handle peak influent flows. PS#5 had multiple instances of exceeding firm capacity. It is recommended that this station have its pumps upgraded to handle peak influent flows. PS#2 has a VFD and operates on both a high and low setting. When the station experiences near 60 minutes running on the hour in its high setting, it is a likely indicator that it' exceeding firm capacity and requires upgrades. It should be noted that I/I reduction efforts described in section 7.3 could delay or eliminate the need for this improvement.
- It is recommended to install pressure gauges and flow monitors at each pump station when they are undergoing upgrades or pump replacements. This allows City staff to record information on pump and influent conditions and assess pump station capacity in real time.
- It is recommended that each pump station currently lacking adequate fall protection be equipped with adequate fall protection. This applies to PS#2, PS#3, PS#4, PS#5, and PS#8. Additionally, it is recommended that each pump station without redundant level sensors be equipped with a redundant level monitoring device, such as an ultrasonic level sensor or backup floats.

Cost estimates for each of the Priority 2 Pump Station improvements are shown in Table 7-3. Cost details can be found in Appendix J.

TABLE 7-3: PRIORITY 2 PUMP STATION IMPROVEMENTS

Project Name	Improvement Cost (rounded)
Pump Station 1 Relocation	$2,400,000
Pump Station 11 Relocation	$3,100,000
Pump Stations 2 - 9 Upgrades	$700,000
Total Project Costs (rounded)	***$6,200,000***

7.2.3 PRIORITY 3 – ACCOMMODATE ADDITIONAL GROWTH

The Priority 3 recommended improvement accommodates anticipated growth. As described in Section 4, PS#7 is undersized for anticipated, 20-year growth. Two industrial areas, a mobile home park, and a portion of mixed-use residential growth are anticipated to develop in the PS#7 basin. It is recommended the pump station firm capacity be increased to approximately 1,400 gpm to accommodate the growth. There is an existing 8-inch force main at the pump station that is currently inactive. It is anticipated that PS#7 will utilize both the existing 6-inch and 8-inch parallel force mains when the firm capacity is increased. The PS#7 improvements are estimated to cost $2,200,000. Cost details can be found in Appendix J.

7.3 RECOMMENDED I/I IMPROVEMENTS

7.3.1 PRIORITY 1 – REDUCE I/I TO REDUCE RISK OF OVERFLOW/SURCHARGING

I/I Reduction

As discussed in Section 3, the City of St. Helens experiences large amounts of I/I. Estimated peak flows in the collection system are 20-25 times higher than annual dry weather flows. The collection system requires significantly increased capacities to handle these peak wet weather flows. They cause much of the surcharging and reported overflows in the collection system. In addition to the surcharging and reported overflows within the collection system, the peak I/I flows also put strain on the City's pump stations and WWTP. While not considered reliable for recording peak flows, the existing WWTP influent flowmeter has recorded peak flows in excess of 25 MGD. An evaluation of the WWTP was not included in the scope of this study. However, in discussion with City staff, the WWTP influent bypass channel is typically used multiple times a year during the wet weather season. It is recommended the City track peak influent flows at the WWTP and assess if they exceed the rated capacity of WWTP unit processes. If I/I in the system is not addressed, the City may need WWTP upgrades to handle peak flows. I/I reduction throughout the system could delay or eliminate the need for many capacity-related improvements throughout the wastewater collection system and WWTP and provide cost savings to the City.

Using the methodology described in Section 3, priority pipelines for inspection and I/I improvements were identified and are displayed in Figure 15 of Appendix A. It is recommended that the City utilize Figure 15 and the table in Appendix K, which highlight the recommended pipelines to begin I/I efforts. Projects that had been replaced or rehabilitated recently were not included in these I/I recommendations. It should be noted that because recent CCTV data was unavailable, specific improvement recommendations for each pipe are not included in this report. Instead, it is recommended that the City utilize this figure and table to inform initial CCTV inspection efforts. Inspection reports can be utilized to identify specific defects in pipelines and manholes to help inform the least intrusive and most cost-effective improvement to rectify defects. Improvements can include pipeline and manhole replacement, slip-lining of existing pipelines, or spot repairs. The City has reported significant I/I issues in defective manholes, and improvements should take special consideration to address manhole as well as pipeline defects. I/I improvements can also include repair and/or replacement of service laterals along the improvement corridor.

It is recommended that the City create an annual budget to fund I/I improvement projects throughout the City. The City currently has an adopted annual replacement budget of $200,000 per year. Rather than have a separate replacement budget and I/I improvement budget, it is recommended the City adopt a combined fund of $500,000 annually. This dollar amount is reflective of the estimated annual pipeline replacement cost discussed in Section 7.8. This annual I/I reduction program would allow City staff to proactively identify and address deficiencies throughout the collection system. The recommended work is anticipated to be a combination of sump pump identification and removal, lateral replacement program, as well as mainline and manhole inspections and rehabilitation/replacement. I/I reductions could delay or eliminate the need for capacity-related pipeline upsizing projects discussed later in the section and provide cost savings to the City over the planning period.

Sump Pump Disconnection

The alternatives for addressing sump pump cross-connections to the wastewater system were presented in Section 5. Based on City staff input, it is recommended the City pursue a combination of Alternatives SP1 (Educational Material), SP5 (Point-of-Sale Inspection), and SP6 (Reward-

Based Disconnect Incentives) as presented in Section 5 of this report. The combination of these alternatives will make up the City's initial Sump Pump Disconnection Program.

A portion of the recommended I/I annual budget should be reserved for the Sump Pump Disconnection Program. The incentive portion of the Disconnection Program may include a direct monetary reward or a billing credit for those who have proven their sump pump has been disconnected.

Concerning the point-of-sale inspection, it is recommended that the City update its code to include language requiring the seller to evaluate and disconnect any sump pumps from the sanitary sewer during inspection and before the property transfers ownership.

7.4 RECOMMENDED CONVEYANCE IMPROVEMENTS

This section summarizes the recommended pipeline improvements to address deficiencies identified in Section 4. All existing system deficiencies are present, with some issues exacerbated, in the 20-year scenario. The improvements presented alleviate potential wastewater overflow and surcharging through the 20-year planning period. Pipeline improvements are sized based on the planning criteria to achieve a d/D of less than 0.85 for the projected 20-year peak flows. All pipelines that are replaced, at a minimum, match the upstream pipeline size and do not exceed the size of the downstream pipeline unless otherwise noted in the descriptions below. This is considered an industry good practice. The pipeline replacements also described below assume open cut construction unless otherwise stated. Alternatively, the City could utilize trenchless rehabilitation technologies such as pipe bursting, cured-in-place-pipe installation, or slip lining. The City has described having success with pipe bursting in projects in the past under certain conditions. The City has also reported having success with horizontal directional drilling (HDD) when installing deeper pipes in the solid basalt rock. These trenchless approaches can be less costly than the open cut construction approach. Evaluation of the appropriate installation method should be completed as a part of the concept or pre-design phase of pipeline replacement projects.

Improvements are organized by priority and are shown in Figure 23 in Appendix A. More detailed planning level cost estimates for recommended improvements can be found in Appendix J.

7.4.1 PRIORITY 1 – ELIMINATE KNOWN OVERFLOWS AND SURCHARGING

The improvements assigned to Priority 1 have been marked as areas of concern by the City and have been reported to have overflows or significant surcharging during wet weather events, which is confirmed by the model. The pre-design and design phases of these projects should be performed in conjunction with Priority 1 I/I improvement projects to assess need and appropriate pipeline sizing for each project as I/I reductions are achieved. It should be noted that if I/I projects significantly reduce peak wet weather flows, the need for these conveyance projects could be reduced, delayed, or eliminated. Costs for these improvements can be found in Table 7-4 (at the end of this section).

<u>Basin 4 Pipeline Upsize and Reroute</u>

It is recommended that the pipeline in Basin 4 west of S 13th Street be upsized to a 12-inch pipeline, and then construct a 15-inch trunkline that reroutes flow from S 13th Street (Manhole M13), along Tualatin and S 7th Street, and to the existing Basin 6 interceptor south of Plymouth Street (Manhole S1). Basin 4 is considered the highest priority of the Priority 3 projects, as this basin contains the largest concentration of potential overflow locations and contributes to the surcharging in Basin 5. By rerouting flow away from Basin 5, the Basin 5 trunkline may experience reduced surcharging. As such, it is recommended that this improvement be constructed prior to the Basin 5 pipeline upsize project.

Basin 5 Pipeline Upsize

The City has reported significant surcharging and overflows in the main trunkline through Basin 5 along S 4th Street. As noted above, the Basin 4 improvements will reduce flows going to Basin 5. In addition, Basin 5 has been reported to have some of the highest I/I in the system. The annual I/I reduction projects could have significant impacts to the peak flows in Basin 5. It is recommended that flow monitoring be included in the concept design phase of this project to evaluate the peak flows in Basin 5 following I/I reduction work and Basin 4 improvements. The model evaluation of Basin 5 improvements, including Basin 4 improvements and assuming no I/I flow reductions, indicates that the trunkline north of the Basin 4 interceptor should be upsized to a 36-inch pipe and the remainder of the trunkline be upsized to a 42-inch pipe.

7.4.2 PRIORITY 2 – NO RECOMMENDATIONS

No conveyance improvements were placed in Priority 2. More immediate concerns for surcharging and overflows are Priority 1. Improvements where City staff have not seen historical flooding or where risk of overflows is lower are included in Priority 3. Consistent I/I mitigation projects could reduce, delay, or eliminate the need for some conveyance improvements. Refer to Section 7.4.1 and 7.4.3 for additional details on conveyance improvement projects.

7.4.3 PRIORITY 3 – REDUCE RISK OF OVERFLOW AND SURCHARGING

The improvements assigned to Priority 3 include areas where the City has reported infrequent or no observations of historical overflows or surcharging, but the hydraulic modeling evaluation identified as areas with capacity limitations within the 20-year planning period. Annual I/I reductions could reduce peak flows in each area resulting in reduction, delay, or elimination of improvements required for capacity limitations. Predesign phases should include updating the design flows and documenting observed I/I reductions. It is generally recommended that downstream improvements occur before upstream improvements within a sewer basin. Upstream improvements can increase peak flows to downstream infrastructure. Downstream impacts should be evaluated for all projects during the pre-design phase. The improvements have been separated by flowmeter basin and arranged based on risk considerations and recommended construction sequence. Costs for the improvements are estimated below in Table 7-4 and in Appendix J.

Basin 6 Pipeline Upsize and Reroute

In the model, Basin 6 is shown to have several potential overflow locations, and the majority of its trunklines along Port Avenue, S 18th Street, Dubois Lane, Kaster Road, and Old Portland Road are shown to be undersized and surcharged during peak flows.

It is recommended that the trunkline along Port Avenue be upsized to a 27-inch pipe, and the pipeline along the Columbia River Highway, Dubois Lane, and S 18th Street be upsized to an 15-inch trunkline. Additionally, a new 15-inch pipe should be constructed that conveys flow from Manhole D1 on S 18th Street to Manhole S20 on Kaster Road, and the connecting pipe from Manhole D1 to Manhole S17 on Old Portland Road should be abandoned. It should be noted that the existing trunkline recommended for upsizing along the Columbia River Highway is believed to cross under Milton Creek. Should this pipeline be scheduled for upsizing, a trenchless technology such as pipe bursting or boring is recommended for the segments beneath the Columbia River Highway. The trenchless technology will also minimize work within the highway right-of-way.

The southern trunkline parallel to Old Portland Road is recommended to be upsized to 30-, 33-, and 36-inch pipeline from Kaster Road east to just past the end of Umatilla Street, upstream of parallel pipes over the lagoon. This pipeline upsize is recommended to accommodate anticipated growth in the 20-year planning period, including significant industrial growth in the southern portion of the City.

The City has not reported observations of historical overflows within the pipelines in Basin 6. A master plan update is anticipated prior to Priority 3 projects being completed and would update planning flow criteria and reassess extents of improvements needed.

Basin 2 Pipeline Upsize and Reroute

Basin 2 is shown by the model to have several potential overflow locations and surcharging along Gable Road, Westshire Lane, Matzen Street, and Sykes Road. As mentioned previously, pre-design phase should include evaluation of potential downstream trunkline impacts to mitigate increasing surcharging or potential overflows in the system. It is recommended that the trunkline along Sykes Road from Matzen Street to Columbia River Highway be upsized to an 18-inch pipeline. The Sykes Road trunkline from Matzen Street to Westshire Lane be upsized to a 15-inch pipeline with a 12-inch connection to the Westshire Lane pipeline. The existing pipelines along Westshire Lane, Archer Drive, and Whitetail Avenue should be upsized to 12-inch pipelines. It is recommended that the Matzen Street trunkline be upsized to a 15-inch from Sykes Road to Campbell Park, and the remainder of the trunkline to the north should be upsized to a 12-inch pipeline.

It is recommended the existing pipeline within Gable Road, upstream of manhole WC9 (located south of Rockwood Drive intersection), be upsized to a 12-inch pipeline. A new 12-inch pipeline should be constructed to reroute flow from manhole WC9 to Manhole W42 at the intersection of Sykes Road and Cedaroak Street.

Basin 1 Pipeline Upsize

Basin 1 has modeled surcharging and potential overflow locations. The City has not observed capacity issues along this line and a new development is being constructed along a portion of the trunkline. Based on the hydraulic evaluation, it would be recommended that the existing trunkline that branches from the north of Manhole N30 (located north of Kelly Street) be upsized to a 15-inch pipeline, and the pipe segment between Manhole N30 and Kelley Street be upsized to an 18-inch pipeline. A master plan update, or concept design phase, is anticipated to occur prior to Priority 3 improvements and would update planning flow criteria and reassess extents of improvements needed at the time the project moves forward.

Basin 3 Pipeline Upsize

The hydraulic evaluation shows Basin 3 with the lowest amount of surcharging. The trunkline along N 10th Street and West Street experiences surcharging. The City has not observed capacity issues along this line, but based on the hydraulic evaluation, it would be recommended this trunkline be upsized to a 15-inch pipeline to address the deficiency identified. A master plan update, or concept design phase, is anticipated to occur prior to Priority 3 improvements and would update planning flow criteria and reassess extents of improvements needed at the time the project moves forward.

TABLE 7-4: RECOMMENDED CONVEYANCE IMPROVEMENTS

Project Name	Improvement Cost (rounded)
Priority 1 Improvments	
Basin 4 Pipeline Upsize and Reroute	$3,600,000
Basin 5 Pipeline Upsize	$4,500,000
Total Priority 1 Costs (rounded)	***$8,100,000***
Priority 3 Improvments	
Basin 6 Pipeline Upsize and Reroute	$6,300,000
Basin 2 Pipeline Upsize and Reroute	$9,400,000
Southern Trunkline Upsize	$3,900,000
Basin 1 Pipeline Upsize	$1,800,000
Basin 3 Pipeline Upsize	$1,200,000
Total Priority 3 Costs (rounded)	***$22,600,000***

It should be noted that these cost estimates include rock excavation contingencies for pipelines being upsized. Due to the unknown field condition of the existing trenches, it was assumed that the trench directly encompassing the existing pipeline would need to be re-excavated to accommodate the upsized pipe. Additionally, when re-constructing roads through existing intersections with sidewalks and pedestrian crossings, the Oregon Department of Transportation (ODOT) and federal law require that ramps be reconstructed to be compliant with the American Disabilities Act (ADA) requirements. The above cost estimates in Table 7-4 account for reconstruction of crosswalk ramps at intersections with existing sidewalk.

7.5 FUTURE SYSTEM IMPROVEMENTS

7.5.1 PRIORITY 2 – PROVIDE WASTEWATER INFRASTRUCTURE FOR PLANNED NEW DEVELOPMENT

As discussed in Section 5.5, the City of St. Helens owns two primary properties and have completed significant planning efforts for potential developments on both. The two properties are the Riverfront District and the Industrial Business Park. Locations and summaries for these developments can be found in Section 5.5. Wastewater loading for these developments was established in Section 2 of this report and can be found in Appendix B. Pipeline improvements are sized based on the planning criteria established in Section 2. This section summarizes the proposed wastewater infrastructure to serve both development properties.

Riverfront District

Proposed infrastructure to serve the Riverfront District is described in Section 7.2.2 as part of relocating PS#1. Figure 21 in Appendix A shows the proposed infrastructure, which includes the relocated pump station, a new force main, and a new 10-inch gravity main. Costs for this infrastructure are shown in Table 7-5 below and detailed in Appendix J.

Industrial Business Park

As discussed in Section 5.5.2 of this report, the City is seeking new opportunities to develop its industrial business park and requires wastewater infrastructure to serve the development. A series of 8- to 15-inch diameter gravity trunklines, a pump station with a firm capacity of approximately 1,300 gpm, and a 10-inch force main are proposed to serve the development. The proposed layout for the gravity lines, pump station, and force main are shown in Figure 22 in Appendix A. It is recommended that two segments on the downstream trunkline near the WWTP be upsized to 36-

inch pipeline as part of the improvements to accommodate the additional flows from the Industrial Business Park. Costs for the proposed wastewater infrastructure are shown in Table 7-5 and detailed in Appendix J.

TABLE 7-5: FUTURE DEVELOPMENT PROPOSED INFRASTRUCTURE

Project Name	Improvement Cost (rounded)
Riverfront District Trunkline and Pump Station 1 Relocation	$2,400,000
Industrial Business Park Trunklines and Pump Station	$13,200,000
Total Project Costs (rounded)	***$15,600,000***

7.6 PLANNING RECOMMENDATIONS

It is recommended that the City update their planning documents every five (5) years. Updates to the planning documents and models allow the City to re-assess needs and properly allocate budgets to address system deficiencies. The next update should include an evaluation of both the wastewater collection system and WWTP. The previous plan for both systems was completed in 1989, and as a result, a Master Plan Update for both the wastewater collection system and the treatment plant has been included in the CIP as a Priority 2 improvement, with an estimated cost of $300,000.

7.7 MAPS

Maps of the existing collection system are provided in Figures 10 and 11 of Appendix A. The recommended I/I improvement locations are shown in Figure 15 in Appendix A. The recommended capital improvements are shown in Figure 23 in Appendix A.

7.8 ENVIRONMENTAL IMPACTS

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

7.8.1 LAND USE / PRIME FARMLAND / FORMALLY CLASSIFIED LANDS

No area within the City limits is classified as prime farmland. All recommended improvements occur within previously disturbed or developed land.

7.8.2 FLOODPLAINS

As shown in Figure 3 in Appendix A, a few portions of the study area (including the wastewater treatment plant) are located inside the 100- and 500-year floodplains of the Columbia River, McNulty Creek, and Milton Creek. None of the alternatives would create new obstructions to these floodplains. Construction that occurs within the 100-year floodplain will require permitting and safeguards against potential flood hazards.

7.8.3 WETLANDS

Improvements to PS#5, PS#8 and PS#11 occur adjacent to wetlands. PS#11 is located adjacent to Wetland MC-9 (from LWI) and McNulty Creek. MC-9 is a type 1 significant wetland to St. Helens and includes a 75' upland protection zone. McNulty Creek is a locally significant riparian area, with a 50' upland protection zone. PS#11 should be relocated to a location outside of the upland protection zones of MC-9 and McNulty Creek. PS#11 relocation is anticipated to cross under a connecting culvert of McNulty Creek. Special precautions should be taken not to disturb McNulty Creek, wetland MC-9, or the creek culvert during construction. As stated in Section 7.2, disturbing culverts with active or historic fish populations may trigger additional environmental permitting and construction constraints. It is recommended that boring or another trenchless method be evaluated during concept or pre-design for pipeline installation across the McNulty Creek. PS#8 is near Milton Creek, also a locally significant riparian area, with a 50' upland protection zone. Upgrades to PS#5 and PS#8 are not expected to impact the adjacent wetlands, streams, or upland protection zones.

Additionally, the upsizing projects in Basin 6 may cross by existing Milton Creek culverts beneath the Columbia River Highway. Similar to the PS#11 improvement, trenchless technology such as pipe bursting is recommended for these sections to avoid disturbing existing culverts.

7.8.4 CULTURAL RESOURCES

None of the recommended improvements are anticipated to impact the above-ground cultural resources identified by the National Register of Historic Places or Ordinance No. 3250 (local historic landmarks). The relocation of PS#1 would involve the abandonment of the existing pump station, which is within the Historic Downtown District. However, the abandonment and construction of the new pump station and gravity pipeline is not anticipated to affect any of the listed historic landmarks or existing structures within the Historic Downtown District.

7.8.5 BIOLOGICAL RESOURCES

For a summary of threatened or endangered plants in the planning area, please see Appendix B. It is important to note that the likelihood of any of these plants existing on the proposed project sites is low because the areas have been previously disturbed, paved, or landscaped.

It is not anticipated that the improvement projects will impact creeks or wetlands where ODFW-listed aquatic species may reside and it is advised that trenchless technology be utilized for pipe installation or upsizing when in proximity to wetlands so impacts to aquatic species or habitat are limited.

7.8.6 WATER RESOURCES

Modifications to the collection system would reduce the risk of overflows and potential to spill into waterways. Design for the PS#11 relocation and force main extension could include boring under the McNulty Creek culvert to minimize impacts. It is recommended that sections of the pipeline upsizing projects on the Columbia River Highway (Basin 6 improvements) be bored, or pipe burst so that impacts to Milton Creek are minimized. There are no other alternatives that involve stream crossings.

7.8.7 SOCIO-ECONOMIC CONDITIONS

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

7.9 LAND REQUIREMENTS

The pipeline rerouting improvements for Basin 2 may require easements through the Avamere parking lot.

7.10 POTENTIAL CONSTRUCTION PROBLEMS

The depth of the water table and rock may affect construction of the improvements. The majority of the city has shallow bedrock that will increase the level of effort and cost of conveyance upgrades. The planning level costs have assumed that new construction will encounter bedrock within three (3) feet of the surface, and that upsizing existing pipelines may require more rock excavation than anticipated due to variable or unknown field conditions of the existing trenches. To provide contingency, it was assumed that the trench volume around the length of upsized pipe will need to be re-excavated. Each project should evaluate the potential use of trenchless technology for construction purposes and cost savings during the predesign and design phases.

Additionally, a portion of the gravity pipelines and the force main for the PS#1 relocation may encounter shallow groundwater. In this case, provisions for dewatering should be anticipated prior to construction. Gravels and sands combined with high groundwater may require extensive dewatering. However, subsurface investigations to better understand these impacts were not within the scope of this planning study.

Construction plans for any of the alternatives would also include provisions to control dust, erosion and sediment, and runoff.

7.11 SUSTAINABILITY CONSIDERATIONS

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

7.11.1 WATER AND ENERGY EFFICIENCY

Installation of an influent flow monitor may minimally increase energy usage at the WWTP. The recommended increase in capacity of PS#1and PS#11 may increase energy use. Alternatively, the incorporation of VFD pumps at the stations may lead to more efficient energy usage when pumping wastewater. The general improvements for the remaining pump stations may minimally increase energy usage to monitor flow, pressure, and level sensors.

Reducing I/I in the collection system would have the largest impact and would result in a decrease in water and energy usage at the pump stations and the WWTP due to an overall reduction in flow needing to be conveyed and treated.

7.11.2 GREEN INFRASTRUCTURE

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

7.12 OPERATION AND MAINTENANCE RECOMMENDATIONS

7.12.1 MAINTENANCE PROGRAM AND STAFFING

The recommended level of service (LOS), O&M, and staffing for the wastewater collection system is summarized in Section 3. As discussed in Section 3, it is estimated that approximately 3.5-4.0 Full Time Equivalent (FTE) are needed to meet the recommended level of O&M to meet the City's LOS goals. As budgeted, the existing wastewater collections FTE staff appears to be adequate, however, the additional projects and work the PW Operations staff are currently requested to complete significantly decreases the budgeted FTE that can be spent on wastewater collections O&M. It is recommended that either additional FTE be budgeted for the PW Operations staff to complete the existing workload requested, or the responsibilities of the PW Operations staff be reduced to focus solely on utility O&M. In addition, the recommended CIP projects would increase workload of the engineering division. The engineering division may need additional staff to manage any sump pump identification and removal program, update and maintain the GIS database, coordinate CCTV inspection and resulting work orders, and manage capital improvements. Additional workload on the engineering and PW operations divisions should be included in planning for any of the recommended improvements and projects. Generally, it is recommended that staffing needs be reevaluated every two to three years.

7.12.2 PIPELINE REPLACEMENT PROGRAM

In addition to regular maintenance, it is recommended that an annual pipeline replacement program be established. As degrading pipe sections and I/I problems are identified through CCTV monitoring and flow monitoring, these areas should be corrected. Pipeline and manhole replacement and rehabilitation needs are likely to increase as the sanitary sewer collection system ages.

Typically, it is recommended to budget for replacing 1/75th of system pipelines annually, assuming average useful life of pipelines is 75 years. For St. Helens, this would lead to a recommendation of the City budgeting for replacement/rehabilitation of an average of 4,200 feet of the collection pipeline system each year. Average useful life of manholes and cleanouts are shown in Table 7-6 below.

As mentioned in Section 7.3, it is recommended that the City budget an annual $500,000 dollars for I/I related replacements, rehabilitation, and sump pump efforts. It is recommended that this amount serve as a combined I/I improvement budget and annual replacement budget. It should be noted that this is an interim amount presented for City budgeting purposes, with the purpose of increasing over time to the recommended $790,000 annual replacement budget for the system. After I/I improvements have sufficiently reduced peak flows to the City's satisfaction, it is recommended the following annual replacement budget be adopted to keep the City's system free of defects.

A reference for the costs associated with funding an on-going replacement and rehabilitation program are summarized in Table 7-6.

TABLE 7-6: REPLACEMENT BUDGETS

Item	Lifespan	Cost/Year
Pipelines	75 Years	$ 570,000
Manholes	50 Years	$ 210,000
Cleanouts	50 Years	$ 5,000
Total (rounded)		***$ 790,000***

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.

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 and other utilities to maximize cost sharing between utilities.

SECTION 8 - CAPITAL IMPROVEMENT PLAN (CIP)

This section outlines the recommended plan to address the wastewater collection system deficiencies identified in previous sections. The alternative evaluation conducted in Section 5 and recommended projects summarized in Section 7 with input from City staff are the basis for the capital improvement plan (CIP) for the wastewater collection system presented in this section.

8.1 BASIS FOR ESTIMATE OF PROBABLE COST

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

The costs are based on experience with similar recent collection system improvement projects. Equipment pricing from manufactures of the flow measuring equipment items was also used to develop the estimates. The total estimated probable project costs include contractor markups and 30% contingencies, which is typical of a planning-level estimate. Overall project costs include total construction costs, costs for engineering design, permitting, construction management services, inspection, as well as administrative costs. For the collection system projects, the contractor's overhead and profit are worked into the line items.

8.2 SUMMARY OF COSTS (20-YEAR CIP)

The cost summary of the 20-year CIP projects is listed in Table 8-1. The system development charge (SDC) eligibility for each project was factored using the expected growth of the existing peak flow to the projected 2040 peak flow. The amount of capacity that can be utilized for future connections up to the projected 20-year planning period is used as the percentage for SDC eligibility. Priority 1 projects are the short-term projects to be completed in the next six years. Costs shown are planning-level estimates and can vary depending on market conditions. These costs should be updated as the project is further refined in the pre-design and design phases. Individual project sheets for Priority 1 projects are included in Appendix J. Each project sheet consists of a project objective, description, location map, and cost estimate.

The primary driver/s for each CIP project is identifed in the third column of Table 8-1. Priorities are set based on modeling performed as part of this facilities planning study and discussions with City staff. Priority 1 collection system improvements address reducing collection system I/I, WWTP influent flow metering, suspected overflows, and more immediate needs of the existing pump stations. Priority 2 collection system projects address identified deficiencies at pump stations or involve the relocation of existing pump stations. Priority 3 collection system projects address surcharging and potential overflows if peak flows are not reduced by Priority 1 or 2 projects.

TABLE 8-1: SUMMARY OF COSTS (20-YEAR CIP)

Project No.	Project Name	Primary Purpose	Total Estimated Cost (2021)	SDC Growth Apportionment		City's Estimated Portion
				%	Cost	
Priority 1 Improvements						
1.1	WWTP Influent Flow Meter	Operations	$ 68,000	10%	$ 7,000	$ 61,000
1.2	Basin 4 Pipeline Upsize and Reroute	Capacity	$ 3,600,000	0%	$ -	$ 3,600,000
1.3	Basin 5 Pipeline Upsize	Capacity	$ 4,500,000	3%	$ 150,000	$ 4,350,000
1.4	Install Overflow Alarms	Operations	$ 9,000	20%	$ 2,000	$ 7,000
1.5	Pump Station 3 On-site Generator	Operations	$ 90,000	0%	$ -	$ 90,000
1.6	Annual I/I Reduction Program (6-Year)	Capacity	$ 3,000,000	20%	$ 590,000	$ 2,410,000
		Total Priority 1 Improvement Cost (rounded)	**$ 11,300,000**			**$ 10,500,000**
Priority 2 Improvements						
2.1	Riverfront District Trunkline and Pump Station 1 Relocation	Capacity, Operations	$ 2,400,000	18%	$ 440,000	$ 1,960,000
2.2	Relocate Pump Station 11	Capacity, Operations	$ 3,100,000	68%	$ 2,110,000	$ 990,000
2.3	Industrial Business Park Trunklines and Pump Station	Capacity, Operations	$ 13,200,000	100%	$ 13,200,000	$ -
2.4	Pump Station Upgrades	Operations, Safety	$ 700,000	20%	$ 140,000	$ 560,000
2.5	Master Plan Update	Operations	$ 300,000	100%	$ 300,000	$ -
2.6	Annual I/I Reduction Program (8-Year)	Capacity	$ 4,000,000	20%	$ 790,000	$ 3,210,000
		Total Priority 2 Improvement Cost (rounded)	**$ 23,700,000**			**$ 6,700,000**
Priority 3 Improvements						
3.1	Basin 6 Pipeline Upsize and Reroute	Capacity	$ 6,300,000	7%	$ 460,000	$ 5,840,000
3.2	Basin 2 Pipeline Upsize and Reroute	Capacity	$ 9,400,000	12%	$ 1,140,000	$ 8,260,000
3.3	Southern Trunkline Upsize	Capacity	$ 3,900,000	26%	$ 1,010,000	$ 2,890,000
3.4	Pump Station 7 Upgrades	Capacity	$ 2,200,000	65%	$ 1,430,000	$ 770,000
3.5	Basin 1 Pipeline Upsize	Capacity	$ 1,800,000	9%	$ 150,000	$ 1,650,000
3.6	Basin 3 Pipeline Upsize	Capacity	$ 1,200,000	3%	$ 40,000	$ 1,160,000
3.7	Annual I/I Reduction Program (6-year)	Capacity	$ 3,000,000	20%	$ 590,000	$ 2,410,000
		Total Priority 3 Improvement Cost (rounded)	**$ 27,900,000**			**$ 23,000,000**
		Total Collection System Improvement Costs (rounded)	**$ 62,900,000**			**$ 40,200,000**

Note: The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2021 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.

8.3 OTHER ANNUAL COSTS

In addition to the capital improvement costs presented in Table 8-1, the following expected annual operating costs are recommended for consideration in setting annual budgets for the collection system:

Additional collection system replacement/rehabilitation needs: Based on linear feet of pipeline, and number of manholes and cleanouts, the City should set a goal to budget a total of $790,000/year for pipeline replacement/rehabilitation (to be either contracted out or completed using City crews). I/I replacement and rehabilitation projects performed as part of the Annual I/I Reduction Program may offset a portion or majority of these recommended costs, as pipeline rehabilitation addresses defects and extends pipeline lifespan. For budgeting purposes, $500,000/year has been recommended as an interim amount. It is recommended this amount increase over time to reach the replacement budget goal of $790,000/year.

The City should target the infiltration and inflow (I/I) projects discussed in Section 5 as a part of the annual pipeline replacement/rehabilitation budget. Prioritizing these projects should help to reduce I/I flows into the system and potentially delay capital improvements triggered by increased system flows.

Collection system cleaning and CCTV needs: It is recommended that the City maintenance staff develop a program to clean the entire collection system every three years, and CCTV the entire collection system every six years. Annual O&M costs for the collection system may increase slightly if Priority 3 improvements are made, as they increase the total linear feet of pipeline in the system.

Overall, if peak inflows from I/I are left unaddressed, the projected increase in influent flows and loadings will increase the total O&M of the system. However, should the Annual I/I Reduction program decrease peak flows, the O&M required to keep the pump stations and WWTP equipment in good working condition is anticipated to decrease by these improvements.

Staffing needs: As recommended in Section 7, the PW Operations division budgeted FTE should be increased or the responsibilities of the division outside of utility maintenance should be decreased. In addition, as the recommended I/I Reduction Program and other CIP projects are implemented, the engineering division will likely require additional staff to manage the program and projects.

8.4 SCHEDULE

An estimated schedule for the next six years is shown in Table 8-2. Again, the costs presented here are planning-level estimates using current (2021) dollar values. The actual cost for each project should be further refined in the pre-design and design phases.

TABLE 8-2: 6-YEAR CIP SCHEDULE

Project No.	Item	Cost (2021)	Opinion of Probable Costs					
			2022	2023	2024	2025	2026	2027
Priority 1 Improvements								
1.1	WWTP Influent Flow Meter	$ 68,000	$ 68,000					
1.2	Basin 4 Pipeline Upsize and Reroute	$ 3,600,000		$ 400,000	$ 3,200,000			
1.3	Basin 5 Pipeline Upsize	$ 4,500,000				$ 500,000	$ 4,000,000	
1.4	Install Overflow Alarms	$ 9,000	$ 9,000					
1.5	Pump Station 3 On-site Generator	$ 90,000	$ 90,000					
1.6	Annual I/I Reduction Program (6-Year)	$ 3,000,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000
	Total (Rounded)	$ 11,300,000	$ 700,000	$ 900,000	$ 3,700,000	$1,000,000	$ 4,500,000	$ 500,000

Note: The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2021 dollars and does not include any escalation.. 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.

8.5 OTHER FINANCIAL CONSIDERATIONS

The City previously had several wastewater debts that were refinanced into a single debt service in 2020. The payment comes out of the enterprise fund as a transfer and pays into a Debt Service Fund that is combined with water and street fund monies. The yearly transfer for this payment is $600,000, and is set to mature in 2034.

The schedule of payments is displayed in Table 8-3 and best correlates with the required payments had the refinance not been done. The City is currently exploring options to paying off the sewer debt sooner, potentially between 2026 and 2031.

TABLE 8-3: CITY WASTEWATER DEBT CURRENT PAYMENT SCHEDULE

Year of Payment	20/21	21/22	22/23	23/24	24/25	25/26	26/27	27/28	28/29	29/30	30/31	31/32	32/33	33/34
Payment Amount	600k	600k	600k	600k	600k	600k	420k	420k	420k	420k	420k	360k	310k	100k

It is recommended the City complete a full-rate study for the wastewater utility to evaluate the potential user rate and system development charge (SDC) impacts of the recommended CIP. Estimated SDC eligibility for each identified capital improvement was included in Table 8-1 above for use in completing a full rate study. It is recommended the City actively pursue opportunities for grant funds, low-interest loans, or principal forgiveness funding sources to mitigate user rate impacts. As the City begins to prepare and proceed on CIP projects, if outside funding is desired, it is recommended the City setup a one-stop meeting with Business Oregon to identify and assess potential funding sources for the sewer projects.

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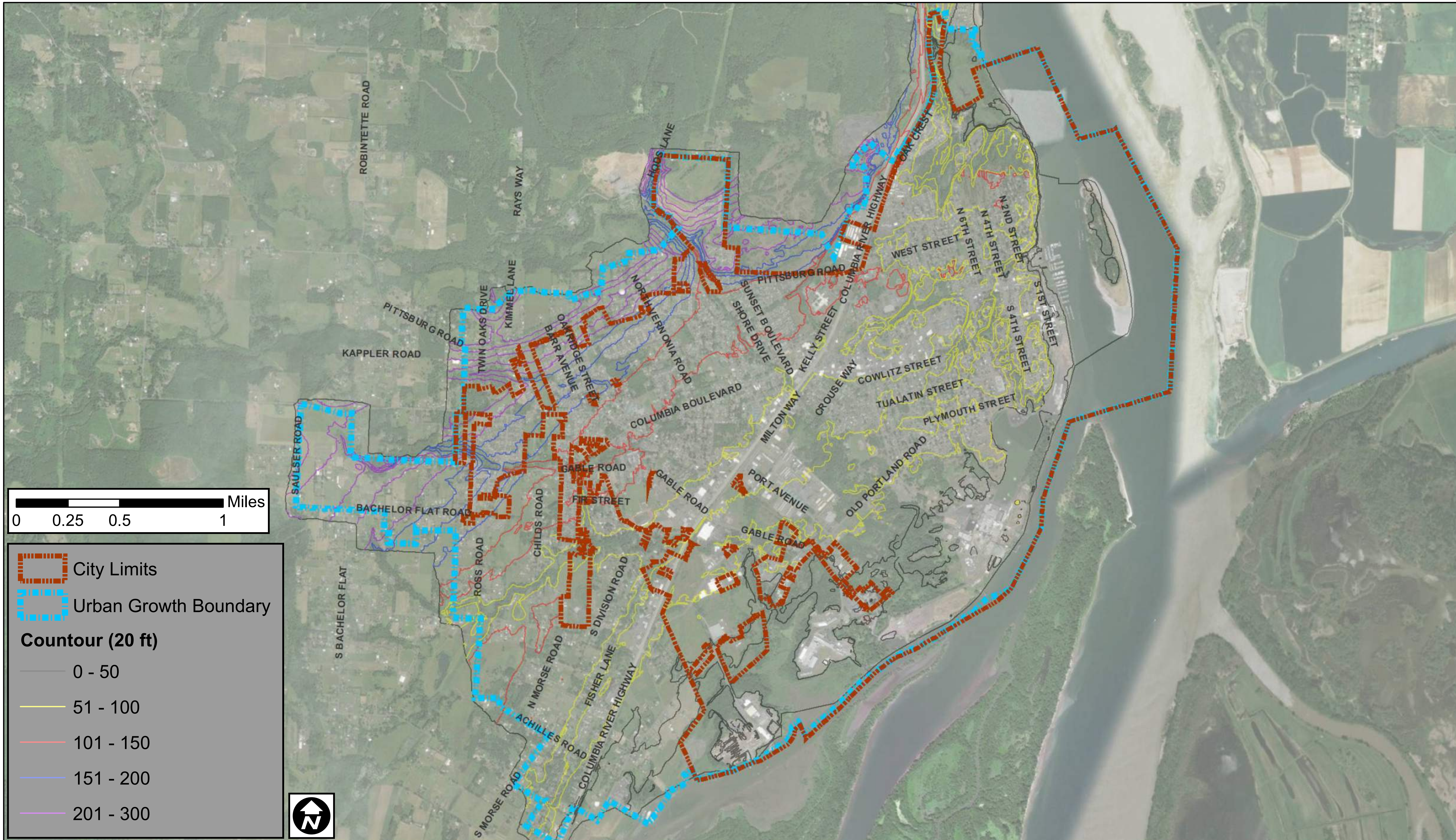
APPENDIX A

Figures

KELLER
ASSOCIATES



KELLER ASSOCIATES

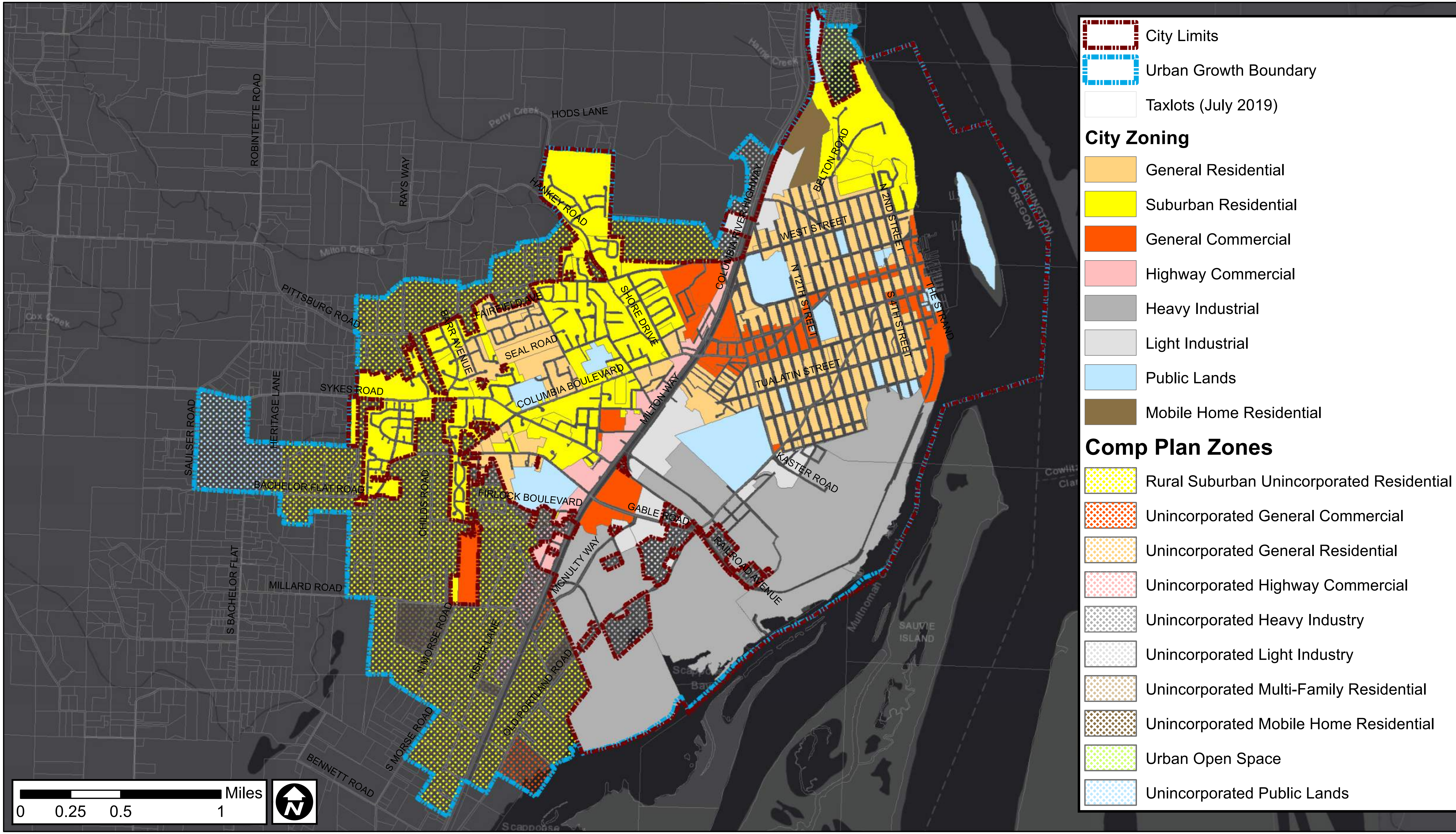
City Limits, UGB, and Topography

Wastewater Master Plan

City of St. Helens
FOUNDED 1850
Oregon

Figure 1

City of St. Helens



Zoning

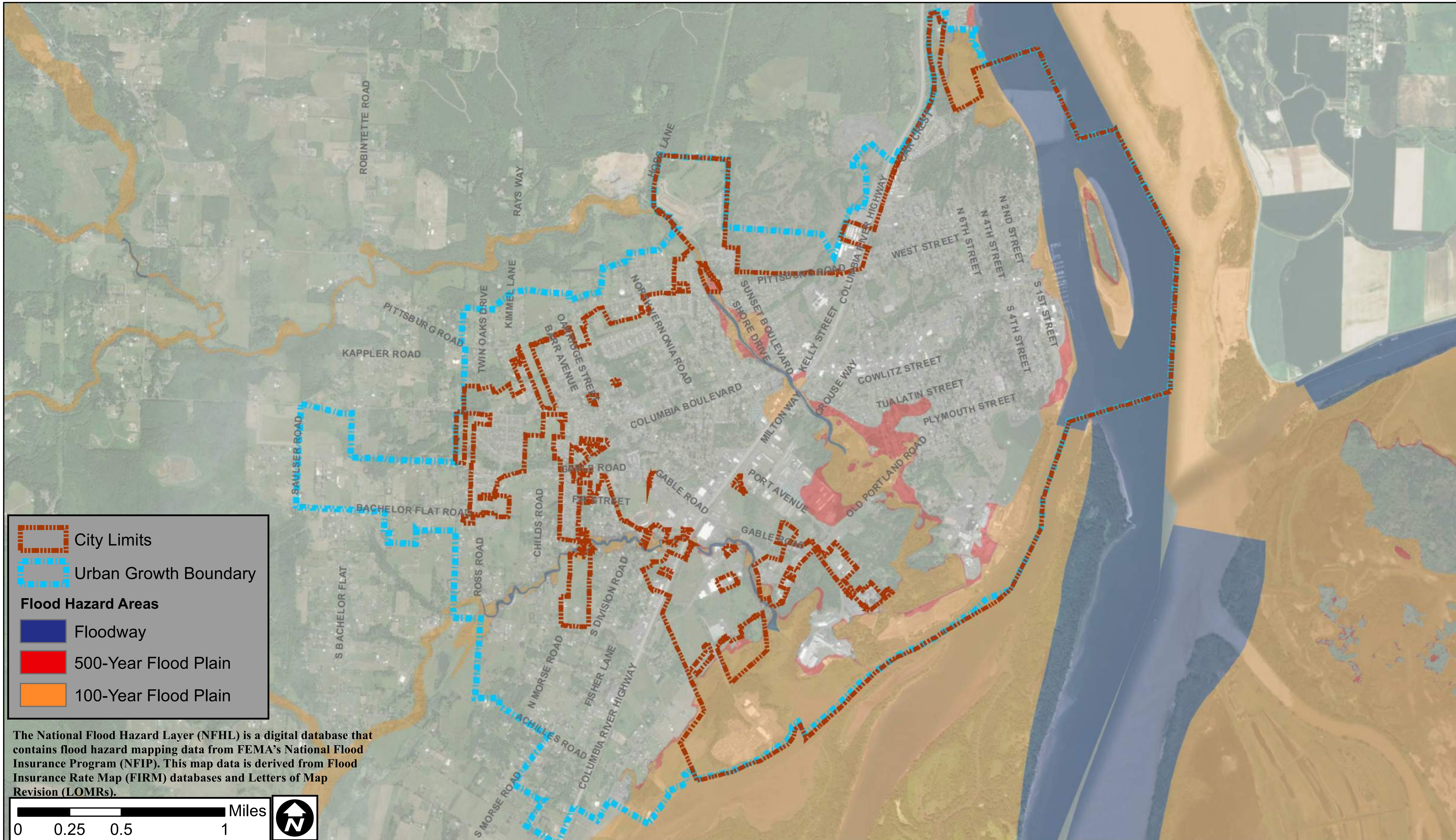
Wastewater Master Plan

KELLER ASSOCIATES

City of St. Helens Founded 1850 Oregon

Figure 2

City of St. Helens



KELLER ASSOCIATES

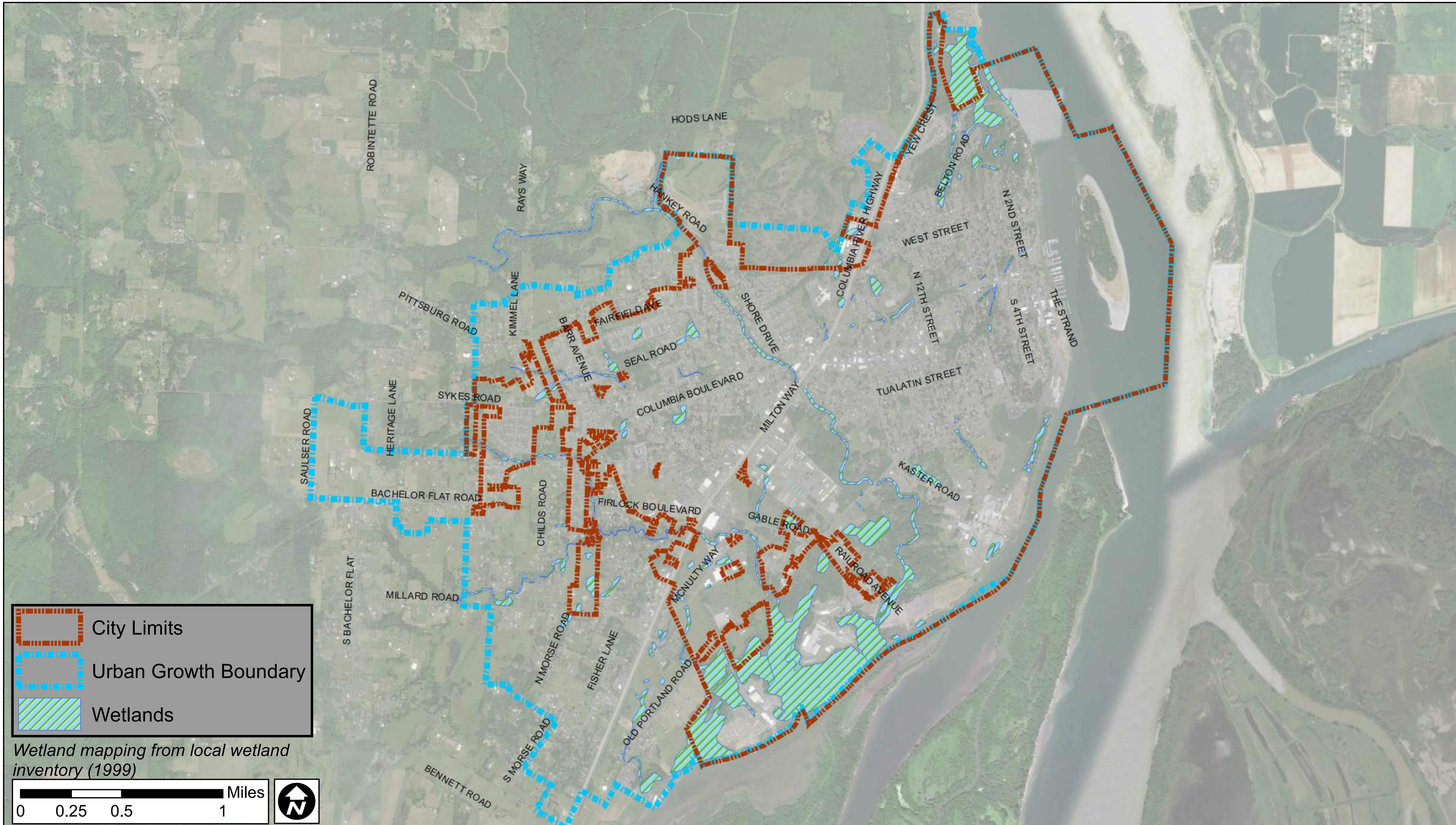
Flood Hazard Zones

Wastewater Master Plan

City of St. Helens, Founded 1850, Oregon

Figure 3

City of St. Helens



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Wetlands

Wastewater Master Plan

City of St. Helens
FOUNDED 1850
Oregon

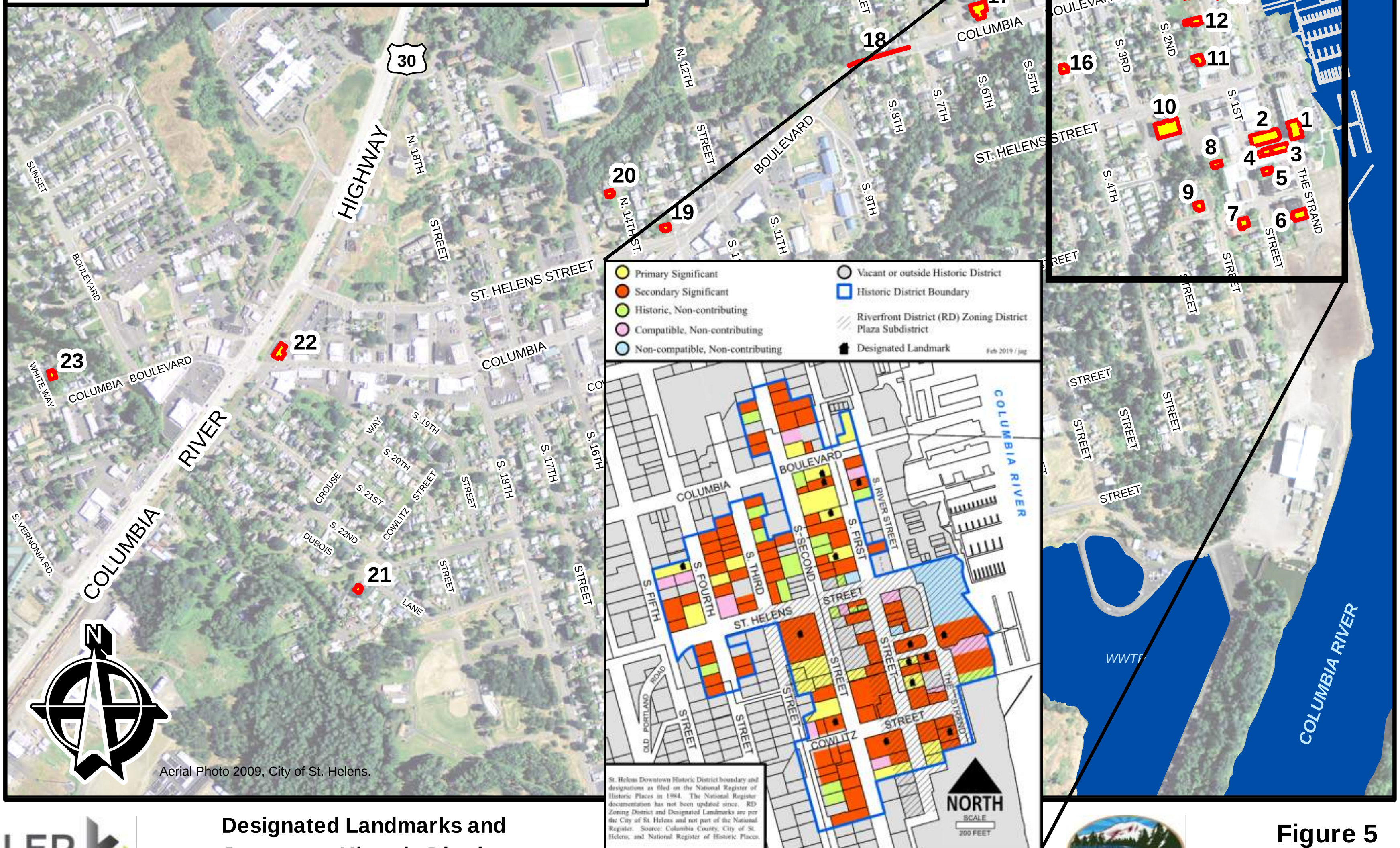
Figure 4

City of St. Helens

City of St. Helens
Designated Landmarks Regsiter Map

Ordinance No. 3250 Aerial Exhibit

SHMC 19.20.030

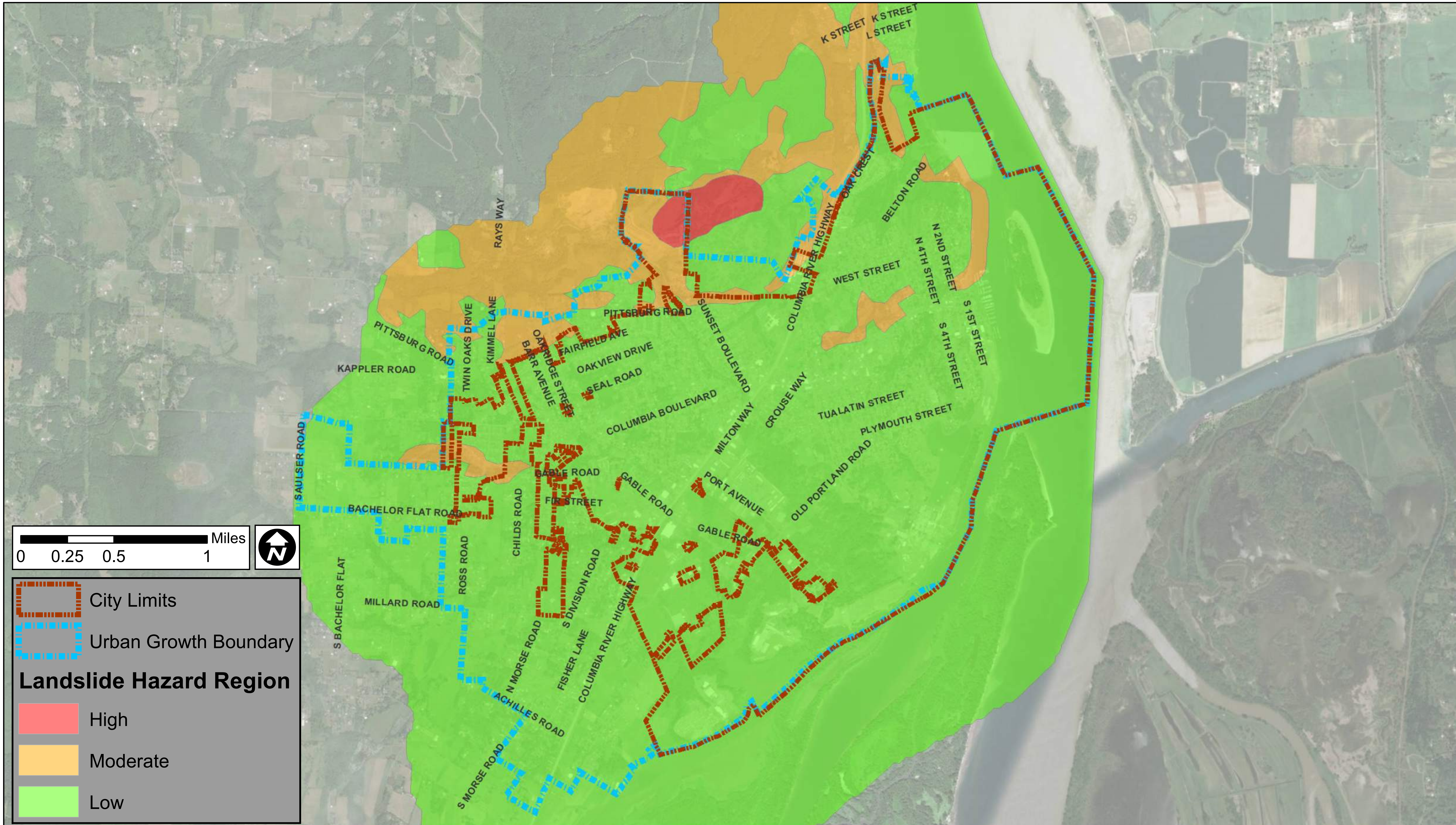


Designated Landmarks and Downtown Historic District

Wastewater Master Plan

Figure 5

City of St. Helens, OR

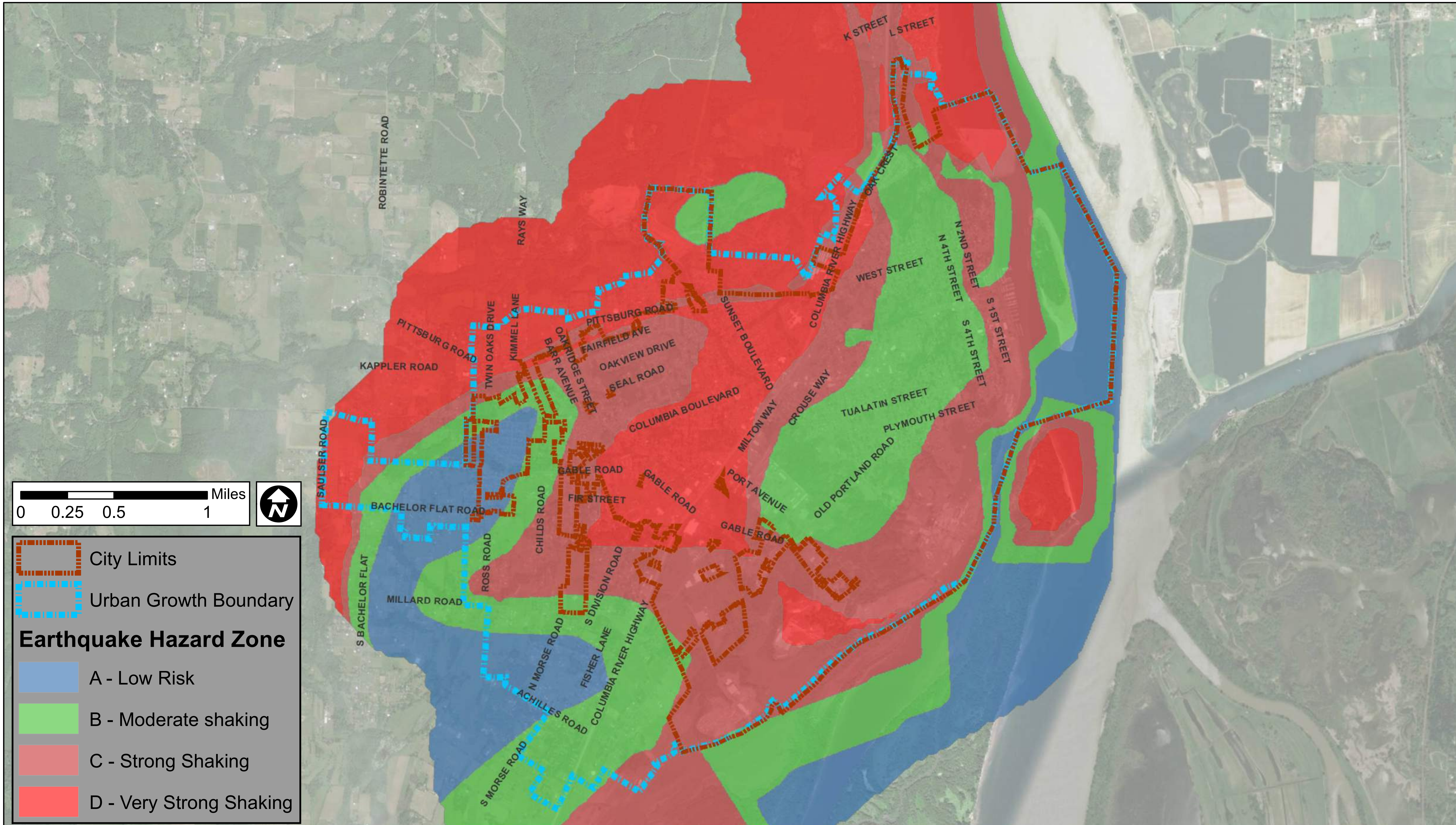


Landslide Hazards

Wastewater Master Plan

Figure 6

City of St. Helens



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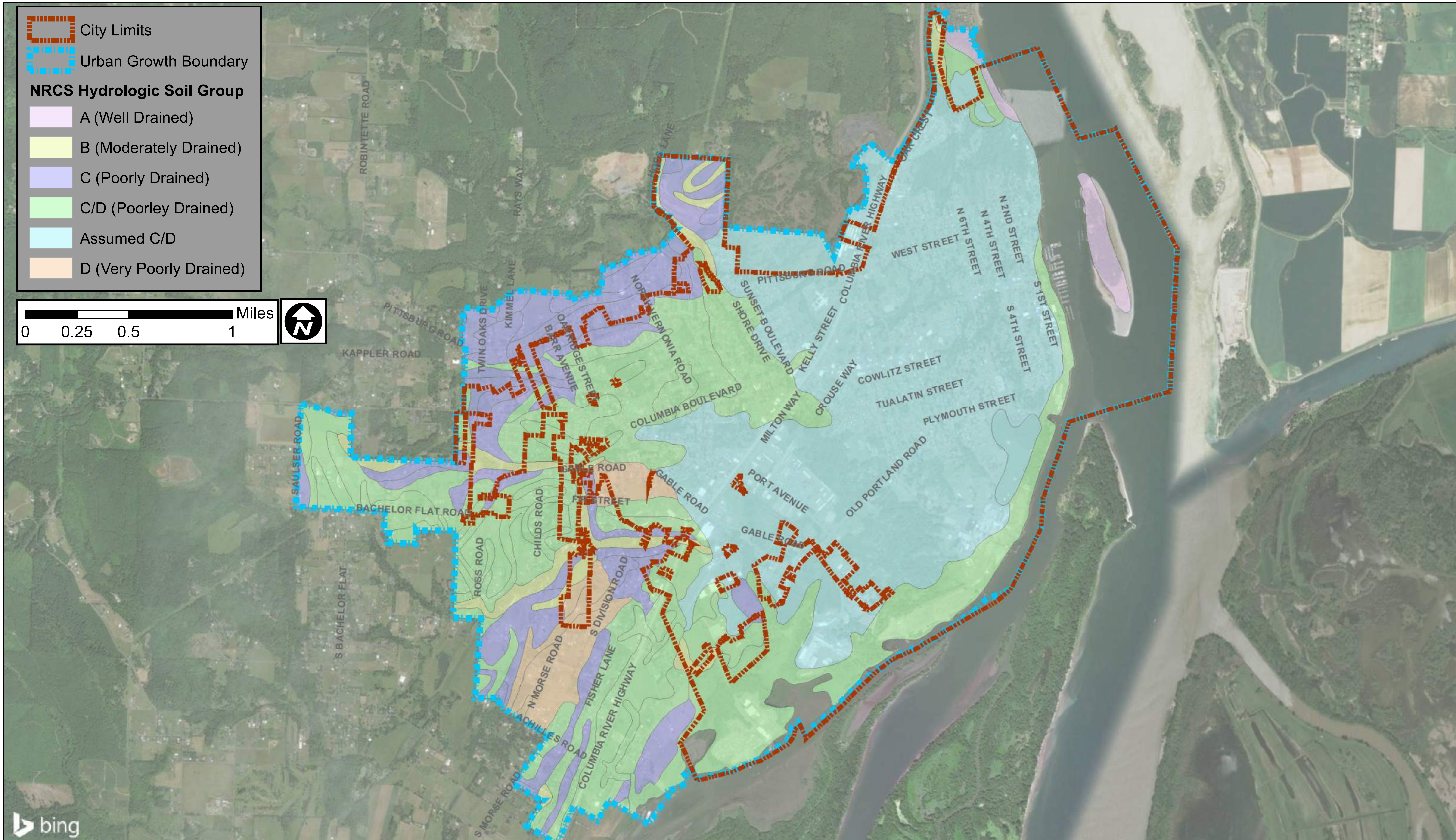
Earthquake Hazards

Wastewater Master Plan

City of St. Helens FOUNDED 1850 Oregon

Figure 7

City of St. Helens



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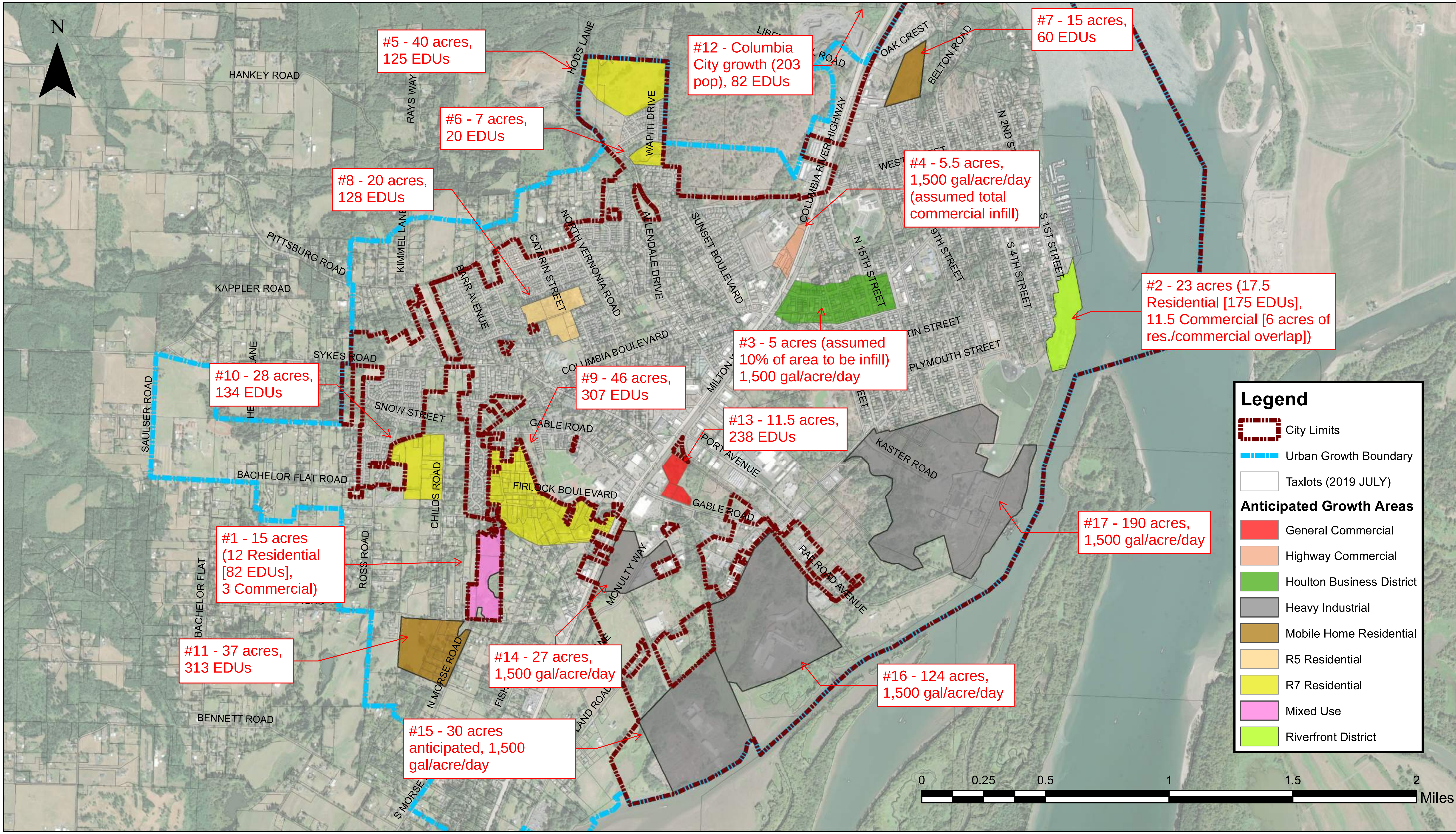
NRCS Hydrologic Soil Categories

Wastewater Master Plan

City of St. Helens Oregon

Figure 8

City of St. Helens

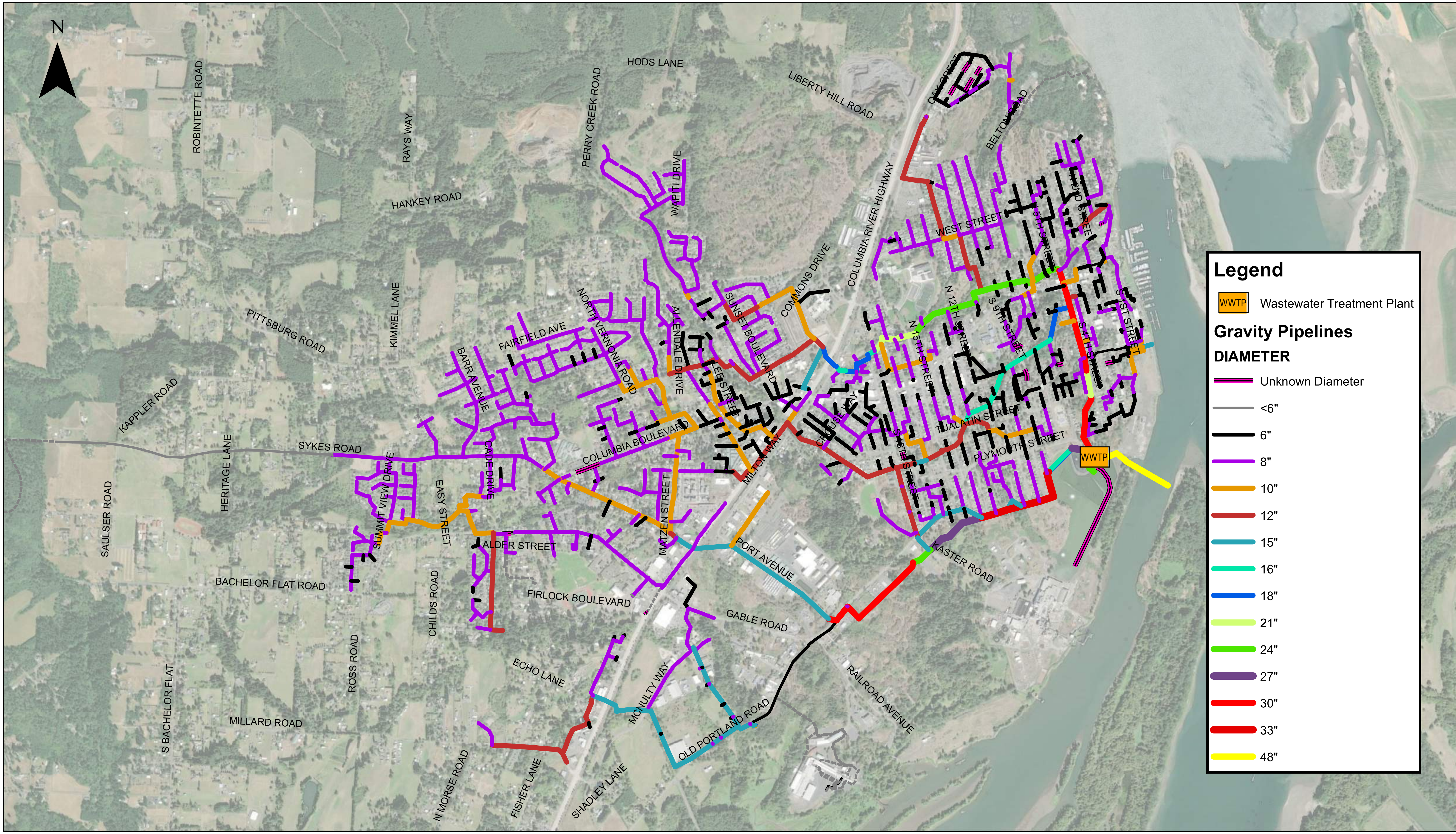


Anticipated Growth Areas (20-Year)

Wastewater Master Plan

Figure 9

City of St. Helens, OR



KELLER ASSOCIATES

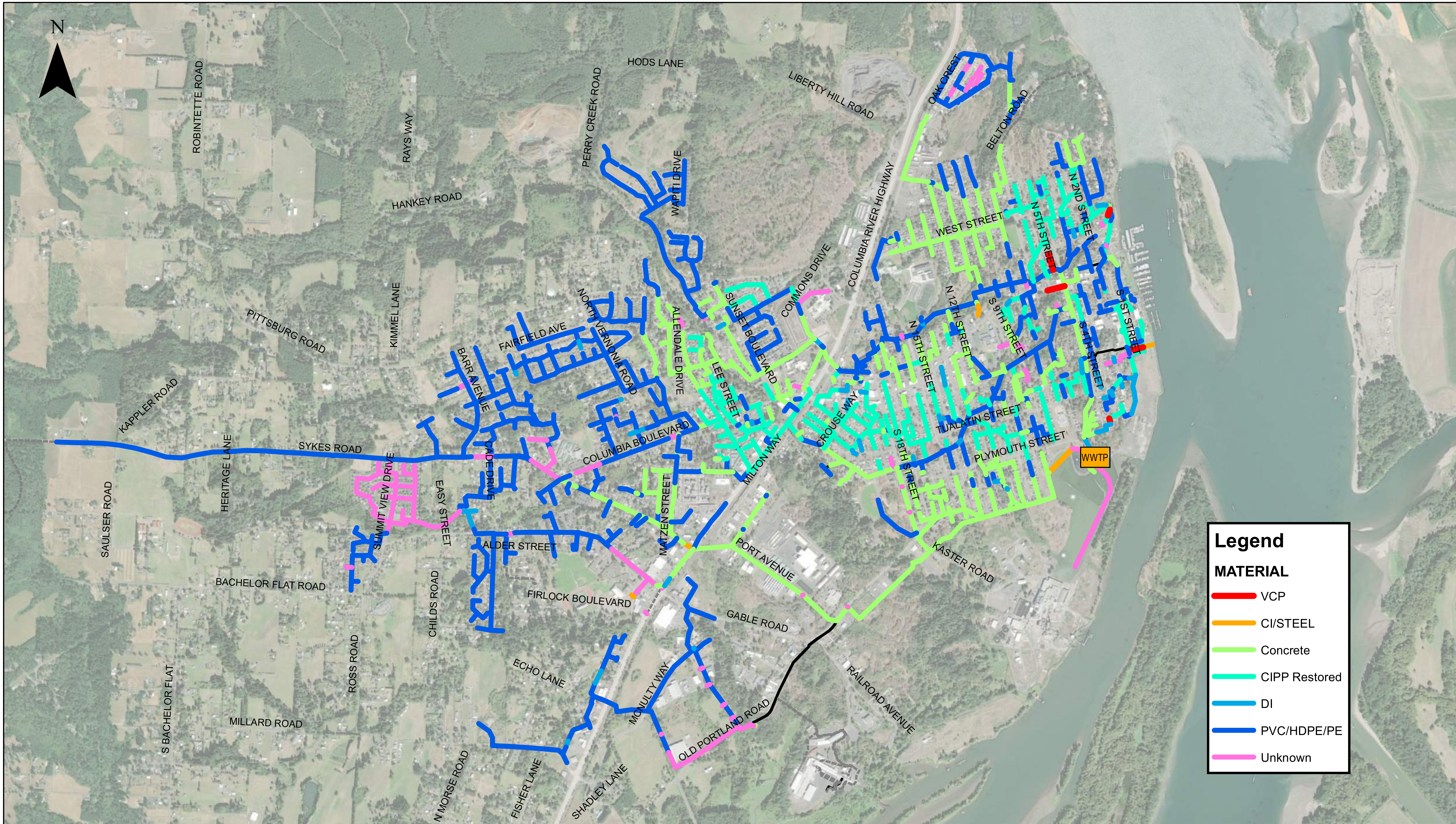
Pipelines by Size

Wastewater Master Plan

City of St. Helens FOUNDED 1850 Oregon

Figure 10

City of St. Helens, OR



KELLER ASSOCIATES

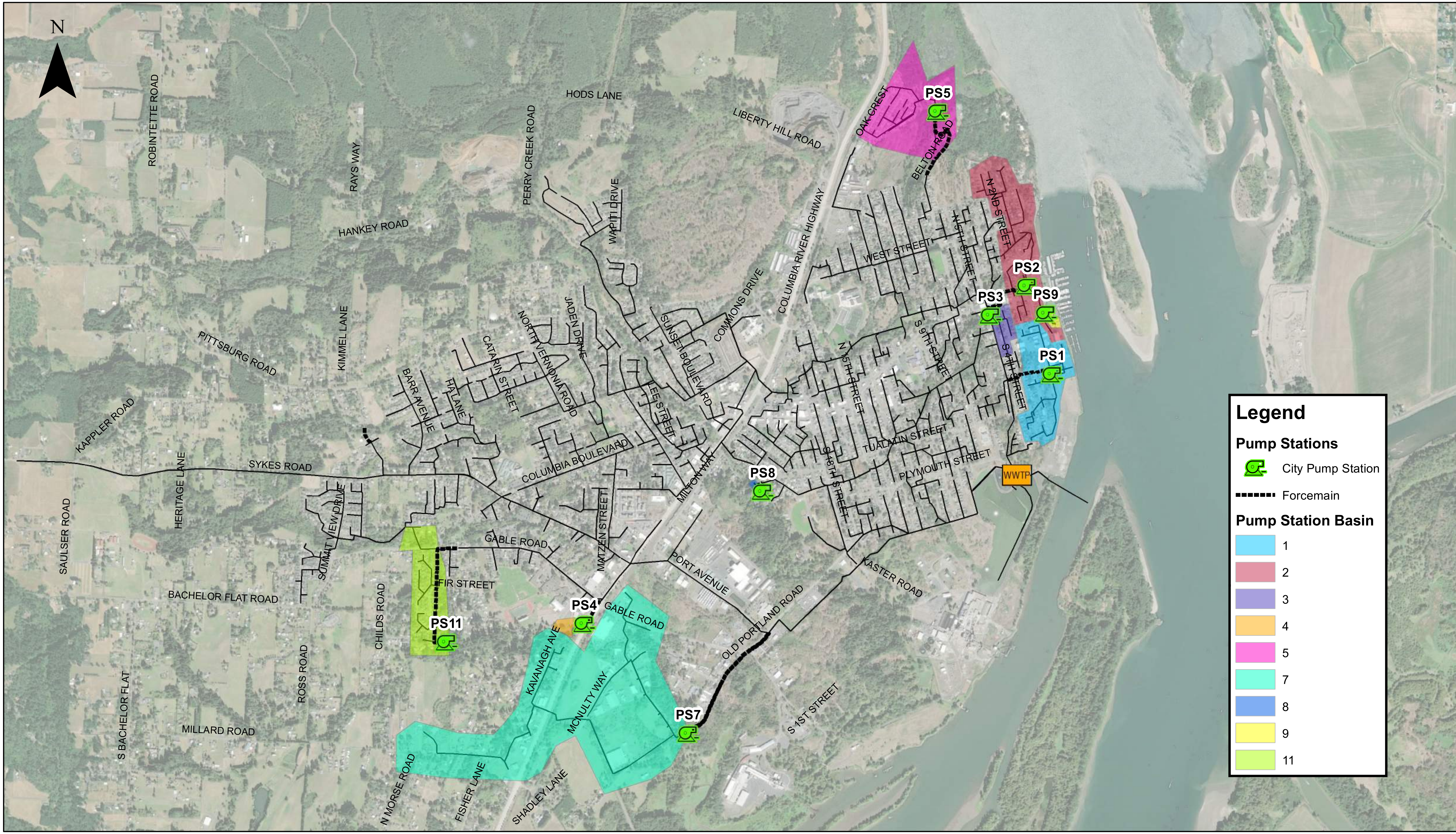
Pipelines by Material

Wastewater Master Plan

City of St. Helens FOUNDED 1850 Oregon

Figure 11

City of St. Helens, OR

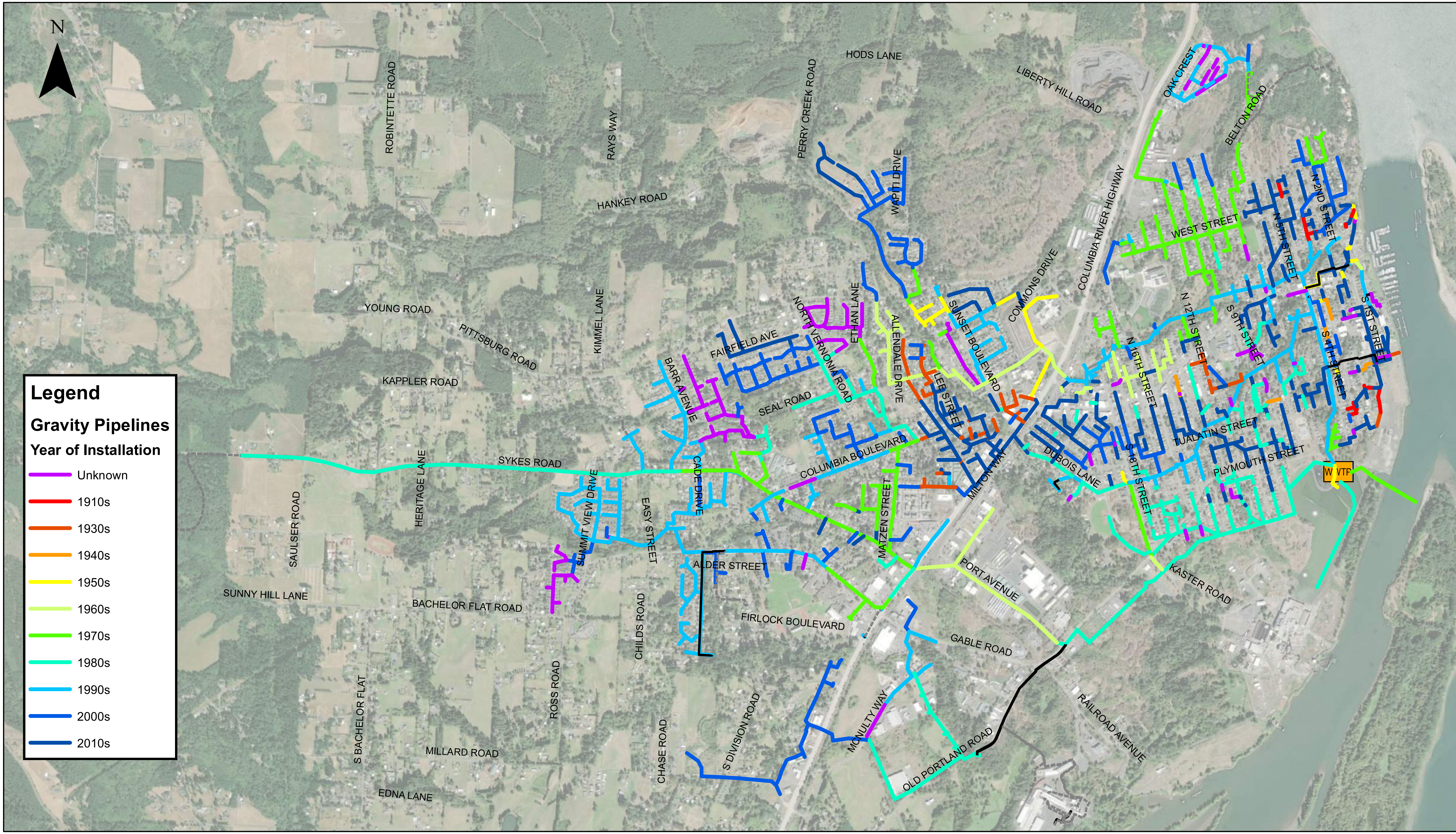


Pump Station Basins

Wastewater Master Plan

Figure 12

City of St. Helens, OR

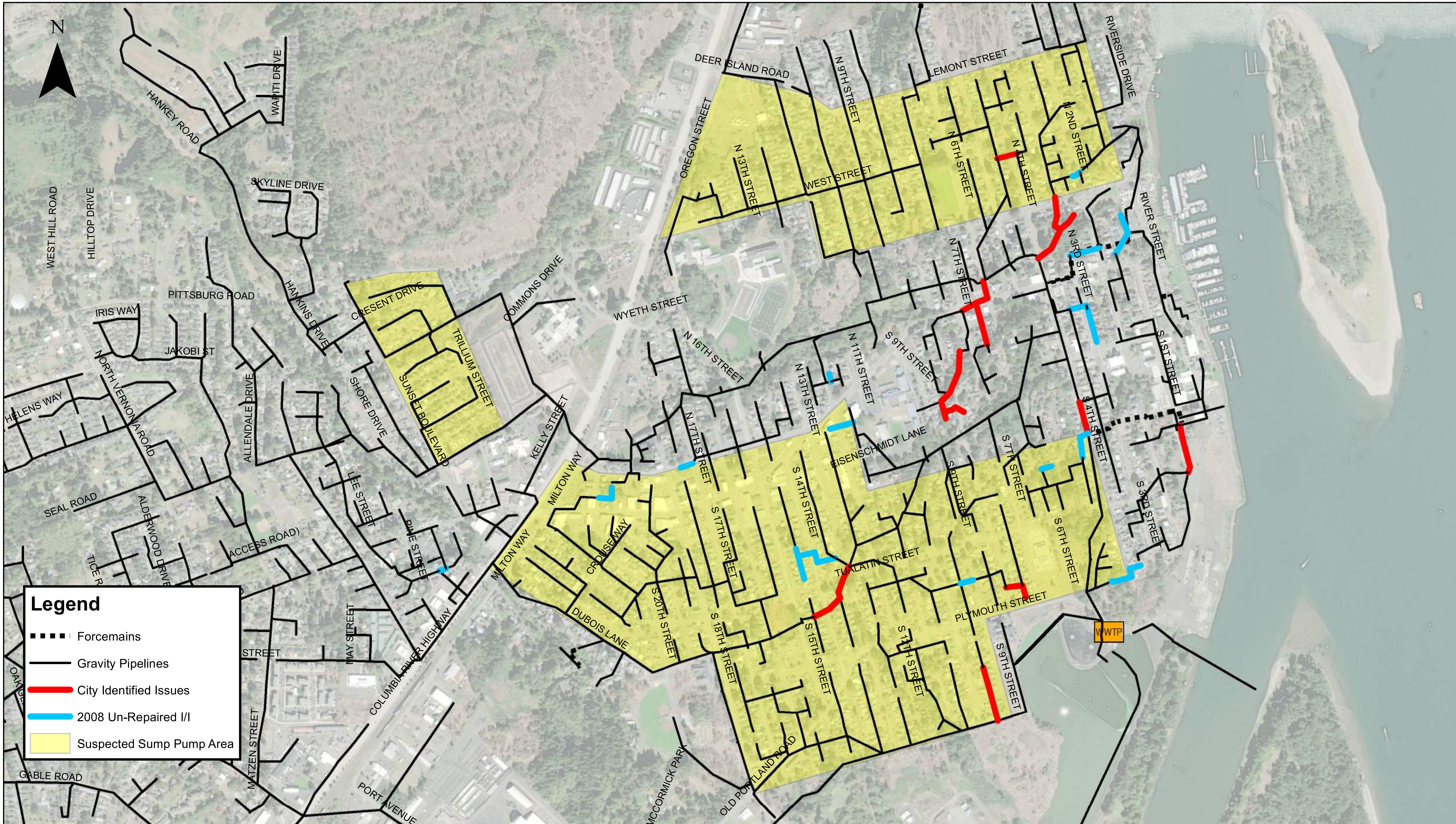


Infiltration and Inflow (I/I) - Pipeline Age

Wastewater Master Plan

Figure 13

City of St. Helens, OR



Infiltration and Inflow (I/I) - Suspected Sump Pump Areas, City Identified Issues, Unrepaired Priority Pipelines from 2008 Study

Wastewater Master Plan

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City of St. Helens FOUNDED 1850 Oregon

Figure 14

City of St. Helens, OR

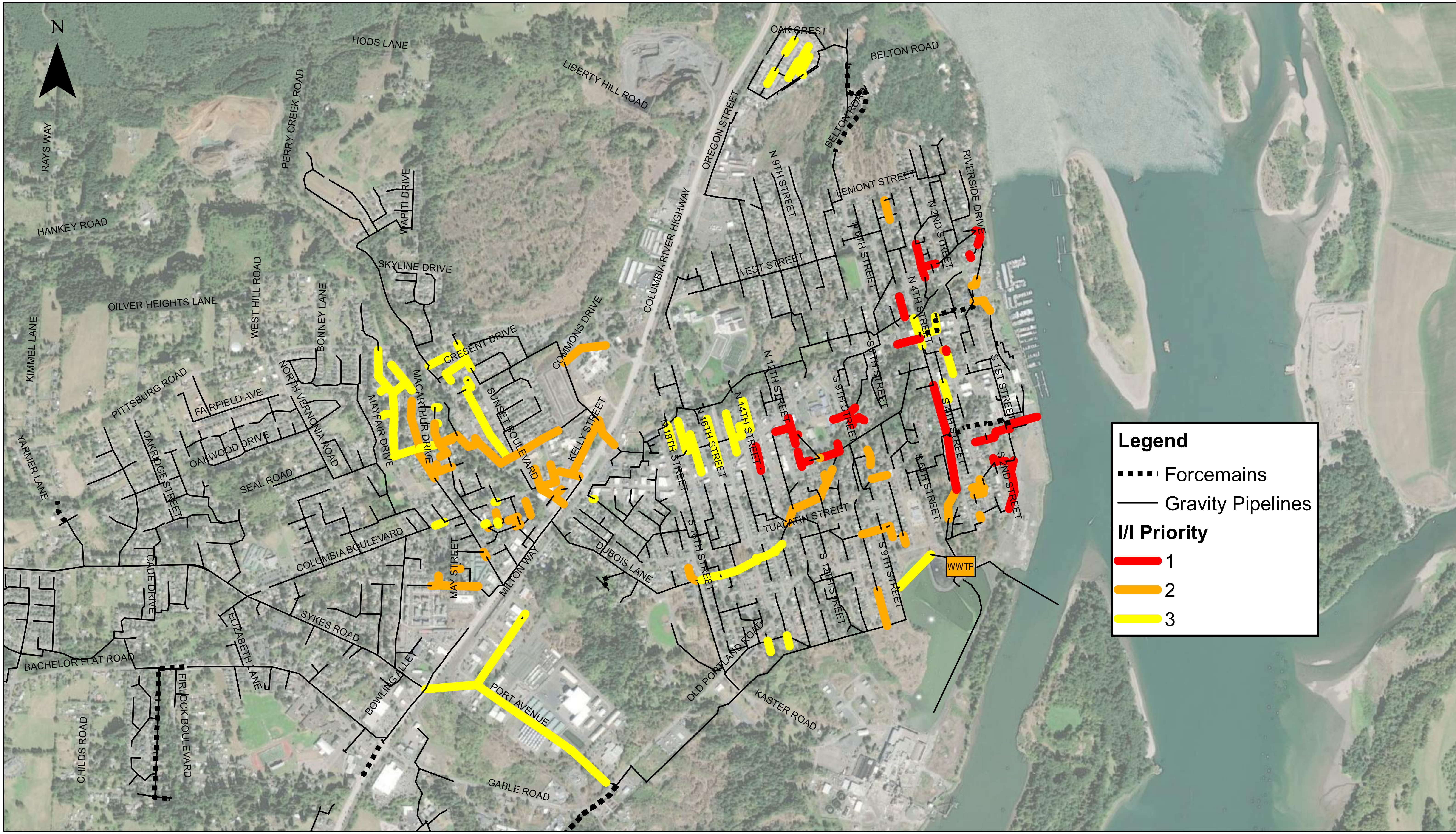
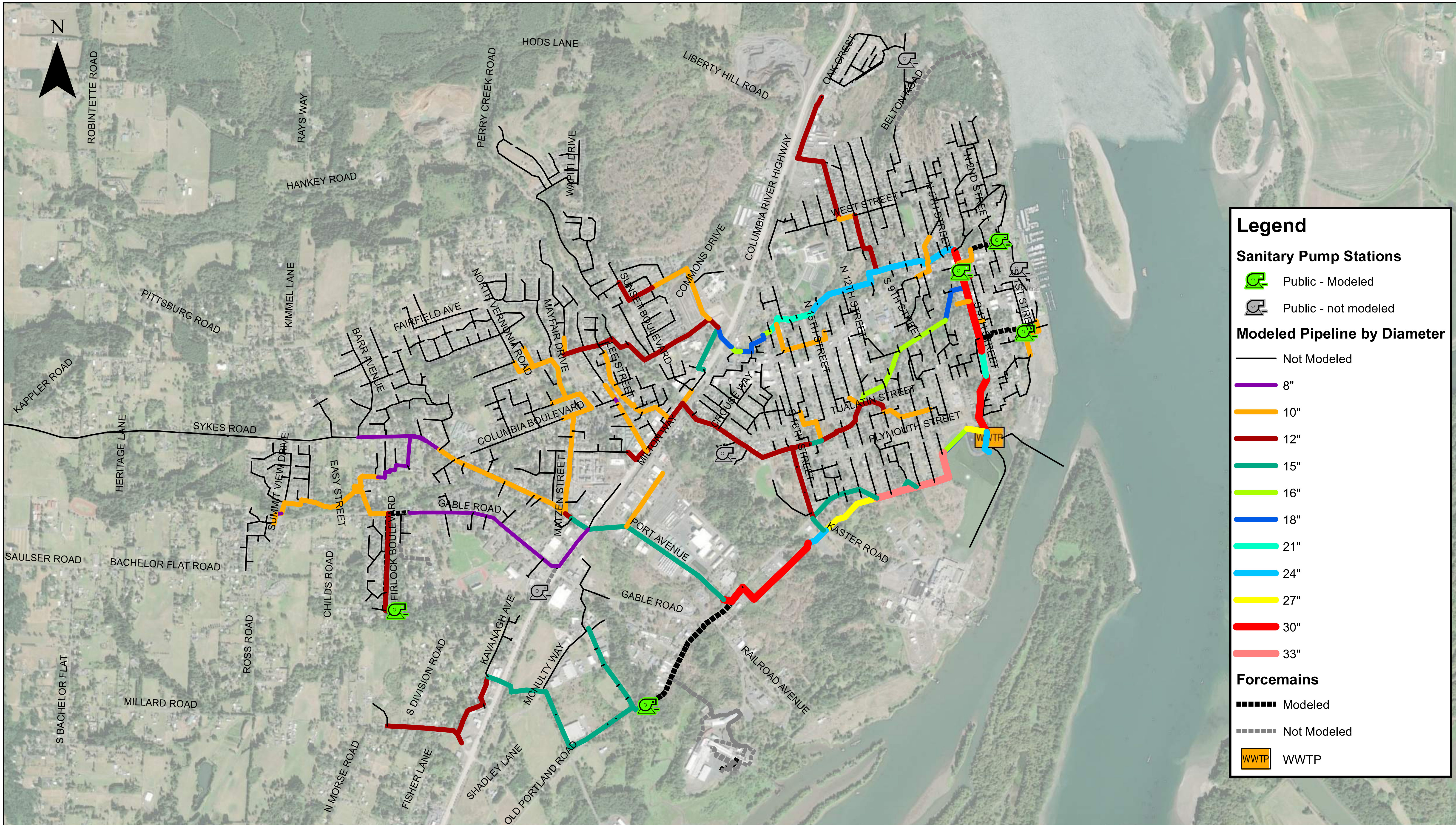


Figure 15

City of St. Helens, OR

Inflow and Infiltration (I/I) - Priority Pipelines

Wastewater Master Plan



Modeled Pipelines by Size

Wastewater Master Plan

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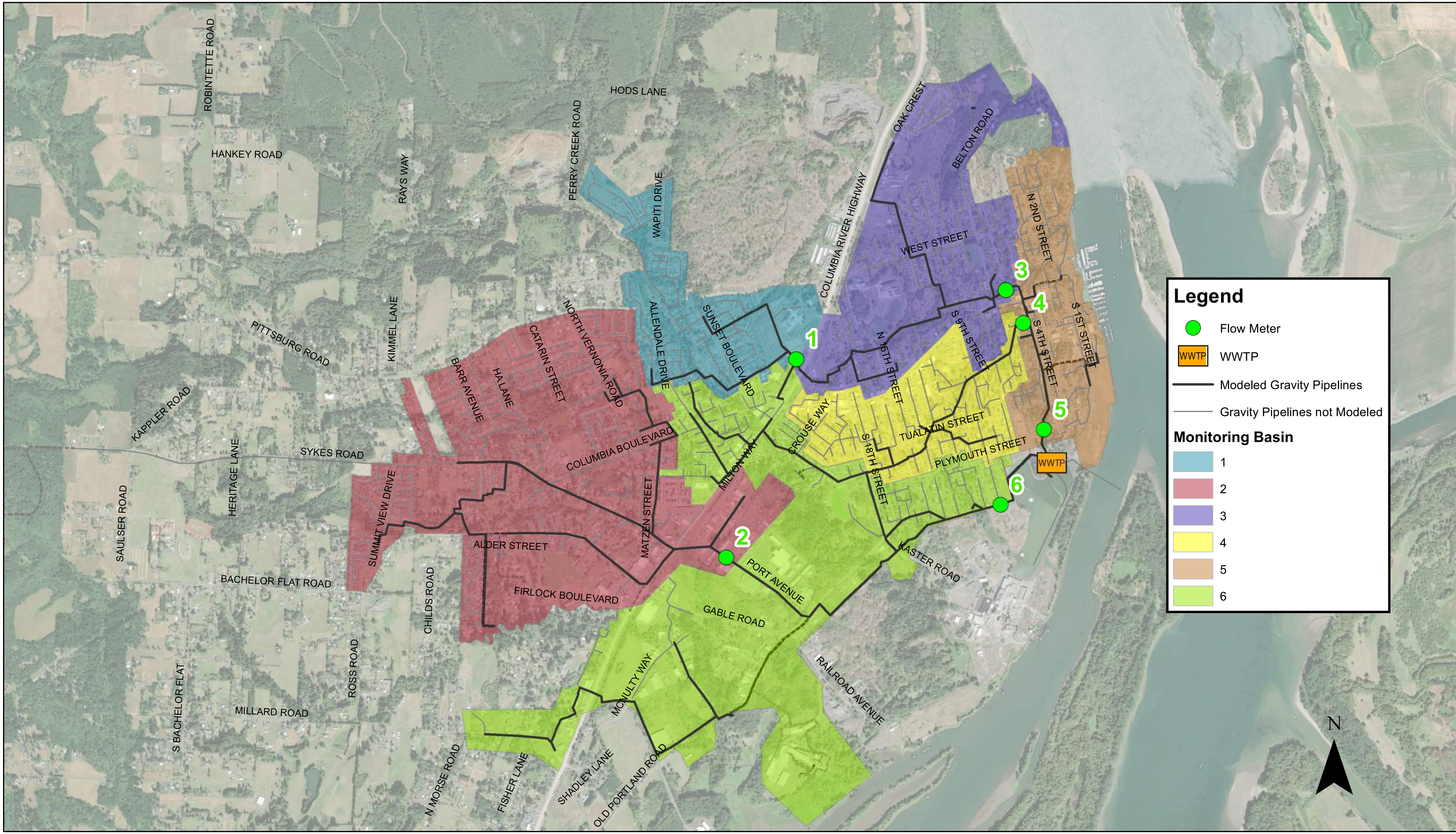
City of St. Helens FOUNDED 1850 Oregon

Figure 16

City of St. Helens, OR



Flow Meter Locations and Basins

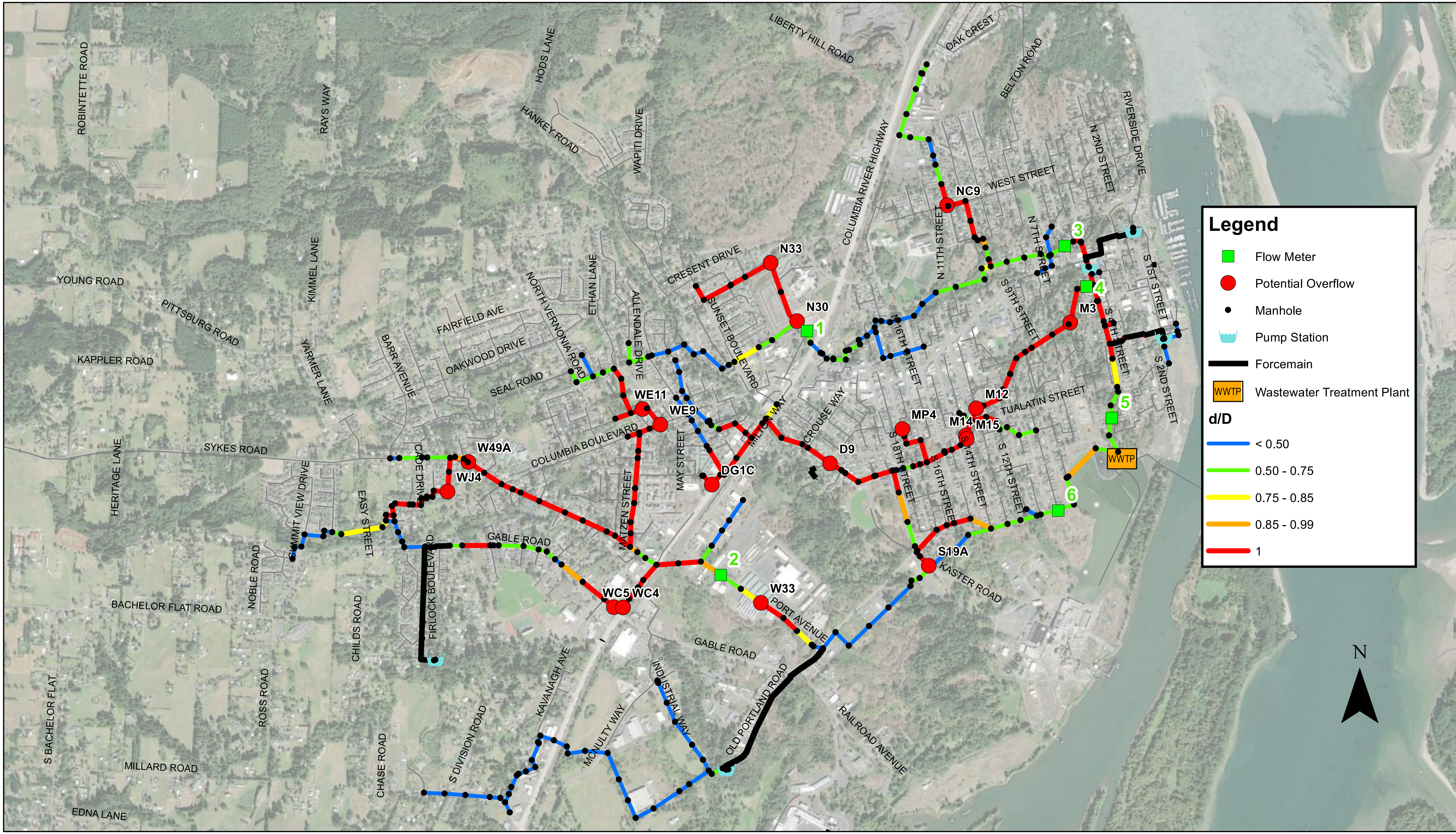
Wastewater Master Plan

Figure 17

City of St. Helens, OR

KELLER ASSOCIATES

City of St. Helens FOUNDED 1850 Oregon



KELLER ASSOCIATES

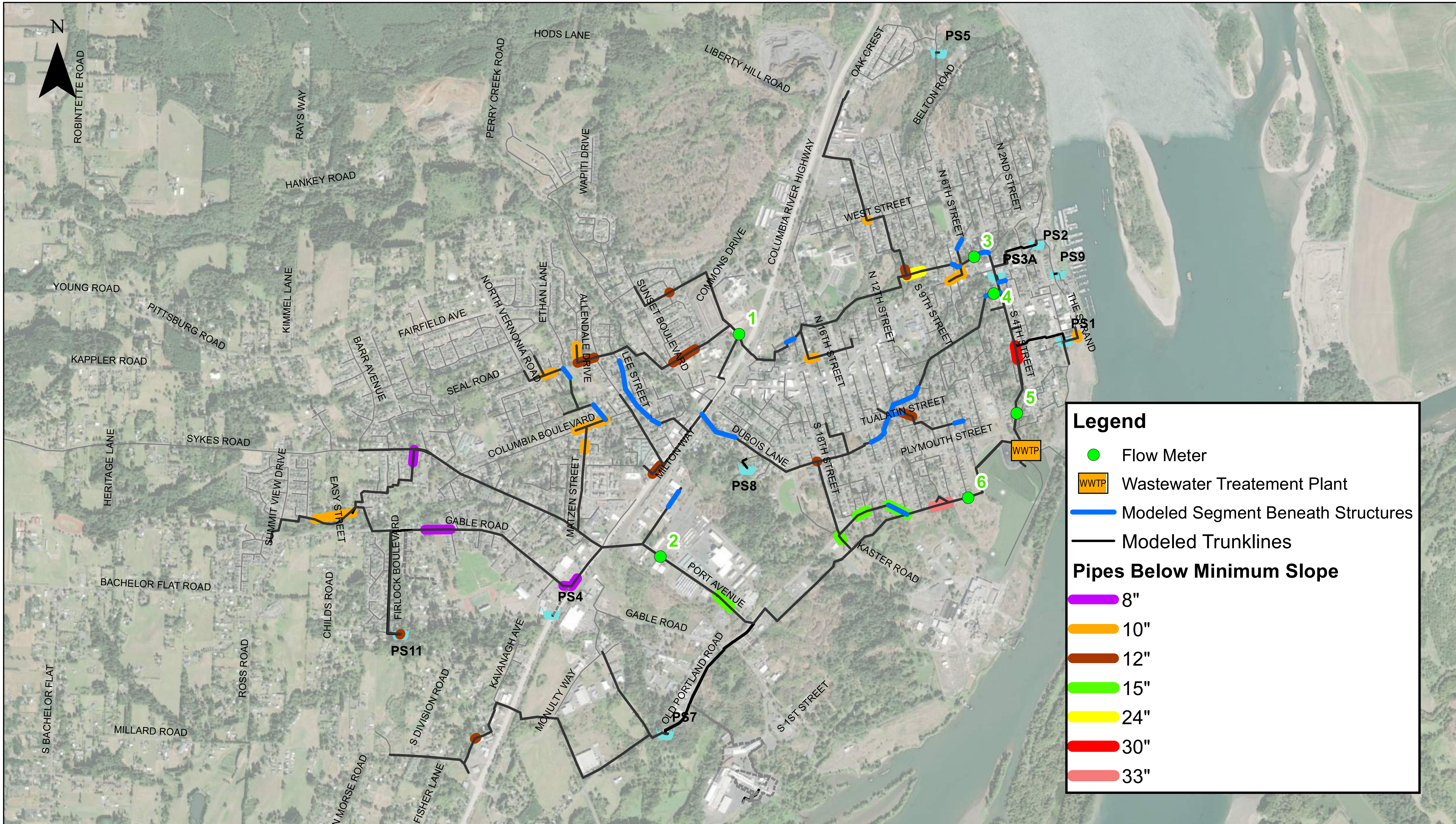
Existing System Evaluation - d/D and Potential Overflow Locations

Wastewater Master Plan

City of St. Helens
FOUNDED 1850
Oregon

Figure 18

City of St. Helens, OR



Existing System - Critical Slope Locations and Pipeline Segments Potentially Beneath Structures

Wastewater Master Plan

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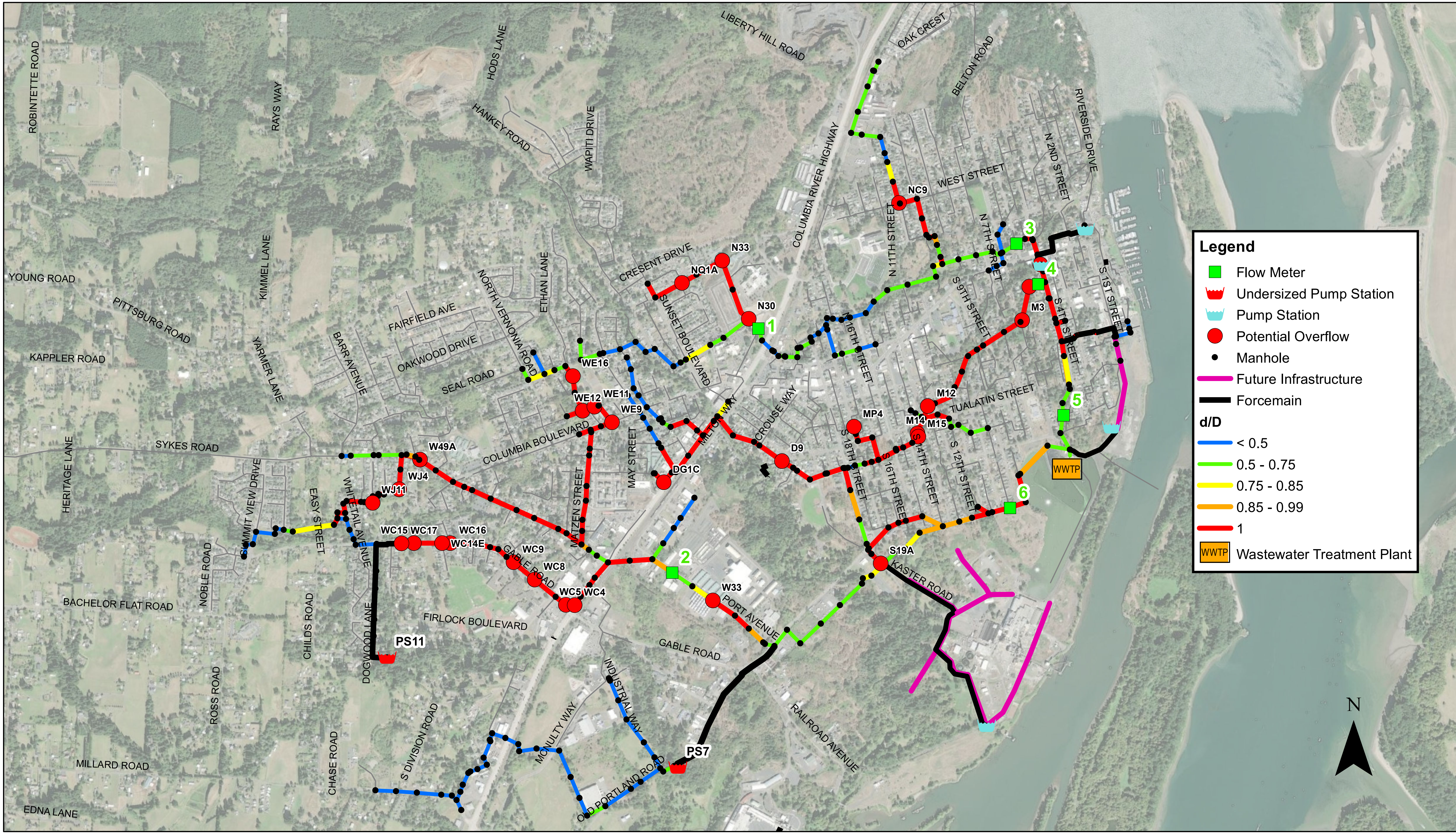
City of St. Helens FOUNDED 1850 Oregon

Figure 19

City of St. Helens, OR

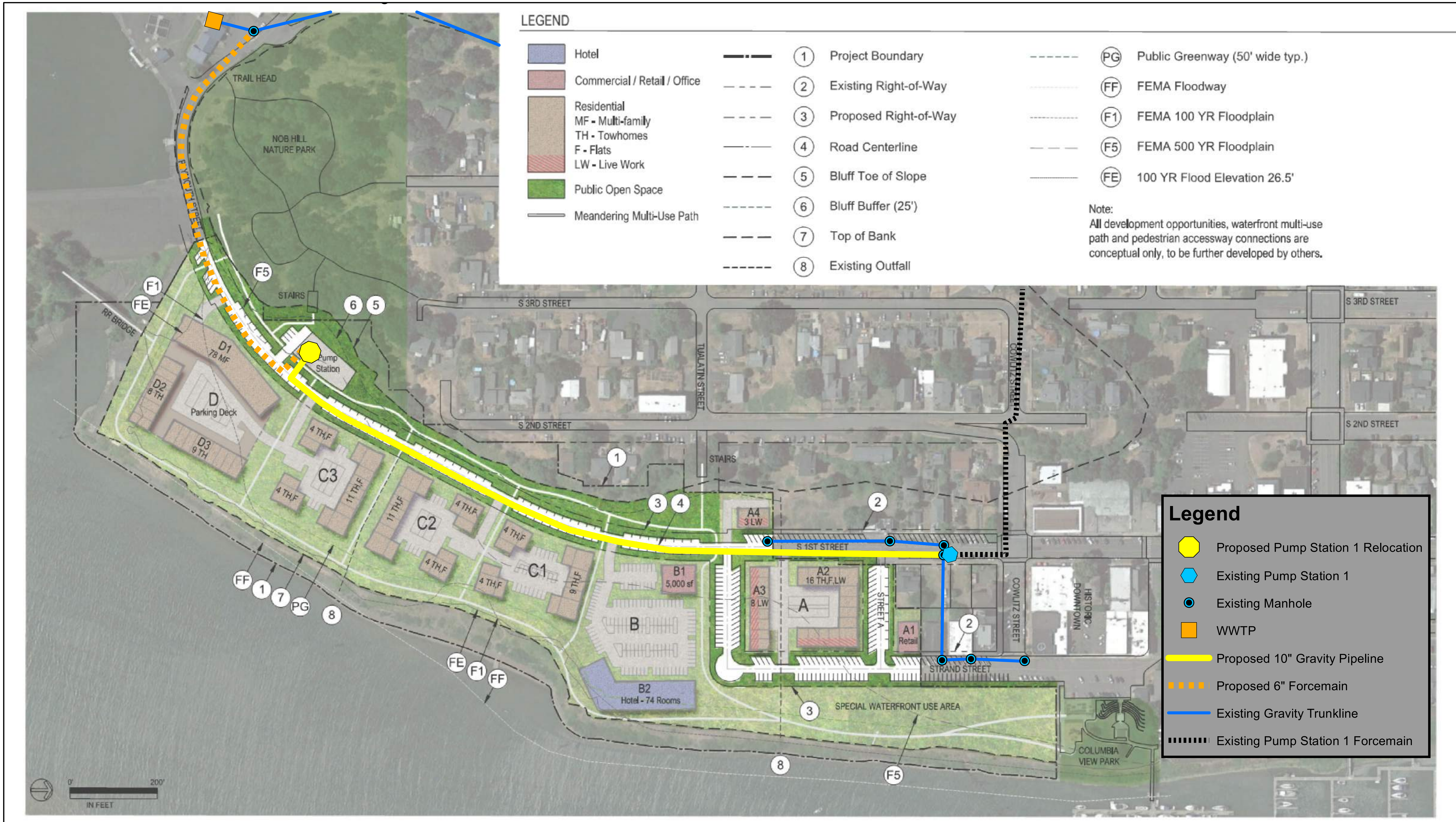


20 Year System Evaluation - d/D and Potential Overflow Locations

Wastewater Master Plan

Figure 20

City of St. Helens, OR



KELLER ASSOCIATES

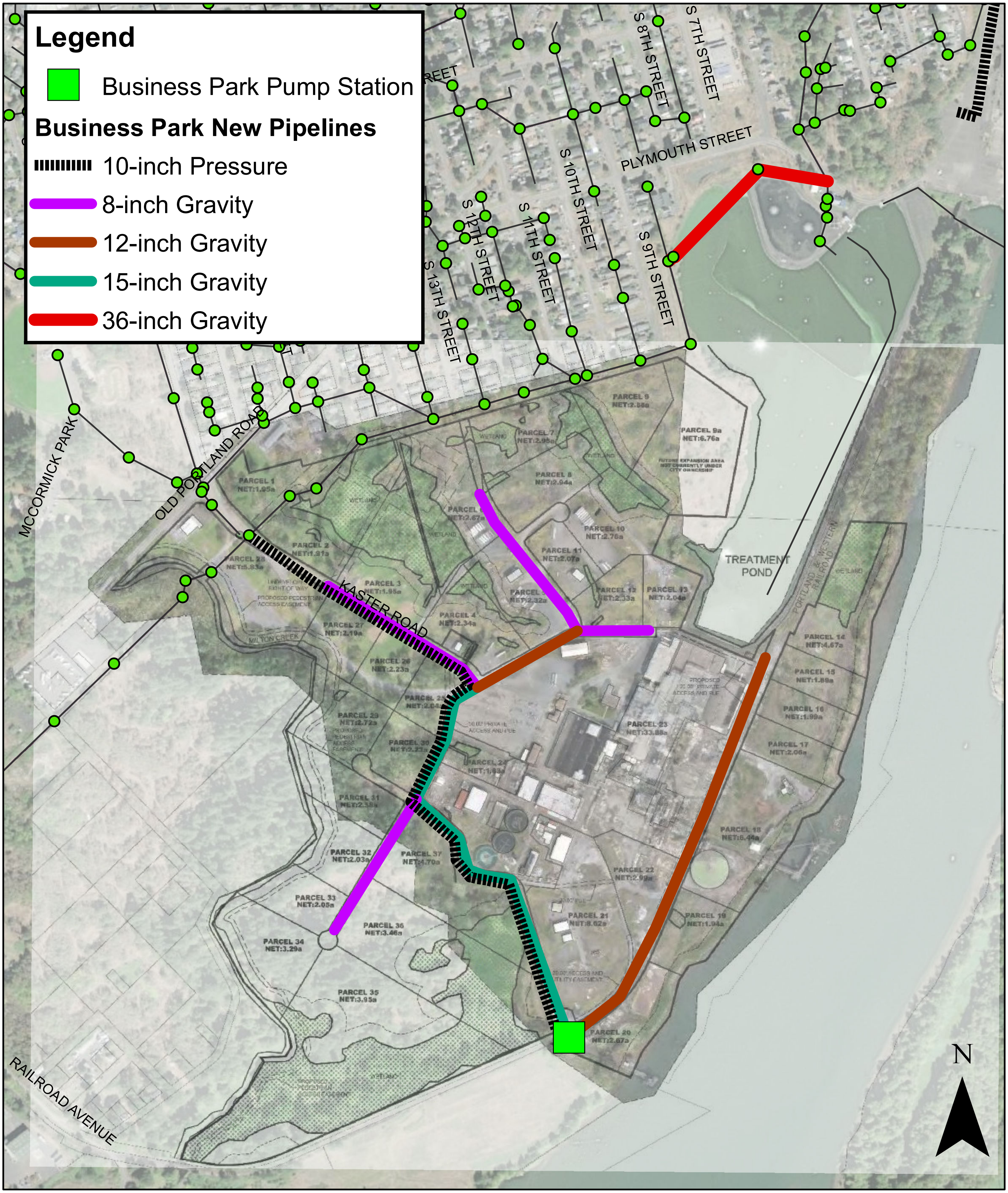
Riverfront District Proposed Infrastructure

Wastewater Master Plan

City of St. Helens FOUNDED 1850 Oregon

Figure 21

City of St. Helens, OR



Industrial Business Park Proposed Infrastructure

Wastewater Master Plan

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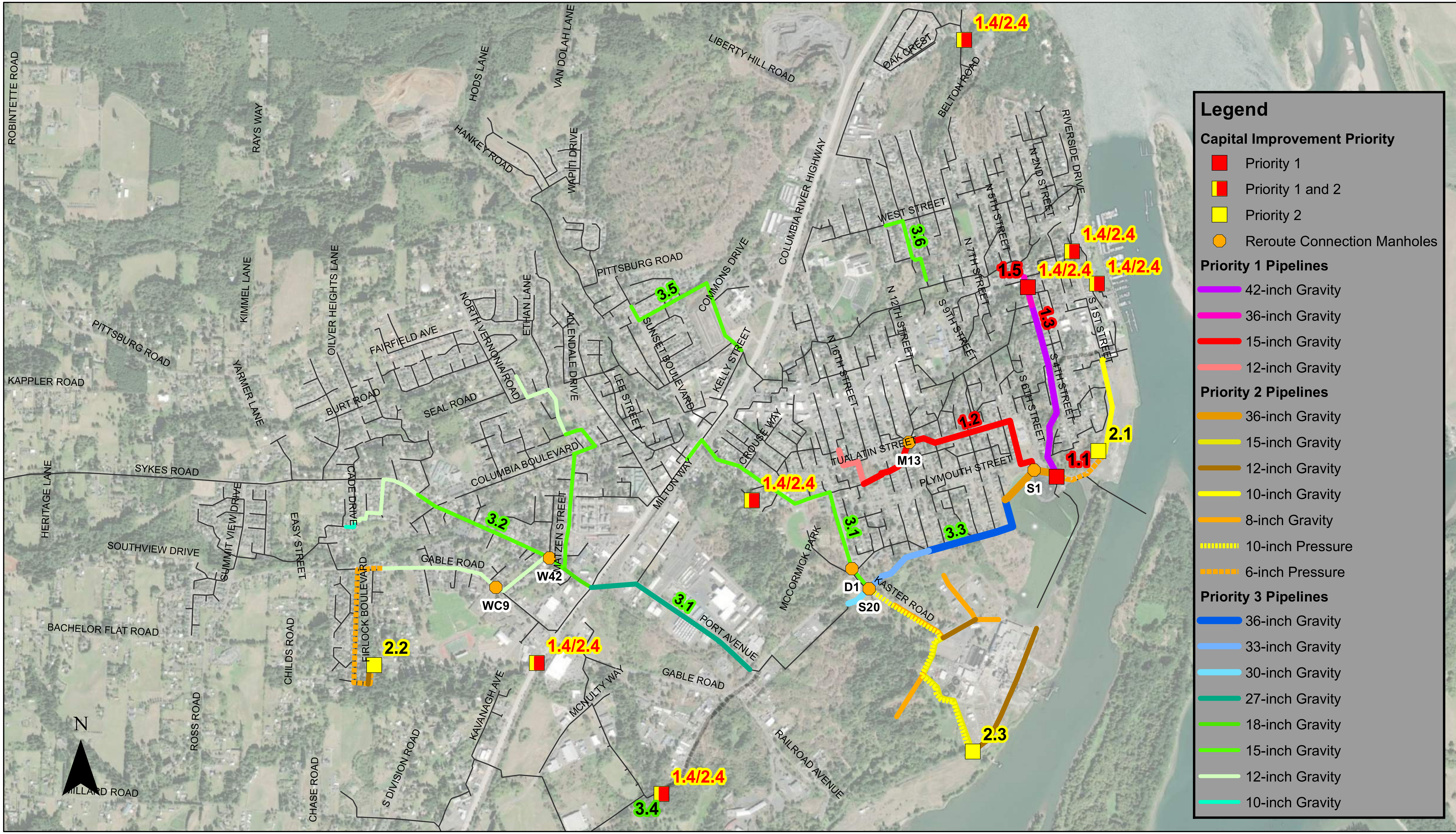
City of St. Helens
FOUNDED 1850
Oregon

Figure 22

City of St. Helens, OR



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Capital Improvement Plan

Wastewater Master Plan

Figure 23

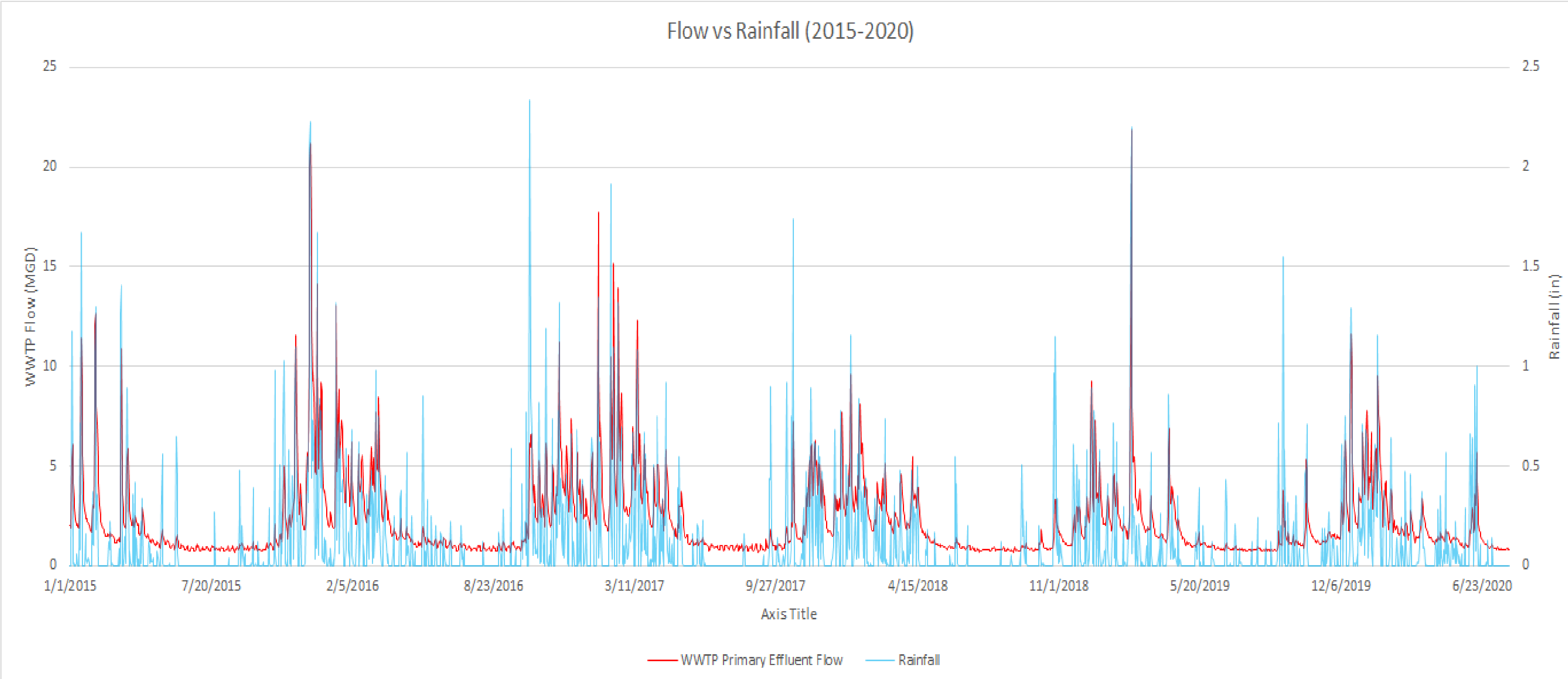
City of St. Helens, OR

Appendix B

Planning Criteria

Columbia County Endangered Species List

Group	Name	Population	Status	Lead Office	Recovery Plan	Recovery Plan Action Status
Snails	Burrington jumping-slug (Hemphillia burringtoni)	Wherever found	Under Review	1		
Fishes	Bull Trout (Salvelinus confluentus)	U.S.A., conterminous, (lower 48 states)	Threatened	1	Coastal Recovery Unit Implementation Plan for Bull Trout (Salvelinus confluentus)	Implementation Progress
Fishes	Bull Trout (Salvelinus confluentus)	U.S.A., conterminous, (lower 48 states)	Threatened	1	Columbia Headwaters Recovery Unit Implementation Plan for Bull Trout (Salvelinus confluentus)	Implementation Progress
Fishes	Bull Trout (Salvelinus confluentus)	U.S.A., conterminous, (lower 48 states)	Threatened	1	Klamath Recovery Unit Implementation Plan for Bull Trout (Salvelinus confluentus)	Implementation Progress
Fishes	Bull Trout (Salvelinus confluentus)	U.S.A., conterminous, (lower 48 states)	Threatened	1	Mid-Columbia Recovery Unit Implementation Plan for Bull Trout (Salvelinus confluentus)	Implementation Progress
Fishes	Bull Trout (Salvelinus confluentus)	U.S.A., conterminous, (lower 48 states)	Threatened	1	Recovery Plan for the Coterminous United States Population of Bull Trout (Salvelinus confluentus)	Implementation Progress
Fishes	Bull Trout (Salvelinus confluentus)	U.S.A., conterminous, (lower 48 states)	Threatened	1	St. Mary Recovery Unit Implementation Plan for Bull Trout (Salvelinus confluentus)	Implementation Progress
Fishes	Bull Trout (Salvelinus confluentus)	U.S.A., conterminous, (lower 48 states)	Threatened	1	Upper Snake Recovery Unit Implementation Plan for Bull Trout (Salvelinus confluentus)	Implementation Progress
Mammals	red tree vole (Arborimus longicaudus)	North Oregon Coast population	Resolved Taxon	1		
Birds	Northern spotted owl (Strix occidentalis caurina)	Wherever found	Threatened	1	Revised Recovery Plan for the Northern Spotted Owl	Implementation Progress
Flowering Plants	Nelson's checker-mallow (Sidalcea nelsoniana)	Wherever found	Threatened	1	Final Recovery Plan for the Prairie Species of Western Oregon and Southwestern Washington	Implementation Progress
Flowering Plants	Kincaid's Lupine (Lupinus sulphureus ssp. kincaidii)	Wherever found	Threatened	1	Final Recovery Plan for the Prairie Species of Western Oregon and Southwestern Washington	Implementation Progress
Flowering Plants	golden paintbrush (Castilleja levisecta)	Wherever found	Threatened	1	Recovery Plan for the Golden Paintbrush (Castilleja levisecta)	Implementation Progress
Birds	Marbled murrelet (Brachyramphus marmoratus)	U.S.A. (CA, OR, WA)	Threatened	1	Recovery Plan for the Threatened Marbled Murrelet (Brachyramphus marmoratus) in Washington, Oregon, and California	Implementation Progress
Flowering Plants	Willamette daisy (Erigeron decumbens)	Wherever found	Endangered	1	Final Recovery Plan for the Prairie Species of Western Oregon and Southwestern Washington	Implementation Progress
Birds	Streaked Horned lark (Eremophila alpestris strigata)	Wherever found	Threatened	1	Draft Recovery Plan for the Streaked Horned Lark	Implementation Progress
Flowering Plants	Bradshaw's desert-parsley (Lomatium bradshawii)	Wherever found	Endangered	1	Final Recovery Plan for the Prairie Species of Western Oregon and Southwestern Washington	Implementation Progress
Flowering Plants	Water howellia (Howellia aquatilis)		Threatened	6	Water Howellia (Howellia aquatilis) Recovery Plan, Public and Agency Review Draft	Implementation Progress
Mammals	Columbian white-tailed deer (Odocoileus virginianus leucurus)	Columbia River (Clark, Cowliz, Pacific, Skamania, and Wahkiakum Counties, WA., and Clatsop, Columbia, and Multnomah Counties, OR.)	Threatened	1	Columbian White-tailed Deer Revised Recovery Plan	Implementation Progress
Birds	Yellow-billed Cuckoo (Coccyzus americanus)	Western DPS: U.S.A. (AZ, CA, CO (western), ID, MT (western), NM (western), NV, OR, TX (western), UT, WA, WY (western)); Canada (British Columbia (southwestern); Mexico (Baja California, Baja California Sur, Chihuahua, Durango (western), Sinaloa, Sonora)	Threatened	2		



Flow vs Rainfall (2015-2020)
WWTP Flow (MGD)
Rainfall (in)
25
20
15
10
5
0
2.5
2
1.5
1
0.5
0
1/1/2015
7/20/2015
2/5/2016
8/23/2016
3/11/2017
9/27/2017
4/15/2018
11/1/2018
5/20/2019
12/6/2019
6/23/2020
Axis Title
WWTP Primary Effluent Flow
Rainfall

St. Helens Soils

Source: USDA Web Soil Survey (WSS)

Columbia County, Oregon (OR009)			
Map Unit Symbol	**Map Unit Name**	**Acres in AOI**	**Percent of AOI**
1A	Aloha silt loam, 0 to 3 percent slopes	738.2	12.50%
1B	Aloha silt loam, 3 to 8 percent slopes	388.9	6.60%
2	Aloha variant silt loam	200.9	3.40%
6D	Bacona silt loam, 3 to 30 percent slopes	27.1	0.50%
10B	Cascade silt loam, 3 to 8 percent slopes	43.2	0.70%
10C	Cascade silt loam, 8 to 15 percent slopes	95.4	1.60%
10D	Cascade silt loam, 15 to 30 percent slopes	46	0.80%
14C	Cornelius silt loam, 8 to 15 percent slopes	114.8	1.90%
14D	Cornelius silt loam, 15 to 30 percent slopes	73.5	1.20%
16	Dayton silt loam	46.3	0.80%
18E	Dowde silt loam, 30 to 60 percent north slopes	22.8	0.40%
19E	Dowde silt loam, 30 to 60 percent south slopes	38.2	0.60%
27B	Latourell silt loam, 3 to 8 percent slopes	12.2	0.20%
31	McBee silt loam	6.6	0.10%

39B	Quafeno loam, 3 to 8 percent slopes	71.5	1.20%
40A	Quatama silt loam, 0 to 3 percent slopes	59.4	1.00%
40B	Quatama silt loam, 3 to 8 percent slopes	272	4.60%
40C	Quatama silt loam, 8 to 15 percent slopes	95.1	1.60%
45	Rock outcrop-Xerumbrepts complex, undulating	2,015.60	34.20%
46	Sauvie silt loam	417.8	7.10%
63	Wapato silt loam	10.9	0.20%
69	Wollent silt loam	404.2	6.90%
70E	Xerochrepts, steep	139	2.40%
71	Xeropsamments, nearly level	56.8	1.00%
W	Water	501.5	8.50%
Totals for Area of Interest		**5,897.80**	**100.00%**

City of St. Helens
Rainfall Event Analysis

Rainfall Events Requested	Peak Day (MGD)	PIF (MGD)	PIF/Peak Day Factor	Rainfall (in)
1/15/2015 - 1/17/2015	11.5	19.3	1.7	1.7
2/5/2015 - 2/7/2015	12.7	14.5	1.1	1.3
12/5/2015 - 12/8/2015	21.2	31.4	1.5	2.2
1/11/2016 - 1/13/2016	13.1	27.4	2.1	1.3
1/16/2017 - 1/18/2017	17.8	24.6	1.4	1.4
2/14/2017 - 2/16/2017	13.9	19.1	1.4	1.3
10/19/2017 - 10/21/2017	7.2	14.1	1.9	1.7
10/25/2018 - 10/27/2018	3.3	5.7	1.7	1.2
2/10/2019 - 2/12/2019	21.9	32.2	1.5	2.2
12/18/2019 - 12/20/2019	11.6	14.2	1.2	1.3
	Average		1.55	

St. Helens WWMP - Anticipated 20-Year Growth, Wet-Weather and Dry Weather Loading Application for Growth Areas

10/5/2021

Population Projection Summary

St. Helens Projected 20-Yr Pop. Growth	3,908
St. Helens Projected 20-Yr EDU Growth	1,569
Columbia City Projected 20-Yr Pop. Growth	203
Columbia City Projected 20-Yr EDU Growth	82
Total System Projected 20-Yr EDU Growth	**1,651**

Notes: 1. See associated figure for allocated growth locations (residential, commercial, and industrial areas shown). EDU = Equivalent Dwelling Unit

Overall System Flow Summary [1]

Existing ADWF (MGD)	**1.11**
Pop. Projected, 20-Year ADWF (MGD) [2]	**1.41**
Anticipated, 20-Year ADWF (MGD) [3]	**1.91**
Residential 20-Year Growth ADWF (MGD)	0.30
Commercial 20-Year Growth ADWF (MGD)	0.03
Industrial 20-Year Growth ADWF (MGD)	0.47

Notes: 1. ADWF = Average Dry Weather Flow

2. Based on PSU projected growth rates.

3. Includes industrial and commercial flows from growth anticipated by the City in the 20-year planning period.

St. Helens - Dry and Wet Weather Loading Application for 20-Year Model

Residential/Commercial Mix

Area Number	Site Name	Acreage	Zoning	Residential Density (assumed)	ROW %	Commercial %	Commercial Area (ac)	Res. EDU count (calculated) [1]	Flow, ADWF (gpd) [2,3]	Flow, ADWF (MGD)	Flow (gpm)	Manhole where DWF load applied	DWF Pattern Applied	Manhole where RDII Hydrograph Applied
1	Residential/Commercial Mix - 15 acres	15	Mixed Use	R5	15%	20%	3	82	18,541	0.019	12.88	PS11/SR1	FM6	SR15/PS11
2	Riverfront District (Mixed Use - 23 acres) [4]	23	Riverfront District	AR	15%	50%	11.5	175	46,247	0.046	32.12	IA9	FM5	IA8
3	Houlton Business District [5]	45	Houlton Business District	N/A	15%	10%	5	0	5,769	0.006	4.01	NI5	FM3	NI4
4	Currently Vacant Commercial Property	5.5	Highway Commercial	N/A	15%	100%	5.5	0	7,013	0.007	4.87	N29	FM1	N28
							Total	**257**	**77,569**	**0.078**				

Notes: 1. From HNA, 2.49 people per EDU assumed. R5 = 8 EDUs/acre, AR (Apartment Residential) = 14 EDUs/acre

2. ADWF = Average Dry Weather Flow

3. Assumed commercial flow rate of 1,500 gallons/acre/day (gpad).

4. Approximately 6 acres designated as mixed use with both commercial and residential flow.

5. The Houlton Business District is already developed, assumed 10% commercial infill.

Residential

Area Number	Site Name	Acreage	Zoning	EDU Count (City Delineated)	ROW %	EDU count (calculated) [1]	Flow, ADWF (gpd) [2]	Flow, ADWF (MGD)	Flow, ADWF (gpm)	Manhole where DWF load applied	DWF Pattern Applied	Manhole where RDII Hydrograph Applied
5	Residential (125 EDUs)	40	R7	125	N/A	125	22,542	0.023	15.7	N38	FM1	N38A
6	Residential (20 EDUs)	7	R7	20	N/A	20	3,607	0.004	2.5	N38	FM1	N38A
7	Residential (60 EDUs)	15	Mobile Home Residential	60	N/A	60	10,820	0.011	7.5	NC18	FM3	NC18
8	Residential (20 acres)	20	R5	N/A	20%	128	23,120	0.023	16.1	WE20	FM2	WE19
9	Residential (64 acres)	64	R7	N/A	20%	307	55,400	0.055	38.5	PS11/SR1	FM2	PS11/SR1
10	Residential (28 acres)	28	R7	N/A	20%	134	24,237	0.024	16.8	WCA3	FM2	WCA3
11	Mobile Home Park (37 acres)	37	Mobile Home Residential	N/A	15%	313	56,475	0.056	39.2	SR17	FM6	SR15
12	Columbia City Growth (203 additional pop.)	N/A	Residential	82	N/A	82	14,702	0.015	10.2	NC18	N/A	N/A
13	Gable Rd. Apartments	11.5	GC (AR)	238	N/A	238	42,920	0.043	29.8	SP5	FM6	SP4A
					Total	**1,407**	**253,824**	**0.254**				

Notes: 1. From HNA, 2.49 people per EDU assumed. R7 = 6 EDUs/acre, R5 = 8 EDUs/acre, Mobile Home Residential = 10 EDUs/acre, AR (Apartment Residential) = 14 EDUs/acre. Wetlands were excluded in area delineation.

2. ADWF = Average Dry Weather Flow

Industrial/Commercial

Area Number	Site Name	Acreage	Zoning	Acres Developed	ROW %	Flow, ADWF (gpd) [2,3]	Flow, ADWF (MGD)	Flow, ADWF (gpm)	Manhole where DWF load applied	DWF Pattern Applied	Manhole where RDII Hydrograph Applied
14	Industrial Site	27	Heavy Industrial	27	15%	34,959	0.035	24.3	SP5	INDUSTRY	SP4A
15	Multnomah Industrial Park [1]	98	Heavy Industrial	30	15%	38,250	0.038	26.6	S37A	INDUSTRY	S37A
16	Old Armstrong Site	124	Heavy Industrial	124	15%	157,588	0.158	109.4	S29	INDUSTRY	S28
17	Industrial Business Park	190	Heavy Industrial	190	15%	242,250	0.242	168.2	S20	INDUSTRY	S20
			Total	**371**	**Total**	**473,047**	**0.47**				

Notes: 1. City anticipates approximately 20-30 acres of this property to develop.

2. ADWF = Average Dry Weather Flow

3. Assumed medium/light industrial flow rate of 1,500 gallons/acre/day (gpad).

SUBMITTED TO:
Keller Associates
245 Commercial St SE,
Suite 210
Salem, Oregon, 97301

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GEOTECHNICAL PLANNING REPORT

St. Helens Wastewater and Stormwater Master Plan Update

ST. HELENS, OREGON

SHANNON & WILSON

November 2021

Shannon & Wilson No: 104961

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Submitted To: Keller Associates
245 Commercial St SE,
Suite 210
Salem, Oregon, 97301
Attn: Peter Olsen, PE

Subject: GEOTECHNICAL PLANNING REPORT, ST. HELENS WASTEWATER AND STORMWATER MASTER PLAN UPDATE, ST. HELENS, OREGON

Shannon & Wilson prepared this report and participated in this project as a subconsultant to Keller Associates. Our scope of services was specified in our contracted dated March 18, 2021 for Keller project number 220060. This report presents the geotechnical planning-related findings based on a review of publicly available documents and was prepared by the undersigned.

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or if we may be of further service, please contact us.

Sincerely,

SHANNON & WILSON, INC.

REGISTERED PROFESSIONAL ENGINEER
77330PE
OREGON
Jan. 10, 2006
ELLIOTT CARL MECHAM
EXPIRES: 12/31/2022

Elliott Mecham, PE
Senior Associate

DSJ:ECM:JLJ/:myw

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1 GENERAL

The City of St. Helens provides sanitary sewer collection services to businesses and residences within the City limits. The sanitary sewer collection system is a combination of 60 miles of gravity and force mains, 9 lift stations, and over 1,700 sanitary sewer manholes, vaults, and cleanouts. All sewage flows are conveyed to the City's wastewater treatment facility. The last complete update to the City's sanitary sewer master plan was in 1989.

The intent of the sanitary sewer master plan is to perform an assessment of the existing sewer system; evaluate the sewer system for its capacity to convey existing and future waste discharges; identify deficiencies, capacity issues, areas for improvement, and identify resiliency issues for critical facilities; and determine and propose solutions.

2 SCOPE OF SERVICES

The purpose of Shannon & Wilson’s task is to prepare and provide GIS maps of the service area with the mapped site geology and the State of Oregon Department of Geology and Mineral Industries' (DOGAMI) mapped seismic hazards, and document the findings in a brief report. The backbone wastewater and stormwater facilities selected and digitized into GIS format by others will be shown on the maps. Our specific scope of work includes the following:

- Mapped site geology;
- Mapped landslides included in DOGAMI’s landslide inventory (if any) along the proposed pipeline alignments or at the treatment plant sites;
- Mapped United States Geology Survey (USGS) Class A or Class B faults that cross pipeline alignments or are located within a 5-mile radius of treatment plant locations;
- Mapped relative earthquake liquefaction hazard based on DOGAMI maps (high, medium, or low hazard);
- Mapped relative landslide risk based on DOGAMI maps (very high, high, moderate, or low hazard); and
- Submitting a brief memo or letter report presenting the geologic maps and a brief discussion summarizing our findings, including a discussion on probable areas where rock excavation could be required, and the potential need to mitigate seismic hazards. The discussions will be limited by the uncertainties and assumptions made during the development of the geologic maps and DOGAMI hazard layers.

3 DESCRIPTION OF PROVIDED MAPS

3.1 Provided Data

Shannon & Wilson was provided GIS files for the City of St. Helens stormwater and wastewater facilities. An overview map of these facilities can be found on Figure 2, Site Plan. Within the files provided were attributes which allowed for the identification of vulnerable assets. The vulnerable pipelines can be found on Figure 3, Pipeline Vulnerabilities.

3.2 Available Mapping

DOGAMI has developed several publications which were used in our assessments related to the stormwater and wastewater facilities. These included site geology, landslide hazard, and peak ground accelerations associated with a Cascadia Subduction Zone earthquake. Datasets of interest for this project include the following:

- Geology: Oregon Geologic Data Compilation release 6 (OGDC-6);
- Landslide Hazard: DOGAMI Open-File Report O-16-02; and
- Cascadia Peak Ground Accelerations: DOGAMI Open-File Report O-13-06.

3.3 Geology

The City of St. Helens is at the northern end of the Portland Basin, a structural depression created by complex folding and faulting of the basement rocks. The most prevalent basement rock of the Portland Basin is a sequence of lava flows called the Columbia River Basalt Group (CRBG), which flowed into the area between about 17 million and 6 million years ago (Beeson and others, 1991). Due to the wet and mild climate of the Pacific Northwest, intense chemical weathering of the geologic units has taken place (Evarts, 2004). This has resulted in the development of soil horizons as thick as 10 m. In some instances, the rocks of the CRBG have been completely converted to soil, destroying all primary rock textures.

The Columbia and Willamette Rivers converge within the Portland Basin and, with their tributaries, have contributed to an extensive sedimentary fill which overlies the basement rock formations. Beeson and others (1991) mapped the local Portland Basin fill sediments as Sandy River Mudstone, overlain by Troutdale Formation. The Troutdale Formation locally consists of well-consolidated friable to moderately well-cemented conglomerate and sandstone, deposited in the Miocene to Pliocene epochs (about 12.5 million to 1.6 million years ago).

The Troutdale Formation is locally overlain by sediments deposited during a series of catastrophic glacial outburst floods. During the late stages of the last great ice age, between about 18,000 and 15,000 years ago, a lobe of the continental ice sheet repeatedly blocked and dammed the Clark Fork River in western Montana, which then formed an immense glacial lake called Lake Missoula. The lake grew until its depth was sufficient to buoyantly lift and rupture the ice dam, which allowed the entire massive lake to empty catastrophically. Once the lake had emptied, the ice sheet again gradually dammed the Clark Fork Valley and the lake refilled, leading to 40 or more repetitive outburst floods at intervals of decades (Allen and others, 2009). During each short-lived episode, floodwaters washed across the Idaho panhandle, through the eastern Washington scablands, and through the Columbia River Gorge. When the floodwater emerged from the western end of the gorge, it spread out over the Portland Basin and up the Willamette Valley as far south as Junction City, depositing a tremendous load of sediment (O'Conner and others, 2001).

The geologic map presented on Figure 4 comes directly from the Oregon Geologic Data Compilation release 6 (OGDC-6).

3.3.1 Regional Seismological Setting

Earthquakes in the Pacific Northwest occur largely as a result of the subduction of the Juan de Fuca plate beneath the North American plate along the Cascadia Subduction Zone (CSZ). The CSZ is located approximately parallel to the coastline from northern California to southern British Columbia. The compressional forces that exist between these two colliding plates cause the oceanic Juan de Fuca plate to descend, or subduct, beneath the continental plate at a rate of about 1.5-inches per year (DeMets and others, 1990). This process leads to volcanism in the North American plate and stresses and faulting in both plates throughout much of the western regions of southern British Columbia, Washington, Oregon, and northern California. Stress between the colliding plates is periodically relieved through great earthquakes at the CSZ plate interface.

Within the regional tectonic framework and historical seismicity, three broad earthquake sources are identified:

- Subduction Zone Interface Earthquakes originate along the CSZ, which is located 25 miles beneath the coastline. Paleoseismic evidence and historic tsunami records from Japan indicate that the most recent subduction zone interface event was in 1700 AD and was an approximately magnitude 9 earthquake that likely ruptured the full length of the CSZ.
- Deep-Focus, Intraplate Earthquakes originate from within the subducting Juan de Fuca oceanic plate as a result of the downward bending and tension in the subducted plate. These earthquakes typically occur 28 to 38 miles beneath the surface. Such events on the

CSZ are estimated to be as large as magnitude 7.5. Historic earthquakes include the 1949 magnitude 7.1 Olympia earthquake, the 1965 magnitude 6.5 earthquake between Tacoma and Seattle, and the magnitude 6.8 2001 Nisqually earthquake. The highest rate of CSZ intraslab activity is beneath the Puget Sound area, with much lower rates observed beneath western Oregon.

- Shallow-Focus Crustal Earthquakes are typically located within the upper 12 miles of the earth's surface. The relative plate movements along the CSZ cause not only east-west compressive strain but dextral shear, clockwise rotation, and north-south compression of the leading edge of the North American Plate (Wells and others, 1998), which is the cause of much of the shallow crustal seismicity of engineering significance in the region. The largest known crustal earthquake in the Pacific Northwest is the 1872 North Cascades earthquake with an estimated magnitude of about 7. Other examples include the 1993 magnitude 5.6 Scotts Mill earthquake and magnitudes 5.9 and 6.0 Klamath Falls earthquakes. According to the USGS Quaternary Fault and Fold database (USGS, 2021), there are no Class A features within approximately 5 miles of the project site.

3.4 Liquefaction Hazard

The statewide liquefaction map of the state is a compilation of liquefaction susceptibility maps from other DOGAMI publications. Within the St. Helens area, this is IMS-7 (Madin and Wang, 1999). While this is a purpose-made liquefaction hazard map for the area, it was based primarily on aerial photo interpretation, geologic mapping from 1946, and water well data. Since the development of IMS-7, new geologic mapping was conducted (Evarts, 2004). In order to allow for a liquefaction hazard map based on the updated geologic mapping, we employed the Youd and Perkins 1978 methodology to convert the mapped geology to liquefaction susceptibility. The resulting map can be seen on Figure 5.

3.5 Landslide Hazard

The landslide hazard map presented on Figure 6 comes from the DOGAMI Open-File Report O-16-02. This overview map encompasses the entire state of Oregon and was designed to be used for regional planning. Susceptibility categories are broken into four categories (low, moderate, high, and very high), where very high denotes areas of mapped landslides.

The relative landslide hazard risk was developed by DOGAMI by creating a generalized geology-landslide intersect map and a percent slope map. Spatial statistics were then used to determine the mean and standard deviation of slope angles within landslides per geologic unit. Thirty percent of the area within the statewide hazard map consists of High or Very High hazard slopes and 80 percent of the landslides are located within this area.

Limitations of the input and modeling mean that the map should only be used for general planning purposes, and the map cannot be used as a substitute for geotechnical explorations, laboratory testing, and detailed site-specific analyses.

4 SUMMARY OF FINDINGS

The majority of the pipelines in need of replacement are located in areas mapped as rock. However, pipeline assets on the western portion of the basin are also mapped in Missoula Flood Deposits with small areas of alluvium. Assets within approximately 500 to 600 feet of the Willamette River pipeline, are located in recent alluvium and fill. The primary geologic hazard in the areas mapped as rock is strong ground motions.

Potential seismic hazards outside of the areas mapped as rock are expected to be related to liquefaction, and liquefaction-related phenomena such as settlement, lateral spreading, and post-seismic soil strength reduction. The risk of other seismic hazards, such as fault rupture, is low within the study area. Additionally, the potential need for rock excavation will be discussed in the following sections.

4.1 Landslides

According to the Department of Geology and Mineral Industries (DOGAMI), the existing pipelines are located within zones of low to high landslide hazard. While none of the mapped facilities are located within a mapped landslide, select stormwater facilities at the northernmost extent of the project area are adjacent to areas of very high landslide hazard indicating there are existing landslides.

4.2 Liquefaction and Lateral Spread

Soil liquefaction occurs in susceptible subsurface soils below the groundwater level. It is a phenomenon in which excess pore water pressure of loose to medium dense, saturated, granular soils increases during ground shaking to a level near the initial effective stress. The increased excess pore pressure results in a reduction of soil shear strength. Given that sands were observed at the ground surface and likely underlie a large portion of the project area, liquefaction is a potential hazard within the project area. A map of liquefaction susceptibility prepared using the Oregon Geologic Data Compilation release 6 (OGDC-6) and the Youd and Perkins, 1978 methodology, and included as Figure 5, indicates that much of the project area has no liquefaction hazard as the area is mapped as rock. However, select pipelines at the westernmost extent of the project area and on the eastern outfalls have moderate to high liquefaction risks. Again, the effects of liquefaction typically include lateral spreading, slope instability, ground settlement, and strength reductions, such as lower allowable soil bearing.

We note that this hazard assessment is based solely on soil type and does not consider ground water presence or the absence of groundwater. If groundwater is not present at the site, the DOGAMI hazard map is likely overestimating the liquefaction potential. The relative density also impacts the liquefaction potential of the sands. Obtaining site specific borings or Cone Penetrometer Tests (CPTs) and laboratory tests on collected soil samples to assess the density of the sand was outside the scope of this study, but we recommend that they be performed during design to further assess the extent of the liquefaction hazard.

Lateral spreading hazards can exist in areas with mild slopes adjacent to a much steeper slope or vertical face. Lateral spreading failure can occur if soil liquefaction develops during a seismic event and the ground acceleration (inertial force) briefly surpasses the yield acceleration (shear strength) of the liquefied soil. This can cause both the liquefied soil and an overlying non-liquefied crust of soil to displace laterally down mild slopes towards an embankment face, or the banks of streams, rivers, and other bodies of water. The displacements are cumulative and permanent in nature. If liquefaction occurs there is risk of post seismic slope instability and potential lateral displacement towards the existing slope to the northeast.

4.2.1 Liquefaction Induced Post-Seismic Settlement

Settlement will likely occur in cohesionless soil below the groundwater table that undergo liquefaction and pore pressure development during ground shaking. The settlement is related to densification and rearrangement of particles during ground shaking, as well as volume change, as the excess pore pressure dissipates after ground shaking. Seismic ground settlement does not typically occur uniformly over an area, and differential settlement may impact existing or proposed structures and infrastructure supported by liquefied soil and/or within the liquified zones. Differential settlement is often estimated to range between 50 and 80 percent of the total settlement. Consequences of seismic-induced settlement would be subsequent settlement of shallow foundations overlying the liquefied soil.

4.2.2 Fault Rupture

Quaternary crustal faults and folds throughout Oregon and Washington have been located and characterized by the United States Geological Survey (USGS). The USGS provides approximate fault locations and a detailed summary of available fault information in the USGS Quaternary Fault and Fold Database. The database defines four categories of faults, Class A through D, based on evidence of tectonic movement known or presumed to be associated with large earthquakes during Quaternary time (within the last 2.58 million years). For Class A faults, geologic evidence demonstrates that a tectonic fault exists and that it has likely been active within the Quaternary period. For Class B faults, there is equivocal geologic evidence of Quaternary tectonic deformation, or the fault may not extend deep enough to be considered a source of significant earthquakes. Class C and D faults lack convincing geologic evidence of Quaternary tectonic deformation or have been studied carefully enough to determine that they are not likely to generate significant earthquakes.

The closest Class A or Class B fault to the site is the Portland Hills Fault, mapped more than 5 miles from the project location, and is shown on the Fault Vicinity Map, Figure 7. In our opinion the risk of fault rupture at the site is low.

4.3 Rock Excavation

Rock excavation may be necessary where buried improvements are located outside or deeper than the existing utility trenches that are planned in areas mapped as rock. In the past, the City of St. Helen's has successfully used pipe bursting. However, the effectiveness and ease of pipe bursting has been a function of the existing trench width, pipe upsize, and depth of cover. We understand the City does not recommend pipe bursting for any pipes with less than 5-6 feet of cover. The City's historical experience with pipe bursting has been successful for increases of 1 to 2 pipe size diameters. The City has also reported successfully using Horizontal Directional Drilling (HDD) in solid basalt rock at depths over 16 feet below ground surface.

Pipe bursting to replace existing pipe where sewer lines are constructed over the top of shallow rock may not be feasible if adequate cover is not present. Additionally, rock or decomposed rock is relatively incompressible. If pipe bursting is performed in areas where pipes are buried in rock, any change in the density of the material surrounding the pipe that is required for upsizing will need to occur within the trench backfill. As was presented in Figure 4, Geologic Map, the majority of city assets are constructed within areas mapped as basalt. Where pipe bursting is considered as a possible remediation or where new sewers will be constructed outside of the existing trench, a review of as-built construction information, historic geotechnical information, or new geotechnical explorations should be considered to identify and mitigate the potential risk of rock related constructability issues in areas mapped as rock.

5 LIMITATIONS

This letter report was prepared for the exclusive use of the Keller and the City of St. Helens and their representatives for the purpose of planning-related geotechnical site evaluation for wastewater facilities. The assessments contained in this letter are based on the information and data provided to us, and information that is publicly available. This letter report should not be viewed as a warranty of conditions described in this report, such as those interpreted from published maps. The maps should be used for planning level purposes only and not a substitute for geotechnical explorations and laboratory testing that will be required for design. Our findings are based on the limitations of our approved scope, schedule, and budget; and our understanding of the project and information provided by Keller Associates.

For any site located on or near a slope, there are slope instability risks that are present and future owners have to accept, including, but not limited to:

- Natural factors: soil and groundwater conditions, steep topography, heavy rainfall events, erosion, and vegetation conditions; and
- Human-related factors: water leaks, pipe breaks, improper drainage, lack of maintenance of vegetation or drainage facilities, fill or debris placement, excavation and/or removal of trees/vegetation.

Similar circumstances or other unknown conditions may also affect slope stability. Our evaluation and planning level assessments described herein are not a guarantee or warranty of slope stability conditions, nor current and future risks.

Please note that our scope of services did not include any environmental assessment or evaluation regarding the presence or absence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below the site.

Shannon & Wilson has prepared the attached, "Important Information About Your Geotechnical/Environmental Report," to assist you and others in understanding the use and limitations of our reports.

6 REFERENCES

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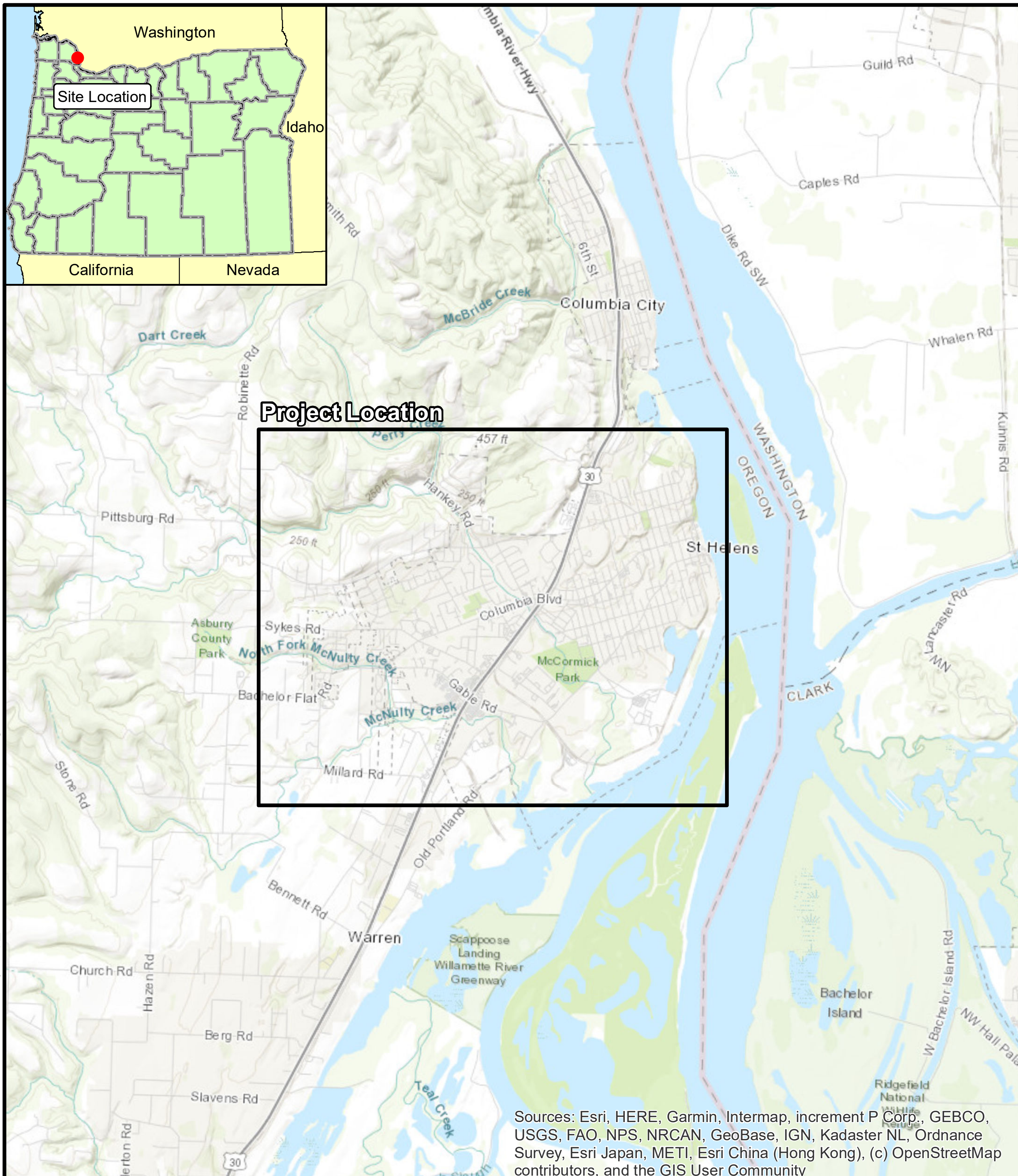

0 0.5 1 2

Scale in Miles

N

St. Helens' Wastewater and Stormwater
Master Plan Update
St. Helens, Oregon

VICINITY MAP

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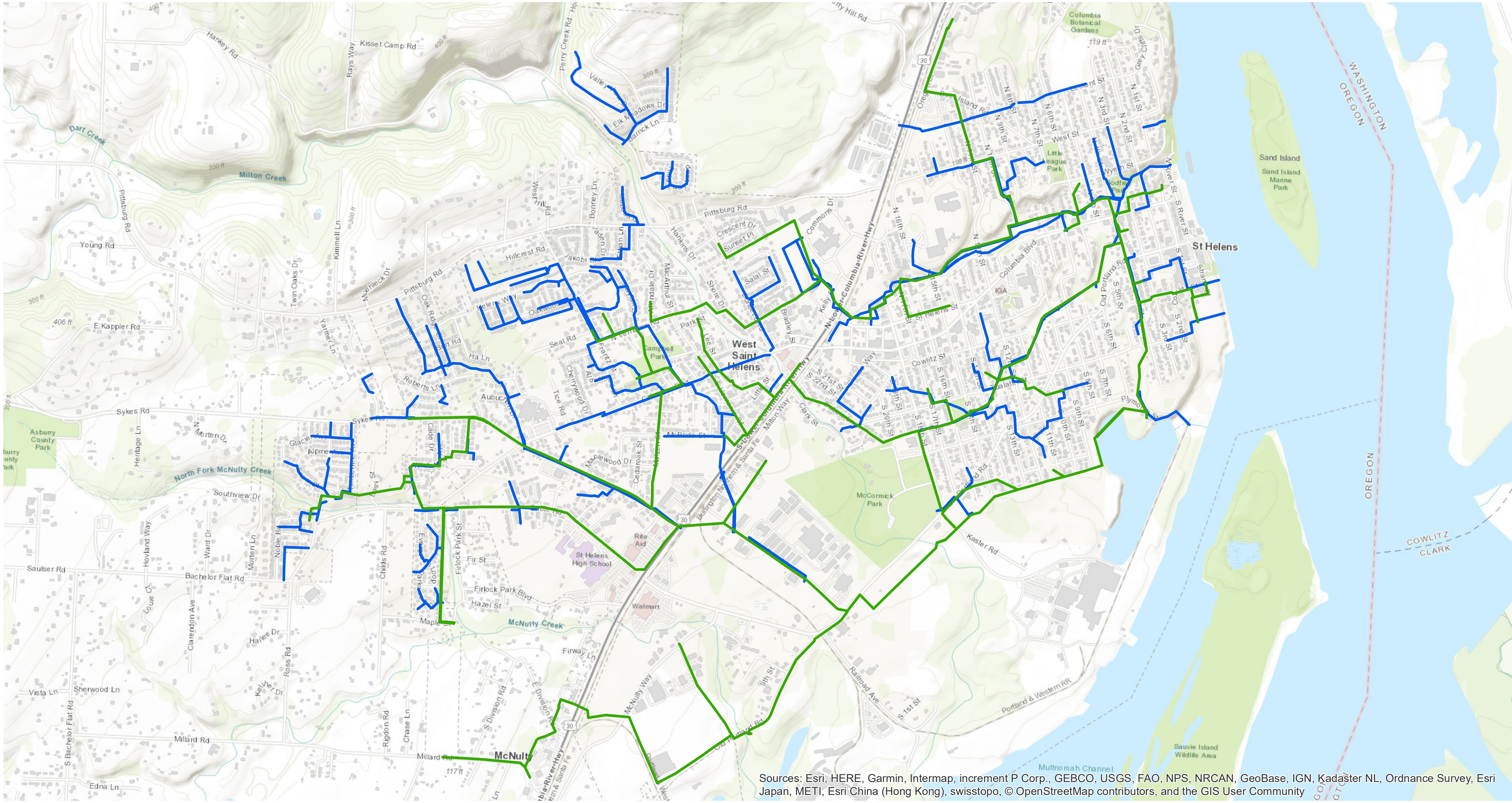
FIG. 1

Filename: I:\EF\PDX\104000s\104961 St. Helen's Sto\GIS Files\2015 Vicinity Map_OR.mxd Date: 6/28/2021 Login: ECM



LEGEND

Sewer Pipelines

Stormwater Conduits

0 800 1,600 3,200

Scale in Feet

NOTES

1. Stormwater and sewer assets provided by Keller Associates on June 14, 2021.

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SITE PLAN

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FIG. 2

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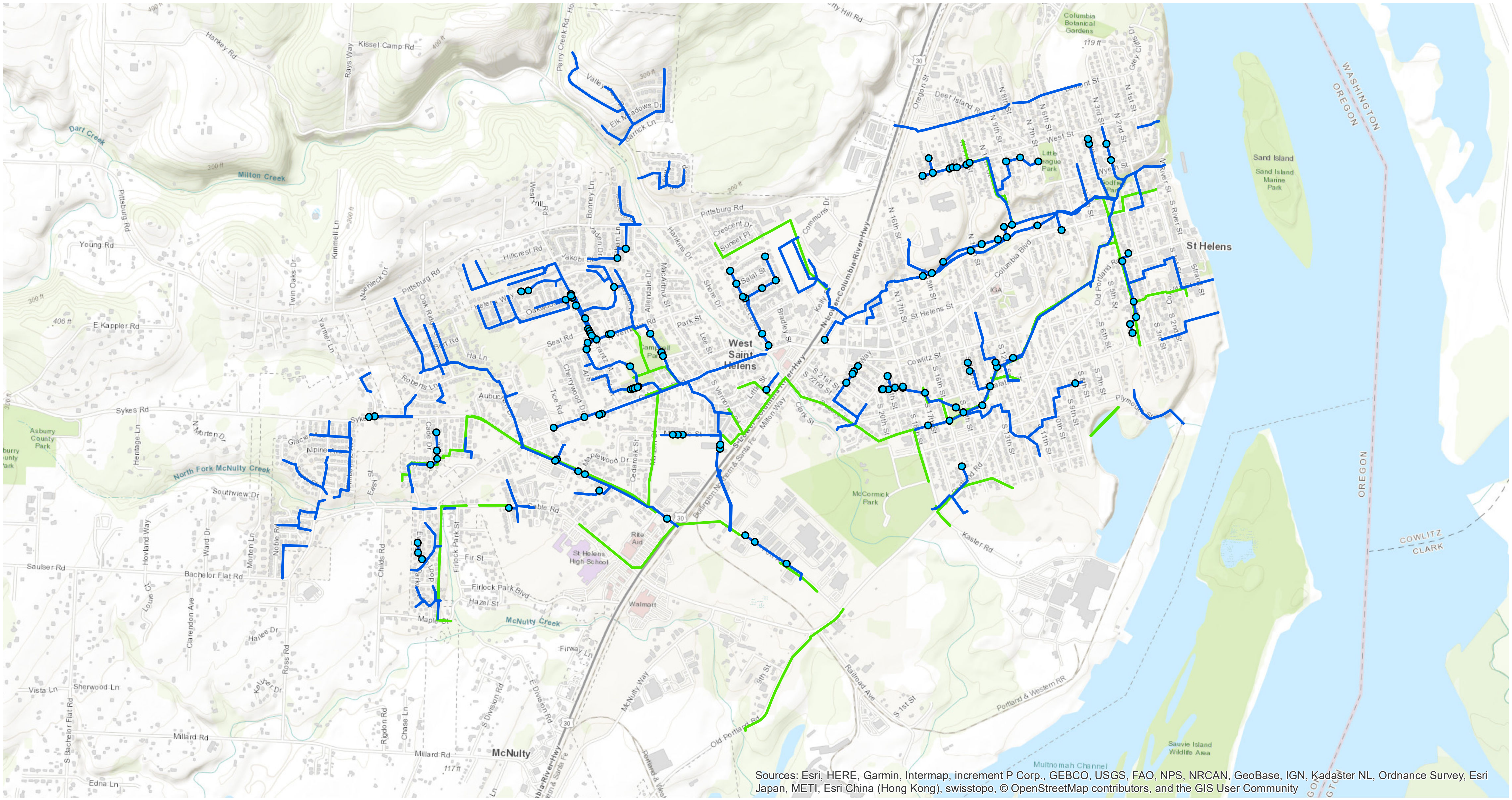
Filename: T:\Projects\PDX\104000s\104961 - St Helens\Pipeline Vulnerabilities.mxd   Date: 6/30/2021   Login: DSJ



LEGEND

- Stormwater Junctions Indicating Need of Improvement
- Sewers in need of Improvement
- Stormwater Conduits

0 800 1,600 3,200

Scale in Feet

NOTES

1. Stormwater and sewer assets provided by Keller Associates on June 14, 2021.

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PIPELINE VULNERABILITIES

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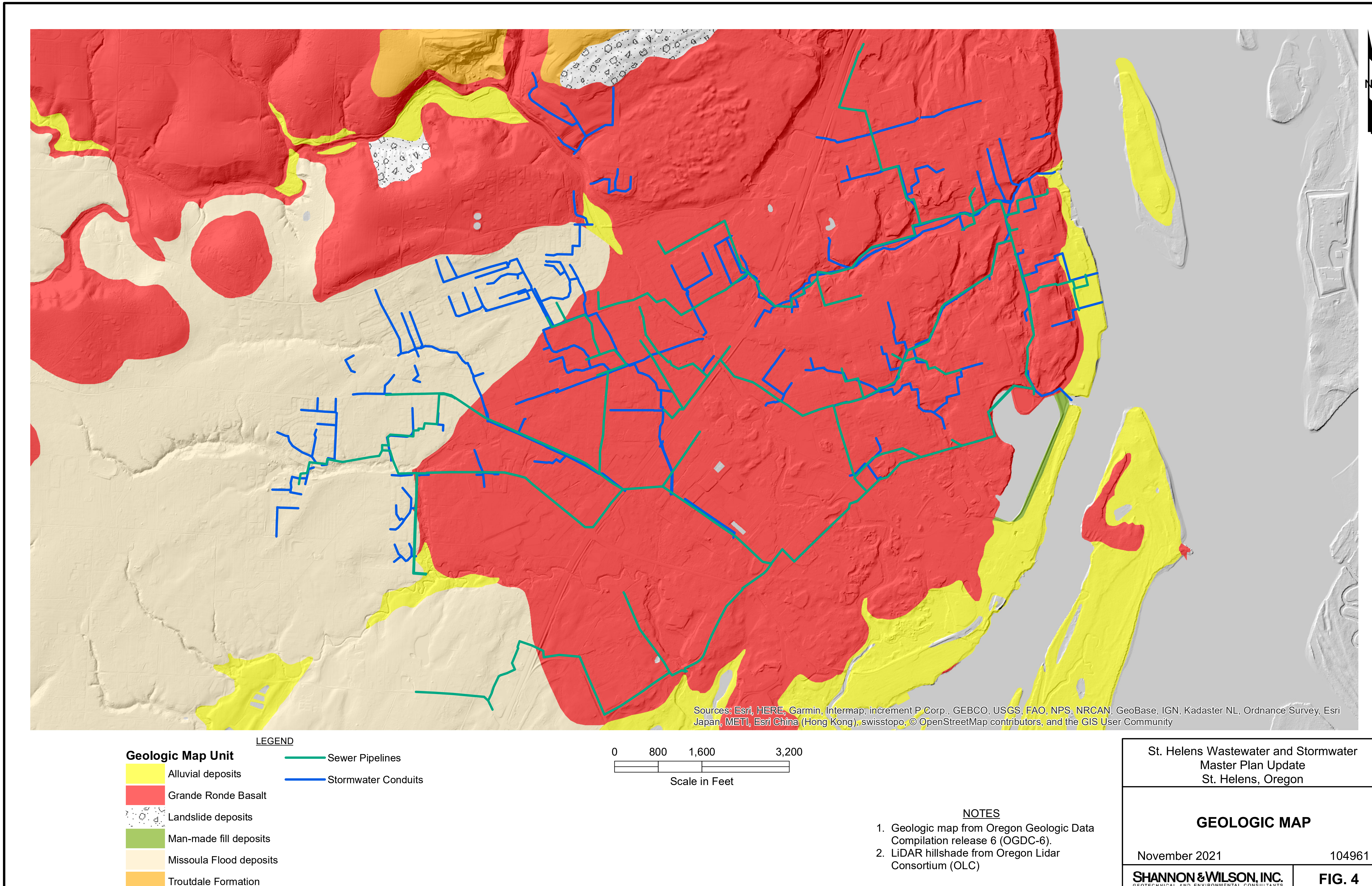
FIG. 3



NOTES
1. Geologic map from Oregon Geologic Data Compilation release 6 (OGDC-6).
2. LiDAR hillshade from Oregon Lidar Consortium (OLC)

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GEOLOGIC MAP

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FIG. 4

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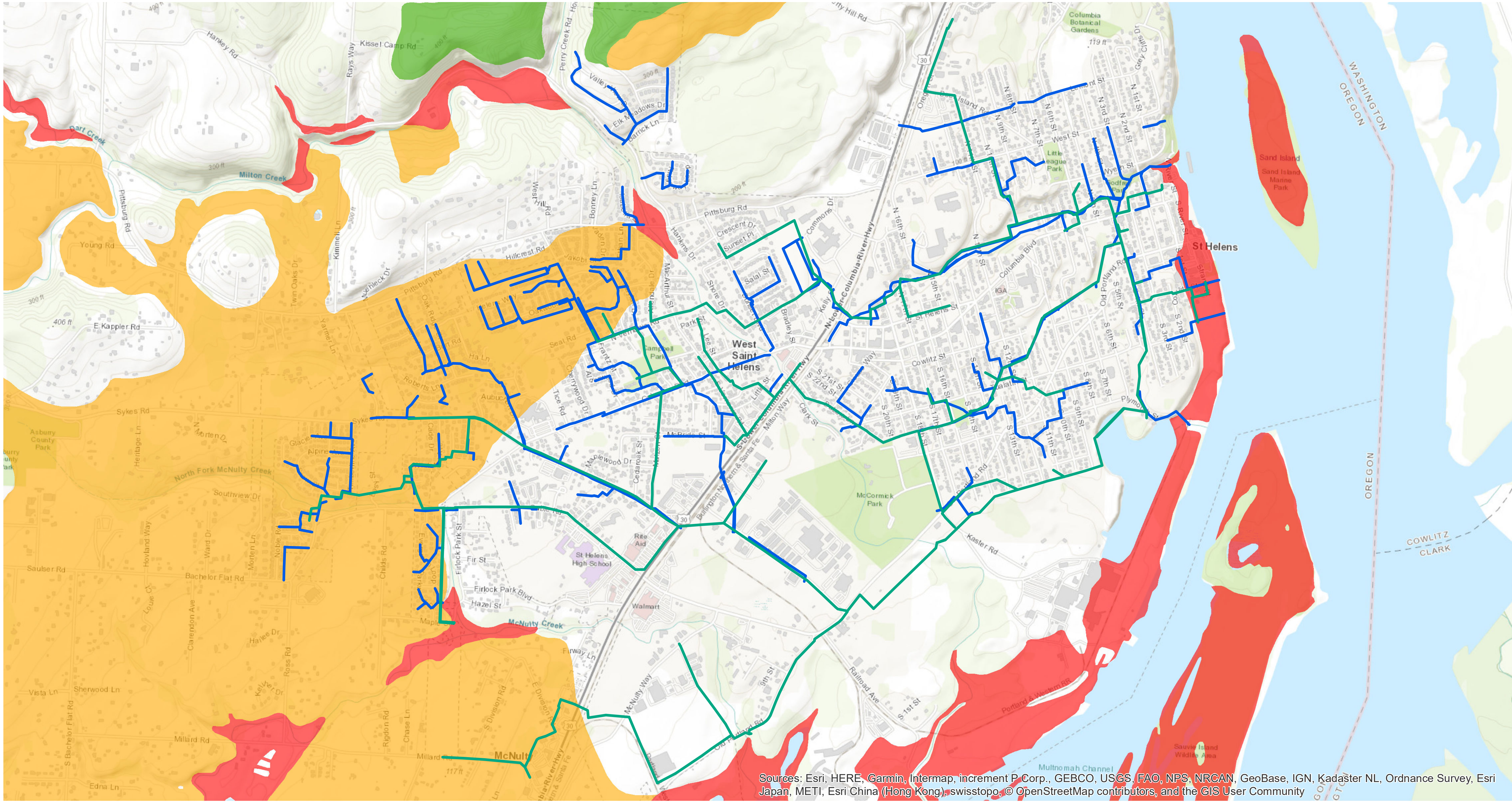

LEGEND

Liquefaction Hazard

- Low
- Moderate
- High

- Sewer Pipelines
- Stormwater Conduits

0 800 1,600 3,200

Scale in Feet

NOTES

1. Liquefaction hazard developed from data provided with OGDC-6, the Youd and Perkins, 1978 metodology, and knowledge of regional liquefaction hazards.
2. Stormwater and sewer assets provided by Keller Associates on June 14, 2021.

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LIQUEFACTION HAZARD

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FIG. 5

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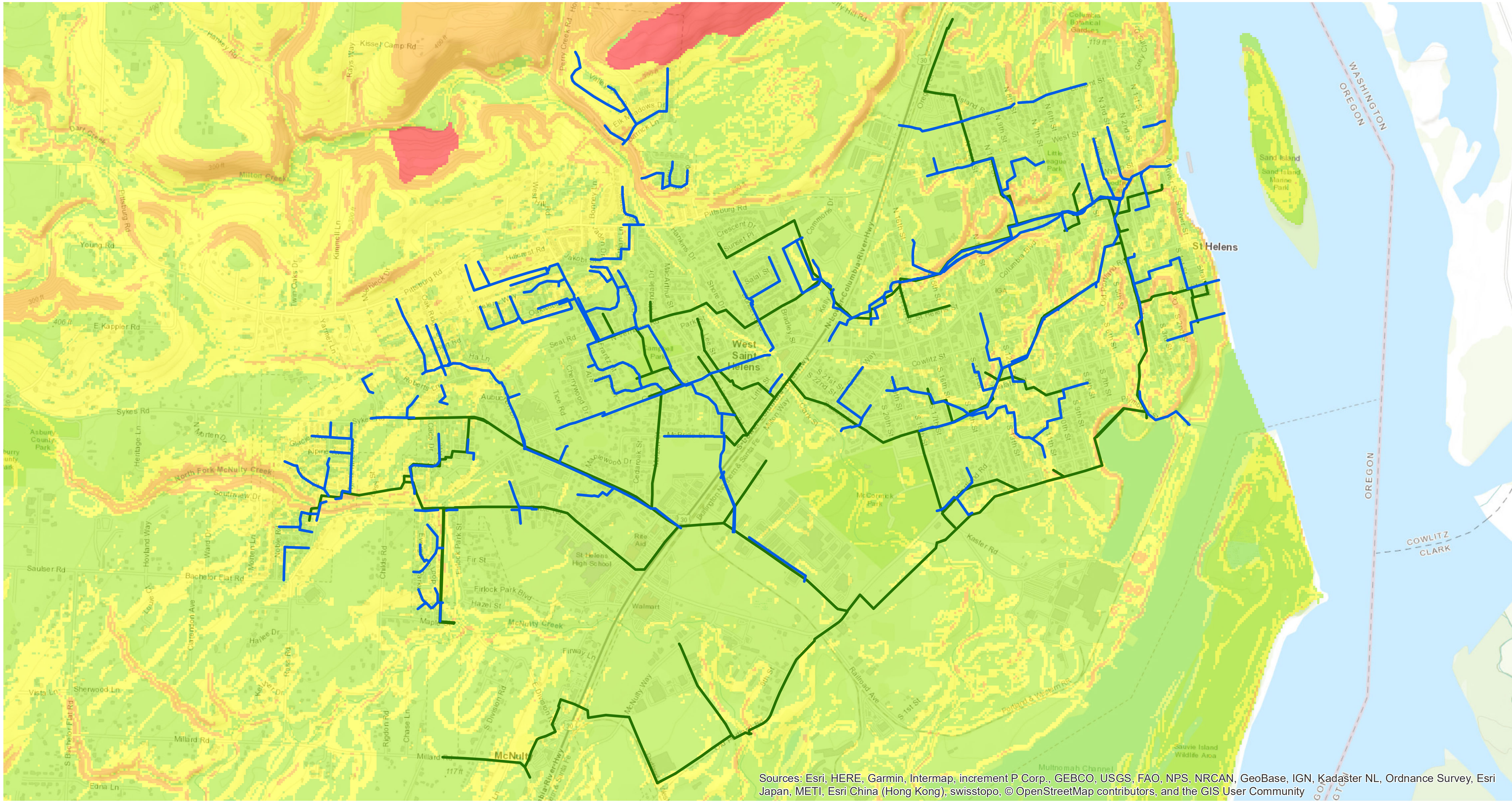

LEGEND

Landslide Susceptibility

- Low
- Moderate
- High
- Very High

- Stormwater Conduits
- Sewer Pipelines

0 800 1,600 3,200

Scale in Feet

NOTES

1. Stormwater and sewer assets provided by Keller Associates on June 14, 2021.
2. Landslide susceptibility from DOGAMI Open-File Report O-16-02.

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LANDSLIDE SUSCEPTIBILITY

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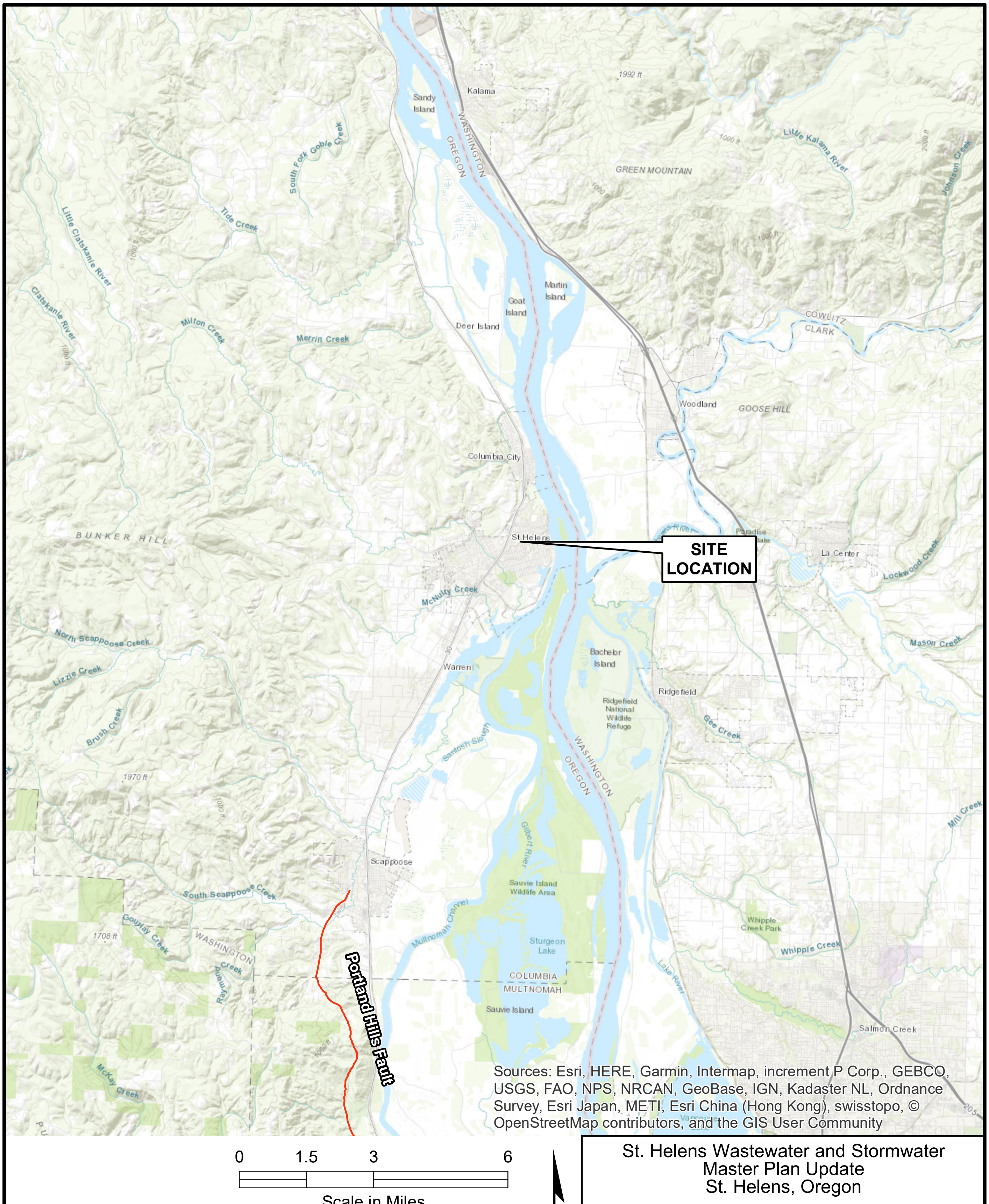
FIG. 6

Filename: T:\Projects\PDX\104000s\104961 - St Helens\Landslide Susceptibility.mxd Date: 6/30/2021 Login: DSJ



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FAULT MAP

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FIG. 7

Filename: T:\Projects\PDX\104000s\104961 - St Helens\Fault Map.mxd Date: 6/30/2021 Login: DSJ

ATTACHMENT A

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

SHANNON & WILSON
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Attachment to and part of Report: 104961
Date: November 2021
To: Peter Olsen
Keller Associates

Important Information About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors that were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the GBA, Silver Spring, Maryland

APPENDIX C

Engineering Standard and Comp Plan Review Tech Memo

KELLER ASSOCIATES

TECHNICAL MEMORANDUM

TO: City of St. Helens

FROM: Peter Olsen, PE
Emily Flock, PE

DATE: 09/13/2021

SUBJECT: **ST. HELENS MUNICIPAL DEVELOPMENT CODE, ENGINEERING STANDARDS MANUAL, AND COMPREHENSIVE PLAN REVIEW – SANITARY SEWER**

1. GENERAL

The City of St. Helens' existing engineering design standards (Title 18), development code (Title 17), and comprehensive plan (Title 19) were reviewed for new development as they pertain to sanitary sewer conveyance and treatment to identify potential deficiencies and provide recommendations for updates. This effort was part of the Wastewater Master Plan (WWMP) process. Sanitary sewer system design criteria encompass the fundamental principles applied in evaluating the existing system and planning for future expansion of the system. The criteria applied in the WWMP come from sources such as neighboring communities, industry standards, and state and federal storm water regulations and are summarized in Section 2 of the WWMP. The aim of the criteria is to accurately define the system demands to mitigate existing deficiencies and prevent future problems. Design criteria addresses design flows, pipeline alignment and geometry, and hydraulic calculation methods.

The following documents were examined during this review effort.

- St. Helens Municipal Code (SHMC) Title 17 Community Development Code
- St. Helens Municipal Code (SHMC) Title 18 Engineering Standards Manual
- St. Helens Municipal Code (SHMC) Title 19 Comprehensive Plan

Note that the recommendations below do not include legal services. Developing draft language and development details for revisions to the Municipal comprehensive plan, development code, and City standards is not included in the scope of this review. Any language provided in this section is intended to assist the City in revising standards and is not intended to be directly incorporated into any City Municipal Code.

2. COMMUNITY DEVELOPMENT CODE

This section discusses the results of reviewing SHMC Title 17 Community Development Code.

2.1 GENERAL AND LAND USE DEFINITIONS (17.16.010)

Title 17 of the SHMC defines specific infrastructure as "Public Facility, Minor" with all undefined infrastructure being a "Public Facility, Major." It is recommended that sanitary sewer force mains and pump stations be excluded from the list of minor public facilities. Additionally, the City should refer to Section 3.10.2 for a list of facilities that are recommended to require special review and approval.

2.2 SANITARY SEWERS (17.152.090)

It is recommended that the City of St. Helens' include a provision at the end of 17.152.090 (2). The provision should require that all sanitary sewers be designed and constructed to meet the requirements of St. Helens Municipal Code Title 18 Engineering Standards Manual.

2.2.1 Oversizing (3)

Title 17 of the SHMC requires that proposed sewer systems consider additional development within the area as projected by the St. Helens comprehensive plan. It is recommended that the City include a reference to the current St. Helens Wastewater Master Plan in this section.

3. ENGINEERING STANDARDS MANUAL

This section discusses the results of reviewing St. Helens Municipal Code Title 18 Engineering Standards Manual.

3.1 SCHEDULING (18.24.010)

The scheduling section of St. Helens' Engineering Standards Manual recommends temporary diverting flow around a new structure "by installing a section of temporary pipe and 45-degree bends around the new manhole and backfilling until testing is completed to the City's satisfaction." It is recommended that the City remove this recommendation and replace it with "the design of wastewater diversion piping and/or bypass pumping shall be the responsibility of the Contractor subject to City approval."

3.2 INTERFERENCES AND OBSTRUCTIONS (18.24.030)

This section adequately defines precautions construction crews should take to retain and protect existing underground utilities during construction. It is recommended that the City use this section to define separation requirements between overhead utilities and the construction equipment or materials. The following separation between equipment and powerlines are required by the Occupational Safety and Health Administration (OSHA):

- < 50 kV line: 10 feet
- 50 – 200 kV line: 15 feet
- 200 – 350 kV line: 20 feet
- 350 – 500 kV line: 25 feet
- 500 – 759 kV line: 35 feet

3.3 PERMANENT SURVEY MONUMENTS (18.24.040)

For additional clarity, it is recommended that the City add a reference to Oregon Revised Statutes (ORS) 209.150 Removal or Destruction of Survey Monument.

3.4 MATERIALS (18.24.050)

The beginning of SHMC Title 18 Engineer Standards Manual states that all sewers shall be designed and constructed to conform to the requirements of the Oregon Department of Environmental Quality (DEQ), the American Public Works Association (APWA), and the City of St. Helens. It is recommended that the City use section 18.24.050 to direct the reader directly to the applicable APWA material specifications. These can be found in ODOT/APWA (Oregon Standard Specifications for Construction (OSSC)). Section 00405 contains specifications for trench excavation, bedding, and backfill.

3.5 GENERAL (18.24.080)

Similar to the recommendations made in the section above, It is recommended that the City add a reference to ODOT/APWA Specifications (OSSC), Section 00405.

3.6 SEWAGE FLOWS (18.24.100)

Requiring sewer facilities to be constructed for conveyance of projected peak flows is an important part of ensuring the City is prepared to handle future flows influenced by inflow and infiltration (I/I). In western Oregon, wastewater design flows are typically calculated in accordance with the DEQ document titled "*Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon: MMDWF, MMWWF, PDAF, and PIF*". These design flows serve as the basis for sizing collection, conveyance, and treatment facilities. The most recently adopted Wastewater Master Plan should provide the following design flows:

- Average Annual Daily Flow (AADF) – The average annual daily flow for the entire year
- Average Dry-Weather Flow (ADWF) – The average daily flow for the period of May 1 through October 31
- Average Wet-Weather Flow (AWWF) – The average daily flow for the period of November 1 through December 31
- Maximum Monthly Dry-Weather Flow ($MMDWF_{10}$) – The flows during the month with the highest flow during the summer months
- Maximum Monthly Wet-Weather Flow ($MMWF_5$) – The flows during the month with the highest flow during the winter months
- Peak Week Flow (PWkF) – The maximum of the average 7-day flow
- Peak Daily Average Flow ($PDAF_5$) – The peak daily average flow during a 5-year storm event
- Peak Instantaneous Flow (PIF_5) – The peak instantaneous flow recorded at the wastewater treatment plant (WWTP)

It is recommended that hydraulic calculations be performed to ensure that pipe size is adequate for conveying PIF_5 flows at full development of the drainage basin in accordance with the current adopted Wastewater Master Plan including all applicable amendments and updates. At the time of this technical memorandum, in accordance with the draft Wastewater Master Plan, pipe size should be adequate for conveying PIF_5 at full development of the basin with pipe flow no more than 85% full depth (d/D). Capacity shall be based on Manning's Equation with "n" = 0.013. This can be noted in SHMC Title 18, Section 18.24.100, which pertains to sewage flows.

3.7 PIPE DESIGN (18.24.110)

Recommendations regarding pipe design on steep slopes, pipe cover, and sanitary sewers in the vicinity of water supplies can be found below.

3.7.1 Steep Slopes (4)

The City's current design documents do not provide guidance on a gravity pipe's maximum velocity. It is recommended the City add a provision requiring pipes where the velocity is greater than 15 feet per second be ductile iron or other material as approved by the City Engineer. Special provisions should be made to protect manholes against erosion and displacement by hydraulic forces. This may include splitting a 90 degree horizontal direction change into two 45 degree incremental changes

3.7.2 Pipe Cover (5)

Current City standards dictate that minimum cover of pipes are as follows:

- Non-reinforced pipe – 36 inches
- Ductile iron – 18 inches

With the measurement points varying depending on the land use directly above the pipe. These requirements provide adequate cover to preserve a pipe's structural integrity; however, there are other items to consider.

It is recommended that all sewers be laid at a depth sufficient to drain (by gravity) the lowest elevation of existing, proposed, and future building sewers to protect against damage by frost or traffic. Depth is measured from the top of the pipe to finish grade at the sewer alignment. Under normal conditions, sewers in residential areas are recommended to be placed under the street with the following minimum depths:

- Main sewers – 6 feet
- Collector, trunk, and interceptor sewers – 8 feet

Sewer serving non-residential developments or residential developments where recommended depths are not attainable should be permitted on an as-approved basis by the City Engineer.

3.7.3 Sanitary Sewer in Vicinity of Water Supplies (6)

The City has published guidance on designing and constructing sanitary sewer lines in the vicinity of water supplies; however, some of the guideline's conflict with Oregon Administrative Rules (OAR) Chapter 333-061-0050. Per St. Helens Engineering Standards Manual, "No sanitary sewer shall be less than 10 feet from any well, spring, or other source of domestic water supply." Per OAR Chapter 333, "no gravity sewer line or septic tank shall be permitted within 50 feet of a well which serves a public water system." It is recommended that the City either 1) revise this section to be in accordance with OAR Chapter 333 or 2) delete this section and replace it with a reference to OAR 333.

3.8 MANHOLE DESIGN (18.24.120)

Manhole design provisions currently state that "manholes shall be provided at least every 400 feet, at every change in alignment, and at every grade change. A manhole shall be located at the upstream end of the pipe except as allowed in SHMC 18.24.130." It is recommended that the maximum distance be reduced from 400 feet to 300 feet. Additionally, it is recommended that the City amend this list to include "at every point where there is a change in pipe size, at each intersection or junction of a sewer, and at any point where an 8-inch diameter or larger private sewer intersects with the public sewer." In general, it is good practice to install manholes in street intersections whenever feasible.

The current minimum manhole size required by the City is 48-inches. It is recommended that minimum manhole diameters be sized based off the diameter of pipes entering the manhole, as shown in Table 3-1.

TABLE 3-1: MINIMUM MANHOLE SIZE

Manhole Interior Diameter (inches)	Maximum Pipe Size with 0 degrees deflection (inches)	Maximum Pipe Size with 90 degrees deflection (inches)
48	18	15
60	30	18
72	42	30
84	54	36

3.9 ADDITIONAL RECOMMENDATIONS

3.9.1 Stream and Creek Crossings – Engineering

The City's current standards provide provisions for contractors constructing stream and creek crossings, but do not provide provisions for designing stream and creek crossings.

It is recommended that, generally, the top of all sewers entering or crossing streams shall be a minimum of three feet below the stream bed and at a sufficient depth below the streambed to protect the sewer main. Inverted siphons shall not be allowed at stream or drainage crossings. Concrete encasement may be required in other cases dependent on soil types, depth of cover, and streambed characteristics.

Sewers located parallel to streams shall be located outside of the streambed and sufficiently removed from the streambed to provide for future possible stream channel widening and in accordance with applicable City code requirements for waterway and riparian area protection.

Sewers crossing streams or drainage channel shall be designed to cross the stream as nearly perpendicular to the stream channel as possible and at a uniform grade. Pipe material shall be DI class 50 with an 18-foot length of pipe centered on the stream or drainage channel centerline. The DI pipe shall extend to a point where a one-to-one slope, which begins at the top of the bank and slopes down from the bank away from the channel centerline, intersects the top of the pipe.

Pipes crossing larger streams or creeks shall be subject special review and approval.

3.9.2 Facilities Not Addressed in Standards

It is recommended that the City add a section to St. Helens Municipal Code Title 18 Engineering Standards Manual in which sanitary sewer 'special' facilities are defined. City engineer standards are generally not intended to address the requirements for all possible public or private facilities. Facilities not addressed in these standards are considered unique and must be designed to meet site specific criteria. For these types of facilities, the design engineer must request a pre-design meeting with the City to review the appropriate design and operation and maintenance (O&M) criteria that will apply to the specific project prior to submittal of any design reports or plans.

The following are examples of facilities that are recommended to require special review and approval:

- Sewer Force Mains
- Relining of Existing Sewers
- Internal Sealing of Existing Sewers
- Wastewater Regulatory Devices
- Wastewater Pump Stations

- Sewer Siphons
- Wastewater Treatment Plants
- Wastewater Flow Measurement/Monitoring Devices
- Stream Crossings
- Extension of Municipal Sewer Service Outside the Urban Growth Boundary

4. COMPREHENSIVE PLAN

There are no recommendations for sanitary sewer provisions in the SHMC Title 19 Comprehensive Plan.

APPENDIX D

Pump Station Pump Curves

KELLER ASSOCIATES

PUMP STATION 1 PUMP CURVE

06/12/2007 14:20 503-241-0399 PACO PUMPS PORTLAND PAGE 06

PACO. PUMPS

Paco Express 5.0.9.127

Pump Performance Datasheet

Customer	:
Customer reference	:
Item number	:
Service	:
Quantity of pumps	: 1

Quote number	:
Pump size	: 401221 QDSC
Stages	: 1
Based on curve number	: RC9889-2 Rev 0
Date last saved	: 12 Jun 2007

Operating Conditions

Flow, rated	: 500 USgpm
Head, rated (requested)	: 120 ft
Head, rated (actual)	: 120 ft
Suction pressure, rated / max	: 0.00 / 0.00 psi.g
NPSH available, rated	: Ample
Frequency	: 60 Hz

Liquid

Liquid type	: Water
Additional liquid description	:
Solids diameter, max	: 0.00 in
Temperature, max	: 68.00 deg F
Fluid density, rated / max	: 0.998 / 0.998 SG
Viscosity, rated	: 1.00 cP

Performance

Speed	: 1,750 rpm
Impeller diameter, rated	: 11.61 in
Impeller diameter, maximum	: 12.00 in
Impeller diameter, minimum	: 9.00 in
Efficiency	: 57.37 %
NPSH required / margin required	: 8.80 / 0.00 ft
Specific speed / Suction specific speed	: 1,747 / 4,740 US units
MCSF	: 144 USgpm
Head, maximum, rated diameter	: 147 ft
Head rise to shutoff	: 22.12 %
Flow, best eff. point (BEP)	: 1,134 USgpm
Flow ratio (rated / BEP)	: 44.09 %
Diameter ratio (rated / max)	: 96.75 %
Head ratio (rated dia / max dia)	: 92.92 %
Viscous coefficients (CQ / CH / CE)	: 1.00 / 1.00 / 1.00
Selection status	: Acceptable

Material

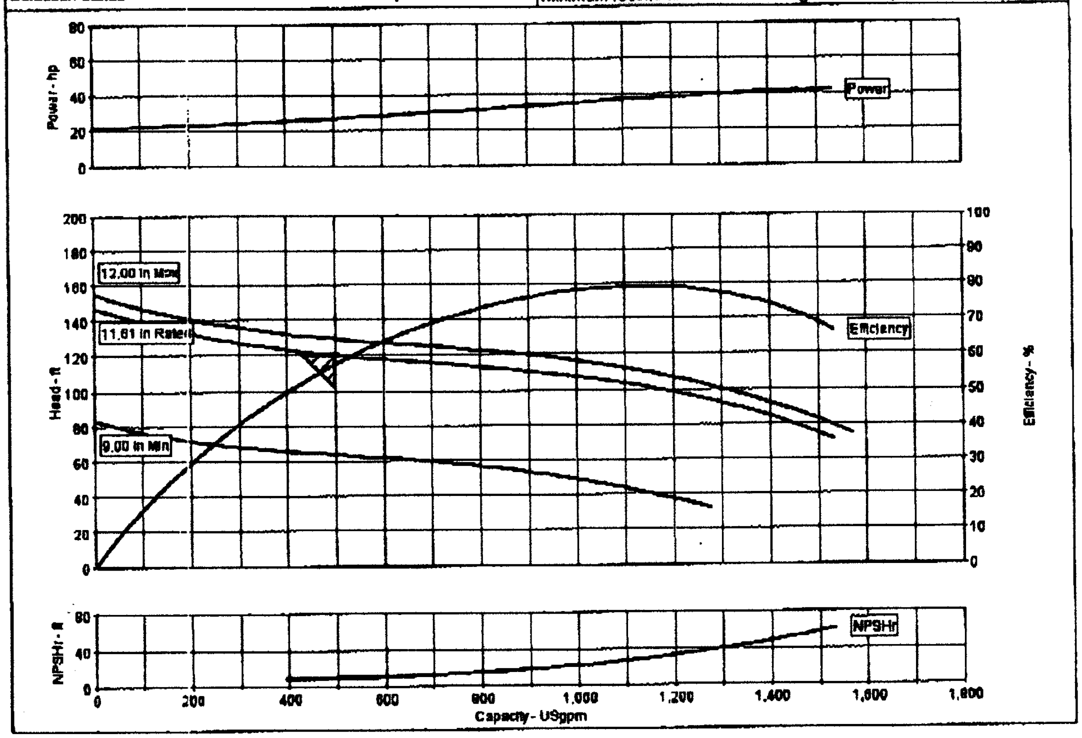
Material requested	: Cast iron
Material selected	: Cast iron

Pressure Data

Maximum working pressure	: 63.42 psi.g
Maximum allowable working pressure	: 175.0 psi.g
Maximum allowable suction pressure	: 175.0 psi.g
Hydrostatic test pressure	: 263.0 psi.g

Driver & Power Data

Driver sizing specification	: Rated power
Margin over specification	: 0.00 %
Service factor	: 1.00
Power, hydraulic	: 15.1 hp
Power, rated	: 26.4 hp
Power, maximum, rated diameter	: 41.4 hp
Minimum recommended driver rating	: 30.0 hp / 22.4 kW

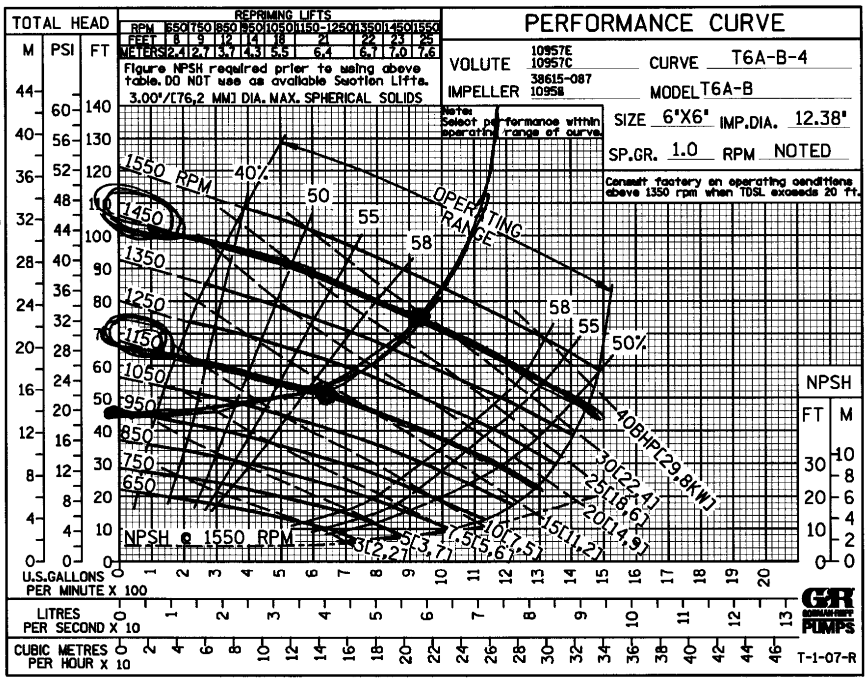


PUMP STATION 2 PUMP CURVE

Tested Pump #1 & #2 on 9/18/12

PUMP MAINTENANCE AND REPAIR – SECTION E

MAINTENANCE AND REPAIR OF THE WEARING PARTS OF THE PUMP WILL MAINTAIN PEAK OPERATING PERFORMANCE.



* **STANDARD PERFORMANCE FOR PUMP MODEL T6A3-B, Including /F, /FM**

*Based on 70° F (21° C) clear water at sea level with minimum suction lift. Since pump installations are seldom identical, your performance may be different due to such factors as viscosity, specific gravity, elevation, temperature, and impeller trim.

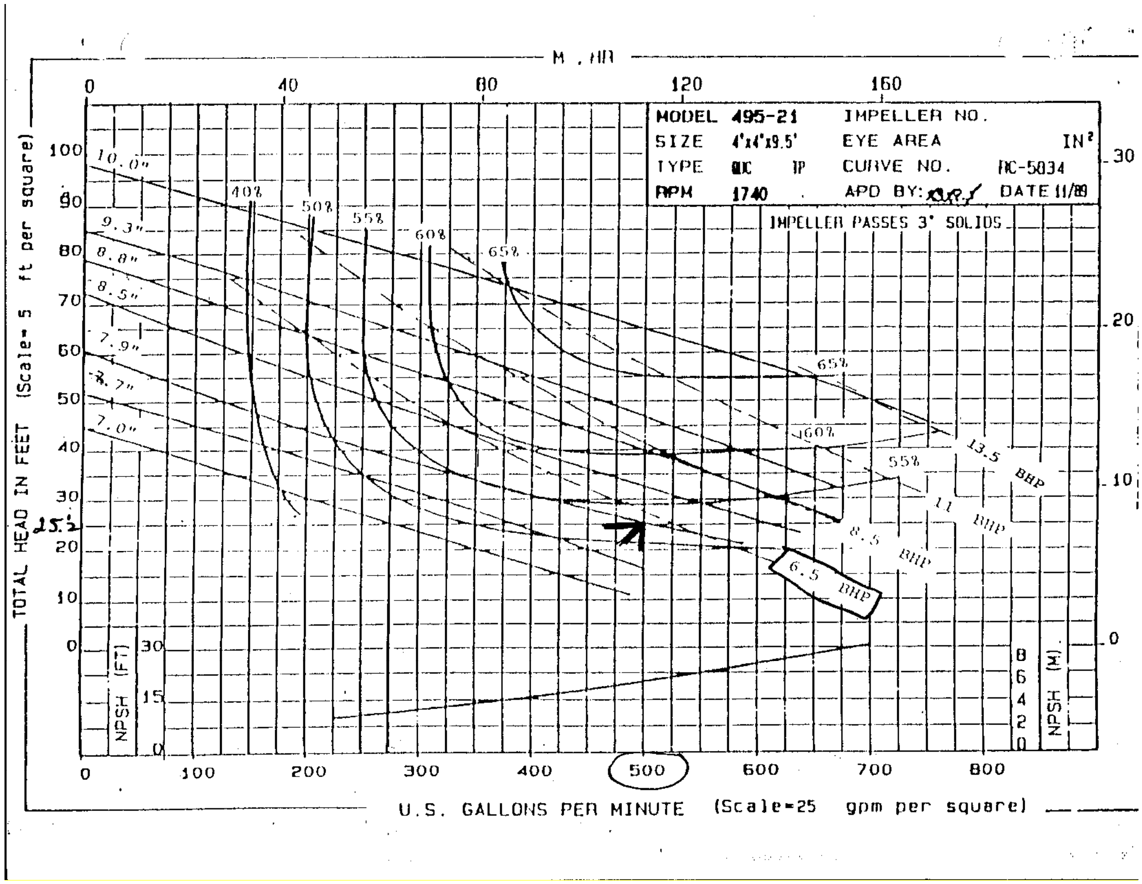
If your pump serial number is followed by an "N", your pump is **NOT** a standard production model.

Contact the Gorman-Rupp Company to verify performance or part numbers.

CAUTION

Pump speed and operating condition points must be within the continuous performance range shown on the curve.

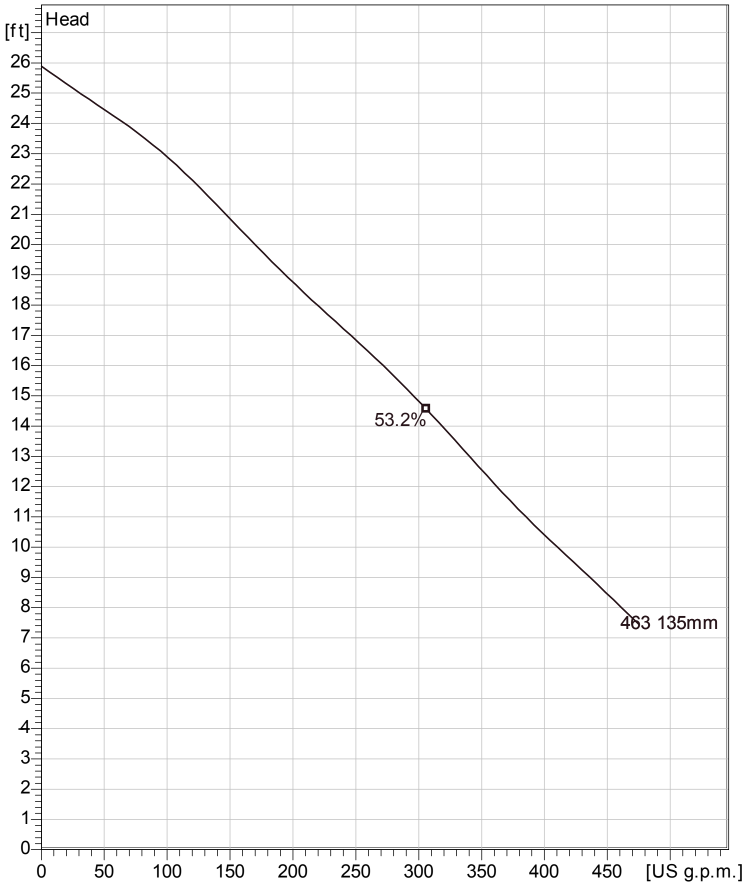
PUMP STATION 3 PUMP CURVE



PUMP STATION 4 PUMP CURVE

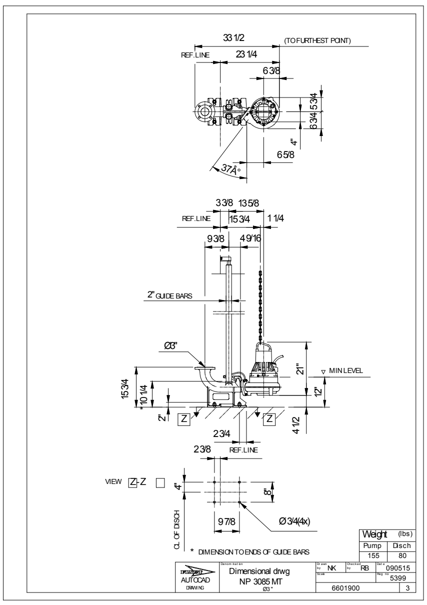
NP 3085 MT 3~ 463

Technical specification



Curve according to: ISO 9906

Installation: P - Semi permanent, Wet



Note: Picture might not correspond to the current configuration.

General

Patented self cleaning semi-open channel impeller, ideal for pumping in waste water applications. Possible to be upgraded with Guide-pin® for even better clogging resistance. Modular based design with high adaptation grade.

Impeller

Impeller material	Grey cast iron
Outlet width	3 1/8 inch
Inlet diameter	111 mm
Impeller diameter	135 mm
Number of blades	2

Motor

Motor #	N3085.092 15-10-4AL-W
Stator variant	
Frequency	60 Hz
Rated voltage	460 V
Number of poles	4
Phases	3~
Rated power	3 hp
Rated current	4.3 A
Starting current	22 A
Rated speed	1700 rpm
Power factor	
1/1 Load	0.83
3/4 Load	0.77
1/2 Load	0.66
Efficiency	
1/1 Load	78.0 %
3/4 Load	79.0 %
1/2 Load	77.0 %

Configuration

Project	Project ID	Created by	Created on	Last update
			2012-05-18	

NP 3085 MT 3~ 463

Performance curve

Pump

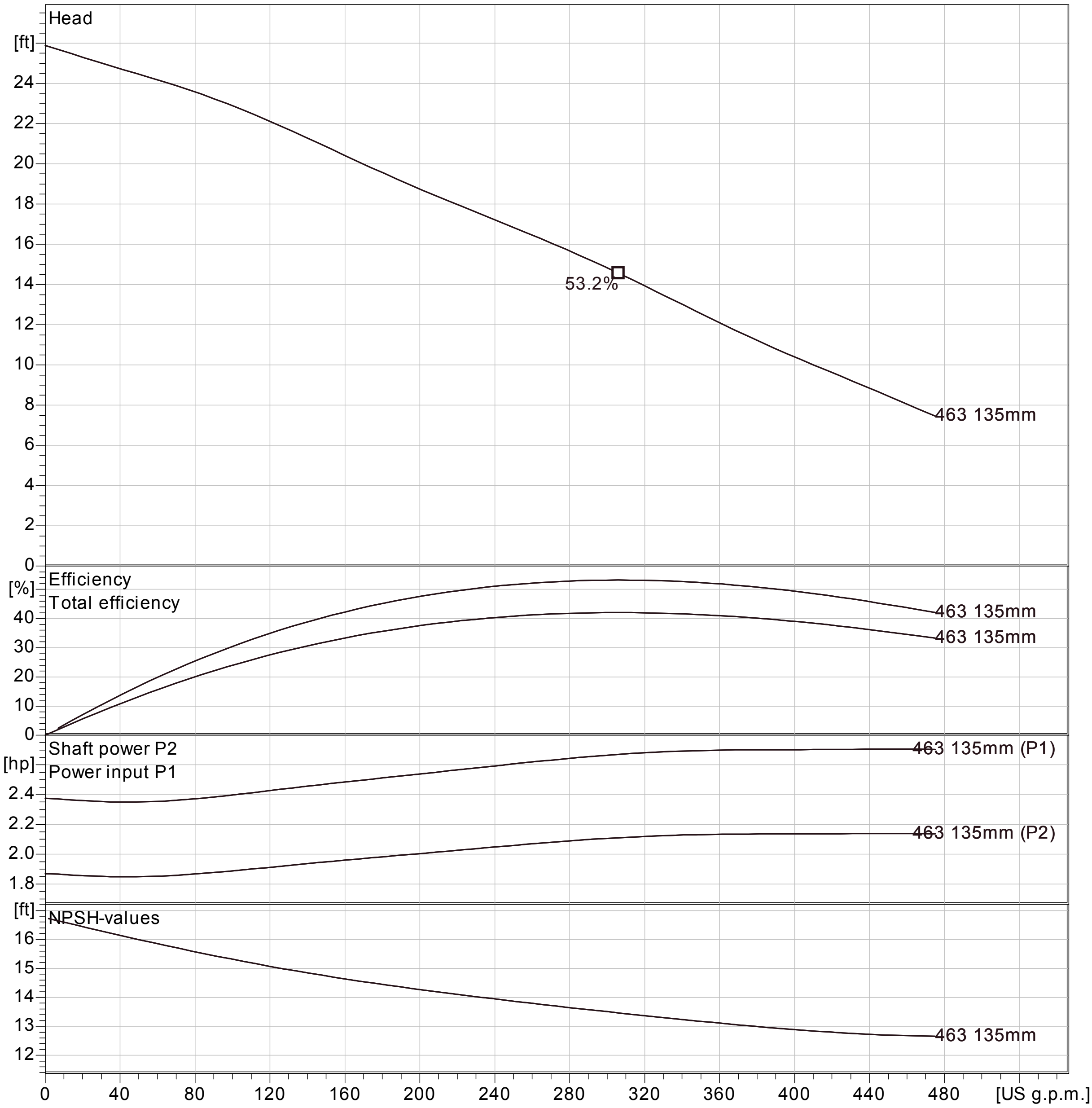
Outlet width	3 1/8 inch
Inlet diameter	111 mm
Impeller diameter	5 5/16"
Number of blades	2

Motor

Motor #	N3085.092 15-10-4AL-W
Stator variant	
Frequency	60 Hz
Rated voltage	460 V
Number of poles	4
Phases	3~
Rated power	3 hp
Rated current	4.3 A
Starting current	22 A
Rated speed	1700 rpm

Power factor	
1/1 Load	0.83
3/4 Load	0.77
1/2 Load	0.66

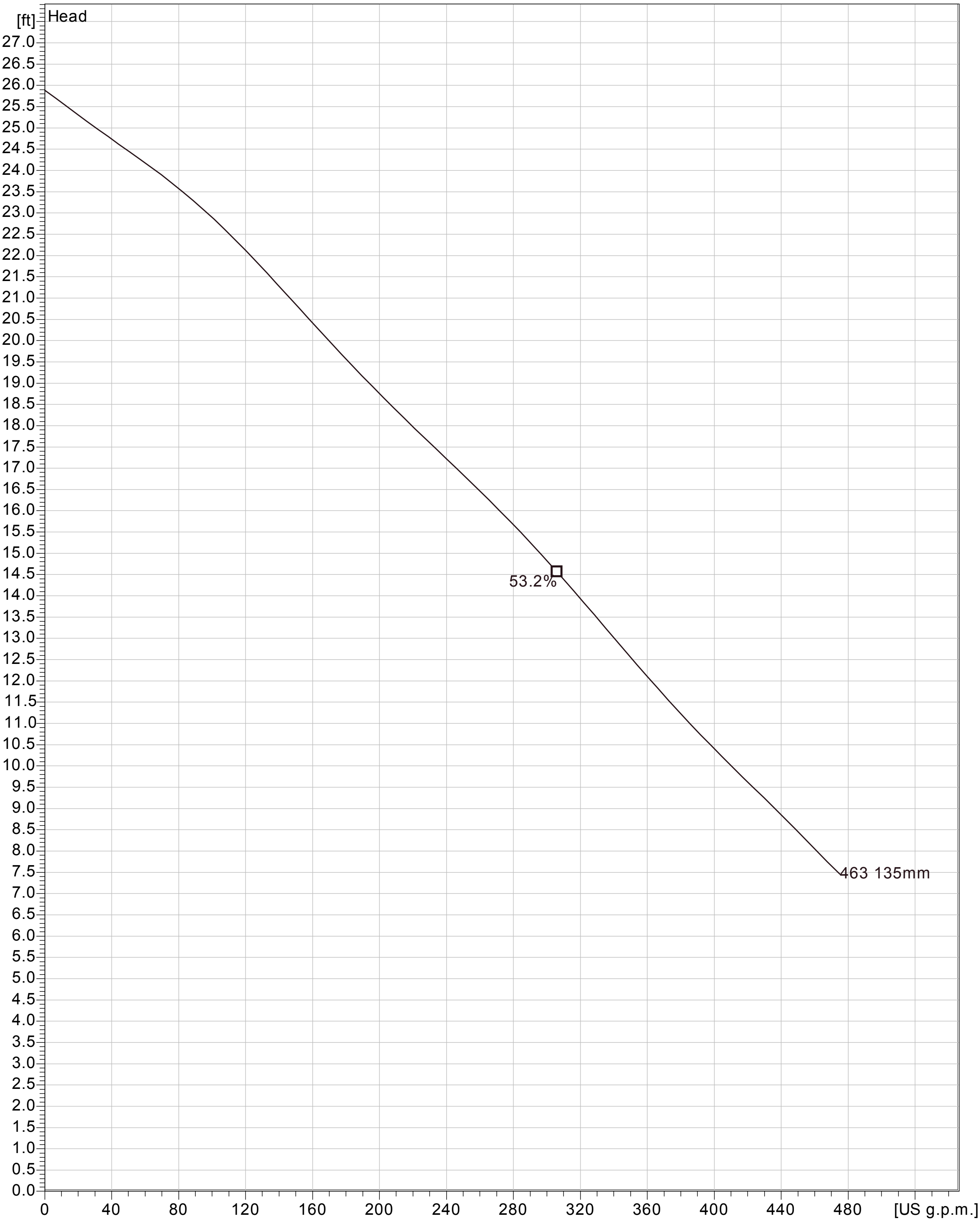
Efficiency	
1/1 Load	78.0 %
3/4 Load	79.0 %
1/2 Load	77.0 %



Curve according to: ISO 9906

Project	Project ID	Created by	Created on	Last update
			2012-05-18	

NP 3085 MT 3~ 463
Duty Analysis

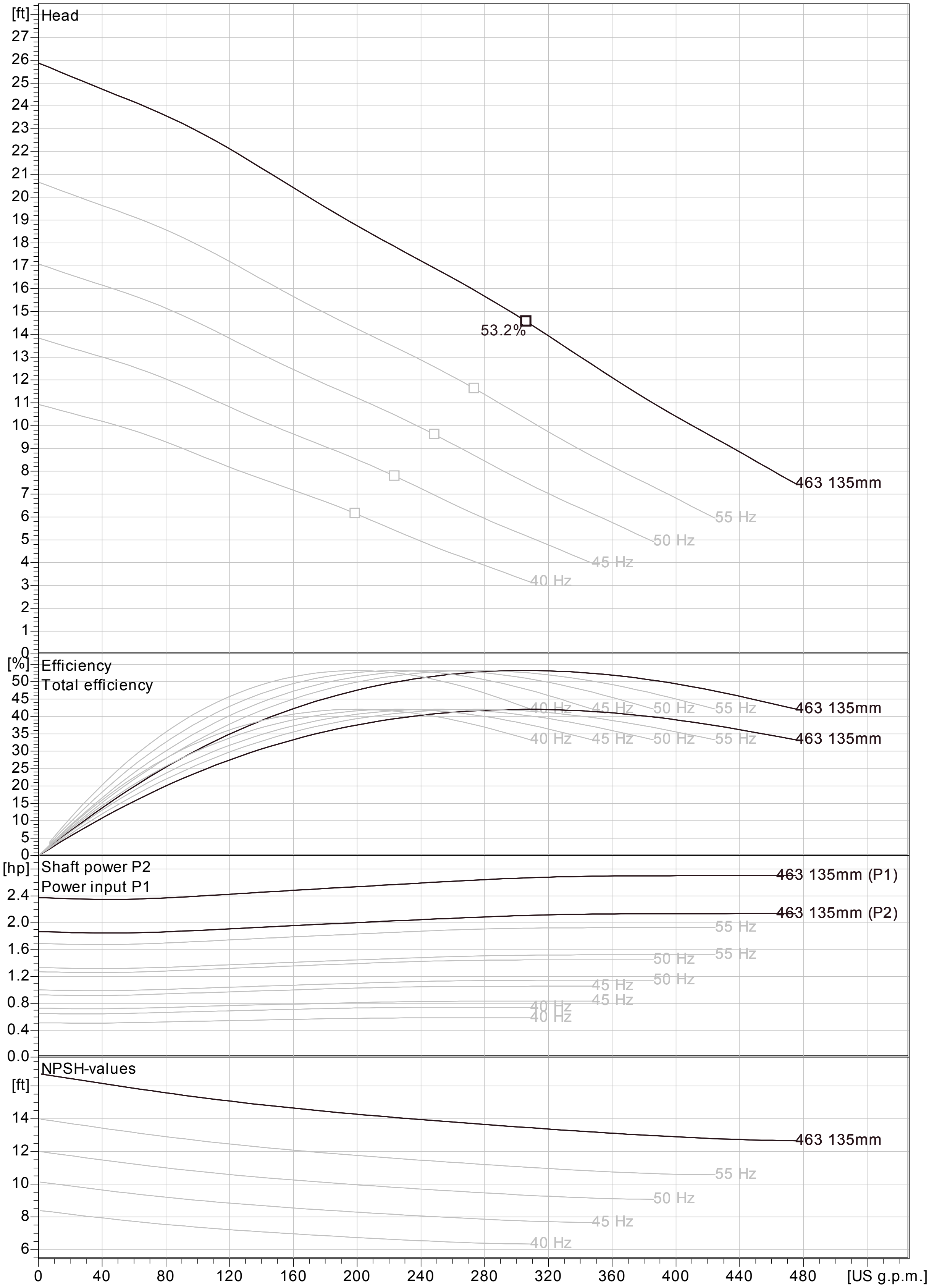


Curve according to: ISO 9906

Project	Project ID	Created by	Created on	Last update
			2012-05-18	

NP 3085 MT 3~ 463

VFD Curve

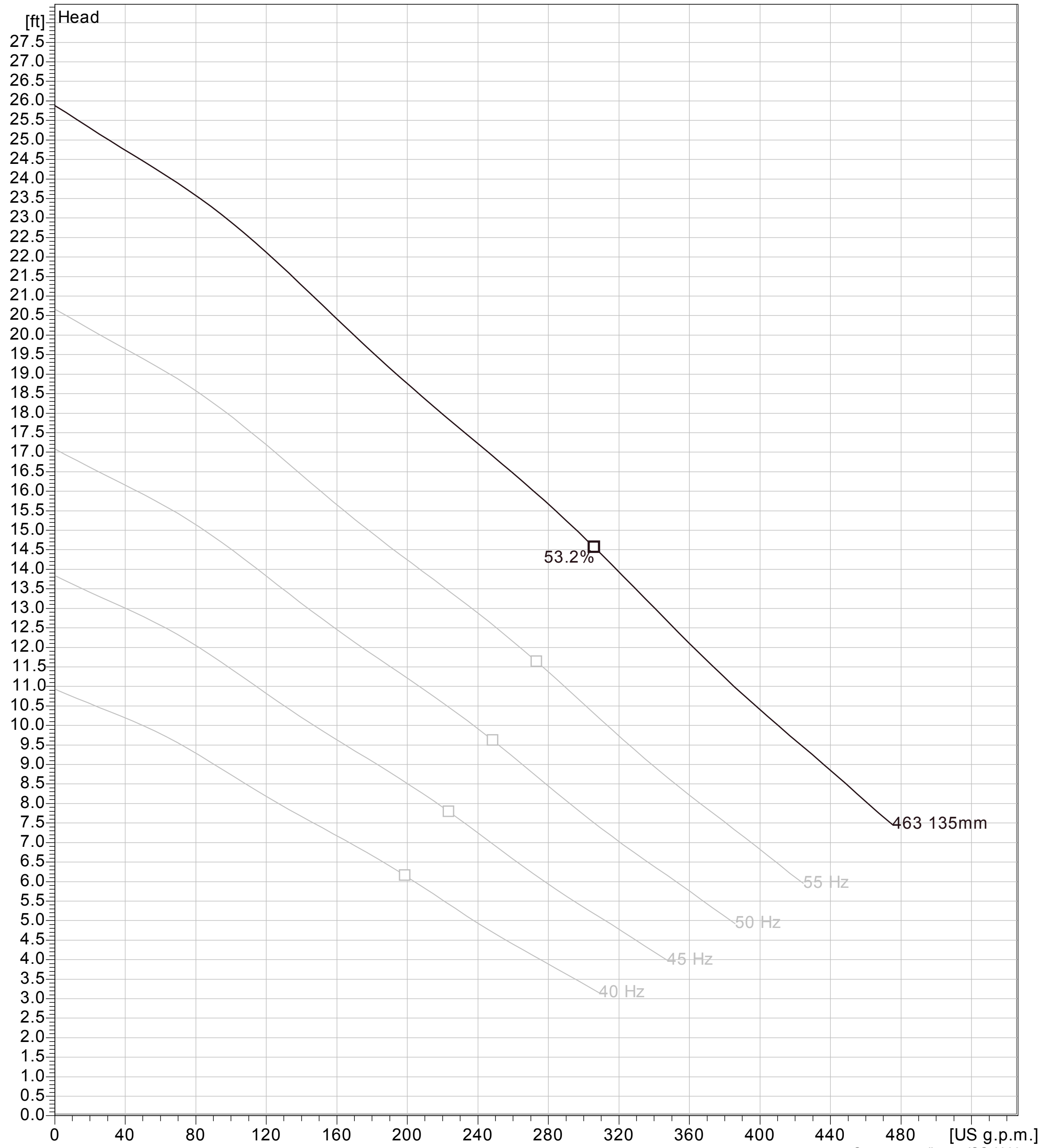


Curve according to: ISO 9906

Project	Project ID	Created by	Created on	Last update
			2012-05-18	

NP 3085 MT 3~ 463

VFD Analysis



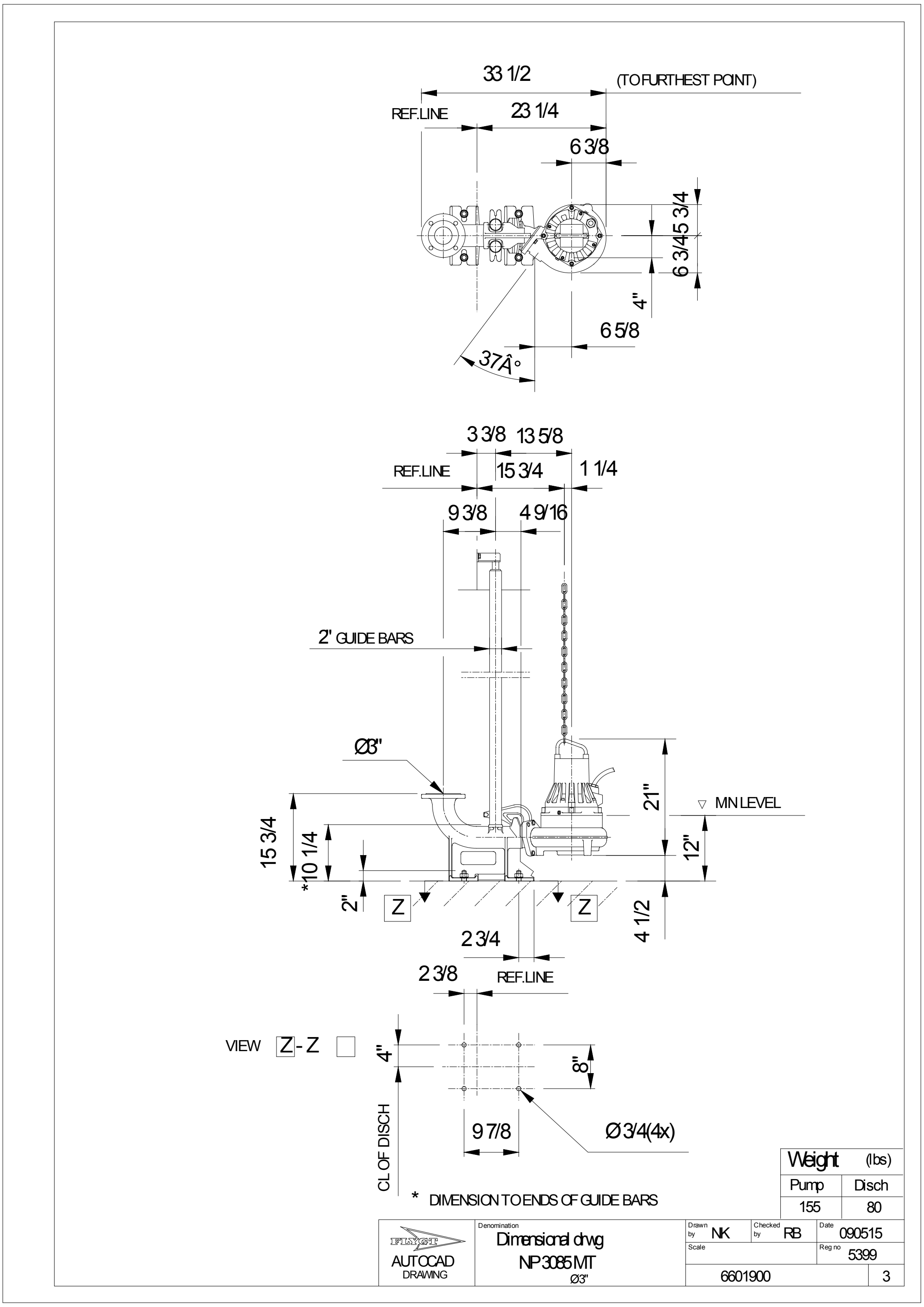
Curve according to: ISO 9906

60 Hz 55 Hz 50 Hz 45 Hz 40 Hz

Project	Project ID	Created by	Created on	Last update
			2012-05-18	

NP 3085 MT 3~ 463

Dimensional drawing



Project	Project ID	Created by	Created on	Last update
			2012-05-18	

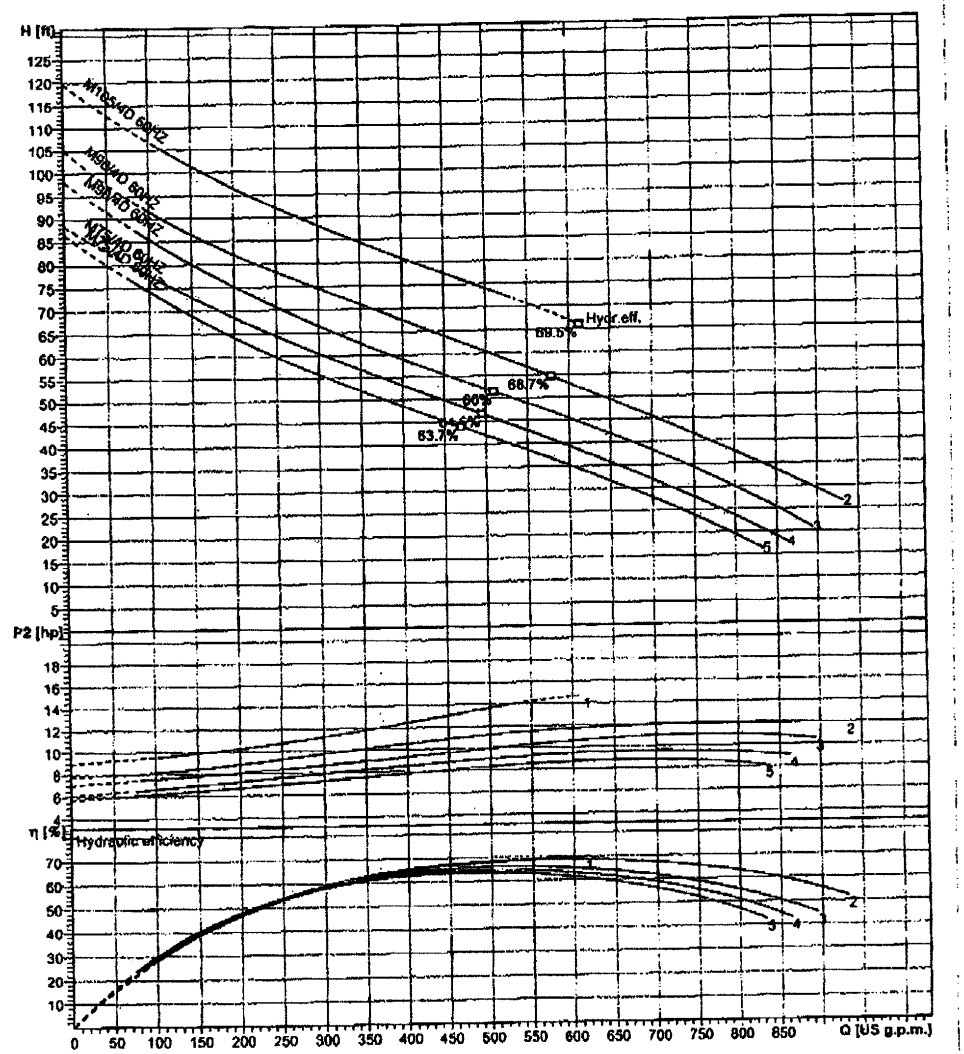
PUMP STATION 5 PUMP CURVE

12-07-'11 14:05 FROM-PUMPTECH, INC 503-659-8718 T-724 P0008/0008 F-006

ABS
COST-EFFECTIVE PUMPING

Pump performance curves
AFP 1049 60 HZ

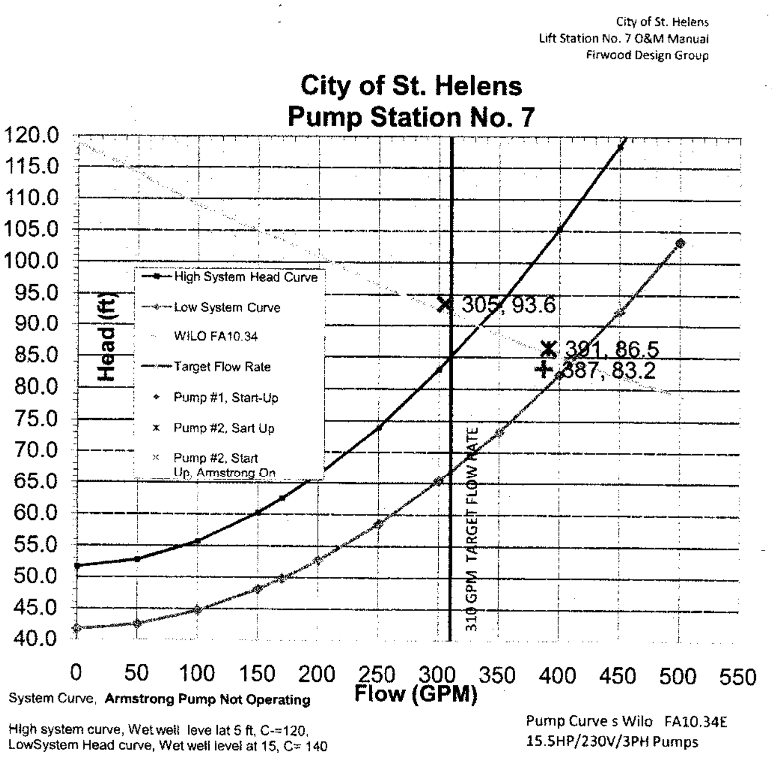
			Discharge 4"	Curve number
				Reference curve AFP 1049
				Frequency 60 Hz
Density 1 kg/dm³	Viscosity 1.562 cSt	Testnorm Hydraulic Institute	Rated speed 1735..1700 rpm	Date 2004-06-04
Flow	Head	Rated power	Hydraulic efficiency	NPSH



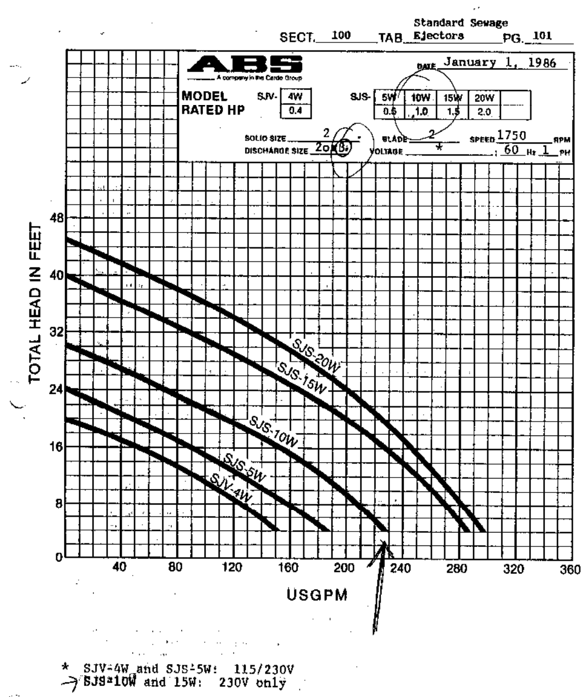
Impeller size	N° of vanes	Impeller	Solid size	Revision 2002-12-10
253..224 mm	1	ContraBlock impeller, 1 vane	3-1/8"	ABSELPRO 1.7.1 / 2004-03-08

ABS reserves the right to change any data and dimensions without prior notice and can not be held responsible for the use of information contained in this software.

PUMP STATION 7 PUMP CURVE



PUMP STATION 8 PUMP CURVE



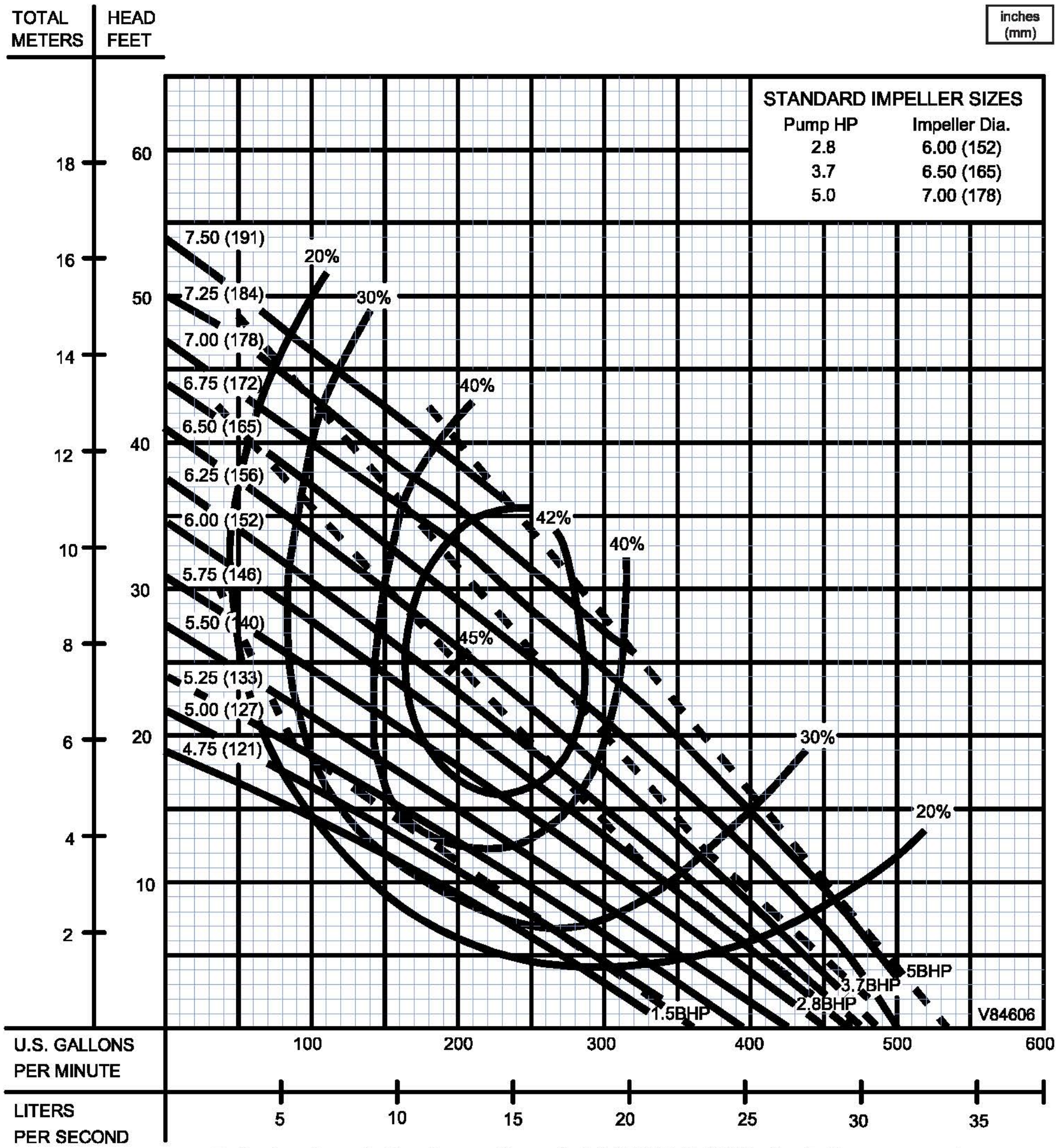
PUMP STATION 9 PUMP CURVE

Series 4SE-L

Performance Curve
2.8, 3.7 & 5.0HP, 1750RPM, 60Hz

BARNES®
www.cranepumps.com

4" Horizontal Discharge - Submersible Non-Clog Pumps



Testing is performed with water, specific gravity 1.0 @ 68° F @ (20°C), other fluids may vary performance

SECTION	1D
PAGE	10
DATE	1/05

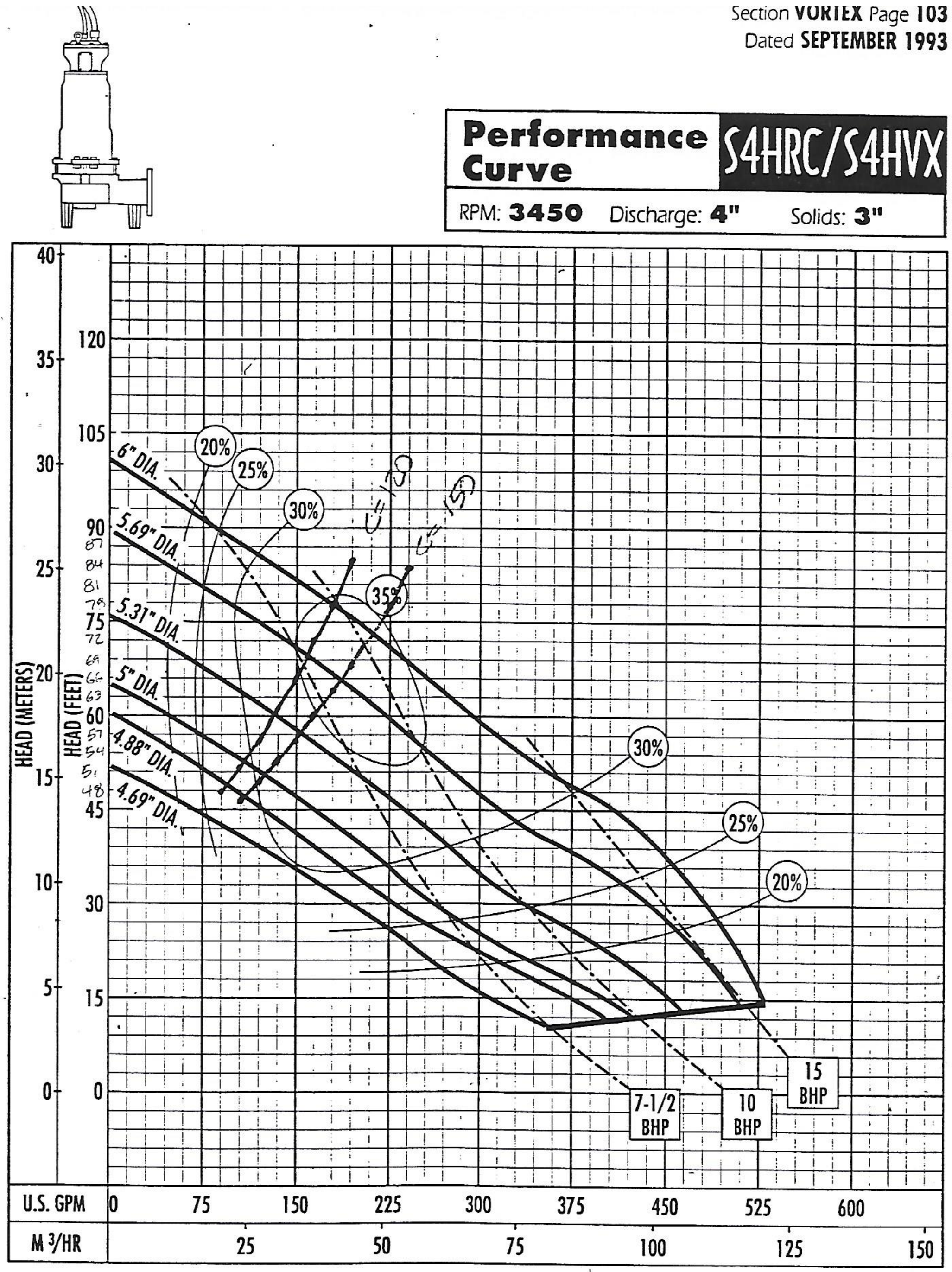
CRANE®
A Crane Co. Company

PUMPS & SYSTEMS

USA: (937) 778-8947 • Canada: (905) 457-6223 • International: (937) 615-3598

PUMP STATION 11 PUMP CURVE

Section VORTEX Page 103
Dated SEPTEMBER 1993



The curves reflect maximum performance characteristics without exceeding full load (Nameplate) horsepower. All pumps have a service factor of 1.2. Operation is recommended in the bounded area with operational point within the curve limit. Performance curves are based on actual tests with clear water at 70° F. and 1280 feet site elevation.

AURORA PUMP
A UNIT OF GENERAL SIGNAL

Conditions of Service:
GPM: 143 TDH: 74

HYDROMATIC™ PUMPS

APPENDIX E

City Identified Wastewater Issues

LIST OF KNOWN SEWER ISSUES:

- PLUG UP/ BACK UP AT 214 S 8TH ST & 791 ST. HELENS ST
- PLUG UP / BACK UP AT 275 S 4TH ST
- PLUG UP / BACK UP AT 285 N 4TH ST
- SEWER & STORM OVERFLOW ISSUES AT 314 S 14TH ST
- PLUG UP / BACK UP AT 495 S 7TH ST
- SEWER ISSUES AT GODFREY PARK
- SEWER ISSUES IN CANYON BEHIND 208 S 9TH ST

SEWER ISSUES AT 214 S 8TH ST & 791 ST. HELENS ST
N
Sewer main backs up into the sewer laterals for these two properties.
S 8TH STREET
S 8TH STREET
S 7TH STREET
8" PVC
8" CP
8" CP
8" CP
8"
6" CIPP
6 PVC
16" PE
185
197
790
721
791
215
205
214
225
216
264
235
274
225
245
284
227
234
255
0
40
80 Feet

SEWER ISSUES AT 275 S 4TH ST

SEWER ISSUES AT 285 N 4TH ST


SEWER ISSUES AT
314 S 4TH ST/ MIDDLE
TRUNK
N
Sewer overflows during
heavy rain events
TUALATIN STREET
S 13TH STREET
S 13TH STREET
S 14TH STREET
12" PE
12" PE
12" DI
6" PVC
12" PE
12" PE
6" CIPP
6" PVC
6" PVC
294
1321
306
315
1421
315
325
344
345
345
355
354
348
355
357
365
364
354
356
365
375
374
0
40
80 Feet

SEWER ISSUES AT
495 N 7TH ST
N
424
435
445
444
8" CIPP
455
8" PVC
465
464
467
469
8" PVC
8" PVC
8" PVC
475
474
S 8TH STREET
S 7TH STREET
6" CIPP
495
484
497
PLYMOUTH STREET
0
40
80 Feet


SEWER ISSUES AT GODFREY PARK
N
Godfrey Park
GODFREY PARK
LUTHERAN CHURCH
CCMH
LIQUOR STORE
KOZY KORNER
COLUMBIA RIVER FIRE & RESCUE
N 4TH STREET
WYETH STREET
N 2ND STREET
N 3RD STREET
S 3RD STREET
COLUMBIA BOULEVARD
8" PVC
6" PVC
6" HDPE
24" PVC
6" VCP
30" CP
10" CP
0
70
140 Feet
225
214
280
204
221
197
261
196
174
164
165
155
145
137
135
134
125
124
115
230
185
127
114
310
380
400
440
271
105


SEWER ISSUES IN
SEWER IN CANYON
N
8" CP
8" PVC
194
8" CP
198
8" CP
204
205
S 9TH STREET
S 8TH STREET
8"
208
8"
8" CP
215
212
225
216
227
220
EISENSCHMIDT LANE
234
255
0
40
80 Feet

APPENDIX F

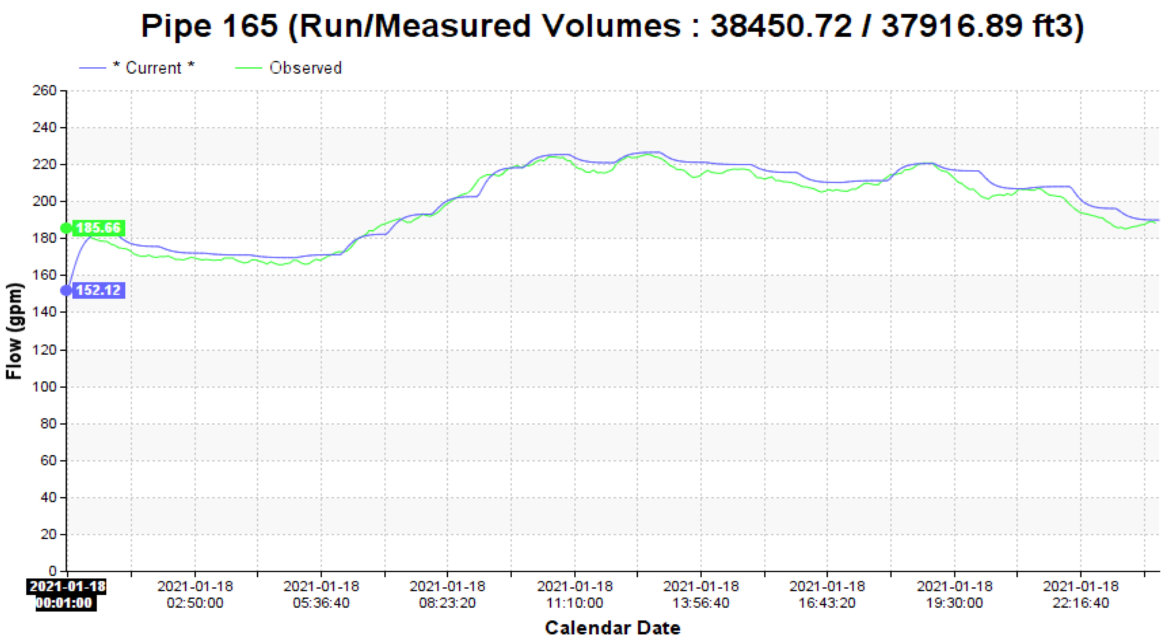
Calibration Information

KELLER
ASSOCIATES

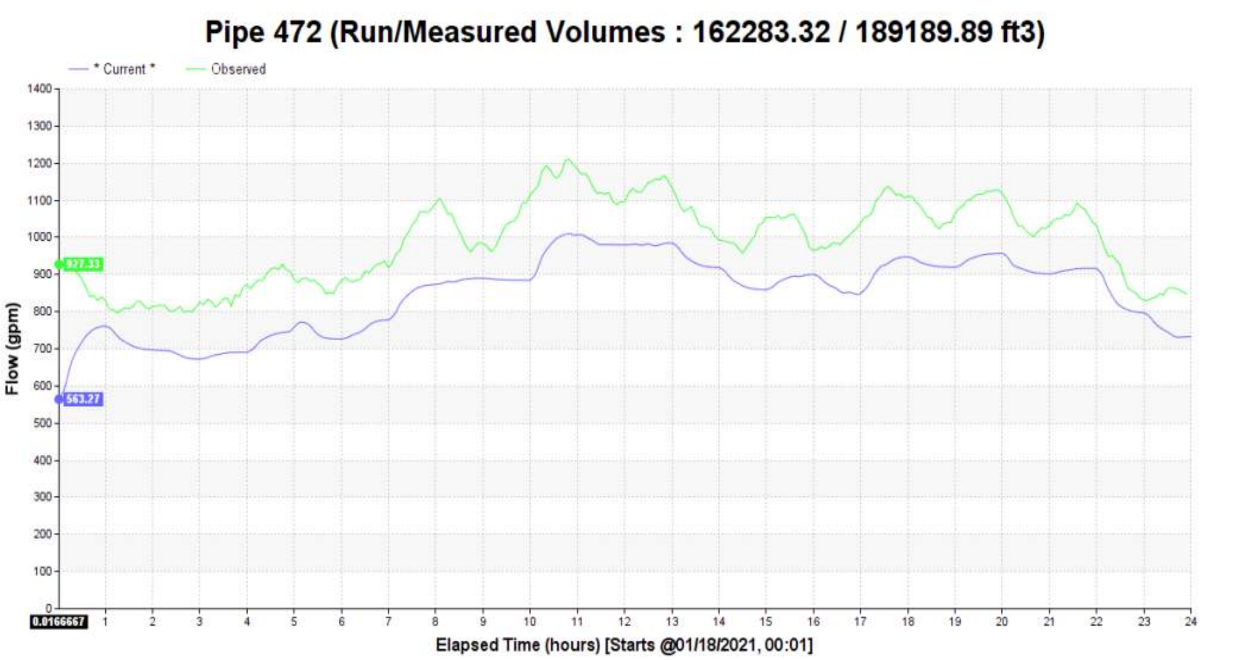
Note: For the following graphs, the green line represents observed flow data from the field, the blue line represents model output

Base Flow Calibration

Site 1 Calibration

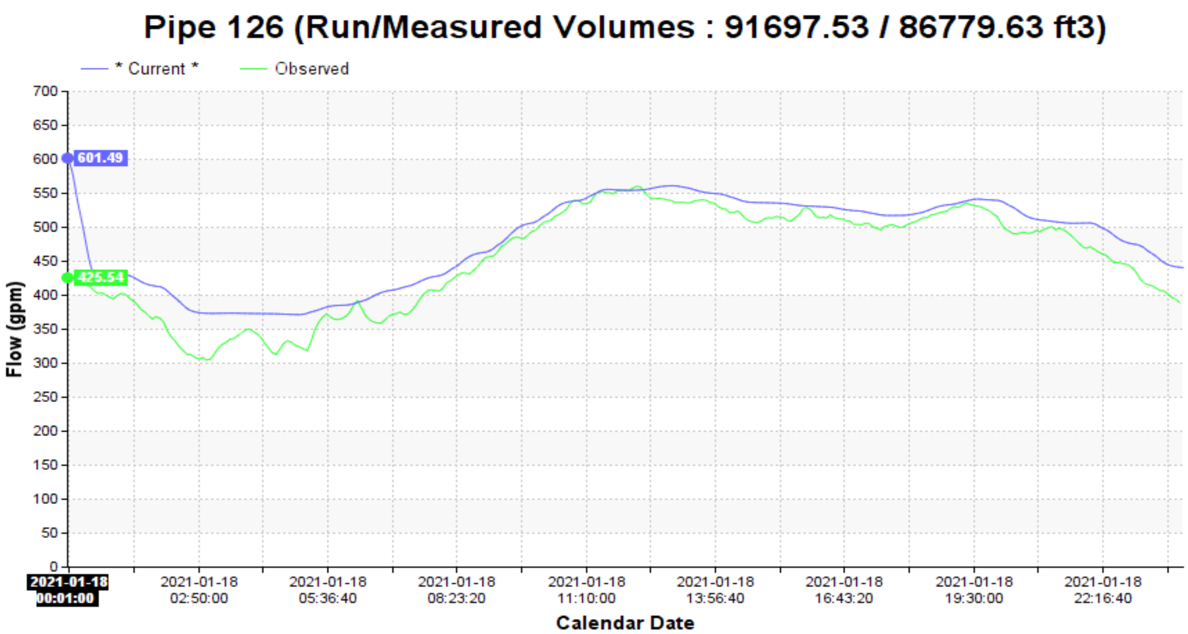


Site 2 Calibration



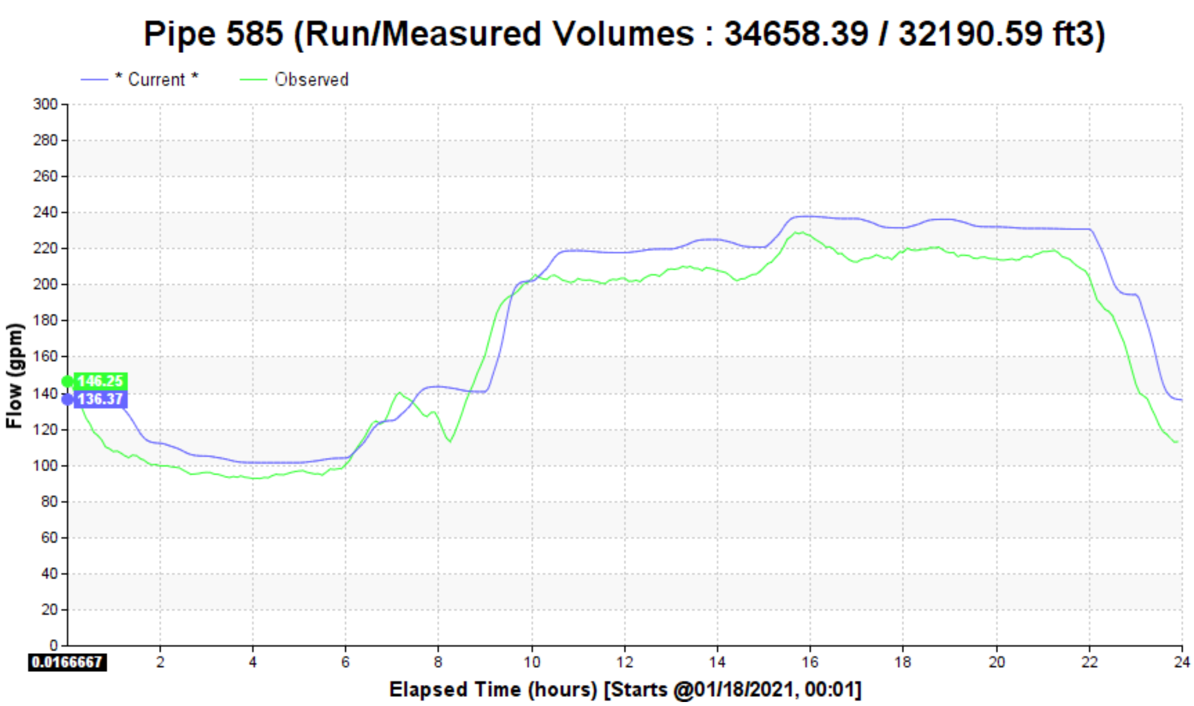
Base Flow Calibration

Site 3 Calibration



Base Flow Calibration

Site 4 Calibration

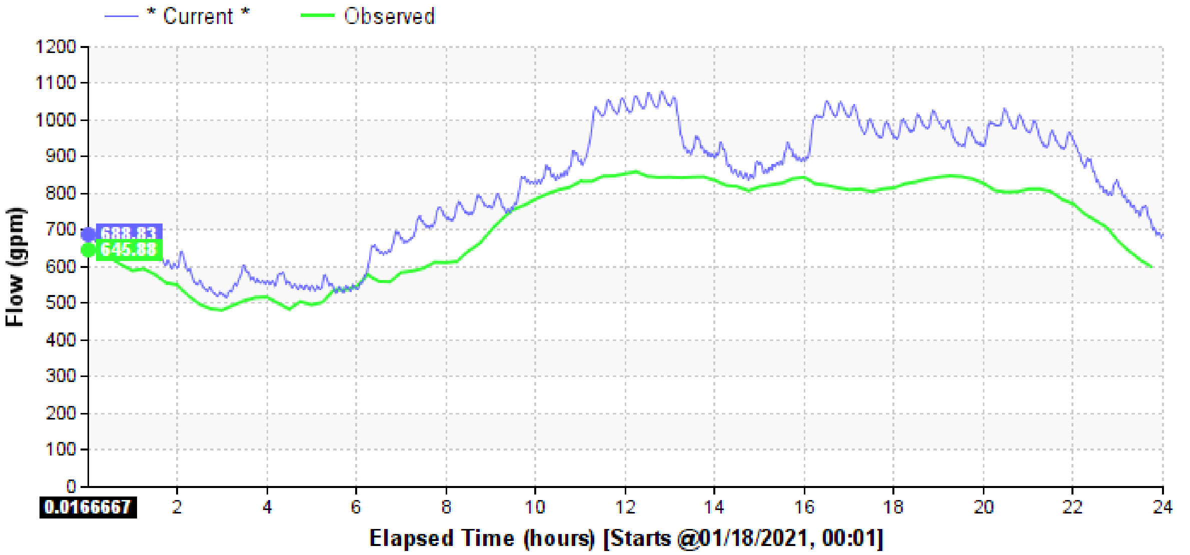


Base Flow Calibration

Site 5 Calibration

- Site 5 was calibrated to the modified calibration curve of site 3 + site 4 + 5% of WWTP flow

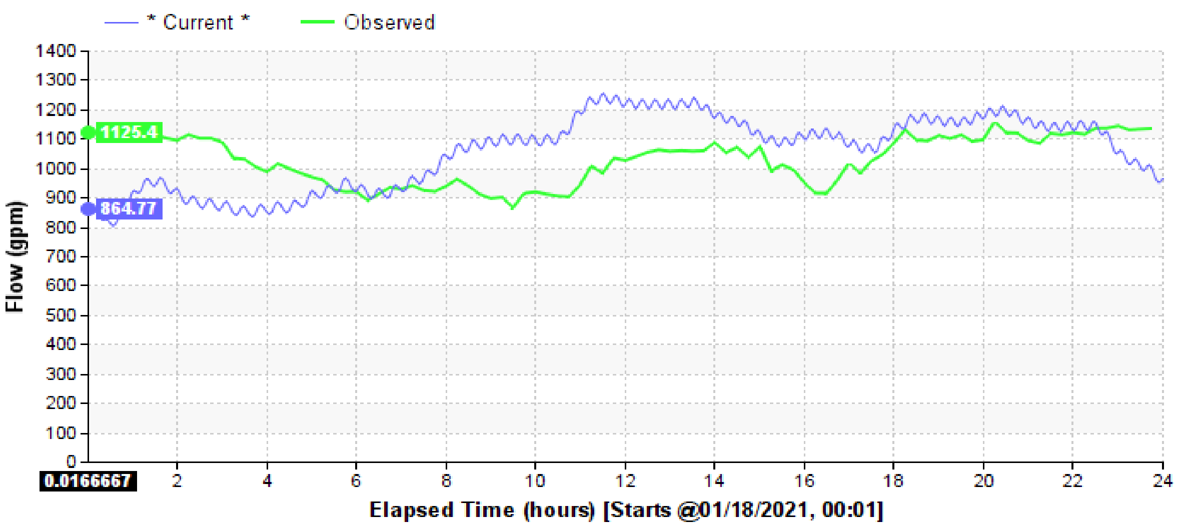
Pipe 260 (Run/Measured Volumes : 154056.26 / 133925.01 ft3)



Site 6 Calibration

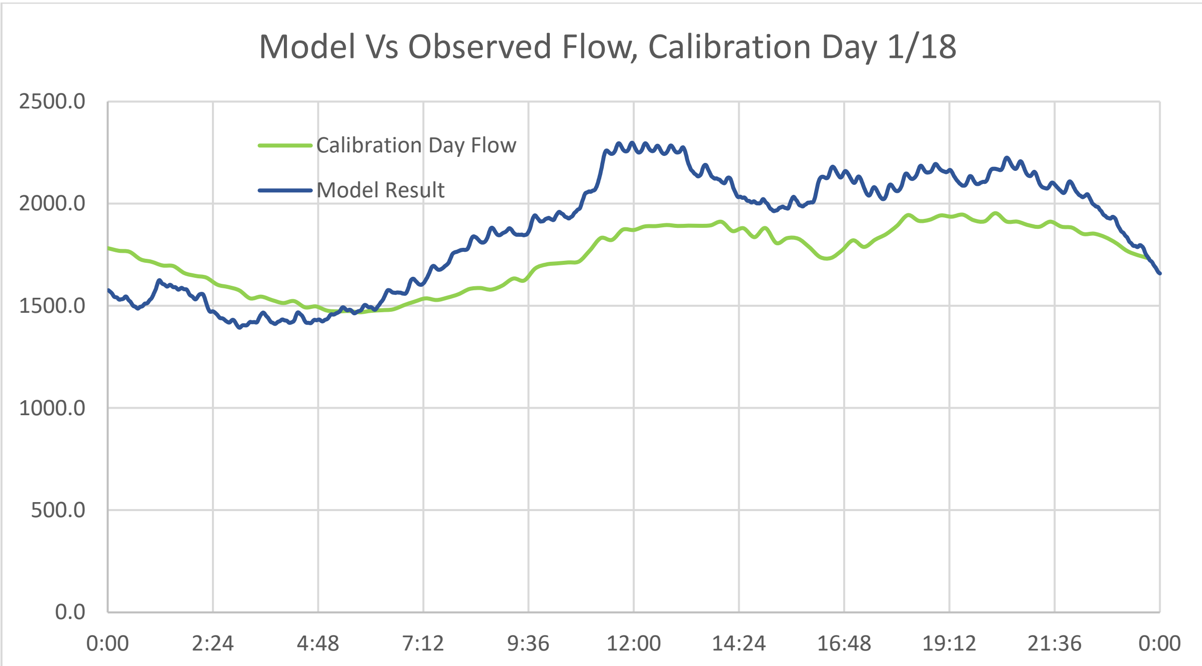
- Site 6 was calibrated to the modified curve of WWTP Flow minus Site 5 flow

Pipe 560 (Run/Measured Volumes : 200352.72 / 193682.04 ft3)



Base Flow Calibration

WWTP Calibration



Base Flow Calibration

Pump Station Calibration

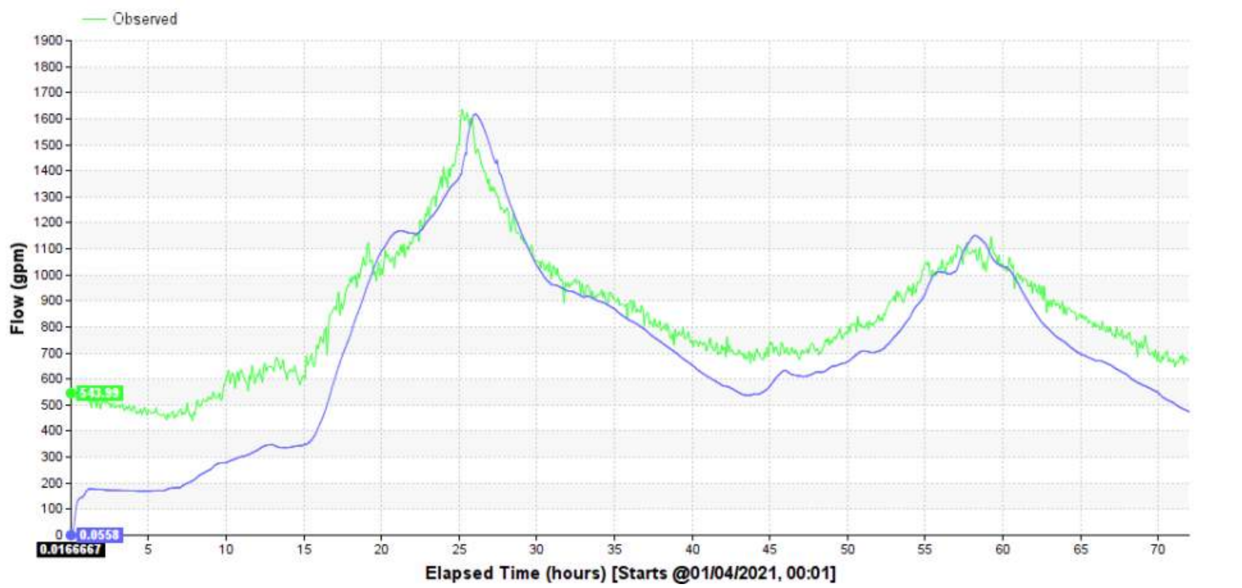
	Pump Station 1	Pump Station 2*	Pump Station 3	Pump Station 7	Pump Station 11
Pump Reported Capacity (gpm)	550	250	500	390	143
Model Average Flow (gpm)	627	275	550	440	133

* Pump Station 2 had its curve modified from the original curve to achieve this flow

Wet Weather Calibration

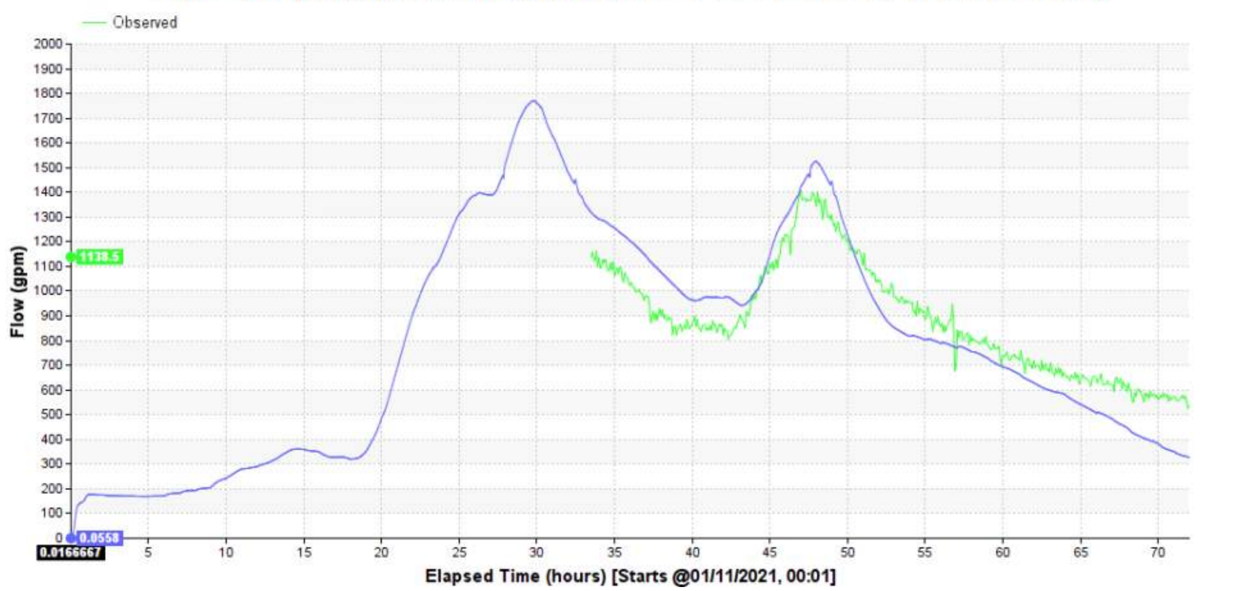
Flowmeter 1 – Calibration Period 1 (Jan 2nd – 4th)

Pipe 163 (Run/Measured Volumes : 414760.02 / 487033.77 ft3)



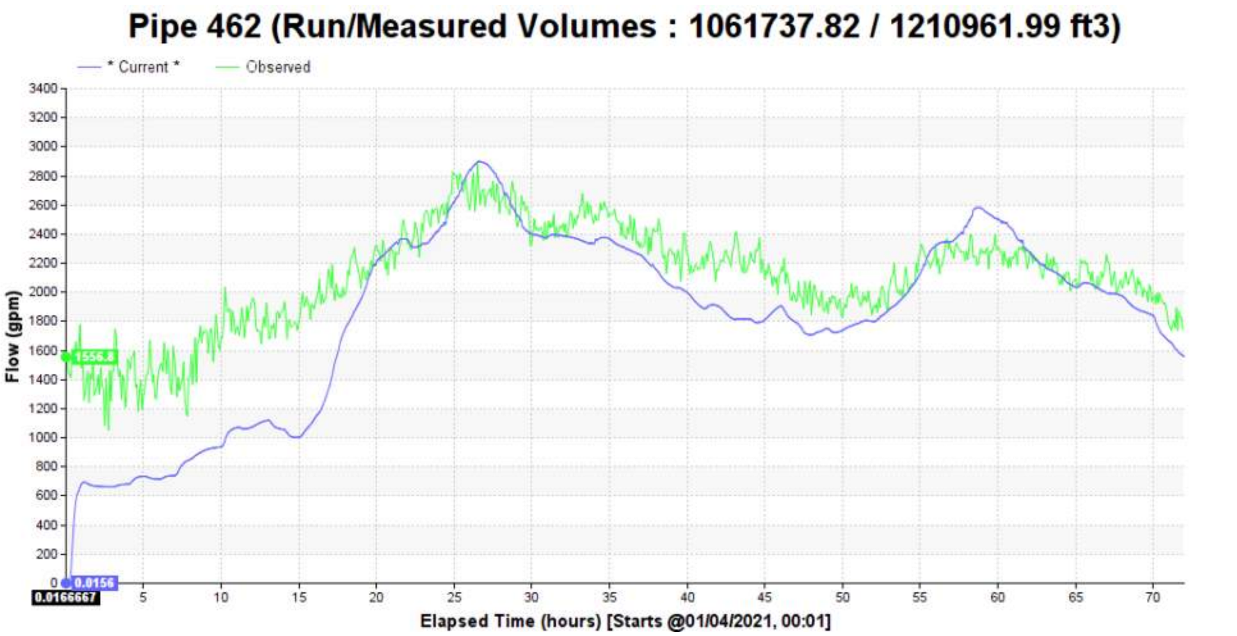
Flowmeter 1 – Calibration Period 2 (Jan 11th – 13th)

Pipe 163 (Run/Measured Volumes : 271438.95 / 274293.48 ft3)

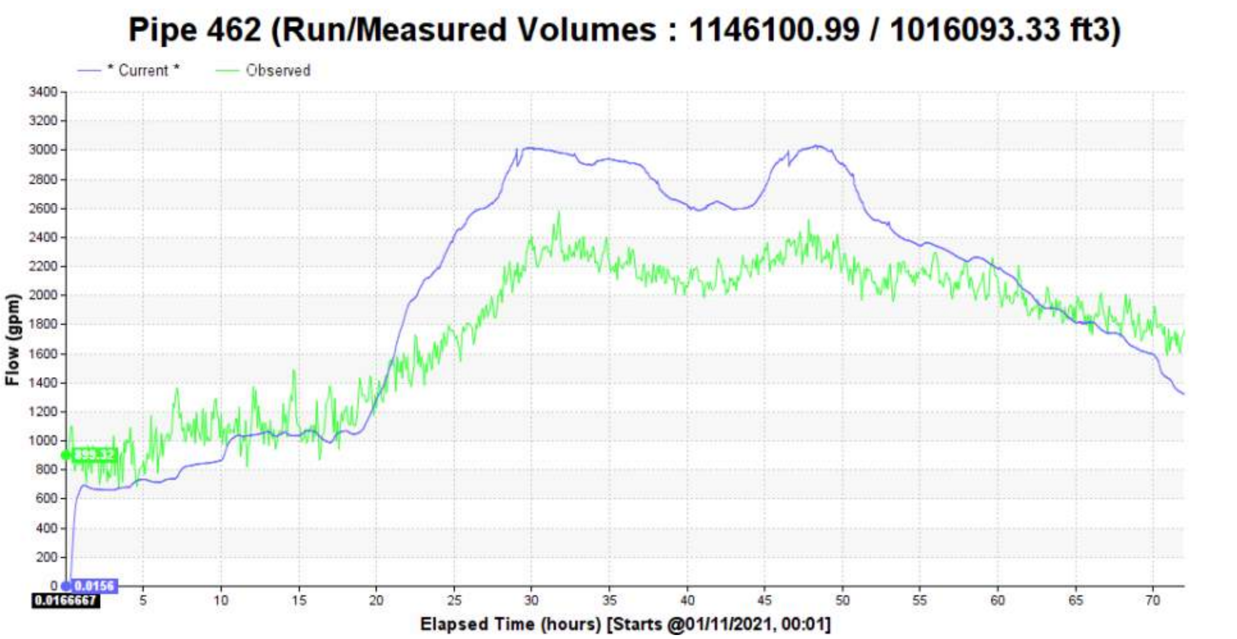


Wet Weather Calibration

Flowmeter 2 – Calibration Period 1 (Jan 2nd – 4th)

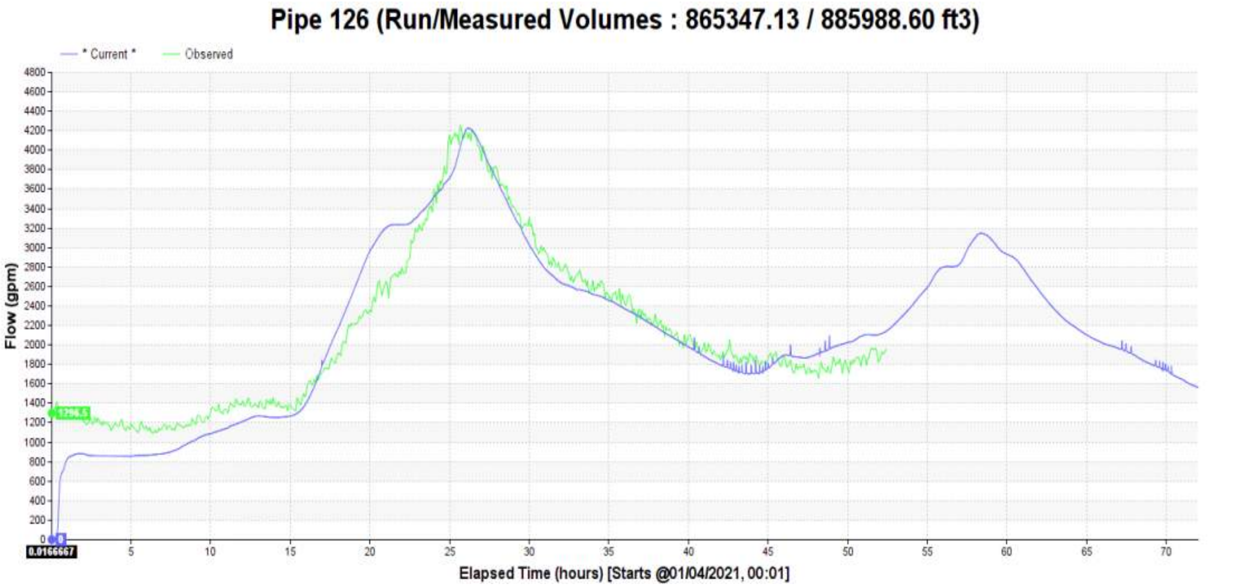


Flowmeter 2 – Calibration Period 2 (Jan 11th – 13th)

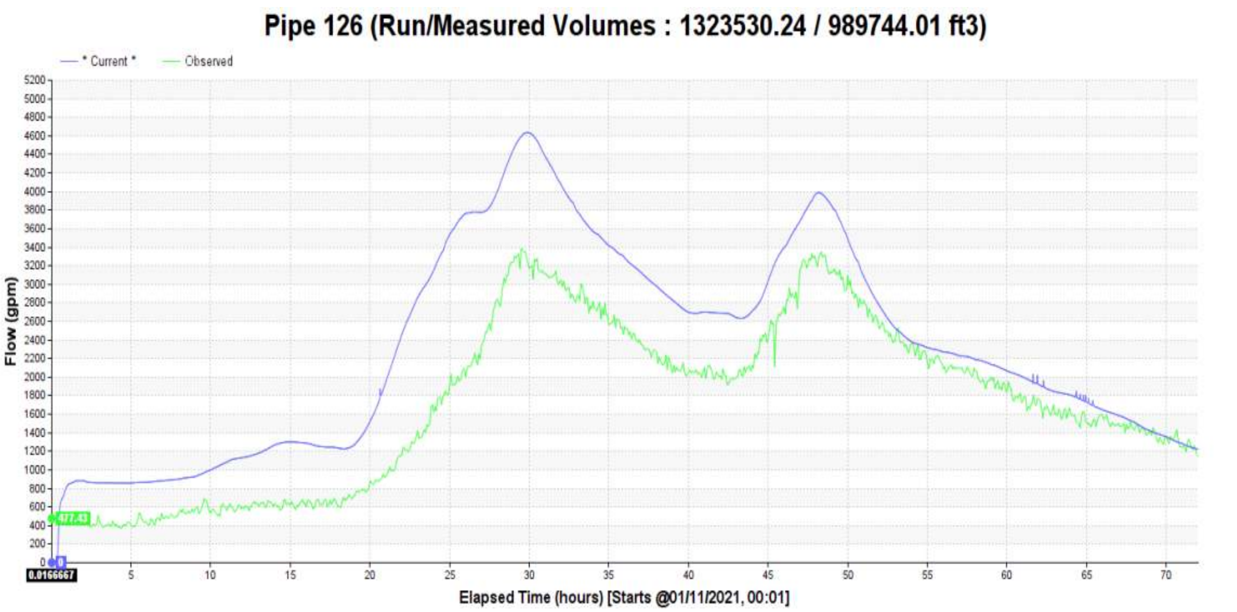


Wet Weather Calibration

Flowmeter 3 – Calibration Period 1 (Jan 2nd – 4th)

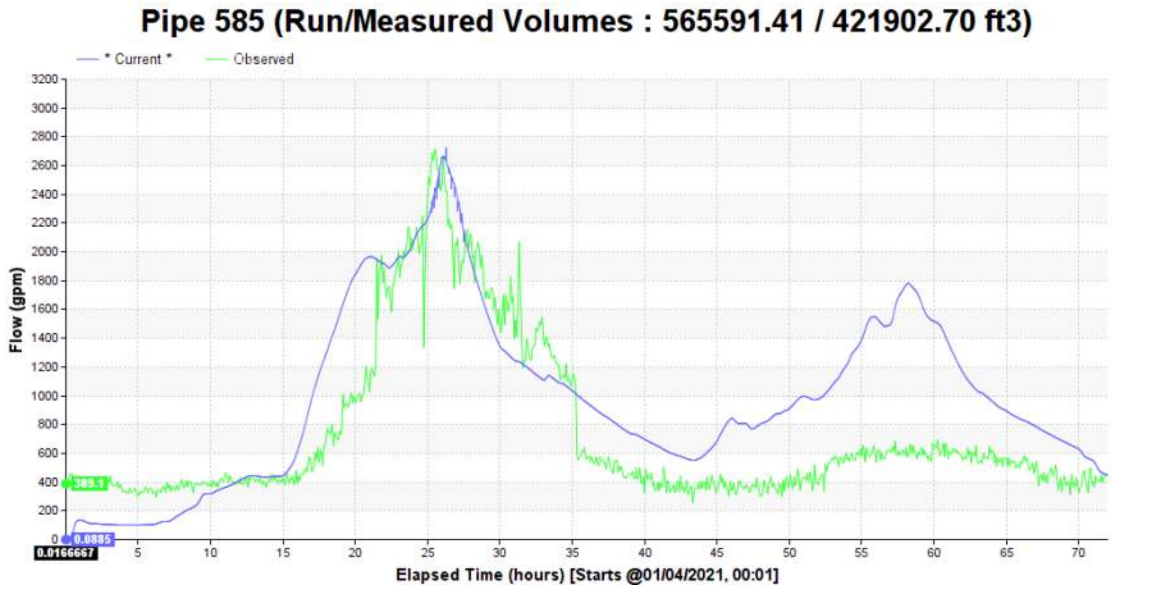


Flowmeter 3 – Calibration Period 2 (Jan 11th – 13th)

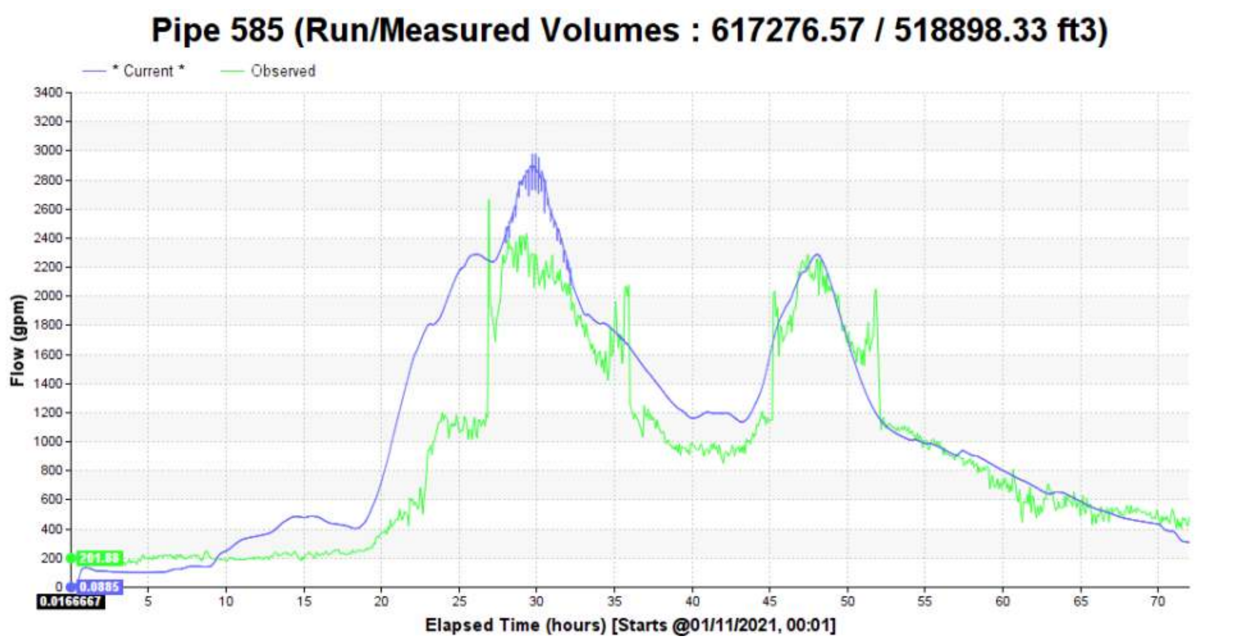


Wet Weather Calibration

Flowmeter 4 – Calibration Period 1 (Jan 2nd – 4th)

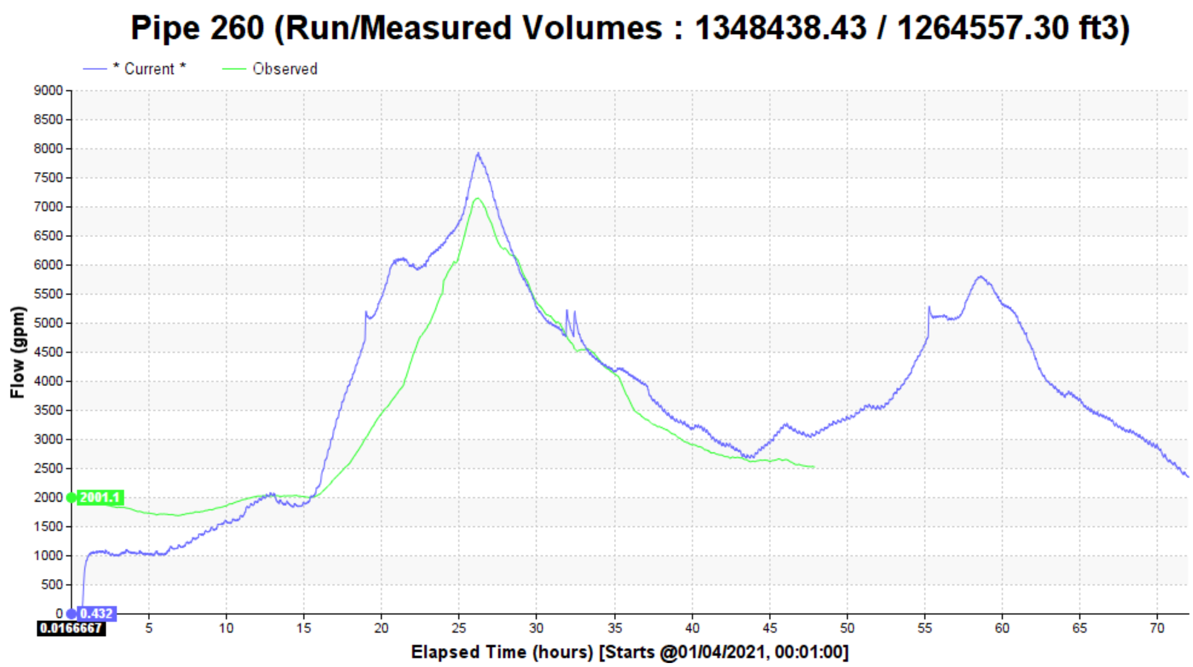


Flowmeter 4 – Calibration Period 2 (Jan 11th – 13th)

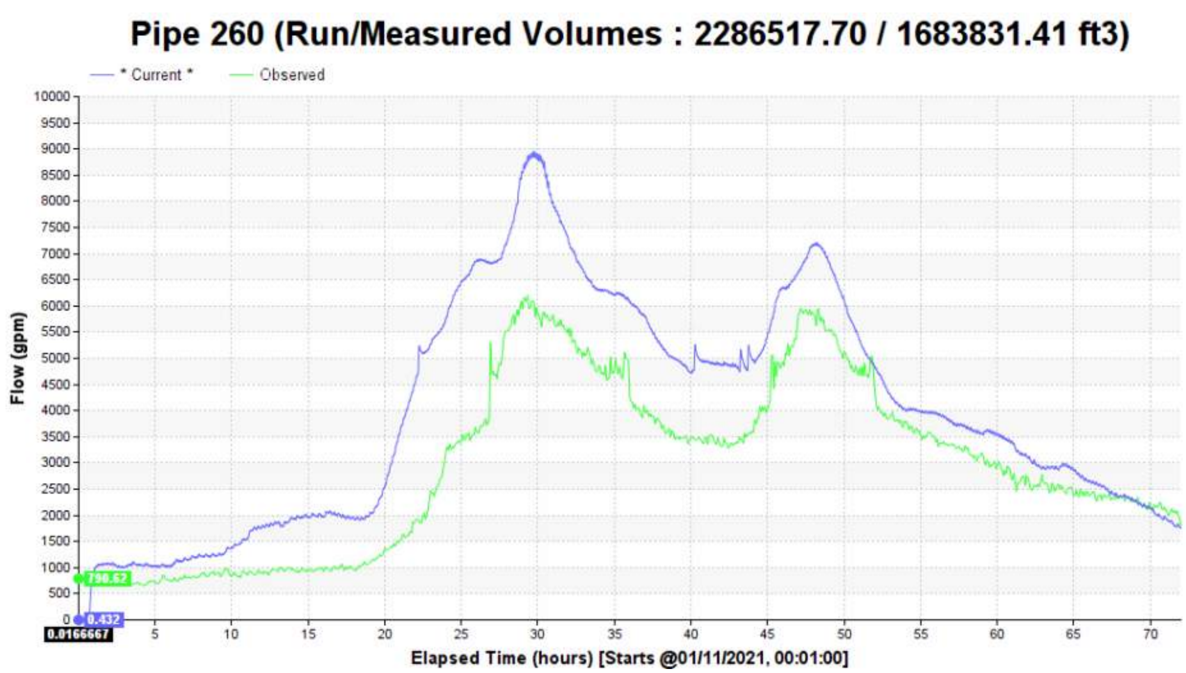


Wet Weather Calibration

Flowmeter 5 – Calibration Period 1 (Jan 2nd – 4th)



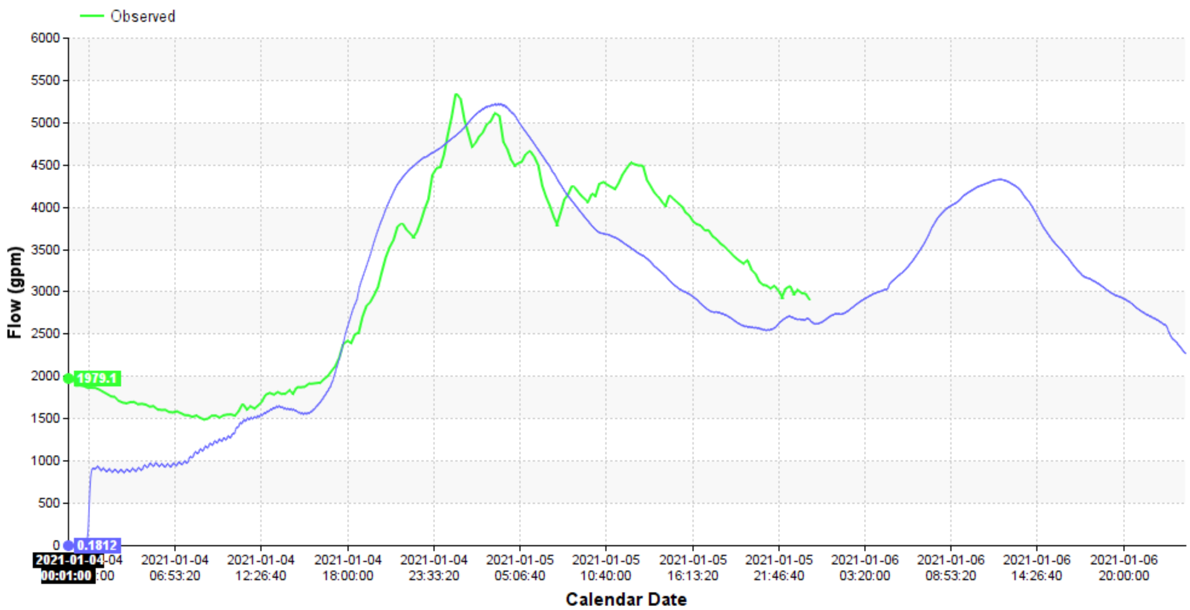
Flowmeter 5 – Calibration Period 2 (Jan 11th – 13th)



Wet Weather Calibration

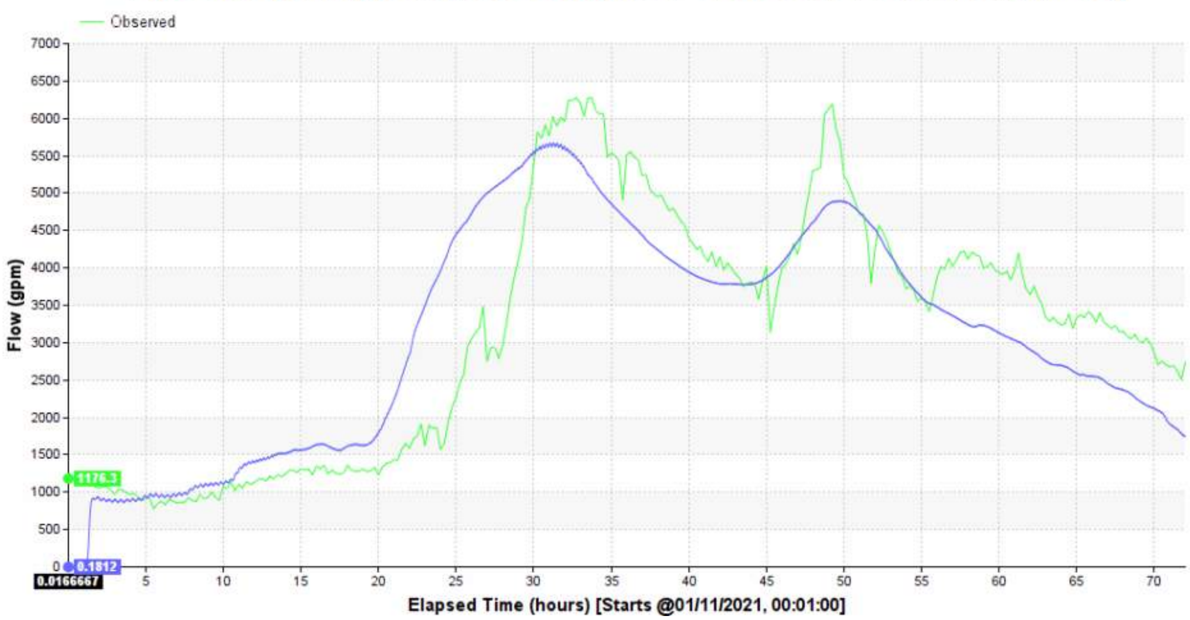
Flowmeter 6 – Calibration Period 1 (Jan 2nd – 4th)

Pipe 560 (Run/Measured Volumes : 1072604.83 / 1186295.73 ft3)



Flowmeter 6 – Calibration Period 2 (Jan 11th – 13th)

Pipe 560 (Run/Measured Volumes : 1787912.94 / 1821127.18 ft3)

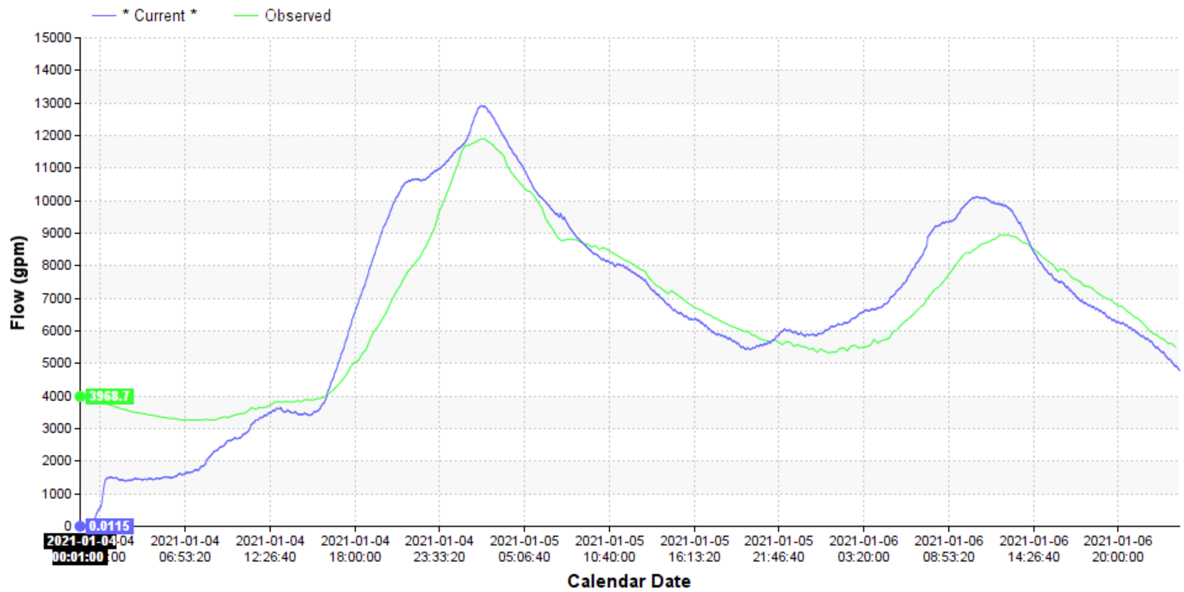


Wet Weather Calibration

WWTP Calibration

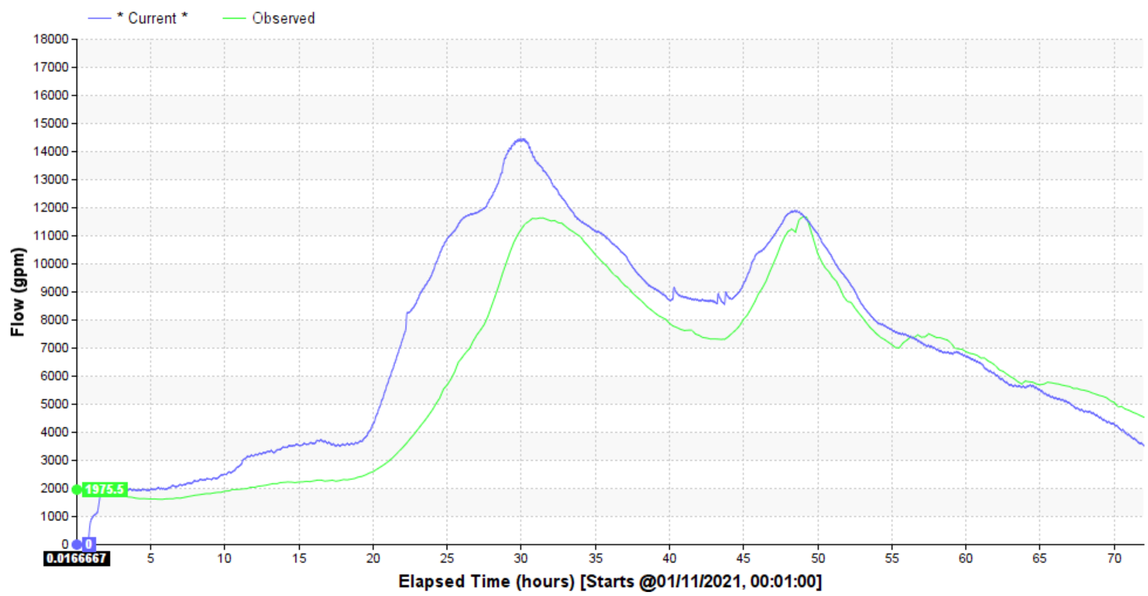
WWTP Flow – Calibration Period 1 (Jan 2nd – 4th)

Pipe 1180 (Run/Measured Volumes : 3851679.11 / 3792653.81 ft3)



WWTP flow – Calibration Period 2 (Jan 11th – 13th)

Pipe 1180 (Run/Measured Volumes : 4085072.04 / 3503366.57 ft3)



Appendix G

Consequence of Failure Analysis

KELLER ASSOCIATES

Consequence of Failure

Size of Lift Station	
Design Flow > 500 gpm	1.5
250 gpm < Design Flow < 500 gpm	1
Design Flow < 250 gpm	0.5

Environmentally Sensitive Areas	
Wetwell overflows to storm system	1
Wetwell located adjacent to wetland/overflows to wetland/creeks	2

Service Parameter	
Critical Government Infrastructure (emergency services/police/fire/etc.)	2
School/Hospital	2
Commercial/Industrial zone	1
Historic Site	1

Proximity to Private Property	
Within 100 feet of private property (high chance of flooding to private property)	2
Between 100 and 250 feet of private property	1
Greater than 250 feet (or low chance of flooding to private property)	0

Portion of Community Served	
>100 EDUs served	3
50-100 EDUs served	2
5-50 EDUs served	1
<5 EDUs served	0

Estimate of Time to Overflow	
Very High Risk (wetwell overflows before pipe surcharges)	3
High Risk (wetwell fills quickly)	2
Moderate Risk	1
Low Risk (wetwell fills slowly)	0

Likelihood of Failure

Liquification Hazard	
High	2
Medium	1
Low	0.5

Backup Power	
No on-site backup power available	1
On-site backup power available	0

Capacity vs. Demand	
Over firm capacity as indicated by runtime	2
Likely over firm capacity as indicated by runtime	1
Under firm capacity as indicated by runtime	0

Landslide Susceptibility	
Very High	3
High	2
Moderate	1
Low	0

Wetwell/ Pipe Condition	
Poor Condition (cracked/broken concrete, disconnected/broken pumps)	2
Moderate Condition (FOG buildup, wear on concrete/electronics/pumps)	1
Good Condition (no concrete damage, operable pipes, no root intrusion)	0

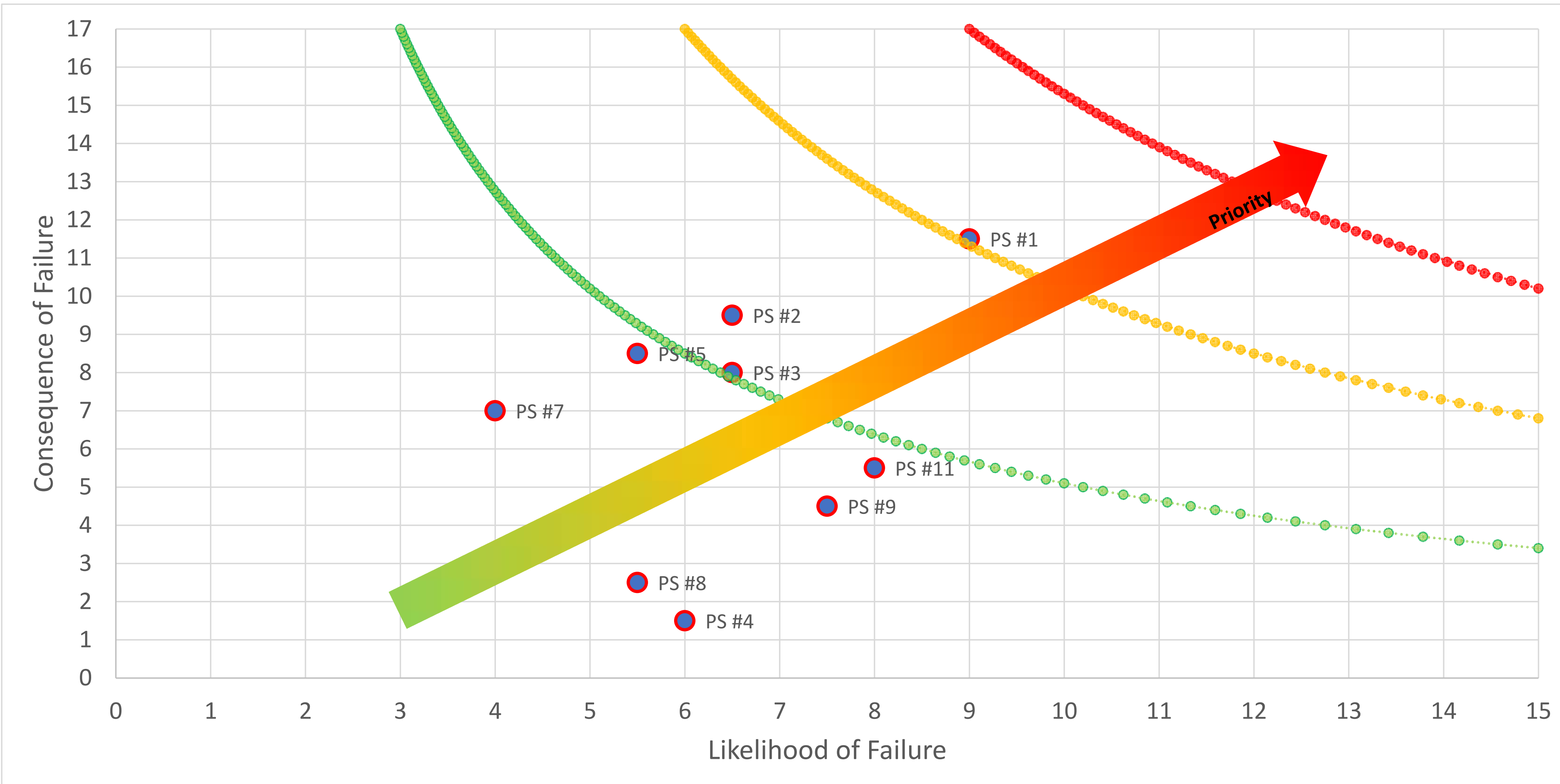
Safety/ Security/ Access	
No safety barrier/ fence	0.5
Difficult to access/repair in an emergency/susceptible to outside damage (traffic)	0.5
Lack of fall protection	0.5

Age	
If Age > 25 years old	2
If Age is between 10 and 25 years old, mechanical updated in last 10 years	1
If Age < 10 years old	0

Sensor and Alarm Redundancy	
No redundancy in level sensors	0.5
Level sensor redundancy	0

Influence from Flooding	
Within 100-year floodplain	1
Outside of 100-year floodplain	0

	Consequence of Failure									Likelihood of Failure										
PS Name	Size of Lift Station	Commercial/ Industry Zone?	School/ Hospital / Critical Gov. Infrastructure/ Historic Site	Portion of Community Served	Environmentally Sensitive Areas	Proximity to Private Property	Estimate of Time to Overflow	Consequence Sum	PS Name	Liquification Hazard	Landslide Susceptibility	Age	Backup Power	Wetwell/ Pipe Condition	Sensor and Alarm Redundancy	Capacity vs. Demand	Safety/ Security/ Access	Influence from Flooding	Likelihood Sum	Risk of Failure
PS #1	1.5	1	1	3	1	2	2	11.5	PS #1	2	0	2	1	1	0	2	1	0	9	104
PS #2	1.5	1	0	3	0	2	2	9.5	PS #2	2	1	2	0	0	0	1	0.5	0	6.5	62
PS #3	1	1	0	1	1	2	2	8	PS #3	0	0	1	1	1	0	2	1.5	0	6.5	52
PS #4	0.5	0	0	0	1	0	0	1.5	PS #4	0	0	1	1	1	0.5	1	1.5	0	6	9
PS #5	0.5	0	0	3	2	0	3	8.5	PS #5	0	1	2	0	0	0	2	0.5	0	5.5	47
PS #7	1	1	0	1	2	0	2	7	PS #7	0	1	1	0	0	0	1	0	1	4	28
PS #8	0.5	0	0	0	2	0	0	2.5	PS #8	0	0	2	1	0	0.5	0	1	1	5.5	14
PS #9	0.5	0	0	0	2	1	1	4.5	PS #9	2	1	1	1	1	0.5	0	0	1	7.5	34
PS #11	0.5	0	0	2	2	0	1	5.5	PS #11	2	1	1	1	1	0.5	0	1.5	0	8	44



Consequence of Failure
Likelihood of Failure
Priority
PS #1
PS #2
PS #5
PS #3
PS #7
PS #11
PS #9
PS #8
PS #4
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17

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Appendix H

Sump Pump Supplemental Material

KELLER ASSOCIATES

Function of Sump Pumps & Downspouts

Rainwater can enter the basement through many sources. The job of a sump pump is to divert the water from inside your basement to a location outside of the house. A sump pump is usually installed in a sump pit which stores the water. When this water reaches a certain level, it triggers the sump pump which pumps the water back outside, away from the house. A downspout's purpose is to direct water from the roof gutters away from the house.

The Problem of Inflow

Inflow is caused by improperly connected foundation (footing) drains, sump pumps, and downspouts. Instead of directing the clear rain water outside and away from the house, it directs the water into the sanitary sewer system. Inflow is a problem because it creates an extra water burden for the sanitary sewer system, and when this system is overloaded, sewage can back up into our streets, buildings, and your home. It also means that our utility bills are higher because we are collectively paying for the unnecessary treatment of clean water!

Rules and Regulations

Inflow is a problem for all of Delaware County's communities and sanitary sewer systems. All municipalities have adopted ordinances which make it illegal to have improper connections to the sanitary sewer. Fees and other enforcement measures can be used to achieve compliance. To avoid fines make sure your sump pumps and downspouts discharge properly.

Homeowners have an impact on preventing or causing the problem of inflow. Your community and neighbors are relying on you to take responsibility for making sure that your connections are not contributing to the problem.

For more information regarding what is being done about inflow in your community, contact your local municipality or sewer authority.

DELCORA

DELAWARE COUNTY REGIONAL WATER QUALITY CONTROL AUTHORITY
100 EAST FIFTH STREET
CHESTER, PA 19013
WWW.DELCORA.ORG
610-876-5523

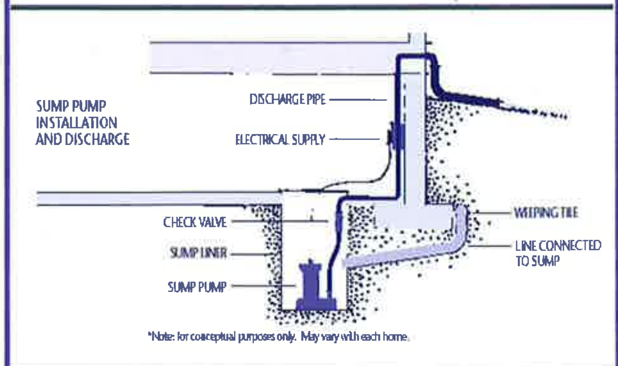
PRINTED WITH THANKS TO THE:
FARIBAULT COUNTY
SOIL AND WATER CONSERVATION DISTRICT
BLUE EARTH, MN 56013
www.faribaultcountyswcd.com

Disconnecting & Redirecting Your Sump Pump & Downspouts

In wet weather it only takes a few improperly connected sump pumps to cause a sanitary sewer backup into basements, streets and waterways.

How Do I Know If My Sump Pump Is Improperly Connected?

Your sump pump is improperly connected to the sanitary sewer if it is connected to the drain or sink in your basement. Unless you are sure your basement drain is not connected to the sanitary sewer, your sump pump is probably improperly connected.



Proper sump pump discharge connections are to the outside of the house only!

How Do I Know If My Downspout Is Improperly Connected?

If your downspouts disappear into the ground rather than discharging into your yard, they may be connected to the sanitary sewer. While connections to the *storm* sewer are permitted, connections to the *sanitary* sewer must be disconnected and redirected.

Downspouts that look like this could be connected to the sanitary sewer.

Disconnecting Your Sump Pump

If your sump pump discharges to the sanitary system in any way, the discharge must be re-directed out of the sanitary sewer system. The change could be as simple as directing the discharge outside the house through a hose. If you aren't familiar with the work, contact a plumbing professional, your local municipality, or your sewer authority for more information.

Each household or business that redirects their stormwater out of the sanitary sewer helps solve the problem of sewage backing up into basements, streets, and waterways.

Disconnecting Your Downspout

Disconnecting your downspout from the sanitary sewer is easy to do yourself.

1. Cut the downspout, leaving enough space to insert the elbow.

2. Tightly cap the end of the pipe sticking out of the ground that leads to the sanitary sewer.

3. Attach an elbow to the end of the downspout and use an appropriate extension to direct the water away from your home.

Where Should I Direct the Flow of My Disconnected Sump Pump and Downspout?

Water should be discharged away from your house or it may seep back into your basement. It should flow to an area where it can seep into the ground or be stored for later use. Direct flow to:

Raingarden

Lawn

Trees

Rain Barrel

Never direct stormwater into a sanitary sewer or onto a neighboring property!

Code of the Town of Derry Sewer Use Ordinance

ARTICLE V Use of Public Sewers

§ 122-30. Discharge of certain waters to sanitary sewer prohibited.

No person shall discharge or cause to be discharged any stormwater, surface water, groundwater, roof runoff, subsurface drainage, cooling water or unpolluted industrial process waters to any sanitary sewer.

§ 122-31. Discharge to storm sewer or natural outlet.

A. Stormwater and all other unpolluted drainage shall be discharged to drains or such sewers as are specifically designated as storm sewers or to a natural outlet approved by the Town.

COMPLIANCE The DPW conducts flow monitoring of areas in the sewer collection system throughout Town identifying suspected areas of sump pump connections. Once an area is identified, video inspection of the sewer mains may be conducted and random inspections made to locate source of stormwater inflow including illicit sump pump connections.

Residents who have any questions or need any assistance in disconnecting their sump pump may call the Derry DPW or their local plumber. By working together we can keep our costs down and reduce risk of damage to other homes and the Town's sanitary sewer facilities.

Town of Derry, NH

Department of Public Works
Derry Municipal Center

Michael A. Fowler, P.E. Director
Thomas A. Carrier, Deputy Director, Water and Sewer Divisions

Phone: 603-432-6144
Fax: 603-432-6130
E-mail: tomcarrier@derrynh.org

«Owner»
«Owner Addr»
« City», «St» «Zip»

TOWN OF DERRY, NH

Guide to Sump Pump Connections

SUMP PUMP CONNECTIONS TO THE MUNICPAL SEWER SYSTEM IS ILLEGAL!

Prepared by:
The Town of Derry
Department of Public Works

SUMP PUMP DISCHARGE REQUIREMENTS

Sump pumps remove groundwater from below building foundations to prevent water damage to the building. Groundwater collected by sump pumps must discharge to the ground surface outside of the building, to a stormwater drain, or to a natural outlet. If your sump pump is frequently operating, rains may have caused the groundwater to rise and flow into the sump pump pit. In some cases, the groundwater may remain high and cause the sump pump to run continually.

Town sewers are not designed to carry the additional flow from sump pumps. An overloaded sewer can create sewer backups in the streets and other homeowner's basements. Also, the groundwater from

the sump pumps would be pumped and treated at the Town's Wastewater treatment facility. The additional flow uses up plant capacity and increases the costs of treatment and in some cases can cause the plant to overflow.

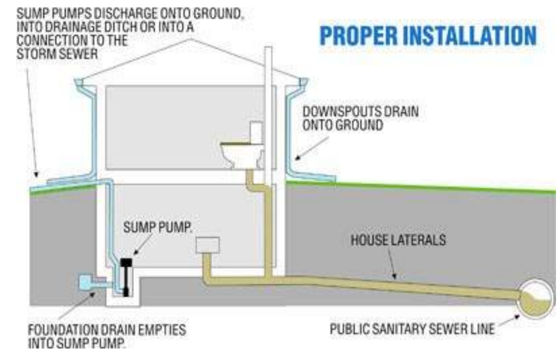
PROHIBITED LOCATIONS FOR SUMP PUMP DISCHARGE

DO NOT connect your sump pump to the sanitary sewer pipes. <u>It is illegal to discharge groundwater from the sump pump to the sanitary sewer.</u> If your sump pump is connected to any other pipe in your home, it is most likely connected incorrectly to the Town sewer system. Such connections are a violation of local Ordinance.

DO NOT pump storm water onto sidewalks or streets. Sump pump water draining onto walkways and streets can cause icy, unsafe conditions as well as reduce the life of the street surface and the curb increasing the Town's maintenance costs .

DO NOT pump storm water onto your neighbors property as this can be a nuisance and result in property damage.

ACCEPTABLE LOCATIONS FOR SUMP PUMP DISCHARGE



The pipe from your basement sump pump should always discharge directly into your yard or stormwater drainage system.

Water should be directed into your yard away from your home so that it doesn't puddle along the wall and seep back into your basement.

Sump Pump Discharge hoses may be connected to the Town's drainage system. Residents MUST FIRST contact the DPW for permission and guidance

Appendix I

Alternatives Cost Analysis

Collection System Project: Basin 1 - Pipeline Upsize
Project Identifier: 1.a

Objective: Resolve undersized pipeline in Basin 1. Construct gravity pipeline capable of conveying anticipated peak hour flows.

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Gravity Pipeline Upszie				
18-inch Pipe - Excavation, Backfill	230	LF	$ 185	$ 42,550
15-inch Pipe - Excavation, Backfill	2,330	LF	$ 170	$ 396,100
Roadway Restoration (Half Lane)	1,315	LF	$ 45	$ 59,180
Landscape Restoration	1,245	LF	$ 20	$ 24,900
Traffic Control w/ Flagging	1	LS	$ 62,000	$ 62,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
48-inch Manhole	8	EA	$ 8,000	$ 64,000
Bypass Pumping - Pipes 24-inch and smaller	2,560	LF	$ 22	$ 56,810
ADA Ramp Reconstruction (Compliance)	6	EA	$ 4,600	$ 27,600
Rock Excavation	589	CY	$ 300	$ 176,770
Replace Service Laterals	18	EA	$ 1,500	$ 27,000
Existing Utility Protection	2,560	LF	$ 4	$ 10,240
			Subtotal (rounded)	***$ 951,000***
Mobilization	1	LS	5%	$ 48,000
Contingency	1	LS	30%	$ 300,000
			Construction Subtotal (rounded)	***$ 1,299,000***
Engineering and CMS	1	LS	20%	$ 260,000
Permitting (Assume 8% of total)	1	LS	$ 103,900	$ 103,900
Geotechnical (Assume 1% of total)	1	LS	$ 13,000	$ 13,000
Surveying	1	LS	$ 40,000	$ 40,000
Legal and Admin	1	LS	$ 20,000	$ 20,000
			Total Project Cost (rounded)	***$ 1,800,000***

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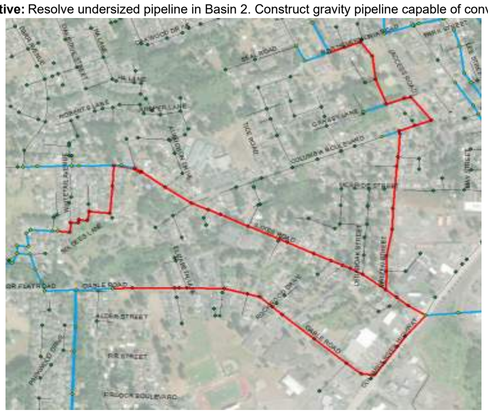
Collection System Project: Basin 2 - Pipeline Upsize
Project Identifier: 2.a

Objective: Resolve undersized pipeline in Basin 2. Construct gravity pipeline capable of conveying anticipated peak hour flows.



Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Sykes Road - Gravity Upsize				
12-inch Pipe - Excavation, Backfill	315	LF	$ 160	$ 50,400
15-inch Pipe - Excavation, Backfill	2,645	LF	$ 170	$ 449,650
18-inch Pipe - Excavation, Backfill	522	LF	$ 185	$ 96,570
Roadway Restoration (Half Lane)	3,482	LF	$ 45	$ 156,690
Traffic Control w/ Flagging	1	LS	$ 65,000	$ 65,000
48-Inch, Standard Manhole	15	EA	$ 8,000	$ 120,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Bypass Pumping - Pipes 24-inch and smaller	3,482	LF	$ 22	$ 77,270
Rock Excavation	786	CY	$ 300	$ 235,760
ADA Ramp Reconstruction (Compliance)	11	EA	$ 4,600	$ 50,600
Replace Service Laterals	55	EA	$ 1,500	$ 82,500
Existing Utility Protection	2,960	LF	$ 4	$ 11,840
			Subtotal	***$ 1,400,000***
Matzen St. - Gravity Pipeline Upsize				
12-inch PVC gravity pipe	1,450	LF	$ 160	$ 232,000
15-inch PVC gravity pipe	2,600	LF	$ 170	$ 442,000
Roadway Restoration (Half Lane)	2,980	LF	$ 45	$ 134,100
Landscape Restoration	1,070	LF	$ 20	$ 21,400
Traffic Control w/ Flagging	1	LS	$ 110,000	$ 110,000
48-Inch, Standard Manhole	16	EA	$ 8,000	$ 128,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Bypass Pumping - Pipes 24-inch and smaller	4,050	LF	$ 22	$ 89,880
Rock Excavation	792	CY	$ 300	$ 237,540
ADA Ramp Reconstruction (Compliance)	9	EA	$ 4,600	$ 41,400
Replace Service Laterals	42	EA	$ 1,500	$ 63,000
Existing Utility Protection	4,050	LF	$ 4	$ 16,200
			Subtotal	***$ 1,519,000***
Gable Rd. - Gravity Pipeline Upsize and Reroute to Sykes Rd.				
12-inch PVC gravity pipe	4,050	LF	$ 160	$ 648,000
Roadway Restoration (Half Lane)	4,050	LF	$ 45	$ 182,250
Traffic Control w/ Flagging	1	LS	$ 105,000	$ 105,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
48-Inch, Standard Manhole	13	EA	$ 8,000	$ 104,000
Existing Utility Protection	4,050	LF	$ 4	$ 16,200
Bypass Pumping - Pipes 24-inch and smaller	4,050	LF	$ 22	$ 89,880
ADA Ramp Reconstruction (Compliance)	11	EA	$ 4,600	$ 50,600
Replace Service Laterals	32	EA	$ 1,500	$ 48,000
Rock Excavation	593	CY	$ 300	$ 177,960
			Subtotal	***$ 1,425,000***
Westshire Ln. - Gravity Pipeline Upsize				
10-inch PVC gravity pipe	250	LF	$ 150	$ 37,500
12-inch PVC gravity pipe	2,300	LF	$ 160	$ 368,000
Roadway Restoration (Half Lane)	2,235	LF	$ 45	$ 100,580
Landscape Restoration	315	LF	$ 20	$ 6,300
Traffic Control w/ Flagging	1	LS	$ 87,000	$ 87,000
48-Inch, Standard Manhole	16	EA	$ 8,000	$ 128,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Bypass Pumping - Pipes 24-inch and smaller	2,550	LF	$ 22	$ 56,590
Rock Excavation	381	CY	$ 300	$ 114,370
ADA Ramp Reconstruction (Compliance)	4	EA	$ 4,600	$ 18,400
Replace Service Laterals	25	EA	$ 1,500	$ 37,500
Existing Utility Protection	2,550	LF	$ 4	$ 10,200
			Subtotal	***$ 968,000***
			Subtotal (rounded)	***$ 5,312,000***
Mobilization	1	LS	5%	$ 266,000
Contingency	1	LS	30%	$ 1,674,000
			Construction Subtotal (rounded)	***$ 7,252,000***
Engineering and CMS	1	LS	20%	$ 1,451,000
Permitting (Assume 8% of total)	1	LS	$ 580,200	$ 580,200
Geotechnical (Assume 1% of total)	1	LS	$ 73,000	$ 73,000
Surveying	1	LS	$ 100,000	$ 100,000
Legal and Admin	1	LS	$ 40,000	$ 40,000
			Total Project Cost (rounded)	***$ 9,500,000***

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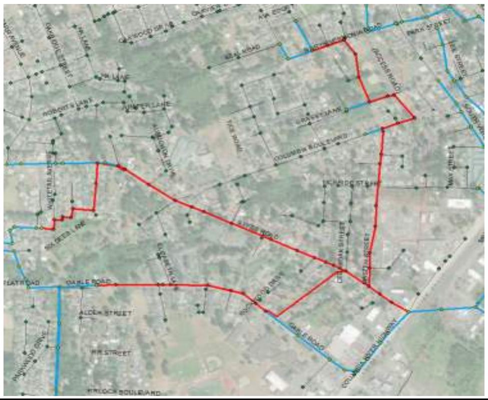
Collection System Project: Basin 2 - Pipeline Upsize and Reroute from Gable Rd. to Sykes Rd.
Project Identifier: 2.b

Objective: Resolve undersized pipeline in Basin 2. Construct gravity pipeline capable of conveying anticipated peak hour flows.



Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Sykes Road - Gravity Upsize				
12-inch Pipe - Excavation, Backfill	315	LF	$ 160	$ 50,400
15-inch Pipe - Excavation, Backfill	2,645	LF	$ 170	$ 449,650
18-inch Pipe - Excavation, Backfill	522	LF	$ 185	$ 96,570
Roadway Restoration (Half Lane)	3,482	LF	$ 45	$ 156,690
Traffic Control w/ Flagging	1	LS	$ 65,000	$ 65,000
48-Inch, Standard Manhole	15	EA	$ 8,000	$ 120,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Bypass Pumping - Pipes 24-inch and smaller	3,482	LF	$ 22	$ 77,270
Rock Excavation	786	CY	$ 300	$ 235,760
ADA Ramp Reconstruction (Compliance)	11	EA	$ 4,600	$ 50,600
Replace Service Laterals	55	EA	$ 1,500	$ 82,500
Existing Utility Protection	2,960	LF	$ 4	$ 11,840
			Subtotal	***$ 1,400,000***
Matzen St. - Gravity Pipeline Upsize				
12-inch Pipe - Excavation, Backfill	1,450	LF	$ 160	$ 232,000
15-inch Pipe - Excavation, Backfill	2,600	LF	$ 170	$ 442,000
Roadway Restoration (Half Lane)	2,980	LF	$ 45	$ 134,100
Landscape Restoration	1,070	LF	$ 20	$ 21,400
Traffic Control w/ Flagging	1	LS	$ 110,000	$ 110,000
48-Inch, Standard Manhole	16	EA	$ 8,000	$ 128,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Bypass Pumping - Pipes 24-inch and smaller	4,050	LF	$ 22	$ 89,880
Rock Excavation	792	CY	$ 300	$ 237,540
ADA Ramp Reconstruction (Compliance)	9	EA	$ 4,600	$ 41,400
Replace Service Laterals	42	EA	$ 1,500	$ 63,000
Existing Utility Protection	4,050	LF	$ 4	$ 16,200
			Subtotal	***$ 1,519,000***
Gable Rd. - Gravity Pipeline Upsize and Reroute to Sykes Rd.				
12-inch Pipe - Excavation, Backfill	3,000	LF	$ 160	$ 480,000
Roadway Restoration (Half Lane)	3,000	LF	$ 45	$ 135,000
Traffic Control w/ Flagging	1	LS	$ 87,000	$ 87,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
48-Inch, Standard Manhole	13	EA	$ 8,000	$ 104,000
Existing Utility Protection	3,000	LF	$ 4	$ 12,000
Bypass Pumping - Pipes 24-inch and smaller	3,000	LF	$ 22	$ 66,580
ADA Ramp Reconstruction (Compliance)	3	EA	$ 4,600	$ 13,800
Replace Service Laterals	23	EA	$ 1,500	$ 34,500
Rock Excavation	1,333	CY	$ 300	$ 400,000
			Subtotal	***$ 1,336,000***
Westshire Ln. - Gravity Pipeline Upsize				
10-inch Pipe - Excavation, Backfill	250	LF	$ 150	$ 37,500
12-inch Pipe - Excavation, Backfill	2,300	LF	$ 160	$ 368,000
Roadway Restoration (Half Lane)	2,235	LF	$ 45	$ 100,580
Landscape Restoration	315	LF	$ 20	$ 6,300
Traffic Control w/ Flagging	1	LS	$ 87,000	$ 87,000
48-Inch, Standard Manhole	16	EA	$ 8,000	$ 128,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Bypass Pumping - Pipes 24-inch and smaller	2,550	LF	$ 22	$ 56,590
Rock Excavation	381	CY	$ 300	$ 114,370
ADA Ramp Reconstruction (Compliance)	4	EA	$ 4,600	$ 18,400
Replace Service Laterals	25	EA	$ 1,500	$ 37,500
Existing Utility Protection	2,550	LF	$ 4	$ 10,200
			Subtotal	***$ 968,000***
			Subtotal (rounded)	***$ 5,223,000***
Mobilization	1	LS	5%	$ 262,000
Contingency	1	LS	30%	$ 1,646,000
			Construction Subtotal (rounded)	***$ 7,131,000***
Engineering and CMS	1	LS	20%	$ 1,427,000
Permitting (Assume 8% of total)	1	LS	$ 570,500	$ 570,500
Geotechnical (Assume 1% of total)	1	LS	$ 71,000	$ 71,000
Surveying	1	LS	$ 100,000	$ 100,000
Legal and Admin	1	LS	$ 40,000	$ 40,000
			Total Project Cost (rounded)	***$ 9,400,000***

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Collection System Project: Basin 3 - Pipeline Upsize
Project Identifier: 3.a

Objective: Resolve undersized pipeline in Basin 3. Construct gravity pipeline capable of conveying anticipated peak hour flows.

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Gravity Pipeline Upszie				
15-inch Pipe - Excavation, Backfill	1,550	LF	$ 170	$ 263,500
Roadway Restoration (Half Lane)	922	LF	$ 45	$ 41,490
Soil Surface Repair	628	LF	$ 5	$ 3,140
Traffic Control w/ Flagging	1	LS	$ 47,000	$ 47,000
48-Inch, Standard Manhole	8	EA	$ 8,000	$ 64,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
ADA Ramp Reconstruction (Compliance)	4	EA	$ 4,600	$ 18,400
Replace Service Laterals	25	EA	$ 1,500	$ 37,500
Bypass Pumping - Pipes 24-inch and smaller	1,550	LF	$ 22	$ 34,400
Rock Excavation	332	CY	$ 300	$ 99,490
Existing Utility Protection	1,550	LF	$ 4	$ 6,200
			Subtotal (rounded)	***$ 619,000***
Mobilization	1	LS	5%	$ 31,000
Contingency	1	LS	30%	$ 195,000
			Construction Subtotal (rounded)	***$ 845,000***
Engineering and CMS	1	LS	20%	$ 169,000
Permitting (Assume 8% of total)	1	LS	$ 67,600	$ 67,600
Geotechnical (Assume 1% of total)	1	LS	$ 8,000	$ 8,000
Surveying	1	LS	$ 20,000	$ 20,000
Legal and Admin	1	LS	$ 10,000	$ 10,000
			Total Project Cost (rounded)	***$ 1,200,000***

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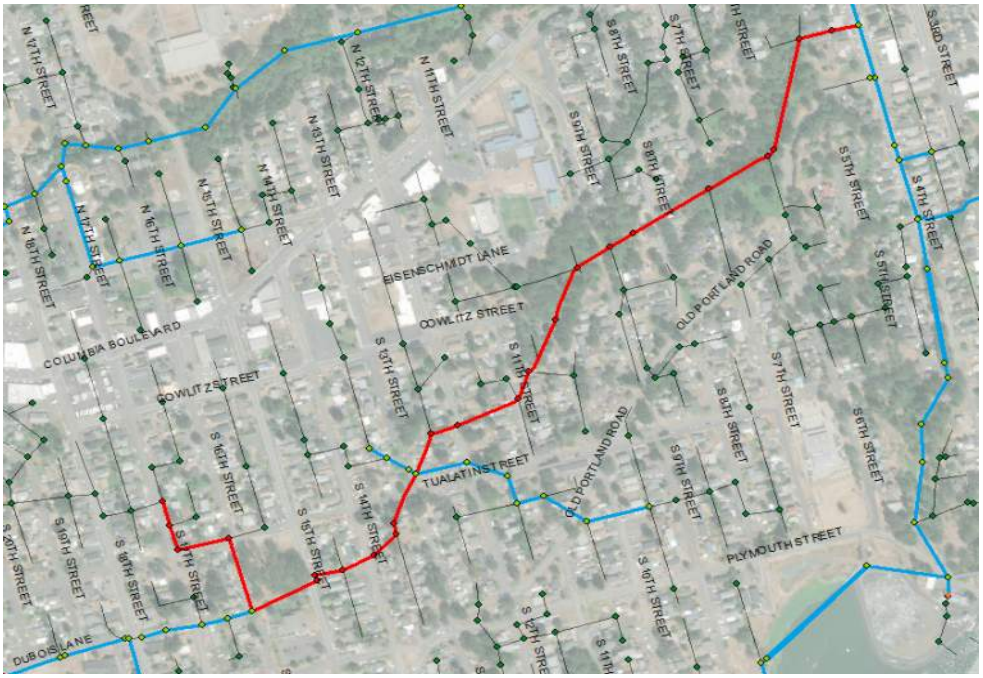
Collection System Project: Basin 4 - Pipeline Upsize
Project Identifier: 4.a

Objective: Resolve undersized pipeline in Basin 4. Construct gravity pipeline capable of conveying anticipated peak hour flows.



Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Gravity Pipeline Upszie				
12-inch Pipe - Excavation, Backfill	860	LF	$ 160	$ 137,600
15-inch Pipe - Excavation, Backfill	1,100	LF	$ 170	$ 187,000
18-inch Pipe - Excavation, Backfill	2,400	LF	$ 185	$ 444,000
21-inch Pipe - Excavation, Backfill	830	LF	$ 195	$ 161,850
Roadway Restoration (Half Lane)	850	LF	$ 45	$ 38,250
Landscape Restoration	4,340	LF	$ 20	$ 86,800
Traffic Control w/out Flagging	4,090	LF	$ 6	$ 24,540
Traffic Control w/ Flagging	1	LS	$ 101,000	$ 101,000
48-Inch, Standard Manhole	30	EA	$ 8,000	$ 240,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Existing Utility Protection	5,190	LF	$ 4	$ 20,760
Replace Service Laterals	42	EA	$ 1,500	$ 63,000
Bypass Pumping - Pipes 24-inch and smaller	5,190	LF	$ 22	$ 115,180
Rock Excavation	1,417	CY	$ 300	$ 425,070
			Subtotal (rounded)	***$ 2,049,000***
Mobilization	1	LS	5%	$ 103,000
Contingency	1	LS	30%	$ 646,000
			Construction Subtotal (rounded)	***$ 2,798,000***
Engineering and CMS	1	LS	20%	$ 560,000
Permitting (Assume 8% of total)	1	LS	$ 223,800	$ 223,800
Geotechnical (Assume 1% of total)	1	LS	$ 28,000	$ 28,000
Surveying	1	LS	$ 50,000	$ 50,000
Legal and Admin	1	LS	$ 20,000	$ 20,000
			Total Project Cost (rounded)	***$ 3,700,000***

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Collection System Project: Basin 4 - Pipeline Upsize and Reroute from Tualatin St. to Basin 6
Project Identifier: 4.b

Objective: Resolve undersized pipeline in Basin 4. Construct gravity pipeline capable of conveying anticipated peak hour flows.

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Gravity Pipeline Upszie				
12-inch Pipe - Excavation, Backfill	860	LF	$ 160	$ 137,600
15-inch Pipe - Excavation, Backfill	3,830	LF	$ 170	$ 651,100
Roadway Restoration (Half Lane)	3,140	LF	$ 45	$ 141,300
Landscape Restoration	1,550	LF	$ 20	$ 31,000
Traffic Control w/out Flagging	860	LF	$ 6	$ 5,160
Traffic Control w/ Flagging	1	LS	$ 122,000	$ 122,000
48-Inch, Standard Manhole	17	EA	$ 8,000	$ 136,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Existing Utility Protection	4,690	LF	$ 4	$ 18,760
Replace Service Laterals	25	EA	$ 1,500	$ 37,500
Bypass Pumping - Pipes 24-inch and smaller	3,160	LF	$ 22	$ 70,130
Rock Excavation	2,114	CY	$ 300	$ 634,330
			Subtotal (rounded)	***$ 1,989,000***
Mobilization	1	LS	5%	$ 100,000
Contingency	1	LS	30%	$ 627,000
			Construction Subtotal (rounded)	***$ 2,716,000***
Engineering and CMS	1	LS	20%	$ 544,000
Permitting (Assume 8% of total)	1	LS	$ 217,300	$ 217,300
Geotechnical (Assume 1% of total)	1	LS	$ 27,000	$ 27,000
Surveying	1	LS	$ 50,000	$ 50,000
Legal and Admin	1	LS	$ 20,000	$ 20,000
			Total Project Cost (rounded)	***$ 3,600,000***

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Collection System Project: Basin 5 - Pipeline Upsize
Project Identifier: 5.a

Objective: Resolve undersized pipeline in Basin 5. Construct gravity pipeline capable of conveying anticipated peak hour flows.

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Gravity Pipeline Upszie				
36-inch Pipe - Excavation, Backfill	470	LF	$ 245	$ 115,150
42-inch Pipe - Excavation, Backfill	2,850	LF	$ 275	$ 783,750
Roadway Restoration (Full Lane)	2,185	LF	$ 75	$ 163,880
Landscape restoration	1,135	LF	$ 20	$ 22,700
Traffic Control w/ Flagging	1	LS	$ 92,000	$ 92,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
72-Inch, Standard Manhole	14	EA	$ 16,500	$ 231,000
Existing Utility Protection	3,320	LF	$ 4	$ 13,280
ADA Ramp Reconstruction (Compliance)	8	EA	$ 4,600	$ 36,800
Replace Service Laterals	27	EA	$ 1,500	$ 40,500
Bypass Pumping - Pipes larger than 24-inch	3,320	LF	$ 27	$ 91,020
Rock Excavation	2,906	CY	$ 300	$ 871,920
Tunnel Bore	475	LF	$ 400	$ 200,000
			Subtotal (rounded)	***$ 2,666,000***
Mobilization	1	LS	5%	$ 134,000
Contingency	1	LS	30%	$ 840,000
			Construction Subtotal (rounded)	***$ 3,640,000***
Engineering and CMS	1	LS	20%	$ 728,000
Permitting	1	LS	$ 15,000	$ 15,000
Geotechnical (Assume 1% of total)	1	LS	$ 36,400	$ 36,400
Surveying	1	LS	$ 30,000	$ 30,000
Legal and Admin	1	LS	$ 10,000	$ 10,000
			Total Project Cost (rounded)	***$ 4,500,000***

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Collection System Project: Basin 6 - Pipeline Upsize
Project Identifier: 6.a

Objective: Resolve undersized pipeline in Basin 6. Construct gravity pipeline capable of conveying anticipated peak hour flows.

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Port Ave - Gravity Upsize				
27-inch Pipe - Excavation, Backfill	3,030	LF	$ 220	$ 666,600
Roadway Restoration (Half Lane)	2,775	LF	$ 45	$ 124,880
Landscape Restoration	255	LF	$ 20	$ 5,100
Traffic Control w/ Flagging	1	LS	$ 76,000	$ 76,000
60-Inch, Standard Manhole	9	EA	$ 14,000	$ 126,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Railroad Boring	140	LF	$ 900	$ 126,000
Highway Permitting	1	LS	$ 5,000	$ 5,000
Replace Service Laterals	8	EA	$ 1,500	$ 12,000
Bypass Pumping - Pipes 24-inch and smaller	3,030	LF	$ 22	$ 67,240
Rock Excavation	1,546	CY	$ 300	$ 463,680
Existing Utility Protection	3,030	LF	$ 4	$ 12,120
			Subtotal	***$ 1,688,000***
S 18th St, Dubois Lane, Columbia River HWY - Gravity Pipeline Upsize				
15-inch Pipe - Excavation, Backfill	4,220	LF	$ 170	$ 717,400
Roadway Restoration (Half Lane)	3,069	LF	$ 45	$ 138,110
Landscape Restoration	760	LF	$ 20	$ 15,200
Highway Permitting	1	LS	$ 5,000	$ 5,000
ODOT Roadway Restoration (Full Lane)	391	LF	$ 225	$ 87,980
Traffic Control w/ Flagging	1	LS	$ 114,000	$ 114,000
Railroad Boring	80	LF	$ 900	$ 72,000
48-Inch, Standard Manhole	16	EA	$ 8,000	$ 128,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
ADA Ramp Reconstruction (Compliance)	10	EA	$ 4,600	$ 46,000
Replace Service Laterals	47	EA	$ 1,500	$ 70,500
Bypass Pumping - Pipes 24-inch and smaller	4,220	LF	$ 22	$ 93,650
Rock Excavation	903	CY	$ 300	$ 270,880
Existing Utility Protection	4,220	LF	$ 4	$ 16,880
			Subtotal	***$ 1,779,000***
Old Portland Rd., Umatilla St. - Gravity Pipeline Upsize				
21-inch Pipe - Excavation, Backfill	1,420	LF	$ 195	$ 276,900
Roadway Restoration (Half Lane)	1,420	LF	$ 45	$ 63,900
Landscape Restoration	375	LF	$ 20	$ 7,500
Traffic Control w/ Flagging	1	LS	$ 24,000	$ 24,000
48-Inch, Standard Manhole	4	EA	$ 8,000	$ 32,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Existing Utility Protection	1,420	LF	$ 4	$ 5,680
Rock Excavation	557	CY	$ 300	$ 167,010
			Subtotal	***$ 580,000***
Southern Trunkline - Gravity Pipeline Upsize				
30-inch Pipe - Excavation, Backfill	420	LF	$ 230	$ 96,600
33-inch Pipe - Excavation, Backfill	1,185	LF	$ 240	$ 284,400
36-inch Pipe - Excavation, Backfill	2,325	LF	$ 245	$ 569,600
36-inch Pipe - Excavation, Backfill, Structure over Lagoon	645	LF	$ 1,170	$ 754,700
Roadway Restoration (Half Lane)	50	LF	$ 45	$ 2,300
Traffic Control w/ Flagging	1	LS	$ 6,000	$ 6,000
72-Inch, Standard Manhole	15	EA	$ 16,500	$ 247,500
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Bypass Pumping - Pipes larger than 24-inch	4,575	LF	$ 27	$ 125,400
Rock Excavation	3,102	CY	$ 300	$ 930,600
Existing Utility Protection	4,575	LF	$ 4	$ 18,300
			Subtotal	***$ 3,039,000***
			Subtotal (rounded)	***$ 7,086,000***
Mobilization	1	LS	5%	$ 355,000
Contingency	1	LS	30%	$ 2,233,000
			Construction Subtotal (rounded)	***$ 9,674,000***
Engineering and CMS	1	LS	20%	$ 1,935,000
Permitting (Assume 5% of total)	1	LS	$ 483,700	$ 483,700
Geotechnical (Assume 1% of total)	1	LS	$ 96,740	$ 96,740
Surveying	1	LS	$ 60,000	$ 60,000
Legal and Admin	1	LS	$ 30,000	$ 30,000
			Total Project Cost (rounded)	***$ 12,300,000***

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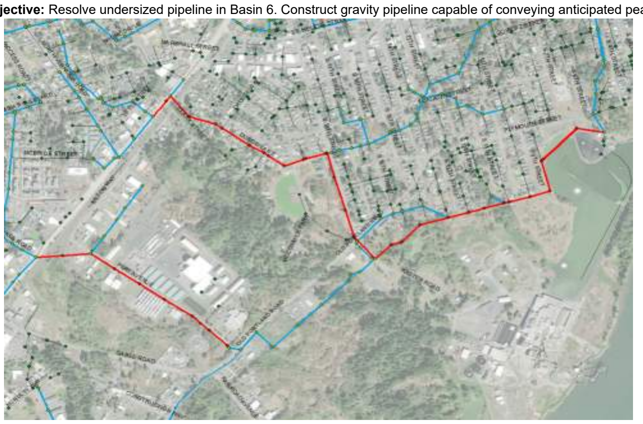
Collection System Project: Basin 6 - Pipeline Upsize and Reroute from Old Portland Rd to Kaster Rd.
Project Identifier: 6.b

Objective: Resolve undersized pipeline in Basin 6. Construct gravity pipeline capable of conveying anticipated peak hour flows.



Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Port Ave - Gravity Upsize				
27-inch Pipe - Excavation, Backfill	3,030	LF	$ 220	$ 666,600
Roadway Restoration (Half Lane)	2,775	LF	$ 45	$ 124,880
Landscape Restoration	255	LF	$ 20	$ 5,100
Traffic Control w/ Flagging	1	LS	$ 76,000	$ 76,000
60-Inch, Standard Manhole	9	EA	$ 14,000	$ 126,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Railroad Boring	140	LF	$ 900	$ 126,000
Highway Permitting	1	LS	$ 5,000	$ 5,000
Replace Service Laterals	8	EA	$ 1,500	$ 12,000
Bypass Pumping - Pipes 24-inch and smaller	3,030	LF	$ 22	$ 67,240
Rock Excavation	1,546	CY	$ 300	$ 463,680
Existing Utility Protection	3,030	LF	$ 4	$ 12,120
			Subtotal	***$ 1,688,000***
S 18th St, Dubois Lane, Columbia River HWY - Gravity Pipeline Upsize				
15-inch Pipe - Excavation, Backfill	3,829	LF	$ 170	$ 650,930
15-inch Pipe - Excludes Excavation, Backfill	391	LF	$ 27	$ 10,480
Directional Bore - 24" Casing	391	LF	$ 573	$ 224,050
Roadway Restoration (Half Lane)	2,678	LF	$ 45	$ 120,510
Landscape Restoration	760	LF	$ 20	$ 15,200
Highway Permitting	1	LS	$ 5,000	$ 5,000
Traffic Control w/ Flagging	1	LS	$ 106,000	$ 106,000
Railroad Boring	80	LF	$ 900	$ 72,000
48-Inch, Standard Manhole	16	EA	$ 8,000	$ 128,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
ADA Ramp Reconstruction (Compliance)	10	EA	$ 4,600	$ 46,000
Replace Service Laterals	47	EA	$ 1,500	$ 70,500
Bypass Pumping - Pipes 24-inch and smaller	3,829	LF	$ 22	$ 84,970
Rock Excavation	819	CY	$ 300	$ 245,780
Existing Utility Protection	3,829	LF	$ 4	$ 15,320
			Subtotal	***$ 1,798,000***
Old Portland Rd., Kaster Rd. - Gravity Pipeline Reroute to 27" Trunkline				
15-inch Pipe - Excavation, Backfill	425	LF	$ 170	$ 72,250
Roadway Restoration (Half Lane)	425	LF	$ 45	$ 19,130
Traffic Control w/ Flagging	1	LS	$ 7,000	$ 7,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Existing Utility Protection	425	LF	$ 4	$ 1,700
Rock Excavation	10	CY	$ 300	$ 3,070
			Subtotal	***$ 107,000***
Southern Trunkline - Gravity Pipeline Upsize				
30-inch Pipe - Excavation, Backfill	420	LF	$ 230	$ 96,600
33-inch Pipe - Excavation, Backfill	1,185	LF	$ 240	$ 284,400
36-inch Pipe - Excavation, Backfill	2,325	LF	$ 245	$ 569,600
36-inch Pipe - Excavation, Backfill, Structure over Lagoon	645	LF	$ 1,170	$ 754,700
Roadway Restoration (Half Lane)	50	LF	$ 45	$ 2,300
Traffic Control w/ Flagging	1	LS	$ 6,000	$ 6,000
72-Inch, Standard Manhole	15	EA	$ 16,500	$ 247,500
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Bypass Pumping - Pipes larger than 24-inch	4,575	LF	$ 27	$ 125,400
Rock Excavation	3,102	CY	$ 300	$ 930,600
Existing Utility Protection	4,575	LF	$ 4	$ 18,300
			Subtotal	***$ 3,039,000***
			Subtotal (rounded)	***$ 6,632,000***
Mobilization	1	LS	5%	$ 332,000
Contingency	1	LS	30%	$ 2,090,000
			Construction Subtotal (rounded)	***$ 9,054,000***
Engineering and CMS	1	LS	20%	$ 1,811,000
Permitting (Assume 5% of total)	1	LS	$ 452,700	$ 452,700
Geotechnical (Assume 1% of total)	1	LS	$ 90,540	$ 90,540
Surveying	1	LS	$ 60,000	$ 60,000
Legal and Admin	1	LS	$ 30,000	$ 30,000
			Total Project Cost (rounded)	***$ 11,500,000***

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Appendix J

Capital Improvement Plan (CIP) Project Sheets

KELLER
ASSOCIATES

St. Helens Wastewater Master Plan
Priority CIP

Project No.	Project Name	Primary Purpose	Total Estimated Cost (2021)	SDC Growth Apportionment		City's Estimated Portion
				%	Cost	
Priority 1 Improvements						
1.1	WWTP Influent Flow Meter	Operations	$ 68,000	10%	$ 7,000	$ 61,000
1.2	Basin 4 Pipeline Upsize and Reroute	Capacity	$ 3,600,000	0%	$ -	$ 3,600,000
1.3	Basin 5 Pipeline Upsize	Capacity	$ 4,500,000	3%	$ 150,000	$ 4,350,000
1.4	Install Overflow Alarms	Operations	$ 9,000	20%	$ 2,000	$ 7,000
1.5	Pump Station 3 On-site Generator	Operations	$ 90,000	0%	$ -	$ 90,000
1.6	Annual I/I Reduction Program (6-Year)	Capacity	$ 3,000,000	20%	$ 590,000	$ 2,410,000
		Total Priority 1 Improvement Cost (rounded)	**$ 11,300,000**			**$ 10,500,000**
Priority 2 Improvements						
2.1	Riverfront District Trunkline and Pump Station 1 Relocation	Capacity, Operations	$ 2,400,000	18%	$ 440,000	$ 1,960,000
2.2	Relocate Pump Station 11	Capacity, Operations	$ 3,100,000	68%	$ 2,110,000	$ 990,000
2.3	Industrial Business Park Trunklines and Pump Station	Capacity, Operations	$ 13,200,000	100%	$ 13,200,000	$ -
2.4	Pump Station Upgrades	Operations, Safety	$ 700,000	20%	$ 140,000	$ 560,000
2.5	Master Plan Update	Operations	$ 300,000	100%	$ 300,000	$ -
2.6	Annual I/I Reduction Program (8-Year)	Capacity	$ 4,000,000	20%	$ 790,000	$ 3,210,000
		Total Priority 2 Improvement Cost (rounded)	**$ 23,700,000**			**$ 6,700,000**
Priority 3 Improvements						
3.1	Basin 6 Pipeline Upsize and Reroute	Capacity	$ 6,300,000	7%	$ 460,000	$ 5,840,000
3.2	Basin 2 Pipeline Upsize and Reroute	Capacity	$ 9,400,000	12%	$ 1,140,000	$ 8,260,000
3.3	Southern Trunkline Upsize	Capacity	$ 3,900,000	26%	$ 1,010,000	$ 2,890,000
3.4	Pump Station 7 Upgrades	Capacity	$ 2,200,000	65%	$ 1,430,000	$ 770,000
3.5	Basin 1 Pipeline Upsize	Capacity	$ 1,800,000	9%	$ 150,000	$ 1,650,000
3.6	Basin 3 Pipeline Upsize	Capacity	$ 1,200,000	3%	$ 40,000	$ 1,160,000
3.7	Annual I/I Reduction Program (6-year)	Capacity	$ 3,000,000	20%	$ 590,000	$ 2,410,000
		Total Priority 3 Improvement Cost (rounded)	**$ 27,900,000**			**$ 23,000,000**
		Total Collection System Improvement Costs (rounded)	**$ 62,900,000**			**$ 40,200,000**

St. Helens Wastewater Master Plan
6-Year CIP

Project No.	Item	Cost (2021)	Opinion of Probable Costs					
			2022	2023	2024	2025	2026	2027
Priority 1 Improvements								
1.1	WWTP Influent Flow Meter	$ 68,000	$ 68,000					
1.2	Basin 4 Pipeline Upsize and Reroute	$ 3,600,000		$ 400,000	$ 3,200,000			
1.3	Basin 5 Pipeline Upsize	$ 4,500,000				$ 500,000	$ 4,000,000	
1.4	Install Overflow Alarms	$ 9,000	$ 9,000					
1.5	Pump Station 3 On-site Generator	$ 90,000	$ 90,000					
1.6	Annual I/I Reduction Program (6-Year)	$ 3,000,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000	$ 500,000
	Total (Rounded)	$ 11,300,000	$ 700,000	$ 900,000	$ 3,700,000	$ 1,000,000	$ 4,500,000	$ 500,000

Collection System Project: Install WWTP Influent Flowmeter
Project Identifier: 1.1

Objective: Provide the St. Helens WWTP with an accurate measurement of influent flows during wet-weather or high-flow periods

Design Considerations:

- Provide adequate upstream and downstream length on either side of flow meter to ensure accurate flow measurement (minimum 18 feet upstream, 35 feet downstream)

- Ensure installation does not prevent WWTP access or operations

SDC Growth Appointment: 10%

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Installation of Flowmeter				
Hach FLO-DAR AV Sensor and Rig	1	EA	$ 16,000	$ 16,000
60-Inch, Standard Manhole	1	LS	$ 14,000	$ 14,000
Roadway Restoration	20	LF	$ 45	$ 900
Construction Subtotal (rounded)				***$ 31,000***
SCADA Integration	1	LS	25%	$ 7,750
Mobilization	1	LS	5%	$ 2,000
Contingency	1	LS	30%	$ 13,000
Engineering and CMS	1	LS	25%	$ 14,000
Total Project Cost (rounded)				***$ 68,000***

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Collection System Project: Basin 4 - Pipeline Upsize and Reroute from Tualatin St. to Basin 6
Project Identifier: 1.2

Objective: Resolve undersized pipelines in Basin 4. Upsize and construct gravity pipeline capable of conveying anticipated 20-year peak hour flows.

Design Considerations:

- Rock excavation for the new pipeline down Tualatn and S 7th St. Assumed pipes to be upsized will require rock excavation from the new pipe crown to bedding.

- Trench modification, manhole modification, and reversing the slope of the existing pipeline in Tualatin St.

- Ensure wastewater service is maintained via bypass pumping during pipeline upsizing and use of existing trunkline during new construction

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Gravity Pipeline Upszie				
12-inch Pipe - Excavation, Backfill	860	LF	$ 160	$ 137,600
15-inch Pipe - Excavation, Backfill	3,830	LF	$ 170	$ 651,100
Roadway Restoration (Half Lane)	3,140	LF	$ 45	$ 141,300
Landscape Restoration	1,550	LF	$ 20	$ 31,000
Traffic Control w/out Flagging	860	LF	$ 6	$ 5,160
Traffic Control w/ Flagging	1	LS	$ 122,000	$ 122,000
48-Inch, Standard Manhole	17	EA	$ 8,000	$ 136,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Existing Utility Protection	4,690	LF	$ 4	$ 18,760
Replace Service Laterals	25	EA	$ 1,500	$ 37,500
Bypass Pumping - Pipes 24-inch and smaller	3,160	LF	$ 22	$ 70,130
Rock Excavation	2,114	CY	$ 300	$ 634,330
			Subtotal (rounded)	***$ 1,989,000***
Mobilization	1	LS	5%	$ 100,000
Contingency	1	LS	30%	$ 627,000
			Construction Subtotal (rounded)	***$ 2,716,000***
Engineering and CMS	1	LS	20%	$ 544,000
Permitting (Assume 8% of total)	1	LS	$ 217,300	$ 217,300
Geotechnical (Assume 1% of total)	1	LS	$ 27,000	$ 27,000
Surveying	1	LS	$ 50,000	$ 50,000
Legal and Admin	1	LS	$ 20,000	$ 20,000
			Total Project Cost (rounded)	***$ 3,600,000***

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Collection System Project: Basin 5 - Pipeline Upsize
Project Identifier: 1.3

Objective: Resolve undersized pipelines in Basin 5. Upsize existing gravity pipeline to be capable of conveying anticipated 20-year peak hour flows.

Design Considerations:

- Upsizing by 2 sizes may be larger than existing trench, assumed pipes to be upsized will require rock excavation from the new pipe crown to bedding.

- When upsizing the parallel pipes beneath the City's tunnel, replace the pipelines with a singular 42-inch pipeline. To re-evaluate flowrates and pipeline sizing after completion of Project 1.2.

- Ensure wastewater service is maintained via bypass pumping when upsizing existing line.

SDC Growth Appointment: 3%

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Gravity Pipeline Upszie				
36-inch Pipe - Excavation, Backfill	470	LF	$ 245	$ 115,150
42-inch Pipe - Excavation, Backfill	2,850	LF	$ 275	$ 783,750
Roadway Restoration (Full Lane)	2,185	LF	$ 75	$ 163,880
Landscape Restoration	1,135	LF	$ 20	$ 22,700
Traffic Control w/ Flagging	1	LS	$ 92,000	$ 92,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
72-Inch, Standard Manhole	14	EA	$ 16,500	$ 231,000
Existing Utility Protection	3,320	LF	$ 4	$ 13,280
ADA Ramp Reconstruction (Compliance)	8	EA	$ 4,600	$ 36,800
Replace Service Laterals	27	EA	$ 1,500	$ 40,500
Bypass Pumping - Pipes larger than 24-inch	3,320	LF	$ 27	$ 91,020
Rock Excavation	2,906	CY	$ 300	$ 871,920
Tunnel Bore	475	LF	$ 400	$ 200,000
			Subtotal (rounded)	***$ 2,666,000***
Mobilization	1	LS	5%	$ 134,000
Contingency	1	LS	30%	$ 840,000
			Construction Subtotal (rounded)	***$ 3,640,000***
Engineering and CMS	1	LS	20%	$ 728,000
Permitting	1	LS	$ 15,000	$ 15,000
Geotechnical (Assume 1% of total)	1	LS	$ 36,400	$ 36,400
Surveying	1	LS	$ 30,000	$ 30,000
Legal and Admin	1	LS	$ 10,000	$ 10,000
			Total Project Cost (rounded)	***$ 4,500,000***

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Collection System Project: Install Overflow Alarms at Pump Stations
Project Identifier: 1.4

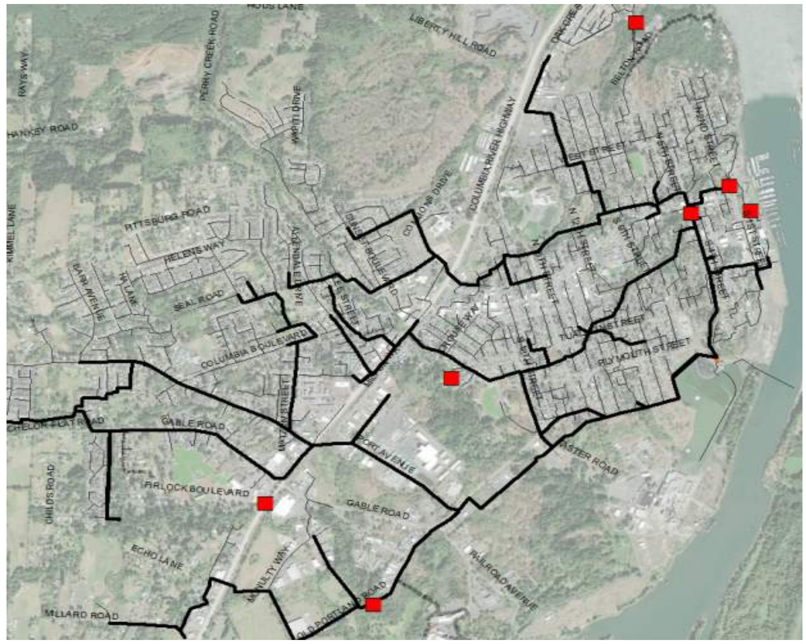
Objective: Provide all of the City's Pump Stations with overflow alarms

Design Considerations:

- Consider coordinating installation of overflow alarms with Priority 2 Pump Station Improvements (Project 2.3)

- Ensure installation doesn't interfere with pump station operations

SDC Growth Appointment: 20%



Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Pump Station Overflow Alarms				
Install overflow alarm - labor and SCADA integration	4	EA	$ 1,000	$ 4,000
Construction Subtotal (rounded)				***$ 4,000***
Mobilization	1	LS	5%	$ 1,000
Contingency	1	LS	30%	$ 2,000
Engineering, SCADA integration, and CMS	1	LS	25%	$ 2,000
Total Project Cost (rounded)				***$ 9,000***

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Collection System Project: Install Pump Station 3 On-Site Generator
Project Identifier: 1.5

Objective: Provide Pump Station 3 with on-site backup power to increase City's

Design Considerations:

- Size generator to service pump station

- Assumed natural gas generator supplied by underground natural gas utility

- The pump station is located within a traffic lane. Traffic control not included in costs, but an increased contigency is included. Contractor to specify traffic control requirements prior to construction.

SDC Growth Appointment: 0%

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Pump Station On-site Generator				
Generator - Includes installation, labor	1	LS	$ 27,000	$ 27,000
Miscellaneous Electrical Materials	1	LS	$ 5,000	$ 5,000
Natural Gas Service	1	LS	$ 4,000	$ 4,000
Automatic Transfer Switch	1	LS	$ 3,500	$ 3,500
Equipment Pad	1	LS	$ 5,000	$ 5,000
Miscellaneous Site Improvements	1	LS	$ 7,000	$ 7,000
Construction Subtotal (rounded)				***$ 52,000***
Mobilization	1	LS	5%	$ 3,000
Contingency	1	LS	30%	$ 17,000
Engineering, SCADA integration, and CMS	1	LS	25%	$ 18,000
Total Project Cost (rounded)				***$ 90,000***

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Collection System Project: Riverfront District Trunkline and Pump Station 1 Relocation
Project Identifier: 2.1

Objective: Demolish existing Pump Station 1 and construct a new 700 gpm pump station to serve the existing basin and new development in the Riverfront District

Design Considerations:

- Connect to existing manhole and abandon/fill pipeline connection to old Pump Station 1 wetwell site. Construction of new road in Riverfront District not included in cost.

- Sequence the demolision/displacment of old Pump Station 1 after construction of new Pump Station 1 to ensure service to existing residents

- Construction of new pipe and pump station may encounter high groundwater table. Pothole to verify water table depth, provide dewatering measures as necessary. Groundwater level may be influenced by tidal changes.

SDC Growth Appointment: 18%

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Relocation of Pump Station 1				
Displace/Demolish Existing Pump Station	1	LS	$ 30,000	$ 30,000
Pump Station, 700 gpm	1	LS	$ 750,000	$ 750,000
10-inch Pipe - Excavation, Backfill, Shoring	1,700	LF	$ 150	$ 255,000
6-inch Force Main - Excavation, Backfill, Shoring	1,100	LF	$ 75	$ 82,500
Roadway Restoration (Half Lane)	1,100	LF	$ 45	$ 49,500
Traffic Control w/ Flagging	1	LS	$ 59,000	$ 59,000
48-Inch, Standard Manhole	6	EA	$ 8,000	$ 48,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Bypass Pumping	1	LS	$ 25,000	$ 25,000
Grounwater Dewatering (Assume 2.5% of subtotal)	1	LS	$ 32,900	$ 32,900
Existing Utility Protection	2,800	LF	$ 4	$ 11,200
			Subtotal (rounded)	***$ 1,347,000***
Mobilization	1	LS	5%	$ 68,000
SCADA Integration	1	LS	$ 30,000	$ 30,000
Contingency	1	LS	30%	$ 434,000
			Construction Subtotal (rounded)	***$ 1,879,000***
Permitting	1	LS	$ 20,000	$ 20,000
Geotechnical	1	LS	$ 20,000	$ 20,000
Surveying	1	LS	$ 40,000	$ 40,000
Engineering and CMS	1	LS	20%	$ 376,000
Legal and Admin	1	LS	$ 20,000	$ 20,000
			Total Project Cost (rounded)	***$ 2,400,000***

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Collection System Project: Pump Station 11 Relocation
Project Identifier: 2.2

Objective: Demolish existing Pump Station 11 and construct a new 550 gpm pump station to serve the existing basin and new development

Design Considerations:

- Purchasing land and/or easement for new pump station and pipelines

- Trenchless bore or minimal impact construction over the McNulty Creek culvert crossing

- Assuming a trenchless directional bore is possible for installing both pipelines beneath McNulty; this avoids replacement of the existing McNulty Creek culvert. Included a 40% contigency and 10% geotechnical line item to account for unseen construction setbacks due to bedrock

SDC Growth Appointment: 68%

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Relocation of Pump Station 11				
Displace/Demolish Existing Pump Station	1	LS	$ 30,000	$ 30,000
Pump Station, 550 gpm	1	LS	$ 600,000	$ 600,000
12-inch Pipe - Trenchless Installation, includes launch and receiving pits, casing	400	LF	$ 595	$ 238,000
6-inch Force Main - Trenchless Installation, includes launch and receiving pits, casing	400	LF	$ 541	$ 216,500
6-inch Force Main - Excavation, Backfill	2,830	LF	$ 75	$ 212,300
Connect to Existing Manhole	2	LS	$ 1,750	$ 3,500
Roadway Restoration (Half Lane)	2,870	LF	$ 45	$ 129,150
Soil Surface Repair	800	LF	$ 30	$ 24,000
Connect to Existing Manhole	1	EA	$ 1,750	$ 1,750
Bypass Pumping	1	LS	$ 25,000	$ 25,000
Rock Excavation	121	BCY	$ 300	$ 36,200
Existing Utility Protection	3,630	LF	$ 4	$ 14,500
			Subtotal (rounded)	***$ 1,531,000***
Mobilization	1	LS	5%	$ 77,000
SCADA Integration	1	LS	$ 30,000	$ 30,000
Contingency	1	LS	40%	$ 656,000
			Construction Subtotal (rounded)	***$ 2,294,000***
Permitting	1	LS	$ 20,000	$ 20,000
Geotechnical (Assume 10% of total)	1	LS	$ 229,000	$ 229,000
Surveying	1	LS	$ 40,000	$ 40,000
Engineering and CMS	1	LS	20%	$ 459,000
Legal and Admin	1	LS	$ 20,000	$ 20,000
			Total Project Cost (rounded)	***$ 3,100,000***

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Collection System Project: Industrial Business Park Trunklines and Pump Station
Project Identifier: 2.3

Objective: Provide wastewater service to Industrial Business Park via new pipelines and pump station

Design Considerations:

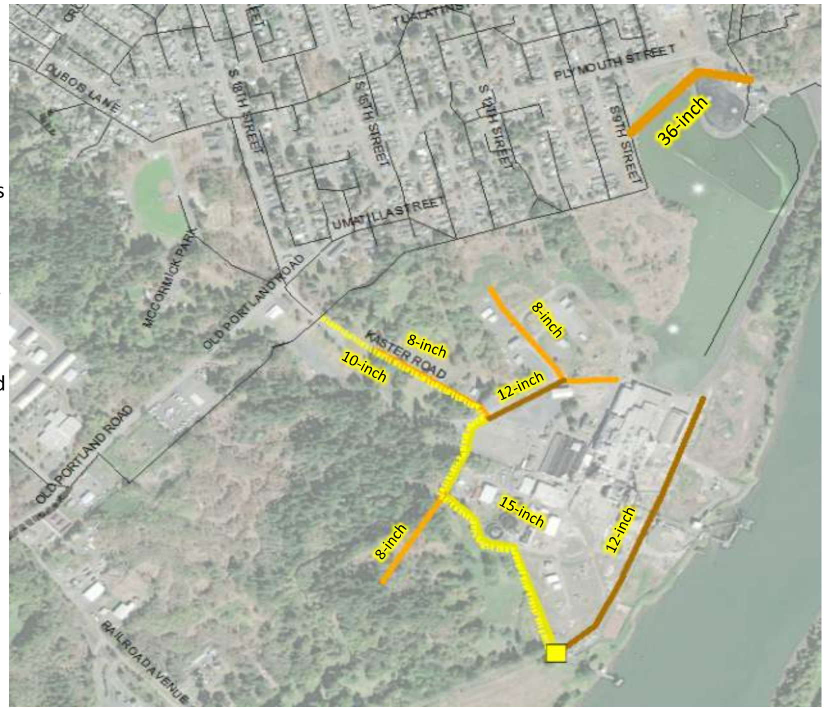
- Restoration of existing road in Industrial Business Park is included in cost. Roadway expansion or upgrades are not included in cost.

- Include construction of 36-inch pipe upstream of WWTP

- Pipelines must be designed to convey anticipated peak hour flows. Flowrates may vary depending on industry and rate of development. Appropriate pipe sizes to be re-evaluated during predesign.

- Costs assume open trench rock excavation for new pipelines. Construction may encounter high groundwater near the Columbia River, assumed 1% of subtotal for dewatering.

SDC Growth Appointment: 100%



Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Construction of Business Industrial Park Infrastructure and Downstream Trunkline				
Pump Station, 1,300 gpm	1	LS	$1,200,000	$ 1,200,000
8-inch Pipe - Excavation, Backfill	3,070	LF	$ 135	$ 414,500
12-inch Pipe - Excavation, Backfill	2,900	LF	$ 160	$ 464,000
15-inch Pipe - Excavation, Backfill	2,210	LF	$ 170	$ 375,700
10-inch Force Main - Excavation, Backfill, Shoring	3,725	LF	$ 95	$ 353,900
36-inch Pipe - Excavation, Backfill, Structure over Lagoon	645	LF	$ 1,170	$ 754,700
36-inch Pipe - Excavation, Backfill	425	LF	$ 245	$ 104,100
Roadway Restoration (Half Lane)	11,905	LF	$ 45	$ 535,700
Traffic Control w/ Flagging	1	LS	$ 206,000	$ 206,000
48-Inch, Standard Manhole	27	EA	$ 8,000	$ 216,000
72-Inch, Standard Manhole	2	EA	$ 16,500	$ 33,000
Connect to Existing Manhole	3	EA	$ 1,750	$ 5,300
Bypass Pumping - Pipes 24-inch and smaller	11,905	LF	$ 22	$ 264,200
Grounwater Dewatering (Assume 1% of subtotal)	1	LS	$ 74,500	$ 74,500
Rock Excavation	8,289	CY	$ 300	$ 2,486,600
Existing Utility Protection	8,180	LF	$ 4	$ 32,700
			Subtotal (rounded)	***$ 7,521,000***
Mobilization	1	LS	5%	$ 377,000
SCADA Integration	1	LS	$ 30,000	$ 30,000
Contingency	1	LS	30%	$ 2,379,000
			Construction Subtotal (rounded)	***$ 10,307,000***
Permitting (Assumed 5% of total)	1	LS	$ 515,350	$ 515,400
Geotechnical (Assume 1% of total)	1	LS	$ 103,070	$ 103,100
Surveying	1	LS	$ 100,000	$ 100,000
Engineering and CMS	1	LS	20%	$ 2,062,000
Legal and Admin	1	LS	$ 40,000	$ 40,000
			Total Project Cost (rounded)	***$ 13,200,000***

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Collection System Project: Pump Station Upgrades
Project Identifier: 2.4

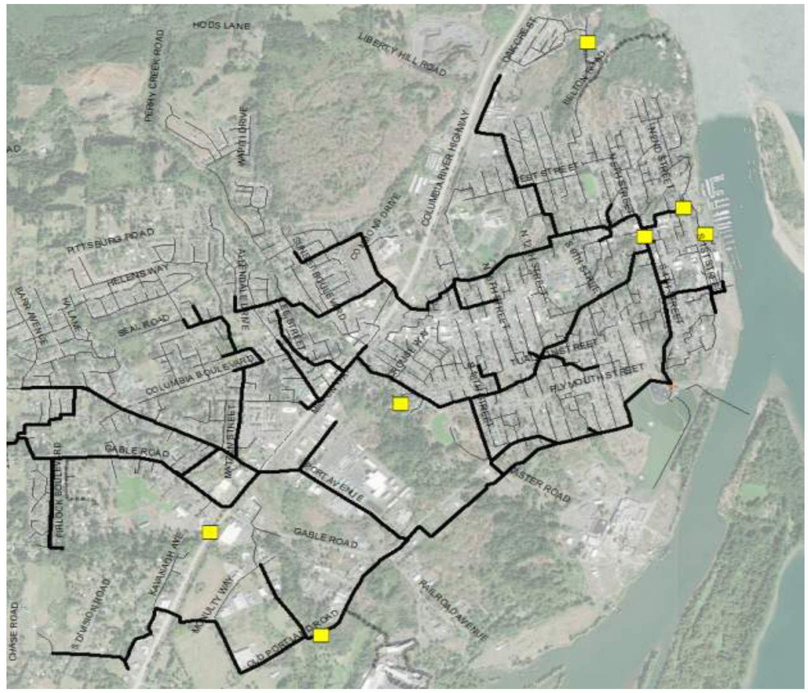
Objective: Provide required and recommended improvements to pump stations to improve operations, data collection, redundancy, and safety

Design Considerations:

- **I**ntegration of new meters and sensors with existing SCADA system

- Mechanical modifications to accomodate new flow monitors and pressure gauges

SDC Growth Appointment: 20%



Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Pump Station 2				
Fall Protection	1	LS	$ 4,000	$ 4,000
Flow Meter (Includes Piping Modifications)	1	LS	$ 20,000	$ 20,000
Pressure Gauge	1	LS	$ 3,500	$ 3,500
			Subtotal	***$ 27,500***
SCADA Upgrades	1	LS	25%	$ 6,875
			Pump Station 2 Subtotal	***$ 34,400***
Pump Station 3				
Fall Protection	1	LS	$ 4,000	$ 4,000
Flow Meter	1	LS	$ 20,000	$ 20,000
Pressure Gauge	1	LS	$ 3,500	$ 3,500
			Subtotal	***$ 27,500***
SCADA Upgrades	1	LS	25%	$ 6,875
			Pump Station 3 Subtotal	***$ 34,400***
Pump Station 4				
Fall Protection	1	LS	$ 4,000	$ 4,000
Flow Meter	1	LS	$ 20,000	$ 20,000
Pressure Gauge	1	LS	$ 3,500	$ 3,500
Ultrasonic Level Sensor	1	LS	$ 5,000	$ 5,000
			Subtotal	***$ 28,500***
SCADA Upgrades	1	LS	25%	$ 7,125
			Pump Station 4 Subtotal	***$ 35,600***
Pump Station 5				
Fall Protection	1	LS	$ 4,000	$ 4,000
Flow Meter	1	LS	$ 20,000	$ 20,000
Pressure Gauge	1	LS	$ 3,500	$ 3,500
Pump Upgrade - 300 gpm	2	EA	$ 30,000	$ 60,000
Electrical Upgrades (Standby Power, Panel)	1	LS	$ 55,000	$ 55,000
			Subtotal	***$ 142,500***
SCADA Upgrades	1	LS	25%	$ 35,625
			Pump Station 5 Subtotal	***$ 178,100***

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Pump Station 7				
Flow Meter	1	LS	$ 20,000	$ 20,000
Pressure Gauge	1	LS	$ 3,500	$ 3,500
			Subtotal	***$ 23,500***
SCADA Upgrades	1	LS	25%	$ 5,875
			Pump Station 7 Subtotal	***$ 29,400***
Pump Station 8				
Fall Protection	1	LS	$ 4,000	$ 4,000
Flow Meter	1	LS	$ 20,000	$ 20,000
Pressure Gauge	1	LS	$ 3,500	$ 3,500
Ultrasonic Level Sensor	1	LS	$ 5,000	$ 5,000
			Subtotal	***$ 32,500***
SCADA Upgrades	1	LS	25%	$ 8,125
			Pump Station 8 Subtotal	***$ 40,600***
Pump Station 9				
Flow Meter	1	LS	$ 20,000	$ 20,000
Pressure Gauge	1	LS	$ 3,500	$ 3,500
Ultrasonic Level Sensor	1	LS	$ 5,000	$ 5,000
			Subtotal	***$ 28,500***
SCADA Upgrades	1	LS	25%	$ 7,125
			Pump Station 9 Subtotal	***$ 35,600***
			Construction Subtotal (rounded)	***$ 389,000***
Mobilization	1	LS	5%	$ 20,000
Contingency	1	LS	30%	$ 123,000
Engineering and CMS	1	LS	25%	$ 133,000
			Total Project Cost (rounded)	***$ 700,000***

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Collection System Project: Master Plan Update
Project Identifier: 2.5

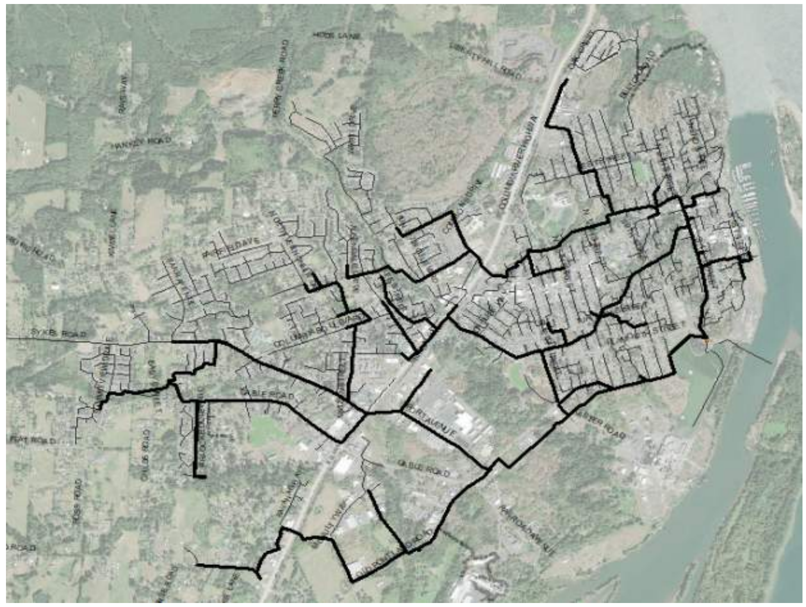
Objective: Update the City of St. Helens Master Plan with new data collected from influent flow meter. Will effect the model and existing/future system evaluation, as well as recommendations and potential future Capital Improvement Projects. Includes Master Planning efforts for treatment.

Design Considerations:

- New areas built-out since previous planning studies

- Combined Wastewater Treatment and Collection System Master Plan Update

SDC Growth Appointment: 100%



Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Planning Update				
Master Plan Update	1	LS	$ 300,000	$ 300,000
			Total Project Cost (rounded)	***$ 300,000***

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Collection System Project: Basin 6 - Pipeline Upsize and Reroute from Old Portland Rd to Kaster Rd.
Project Identifier: 3.1

Objective: Resolve undersized pipelines in Basin 6. Upsize existing and construct new gravity pipeline to be capable of conveying anticipated 20-year peak hour flows.

Design Considerations:
- There is a crossing beneath Milton Creek within the Columbia River Highway. Assume trenchless bore to avoid interference with Milton Creek.

- Anticipate rock excavation for new pipeline from Old Portland Rd to Kaster Rd. Assumed pipes to be upsized will require rock excavation from the new pipe crown to bedding.

- Ensure wastewater service is maintained via bypass pumping when upsizing existing line. Utilize existing trunkline along Umatilla St. to maintain service during construction of new pipeline.

SDC Growth Appointment: 7%

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Port Ave - Gravity Upsize				
27-inch Pipe - Excavation, Backfill	3,030	LF	$ 220	$ 666,600
Roadway Restoration (Half Lane)	2,775	LF	$ 45	$ 124,880
Landscape Restoration	255	LF	$ 20	$ 5,100
Traffic Control w/ Flagging	1	LS	$ 76,000	$ 76,000
60-Inch, Standard Manhole	9	EA	$ 14,000	$ 126,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Railroad Boring	140	LF	$ 900	$ 126,000
Highway Permitting	1	LS	$ 5,000	$ 5,000
Replace Service Laterals	8	EA	$ 1,500	$ 12,000
Bypass Pumping - Pipes 24-inch and smaller	3,030	LF	$ 22	$ 67,240
Rock Excavation	1,546	CY	$ 300	$ 463,680
Existing Utility Protection	3,030	LF	$ 4	$ 12,120
			Subtotal	***$ 1,688,000***
S 18th St, Dubois Lane, Columbia River HWY - Gravity Pipeline Upsize				
15-inch Pipe - Excavation, Backfill	3,829	LF	$ 170	$ 650,930
15-inch Pipe - Excludes Excavation, Backfill	391	LF	$ 27	$ 10,480
Directional Bore - 24" Casing	391	LF	$ 573	$ 224,050
Roadway Restoration (Half Lane)	2,678	LF	$ 45	$ 120,510
Landscape Restoration	760	LF	$ 20	$ 15,200
Highway Permitting	1	LS	$ 5,000	$ 5,000
Traffic Control w/ Flagging	1	LS	$ 106,000	$ 106,000
Railroad Boring	80	LF	$ 900	$ 72,000
48-Inch, Standard Manhole	16	EA	$ 8,000	$ 128,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
ADA Ramp Reconstruction (Compliance)	10	EA	$ 4,600	$ 46,000
Replace Service Laterals	47	EA	$ 1,500	$ 70,500
Bypass Pumping - Pipes 24-inch and smaller	3,829	LF	$ 22	$ 84,970
Rock Excavation	819	CY	$ 300	$ 245,780
Existing Utility Protection	3,829	LF	$ 4	$ 15,320
			Subtotal	***$ 1,798,000***

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Old Portland Rd., Kaster Rd. - Gravity Pipeline Reroute to 27" Trunkline				
15-inch Pipe - Excavation, Backfill	425	LF	$ 170	$ 72,250
Roadway Restoration (Half Lane)	425	LF	$ 45	$ 19,130
Traffic Control w/ Flagging	1	LS	$ 7,000	$ 7,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Existing Utility Protection	425	LF	$ 4	$ 1,700
Rock Excavation	10	CY	$ 300	$ 3,070
			Subtotal	***$ 107,000***
			Subtotal (rounded)	***$ 3,593,000***
Mobilization	1	LS	5%	$ 180,000
Contingency	1	LS	30%	$ 1,132,000
			Construction Subtotal (rounded)	***$ 4,905,000***
Engineering and CMS	1	LS	20%	$ 981,000
Permitting (Assume 5% of total)	1	LS	$ 245,300	$ 245,300
Geotechnical (Assume 1% of total)	1	LS	$ 49,050	$ 49,050
Surveying	1	LS	$ 60,000	$ 60,000
Legal and Admin	1	LS	$ 30,000	$ 30,000
			Total Project Cost (rounded)	***$ 6,300,000***

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Collection System Project: Basin 2 - Pipeline Upsize and Reroute from Gable Rd. to Sykes Rd.
Project Identifier: 3.2

Objective: Resolve undersized pipelines in Basin 2. Upsize existing gravity pipeline and construct new pipeines to be capable of conveying anticipated 20-year peak hour flows.

Design Considerations:
- Upsizing by 2 sizes may be larger than existing trench, assumed pipes to be upsized will require rock excavation from the new pipe crown to bedding.

- Anticipate rock excavation when constructing new pipeline from Gable Rd to Sykes Rd.

- Ensure wastewater service is maintained via bypass pumping when upsizing existing line. Utilize existing trunkline along Gable Rd. to maintain service during construction of new pipeline.

SDC Growth Appointment: 12%

Item	EST. QTY	UNIT	UNIT PRICE		Cost (2021)	
Sykes Road - Gravity Upsize						
12-inch Pipe - Excavation, Backfill	315	LF	$	160	$	50,400
15-inch Pipe - Excavation, Backfill	2,645	LF	$	170	$	449,650
18-inch Pipe - Excavation, Backfill	522	LF	$	185	$	96,570
Roadway Restoration (Half Lane)	3,482	LF	$	45	$	156,690
Traffic Control w/ Flagging	1	LS	$	65,000	$	65,000
48-Inch, Standard Manhole	15	EA	$	8,000	$	120,000
Connect to Existing Manhole	2	EA	$	1,750	$	3,500
Bypass Pumping - Pipes 24-inch and smaller	3,482	LF	$	22	$	77,270
Rock Excavation	786	CY	$	300	$	235,760
ADA Ramp Reconstruction (Compliance)	11	EA	$	4,600	$	50,600
Replace Service Laterals	55	EA	$	1,500	$	82,500
Existing Utility Protection	2,960	LF	$	4	$	11,840
				Subtotal	***$***	***1,400,000***
Matzen St. - Gravity Pipeline Upsize						
12-inch Pipe - Excavation, Backfill	1,450	LF	$	160	$	232,000
15-inch Pipe - Excavation, Backfill	2,600	LF	$	170	$	442,000
Roadway Restoration (Half Lane)	2,980	LF	$	45	$	134,100
Landscape Restoration	1,070	LF	$	20	$	21,400
Traffic Control w/ Flagging	1	LS	$	110,000	$	110,000
48-Inch, Standard Manhole	16	EA	$	8,000	$	128,000
Connect to Existing Manhole	2	EA	$	1,750	$	3,500
Bypass Pumping - Pipes 24-inch and smaller	4,050	LF	$	22	$	89,880
Rock Excavation	792	CY	$	300	$	237,540
ADA Ramp Reconstruction (Compliance)	9	EA	$	4,600	$	41,400
Replace Service Laterals	42	EA	$	1,500	$	63,000
Existing Utility Protection	4,050	LF	$	4	$	16,200
				Subtotal	***$***	***1,519,000***

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Gable Rd. - Gravity Pipeline Upsize and Reroute to Sykes Rd.				
12-inch Pipe - Excavation, Backfill	3,000	LF	$ 160	$ 480,000
Roadway Restoration (Half Lane)	3,000	LF	$ 45	$ 135,000
Traffic Control w/ Flagging	1	LS	$ 87,000	$ 87,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
48-Inch, Standard Manhole	13	EA	$ 8,000	$ 104,000
Existing Utility Protection	3,000	LF	$ 4	$ 12,000
Bypass Pumping - Pipes 24-inch and smaller	3,000	LF	$ 22	$ 66,580
ADA Ramp Reconstruction (Compliance)	3	EA	$ 4,600	$ 13,800
Replace Service Laterals	23	EA	$ 1,500	$ 34,500
Rock Excavation	1,333	CY	$ 300	$ 400,000
			Subtotal	***$ 1,336,000***
Westshire Ln. - Gravity Pipeline Upsize				
10-inch Pipe - Excavation, Backfill	250	LF	$ 150	$ 37,500
12-inch Pipe - Excavation, Backfill	2,300	LF	$ 160	$ 368,000
Roadway Restoration (Half Lane)	2,235	LF	$ 45	$ 100,580
Landscape Restoration	315	LF	$ 20	$ 6,300
Traffic Control w/ Flagging	1	LS	$ 87,000	$ 87,000
48-Inch, Standard Manhole	16	EA	$ 8,000	$ 128,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Bypass Pumping - Pipes 24-inch and smaller	2,550	LF	$ 22	$ 56,590
Rock Excavation	381	CY	$ 300	$ 114,370
ADA Ramp Reconstruction (Compliance)	4	EA	$ 4,600	$ 18,400
Replace Service Laterals	25	EA	$ 1,500	$ 37,500
Existing Utility Protection	2,550	LF	$ 4	$ 10,200
			Subtotal	***$ 968,000***
			Subtotal (rounded)	***$ 5,223,000***
Mobilization	1	LS	5%	$ 262,000
Contingency	1	LS	30%	$ 1,646,000
			Construction Subtotal (rounded)	***$ 7,131,000***
Engineering and CMS	1	LS	20%	$ 1,427,000
Permitting (Assume 8% of total)	1	LS	$ 570,500	$ 570,500
Geotechnical (Assume 1% of total)	1	LS	$ 71,000	$ 71,000
Surveying	1	LS	$ 100,000	$ 100,000
Legal and Admin	1	LS	$ 40,000	$ 40,000
			Total Project Cost (rounded)	***$ 9,400,000***

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Collection System Project: Basin 6 - Southern Trunkline Upsize
Project Identifier: 3.3

Objective: Resolve undersized trunkline in Basin 6. Upsize existing gravity pipeline to be capable of conveying anticipated 20-year peak hour flows.

Design Considerations:
- Upsizing by one to two sizes may be larger than existing trench, assumed pipes to be upsized will require rock excavation from the new pipe crown to bedding.

- Ensure wastewater service is maintained via bypass pumping when upsizing existing line.

SDC Growth Appointment: 26%

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Southern Trunkline - Gravity Upsize				
30-inch Pipe - Excavation, Backfill	420	LF	$ 230	$ 96,600
33-inch Pipe - Excavation, Backfill	1,185	LF	$ 240	$ 284,400
36-inch Pipe - Excavation, Backfill	1,900	LF	$ 245	$ 465,500
Roadway Restoration (Half Lane)	50	LF	$ 45	$ 2,250
Traffic Control w/ Flagging	1	LS	$ 6,000	$ 6,000
72-Inch, Standard Manhole	13	EA	$ 16,500	$ 214,500
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
Bypass Pumping - Pipes larger than 24-inch	3,505	LF	$ 27	$ 96,090
Rock Excavation	3,102	CY	$ 300	$ 930,630
Existing Utility Protection	3,505	LF	$ 4	$ 14,020
			Subtotal	***$ 2,113,000***
			Subtotal (rounded)	***$ 2,113,000***
Mobilization	1	LS	5%	$ 106,000
Contingency	1	LS	30%	$ 666,000
			Construction Subtotal (rounded)	***$ 2,885,000***
Engineering and CMS	1	LS	20%	$ 577,000
Permitting (Assume 8% of total)	1	LS	$ 230,800	$ 230,800
Geotechnical (Assume 1% of total)	1	LS	$ 29,000	$ 29,000
Surveying	1	LS	$ 100,000	$ 100,000
Legal and Admin	1	LS	$ 40,000	$ 40,000
			Total Project Cost (rounded)	***$ 3,900,000***

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Collection System Project: Pump Station 7 Upgrades
Project Identifier: 3.4

Objective: Upgrade Pump Station 7 with new pumps to handle anticipated 20-year flows

Design Considerations:

- Station will continue to use parallel 6" and 8" forcemains to convey wastewater

- Install new pumps in existing pump station

- Revise pump station capacity with anticipated loading during pre-design

- Construction may encounter high groundwater table. Pothole to verify water table depth, provide dewatering measures as necessary. Groundwater level may be influenced by tidal changes.

- Ensure wastewater service is maintained via bypass pumping.

SDC Growth Appointment: 65%

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
New/Significant Upgrades to Pump Station 7				
Pump Station, 1,400 gpm	1	LS	$ 1,200,000	$ 1,200,000
Bypass Pumping	1	LS	$ 30,000	$ 30,000
			Subtotal (rounded)	***$ 1,230,000***
Mobilization	1	LS	5%	$ 62,000
SCADA Integration	1	LS	$ 30,000	$ 30,000
Contingency	1	LS	30%	$ 397,000
			Construction Subtotal (rounded)	***$ 1,719,000***
Permitting	1	LS	$ 20,000	$ 20,000
Geotechnical	1	LS	$ 20,000	$ 20,000
Surveying	1	LS	$ 40,000	$ 40,000
Engineering and CMS	1	LS	20%	$ 344,000
Legal and Admin	1	LS	$ 20,000	$ 20,000
			Total Project Cost (rounded)	***$ 2,200,000***

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Collection System Project: Basin 1 - Pipeline Upsize
Project Identifier: 3.5

Objective: Resolve undersized pipelines in Basin 1. Upsize existing gravity pipeline to be capable of conveying anticipated 20-year peak hour flows.

Design Considerations:

- Restore existing landscaping south of Sunset Pl to pre-disturbed condition or better.

- Assumed pipes to be upsized will require rock excavation from the new pipe crown to bedding.

- Ensure wastewater service is maintained via bypass pumping when upsizing existing line.

SDC Growth Appointment: 9%

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Gravity Pipeline Upszie				
18-inch Pipe - Excavation, Backfill	230	LF	$ 185	$ 42,550
15-inch Pipe - Excavation, Backfill	2,330	LF	$ 170	$ 396,100
Roadway Restoration (Half Lane)	1,315	LF	$ 45	$ 59,180
Landscape Restoration	1,245	LF	$ 20	$ 24,900
Traffic Control w/ Flagging	1	LS	$ 62,000	$ 62,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
48-inch Manhole	8	EA	$ 8,000	$ 64,000
Bypass Pumping - Pipes 24-inch and smaller	2,560	LF	$ 22	$ 56,810
ADA Ramp Reconstruction (Compliance)	6	EA	$ 4,600	$ 27,600
Rock Excavation	589	CY	$ 300	$ 176,770
Replace Service Laterals	18	EA	$ 1,500	$ 27,000
Existing Utility Protection	2,560	LF	$ 4	$ 10,240
			Subtotal (rounded)	***$ 951,000***
Mobilization	1	LS	5%	$ 48,000
Contingency	1	LS	30%	$ 300,000
			Construction Subtotal (rounded)	***$ 1,299,000***
Engineering and CMS	1	LS	20%	$ 260,000
Permitting (Assume 8% of total)	1	LS	$ 103,900	$ 103,900
Geotechnical (Assume 1% of total)	1	LS	$ 13,000	$ 13,000
Surveying	1	LS	$ 40,000	$ 40,000
Legal and Admin	1	LS	$ 20,000	$ 20,000
			Total Project Cost (rounded)	***$ 1,800,000***

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Collection System Project: Basin 3 - Pipeline Upsize
Project Identifier: 3.6

Objective: Resolve undersized pipelines in Basin 3. Upsize existing gravity pipeline to be capable of conveying anticipated 20-year peak hour flows.

Design Considerations:

- Assumed pipes to be upsized will require rock excavation from the new pipe crown to bedding.

- Ensure wastewater service is maintained via bypass pumping when upsizing existing line.

SDC Growth Appointment: 3%

Item	EST. QTY	UNIT	UNIT PRICE	Cost (2021)
Gravity Pipeline Upszie				
15-inch Pipe - Excavation, Backfill	1,550	LF	$ 170	$ 263,500
Roadway Restoration (Half Lane)	922	LF	$ 45	$ 41,490
Soil Surface Repair	628	LF	$ 5	$ 3,140
Traffic Control w/ Flagging	1	LS	$ 47,000	$ 47,000
48-Inch, Standard Manhole	8	EA	$ 8,000	$ 64,000
Connect to Existing Manhole	2	EA	$ 1,750	$ 3,500
ADA Ramp Reconstruction (Compliance)	4	EA	$ 4,600	$ 18,400
Replace Service Laterals	25	EA	$ 1,500	$ 37,500
Bypass Pumping - Pipes 24-inch and smaller	1,550	LF	$ 22	$ 34,400
Rock Excavation	332	CY	$ 300	$ 99,490
Existing Utility Protection	1,550	LF	$ 4	$ 6,200
			Subtotal (rounded)	***$ 619,000***
Mobilization	1	LS	5%	$ 31,000
Contingency	1	LS	30%	$ 195,000
			Construction Subtotal (rounded)	***$ 845,000***
Engineering and CMS	1	LS	20%	$ 169,000
Permitting (Assume 8% of total)	1	LS	$ 67,600	$ 67,600
Geotechnical (Assume 1% of total)	1	LS	$ 8,000	$ 8,000
Surveying	1	LS	$ 20,000	$ 20,000
Legal and Admin	1	LS	$ 10,000	$ 10,000
			Total Project Cost (rounded)	***$ 1,200,000***

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Appendix K

Inflow and Infiltration (I/) Priority Pipelines

KELLER ASSOCIATES

Priority	GIS - Field ID	GIS - Object ID	Diameter	Material	Upstream Manhole	Downstream Manhole	Length (ft)
1	243	265	21"	CP STACKED	I5	I4A	470
1	244	266	30"	CP	I4A	I4	76
1	245	267	20"	PE STACKED	I5	I4A	472
1	246	268	30"	CP	I8	I17A	347
1	262	285	30"	CP	I6	I5	246
1	390	427	6"	PVC	IF48B	IF48	64
1	391	428	8"	PVC	IF52	IF50	167
1	395	432	8"	PVC	IF48	IF50	143
1	396	433	8"	PVC	IF54	IF52	181
1	508	548	6"	VCP	NA1	N1	300
1	631	678	8"	CP	IF30	IF29	67
1	906	971	10"	CP	IA7	IA7A	22
1	1066	1138	8"	VCP	IF62	IF28	116
1	1115	1188	30"	CP	I7A	I7	75
1	1116	1189	30"	CP	I7	I6	305
1	1290	1370	8"	PVC	IA11	IA10B	68
1	1291	1371	8"	PVC	IA10B	IA10A	102
1	1292	1375	8"	DI	IA10	IA19	169
1	1305	1391	6"	DI	IA22	IA21	252
1	1306	1392	6"	DI	IA21	IA20	165
1	1307	1393	8"	DI	IA20	IA19	118
1	1308	1394	6"	CP	M11	MF5	230
1	1321	1409	6"	CP	MF5	MF4	129
1	1322	1410	6"	PVC	MF4	MF3	68
1	1323	1411	6"	CP	MF7	MF1	130
1	1324	1412	6"	CP	MF7A	MF7	58
1	1325	1413	6"	CP	MF6	MF7	164
1	1326	1414	6"	CP/PVC	MF9	MF5	138
1	1327	1415	6"	CP	STUB	M8	117
1	1328	1416	6"	CP	MF10	MF4	115
1	1329	1417	6"	CP/PVC	MF1A	M8	288
1	1387	1477	15"	CI	UNKN	IA25	182
1	1473	1565	8"	PVC	IA11A	IA11	68
1	1480	1572	6"	DI	MK6B	MK6A	172
1	1483	1575	6"	DI/CP	MK6A	MK6	147
1	1530	1626	6"	VCP	I10A	I10	244
1	1531	1627	6"	VCP	I10B	I10A	85
1	1542	1638	6	PVC	IE11	IE6	31
1	1551	1649	8"	PVC	IF50	IF51	147
1	1648	1753			IA7B	IA7	19
1	1682	1791	6"		IA28	IA7D	196
1	1683	1792			UNKN	IA28	166
1	1685	1794			IF64B	IF64A	5
1	1689	1799			IF64A	IF64	49
1	1690	1800	6"		IA7D	IA7B	142
1	1708	1825	10"	VCP		IA25	247
1	1709	1826			IA7B1	IA7	22
1	1710	1830			NE7A		120
1	1716	1844			NE7B	NE7A	87
1	1739	1877	6"	DI/CP	MK6A	MK6	26
2	1795	0	6"	PVC			118
2	98	110	12"	CP	NN6	NN5	419
2	102	114	12"	CP	NN4	NN3	100
2	103	115	12"	CP	NN7	NN6	157
2	104	116	12"	CP	NN8	NN7	228
2	105	117	12"	CP	NN8A	NN8	282
2	106	118	12"	CP	NN5	NN4	130
2	126	138	8"	CP	D25	D24	479
2	142	156	10"	CP	N31	N30	396
2	146	160	12"	CP	NN3	NN2	464
2	147	161	15"	CP	DD9B	DD9	856
2	148	162	12"	CP	NN1A	NN1	244

Priority	GIS - Field ID	GIS - Object ID	Diameter	Material	Upstream Manhole	Downstream Manhole	Length (ft)
2	150	164	6"		NO2	NO1	355
2	151	165	18"	CP	N29	N28	197
2	175	191	6"	PVC	DE18A	DE18	174
2	183	200	6"		N32	NO1	311
2	238	260	30"	CP	I3	I2	189
2	242	264	30"	CP	I4	I3	257
2	268	292	8"	DI	IF22	IF21	55
2	378	409	6"		DD10	DD9	175
2	379	410	15"	CP	DD9	DD8	106
2	380	411	18"	CP	N28	N27	232
2	385	421	10"	PVC	DE4A	DE4	75
2	387	423	8"	CP	DG2	DG1B	232
2	413	450	8"	DI	IF23	IF22	76
2	511	551	8"	CP	IF13	IF4	145
2	512	552	8"	CP	IF14	IF13	44
2	604	649	6"		DL1	D5	228
2	708	761	16"	PE	M10	M9	144
2	709	762	16"	PE	M11	M10	300
2	710	763	16"	PE	M12	M11	126
2	711	764	16"	PE	M13	M12	212
2	717	770	16"	PE	M9	M8A	285
2	769	824	8"	CP	NN41	NN6	91
2	806	866	6"	CIPP	ND26	ND7	56
2	807	867	8"	PVC	ND8	ND7	230
2	817	878	6"	CP	DD2B	DD2	150
2	856	919	8"	CP	SB1	S6	342
2	857	920	8"	CP	SB2	SB1	180
2	1044	1115	8"	CP	IF15	IF14	133
2	1088	1160	10"	PVC	ML8	ML7	158
2	1092	1164	6"	CIPP	ML24	ML10	116
2	1127	1201	8"		STUB	IF22	10
2	1331	1419	6"	CP	NN35	NN34	243
2	1332	1420	6"	CP	NN34	NN10	446
2	1338	1426	10"	PVC	D19	D18A	82
2	1341	1431	6"	CP	DG1B	DG1	324
2	1347	1437	8"	PVC	DE13	DE2	69
2	1369	1459	6"	CP	DE28	DE8	195
2	1370	1460	6"	CP	DE31	DE28	108
2	1372	1462	8"	CP	NN9	DE9	176
2	1378	1468	8"	CP	NN30	NN29	135
2	1379	1469	8"	CP	NN29	NN9	165
2	1380	1470	6"	CP	NN9A	NN9	193
2	1386	1476	6"	CP	DD5	DD4	230
2	1388	1478	8"	CP	DD3	DD2	194
2	1389	1479	8"	PVC	DD2	DD1	185
2	1390	1480	8"	CP	DD4	DD3	41
2	1391	1481	6"	CP	DD6	DD5	259
2	1392	1482	6"	CP	DD13	DD4	266
2	1465	1557	8"	PVC	DG8	DG2	108
2	1466	1558	8"	CP	DG3	DG2	132
2	1477	1569	8"	PVC	ML8A	ML8	82
2	1478	1570	8"	PVC	ML9	ML8A	85
2	1513	1609	6"	CP	DG7	DG5	83
2	1525	1621	6"	CP/PVC	MF1	MF1A	10
2	1536	1632	8"	DI	IA15	IA14	84
2	1537	1633	8"	DI	IA15A	IA15	137
2	1538	1634	8"	PVC	IA16	IA15	50
2	1539	1635	8"	PVC	IA16A	IA16	25
2	1540	1636	8"	PVC	IA17	IA16	63
2	1541	1637	6"	PVC	IE6	IE5	39
2	1543	1639	6"	CP	DE17B	DE17	215
2	1547	1643	6"	PVC/CP	DE13A1	DE13A	138

Priority	GIS - Field ID	GIS - Object ID	Diameter	Material	Upstream Manhole	Downstream Manhole	Length (ft)
2	1550	1648	8"	PVC	IA18	IA17	138
2	1584	1686	6"	CP	DE9A	DE9	104
2	1679	1788	6"	CIPP	ME8	ME8A	150
2	1790	1788	6"	CP	ME8A	ME10	68
2	1715	1840	6"	VCP		IG11	58
2	1717	1846				ME9	145
2	1718	1847				ME9	79
2	1722	1853	10"	CP	N31	N30	160
2	1737	1875	8"	PVC	ML9	ML8A	78
2	1738	1876	10"	CP/PVC	ML8	ML7	124
3	1813	0					226
3	1814	0	6"				317
3	4	5	10"	CP	NI2	NI1	438
3	5	6	10"	CP	NI6	NI5	133
3	8	9	10"	CP	NI3	NI2	123
3	9	10	8"	CP	NI15	NI5	362
3	10	11	8"	CP	NI13	NI4	364
3	11	12	8"	CP	NI12	NI4	297
3	12	13	8"	CP	NI14	NI5	209
3	13	14	8"	CP	NI11	NI3	213
3	14	15	8"	CP	NI10	NI3	346
3	15	16	8"	CP	NJ2	NJ1	137
3	16	17	8"	CP	NJ1	N23	178
3	17	18	10"	CP	NI7	NI6	137
3	18	19	8"	CP	NI8	NI7	136
3	19	20	8"	CP	NI9	NI8	67
3	20	21	8"	CP	NI16	NI8	347
3	21	22	6"	CP	STUB	NI12	10
3	58	69	6"	CP	DK3	DK1	143
3	91	103	8"	CP	NN19	NN20	400
3	92	104	8"	CP	NN21	NN20	323
3	93	105	6"	CP	NN22	NN21	82
3	94	106	6"	CP	NN23	NN22	111
3	96	108	8"	CP	STUB	NN22	4
3	97	109	8"	CP	NN19	NN4	434
3	107	119	8"	CP	NR1	N39	264
3	108	120	12"	CP	N39	N38	132
3	153	167	6"	CP	N43A	N43	302
3	174	190	6"	PVC	IE7	IE11	324
3	248	270	30"	CP	I11	I10	53
3	270	294	30"	CP	I12	I11	245
3	277	301	8"	PE	IE3A	IE3	259
3	330	360	18"	STEEL	S2	S1	644
3	331	361	16"	STEEL	S2	S1	644
3	424	461	15"	CP	W37	W36	372
3	425	462	15"	CP	W36	W35	387
3	426	463	10"	CP	WA1	W36	312
3	427	464	10"	CP	WA2	WA1	228
3	1855	464	10"	CP	WA2	WA1	67
3	1856	464	10"	PVC	WA2	WA1	5
3	428	465	10"	CP	WA3	WA2	200
3	429	466	10"	CP	WA4	WA3	329
3	1857	466	10"	CP	WA4	WA3	51
3	430	467	15"	CP	W35	W34	400
3	431	468	15"	CP	W34	W33	400
3	432	469	15"	CP	W33	W32	443
3	433	470	15"	CP	W31	W30	366
3	434	472	15"	CP	W38	W37	354
3	469	507	8"	CP	NN17	NN16	191
3	470	508	8"	CP	NN14	NN13	284
3	471	509	10"	CP	NN13	NN12	314
3	472	510	12"	CP	NN12	NN11	323

Priority	GIS - Field ID	GIS - Object ID	Diameter	Material	Upstream Manhole	Downstream Manhole	Length (ft)
3	473	511	8"	CP	NN16	NN15	120
3	474	512	8"	CP	NN15	NN14	294
3	475	513	8"	CP	NN38	NN15	244
3	476	514	8"	CP	NN39	NN38	324
3	477	515	8"		NN40	NN38	137
3	478	516	8"	CP	NN37	NN16	115
3	479	517	8"	CP	NN36	NN14	116
3	480	518	12"	CP	NN11	NN10	60
3	481	520	6"	CP	NN36B	NN36	50
3	573	618	15"	PE	M21	M20	117
3	619	666	6"		STUB	M18A	10
3	620	667	6"		STUB	M18	10
3	623	670	6"	PVC	STUB	M16	10
3	624	671	6"	PVC	STUB	M16	15
3	625	672	10"	CP	IF1	I11	258
3	626	673	10"	CP	IF2	IF1	199
3	714	767	12"	DI	M16	M15	141
3	835	896	4	CP	UNKN	DE15	32
3	853	916	12"	PE	M17	M16	163
3	864	928	12"	PE	M18	M17	124
3	865	929	12"	PE	M18	M18A	25
3	866	930	12"	PE	M19	M18A	330
3	867	931	15"	PE	M20	M19	117
3	868	932	12"	PE	M21A	M21	163
3	869	933	12"	PE	M22	M21A	118
3	880	945	8"	CP	STUB	MP15	19
3	1275	1353	8"	CO	NN18	NN17	376
3	1280	1359	8"		STUB	WA4	10
3	1281	1360	15"	CP	W32	W31A	307
3	1282	1362	15"		STUB	W32	10
3	1333	1421	12"	CP	NN10	NN9	278
3	1374	1464	6"	CP	NN31	NN31A	49
3	1377	1467	6"	CP	NN30A	NN30	47
3	1417	1507	30"	CP	I10	I9A	112
3	1486	1580	8"	CP	N40	N39	236
3	1487	1581	8"	CP	N41	N40	115
3	1545	1641	10"	PVC	DE5A	DE4A	32
3	1546	1642	6"	CP	DE18B1	DE18B	41
3	1573	1671	6"	CP	N41A	N41	118
3	1752	1892	6"	CP			164
3	1763	1904	6"	CP			156
3	1772	1919					174
3	1773	1922					272
3	1775	1925				NCC49	213