City of Scappoose
Facilities Plan Update

FACILITIES PLAN UPDATE

FINAL DRAFT | March 2018

City of Scappoose
Facilities Plan Update

FACILITIES PLAN UPDATE

FINAL DRAFT | March 2018

Contents

Appendices

Tables

Figures

-This Page Intentionally Left Blank-

Abbreviations

°C	degrees Celsius
ADWF	average dry weather flow
AWWF	average wet weather flow
BOD	biochemical oxygen demand
BWF	base wastewater flow
Carollo	Carollo Engineers, Inc.
City	City of Scappoose
COD	chemical oxygen demand
DO	dissolved oxygen
DWF	dry weather flow
gpcd	gallons per capita day
gpda	gallons per day per acre
gpm	gallons per minute
GWI	groundwater infiltration
hp	horsepower
I/I	inflow and infiltration
IPS	influent pumping station
LS	lift station
mg/L	milligrams per liter
mgd	million gallons per day
MLSS	mixed liquor suspended solids
MMDWF	maximum month dry weather flow
MMWWF	maximum month wet weather flow
NPDES	National Pollution Discharge Elimination System
O&M	operation and maintenance
PDF	peak day flow
PIF	peak instantaneous flow
ppd	pounds per day
PSU PRC	Portland State University Population Research Center
RAS	return activated sludge
ROW	right-of-ways
RPA	reasonable potential analysis
SOR	surface overflow rate
SRT	solids retention time
SSO	sanitary sewer overflow
SVI	sludge volume index
SWD	side water depth

TSS	total suspended solids
UGB	urban growth boundary
VSS	volatile suspended solids
WAS	waste activated sludge
WTP	water treatment plant
WWF	wet weather flow
WWTP	wastewater treatment plant

EXECUTIVE SUMMARY

ES.1 Project Objectives

The Master Plan Update was prepared to identify a logical path forward for the Scappoose Wastewater Treatment Plant (WWTP) for the next twenty years. Treatment facility improvements needed to accommodate projected growth in the wastewater service area, maintain assets, and accommodate anticipated future regulatory requirements were identified and scheduled through a 20-year Capital Improvement Program (CIP). A financial plan was developed to establish rates and system development charges to implement the CIP. Key elements addressed in the Master Plan Update include:

- Wastewater flow and load projections from current conditions through the planning period,
- Flow monitoring data review of the collection system and flow projections through the planning period,
- A plan for treatment facility projects that addresses current operational issues, improves plant aesthetics, accommodates growth, and provides flexibility to adapt to a variety of potential regulatory scenarios, including changes to the current permit requirements that pertain to nitrification, and nutrient limits, and
- Financial plan to implement the treatment facility projects throughout the 20 year CIP.

The objectives for this project are:

- Provide the City with an amended planning document that identifies and details necessary WWTP capital improvements over the 20-year planning period,
- Provide the City with a state-approvable Facilities Plan that satisfies all requirements for the City applying for State Revolving Fund financing for near-term improvements, and
- Provide a predesign of the capital improvements that the City needs to complete in the near-term (first five years).

ES.2 Basis of Planning

The City manages the sewer collection system and maintains sanitary sewer lines, ranging from 4-inch to 21-inch in diameter, and five lift stations. The City's study area is illustrated in Figure ES.1.

The City operates the WWTP located on 34485 E Columbia Ave. The planning area for the current and future service area is 2,845 acres, or approximately 4.4 square miles, and is consistent with the City’s Comprehensive Plan. For this Master Plan Update, it is assumed that the City will increase its density and annex within the urban growth boundary (UGB) when required to accommodate growth.

-This Page Intentionally Left Blank-

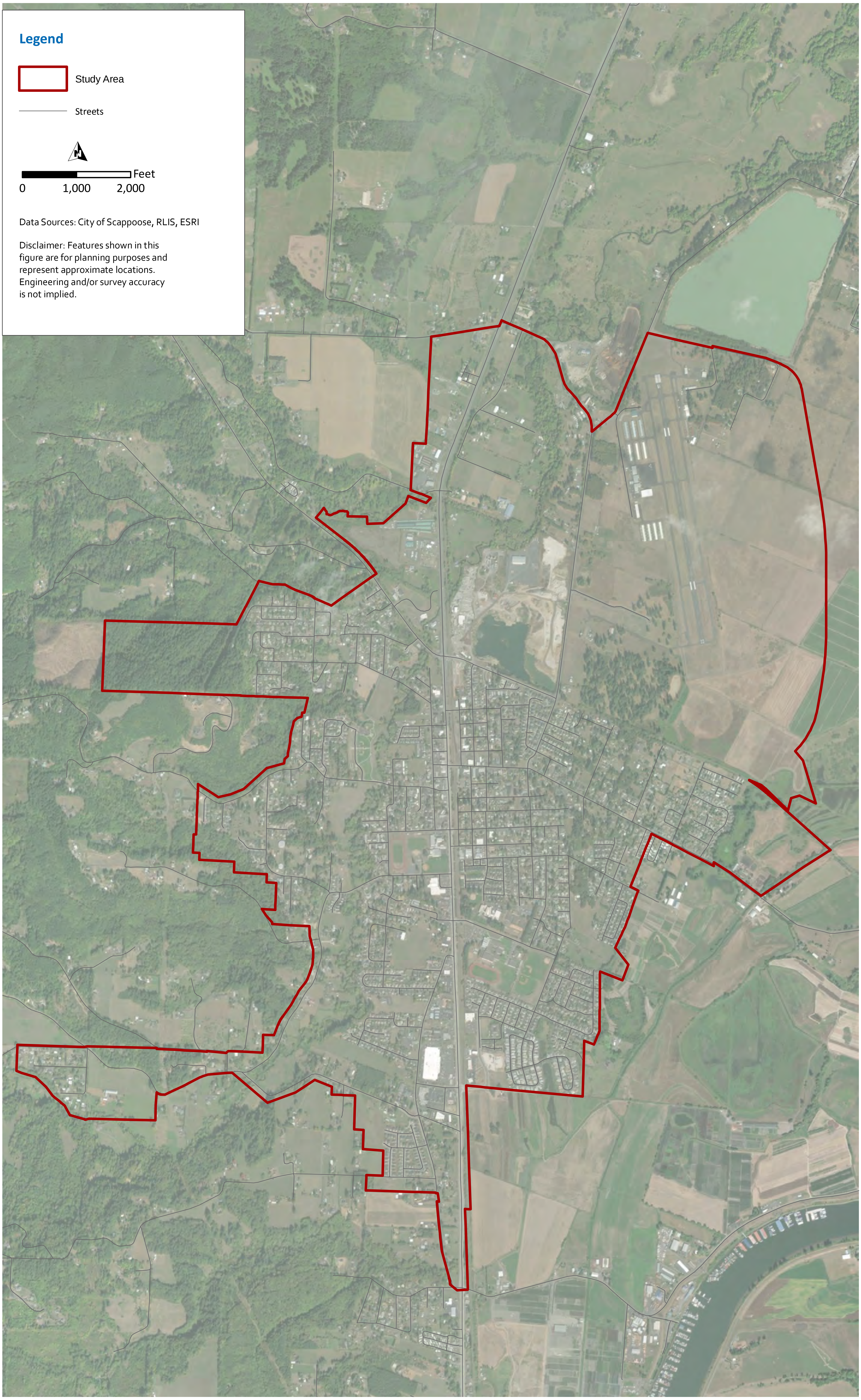


Figure ES.1 WWTP Study Area

-This Page Intentionally Left Blank-

ES.2.1.1 Population, Flow, and Load Projections

The current certified population estimates for the City of Scappoose were prepared by Portland State University (PSU) Population Research Center (PRC) based on 2010 census data. For this report, future population served by WWTP was projected based on the growth rates developed as part of that study. Figure ES.2 presents the current and projected population through year 2035 based on this approach. A summary of the current and projected flows and loads based on the projected growth is provided in Table ES.1.

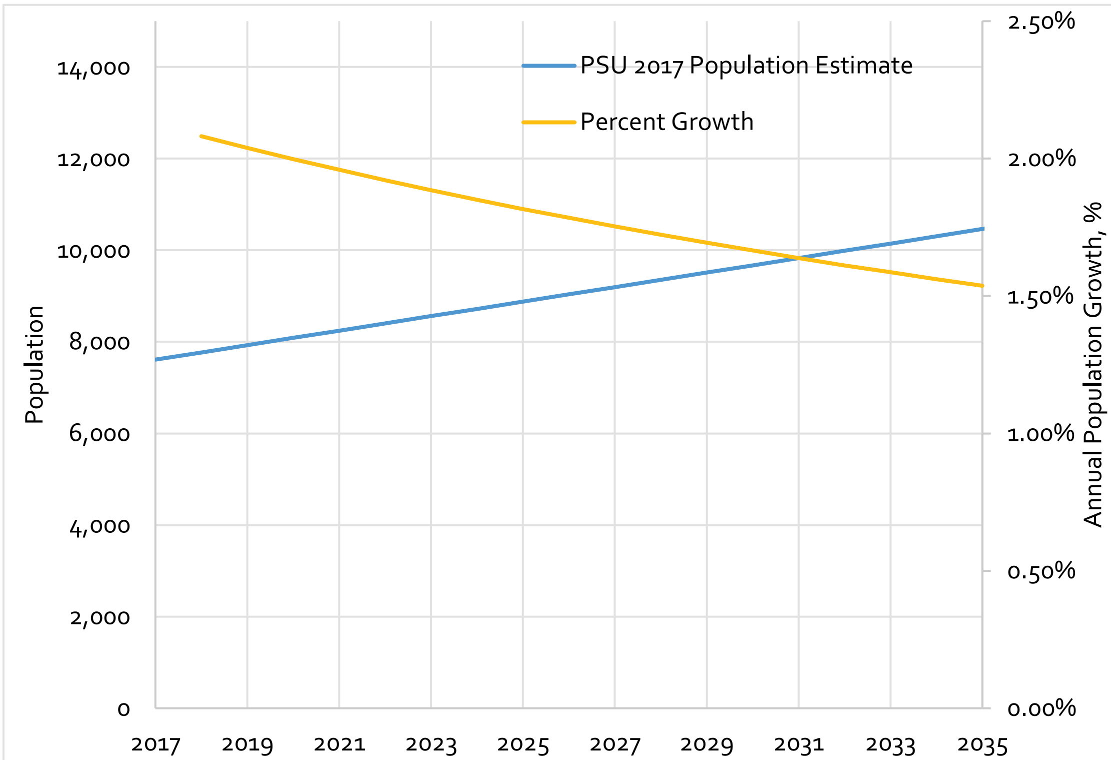


Figure ES.2 Projected Population Growth for Scappoose UGB

Table ES.1 Wastewater Flow and Load Projections

Parameter	Current	2035
Population	7,610	10,461
Flow Summary		
BWF, mgd	0.607	0.899
ADWF, mgd	0.646	0.956
AWWF, mgd	0.878	1.30
MMDWF, mgd	0.871	1.22
MMWWF, mgd	1.20	1.68
PDF, mgd	2.77	3.88
PIF, mgd	3.96	5.56

Table ES.1 Wastewater Flow and Load Projections (Continued)

Parameter	Current	2035
Load Summary		
BOD_5		
Average Annual Load, ppd	1,430	1,990
Maximum Month Load, ppd	2,180	3,030
TSS		
Average Annual Load, ppd	1,670	2,330
Maximum Month Load, ppd	2,540	3,510
Ammonia		
Average Annual Load, ppd	264	366
Maximum Month Load, ppd	450	637

ES.2.1.2 Regulatory Considerations

Water quality standards and regulations continue to evolve and there are a number of new regulatory initiatives being discussed and/or implemented at the state and federal levels. The WWTP currently discharges treated effluent into the Multnomah Channel, which is a side channel of the Lower Willamette River. The City's current National Pollutant Discharge Permit (NPDES) was last issued in 2009 and expired in October of 2014. However, it remains in effect as allowed by OAR 340-045-0040. To anticipate permit limits in the new NPDES permit, a Reasonable Potential Analysis (RPA) was completed to determine limits that would be protective of current water quality standards, including established beneficial uses and water quality criteria developed in rulemaking to protect those beneficial uses. Based on a review of the potential regulatory issues, Table ES.2 presents the anticipated future NPDES limits.

Table ES.2 Anticipated NPDES Permit Limits

Parameter	Average Monthly	Average Weekly	Maximum Daily
BOD_5 (1) (May 1 – October 31)	10 mg/L 125 ppd(21) 85% removal	15 mg/L 190 ppd	225 ppd
TSS(3) (May 1 – October 1)	10 mg/L 125 ppd 85% removal	15 mg/L 190 ppd	225 ppd
BOD_5 (November 1 – April 30)	25 mg/L 315 ppd 85% removal	37 mg/L 475 ppd	630 ppd
TSS (November 1 – April 30)	25 mg/L 315 ppd 85% removal	37 mg/L 475 ppd	630 ppd
Ph	Daily minimum and maximum between 6.0 and 9.0		
E.coli Bacteria	126/100 mL		
Total Phosphorus	No limit	No limit	No limit
Ammonia	< 5.0 mg/L (Summer)		

Notes:
(1) BOD5 = 5-day Biochemical Oxygen Demand.
(2) Ppd = pounds per day.
(3) TSS = total suspended solids.

ES.3 Collection System Flow Monitoring Data Review

Collection System flow monitoring was conducted by Keller Associates in January and February 2016. Figure ES.3 shows the locations of the five flow monitoring locations. Flow metrics such as flow versus time with rainfall, scattergraphs, and rainfall data were used to perform detailed analysis to determine suitability of five flow meters. Table ES.3 summarizes the results and explains the different data quality types.

Table ES.3 Meter Data Review Results Summary

Meter ID	Suitability	Comments
Meter 1	Unsuitable	Sign of overflow caused by blockage in the sewer line Very limited flow response to rainfall during the metering period.
Meter 2	Unsuitable	Potential velocity sensor drifting or potential influence from Smith Rd LS
Meter 3	Partially	Scattergraph looks good. Very limited flow response to rainfall, potentially not sufficient for calibration.
Meter 4	Partially	Scattergraph looks good. Very limited flow response to rainfall, potentially not sufficient for calibration.
Meter 5	Partially	Scattergraph looks reasonable. Only one rainfall events showed system flow response, when at least three events are recommended for wet weather calibration.

Based on the detailed analysis of flow monitoring data, it was concluded that there is not enough suitable data and significant flow response to rainfall to perform a good and accurate model calibration. It is recommended that the City deploy at least three flow monitors during the next rainy season to allow for wet weather model calibration. Level information is critical during model calibration as this is the metric triggering capacity deficiencies and ultimately the timing and extent of recommended improvements. The collection system CIP will be completed following the wet weather model calibration which is anticipated to occur in winter 2018.

ES.4 Existing Wastewater Treatment Facility

Figure ES.4 summarizes the current operation of the WWTP. This Master Plan Update presents the hydraulic and process capacities at the WWTP, as detailed in Chapter 3. The condition assessment completed as part of Draft Facilities Plan Update (Keller Associates, December 2016) and is attached as Appendix C to this Master Plan Update. Both these documents were used to provide a baseline for identifying improvements at the plant to address current capacity and operational issues, as well as future improvements required to meet potential changes in regulations and increases in flows and loads due to growth.

A summary of hydraulic capacity of each process area is presented in Table ES.4.

The plant secondary process was analyzed using BioWin Version 5.1. Based on the calibrated model, the existing secondary process has sufficient capacity to meet existing and anticipated NPDES permit scenarios. However, adequate redundancy and reliability is not provided through the planning period. Only one aerated lagoon is in operation. The secondary clarifiers do not have adequate capacity to pass peak flows. Additionally, with one secondary clarifier out of service, the remaining clarifier does not have capacity to pass the current and future summer peak hour flows.

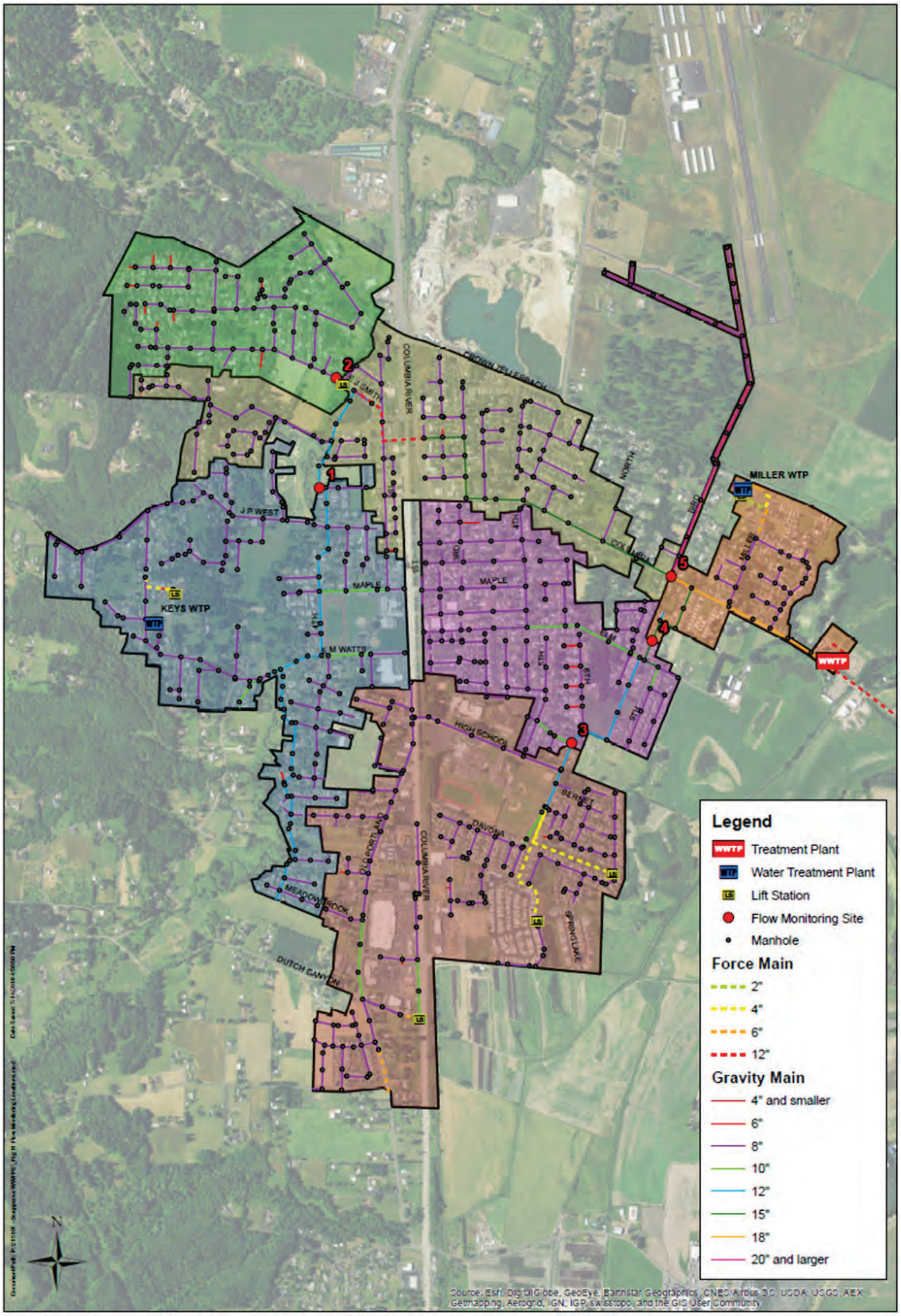


Figure ES.3 Flow Monitoring Locations

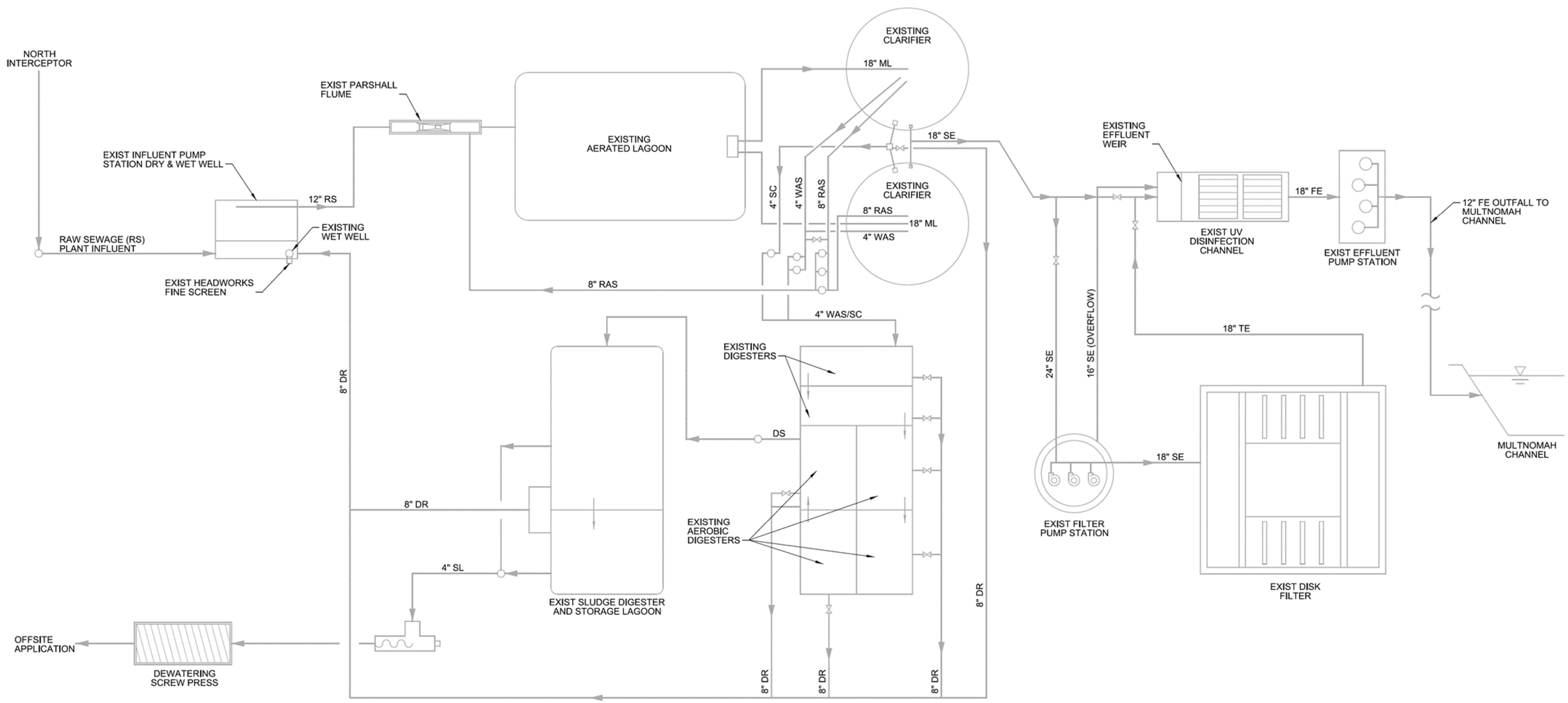


Figure ES.4 Current Plant Process Flow Diagram

-This Page Intentionally Left Blank-

Table ES.4 Hydraulic Capacity of Existing Processes

Process Area	Existing Hydraulic Capacity, mgd	Required Hydraulic Capacity, mgd[1]
Influent Screening	4.0 mgd	5.6 mgd
Influent Pumping Station	4.3 mgd	5.6 mgd
Aeration Basin	7.0 mgd[2]	5.6 mgd
Secondary Clarifiers	4.7 mgd	5.6 mgd
Tertiary Filtration System - Intermediate Pumping Station - Filtration	4 mgd 2.4 mgd	3.4 mgd[4] 2.4 mgd[5]
UV Disinfection	5.6 mgd[3]	5.6 mgd
Effluent Pump Station	4.4 mgd	5.6 mgd
Outfall	4.3 mgd	5.6 mgd

Notes:
Through planning period (year 2035)
With upstream and downstream restrictions removed.
At this flow, the V-notch weir is completely submerged and exceeds UV bank submergence limits. This limits the accurate flow measurement over the weir.
Year 2035 Summer Peak Instantaneous Flow
Year 2035 Summer Peak Day Flow

On the solids processing, the City generally stabilizes biosolids adequately with long retention time in the aeration basin, aerobic digester, and in the storage lagoon. However the existing aerobic digesters are pretty close to their capacity with existing loading conditions.

ES.5 Recommended WWTP Improvements

Improvements required to address current operational issues, accommodate growth, and provide flexibility to adapt to potential regulatory scenarios were identified through evaluation of unit process alternatives and integration of overall plant facilities. The identified projects were costed and reviewed with City staff to capture WWTP requirements for growth, redundancy, reliability, condition, and current operational issues.

The cost estimates are for general planning purposes and for guidance in project evaluation and implementation. Costs were estimated by developing preliminary layouts and quantity takeoffs, and using quotes from equipment manufacturers. The costs are based on an Engineering News Record Construction Cost Index (ENR CCI) 20-City Average of 10,842 (August 2017). Construction costs do not include contingencies. Electrical/instrumentation /controls costs were estimated by applying a factor to the base construction cost. Factors for mobilization/demobilization and contractor's overhead and profit were applied to the sum of the base construction costs and costs for electrical/instrumentation/controls. The total estimated construction costs include the work items described, plus mark-up costs for electrical/instrumentation controls, mobilization/demobilization, and contractor's overhead and profit. Contingencies were estimated by multiplying the sum of the estimated construction costs by a mark-up factor. The total project costs were calculated by multiplying the sum of the estimated construction cost and contingencies by a factor to account for engineering, legal and administrative costs. These mark-ups are shown in Table ES.5.

Table ES.5 Mark up Factors Used in Developing Cost Estimates for Alternatives

Item	Markup
Mobilization/Demobilization	10%
Contractor Overhead and Profit	15%
Contingencies	30%
Engineering, Legal and Administrative	25%

ES.5.1 Capital Improvement Plan (CIP)

Recommended unit process improvements were reviewed with City staff and grouped into a program of four capital improvement phases. The recommended capital improvements at the WWTP are identified on Table ES.6 and shown on Figure ES.5.

The first phase of capital improvement projects address deficiencies at the Spring Lake Lift Station in the collection system, and UV bank replacement, hydraulic improvements, a new secondary clarifier along with RAS/WAS upgrades, and aerobic digester life safety improvements at the WWTP. These improvements address immediate failing process/equipment and provide redundancy/reliability to the secondary process.

The second phase of capital improvement projects address condition, hydraulic deficiencies and accommodate growth in a headworks facility. The projects include a new headworks, influent pumping station, and operational improvements.

The third phase of capital improvement projects include upgrades and expansion to alleviate hydraulic issues and provide redundancy to critical processes. The projects include the addition of a second UV disinfection channel, upgrades to effluent pump station and a new parallel outfall.

The fourth phase addresses capacity and accommodates growth on the solids treatment with a new aerobic digester.

Table ES.6 Implementation Program and Phasing Schedule

Item Description	Description	Construction Year				Total
		Phase 1 2018 – 2021	Phase 2 2021 - 2023	Phase 3 2023 - 2026	Phase 4 2026 – 2028	
Phase 1 Improvements						
Spring Lake Lift Station	Replacement of pumps and corroded piping; addition of valve vault to measure flow.	$271, 600				$6,430,600
UV Disinfection	Replacement of existing UV banks with newer technology UV system (Trojan 3000Plus).	$616,600				
Hydraulic Improvements	Rebuilding secondary splitter structure.	$519,700				
Secondary Clarifier and RAS/WAS Pumping Upgrades	Addition of third 50-foot secondary clarifier, update existing clarifier wiring, and expand RAS/WAS pumping.	$4,590,100				
Aerobic Digester Life Safety Improvements	Replace damaged coarse bubble diffusers, replace two blowers, sludge pump, sludge flow meter, fix hand rails.	$432,600				
Phase 2 Improvements						
Headworks and Influent Pump Station	New two fine screens with passive bypass channel and submersible influent pump station.		$5,504,400			$7,204,400
Operational Improvements	SCADA integration and new lab.		$1,700,000			
Phase 3 Improvements						
UV Disinfection	Addition of second channel and equipment.			$1,685,900		$4,393,200
Effluent Pump Station	Replacement of pumps, modification of skylights, addition of flow meter, electrical improvements.			$536,600		
Outfall	New parallel 15-inch outfall.			$2,170,700		
Phase 4 Improvements						
Aerobic Digester	New aerobic digester to achieve Class B biosolids.				$2,486,900	$2,486,900
TOTAL WWTP CIP						**$20,515,000**

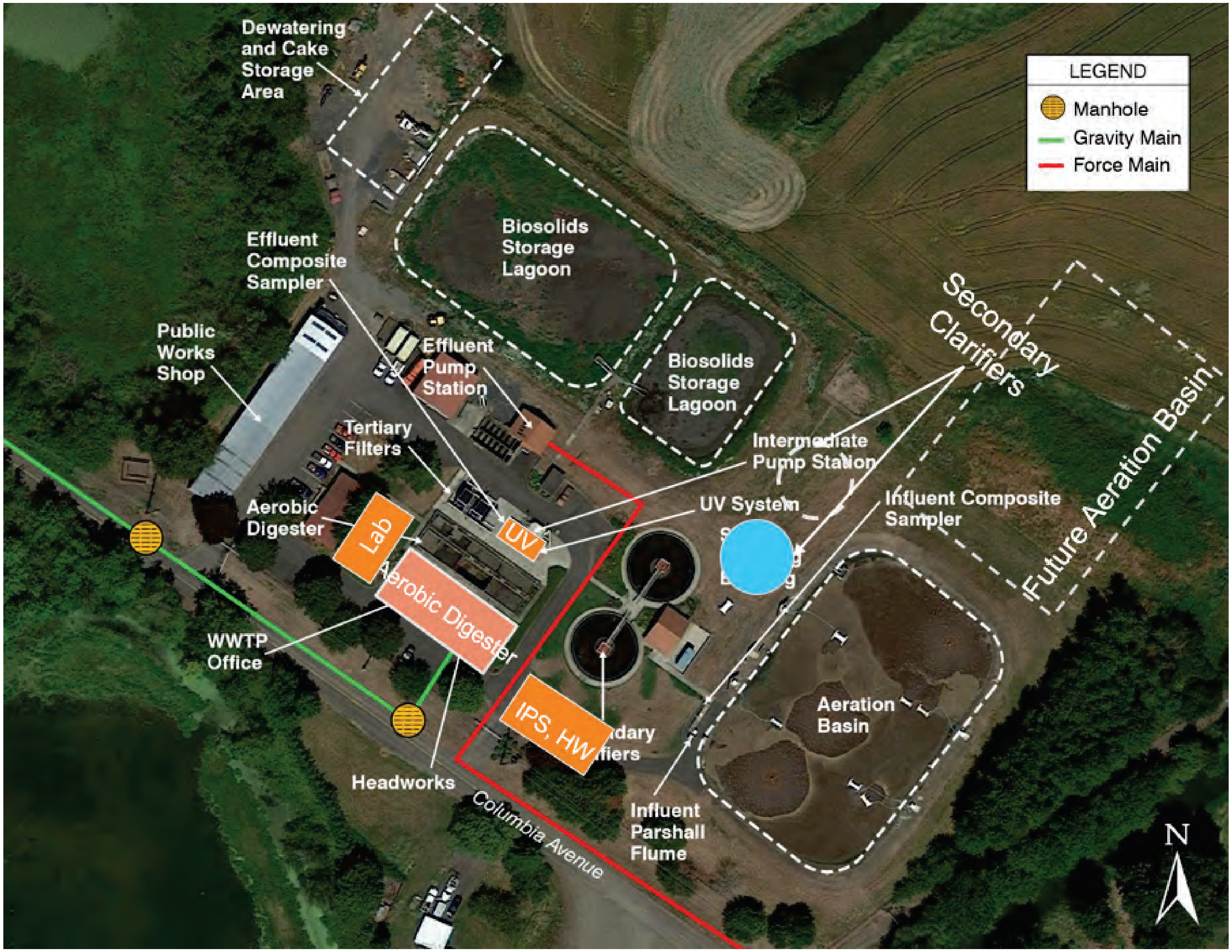


Figure ES.5 Recommended Site Layout

-This Page Intentionally Left Blank-

Chapter 1

BASIS OF PLANNING

1.1 Purpose

The purpose of this Basis of Planning Report is to define the information needed to select and size the facilities for the Wastewater Treatment Plant (WWTP). This report presents the following:

- Current and anticipated discharge requirements,
- Historical wastewater flows and loads,
- Flow and load projections based on anticipated population growth,
- Reliability and redundancy requirements for each facility under consideration for replacement or expansion, and
- Design criteria for each facility under consideration for replacement or expansion.

1.2 Background

The City of Scappoose owns and operates a wastewater treatment plant (WWTP) which services the City of Scappoose as well as several industrial clients. The WWTP currently discharges treated effluent into the Multnomah Channel, which is a side channel of the Lower Willamette River. The City's current National Pollutant Discharge Permit (NPDES) was last issued in 2009 and expired in October of 2014. However, it remains in effect as allowed by OAR 340-045-0040.

The objectives for this project are:

- Provide the City with an amended planning document that identifies and details necessary WWTP capital improvements over the 20-year planning period,
- Provide the City with a state-approvable Facilities Plan that satisfies all requirements for the City applying for State Revolving Fund financing for near-term improvements, and
- Provide a predesign of the capital improvements that the City needs to complete in the near-term (first five years).

1.3 Current Discharge Permit Requirements

Discharges from wastewater treatment plants to surface waters must be permitted by a NPDES Permit as required by the federal Clean Water Act and the promulgated regulations implementing the requirements of the Clean Water Act. NPDES permit limitations on discharges are established so that in-stream water quality criteria are met as well as other water quality standards and regulations. The City's current effluent limits are summarized in Table 1.1.

Table 1.1 Existing NPDES Permit Limits

Parameter	Average Monthly	Average Weekly	Maximum Daily
BOD_5 [1] (May 1 – October 31)	10 mg/L 125 ppd[2] 85% removal	15 mg/L 190 ppd	225 ppd
TSS[3] (May 1 – October 1)	10 mg/L 125 ppd 85% removal	15 mg/L 190 ppd	225 ppd
BOD_5 (November 1 – April 30)	25 mg/L 315 ppd 85% removal	37 mg/L 475 ppd	630 ppd
TSS (November 1 – April 30)	25 mg/L 315 ppd 85% removal	37 mg/L 475 ppd	630 ppd
pH	Daily minimum and maximum between 6.0 and 9.0		
E.coli Bacteria	126/100 mL		
Total Phosphorus	No limit	No limit	No limit
Ammonia	No limit	No limit	No limit

Notes:
(1) BOD_5 = 5-day Biochemical Oxygen Demand.
(2) ppd = pounds per day.
(3) TSS = total suspended solids.

1.4 Expected Future Discharge Permit

To anticipate permit limits in the new NPDES permit, a Reasonable Potential Analysis (RPA) was completed to determine limits that would be protective of current water quality standards, including established beneficial uses and water quality criteria developed in rulemaking to protect those beneficial uses.

The RPA analysis was completed using current Excel Workbooks authored by DEQ for evaluating the potential for both ammonia toxicity and metals toxicity. The ammonia RPA workbook had been recently revised to reflect newly adopted ammonia criteria based on the EPA's 2013 Ammonia Toxicity Criteria revisions – developed to protect endangered fresh water mussels. The analysis of potential metals toxicity evaluated potential permit limits driven by both water quality criteria and human health criteria. Although the workbook for the human health criteria allows evaluation of metals and a wide range of organic toxins, no data on potential organic toxins were available and the RPA was limited to human health criteria for very limited data on metals collected in 2006 – 2008 only.

Based on this limited metals data the RPA determined that there was no reasonable potential for end-of-pipe exceedances of water quality criteria for any metals discharged from the WWTP. Therefore, no NPDES permit limits for metals are anticipated. However, zinc concentrations seemed higher than normal and is recommended that the City proactively pursue source reduction. The RPA did indicate that there is a potential for water quality criteria violations, when effluent ammonia concentrations exceed 5.0 mg/l. It is therefore likely that the new NPDES permit will have ammonia limits less than 5 mg/l. Table 1.2 summarizes the expected future permit limits for the WWTP.

Table 1.2 Expected Future NPDES Permit Limits

Parameter	Average Monthly	Average Weekly	Maximum Daily
BOD_5 (1) (May 1 – October 31)	10 mg/L 125 ppd(21) 85% removal	15 mg/L 190 ppd	225 ppd
TSS(3) (May 1 – October 1)	10 mg/L 125 ppd 85% removal	15 mg/L 190 ppd	225 ppd
BOD_5 (November 1 – April 30)	25 mg/L 315 ppd 85% removal	37 mg/L 475 ppd	630 ppd
TSS (November 1 – April 30)	25 mg/L 315 ppd 85% removal	37 mg/L 475 ppd	630 ppd
Ph	Daily minimum and maximum between 6.0 and 9.0		
E.coli Bacteria	126/100 mL		
Total Phosphorus	No limit	No limit	No limit
Ammonia	< 5.0 mg/L (Summer)		

Notes:
(1) BOD5 = 5-day Biochemical Oxygen Demand.
(2) Ppd = pounds per day.
(3) TSS = total suspended solids.

1.5 Flow and Load Analysis

The WWTP receives domestic, commercial, and industrial wastewater from the City of Scappoose. The following sections summarize the existing and future loads for the WWTP.

1.5.1 Definitions

The flow parameters of primary interest for this Project are defined as follows:

1. **Base Wastewater Flow (BWF).** The average daily flow over the three-month summer period of July through September.
2. **Average Dry Weather Flow (ADWF).** The average of daily flows over the dry weather season between May and October.
3. **Average Wet Weather Flow (AWWF).** The average flow at the plant during the wet weather season (November through May) during a year with average rainfall.
4. **Maximum Month Dry Weather Flow (MMDWF).** Defined by DEQ as the maximum monthly average dry weather flow with a 10% probability of occurrence.
5. **Maximum Month Wet Weather Flow (MMWWF).** Defined by DEQ as the maximum monthly average wet weather flow with a 20% probability of occurrence.
6. **Peak Day Flow (PDF).** Defined by DEQ as the peak daily average flow associated with a 5-year storm.
7. **Peak Instantaneous Flow (PIF).** Defined by DEQ as the peak hour flow sustained for one hour during a 5-year PDF.

1.5.2 Rainfall Records

Rainfall increases flow rates during the wet weather season. DEQ flow analysis guidelines incorporate rainfall records into the recommended statistical analysis. Daily rainfall data collected at the plant was used to develop the flow analysis. Statistical summaries of climatological data prepared by National Oceanic and Atmospheric Administration (NOAA) were also used in the analysis. NOAA prepares statistical summaries of climatologic data for selected meteorological stations. The most recent climatologic statistical summary for weather stations surrounding Scappoose was issued in 2004 and is based on data collected from 1971 through 2000.

1.5.3 Population

The current certified population estimates for the City of Scappoose were prepared by Portland State University (PSU) Population Research Center (PRC) based on 2010 census data. For this Report, future population served by WWTP was projected based on the growth rates developed as part of that study. Figure 1.1 presents the current and projected population through year 2035 based on this approach.

Population data is used to assign a per capita flow factor and assess future flows and loads. The historical per capita flow is calculated from base wastewater flow and population. The per capita flow factor, along with population projections, is used to predict future flow at the WWTP.

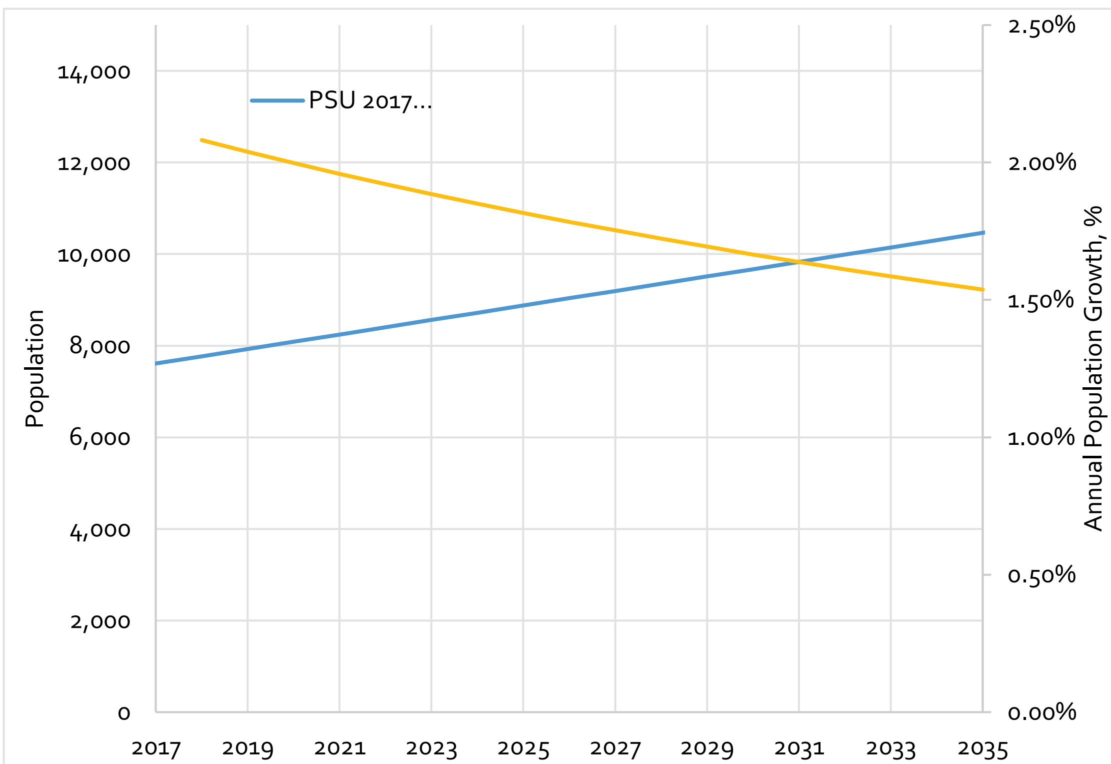


Figure 1.1 Projected Population Growth for Scappoose UGB

1.5.4 Industrial Contributions

The City of Scappoose Economic Opportunities Analysis (EOA) was prepared by Johnson Reid and adopted as an amendment to the Scappoose Comprehensive Plan in 2011. The EOA's forecast anticipates the commercial and industrial land demand (including streets, utilities, etc.) as 483 acres by 2030. Estimates made by the City Development Code were used to establish average lot size by zoning category. Based on the forecasting, the City of Scappoose expects to see approximately 440 acres of future commercial growth equating to a population growth of 6,400.

1.5.5 Historical Flow Analysis

The current flows for the WWTP were established through analysis of historical flow records from January 2010 through February 2017.

1.5.5.1 Base Wastewater Flow

BWF is determined based on the average plant flow during periods of minimal rainfall, defined in this case as the plant flow during the months of July through September. BWF is used as the basis for establishing peaking factors and per capita flows. The BWF of 0.607 mgd for the WWTP was calculated by averaging the observed BWF over the 2014-2016 period, since the most recent BWF values appear to have decreased significantly since 2010. These observed BWF values are depicted in Figure 1.2 and Table 1.1. The estimated per capita flow over this period was 80.8 gpcd.

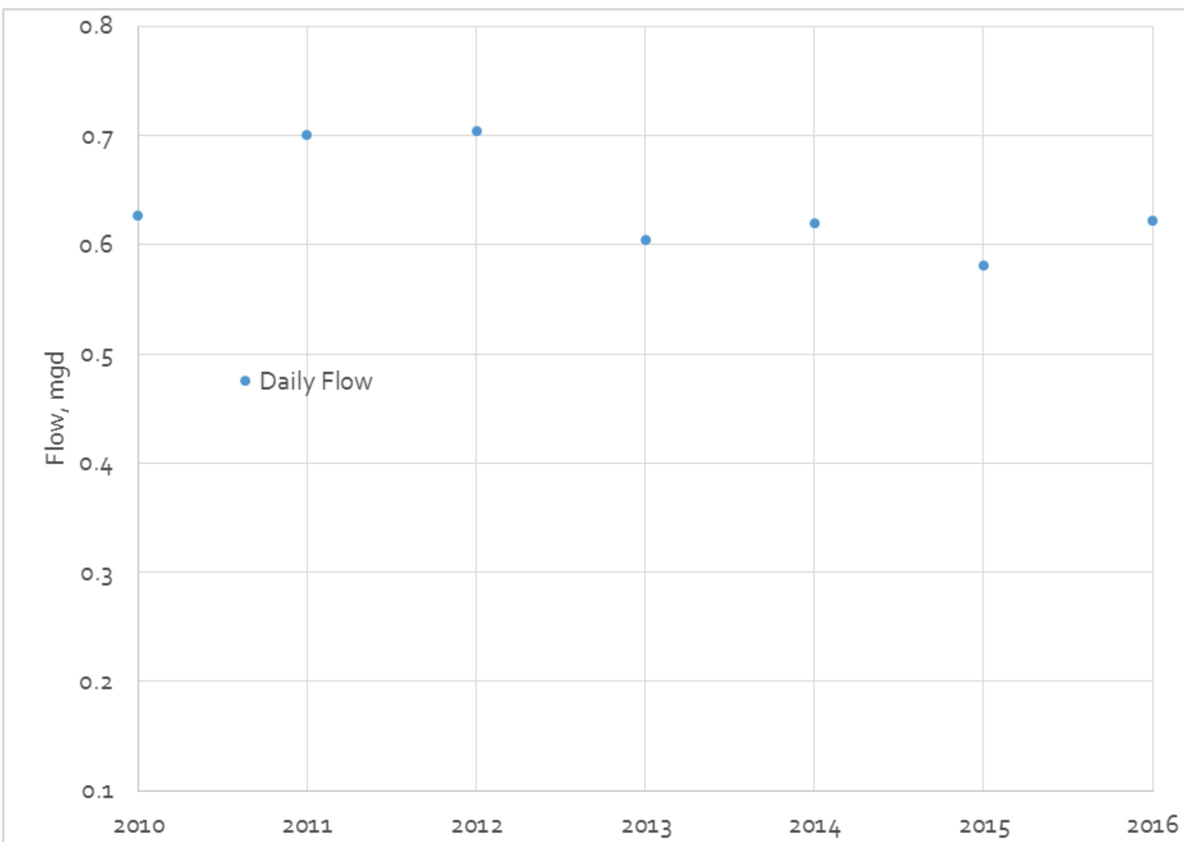


Figure 1.2 Historical Base Wastewater Flow

Table 1.3 Historical Base Wastewater Flows

Year	Population	Base Wastewater Flow, mgd	Per Capita Flow, gpcd
2010	7,269	0.627	86.3
2011	7,318	0.700	95.7
2012	7,366	0.704	95.5
2013	7,415	0.604	81.5
2014	7,464	0.619	83.0
2015	7,513	0.581	77.3
2016	7,561	0.622	82.2
Overall Average		0.623	85.9
2014-2016 Average		0.607	80.8

1.5.5.2 Average Dry and Wet Weather Flows

Table 1.4 presents the seasonal summary of rainfall and influent plant flows for the period from January 2010 through February 2017. The DEQ methodology uses the relationship between plant flow and mean climatological conditions experienced in the WWTP's service area to estimate ADWF and AWWF. However, the seasonal values shown in Table 1.4, Figure 1.3 (ADWF), and Figure 1.4 (AWWF) indicate that there is no strong correlation between influent flows and rainfall. Therefore an average of data from 2010 through 2016 was used to estimate both ADWF and AWWF. The estimated ADWF was 0.660 mgd, and the estimated AWWF was 0.892 mgd.

Table 1.4 Historical Average Dry and Wet Weather Flows

Season	Year	Total Rainfall, inches	Average Plant Influent Flow, mgd
Dry Season	2010	14.49	0.677
	2011	7.95	0.695
	2012	12.18	0.718
	2013	13.88	0.659
	2014	11.58	0.634
	2015	7.23	0.586
	2016	14.87	0.649
	Average	11.74	0.660
Wet Season	2010	37.4	0.870
	2011	33.55	0.871
	2012	43.49	0.960
	2013	13.2	0.771
	2014	31.07	0.824
	2015	34.73	0.928
	2016	35.53	1.017
	Average	32.71	0.892

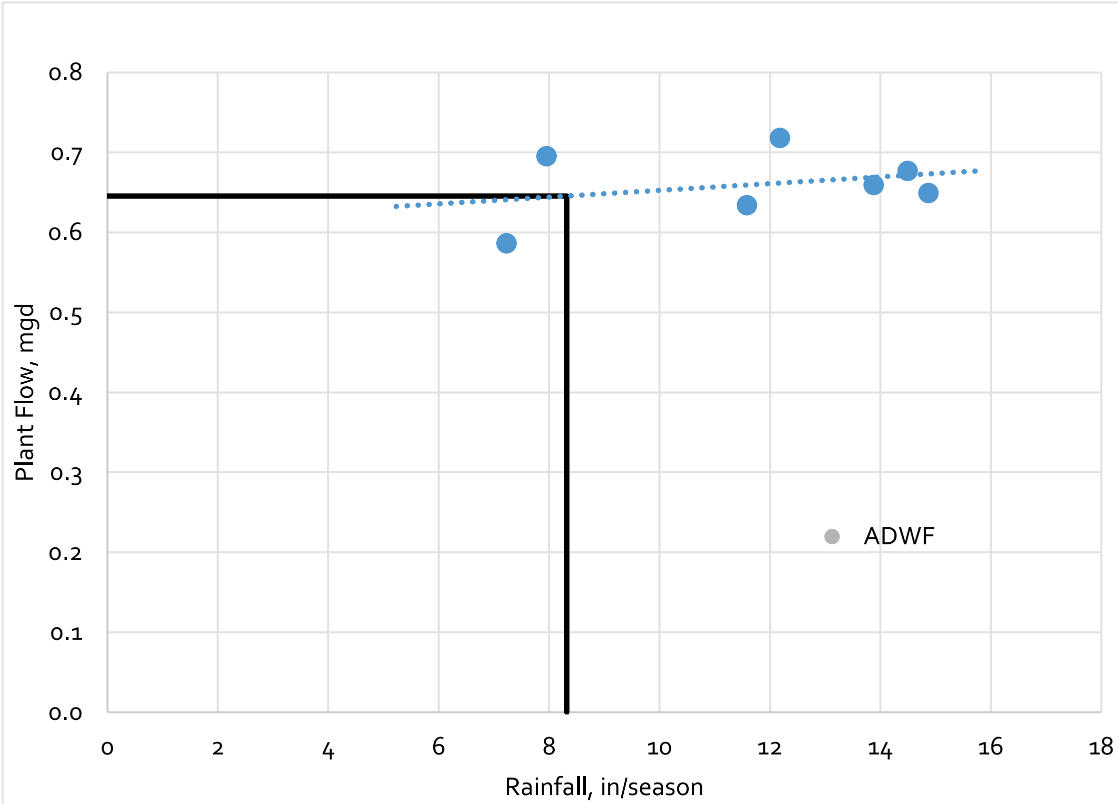


Figure 1.3 Average Dry Weather Flows

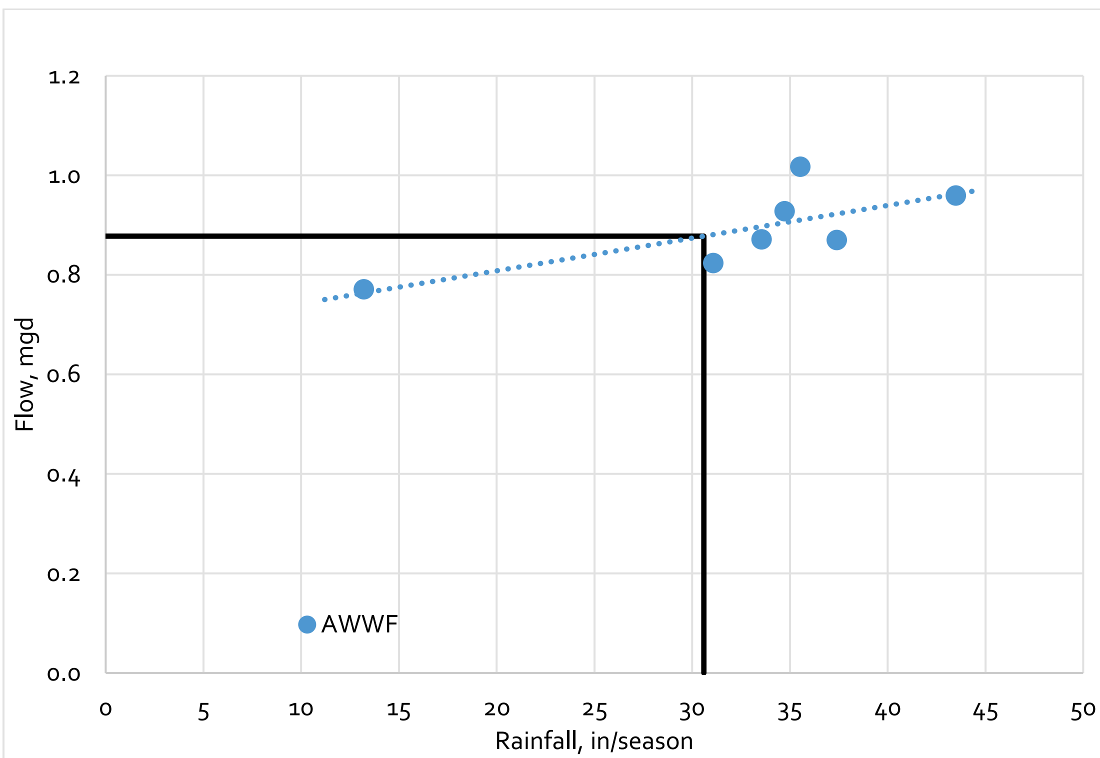


Figure 1.4 Average Wet Weather Flows

1.5.5.3 Maximum Month Flows

The DEQ methodology for estimating maximum month flows consists of plotting monthly average plant flow for the months of January through May against the corresponding monthly rainfall, and developing a linear relationship between flow and rainfall.

The MMDWF is defined as the flow that is expected to occur with a rainfall with a 1-in-10 year probability for the wettest month of the dry weather season. October is the wettest dry weather month for the area, but the average May rainfall is used for this analysis because groundwater levels are higher in the spring resulting in higher flows due to increased infiltration. For Scappoose, the 1-in-10 year May rainfall is 4.16 inches. By approximating a linear relationship between the influent flow and rainfall, as illustrated in Figure 1.5, the MMDWF is estimated at 0.871 mgd.

Similarly, the MMWWF is defined as the flow expected to occur when rainfall is at the 1-in-5 year high rainfall (9.58 inches). For the WWTP service area, December is the wettest month of the year, but the average January rainfall is used for this analysis because groundwater levels are higher in January. From Figure 1.5, the MMWWF is estimated at 1.20 mgd.

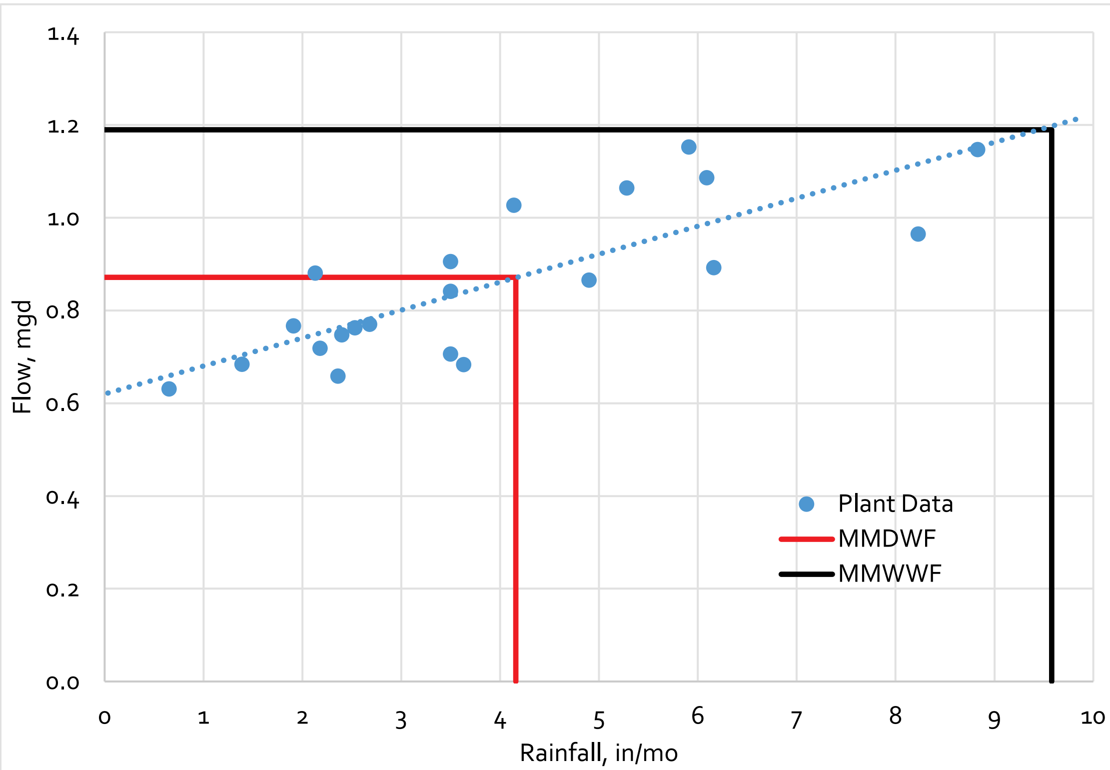


Figure 1.5 Maximum Month Flows

The observed max month flows over the study period are shown in Table 1.5. The range of MMDWFs observed from 2010 through 2016 is 0.690 to 0.905 mgd, while the range of MMWWFs observed during the same period is 1.05 to 1.38 mgd (Table 1.3).

The MMDWF of 0.871 mgd calculated using the DEQ method corresponds well with the maximum month flows observed during the last 7 years, and was therefore selected as the current design MMDWF.

Table 1.5 Historical Maximum Month Dry and Wet Weather Flows

Year	MMDWF, mgd	MMWWF, mgd
2010	0.905	1.05
2011	0.835	1.11
2012	0.904	1.17
2013	0.821	1.15
2014	0.841	1.05
2015	0.690	1.33
2016	0.889	1.38

1.5.5.4 Peak Day and Peak Instantaneous Flows

The MMWWF of 1.20 mgd calculated using the DEQ method corresponds well with the maximum month flows observed during the last 7 years, and was therefore selected as the current design MMWWF.

The peak day flow (PDF) is defined as the daily average plant flow rate that occurs during the 1-in-5 year, 24-hour storm event. For the WWTP service area, the 1-in-5 year, 24-hour storm corresponds to 2.8 inches of rain (NOAA Atlas 2 Volume X, Figure 26). According to DEQ's methodology, PDF is estimated based on the linear relationship that exists between the daily average plant influent flow data during significant wet season storm events and daily rainfall. In Figure 1.6, only those days with over 1.5 inches of recorded rainfall and days with at least 2 inches of cumulative rainfall in the previous four days were considered. This ensures that the soils were saturated and I/I contributions were significant. Linear regression of historical data for rainfall and plant flow observed the next day indicates a correlation, and extrapolating the line to the 1-in-5 year, 24-hour storm rainfall yields an estimated PDF of 2.77 mgd. This matches well with the observed flow of 2.63 mgd observed the day after a 2.68 inch rainfall in November 2016, close to the 1-in-5 year, 24-hour storm rainfall.

The DEQ methodology for estimating PIF uses a probability analysis. This analytical technique assumes that the AAF, MMWWF, PDF, and PIF will occur such that the recurrence probabilities associated with each of the flows are as follows:

- AAF is exceeded half the time (50% probability).
- MMWWF is exceeded during one month (8.3% probability).
- PDF is exceeded on one day (0.27% probability).
- PIF is exceeded during one hour (0.011% probability).

The resulting PIF of 6.03 mgd is presented in Figure 1.7. This PIF is unrealistically high based on observed PDF to PIF peaking factors during high storm events (1.10 to 1.50). Using a PDF to PIF peaking factor of 1.24 observed during November 2016 event, the recommended PIF is 3.43 mgd. Including this initial estimate in the probability analysis used to estimate PIF yields a more realistic value of 3.96 mgd, which was selected as the design value in order to provide some additional conservatism over the 3.43 mgd value.

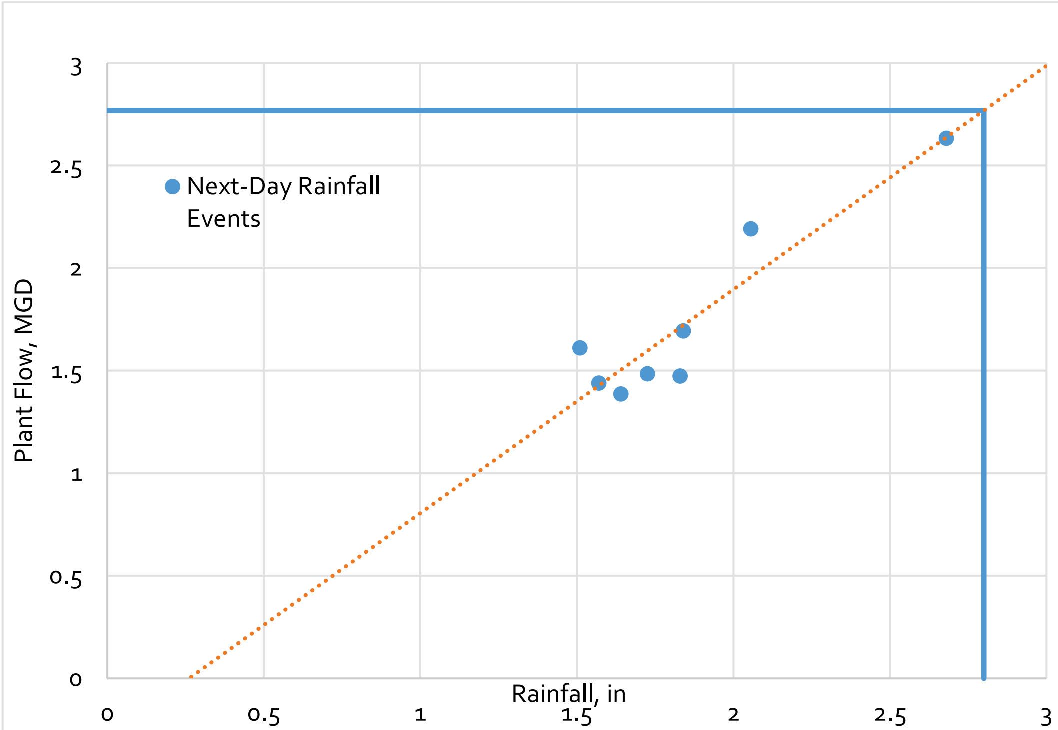


Figure 1.6 Peak Day Flows

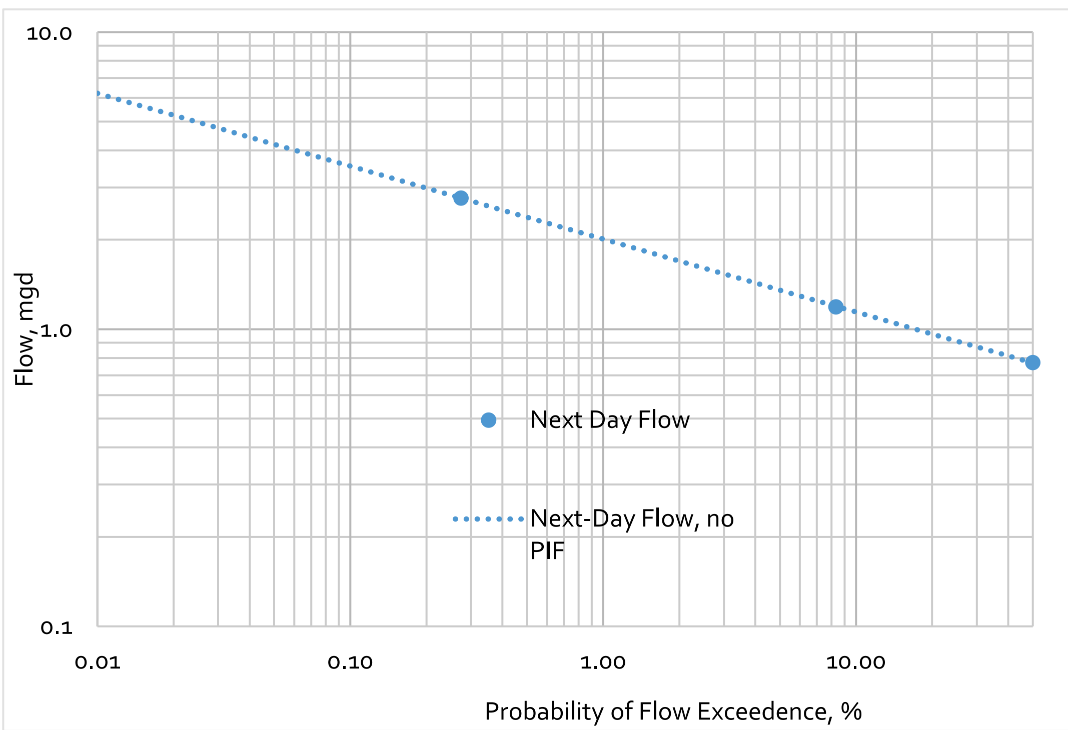


Figure 1.7 PIF using DEQ Methodology

1.5.5.5 Influent Flow Summary

Table 1.6 provides a summary of the current influent flows from the WWTP service area, along with their associated peaking factors. The peaking factor is calculated based on the BWF. As an example, the peaking factor for MMWWF is the ratio of the MMWWF of 1.20 mgd to the BWF of 0.607 mgd which results in a peaking factor of 1.97.

Table 1.6 Current Wastewater Flows

Flow Parameter	Flow, mgd	Peaking Factor(1)
Base Wastewater Flow (BWF)	0.607	1.00
Average Dry Weather Flow (ADWF)	0.646	1.06
Average Wet Weather Flow (AWWF)	0.878	1.45
Maximum Month Dry Weather Flow (MMDWF)	0.871	1.43
Maximum Month Wet Weather Flow (MMWWF)	1.20	1.97
Peak Day Flow (PDF)	2.77	4.56
Peak Instantaneous Flow (PIF)	3.96	6.52

Notes:
(1) Peaking factor based on BWF.

1.5.6 Historical Wastewater Loads Analysis

1.5.6.1 BOD Loading Analysis

Daily BOD_5 loads for the period of January 2010 to February 2017 are presented in Figure 1.8. The BOD_5 loading has increased steadily over this period. Because of the increasing trend in BOD loads, average annual load was determined by averaging the daily loads over the most recent three-year period from January 2014 to December 2016. The maximum month BOD load was calculated as the maximum of the maximum month loads observed in the entire period of record. Based on these data, the average annual wastewater loading was 1,430 ppd, and the maximum month loading was 2,240 ppd as presented in Table 1.7. The BOD peaking factor for the maximum month loading is defined as the maximum month BOD load divided by the average annual load, resulting in a maximum month peaking factor of 1.57.

Table 1.7 Current BOD Loading

Load Parameter	BOD Load, ppd	Peaking Factor
Average Annual Load (AA)	1,430	1.00
Maximum Month Load (MM)	2,180	1.51

1.5.6.2 TSS Loading Analysis

Daily TSS loads for the period of January 2010 to February 2017 are presented in Figure 1.9. Similar to the BOD loads, the TSS loading has also consistently increased over this period. The average annual TSS load was determined by averaging the daily loads over the most recent three-year period from January 2014 to December 2016. The maximum month TSS load was calculated as the average of maximum month loads observed in the entire period of record. Based on these data, the average annual wastewater TSS loading was 1,670 ppd, and the maximum month loading was 2,540 ppd (Table 1.8). The TSS peaking factor for the maximum month loading is defined as the maximum month TSS load divided by the average annual load, resulting in a maximum month peaking factor of 1.52.

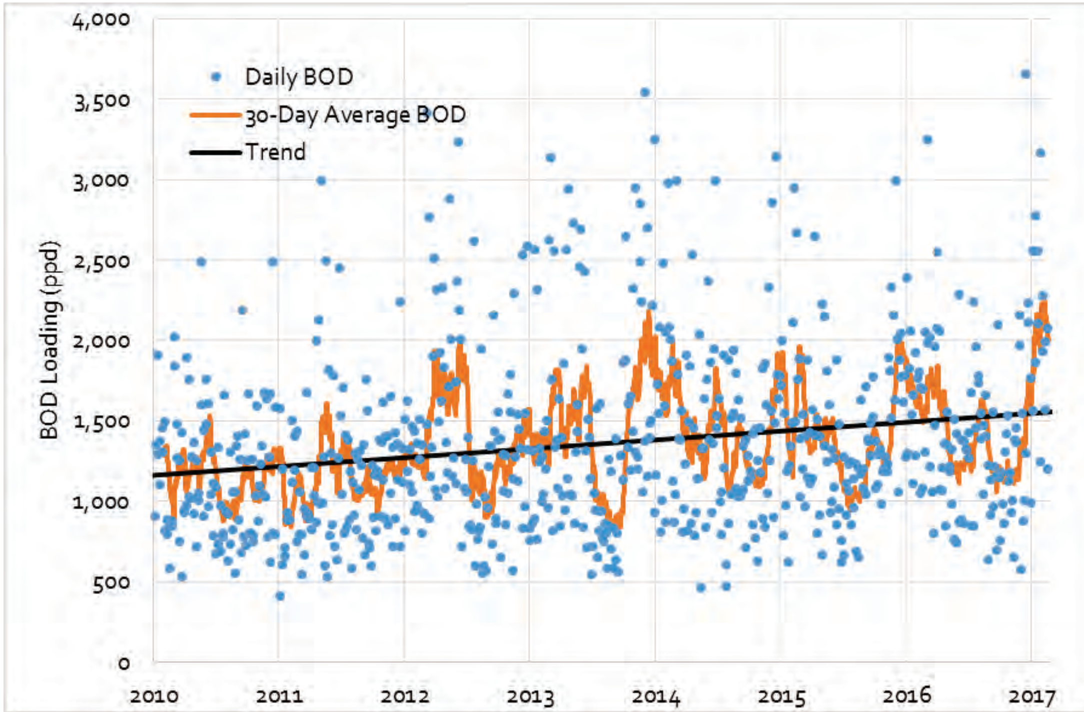


Figure 1.8 Historical BOD Loading

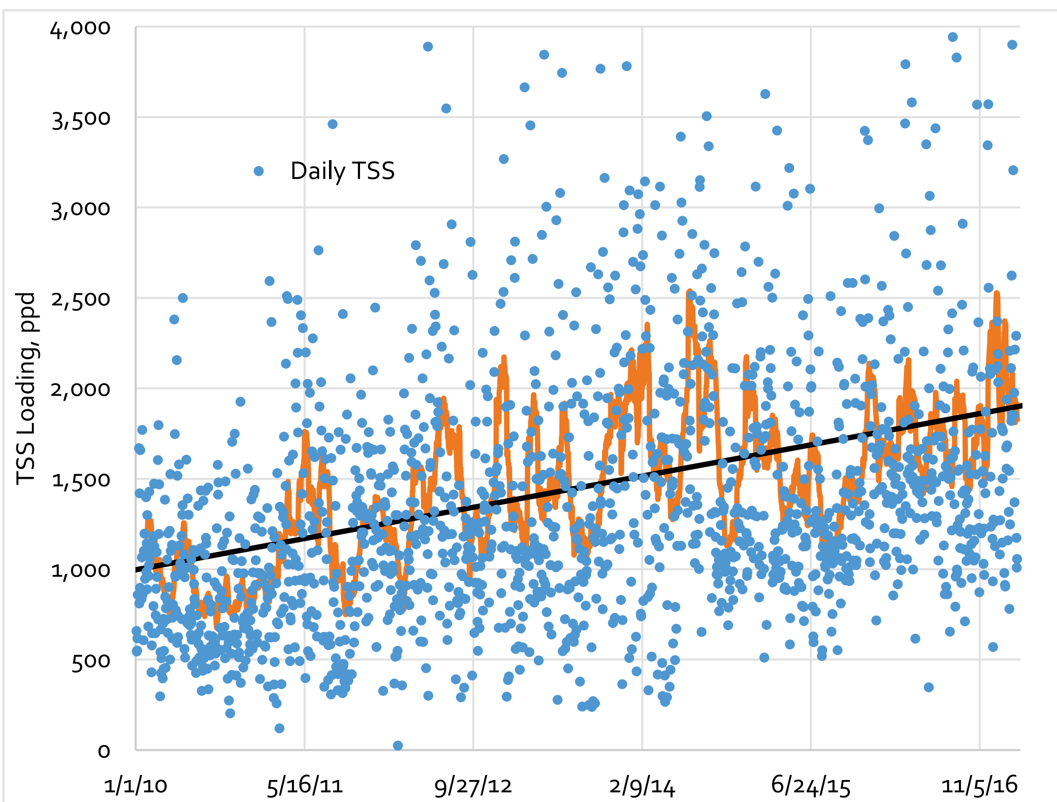


Figure 1.9 Historical TSS Loading

Table 1.8 Current TSS Loading

Load Parameter	TSS Load, ppd	Peaking Factor
Average Annual Load (AA)	1,670	1.00
Maximum Month Load (MM)	2,540	1.52

1.5.6.3 Ammonia Loading Analysis

Daily ammonia loads for the period of January 2010 to February 2017 are presented in Figure 1.10. Similar to the BOD and TSS loads, the ammonia loading has also increased over this period, with a relatively sharp increase in 2016 due to sampling and testing methodology change. The average annual ammonia load was determined by averaging the daily loads over the most recent three-year period from January 2014 to December 2016. The maximum month ammonia load was calculated as the average of maximum month loads observed in the entire period of record. Based on these data, the average annual wastewater ammonia loading was 263 ppd, and the maximum month loading was 467 ppd (Table 1.9). The ammonia peaking factor for the maximum month loading is defined as the maximum month ammonia load divided by the average annual load, resulting in a maximum month peaking factor of 1.78.

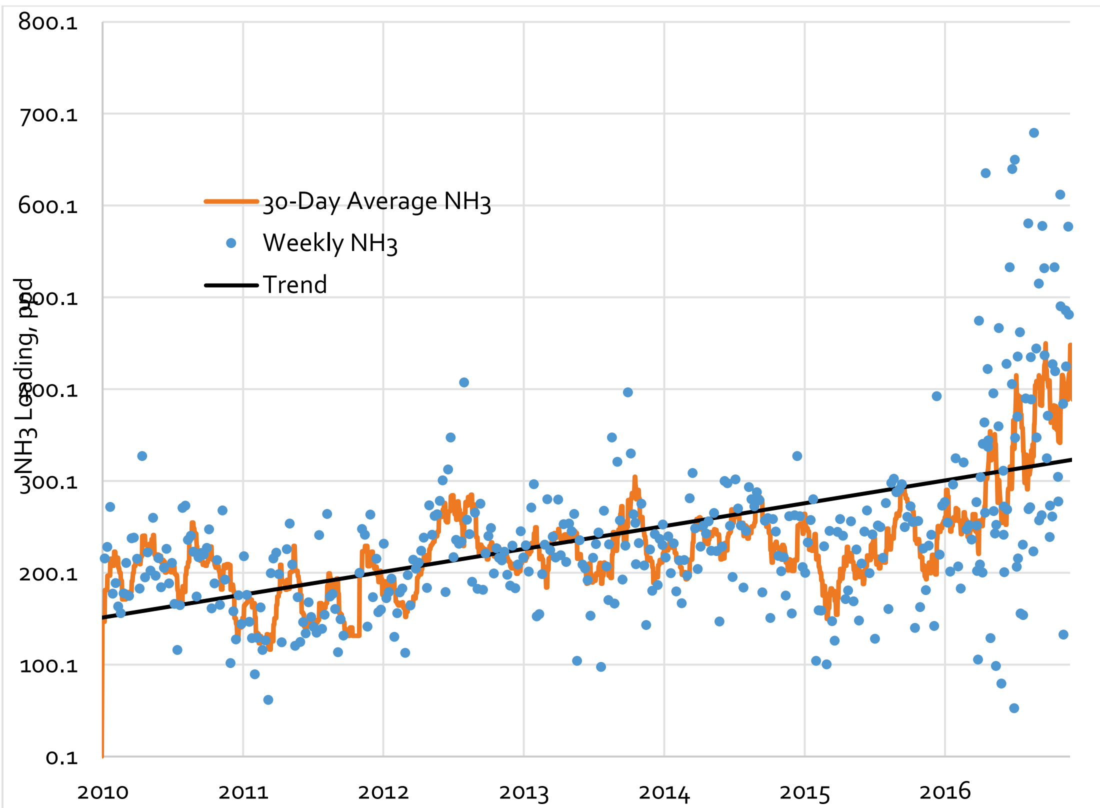


Figure 1.10 Historical Ammonia Loading

Table 1.9 Current Ammonia Loading

Load Parameter	Ammonia Load, ppd	Peaking Factor
Average Annual Load (AA)	263	1.00
Maximum Month Load (MM)	467	1.78

1.5.7 Flow Projections

Table 1.10 provides the WWTP flow projections to 2035. The future BWF, ADWF, and AWWF are projected by multiplying the anticipated population served by the WWTP by the estimated per capita flow. MMDWF, MMWWF, PDF, and PIF are projected by growing each flow by the anticipated population growth rate.

Table 1.10 Wastewater Flow Projections

Parameter	Current	2035
Population	7,610	10,461
BWF, mgd	0.607	0.899
ADWF, mgd	0.646	0.956
AWWF, mgd	0.878	1.30
MMDWF, mgd	0.871	1.22
MMWWF, mgd	1.20	1.68
PDF, mgd	2.77	3.88
PIF, mgd	3.96	5.56

1.5.8 Load Projections

The future annual average influent load projections are also based on unit loads, which are summarized in Table 1.11. Similar to the flow estimates, the estimated per capita TSS and BOD_5 loads are based on the average value from 2012 to 2017. The average annual loads are expected to grow at the same rate as population. Therefore, the projected average annual loads are projected by multiplying the anticipated population served by the WWTP by the estimated per capita load. The maximum month loads are then estimated by applying appropriate peaking factor. The resulting load projections are presented in Table 1.12.

Table 1.11 Per Capita Loading

Year	Population	BOD5, ppcd	TSS, ppcd	Ammonia, ppcd
2010	7,269	0.160	0.126	0.0287
2011	7,318	0.157	0.164	0.0224
2012	7,366	0.189	0.194	0.0300
2013	7,415	0.194	0.204	0.0307
2014	7,464	0.195	0.232	0.0314
2015	7,513	0.187	0.199	0.0294
2016	7,561	0.189	0.236	0.0440
Selected		0.190	0.223	0.0349

Table 1.12 Load Projections

Parameter	Current	2035
Population	7,610	10,461
BOD_5		
Average Annual Load, ppd	1,430	1,990
Maximum Month Load, ppd	2,180	3,030
TSS		
Average Annual Load, ppd	1,670	2,330
Maximum Month Load, ppd	2,540	3,510
Ammonia		
Average Annual Load, ppd	264	366
Maximum Month Load, ppd	450	637

Establishing core design criteria and reliability/redundancy requirements provides a method to calculate current unit process capacity and a sound rationale for selecting design alternatives. A summary of these criteria is presented in Table 1.13 and are based on typical criteria approved by DEQ and Carollo's experience with other utilities.

Table 1.13 Core Design and Reliability/Redundancy Criteria

Unit Process	Design Parameter	Recommended Design Criteria	Reliability/Redundancy Criteria
Influent Pump Station	PIF	• Required Year 2035 Capacity = 5.56 mgd • Pump current low flows without cycling	• Capacity for PIF with largest unit out of service
Screening	PIF	• Required Year 2035 Capacity = 5.56 mgd • Screen Opening = 3/8"	• Minimum 2 screens • Ability pass PIF with one unit out of service and a passive bypass channel
Grit Removal	PIF	• Required Year 2035 Capacity = 5.56 mgd	• Single unit to pass PIF with passive bypass
Aeration	MMWW	• Aerobic SRT = 12 days	• All units in service under MMWW conditions
Aeration Blowers	Peak Day Load	• Design DO = 2.0 mg/L	• One blower out of service during peak day load • Maintain DO at 2.0 mg/L
Secondary Clarifiers	PIF	• Required Year 2035 Capacity = 5.56 mgd • SOR = 1500 gal/SF/day	• Largest clarifier out of service for ADW conditions
Tertiary Treatment	MMF (summer)	• Required Year 2035 Capacity = 1.22 mgd	• All units in service
Disinfection	PIF	• Dose 20 - 25 mJ/cm2 with one log safety	• Capacity for PIF with one bank out of service

Table 1.13 Core Design and Reliability/Redundancy Criteria (Continued)

Unit Process	Design Parameter	Recommended Design Criteria	Reliability/Redundancy Criteria
Effluent Pump Station	PIF	• Required Year 2035 Capacity = 5.56 mgd • Pump current low flows without cycling	• Capacity for PIF with largest unit out of service
Outfall	PIF	• Convey up to PIF within acceptable velocity range of 2 – 10 fps	• All units in service
Digesters	MM SRT	• 15 days @20deg C	• All units in service
Dewatering	MM Solids hydraulic Loading	• Solids loading rate of 350 lb/hr	• 8 hours/day operation; 5 days a week • One full back up unit OR 1 month sludge storage

Appendix A

CURRENT NPDES PERMIT

Chapter 2

FLOW MONITORING DATA REVIEW AND FLOW PROJECTIONS

2.1 Introduction

This Chapter reviews the existing wastewater flows and presents the projected wastewater flows for future conditions within the City of Scappoose's (City) collection system. Existing flows are based on data collected by the City's recent flow monitoring program. Future flows were developed for planning year 2035, and build condition, estimated for planning year 2067.

2.2 Study Area

The study Area is served by the City owned and operated sewer utility, conveying flows from the south and west portions of the city to the wastewater treatment plant. The City's study area is illustrated in Figure 2.1. The Study Area corresponds to the City's Urban Growth Boundary (UGB), which is planned to become build-out by 2067. Wastewater flows in this chapter will be projected for build-out condition and estimated backwards to simulate the duration of this Plan (i.e. planning year 2035).

2.3 Sewer Collection System Flows

Sewer collection systems are intended to convey sanitary flows, but frequently have additional flows from other sources. The different flow components are described in the section below and illustrated in Figure 2.2.

2.3.1 Dry Weather Flow Components

There are two primary components of Dry Weather Flow (DWF).

2.3.1.1 Base Wastewater Flow

The Base Wastewater Flow (BWF) is the sanitary flow generated by routine water usage of the City's residential, commercial, and industrial customers. Conveying this flow is the primary function of the collection system. The flow has a diurnal pattern that varies by customer. Typically, a residential diurnal pattern has two peaks with the more pronounced peak following the wake-up hours of the day, and a less pronounced peak occurring in the evening. Commercial and industrial patterns, though they vary depending on the type of use, typically have more consistent higher flow patterns during business hours, and lower flows at night. Furthermore, the diurnal flow pattern of a weekend may vary from the diurnal flow experienced during a weekday.

-This Page Intentionally Left Blank-

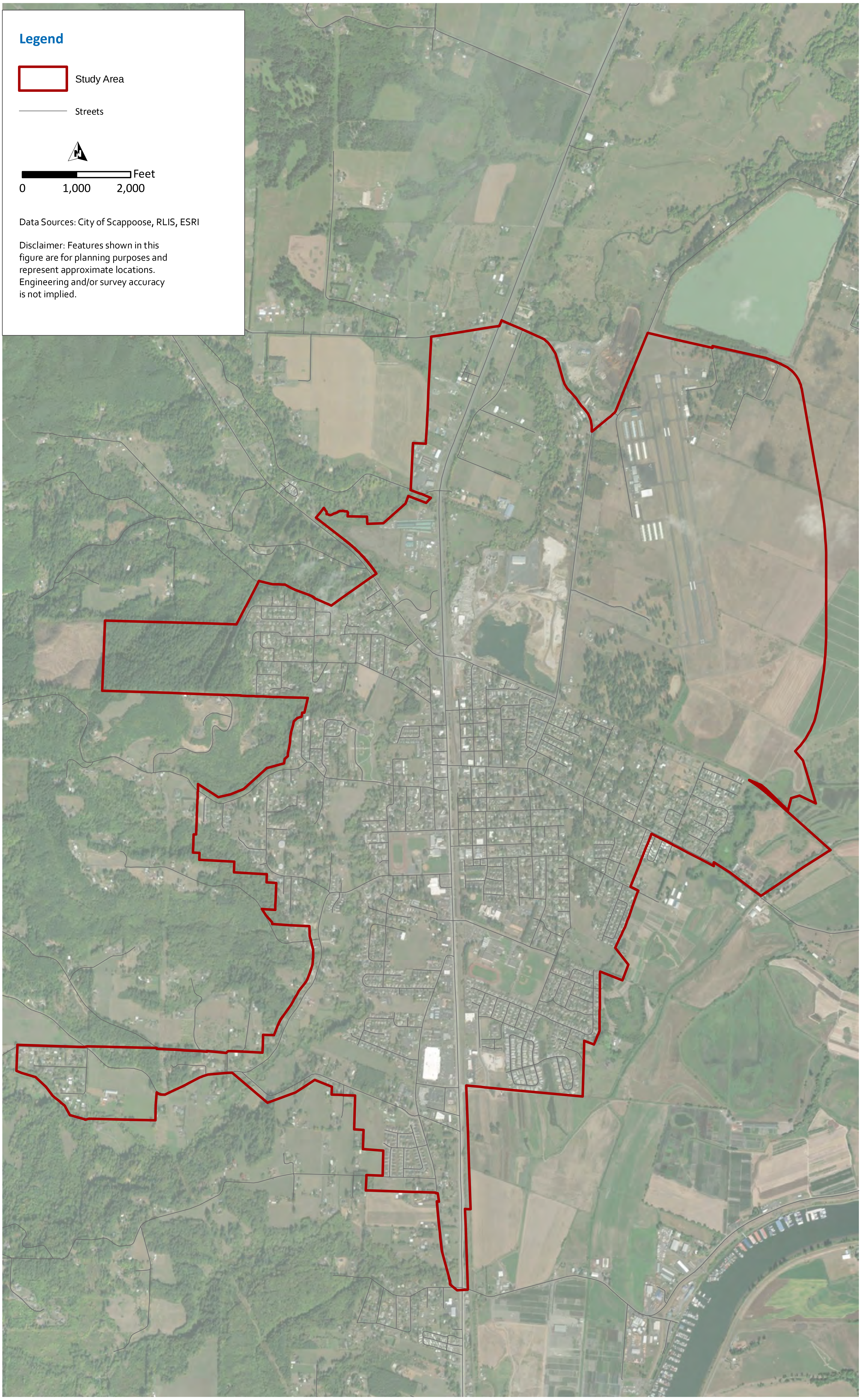


Figure 2.1 Study Area

-This Page Intentionally Left Blank-

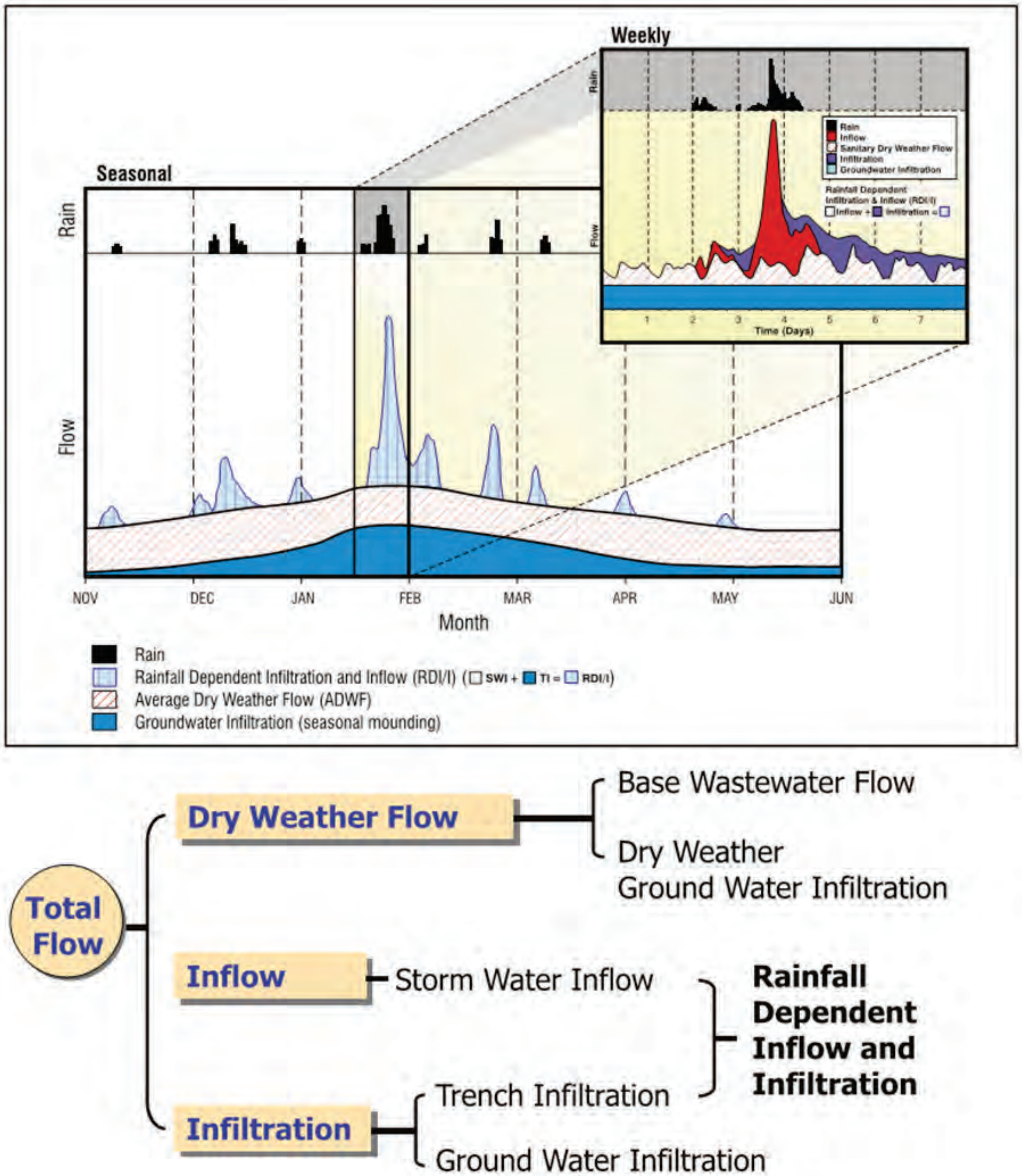


Figure 2.2 Typical Wastewater Flow Components

(Note: This figure is not based on flow data specific to the City or this Master Plan)

2.3.1.2 Dry Weather Groundwater Infiltration

Dry weather Groundwater Infiltration (GWI) will enter the sewer system when the relative depth of the groundwater table is higher than the depth of the pipeline and the sanitary sewer pipe allows infiltration through defects such as cracks, misaligned joints, and broken pipelines. Dry weather GWI (or base infiltration) cannot easily be separated from BWF by flow measurement techniques. Therefore dry weather GWI is typically grouped with BWF.

2.3.1.3 Average Dry Weather Flow

Average Dry Weather Flow (ADWF) is the average flow that occurs on a daily basis during the dry weather season. The ADWF serves as the baseline flow in the hydraulic model. Diurnal (24-hour) patterns are applied to ADWFs, and cumulatively make up the collection system flows.

2.3.2 Wet Weather Flow

Wet Weather Flow (WWF) includes two components:

1. Inflow and Infiltration (I/I): The stormwater I/I response in the sewer system to rainfall is seen immediately (inflow) or within hours after the storm (infiltration).
2. GWI: Wet weather GWI is not specific to a single rainfall event, but rather to the effects on the sewer system over the entire wet weather season. The depth of the groundwater table rising above the pipe invert elevation causes GWI. Sewer pipes within close proximity to a body of water can be greatly influenced by groundwater effects.

2.3.2.1 Inflow and Infiltration

Inflow is stormwater that enters the sewer system via a direct connection to the system, such as roof drain and downspout connections, leaky manhole covers, and inappropriately connected storm drains. Infiltration is stormwater that enters the sewer system by percolating through the soil and then through defects in pipelines, manholes, and joints. Some of the most common sources of I/I are shown in Figure 2.3. The adverse effects of I/I entering the sewer system are that it increases both the flow volume and peak flows such that the sewer system could be operating at or above its capacity, as illustrated in Figure 2.4. If too much I/I enter the sewer system, sanitary sewer overflows (SSOs) could occur.

2.3.2.2 Groundwater Infiltration

GWI, one of the components of I/I, is associated with extraneous water entering the sewer system through defects in pipes and manholes while the ground is saturated during the wet weather season. GWI is related to the condition of the sewer pipes, manholes, and groundwater levels. GWI may occur throughout the year.

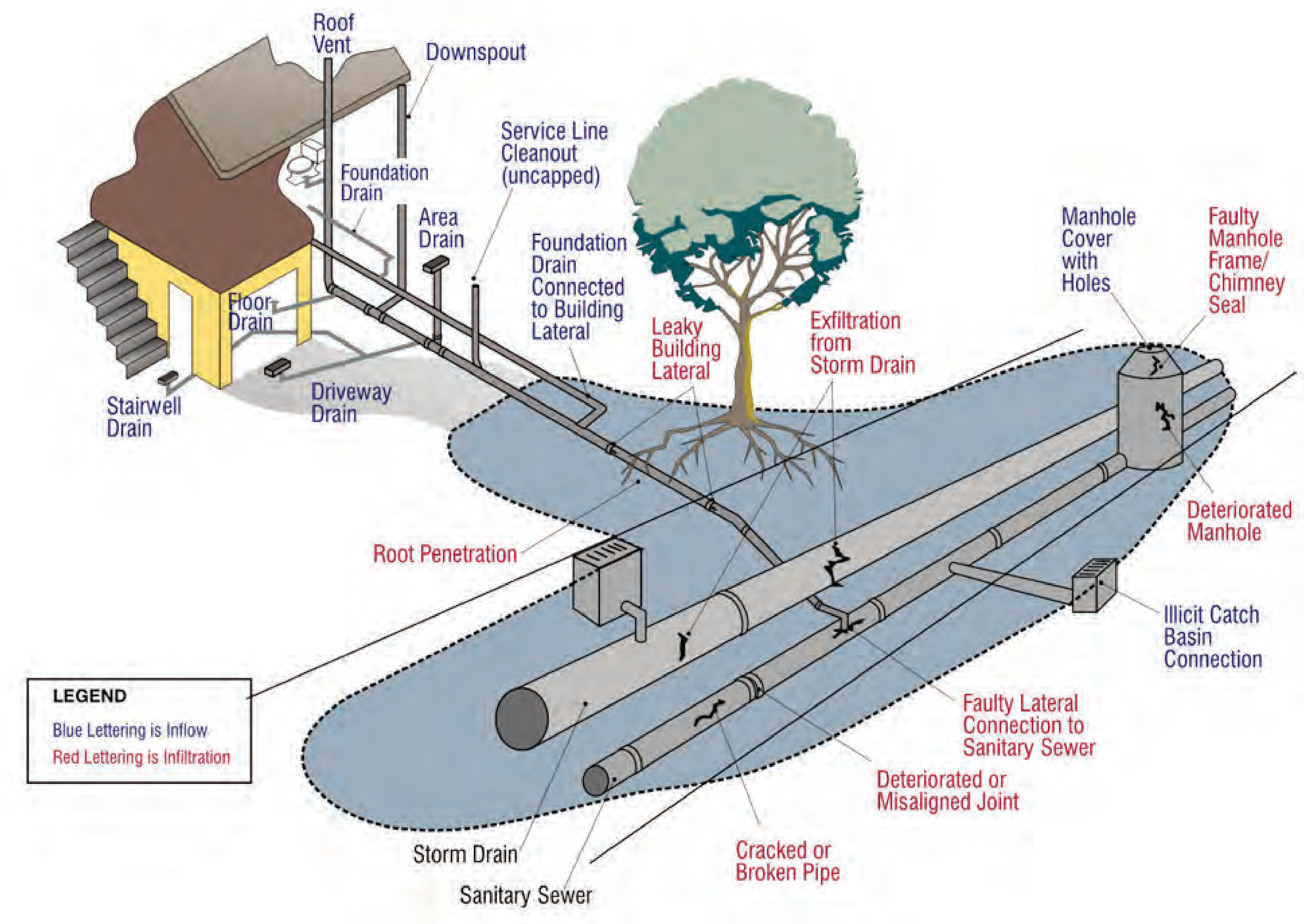


Figure 2.3 Typical Sources of Inflow and Infiltration

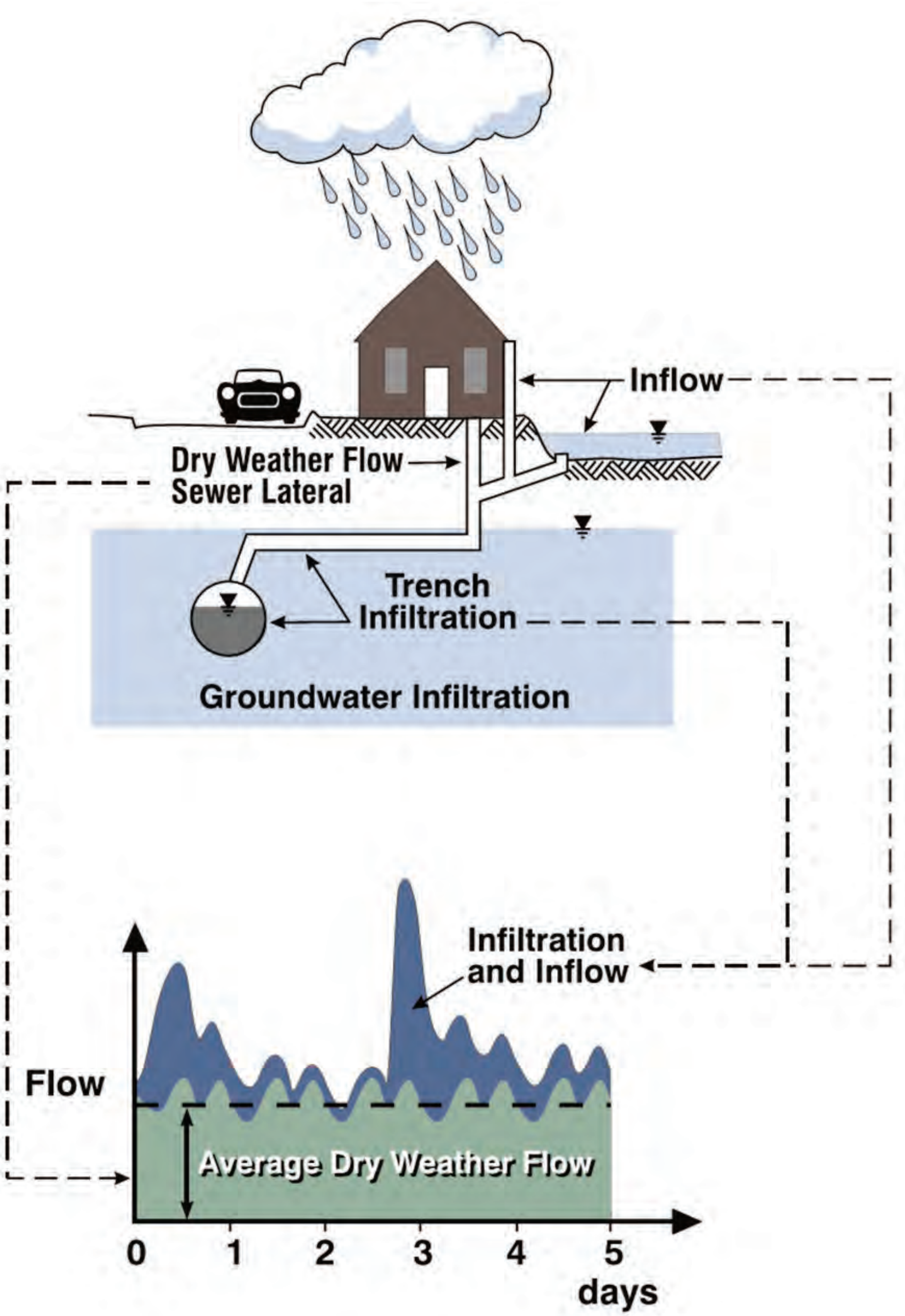


Figure 2.4 Typical Effects of Inflow and Infiltration

2.4 Data Overview for Flow Projections

A number of data components are needed to develop flow projections for the collection system area. Data needs are listed below and detailed in the following sections:

- Flow Monitoring Data.
- Current Zoning Data and Developed Area.
- Existing ADWF and Wastewater Flow Coefficients.
- Future Land Use Data.

2.5 Flow Monitoring Data Quality Review

Carollo reviewed the flow monitoring data collected in the previous Master Plan. The flow monitoring data was collected for one month between January 14, 2016 and February 11, 2016. Below are summary results of the analysis performed. Figure 2.5 shows the locations of the flow monitoring locations for reference.

Below is some background of what the data analysis entailed. Several metrics are used to review the flow monitoring data and are described below:

- **Flow vs Time with Rainfall** – Hydrograph plots show flow vs time data and help as a first good check of the data. We want to make sure the flow data looks good and that there are no anomalies, and we also want to look at the flow and rainfall correlation, make sure we have good system response to calibrate to.
- **Scattergraphs (velocity vs level)** - Scattergraph plots are velocity versus depth data points that are instructive in identifying hydraulic anomalies such as backwater conditions (i.e., signs of downstream or upstream obstructions or restrictions). The resulting patterns form characteristics signatures that reveal important information about conditions within a sewer and the impact that these conditions have on sewer capacity. The scatter plots are compared with the Manning's flow curve for the monitored sewer. The Manning's equation is an empirical formula that calculates flow based on depth and velocity of open channel flows.
- **Rainfall** - The rainfall data presented in previous reports was daily average data of unknown source. Daily rainfall data has limited value in the calibration of hydrologic/hydraulic sewer model, and, therefore, hourly data is recommended for this analysis. Data from USGS was used for this analysis from the following source: (https://or.water.usgs.gov/precip/raingage_info/clickmap.html). Rain gage 3 seems to be the closest available rain gage from the City.
  The rainfall data is compared against the recorded flow data to identify system flow response from the collection system. Wet weather calibration standards require significant flow response for at least three separate storm events, therefore, flow response to events will also be looked at in the data to determine whether response is sufficient.

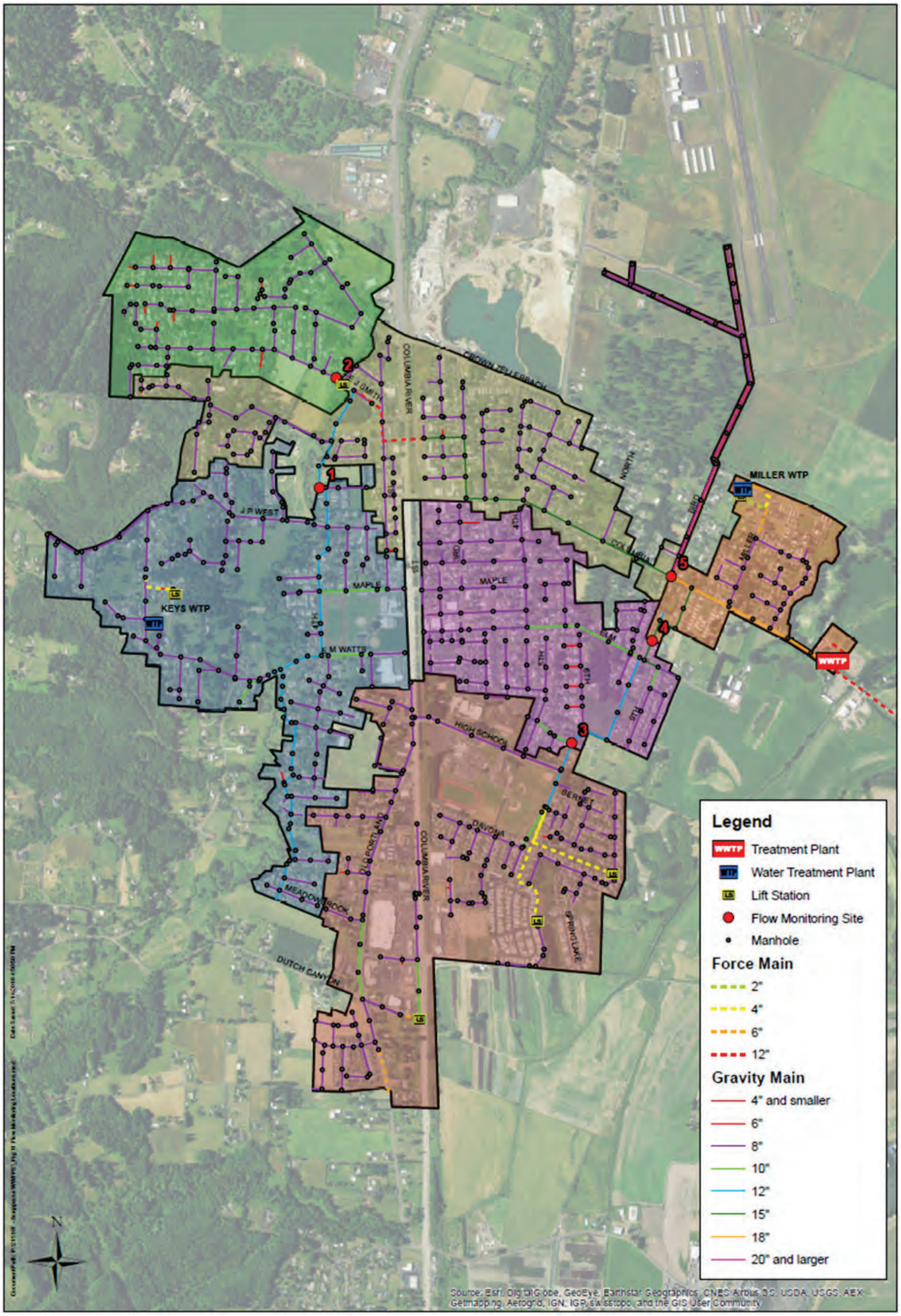


Figure 2.5 Flow Monitoring Locations

2.5.1 West side of the Highway

The following sections discuss the results of the flow monitoring data analysis performed for the flow meters located to the west of the Highway, upstream of Smith Rd Lift Station (LS).

2.5.1.1 Flow Meter 1

The following observations and conclusion were made from the data analysis for Meter 1. Figure 2.6 shows both scattergraph (top) and flow (bottom) graphs for this meter.

The data show surcharging occurring along the sewer main where Meter 1 is located, as shown circled in red below. City staff noted that they experienced a blockage in that line potentially during the course of the flow monitoring program. The scattergraph signature confirms a potential blockage for at least half of the flow monitoring duration.

The data show very limited response to rainfall events in the first half of the metering period. The second half of the data shows sharp spikes (approximately once a day) with no correlation to rainfall. The City noted that these daily peaks are caused by the backwash events at Keys Water Treatment Plant (WTP) (i.e. well filter backwash, surface water backwash).

In conclusion, it was determined that the flow response to rainfall was limited during the metering period and may not be sufficient for wet weather calibration. It is recommended that the meter captures more significant events to have more confidence in the hydraulic model when running the design storm.

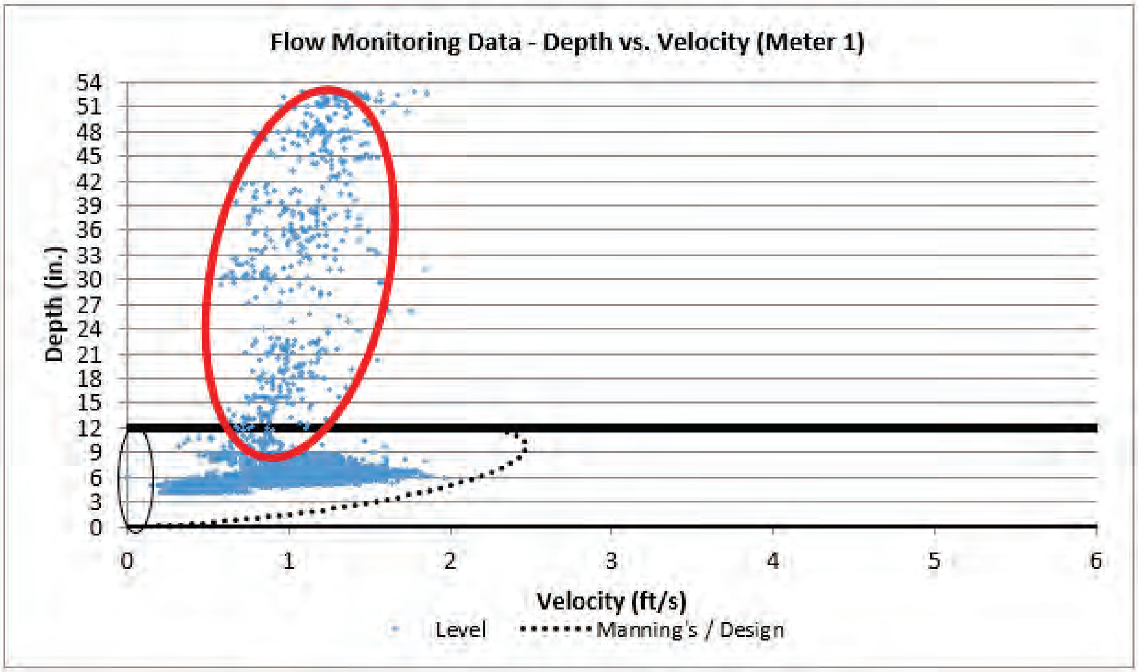
2.5.1.2 Flow Meter 2

The following observations and conclusion were made from the data analysis for Meter 2. Figure 2.7 shows both scattergraph (bottom) and flow (top) graphs for this meter.

The flow data, illustrated at the top of Figure 2.7, show a large drop in flow over the course of the flow monitoring period from approximately 140 gpm in the week of January 11, 2016 down to approximately 30 gpm in the week of February 10, 2016. Additionally, the recorded velocity data get extremely spiky during the second half of the flow monitoring period, which usually is a sign of a malfunctioning sensor.

The scattergraph shows velocities drifting over a wide range of values without a corresponding change in depth, as circled in red in Figure 2.7. This is usually a sign confirming that the velocity sensor might have been malfunctioning.

In conclusion, the data for Meter 2 appear unsuitable for calibration and it is recommended that it is not used.



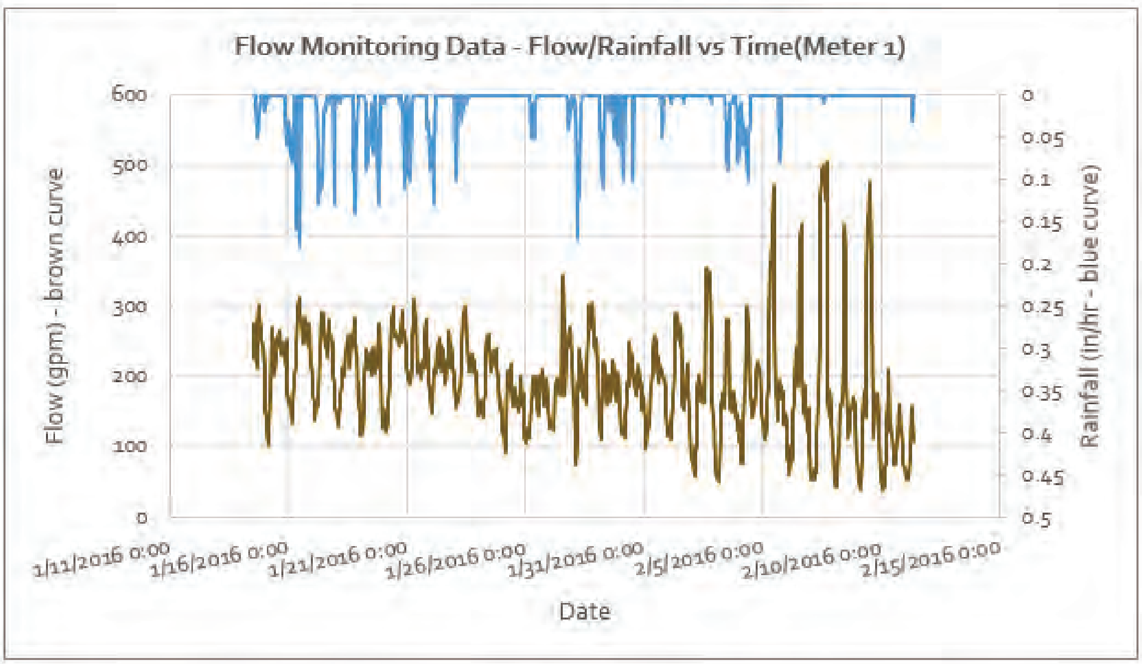


Figure 2.6 Flow Meter 1 Data Analysis

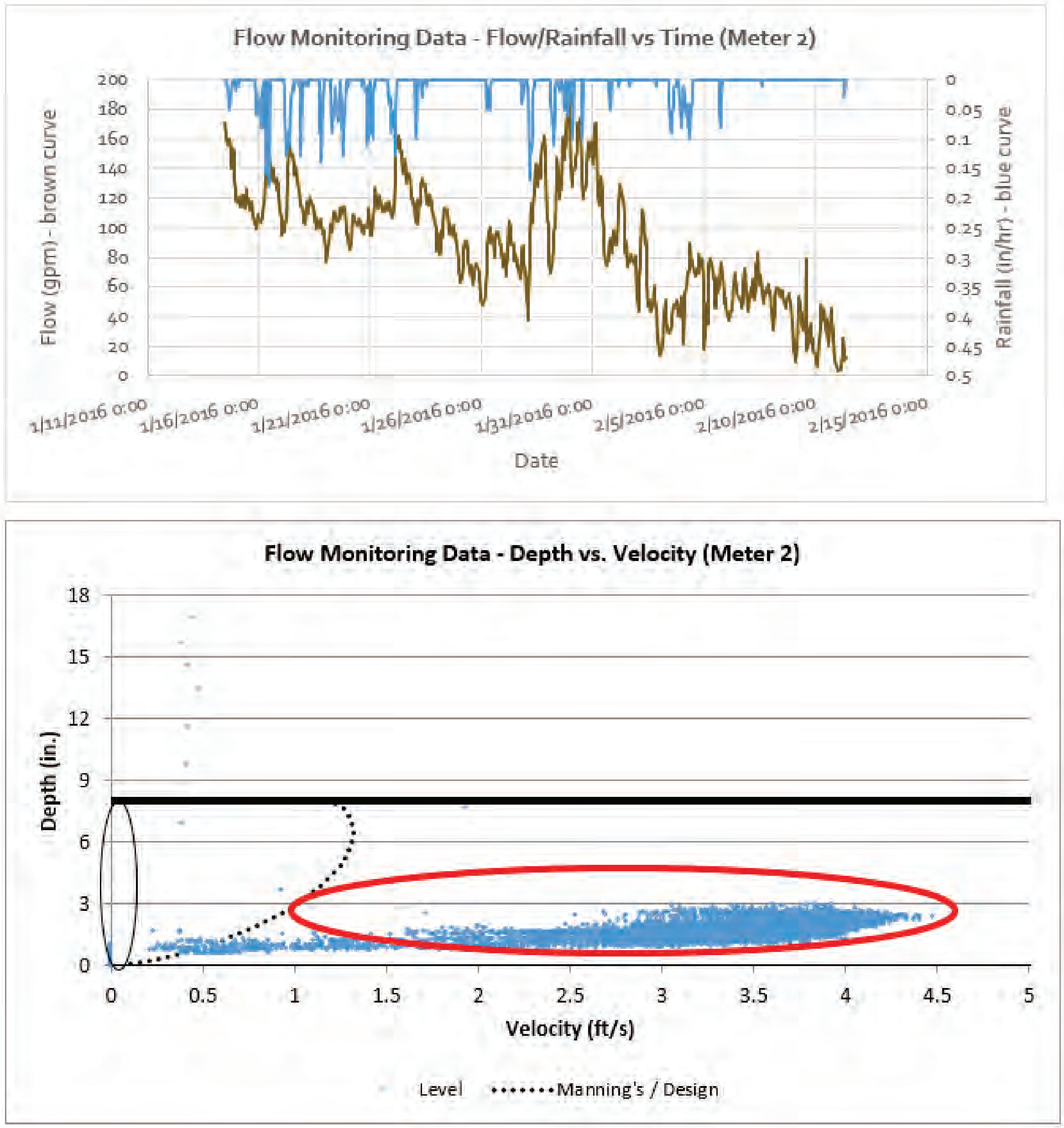


Figure 2.7 Flow Meter 2 Data Analysis

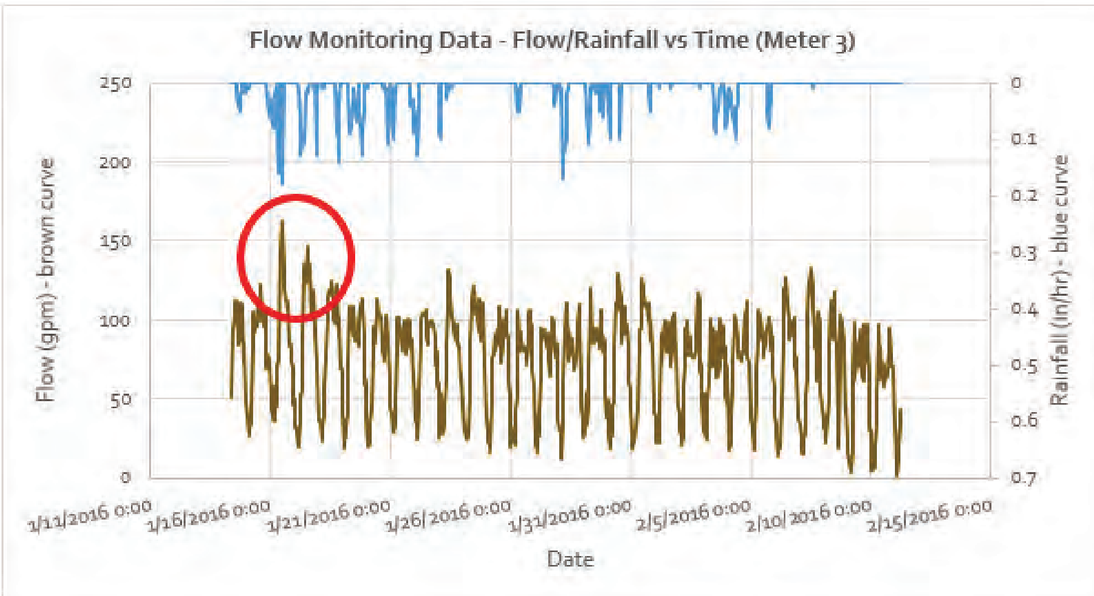
2.5.2 East side of the Highway

2.5.2.1 Flow Meters 3 and 4

The following observations were made from the data analysis for both Meters 3 and 4. Figure 2.8 shows the flow data graph for Meter 3 at the top and flow data for Meter 4 at the bottom.

The data collected for Meter 3 and Meter 4 appear suitable; both scattergraphs and flow data graphs seem reasonable. However, both flow data graphs show very limited response to rainfall, especially during the second half of the flow monitoring period. One storm (January 16, 2016) shows a slight response, as circled in red below, and might be able to be used for calibration.

The recommended criteria for wet weather calibration is that the model results be compared and calibrated to three significant rainfall events. The flow data for both Meters 3 and 4 do not show sufficient response to meet these calibration criteria and it was concluded that this data may not be sufficient for calibration.



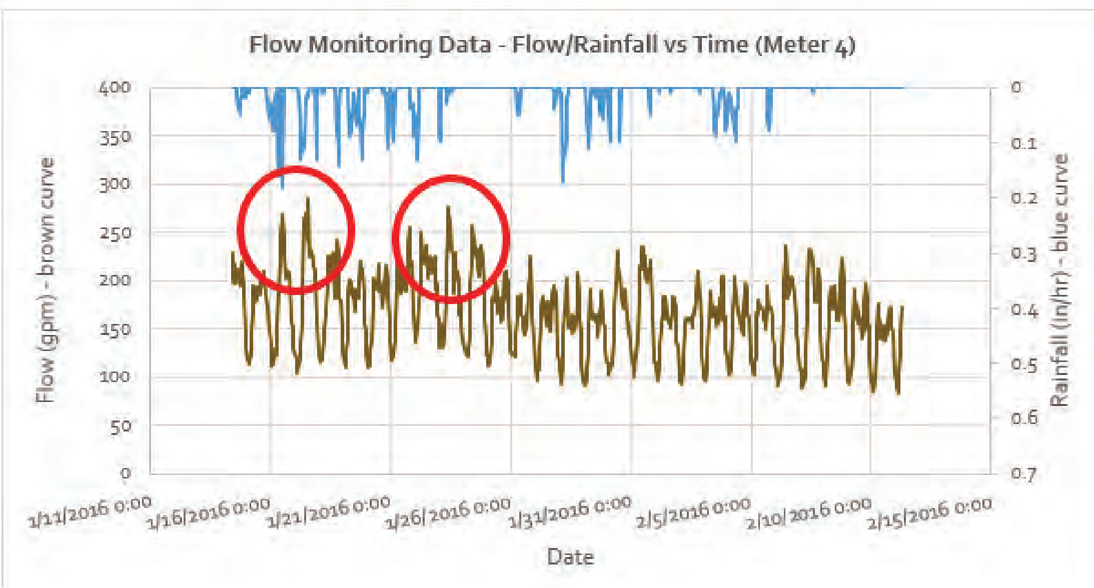


Figure 2.8 Flow Meters 3 and 4 Data Analysis

Flow Meter 5

The following observations were made from the data analysis for Meter 5. Figure 2.9 shows both scattergraph (bottom) and flow (top) graphs for this meter

After reviewing the flow vs time graph (top), the first half of the data might be suitable for model calibration an shows some response to the January 26, 2016 rainfall event. The second half of the data shows sharp spikes (approximately once a day) with no correlation to rainfall. Similar to Meter 2, it is suspected that these daily peaks are due to backwash at the Keys WTP.

Per the recommended calibration standards, it was concluded from this analysis that the data for Meter 5 may not be sufficient for wet weather calibration.

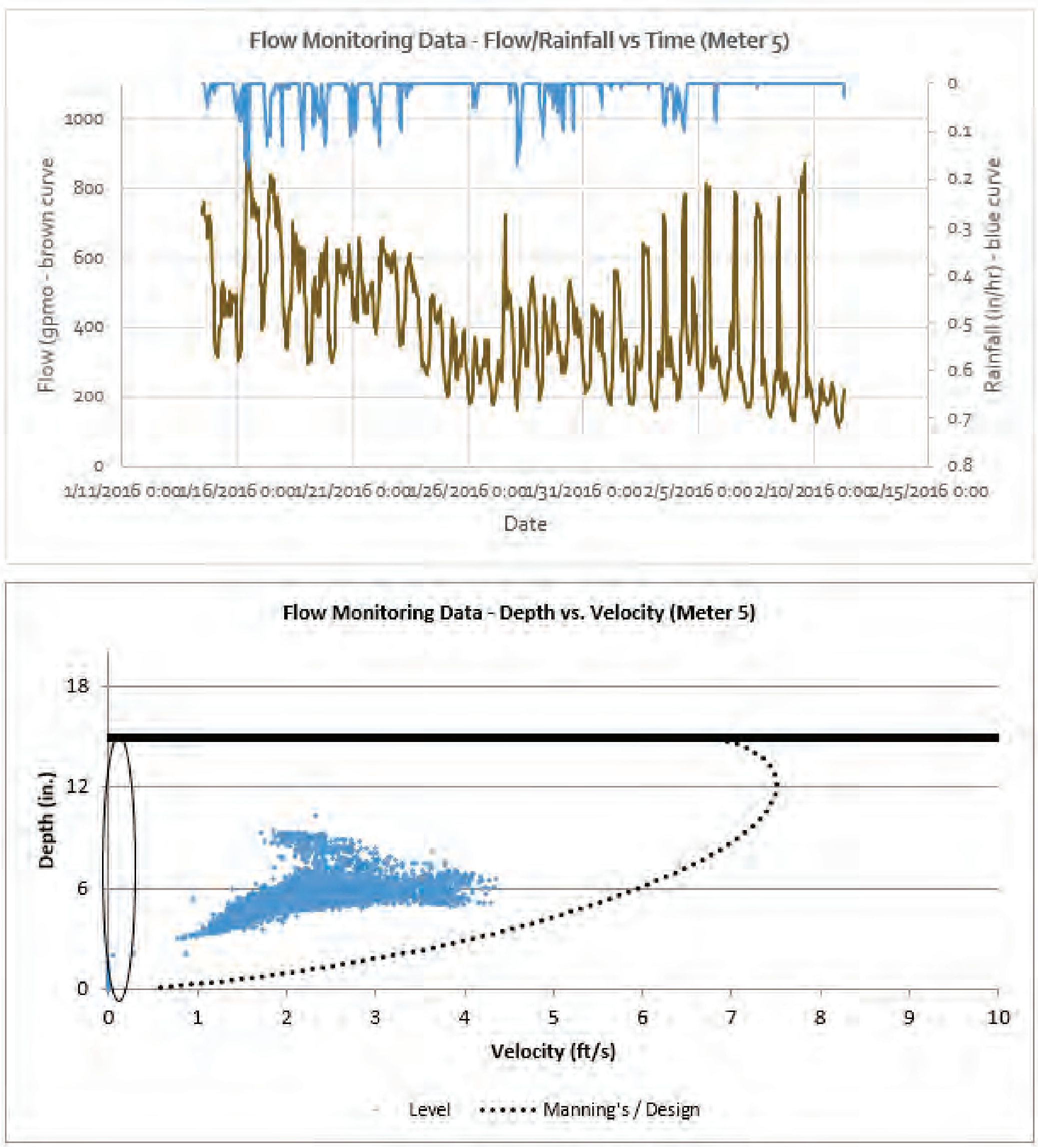


Figure 2.9 Flow Meter 5 Data Analysis

2.5.3 Flow Monitoring Data Review Summary and Recommendations:

As detailed in the sections above, the suitability of each of the five flow meters was evaluated based on both hydrographs and flow response to rainfall, and scattergraphs. Table 2.1 summarizes the results and explains the different data quality types. The data at each meter location were classified into three levels of data quality:

- Suitable – most data is generally good;
- Partially suitable – some data is good;
- Unsuitable – data will not work for model calibration, see comments.

Table 2.1 Meter Data Review Results Summary

Meter ID	Suitability	Comments
Meter 1	Unsuitable	Sign of overflow caused by blockage in the sewer line Very limited flow response to rainfall during the metering period.
Meter 2	Unsuitable	Potential velocity sensor drifting or potential influence from Smith Rd LS
Meter 3	Partially	Scattergraph looks good. Very limited flow response to rainfall, potentially not sufficient for calibration.
Meter 4	Partially	Scattergraph looks good. Very limited flow response to rainfall, potentially not sufficient for calibration.
Meter 5	Partially	Scattergraph looks reasonable. Only one rainfall events showed system flow response, when at least three events are recommended for wet weather calibration.

Overall, more than half of the flow monitoring data collected do not appear suitable or sufficient and are not recommended for use in hydraulic model calibration.

- The two meters (Meter 1 and Meter 2) located west of the highway (upstream of Smith Rd LS) did not record good quality data. Available data at the Smith Rd LS, if hourly run time data is available, could be used to help with calibration. However, the sewer line along 4th Street seems to be surcharging, potentially experiences overflows, and might be capacity restricted. Recording depth data in the 4th Street sewer is recommending for accurate calibration. Evaluation criteria are usually based on surcharging and water depth in the pipe, therefore, calibration not only to flow but also levels is primordial for model accuracy and trust.
- On the east side of the highway, the three meters show relatively good data, however, only one "main" storm was recorded and did not translate to a significant system flow response. These data were concluded as not sufficient for wet weather calibration.
- The treatment plant data might be used to calibrate the total flow coming from the system during rainfall events. Hourly data during at least three major rainfall events would be needed.

After reviewing the data, it was concluded that there is not enough suitable data and significant flow response to rainfall to perform a good and accurate model calibration. It is recommended that the City deploy at least three flow monitors during the next rainy season to allow for wet weather model calibration. Level information is critical during model calibration as this is the metric triggering capacity deficiencies and ultimately the timing and extent of recommended improvements.

2.6 Zoning and Land Use Data

Land use designations and regulations provide important information in evaluating existing sewer system capacity. Land use determines the area available for various types of development including Low, Medium, High Density Residential development, as well as commercial and other types of land use that provide the economic base necessary to support residential development.

Land use information is an integral component in estimating the amount of wastewater generated within any City. The type of land use in an area will affect the volume of the wastewater generated. Additionally, the service area is typically comprised of both sewered and unsewered areas: sewered areas contribute flow to the collection system, while unsewered areas are vacant or undeveloped land and do not currently contribute flow to the collection system. The following section describes the land use assumptions for the existing sewer service area and Study Area.

2.6.1 Zoning and Existing Served and Connected Areas

The existing zoning within the study area is shown in Figure 2.10. Of the City's Study Area of 2,850 acres, approximately 1,100 acres (39 percent) is sewered (i.e., contribute flows to the sewer system), 1,390 acres are unsewered (vacant, undeveloped, etc.). Right-of-ways (ROW) were removed from this analysis, as streets and ROW are assumed to contribute no sanitary flows to the collection system and accounted for approximately 360 acres. Table 2.2 provides a summary of the land use zoning categories and acreages for the existing service area.

Table 2.2 Existing Zoning Acreage

Zone Description	Acreage (acres)
Expanded Commercial	127.2
General Commercial	43.8
High Density Residential	28.6
Light Industrial	23.0
Low Density Residential	289.7
Manufactured Housing Residential	71.8
Moderate Density Residential	294.6
Public Lands-Institutional	2.7
Public Lands-Recreation	7.3
Public Lands-Utility	8.4
Public Use Airport	200.8
Right-of-Way (ROW)	359.9
Vacant	1,387.0
Total Acreage	**2,844.8**

Of the total service area (sewered and unsewered), the largest land use category is residential, which accounts for approximately 660 acres or 23 percent of total acreage. Commercial, business, and office space make up approximately 171 acres, or 6 percent of the total.

-This Page Intentionally Left Blank-

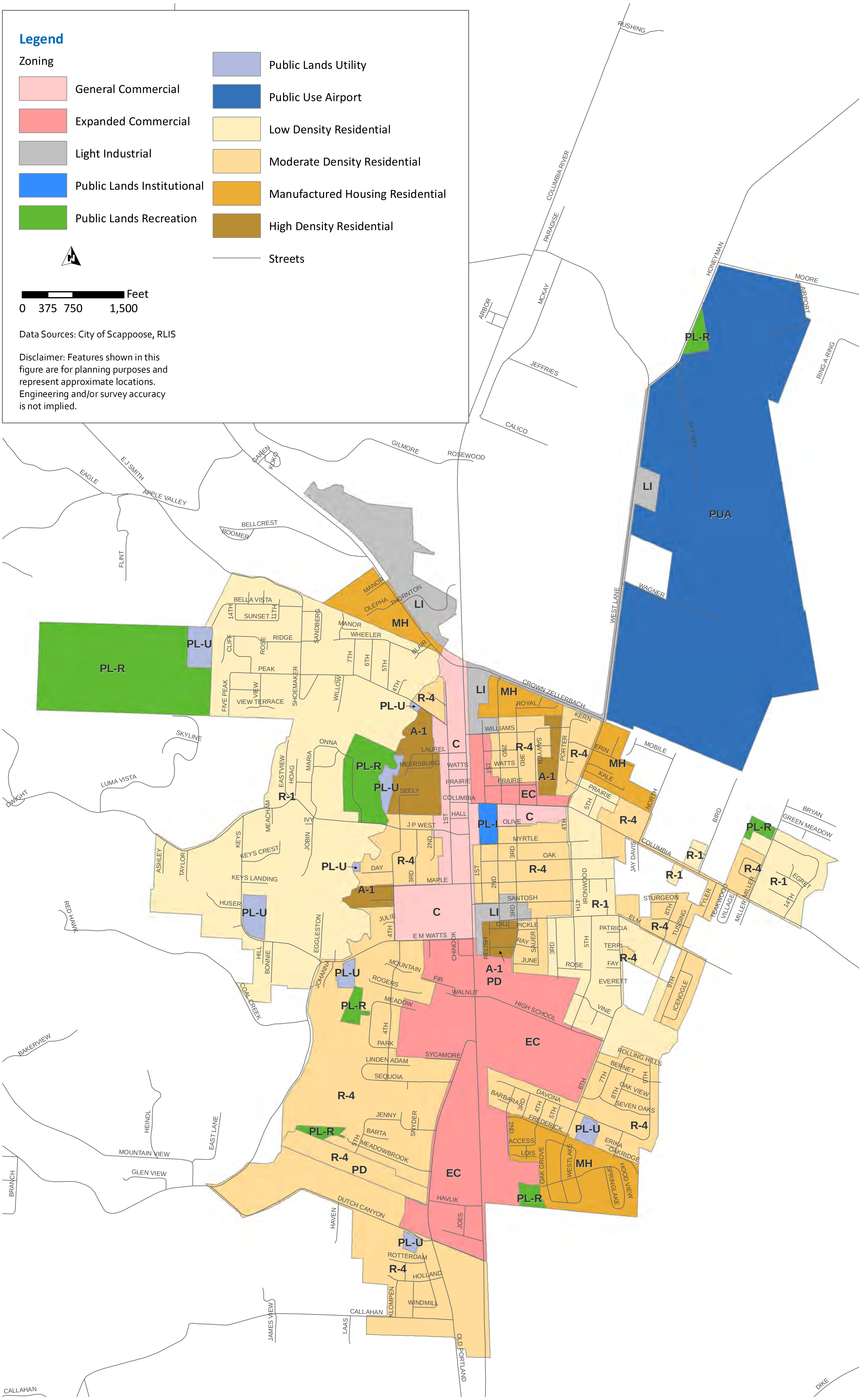


Figure 2.10 Existing Zoning

-This Page Intentionally Left Blank-

2.6.2 Future Land Use

The future land use is presented in Figure 2.11 and corresponds to the adopted Comprehensive Land Use Plan. The comprehensive plan has consolidated the 11 zoning categories to 7 land use designations. The future service area includes build-out of the entire Study Area, shown in Figure 2.1. Therefore, the future sewered service area includes existing sewered service area and infill of existing unsewered areas. Table 2.3 provides land use summary for the whole Study Area, including the current development. Approximately 1,330 acres will be added to the currently developed area to make the future build-out service area. Streets and ROW are removed from this analysis and are assumed to contribute zero sanitary sewer flow. It is also assumed that for the future Airport Employment area, approximately 20 percent of the area is assumed to be ROW and taxiway access.

Table 2.3 Future Land Use Acreage

Comprehensive Plan Designation	Total Acreage (acres)
Airport Employment	517.5
Runway Protection Zone	74.6
Commercial	179.3
General Residential	175.6
Industrial	267.2
Manufactured Home	85.8
Public Lands	262.1
Suburban Residential	868.0
Right-of-Way (ROW)	414.8
Total Acreage	**2,844.8**

2.6.3 Zoning and Land Use Correspondence

As presented above, future land use designations are different from the existing zoning designations. A relationship between existing and future designations was developed. Table 2.4 summarizes the correlation between zoning and land use, and will be used to develop wastewater flow coefficients (see Section 7.0).

Table 2.4 Zoning vs. Land Use Correspondence

Zoning Description	Land Use Classification
Expanded Commercial	Commercial
General Commercial	Commercial
High Density Residential	General Residential
Light Industrial	Industrial
Low Density Residential	Suburban Residential
Manufactured Housing Residential	Manufactured Home
Moderate Density Residential	General Residential
Public Lands-Institutional	Public Lands

Table 2.4 Zoning vs. Land Use Correspondence (Continued)

Zoning Description	Land Use Classification
Public Lands-Recreation	Public Lands
Public Lands-Utility	Public Lands
Public Use Airport	Airport Employment

2.7 Wastewater Flow Coefficients

In order to develop wastewater flow projections and allocate future flows to the collection system, relationships between land use and wastewater generation were developed. These relationships, called wastewater flow factors are established based on the average wastewater flow generated for each existing land use type. The land use flow factors were established to project the estimated ADWF through future development of the City's wastewater collection system and project future flows within the Study Area boundary.

Average wastewater flow coefficients are rates, usually expressed in gallons per day per acre (gpda), applied to either gross or net acres for calculating average day flow generated from a particular land use. A flow coefficient was developed for each of the land use classifications discussed previously. The flow coefficient provides a means to transform a land use category from acreage into wastewater flow. The resulting flow is then inputted into the appropriate sewer area in the sewer system model. Wastewater flow coefficients for residential areas can range between 300 to 3,000 gpda, and commercial and industrial areas might range from 900 to 4,000 gpda, with typical values averaging approximately 1,000 gpda. Land uses designated as open space and parks are assumed to generate negligible amounts of sewage flow, and as a result have a flow coefficient of zero.

The coefficients are developed using the following procedure:

- Average flows for the system is extracted from the wastewater treatment plant flows.
- Using GIS, the acres for each land use type are calculated for the collection system.
- Preliminary coefficients for each land use type are assumed based on typical values, which can be estimated from the approximate number of dwelling units per acre, the assumed per capita wastewater generation rates, and/or the typical number of people per dwelling unit for each land use type.
- The coefficients are adjusted up or down (balanced) within a reasonable range (based on engineering judgment) until the calculated average flows for the system matched what was measured during the flow monitoring period. If the flow coefficients produced average flows that were significantly different from the field measured flows, further investigation was conducted to verify that the tributary basins were delineated correctly and if the collection system configuration was correct.

Table 2.5 presents the wastewater flow coefficients and land use areas that represent the existing ADWF. The land use coefficients generate an ADWF for the collection system of 0.66 mgd, a 0.3 percent different from the historical wastewater treatment plant. As with most cities, residential land use makes up the majority of developed land and wastewater flow. For Scappoose, residential customers make up approximately 67.8 percent of the current flow.

Table 2.5 Land Use Flow Factors Development

Land Use Designation	Total Acreage	Flow Factor (gpad)	Flow (mgd)
Airport Employment	200.8	90	0.018
Commercial	170.9	950	0.162
General Residential	323.2	810	0.262
Industrial	23.0	1,200	0.028
Manufactured Home	71.8	780	0.056
Public Lands	18.3	300	0.006
Suburban Residential	289.7	450	0.130
Total	**1,097.8**		**0.662**
	Flow Monitoring ADWF (mgd)		**0.660**
		Percent Difference	**0.26%**

2.8 Collection System Flow Projections

2.8.1 Dry Weather Flows Projections

Flow projections are done in two steps based on the type of customer:

2.8.1.1 Land Use Based Flow Projections

The projected DWFs were developed based on the future land use map (Figure 2.11). Projected flows were generated in a similar manner as the existing ADWF. The flow coefficients developed for the existing land use categories were applied to the future build-out land use (acres) to project the wastewater flow generated from infill and new development. The resulting flows represent the projected inflow in the hydraulic model.

The study area is projected to be build-out by planning year 2067, however, the planning period for the Plan is 2035. Population projections from the Portland State University Population Research Center (PSU PRC) were used to estimate intermediate ADWF for the collection system. An annual population increase of 1.8% every year between 2016 and 2035.

-This Page Intentionally Left Blank-

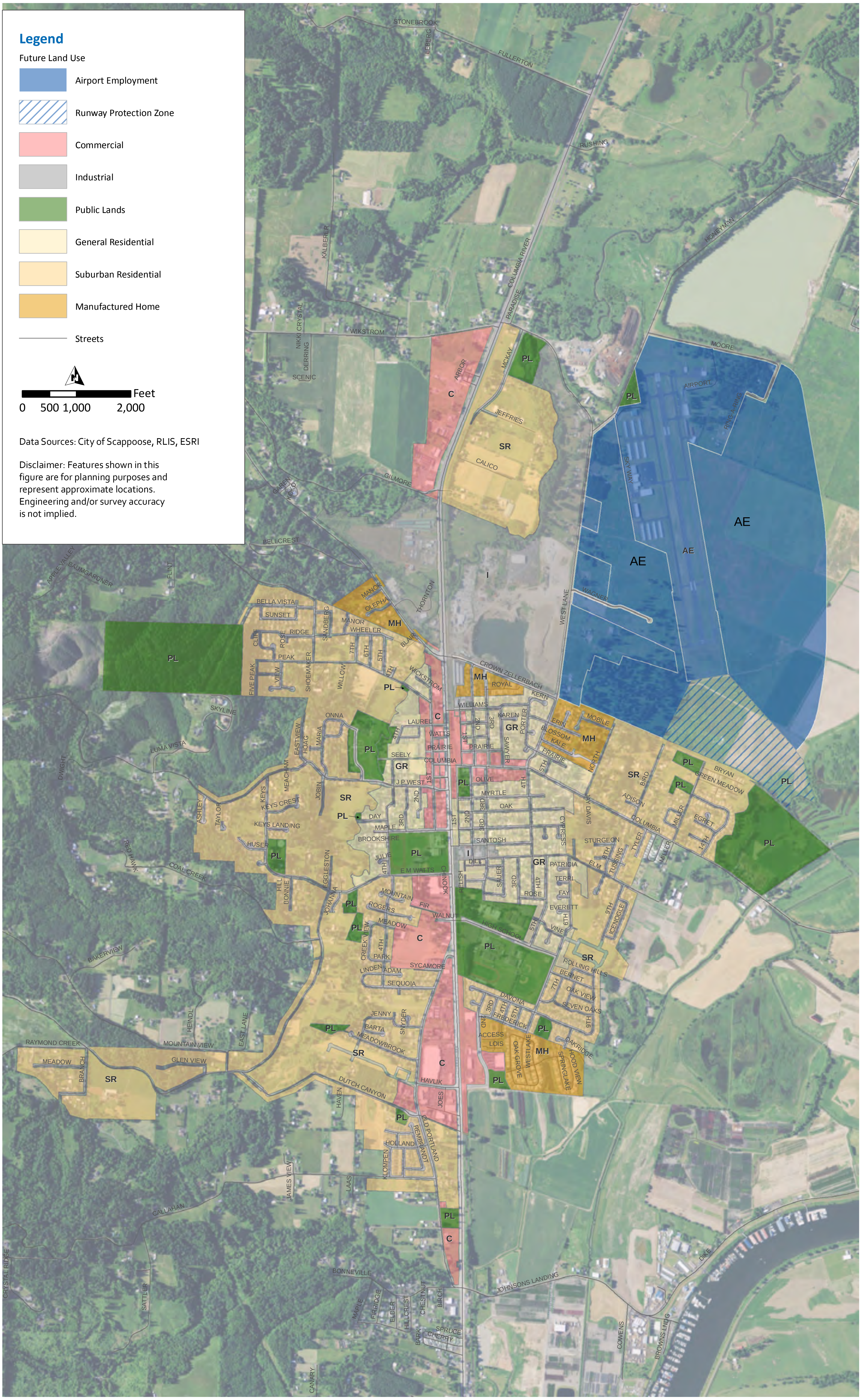


Figure 2.11 Future Land Use

-This Page Intentionally Left Blank-

2.8.1.2 Largest Customer Projections

The largest future customers were identified by the City and ADWFs were projected for each of these customers. A description of the future largest customers can be found in Table 2.6.

Projections for these customers were individually developed based on the type of development. The DOE Orange book was used to estimate future flows for these customers. Table 2.6 summarizes the largest customers and corresponding projected ADWFs. Total ADWF from these customers is estimated at 0.14 mgd.

Table 2.6 Largest Customer Projected ADWF

Customer #	Large Customer	# of student/ employee/...		Flow Unit		ADWF (gpd)	ADWF (gpm)
1.	**Community College**	500	Students (next couple years)	15	gpd/per student	**7,500**	5
		1500	Students (by 2035)	15	gpd/per student	**22,500**	16
2.	**50 acres of Hotels**	200	rooms	150	gpd/room	**30,000**	21
3.	**Metallurgy R&D – 10 acres**	50	employee	30	gpd/per employee	**1,500**	1
4.	**42 acres of light/heavy industrial**			flow factors→ 900 gpad		**37,800**	26
5.	**50 acres of light/heavy industrial**			flow factors→ 900 gpad		**45,000**	31

2.8.2 Wet Weather Flows Projections

Wet weather flows were developed using the following assumptions:

- Future I/I for new development estimated at 1,500 gpd/acre.
- Existing I/I is estimated based on wastewater treatment plant flows. It is recommended that the hydraulic model calibration be checked and run after removing hydraulic restrictions within the system to estimate the maximum potential Peak Wet Weather Flows.
- No degradation of the system over the years is taken into account. It is assumed that the City's repair and replacement program will offset the impact of aging of the system on I/I.

Additional I/I flows for new development is estimated at 1.04 mgd, while total PWWF at the treatment plant is predicted at approximately 5.6 mgd. It is to be noted that this value might increase when the City removes any hydraulic capacity restrictions in the system.

2.8.3 Existing and Projected Flow Summary

Developing an accurate estimate of the future quantity of wastewater generated at build-out of the collection system is an important step in maintaining and sizing sewer system facilities, for both existing conditions and future scenarios. Table 2.7 summarizes flows under existing condition, planning year 2035, and build-out condition for the Study Area (i.e. planning year 2067), which are illustrated in Figure 2.12.

Table 2.7 Projected Flows Summary

Flow Condition	Existing Condition	Year 2035	Year 2067 (build-out)
ADWF (w/o largest customer) (mgd)	0.66	0.90	1.14
Largest Customers ADWF (mgd)	0.00	0.14	0.14
Total ADWF (mgd)	**0.66**	**1.04**	**1.28**
Total PWWF (mgd)	**3.97**	**5.57**	**6.59**

Figure 2.12 ADWF and PWWF Projections

Chapter 3

HYDRAULIC AND PROCESS CAPACITY ANALYSIS

3.1 Introduction

This chapter summarizes the hydraulic and process capacities at the Scappoose Wastewater Treatment Plant (WWTP). The capacity assessment along with condition assessment completed as part of Draft Facilities Plan Update (Keller Associates, December 2016) will be used to provide a baseline for identifying improvements at the plant to address current capacity and operational issues, as well as future improvements required to meet potential changes in regulations and increases in flows and loads due to growth.

3.2 Current Plant Operations

Figure 3.1 summarizes the current operation of the WWTP. Raw sewage enters the plant at the influent pump station. The raw sewage flows through a fine screen with 1/4 inch openings to remove rags and other debris from the flow stream. The plant influent is then pumped up to the operating level by four 15 hp, variable-frequency pumps .

The WWTP has a secondary treatment process consisting of a single aerated lagoon and two secondary clarifiers. The aerated lagoon has a volume of approximately 1.9 million gallons with a side water depth (SWD) of 11 feet. The lagoon is aerated and mixed by two 40 hp high-speed floating surface aerators and two triton 40 hp mixers. The mixed liquor suspended solids (MLSS) exits the lagoon at a weir and is split to the secondary clarifiers.

Each secondary clarifier has a diameter of 50 feet and a SWD of 15 feet. The clarified effluent flows over peripheral weirs along the clarifier walls and flows by gravity to the WWTP's tertiary treatment facility.

Tertiary treatment at the plant consists of disk filtration and ultraviolet (UV) disinfection. Clarifier effluent is pumped to the disk filters via two, 16 hp pumps. Water passes through the disk cloth media, resulting in the separation of particulates from the flow. The filter effluent and any unfiltered secondary effluent flow by gravity to the UV disinfection channel where ultraviolet light is used to disinfect the treated water prior to discharge.

The plant effluent is pumped to the outfall in the Multnomah Channel with four 40 hp vertical turbine pumps.

-This Page Intentionally Left Blank-

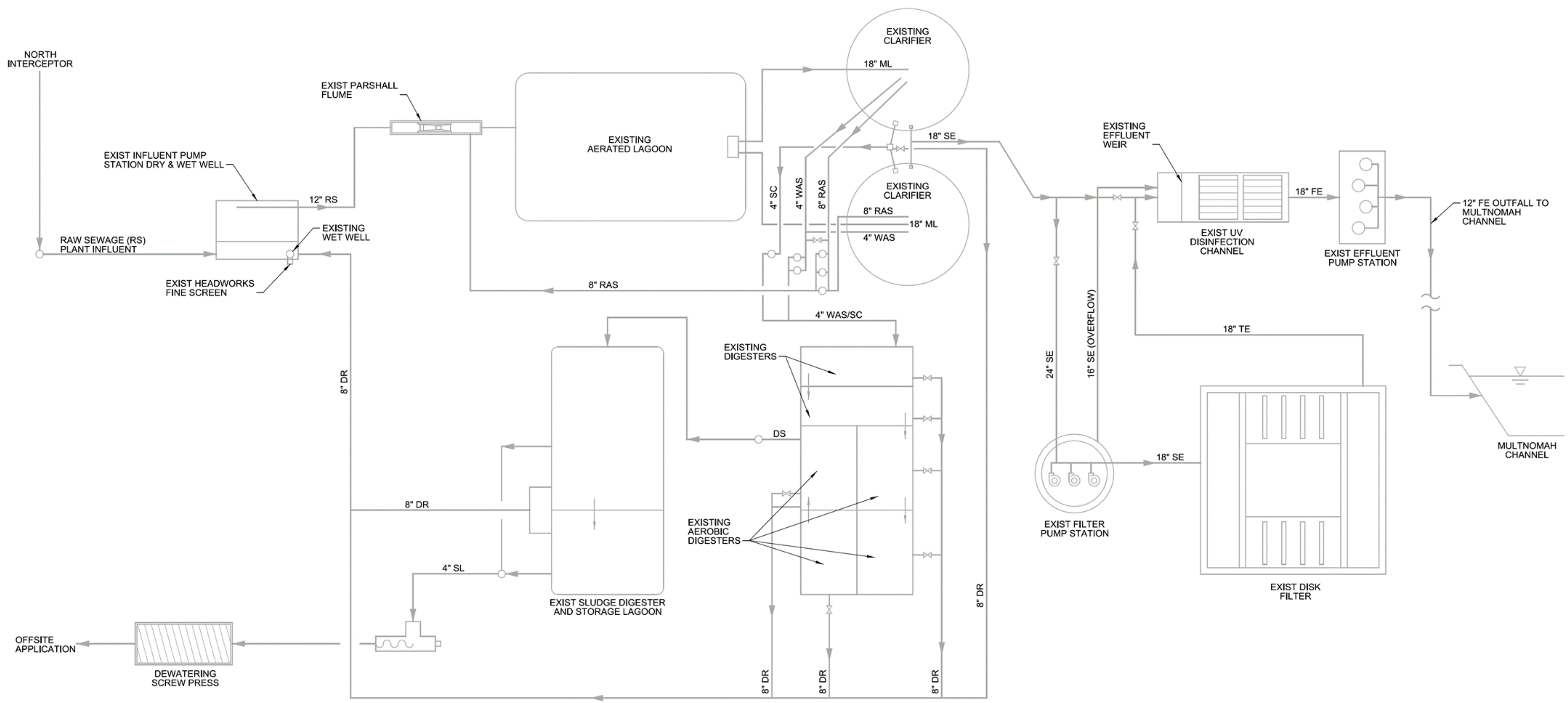


Figure 3.1 Current Plant Process Flow Diagram

-This Page Intentionally Left Blank-

Settling sludge is collected in a hopper at the center of each clarifier and pumped back to the discharge of a Parshall flume upstream of the aerated lagoon via the return activated sludge (RAS) pumps. There are three RAS pumps, designed to operate at 700 gpm each. Two waste activated sludge (WAS) pumps draw sludge from the sludge return line and discharge to the aerobic digester. The WAS pumps operate at 100 gpm and are run at set time intervals and durations to maintain the desired MLSS concentration in the aerated lagoon. Scum is also collected from the water surface in the clarifiers and deposited in a scum pit, where it is subsequently pumped to the aerobic digester.

WAS and scum from the secondary clarifiers are digested in a six-compartment aerobic digester. Sludge flows through each of the six chambers in series and is subsequently decanted and pumped to the sludge storage lagoons by a sludge transfer pump. Air is supplied to all six compartments by two 25 hp air blowers. The digested sludge is stored in one of two sludge storage lagoons (one of which is twice as big as the other) until the sludge is removed and subsequently land-applied.

3.3 Plant Hydraulic Capacity Analysis

3.3.1 Modeling Assumptions

The hydraulic capacity of a system refers to the physical capacity of a wastewater treatment plant to pass wastewater through the plant. A plant may have a peak hydraulic capacity to pass a given flow, which the unit process of the plant would be unable to adequately treat to required levels of effluent quality.

Carollo’s Hydraulix® software was used to develop a hydraulic profile for the WWTP using information from the design drawings for 1992 and 2011 expansions. The software models a treatment plant's flow by calculating the energy and hydraulic grade lines. The energy grade line is the sum of the elevation head, pressure head, and velocity head. Subtracting the velocity head from the energy grade line results in the hydraulic grade line, which measures the system's potential energy at a particular location. Energy losses associated with the flow of wastewater through the plant are added to the energy grade line downstream of each segment of flow to determine the energy grade upstream.

The hydraulic profile developed from *Hydraulix®* was used to determine the hydraulic capacity for each unit process, based on the ability of each process to physically pass wastewater through it. This capacity is defined by the maximum flow resulting in a water surface elevation either at which there is at least 6 inches of free discharge over a weir, or at which there are at least 12 inches of freeboard.

The following assumptions were made about the current plant operation to establish the hydraulic profile:

- All unit processes are on-line during a peak hour event.
- The cylindrical screen openings are 50 percent blocked.
- The overall RAS pumping rate is equal to 1,200 gpm.
- Both secondary clarifiers are in operation during peak flows. The flow split between the two clarifiers is such that the surface overflow rate between the two clarifiers is the same, and the underflow rates are proportional to the corresponding effluent flow rates.
- Multnomah Channel level is at 113 feet, which is assumed to be the 100-year flood.

3.3.2 Results

The hydraulic capacities of each process area at the WWTP are shown in Table 3.1 below, and summarized in the subsequent sections.

Table 3.1 Flow Capacity of Existing Processes

Process Area	Existing Hydraulic Capacity, mgd	Required Hydraulic Capacity, mgd[1]
Influent Screening	4.0 mgd	5.6 mgd
Influent Pumping Station	4.3 mgd	5.6 mgd
Aeration Basin	7.0 mgd[2]	5.6 mgd
Secondary Clarifiers	4.7 mgd	5.6 mgd
Tertiary Filtration System - Intermediate Pumping Station - Filtration	 4 mgd 2.4 mgd	 3.4 mgd[4] 2.4 mgd[5]
UV Disinfection	5.6 mgd[3]	5.6 mgd
Effluent Pump Station	4.4 mgd	5.6 mgd
Outfall	4.3 mgd	5.6 mgd

Notes:
(1) Through planning period (year 2035)
(2) With upstream and downstream restrictions removed.
(3) At this flow, the V-notch weir is completely submerged and exceeds UV bank submergence limits. This limits the accurate flow measurement over the weir.
(4) Year 2035 Summer Peak Instantaneous Flow
(5) Year 2035 Summer Peak Day Flow

3.3.2.1 Influent Screening

The current capacity of the influent screening is 4.0 mgd and is insufficient to accommodate the PIF of 5.6 mgd. Additionally, there is no redundancy for the screening equipment, therefore the 4 mgd hydraulic capacity is not reliable capacity.

3.3.2.2 Influent Pumping Station

The current capacity of the influent pump station, 4.3 mgd (total) and 3.24 mgd (firm), is insufficient to accommodate the PIF of 5.6 mgd.

3.3.2.3 Aeration Basin

The influent parshall flume limits the flow conveyed to the aeration basin to 5.7 mgd. Additionally, the model predicted flooded conditions in the aeration basin effluent weir structure at peak flow (5.6 mgd) which controls the level in the basin. The aeration basin is therefore restricted to peak flow of 5.6 mgd. The capacity can be increased to approximately 7 mgd if upstream and downstream restrictions are removed.

3.3.2.4 Secondary Clarifiers

At the surface overflow rate of 1500 gpd/sf as outlined in Chapter 1, the secondary clarifiers have sufficient capacity (2.9 mgd/clarifier) and the weirs are not flooded during peak flow condition.

3.3.2.5 Tertiary Filtration System

The plant currently does not have any nutrient limits and uses tertiary disk filtration to polish wastewater prior to disinfection and discharge. It is anticipated that future ammonia limits will be imposed during summer months. Therefore the required capacities for the intermediate pumping station and disk filtration are:

- Intermediate pumping station: meet year 2035 summer peak instantaneous flow of 3.4 mgd.
- Disk filters is to meet year 2035 summer peak day flow of 2.4 mgd.

With a current total capacity of the intermediate pump station at 4 mgd and disk filter at 2.4 mgd, the WWTP has sufficient capacity to meet projected flows.

3.3.2.6 UV Disinfection

The UV channel can pass through the projected peak instantaneous flow (PIF) of 5.6 mgd. The model assumes that the downstream tipping weir maintains the water surface elevation in the UV channel at 113.0 during PIF condition. At this level the upstream V-notch weir is completely submerged and limits treatment capacity during peak flow conditions by exceeding bank submergence limits. In addition when the upstream weir is completely submerged, it limits accurate flow measurement over the weir.

The UV system should meet two conditions:

- Provide minimum dose at PIF:
  - The system may be unable to provide sufficient dose during PIF condition.
- Provide sufficient capacity for peak day summer flow with one unit out of service:
  - With one bank out of service, the system does not have enough capacity to meet future summer PIF of 3.4 mgd.

3.3.2.7 Effluent Pumping Station

The current capacity of the effluent pump station, 4.4 mgd (total) and 3.3 mgd (firm), is insufficient to accommodate the PIF of 5.6 mgd.

3.3.2.8 Outfall

The existing outfall is a 12-inch line and has a capacity of 4.3 mgd to keep the velocities under 8.5 fps. Upsizing of the existing outfall or a new parallel line will be required to discharge future PIF.

3.4 Plant Process Capacity Analysis

The secondary treatment processes at the Scappoose WWTP consist of an aerated lagoon and two secondary clarifiers. Currently, these processes are chiefly responsible for removing biochemical oxygen demand (BOD) and total suspended solids (TSS). In addition to BOD and TSS, it is anticipated that future national pollution discharge elimination system (NPDES) permits for the WWTP will limit the discharge of ammonia. In light of these future regulations, this section evaluates alternatives for the WWTP to remove ammonia while providing capacity for growth.

3.4.1 Model Calibration

To develop recommendations for the future process, a whole-plant process model was developed for the current plant configuration in BioWin version 5.1. The dynamic model was calibrated to the latest one year of operation (2/25/2016 – 2/25/2017) using the influent chemical oxygen demand (COD) fractions shown in Table 3.2.

Table 3.2 Model Influent Characteristics

Influent COD fraction	Selected Parameter	BioWin Default
F_{bs} (fraction of the influent COD which is readily degradable)	0.160	0.160
F_{ac} (fraction of F_{bs} which is volatile fatty acid [VFA])	0.150	0.150
F_{xsp} (fraction of the slowly degradable COD which is particulate)	0.825	0.750
F_{us} (fraction of the influent COD which is soluble unbiodegradable)	0.050	0.050
F_{up} (fraction of the influent COD which is particulate unbiodegradable)	0.230	0.130
F_{zbh} (fraction of the influent COD which is biomass)	0.020	0.020
COD/BOD ratio of F_{ac}	1.360	1.360
COD/BOD ratio of rapidly degradable and colloidal COD	1.420	1.420
COD/BOD ratio of F_{xsp}	1.890	1.890
COD/BOD ratio of F_{zbh}	1.790	1.790
COD/VSS ratio for slowly degradable COD	1.600	1.600
COD/VSS ratio of F_{zbh}	1.420	1.420
COD/VSS ratio of F_{up}	1.600	1.600
Overall COD/BOD ratio	2.341	2.034
Overall COD/VSS ratio	2.291	2.646

Since influent COD is not measured, the process model matched the measured influent BOD concentration using an overall COD/BOD ratio of 2.34. During the calibration period, the influent BOD concentration averaged 217 mg/L as shown in Figure 3.2. The process model calculates the influent TSS concentration for each day from the overall COD to volatile suspended solids (VSS) ratio of 2.29. During the calibration period, the plant measured and model calculated influent TSS concentration both averaged 272 mg/L as shown in Figure 3.3.

During the calibration period, the solids retention time (SRT) averaged 56.2 days. The modeled WAS flow was set in the model to match the daily measured SRT as shown in Figure 3.4. The measured MLSS concentration averaged 4,500 mg/L for the year-long calibration period, and the model calculated MLSS concentration was slightly less at 4,200 mg/L, but still followed the observed trends in MLSS concentration closely, as shown in Figure 3.4. The average measured WAS load was 1,780 ppd and the model calculated WAS load was 1,750 ppd as shown in Figure 3.5.

During the calibration period, the secondary effluent ammonia concentration averaged 0.06 mg/L, indicating complete nitrification. With an average dissolved oxygen DO concentration in the aerated lagoon of 2.0 mg/L, the model calculated an average secondary effluent ammonia concentration of 0.12 mg/L (roughly double the observed average, though still indicative of effective nitrification). As shown in Figure 3.6, the model predicted secondary effluent ammonia concentration matched the low observed concentrations. Spikes in the average observed SE ammonia concentration were generally captured in the modeled secondary effluent ammonia concentration.

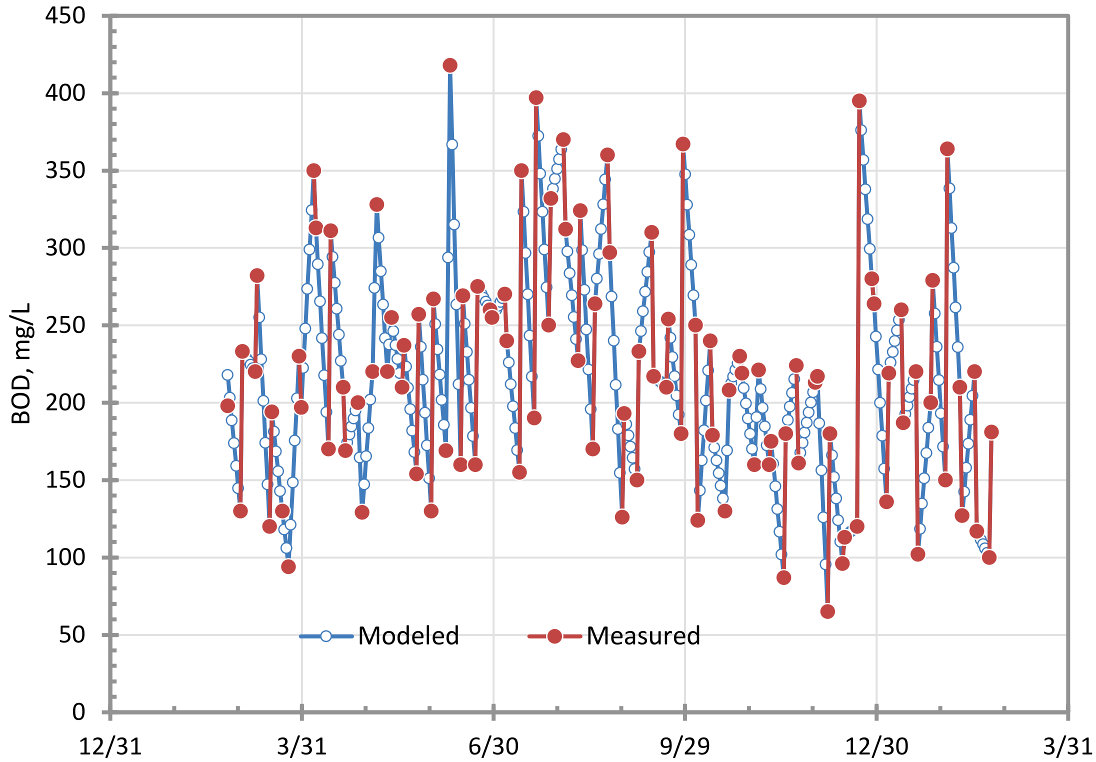


Figure 3.2 Plant Measured and Model Calculated Influent BOD Concentration

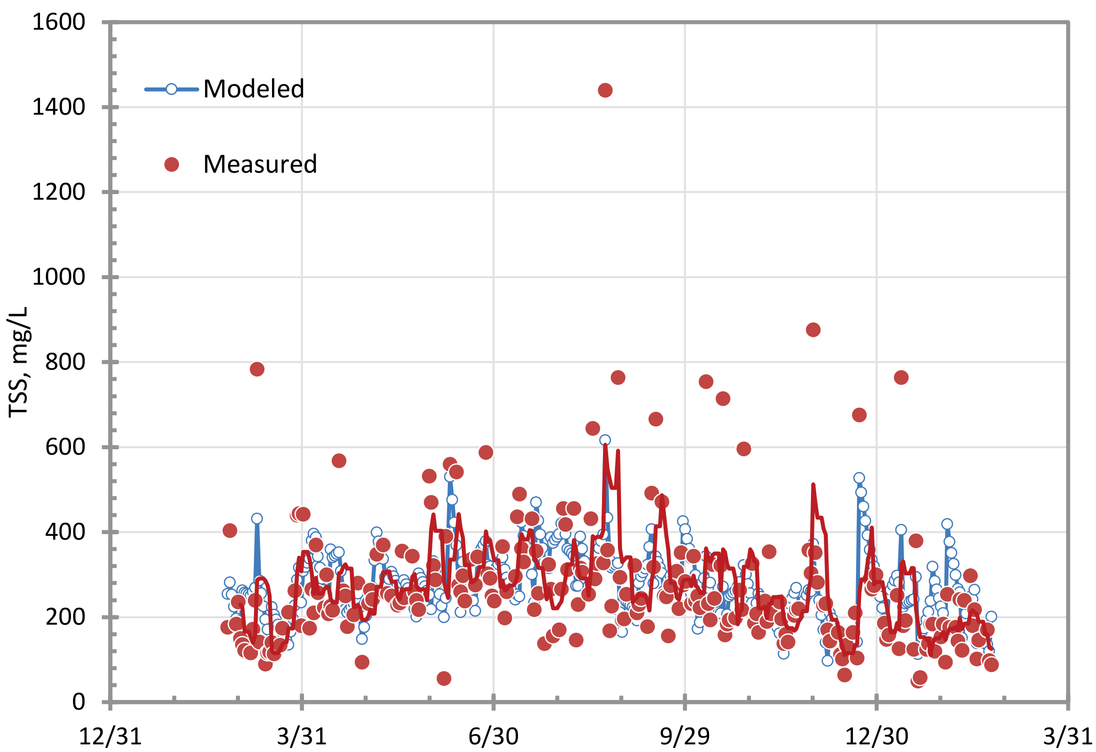


Figure 3.3 Plant Measured and Model Calculated Influent TSS Concentration

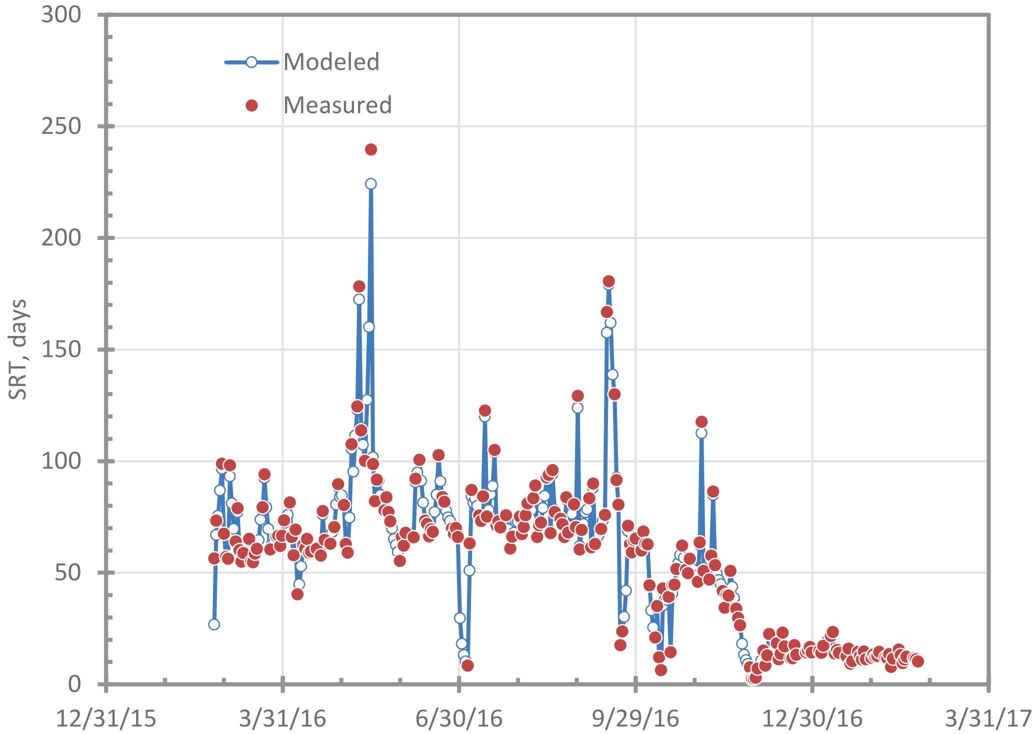


Figure 3.4 Plant Measured and Model Calculated MLSS Concentration

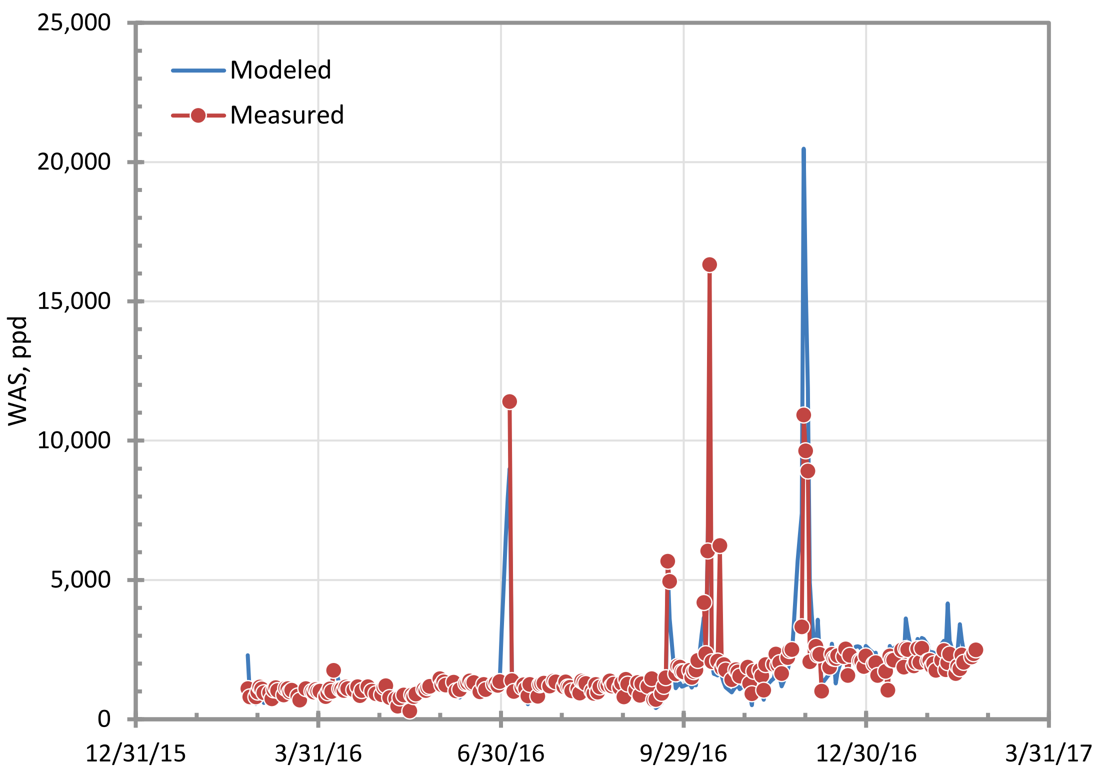


Figure 3.5 Plant Measured and Model Calculated WAS Load

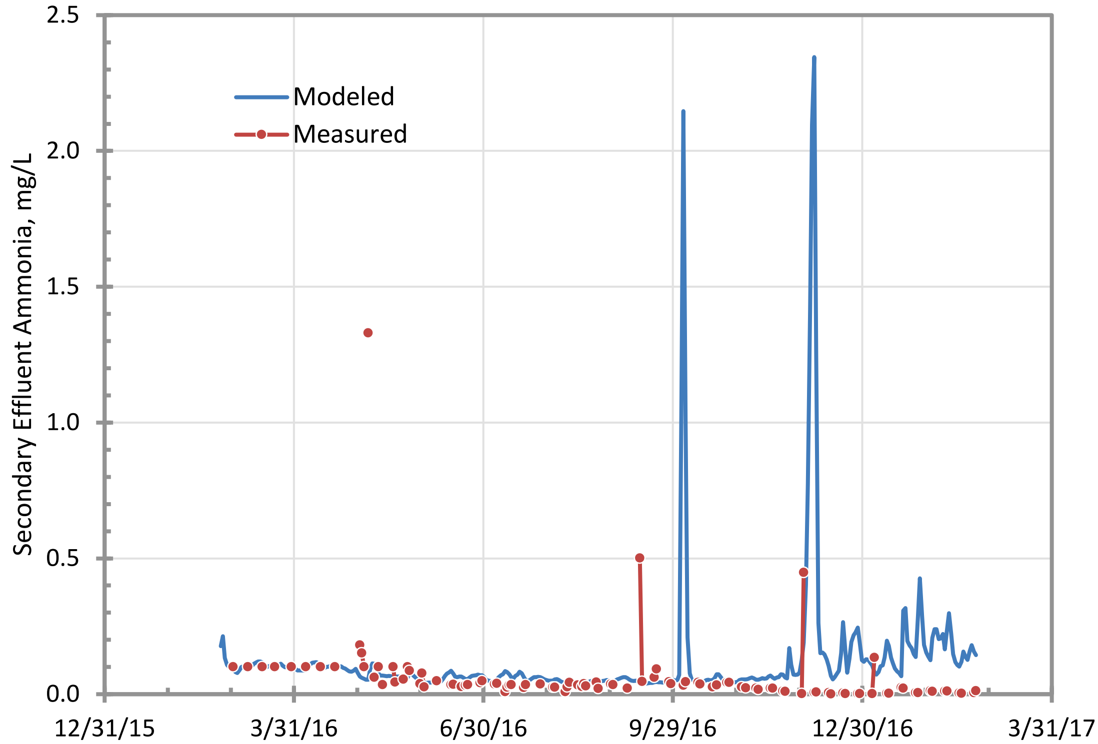


Figure 3.6 Plant Measured and Model Calculated Secondary Effluent Ammonia Concentration

3.4.2 Basis of Planning

Table 3.3 summarizes the effluent quality goals presented in Chapter 1. Based on the reasonable potential analysis (RPA) completed for this project, a reasonable potential was found for end of pipe water quality violations of the newly adopted ammonia criteria developed to protect endangered fresh water mussels when effluent ammonia concentrations exceeded 5 mg/L. For this reason, the summer monthly effluent ammonia concentration goal is 5 mg/L.

Table 3.3 Effluent Quality Goals

Parameter	Average monthly limit Summer	Average monthly limit Winter
BOD_5	10 mg/L	25 mg/L
TSS	10 mg/L	25 mg/L
Ammonia	5 mg/L	No limit

Based on the anticipated permit limit, the WWTP will need to remove ammonia. Most commonly, ammonia is removed through nitrification in the aerated lagoons. The effectiveness of nitrification increases as the SRT increases and is dependent on the wastewater temperature. A minimum month wastewater temperature of 10°C was assumed. For planning purposes, the SRT will be sufficiently long to sustain nitrification with wastewater temperatures as low as 10°C.

The process model predicts that secondary effluent ammonia concentrations will drop below 1 mg/L, once the SRT is greater than 6 days for a wastewater temperature of 10°C as shown in Figure 3.7. Typically, a nitrification safety factor of 1.5 to 2 is used to ensure consistent nitrification. Therefore to reliably nitrify year round, a minimum SRT of 10 days is recommended. As is shown in Figure 3.8, the aerated lagoons already nitrify and produce a 30 day average effluent ammonia concentration less 5 mg/L.

As is shown in Figure 3.8, historically the SRT of the aerated lagoon ranged from about 30 to 80 days and during this time the 30 day average secondary effluent ammonia concentrations were less than 5 mg/L. Within the last year, the average SRT of the aerated lagoon has dropped to between 10 to 15 days and the WWTP has still been able to maintain secondary effluent ammonia concentrations less than 5 mg/L. The more recent plant data supports the SRT setpoint of 10 days for complete nitrification.

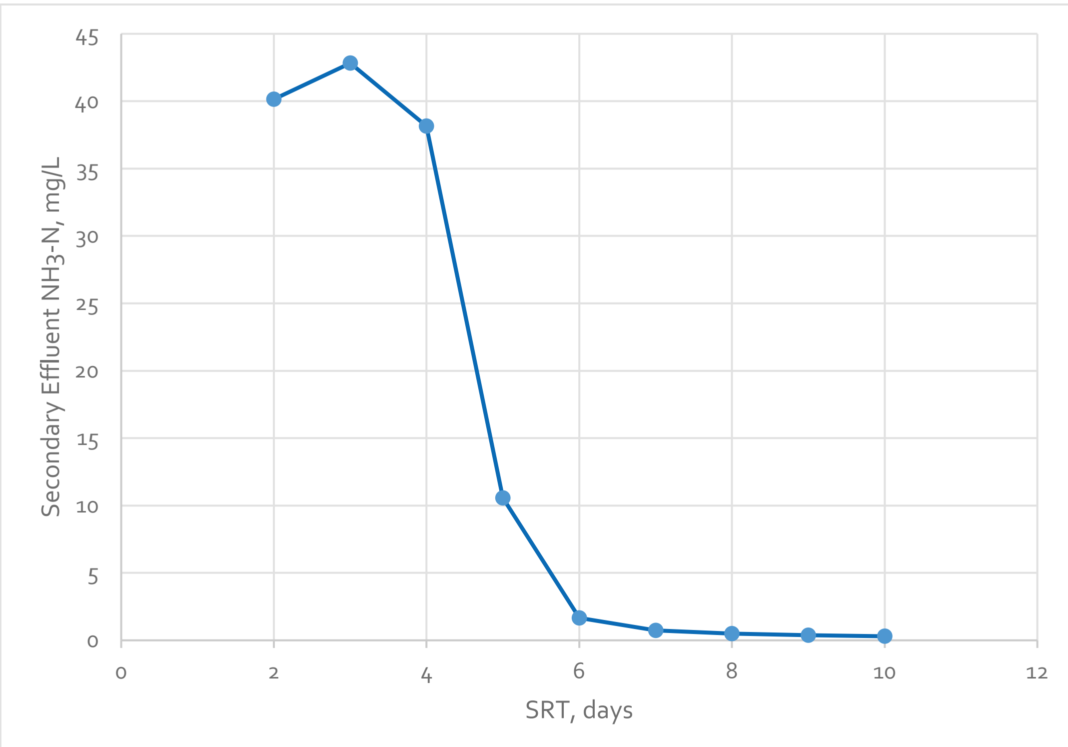


Figure 3.7 Model Predicted Secondary Effluent Ammonia Concentrations versus SRT

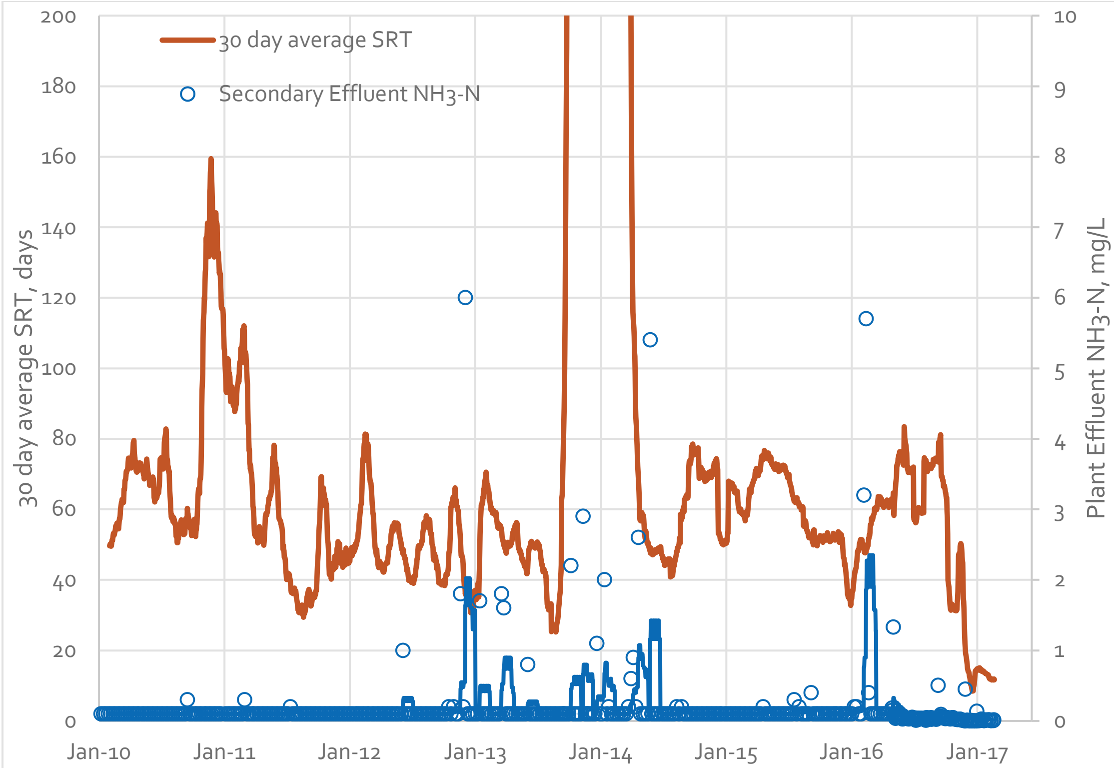


Figure 3.8 Plant Measured SRT and Secondary Effluent Ammonia Concentrations

The capacity of the secondary clarifiers depends on the settleability of the MLSS. Currently, the plant monitors the MLSS settlability though daily measurements of the sludge volume index (SVI). A well settling wastewater will typically have an SVI in the range of 125 to 150 mL/g. As shown in Figure 3.9, the maximum 30-day running average SVI reached as high of approximately 220 mL/g during the first part of 2014. Since the plant was recycling the solids during this time, the SVI is artificially high. Ignoring this period, the highest 30 day average SVI was 129 mL/g. For this reason, the secondary clarifier capacity will be based on a SVI of 129 ml/g.

Based on a state point analysis with two secondary clarifiers in operation (Figure 3.10), with a SVI of 129 mL/g and a peak hour flow of 5.6 mgd, the maximum MLSS concentration is 2,300 mg/L and the resultant surface overflow rate (SOR) is 1400 gpd/sf which is higher than the core design and reliability/redundancy criteria of 1,250 gpd/SF/day, as outlined in Chapter 1. In addition, with one secondary clarifier out of service for repairs, the second secondary clarifier cannot pass the summer peak hour flow as the SOR increases above our maximum design criteria of 1,250 gpd/sf to 1,730 gpd/sf. With only one secondary clarifier in operation, the peak flow that the clarifier can pass is 2.5 mgd and the maximum MLSS concentration under this condition is 2000 mg/L (Figure 3.11). Therefore, to provide adequate peak wet weather capacity and sufficient redundancy to take one clarifier out of service during summer months, it is recommended that one additional secondary clarifier is provided. With three secondary clarifiers in service during the peak hour flow, the state point analysis predicts that the maximum MLSS concentration can increases to 3,400 mg/L (Figure 3.12).

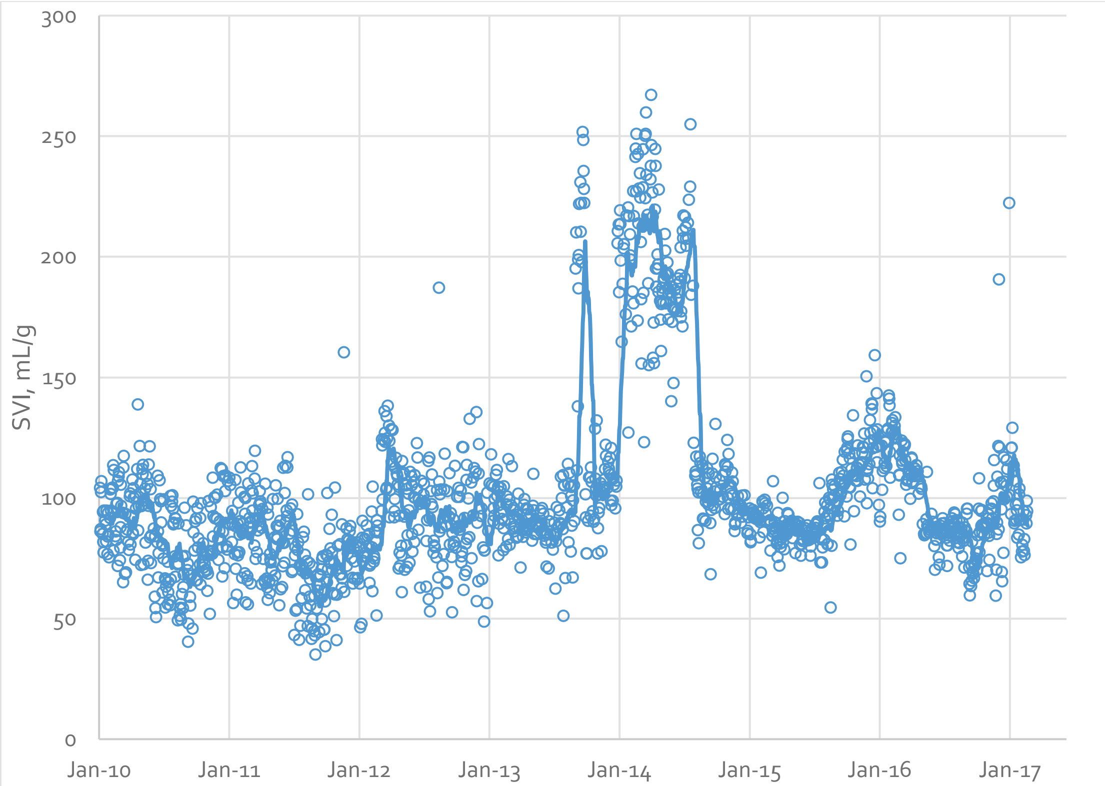


Figure 3.9 Plant Measured SVI

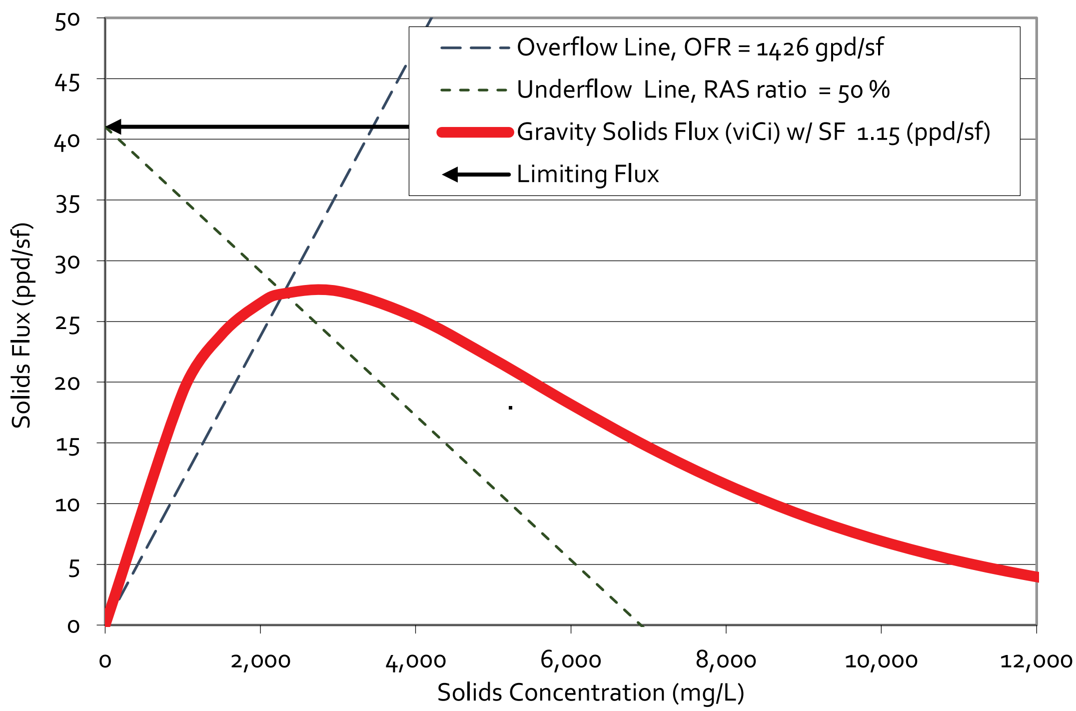


Figure 3.10 State Point Analysis for 2 Secondary Clarifiers in Service during Peak Hour Flow

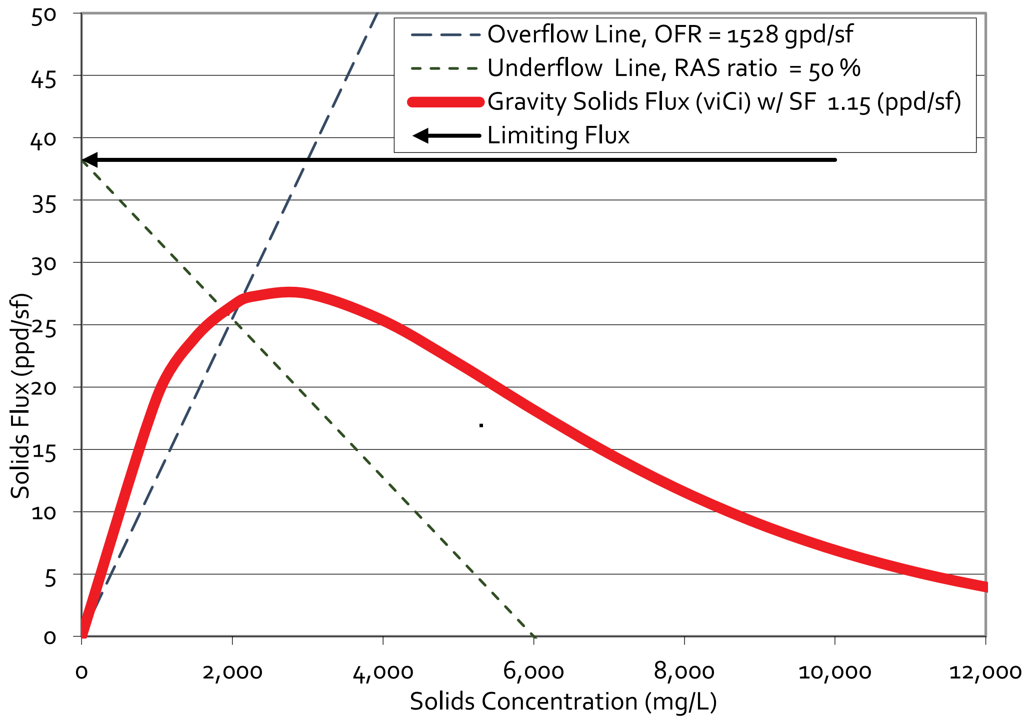


Figure 3.11 State Point Analysis for 1 Secondary Clarifier in Service with 3 mgd Peak Flow

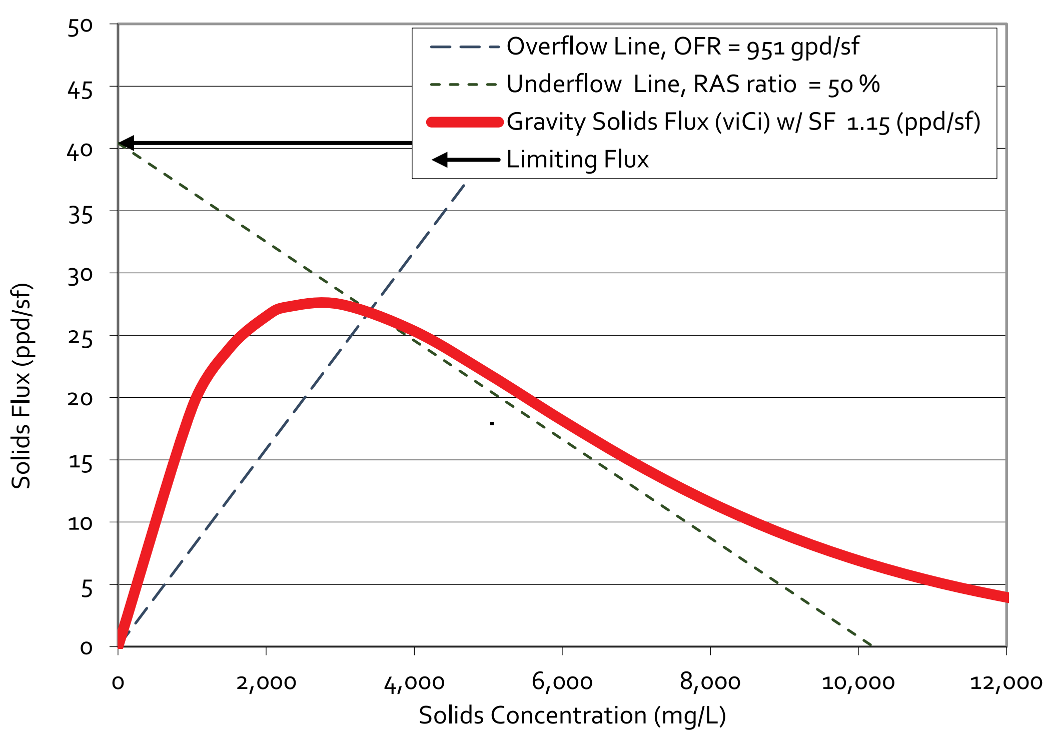


Figure 3.12 State Point Analysis for 3 Secondary Clarifiers in Service during Peak Hour Flow

Table 3.4 summarizes the aerated lagoon and secondary clarifier design and redundancy criteria assumed in the analysis.

Table 3.4 General Secondary Process Design Criteria

Process Area	Design Parameter	Design Criteria	Reliability/Redundancy
Aerated lagoon	Max month flows/loads	• SRT = 10 days at minimum wastewater temperature of 10°C(1) • MLSS < 3400 mg/L with 3 secondary clarifiers • MLSS < 2300 mg/L with 2 secondary clarifiers	• Only one aerated lagoon. Provide provisions to take part of the lagoon out of service for maintenance.
Secondary Clarifiers	Peak Hour Flow	• SOR = 1250 gpd/sf • SVI = 129 mL/g(2)	• One unit out of service during summer peak hour flows.(3)

Notes:
(1) No data available, assumed.
(2) Maximum 30-day running average SVI from plant data from 2015 - 2016.
(3) The system does not currently have capacity to take a secondary clarifier out of service during summer peak hour flows. Summer peak hour flow redundancy will be provide once the third secondary clarifier is added.

3.4.3 Capacity of System for Future Loads

The calibrated BioWin model was run to simulate one year of operation at 2035 flows and loads with the SRT held to 10 days during the entire year of operation. Under these conditions, the model predicted that the MLSS concentration would range from approximately 1,200 to 2,200 mg/L which are less than the maximum MLSS concentration determined from the state point analysis assuming two secondary clarifiers online as shown in Figure 3.13. The model predicted that the 30 day average secondary effluent ammonia convention would be less than 5 mg/L for the entire year. Based on these results, the current system has capacity to provide treatment for the projected year 2035 flows and loads.

Although the current system has sufficient capacity to treat the year 2035 flows and loads, adequate redundancy is not currently provided. Only one aerated lagoon is in operation. Additionally, the secondary clarifiers do not have adequate capacity to pass peak flows through the planning period and with one secondary clarifier out of service, the remaining one clarifier does not have capacity to pass the current and future summer peak hour flows. For this reason, a third secondary clarifier is recommended.

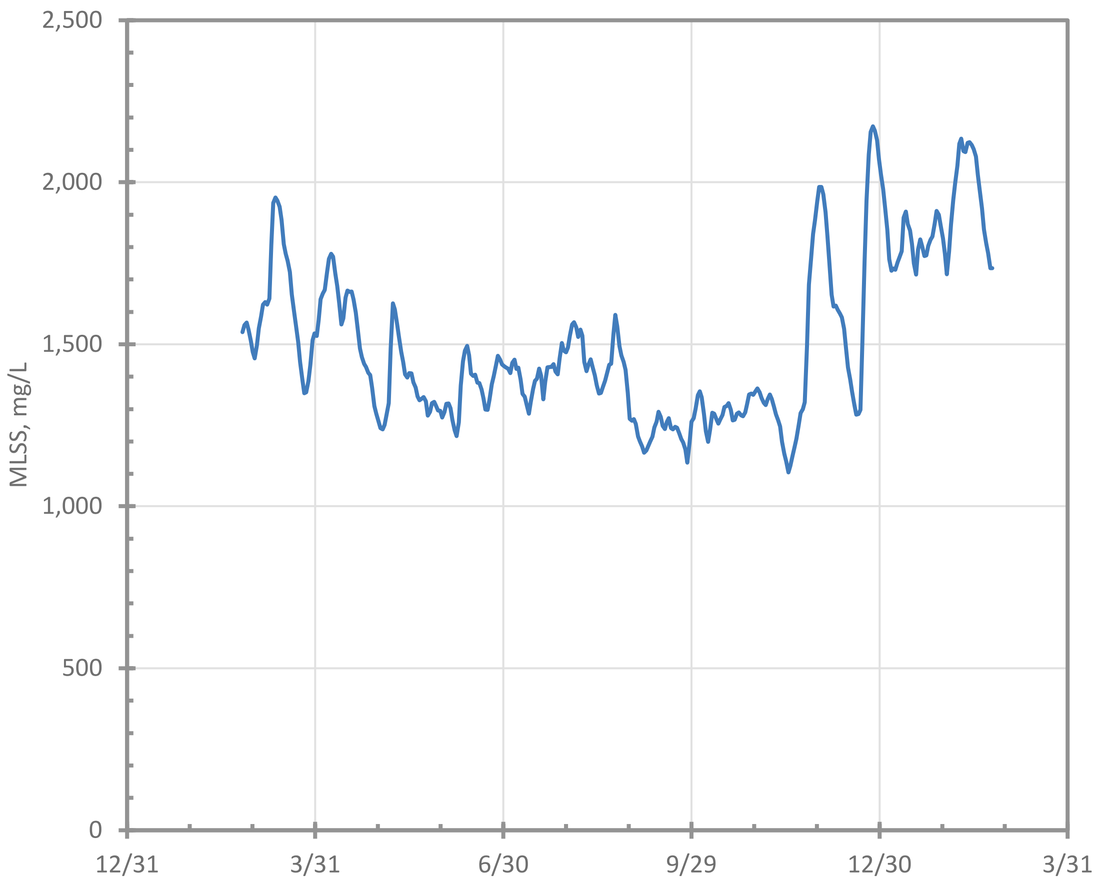


Figure 3.13 Model Predicted Future MLSS Concentration

3.5 Solids Process Capacity Analysis

On the solids processing, raw sewage from the headworks enter the aeration basin where activated sludge is formed. The aeration basin currently has a solids retention time (SRT) of 12 to 92 days. The activated sludge enters the secondary clarifiers, where settling occurs. Waste activated sludge at approximately 1.3 percent solids is pumped from the secondary clarifiers to the aerobic digesters, where stabilization and volatile solids reduction takes place. The current SRT in the digesters is 9 to 20 days (averaging approximately 12 days), depending on the loading, and the operating temperature in the digesters range from 11°C to 20°c, depending on the time of year. At 10 day SRT and 55 percent RAS return rate, the digesters will have an average sludge concentration ranging from 0.3 to 0.5 percent (0.4 average). From the digester, the sludge is pumped to the sludge storage lagoons. Liquid in the sludge either evaporates into the atmosphere or, if the water level is high enough, overflows into a decant box and flows by gravity back to the influent pump station. The solids content of the sludge in the lagoons is typically around 6 percent. The City applies the biosolids from the sludge lagoons to the fields adjacent to the treatment plant site once every one or two years. This year the City purchased a screw press for dewatering and the new protocol is to dewater and store cake prior to land applying.

The City generally stabilizes biosolids adequately with long retention time in the aeration basin, aerobic digester, and in the storage lagoon. However there were a few times when the volatile reduction was not met. This elucidates that the existing aerobic digesters are pretty close to their capacity. In addition, a minimum of 15 days SRT is recommended to alleviate odor issues. Therefore as the flows increase within the system and the retention time decreases the City risks not meeting Class B requirements. It is recommended to build additional aerobic digester capacity within the planning period. The sludge storage lagoons and cake storage have adequate capacity through the planning period.

Chapter 4

RECOMMENDED IMPROVEMENTS

4.1 Objective

The objective of this chapter is to identify the recommended liquid and solids treatment process improvements needed through the planning period. Recommended improvements were developed to:

- Meet expected regulatory scenario developed in Chapter 1 – Basis of Planning,
- Meet projected flows and loads as identified in Chapter 1 – Basis of Planning, and Chapter 3 – Hydraulic and Process Capacity Analysis,
- Provide sufficient redundancy, and
- Alleviate condition and life safety issues identified in Draft Wastewater Facilities Planning Study (Keller Associates, December 2016).

4.2 Cost Estimation

The cost estimates presented in this Chapter have been prepared for general planning purposes and for guidance in project evaluation and implementation. Costs were estimated by developing preliminary layouts and quantity takeoffs, and using quotes from equipment manufacturers. The costs are based on an Engineering News Record Construction Cost Index (ENR CCI) 20-City Average of 10,842 (August 2017). Construction costs do not include contingencies. Electrical/instrumentation /controls costs were estimated by applying a factor to the base construction cost. Factors for mobilization/demobilization and contractor's overhead and profit were applied to the sum of the base construction costs and costs for electrical/instrumentation/controls. The total estimated construction costs include the work items described, plus mark-up costs for electrical/instrumentation controls, mobilization/demobilization, and contractor's overhead and profit. Contingencies were estimated by multiplying the sum of the estimated construction costs by a mark-up factor. The total project costs were calculated by multiplying the sum of the estimated construction cost and contingencies by a factor to account for engineering, legal and administrative costs. These mark-ups are shown in Table 4.1.

Table 4.1 Mark up Factors Used in Developing Cost Estimates for Alternatives

Item	Markup
Mobilization/Demobilization	10%
Contractor Overhead and Profit	15%
Contingencies	30%
Engineering, Legal and Administrative	25%

4.3 Liquid Stream Treatment

Based on the evaluations completed in Chapter 3, the following unit processes need additional capacity and/or replacement due to age:

- Headworks (Influent Screening and Pumping): Insufficient capacity (screening and pumping), redundancy (screening), and poor condition (pumping).
- Secondary Clarification: Insufficient capacity and redundancy.
- UV Disinfection: Insufficient capacity, redundancy, and aged infrastructure.
- Effluent Pumping and Outfall: Insufficient capacity.

4.3.1 Headworks

Issues at the headworks (influent screening, pumping and flow measurement) include:

- Insufficient equipment capacity for future PIF,
- Insufficient wet well volume for future PIF,
- Lack of freeze protection on the influent screen,
- No grit removal,
- Inaccurate level measurement in the influent wet well,
- Influent pumps that are not controlled as a system and are located in a wet and hazardous environment,
- HVAC system that is not sized to create an unclassified environment according to NFPA 820 requirements for the influent pump station, and
- Sump pump in the pump dry pit that does not have redundancy.

Two alternatives were evaluated to address these deficiencies and are detailed below. A

4.3.1.1 Alternative 1: Repair and Expand Existing Headworks

This alternative would include addition of an influent flow meter, and the influent pumps would be replaced to provide capacity for future PIF. Additional heat tracing would be added to the influent screen to address the freezing concerns. The bubbler would be replaced with an ultrasonic level sensor or pressure transducer. Electrical controls for the pumps would be replaced and the HVAC system would be upgraded to meet current code. An additional sump pump would be installed and piped into the existing sump piping. The total cost for this alternative would be approximately $5M (estimate from Draft Wastewater Facilities Planning Study (Keller Associates, December 2016).

4.3.1.2 Alternative 2: Construct New Headworks

Under this alternative, a new headworks with influent screens and pumps would be constructed. It may be possible for the existing influent screen to be relocated and reused inside the new headworks. However, to be reasonably conservative for budgeting purposes, new screening units are assumed. This alternative would include the following:

- Redundant ¼-inch multi-rake screens with passive bypass channel.
- Submersible, non-clog influent pumps installed in a wet-pit. Installation of submersible pumps in a wet-pit offers substantial capital cost savings because construction of a dry-pit structure is eliminated along with the associated ventilation, access facilities, and utilities that are required to allow personnel entry.
- Flexibility to include grit removal in future, as required.

The estimated total cost for this alternative would be $5.5M.

4.3.1.3 Recommended Headworks Alternative

While Alternative 1 utilizes existing space, there are several disadvantages including:

- Even with new influent pumps, the wet well volume is insufficient and will create pump cycling issues,
- There is no additional space to provide additional screen for capacity and redundancy,
- The alternative has higher construction risk since the treatment plant needs to be in operation during construction,
- Provides no future flexibility to add grit removal, and
- Does not provide significant cost savings as compared to Alternative 2.

Therefore Alternative 2 is recommended. This will provide the City with the required capacity and flexibility through the planning years and build-out.

4.3.2 Secondary Clarification

The existing secondary system (aeration basin and secondary clarifiers) have sufficient capacity to meet current flows and loads. However, the secondary clarifiers do not have sufficient capacity for future flows within the planning period. Additionally, the flow split to the clarifiers is not equal (resulting in periodic overloading of the clarifiers), and the wiring to the clarifier drives is in need of repair. Therefore a new 50-foot secondary clarifier is recommended. This will allow for sufficient clarifier capacity to allow the existing secondary clarifiers to be taken out of service for maintenance and rehabilitation during summer time.

When the new clarifier is added, the existing RAS and WAS pumping should also be upgraded.

4.3.3 UV Disinfection

The UV system is critical to meet permit requirements and currently has the following issues:

- Insufficient capacity and redundancy to meet future flows,
- The plant is unable to obtain lamps and parts as the installed UV system is no longer supported by the manufacturer,
- Diminishing system efficiency likely due to its age.

There are three alternatives were considered to address these deficiencies and are presented below.

4.3.3.1 Alternative 1: Replace Aged Equipment with Updated Technology with Bypass Channel

Under this alternative, the equipment in the existing channel will be replaced with updated with current technology from the same manufacturer (Trojan 3000Plus). A bypass channel will be added to add equipment in future when flows increase. The estimated total cost for this alternative would be $1.5M.

4.3.3.2 Alternative 2: Replace Aged Equipment with Updated Technology with Bypass Channel Equipped with UV Banks

This alternative is similar to Alternative 1 and provides equipment in the second channel, satisfying both capacity and redundancy requirements. The estimated total cost for this alternative would be $2.0M.

4.3.3.3 Alternative 3: Leave Existing As-Is and Add New channel with Latest Technology

Under this alternative, a new channel sized for the entire future PIF would be added. A newer technology (Low Power High Output – LPHO) UV system would be added to the new channel. The new LPHO system would control UV dose based on influent flow characteristics (sensor based) and save significant energy over other systems. Additionally, the lamps are 2-4 times larger in wattage, reducing the number of lamps in the system that need to be maintained. The existing channel and old UV system would be left "as-is" to provide redundancy for the new UV system. If the existing equipment needs to be replaced, channel modifications will be required to retrofit with newer LPHO system. . The estimated cost for this alternative would be $1.8M.

4.3.3.4 Recommended UV Disinfection Alternative

The existing UV disinfection system does not have sufficient capacity for the projected peak hour flows. Also, several operation and maintenance issues need to be addressed within the short-term for the existing system. Therefore, the recommended plan includes replacing the existing system with an updated technology as soon as possible and adding a second channel with redundant UV equipment for additional flows through the planning period as outlined in Alternative 2.

4.3.4 Effluent Pumping and Outfall

The effluent pumps have insufficient capacity and redundancy for current and projected PIF and are at the end of their useful life. With a single 12-inch outfall, they also do not have sufficient head to convey final effluent to the Multnomah Channel. Additionally, the 12-inch outfall does not have capacity for projected flows. A new parallel 16-inch outfall is recommended.

Although the pumps and minor improvements to the effluent pump station can be separated from outfall improvements, it is recommended to combine effluent pumping improvements in conjunction with outfall. This enables the City to build pumps to the required head conditions. The estimated total cost for effluent pumping and outfall is $2.9M.

4.4 Solids Stream Treatment

The WWTP's solids stream deficiencies are:

- Aerobic digester: Insufficient capacity and aged equipment.
- Dewatering: Insufficient redundancy.

4.4.1 Digestion

The existing aerobic digester is currently not adequately sized to achieve Class B biosolids, as required for currently land application practices. Additionally, the existing aerobic digester has the following condition issues:

- Some of the coarse bubble diffusers are not working, which means that some cells are not receiving adequate oxygen.
- Digester blowers are not rated to meet current NFPA 820 requirements.
- Safety railing is falling apart,
- No redundancy for the backup sludge pump, and
- Sludge flow meter is currently not working.

The recommendation for the aerobic digester is to repair and upgrade the existing digester immediately. It is anticipated that a second digester will be required within the planning period to achieve Class B biosolids. The estimated cost for repair and upgrade of the existing aerobic digester is $300,000 and the estimated cost for the second aerobic digester is $2.8M.

4.4.2 Dewatering

The City's screw press has adequate capacity through the planning period. When the screw press needs to be serviced, sludge can be stored in the biosolids storage lagoons, satisfying the redundancy requirements. Therefore, a second screw press is not recommended within the planning period.

Chapter 5

IMPLEMENTATION PLAN

5.1 Introduction

The purpose of this chapter is to identify how the improvements identified in the previous chapters will be implemented at the Scappoose Wastewater Treatment Plant. Specific attention was paid to those tasks requiring implementation within the next five years. The implementation plan was developed by weighing the trigger dates determined by the process capacity criteria with the implementation priority determined through discussions with City staff.

5.2 Implementation Program and Schedule

The recommended implementation program and schedule is presented in Table 5.1. The timing of the improvements is typically based on one or more of the following generalized factors:

- Need for additional capacity.
- Need for performance improvements.
- Need for life safety improvements.
- Available financial resources.

For projects of significant size, a 3-year duration was estimated. This window provides adequate time for preliminary design, final design, bidding, and construction. Some projects such as the recommended UV disinfection alternative is split into two phases to allow schedule allowance for funding acquisition. A summary of cost estimates is provided in Appendix B.

Table 5.1 Implementation Program and Phasing Schedule

Item Description	Description	Construction Year				Total
		Phase 1 2018 – 2021	Phase 2 2021 - 2023	Phase 3 2023 - 2026	Phase 4 2026 – 2028	
Phase 1 Improvements						
Spring Lake Lift Station	Replacement of pumps and corroded piping; addition of valve vault to measure flow.	$271, 600				$6,430,600
UV Disinfection	Replacement of existing UV banks with newer technology UV system (Trojan 3000Plus).	$616,600				
Hydraulic Improvements	Rebuilding secondary splitter structure.	$519,700				
Secondary Clarifier and RAS/WAS Pumping Upgrades	Addition of third 50-foot secondary clarifier, update existing clarifier wiring, and expand RAS/WAS pumping.	$4,590,100				
Aerobic Digester Life Safety Improvements	Replace damaged coarse bubble diffusers, replace two blowers, sludge pump, sludge flow meter, fix hand rails.	$432,600				
Phase 2 Improvements						
Headworks and Influent Pump Station	New two fine screens with passive bypass channel and submersible influent pump station.		$5,504,400			$7,204,400
Operational Improvements	SCADA integration and new lab.		$1,700,000			
Phase 3 Improvements						
UV Disinfection	Addition of second channel and equipment.			$1,685,900		$4,393,200
Effluent Pump Station	Replacement of pumps, modification of skylights, addition of flow meter, electrical improvements.			$536,600		
Outfall	New parallel 15-inch outfall.			$2,170,700		
Phase 4 Improvements						
Aerobic Digester	New aerobic digester to achieve Class B biosolids.				$2,486,900	$2,486,900
TOTAL WWTP CIP						**$20,515,000**

Chapter 6

PREDESIGN REPORT

6.1 Introduction

This chapter presents detailed description of proposed CIP Improvements (as outlined in Chapter 5) for the City of Scappoose WWTP. The following information regarding near-term projects is provided in this report:

- Proposed flow and process schematics,
- Design criteria and capacities,
- Preliminary electrical and instrumentation and control improvements required, and
- Overall site plan.

6.2 Phase 1 Improvements

Three projects were included in the immediate improvements:

- Spring Lake lift station improvements
- UV Disinfection equipment replacement,
- Hydraulic Improvements,
- Secondary clarifier and RAS/WAS pumping upgrades, and
- Life safety and condition improvements at the aerobic digester.

6.2.1 Spring Lake Lift Station Improvements

The Spring Lake Lift Station is located near the intersection of Westlake Drive and SE 6th Street. The lift station is a prefabricated package style unit and consists of a wet well with four float switches, two 3-hp pumps mounted on top of the wet well, and a control panel.

The station is fed by three gravity sewers. Two of the three gravity lines are private lines. The pumps are controlled by the float switches using a lead/lag operational strategy. Mission Controls sends alarms and operating data to the operator. The station is powered by single phase electrical supply and has a manual power transfer switch with receptacle. In the event of a power failure, a portable generator is used for backup power.

Each pump at the station has a capacity of 140 gpm. There have been no known issues with the lift station overflowing or with pumps running continuously for an extended period of time. The lift station overflow discharges directly into Spring Lake.

The lift station regularly alarms due to insufficient priming issues with the pumps. Additionally, City staff noted that the force main has a high point prior to discharging to the gravity collection system.

The following is recommended for immediate improvements:

- Perform engineering analysis to determine the correct size of the pump and suction piping,

- Replace pumps and associated valves,
- Replace corroded piping within the wet well,
- Perform electrical system review and provide 3-phase service, and
- Provide SCADA upgrades.

The total estimated cost for Spring Lake lift station improvements is estimated at $272,000.

6.2.2 UV Disinfection Equipment Replacement

The existing UV system is a Trojan UV3000 model that utilizes low-pressure, low-output UV lamps and installed in 1993. The system does not have sufficient redundancy required to meet projected flows through the planning period as presented in Chapter 3. In addition, in recent years, the City had delays acquiring some key replacement parts since Trojan Technologies is no longer supporting this system. This along with the service life of the installed system, is the basis of recommended alternative which is to initiate replacement with a new UV system with a more current technology. The selected replacement UV system must balance capital cost with reliable, energy efficient operation and meet the following UV equipment replacement project objectives:

- Utilize existing infrastructure,
- Optimize performance and efficiency, and
- Design for long-term operation and maintenance (O&M) value.

The existing channels have the hydraulic capacity to meet the build-out flows. Reusing existing channels by carefully orchestrating project sequencing saves money as the cost of a UV system is largely driven by two factors: the number of lamps in the system, and the structural/mechanical modifications needed to complete the installation.

Based on channel dimensions, the most cost-effective approach is to replace the existing system with an open-channel horizontal lamp reactor that will fit within the existing channel. Preliminary selection discussions with UV vendors suggests that Trojan UV3000Plus offers the best fit with no channel modifications. Replacement with a newer Trojan product also offers the City with significant reduction in the number of lamps which will reduce capital and O&M costs.

The process design criteria for the new UV equipment is presented in Table 6.1.

Table 6.1 UV Process Design Criteria

Parameter	DEQ Design Criteria	UV Channel Capacity
Total peak flow through the disinfection system	Provide minimum of two units that will provide required dose at peak instantaneous flow with all units in service. Required Flow for Design Year 2035 is 5.6 mgd.	5.6 mgd
Design Dose	Minimum 30 mJ/cm2; or per collimated beam testing.	30 mJ/cm2
Design UVT	Based on historical data.	68%
Sleeve Fouling Factor(1)	N/A	0.80
Lamp Aging Factor(2)	N/A	0.80

Notes:
(1) Sleeve fouling is a measure of relative sleeve transmittance over time. Fouling can decrease the delivered UV dose over time and can be mitigated through cleaning practices
(2) The output from UV lamps degrade over time, resulting in a decrease in the UV output required for disinfection. Therefore, this degradation must be taken into account during the design process

6.2.3 Hydraulic Improvements

Based on the hydraulic modeling completed, the effluent structure from the aeration basin conveying flows to the secondary clarifiers will be flooded under design flow condition. This will be replaced to accommodate sufficient weir length to capture even flow splitting between the existing secondary clarifiers and provisions for adding two additional secondary clarifiers in future.

6.2.4 Secondary Clarifier and RAS/WAS Pumping Upgrades

6.2.4.1 Secondary Clarifier

The existing secondary system (aeration basin and (2) 50-foot secondary clarifiers) have sufficient capacity to meet current flows and loads. However, the secondary clarifiers do not have sufficient capacity for future flows within the planning period. As identified in Chapter 3, the state point analysis results show that the surface overflow rate is higher than the core design criteria identified for the facility. Additionally, with once clarifier out of service during summer time, the second clarifier cannot pass peak summer flows. Therefore, to provide adequate peak wet weather capacity and sufficient redundancy to take one clarifier out of service during summer months, a new secondary clarifier is needed. The analysis assumes that the third secondary clarifier will be of similar size as existing (50-foot diameter).

6.2.4.2 RAS/WAS Pumping Upgrade

The Sludge Pumping Building holds three (3) 7.5 hp vertical centrifugal RAS pumps and two (2) 2 hp vertical centrifugal WAS pumps. The RAS pumps are each designed to pump 700 gpm at 17 ft. TDH, with one of the RAS pumps as a standby. The RAS pumps were last rebuilt in 2010-2011. The WAS pumps have a combined capacity of approximately 265 gpm at 20 ft. TDH and a firm capacity (with one pump operating) of approximately 135 gpm.

With the addition of a new secondary clarifier, RAS and WAS pumping capacity needs to be increased. However the Sludge Pumping Building does not have space for any additional pumps. This upgrade would include increasing building size and adding additional capacity for RAS and WAS pumping, upgrading existing WAS pumps, and upgrading HVAC system.

6.2.5 Life Safety and Condition Improvements at the Aerobic Digester

Based on the condition assessment and discussions with the City staff, the following is needed to fix condition and improve life safety at the aerobic digesters:

- **Replace plugged coarse bubble diffusers and broken Swingfuser:**
  Coarse bubble diffusers in the bottom of the digester cells distribute the air from the blowers. However, some of the diffusers are plugged and it is difficult to take down a cell to clean and still meet Class B requirements. Additionally one of the "swingfusers" is broken, which doesn't allow air to reach the cell. The basin is not being adequately mixed, which is reducing the volatile solids destruction.

- **Review location of blowers for code compliance and replace as needed:**
  Two (2) 25 hp blowers are located on the main floor in the Headworks building and are used to provide air to the digester. The blowers draw air in from the building through their inlet filters. A review of the electrical system should be a part of any upgrade to

ensure compliance with the Standard for Fire Protection in Wastewater Treatment and Collection Facilities (NFPA 820).

- **Add a redundant sludge pump and replace sludge flow meter:**
  A suction line in the bottom of the last cell is connected to a 5 hp Wemco Model EVM sludge transfer pump located in the Headworks. There is no redundancy for this pump. In the event of failure, the plant cannot transfer digested sludge to the Biosolids Storage Lagoons. In addition, the 6-inch electromagnetic flow meter is normally used to measure the sludge flow to the lagoons is currently malfunctioning and needs replacement.

- **Fix broken railing and complete other life safety fixes to comply with OSHA standards:**
  Some of the safety railing near the basin is corroded or missing. This along with other access area issues should be addressed.

6.3 Phase 2 Improvements

Phase 2 includes the following improvements:

- Headworks and influent pumping, and
- Operational improvements.

6.3.1 Headworks and Influent Pumping

The existing headworks and influent pumping station (IPS) has both capacity and condition issues as outlined in Chapter 3. The influent wet well is undersized and the existing headworks does not accommodate expansion of the screens to add redundancy. Therefore a new headworks (new screening and influent pumps) with ability to add grit removal system is recommended. Figure 6.1 presents the process flow schematic for the proposed improvement.

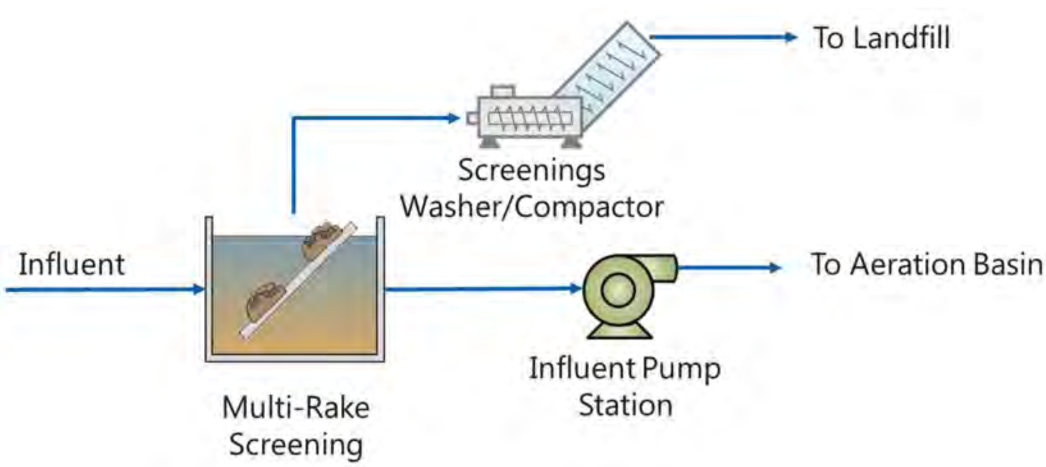


Figure 6.1 Headworks Process Flow Schematic

6.3.1.1 Screening

There are several proven screening technologies that can effectively remove debris from influent wastewater flow. These include multi-rakes, step screens, perforated plate screens. Based on Carollo's experience, multi-rake screens offer several advantages including:

- Lower head loss,
- Frequent cleaning passes and high screenings removal rate,
- Control of cycling frequency using multi-speed drives,
- Easily removes large floatables,
- Depending on the manufacturer there are no submerged bearings or moving parts in the screening channel, and
- No spray water and/or brushes required.

Therefore these screens are included in the pre-design. It is recommended that the City tour different screening technologies before final design. A brief description of the mechanics of multi-rake screens is included in this section. The multi-rakes comprise of parallel bars on a bar rack, with multiple rake bars mounted onto chains on both sides of a self-contained frame. The screens are configured so the rakes clean and return in front of the bar rack to prevent carryover of material to the downstream channel. The screen sits submerged within a channel collecting debris as influent flow passes through. The screen works on a level differential, and the cleaning rakes are signaled to start when the head loss across the screen reaches an operator setpoint. The cleaning rakes comb through the bars conveying debris and discharging through the discharge chute to either a washer/compactor or a conveyor. The screen is capable of pivoting out of the channel for maintenance. Figure 6.2 shows a diagram of a typical multi-rake bar screen (Source: Headworks, Inc.).

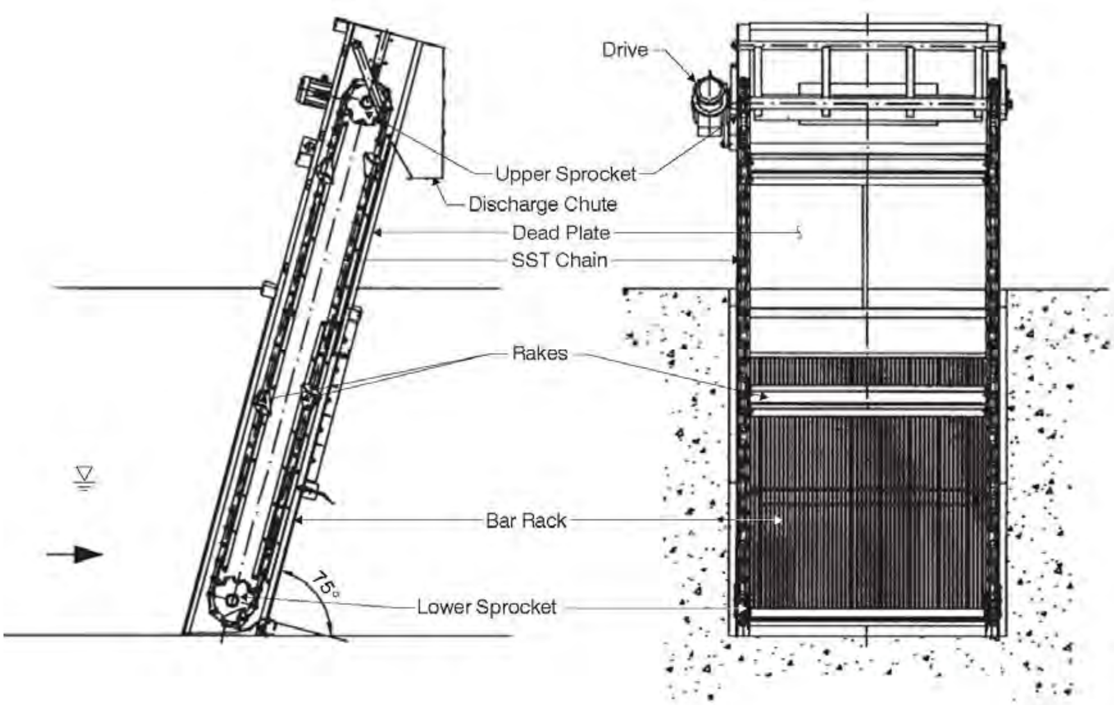


Figure 6.2 Typical Multi-Rake Bar Screen

6.3.1.2 Screenings Handling

Screenings Conveyance

Screenings must be conveyed to a washer/compactor for further washing and dewatering. Conveyance technologies that can be used for screenings conveyance include belt conveyors, sluiceways, and shaftless screw conveyors. Belt conveyors are not recommended for this application because they are odorous and require more operations and maintenance than screw conveyors. Screenings material placed on a belt conveyor is difficult to contain, can spill over the sides of the conveyor and requires frequent maintenance.

A sluiceway for screenings conveyance is also not recommended due to the amount of flush water required. Sluiceways typically require 300 to 400 gpm of flush water to convey screenings properly. Although this flow would be used in the screenings washing process, it would then need to be returned to the influent flow stream and re-routed through the plant process at a cost.

The recommended method for conveying screenings is a shaftless screw conveyor. A shaftless screw conveyor can accommodate multiple inlet points collecting screenings from multiple screens to convey to a common washer/compactor. The equipment consists of a hardened steel spiral installed within a stainless steel U-shaped trough and driven by a motor and gearbox as shown in Figure 6.3 (Source: American Bulk Conveying). The screw conveyor is equipped with removable stainless steel covers that completely enclose the screenings, containing odors and preventing material from spilling out of the conveyor. Shaftless screw conveyors are a simple, proven, and reliable technology for screenings conveyance.

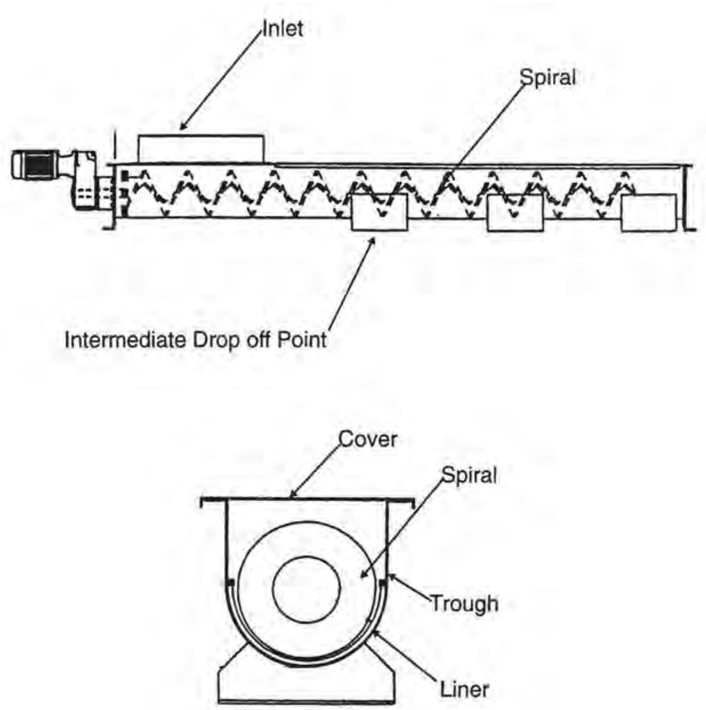


Figure 6.3 Shaftless Screw Conveyor

Screenings Washer/Compactor

In addition to rags and inert material, at the selected bar spacing of 1/4 inch, the screens will also remove fecal and other organic matter resulting in odorous materials that must be handled. By washing the screenings, the organic matter can be separated from the inert material before dewatering and disposal. Lowering the organic content will reduce odors, improve dewatering, and reduce the weight, volume, and cost of materials hauled to landfill. Therefore a screenings washer/compactor is recommended.

Screenings washing and dewatering can be accomplished with a single piece of equipment known as a washer/compactor. A typical unit consists of a motor-driven shafted or shaftless helical screw auger, an inlet hopper, and a discharge tube.

The washer/compactor uses a water spray applying high pressure utility water at several points in the compaction unit. Water sprays have limited success in breaking up organic matter, which is why some washer units also conduct batch washing process where the auger cycles backward and forward a number of times to break the organic material apart.

Following the washing cycle, dewatering of washed screenings occurs. Figure 6.4 provides a drawing of a washer/compactor (Source: Vulcan). Turning the auger pushes the wet screenings material from the inlet hopper into the discharge tube, which is tapered for a short length at its inlet. This reduction in diameter at the inlet of the discharge tube causes a "squeezing" action that removes the free water from the screenings material. The water drains out of the unit through a perforated plate at the bottom of the unit and is returned to the flow stream for further treatment.

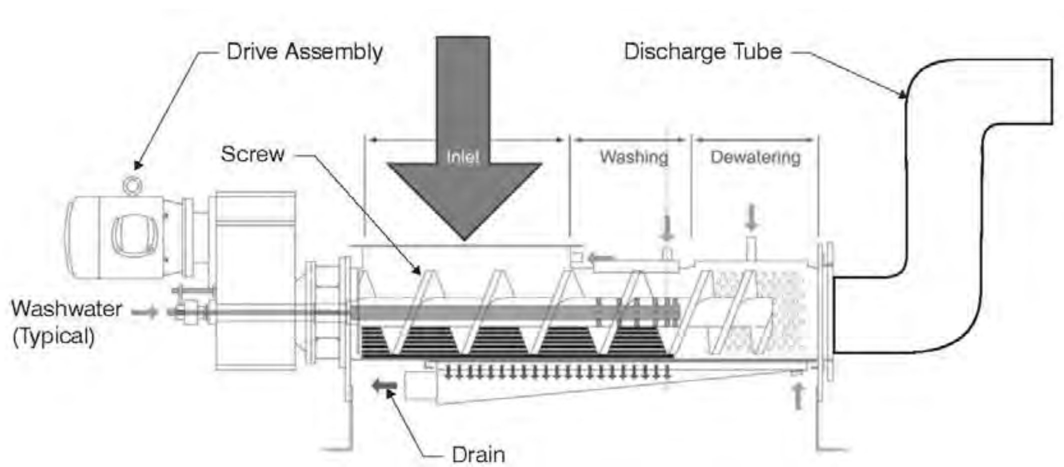


Figure 6.4 Screenings Washer/Compactor

Washer/compactors provide moderate washing with limited breakup of organic material, but can reduce volume up to 70 percent and weight up to 40 percent, decreasing the amount hauled to landfill for disposal. Washer/compactors are a proven and reliable technology, which are available from several manufacturers.

6.3.1.3 Influent Pumping

Two types of pumps were evaluated for the new IPS:

- Submersible non-clog pumps.
- Vertical non-clog centrifugal pumps in a dry pit.

Both types of pumps are designed to handle solids bearing liquids while maintaining relatively high pump efficiencies, typically between 75 and 85 percent at their best efficient point.

Submersible Non-Clog Pumps

Submersible non-clog pumps are a proven technology used in many wastewater treatment applications. Pumps are available in a wide range of sizes and capabilities and are available from several manufacturers.

Submersible non-clog pumps can be installed in either a dry-pit or wet-pit configuration. Installation of submersible pumps in a wet-pit can offer substantial capital cost savings because construction of a dry-pit structure is eliminated along with the associated ventilation, access facilities, and utilities that are required to allow personnel entry.

Vertical Non-Clog Centrifugal Pumps in a Dry Pit

These pumps are a proven technology for this application and have been successfully used at many other wastewater treatment plants. These pumps are also available in a wide range of sizes and capabilities and are available from several manufacturers.

Non-clog centrifugal pumps would be installed in a dry pit connected by drive shafting to motors located above grade, similar to the current layout at the existing IPS. Placing the motors at-grade and above the maximum flood line would assure continued operation of the IPS in the event flooding occurred within the dry pit.

Pump Comparison

In terms of capacity, efficiency, and solids handling capability, both types of pumps are comparable. Table 6.2 provides a comparison between the two pumps.

Table 6.2 Submersible versus Dry Pit-Type Influent Pumps

	Advantages	Disadvantages
Submersible Pumps	• Proven technology. • Requires smaller wet well. • Submersible motor provides reliability if IPS is flooded. • Typically cheaper	• Requires pump removal for service and maintenance. • Fewer submersible pumps on the market.
Dry Pit-Type Pumps	• Proven technology. • Accessible for service and maintenance. • More dry pit pumps available on the market.	• Requires larger structure. • Wet well is inaccessible. • Higher costs associated with larger structure.

Submersible non-clog pumps are recommended at the Scappoose WWTP.

6.3.1.4 Recommended Alternative Design Criteria

The recommended headworks design criteria is presented in Table 6.3.

Table 6.3 Recommended Headworks Design Criteria

Parameter	Value
Screening	
Type	Multi-Rake Bar Screen
Number of Screens	2
Number of Channels	3 (2 duty, 1 passive bypass)
Capacity, Each	5.6 mgd
Screen Opening	1/4in.
Channel Dimensions	
Width	4 ft
Length	30 ft
Depth	15.5 ft
Incline Angle	75 degrees
Screenings Conveyor Type	Shaftless Screw Conveyor
Screenings Washer/Compactor Type	Screw Auger with Spray Water
Influent Pumping	
Type of Pump	Submersible Non-Clog
Number of Structures	1
Firm Capacity	5.6 mgd
Number of Pumps	4
Capacities	Two 1.5-mgd Two 3.0-mgd

6.3.2 Operational Improvements

6.3.2.1 SCADA System

The plant currently has no SCADA system, which makes trending and process monitoring and control difficult. Each of the buildings has its own control panel and motor control centers. The control panel in the WWTP Office controls the influent screen, influent pumps, digester blowers, and sludge transfer pump. It also provides status for all of the pump and motors throughout the plant and displays flow measurements. The autodialer (Mission Control) is also located in this panel. The control panel in the Sludge Pumping Building controls the lagoon aerators, RAS and WAS pumps, scum pump, and clarifiers. The control panel in the Effluent Pump Station controls the effluent pumps and sludge spray irrigation pump. The UV System, Intermediate Pump Station, and the Tertiary Filters each contain their own control panel and motor starters.

The autodialer (Mission Control) provides information on the influent and effluent pump runtimes, influent and effluent flow rates, and alarms for high level in the influent or effluent pump wet well or if the influent pump, effluent pump, UV, clarifier, influent screen, or control power fails. Since the control power is normally backed up with a battery, the operators do not get an alarm when there is a brief power outage.

Under this project, a formal SCADA system would be added to provide process trending and alarms, as required, to the City staff.

6.3.2.2 WWTP Lab

The WWTP lab currently houses both the laboratory equipment, kitchen equipment, and operator offices in the same room. The laboratory equipment should be separate from the food preparation and eating area, as the wastewater samples and chemicals used in the laboratory pose health hazards and should not be ingested. A new 1500 SF lab space to house the laboratory equipment and SCADA system is included in this project.

6.4 Phase 3 Improvements

Phase 3 includes the following improvements:

- UN Disinfection Channel Addition,
- Effluent Pump Station Improvements, and
- Parallel Outfall.

6.4.1 UV Disinfection Channel Addition

The immediate improvement replaces the existing UV system is a Trojan UV3000 model with Trojan UV3000Plus. This provides the required capacity and reliability under existing conditions but does not meet the design year requirements. Additionally the hydraulic modeling indicate that under projected peak flow condition the UV banks will be submerged to the extent that they will not provide sufficient kill to meet the NPDES permit. Therefore, this project adds a second channel similar to the existing channel. The addition will provide adequate reliability, redundancy, and hydraulic capacity through the planning period.

6.4.2 Effluent Pumping Station Improvements

The effluent pump station currently has four (4) 40 hp vertical turbine effluent pumps located in dry well that are the end of their expected lifespan. Each effluent pump is rated for a capacity of 760 gpm at 120 ft. TDH. The firm capacity of the effluent pumps (with one pump out of service) is approximately 2,280 gpm (3.3 MGD), which is less than the current peak instantaneous flow. Additionally, the head required for pumping design year flows is approximately 175 feet higher than the pump design.

An overhead crane is located above the pumps for maintenance and removal. Although there are access ports on the roof, the roof pitch makes it hazardous to unfasten the ports when the weather is poor. This project along with parallel outfall replaces the pumps to provide adequate capacity and redundancy during peak flows for the treatment plant. In addition, the roof access ports are upgraded to make them easier to operate.

6.4.3 Parallel Outfall

Currently, the wastewater is pumped through a 12-inch diameter pipe approximately one mile to the Multnomah Channel. A single-port diffuser under the channel is used to mix the discharged effluent with the channel flow. At the current peak instantaneous flows the velocity in the outfall is 7.9 feet per second (fps), increasing to 11 fps during design year flows. The higher velocity increases head requirement at the effluent pump station and erodes the outfall pipe. Therefore a second 16-inch outfall was recommended. This will ensure that the treatment plant can be easily operated during construction and provide sufficient capacity and redundancy to the system.

6.5 Recommended Site Layout

The recommended site layout is presented in Figure 6.5.

Figure 6.5 Recommended Site Layout

-This Page Intentionally Left Blank-

Appendix A

CURRENT NPDES PERMIT

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM
WASTE DISCHARGE PERMIT

Department of Environmental Quality
Northwest Region – Portland Office
2020 SW 4th Ave., Suite 400, Portland, OR 97201
Telephone: (503) 229-5263

Issued pursuant to ORS 468B.050 and The Federal Clean Water Act

ISSUED TO:

City of Scappoose
33568 East Columbia Avenue
Scappoose, OR 97056

SOURCES COVERED BY THIS PERMIT:

Type of Waste	Outfall Number	Outfall Location
Treated Wastewater	001	R.M. 10.6

FACILITY TYPE AND LOCATION:

Activated Sludge/Extended Aeration
Scappoose STP

34485 East Columbia Avenue
Scappoose OR 97056

RECEIVING STREAM INFORMATION:

Basin: Willamette
Sub-Basin: Lower Willamette

Receiving Stream: Multnomah Channel
LLID: 1227863458618-10.5796 D
County: Columbia

Treatment System Class: Level III
Collection System Class: Level II

EPA REFERENCE NO: OR0022420

This permit is issued in response to Application No. 971614 received June 9, 2009.

This permit is issued based on the land use findings in the permit record.

____ ____________________

Gregory L. Geist, Manager, Water Quality Source Control Section — Date
Northwest Region

PERMITTED ACTIVITIES

Until this permit expires or is modified or revoked, the permittee is authorized to construct, install, modify, or operate a wastewater collection, treatment, control and disposal system and discharge to public waters adequately treated wastewaters only from the authorized discharge point or points established in Schedule A and only in conformance with all the requirements, limitations, and conditions set forth in the attached schedules as follows:

Unless specifically authorized by this permit, by another NPDES or WPCF permit, or by Oregon Administrative Rule, any other direct or indirect discharge of waste is prohibited, including discharge to waters of the state or an underground injection control system.

SCHEDULE-A

1. Waste Discharge Limitations not to be exceeded after permit issuance.

a. Treated Effluent Outfall 001

(1) May 1 - October 31:

Parameter	Average Effluent Concentrations Monthly	Average Effluent Concentrations Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
BOD_5	10 mg/L	15 mg/L	125	190	255
TSS	10 mg/L	15 mg/L	125	190	255

(2) November 1 - April 30:

Parameter	Average Effluent Concentrations Monthly	Average Effluent Concentrations Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
BOD_5	25 mg/L	37 mg/L	315	475	630
TSS	25 mg/L	37 mg/L	315	475	630

* Average dry weather design flow to the facility equals **1.515 MGD**. Mass load limits are based upon average dry weather design flow to the facility.

(3)

Other parameters (year-round)	Limitations
E. coli Bacteria	Shall not exceed 126 organisms per 100 mL monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL. (See Note 1)
pH	Shall be within the range of 6.0 - 9.0
BOD_5 and TSS Removal Efficiency	Shall not be less than 85% monthly average for BOD_5 and 85% monthly for TSS.

(4) Mixing Zone: Except as provided for in OAR 340-045-0080, no wastes shall be discharged and no activities shall be conducted which violate Water Quality Standards, as adopted in OAR 340-041; except in the following defined mixing zone:

> The allowable mixing zone is that portion of Multnomah Channel contained within a band extending out 100 feet from the shore side of the outfall, and 200 feet downstream and 200 feet upstream from the outfall. The Zone of Immediate Dilution (ZID) shall be defined as that portion of the allowable mixing zone that is within a 20 foot radius of the discharge point.

(5) Chlorine: Ultra-Violet Disinfection of effluent is required at this facility. Chlorine and chlorine compounds must not be used as a disinfecting agent of the treated effluent, and no

chlorine residual is allowed in the discharged effluent due to chlorine used for maintenance purposes.

b. Groundwater: No activities shall be conducted that could cause an adverse impact on existing or potential beneficial uses of groundwater. All wastewater and process related residuals shall be managed and disposed in a manner that will prevent a violation of the Groundwater Quality Protection Rules (OAR 340-040).

NOTES:

1. If a single sample exceeds 406 organisms per 100 mL, then five consecutive re-samples may be taken at four-hour intervals beginning within 28 hours after the original sample was taken. If the log mean of the five re-samples is less than or equal to 126 organisms per 100 mL, a violation shall not be triggered.

SCHEDULE-B

1. Minimum Monitoring and Reporting Requirements

The permittee shall monitor the parameters as specified below at the locations indicated. The laboratory used by the permittee to analyze samples shall have a quality assurance/quality control (QA/QC) program to verify the accuracy of sample analysis. If QA/QC requirements are not met for any analysis, the results shall be included in the report, but not used in calculations required by this permit. When possible, the permittee shall re-sample in a timely manner for parameters failing the QA/QC requirements, analyze the samples, and report the results.

a. Influent

The facility influent sampling locations are the following: Influent grab samples and measurements and composite samples are taken at the inlet to the influent Flow Measurement Structure (Parshall flume). All samples for toxics are taken in the same location.

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD)	Daily	Measurement
Flow Meter Calibration	Semi-Annual	Verification
BOD_5	2/Week	Composite
TSS	2/Week	Composite
pH	3/Week	Grab

b. Treated Effluent Outfall 001

The facility effluent sampling locations are the following: Effluent grab samples, measurements, and composite samples are taken at the outlet to the Ultra-Violet (UV) disinfection unit. All samples for toxics are taken at the same location.

Item or Parameter	Minimum Frequency	Type of Sample
BOD_5	2/Week	Composite
TSS	2/Week	Composite
pH	3/Week	Grab
E. coli	2/Week	Grab
Ammonia (Measured as N)	Weekly	Grab
Dissolved Oxygen	Weekly	Measurement
UV Radiation Intensity	Daily	Reading (See Note 1)
Pounds Discharged (BOD_5 and TSS)	2/Week	Calculation of Daily Mass Load
Pounds Discharged (BOD_5 and TSS)	Weekly	Calculation of Weekly Average Mass Load
Pounds Discharged (BOD_5 and TSS)	Monthly	Calculation of Monthly Average Mass Load
Average Percent Removed (BOD_5 and TSS)	Monthly	Calculation

c. Biosolids Management

Item or Parameter	Minimum Frequency	Type of Sample
Biosolids analysis including: Total Solids (% dry wt.) Volatile solids (% dry wt.) Biosolids nitrogen for: NH_3-N; NO_3-N; & TKN (% dry wt.) Phosphorus (% dry wt.) Potassium (% dry wt.) pH (standard units) Biosolids metals content for: As, Cd, Cu, Hg, Mo, Ni, Pb, Se & Zn, measured as total in mg/kg	Annually	Composite sample to be representative of the product to be land applied from the Storage lagoon or pond (See Note 2).
Record of locations where biosolids are applied on each DEQ approved site. (Site location maps to be maintained at treatment facility for review upon request by DEQ)	Each Occurrence	Date, volume, & locations where biosolids were applied recorded on site location map.
Record of % volatile solids reduction accomplished through stabilization	Annually	Calculation (See Note 3).
Fecal coliform per gram total solids (dry weight basis) or Salmonella sp. bacteria per four grams total solids (dry weight basis)	Monthly (when land applying biosolids)	At least seven (7) individual samples representative of the product to be land applied from the storage lagoon or pond (See Note 4).

d. Temperature Monitoring (Monitored only required May 1 - October 31)

Item or Parameter	Minimum Frequency	Type of Sample
Influent Temperature (ºC), Daily Maximum	3/Week	Measurement between 3 and 5 PM.
Effluent Temperature (ºC), Daily Maximum	3/Week	Measurement between 3 and 5 PM.
Multnomah Channel Temperature (ºC), Daily Max	3/Week	Measurement between 3 and 5 PM.
Effluent Temperature (ºC), Average of Daily Maximums	Weekly	Calculation

2. Monitoring Report Procedures

a. Monitoring results must be reported on approved forms. The reporting period is the calendar month. Reports must be submitted to the Department's Northwest Region - Portland office by the 15th day of the following month.

b. State monitoring reports must identify the name, certificate classification and grade level of each principal operator designated by the permittee as responsible for supervising the wastewater collection and treatment systems during the reporting period. Monitoring reports must also identify each system classification as found on page one of this permit.

c. Monitoring reports must include a record of the quantity and method of use of all sewage sludge removed from the treatment facility; and must record all applicable equipment breakdowns and bypassing.

3. Biosolids Report Submittals

Biosolids Report: For any year in which biosolids are land applied, a report shall be submitted to the Department by February 19 of the following year that describes biosolids handling activities for the previous year and includes, but is not limited to, the required information outlined in OAR 340-050-0035(6)(a)-(e).

NOTES:

1. UV Intensity Measurement. The intensity of UV radiation passing through the water column affects the system's ability to kill organisms. To track the reduction in intensity, the UV disinfection system must include a UV intensity meter with a sensor located in the water column at a specified distance from the UV bulbs. This meter will measure the intensity of UV radiation in mWatts-seconds/cm^2. The daily UV radiation intensity shall be determined by reading the meter each day. If more than one meter is used, the daily recording will be an average of all meter readings each day. Intensity meter(s) must be calibrated at a frequency recommended by the manufacturer. The manufacturer's UV intensity curves shall be used to determine when UV bulbs must be replaced or cleaned. Record all daily UV intensity readings in the treatment facility's log book. Record any change of UV bulbs. **Daily UV intensity readings are required for at least 5 days per week**.

2. Biosolids Sampling. Biosolids composite samples from the storage lagoon or pond shall be taken from reference areas in the storage lagoon or pond pursuant to Test Methods for Evaluating Solid Waste, Volume 2; Field Manual, Physical/Chemical Methods, November 1986, Third Edition, Chapter 9.

   Inorganic pollutant monitoring must be conducted according to Test Methods for Evaluating Solid Waste, Physical/Chemical Methods, Second Edition (1982) with Updates I and II and Third Edition (1986) with Revision I.

3. Volatile Solids Reduction. Calculation of the % volatile solids reduction is to be based on comparison of a representative grab sample of total and volatile solids entering each digester (a weighted blend of the primary and secondary clarifier solids) and a representative composite sample of solids exiting each digester withdrawal line.

4. Fecal Coliform Sampling. Analyze and report a fecal coliform result for each sample separately. Calculate and report the geometric mean of all the samples.

5. Temperature Monitoring. After two years of temperature monitoring, and if approved in writing by the Department; monitoring may be waived for those months when the 7-day average of effluent temperature does

not exceed the stream temperature standard of 18.0 °C (64.4 °F). Temperature monitoring results must be reported on the monthly DMR.

SCHEDULE-C

Permit Compliance Conditions

There are no compliance conditions.

SCHEDULE-D

Special Conditions

1. Sewer Cleaning Report: Permittee is required to inspect via television camera and clean 20% of its sanitary sewer system each year. A Sewer Cleaning Report shall be submitted by February 19 of the following year. The Report shall identify the sewer lines cleaned, structural defects noted, and any repairs made to correct identified structural defects.

2. Biosolids Management: All biosolids shall be managed in accordance with the current, DEQ approved Biosolids Management Plan, and the site authorization letters issued by the DEQ. Any changes in biosolids management activities that significantly differ from operations specified under the approved plan require the prior written approval of the DEQ.

   All new biosolids application sites shall meet the site selection criteria set forth in OAR 340-050-0070 and must be located within DEQ approved application sites in Columbia County. All currently approved sites are located adjacent to the wastewater treatment facility on 35 acres, where 28.1 acres are available for actual disposal. No new Public Notice is required for continued use of the currently approved sites; however, a copy of the latest approved Biosolids Management Plan is included for review with renewal permit documentation during the Public Comment Period. Property owners adjacent to any newly approved application sites shall be notified, in writing or by any method approved by DEQ, of the proposed activity prior to the start of application. Proposed new application sites that are deemed by the DEQ to be sensitive with respect to residential housing, runoff potential, or threat to groundwater are subject to public comment in accordance with OAR 340-050-0030.

3. Changes in Biosolids Standards: This permit may be modified to incorporate any applicable standard for biosolids use or disposal promulgated under section 405(d) of the Clean Water Act, if the standard for biosolids use or disposal is more stringent than any requirements for biosolids use or disposal in the permit, or controls a pollutant or practice not limited in this permit.

4. Operator Certification: The permittee shall comply with Oregon Administrative Rules (OAR), Chapter 340, Division 049, "Regulations Pertaining To Certification of Wastewater System Operator Personnel" and accordingly:

   a. The permittee shall have its wastewater system supervised by one or more operators who are certified in a classification and grade level (equal to or greater) that corresponds with the classification (collection and/or treatment) of the system to be supervised as specified on page one of this permit.

      Note: A "supervisor" is defined as the person exercising authority for establishing and executing the specific practice and procedures of operating the system in accordance with the policies of the permittee and requirements of the waste discharge permit. "Supervise" means responsible for the technical operation of a system, which may affect its performance or the quality of the effluent produced. Supervisors are not required to be on-site at all times.

   b. The permittee’s wastewater system may not be without supervision (as required by Special Condition-4.a. above) for more than thirty (30) days. During this period, and at any time that the supervisor is not available to respond on-site (i.e. vacation, sick leave or off-call), the permittee must make available another person who is certified at no less than one grade lower then the system classification.

   c. If the wastewater system has more than one daily shift, the permittee shall have the shift supervisor, if any, certified at no less than one grade lower than the system classification.

d. The permittee is responsible for ensuring the wastewater system has a properly certified supervisor available at all times to respond on-site at the request of the permittee and to any other operator.

e. The permittee shall notify the Department of Environmental Quality in writing within thirty (30) days of replacement or redesignation of certified operators responsible for supervising wastewater system operation. The notice shall be filed with the Water Quality Division, Operator Certification Program, 811 SW 6th Ave, Portland, OR 97204. This requirement is in addition to the reporting requirements contained under Schedule B of this permit.

f. Upon written request, the Department may grant the permittee reasonable time, not to exceed 120 days, to obtain the services of a qualified person to supervise the wastewater system. The written request must include: (1) Justification for the time needed, (2) A schedule for recruiting and hiring, (3) The date the system supervisor’s availability ceased, and (4) The name of the alternate system supervisor(s) as required by 4.b. above.

5. Groundwater: The permittee shall not be required to perform a hydro-geologic characterization or groundwater monitoring during the term of this permit provided:

a. The facilities are operated in accordance with the permit conditions; and

b. There are no adverse groundwater quality impacts (complaints or other indirect evidence) resulting from the facility's operation.

If warranted at permit renewal, the Department may evaluate the need for a full assessment of the facilities impact on groundwater quality.

6. Notification: The permittee shall notify the DEQ Northwest Region - Portland Office (phone: (503) 229-5263) in accordance with the response times noted in the General Conditions of this permit, of any malfunction so that corrective action can be coordinated between the permittee and the Department.

SCHEDULE-E

NOT APPLICABLE

(Pretreatment is not required)

SCHEDULE-F

NPDES GENERAL CONDITIONS – DOMESTIC FACILITIES

(Schedule-F, last update 9.18.2009)

SECTION-A, STANDARD CONDITIONS

1. Duty to Comply with Permit
The permittee must comply with all conditions of this permit. Failure to comply with any permit condition is a violation of Oregon Revised Statutes (ORS) 468B.025 and the federal Clean Water Act and is grounds for an enforcement action. Failure to comply is also grounds for the Department to terminate, modify and reissue, revoke, or deny renewal of a permit.

2. Penalties for Water Pollution and Permit Condition Violations
The permit is enforceable by DEQ or EPA, and in some circumstances also by third-parties under the citizen suit provisions 33 USC §1365. DEQ enforcement is generally based on provisions of state statutes and EQC rules, and EPA enforcement is generally based on provisions of federal statutes and EPA regulations.

ORS 468.140 allows the Department to impose civil penalties up to $10,000 per day for violation of a term, condition or requirement of a permit. The federal Clean Water Act provides for civil penalties not to exceed $32,500 and administrative penalties not to exceed $11,000 per day for each violation of any condition or limitation of this permit.

Under ORS 468.943, unlawful water pollution, if committed by a person with criminal negligence, is punishable by a fine of up to $25,000, imprisonment for not more than one year, or both. Each day on which a violation occurs or continues is a separately punishable offense. The federal Clean Water Act provides for criminal penalties of not more than $50,000 per day of violation, or imprisonment of not more than 2 years, or both for second or subsequent negligent violations of this permit.

Under ORS 468.946, a person who knowingly discharges, places, or causes to be placed any waste into the waters of the state or in a location where the waste is likely to escape into the waters of the state is subject to a Class B felony punishable by a fine not to exceed $200,000 and up to 10 years in prison. The federal Clean Water Act provides for criminal penalties of $5,000 to $50,000 per day of violation, or imprisonment of not more than 3 years, or both for knowing violations of the permit. In the case of a second or subsequent conviction for knowing violation, a person shall be subject to criminal penalties of not more than $100,000 per day of violation, or imprisonment of not more than 6 years, or both.

3. Duty to Mitigate
The permittee must take all reasonable steps to minimize or prevent any discharge or sludge use or disposal in violation of this permit that has a reasonable likelihood of adversely affecting human health or the environment. In addition, upon request of the Department, the permittee must correct any adverse impact on the environment or human health resulting from noncompliance with this permit, including such accelerated or additional monitoring as necessary to determine the nature and impact of the noncomplying discharge.

4. Duty to Reapply

If the permittee wishes to continue an activity regulated by this permit after the expiration date of this permit, the permittee must apply for and have the permit renewed. The application must be submitted at least 180 days before the expiration date of this permit.

The Department may grant permission to submit an application less than 180 days in advance but no later than the permit expiration date.

5. Permit Actions

This permit may be modified, revoked and reissued, or terminated for cause including, but not limited to, the following:

a. Violation of any term, condition, or requirement of this permit, a rule, or a statute;
b. Obtaining this permit by misrepresentation or failure to disclose fully all material facts;
c. A change in any condition that requires either a temporary or permanent reduction or elimination of the authorized discharge;
d. The permittee is identified as a Designated Management Agency or allocated a wasteload under a Total Maximum Daily Load (TMDL);
e. New information or regulations;
f. Modification of compliance schedules;
g. Requirements of permit reopener conditions;
h. Correction of technical mistakes made in determining permit conditions;
i. Determination that the permitted activity endangers human health or the environment;
j. Other causes as specified in 40 CFR 122.62, 122.64, and 124.5;
k. For communities with combined sewer overflows (CSOs);
   (1) To comply with any state or federal law regulation that addresses CSOs that is adopted or promulgated subsequent to the effective date of this permit;
   (2) If new information, not available at the time of permit issuance, indicates that CSO controls imposed under this permit have failed to ensure attainment of water quality standards, including protection of designated uses;
   (3) Resulting from implementation of the Permittee's Long-Term Control Plan and/or permit conditions related to CSOs.

The filing of a request by the permittee for a permit modification, revocation or reissuance, termination, or a notification of planned changes or anticipated noncompliance, does not stay any permit condition.

6. Toxic Pollutants

The permittee must comply with any applicable effluent standards or prohibitions established under Oregon Administrative Rules (OAR) 340-041-0033 and 307(a) of the federal Clean Water Act for toxic pollutants, and with standards for sewage sludge use or disposal established under Section 405(d) of the Clean Water Act, within the time provided in the regulations that establish those standards or prohibitions, even if the permit has not yet been modified to incorporate the requirement.

7. Property Rights and Other Legal Requirements

The issuance of this permit does not convey any property rights of any sort, or any exclusive privilege, or authorize any injury to persons or property or invasion of any other private rights, or any infringement of federal, tribal, state, or local laws or regulations.

8. Permit References

Except for effluent standards or prohibitions established under Section 307(a) of the federal Clean Water Act and OAR 340-041-0033 for toxic pollutants, and standards for sewage sludge use or disposal established under Section 405(d) of the Clean Water Act, all rules and statutes referred to in this permit are those in effect on the date this permit is issued.

9. Permit Fees

The permittee must pay the fees required by Oregon Administrative Rules.

SECTION-B, OPERATION AND MAINTENANCE OF POLLUTION CONTROLS

1. Proper Operation and Maintenance

The permittee must at all times properly operate and maintain all facilities and systems of treatment and control (and related appurtenances) that are installed or used by the permittee to achieve compliance with the conditions of this permit. Proper operation and maintenance also includes adequate laboratory controls and appropriate quality assurance procedures. This provision requires the operation of back-up or auxiliary facilities or similar systems that are installed by a permittee only when the operation is necessary to achieve compliance with the conditions of the permit.

2. Need to Halt or Reduce Activity Not a Defense

For industrial or commercial facilities, upon reduction, loss, or failure of the treatment facility, the permittee must, to the extent necessary to maintain compliance with its permit, control production or all discharges or both until the facility is restored or an alternative method of treatment is provided. This requirement applies, for example, when the primary source of power of the treatment facility fails or is reduced or lost. It is not a defense for a permittee in an enforcement action that it would have been necessary to halt or reduce the permitted activity in order to maintain compliance with the conditions of this permit.

3. Bypass of Treatment Facilities

a. Definitions
   (1) "Bypass" means intentional diversion of waste streams from any portion of the treatment facility. The permittee may allow any bypass to occur which does not cause effluent limitations to be exceeded, provided the diversion is to allow essential maintenance to assure efficient operation. These bypasses are not subject to the provisions of paragraphs b. and c. of this section.
   (2) "Severe property damage" means substantial physical damage to property, damage to the treatment facilities which causes them to become inoperable, or substantial and permanent loss of natural resources that can reasonably be expected to occur in the absence of a bypass. Severe property damage does not mean economic loss caused by delays in production.

b. Prohibition of bypass.
   (1) Bypass is prohibited and the Department may take enforcement action against a permittee for bypass unless:

i. Bypass was unavoidable to prevent loss of life, personal injury, or severe property damage;
ii. There were no feasible alternatives to the bypass, such as the use of auxiliary treatment facilities, retention of untreated wastes, or maintenance during normal periods of equipment downtime. This condition is not satisfied if adequate backup equipment should have been installed in the exercise of reasonable engineering judgment to prevent a bypass that occurred during normal periods of equipment downtime or preventative maintenance; and
iii. The permittee submitted notices and requests as required under General Condition B.3.c.

(2) The Department may approve an anticipated bypass, after considering its adverse effects and any alternatives to bypassing, when the Department determines that it will meet the three conditions listed above in General Condition B.3.b.(1).

c. Notice and request for bypass.
(1) Anticipated bypass. If the permittee knows in advance of the need for a bypass, a written notice must be submitted to the Department at least ten days before the date of the bypass.
(2) Unanticipated bypass. The permittee must submit notice of an unanticipated bypass as required in General Condition D.5.

4. <u>Upset</u>

a. Definition. "Upset" means an exceptional incident in which there is unintentional and temporary noncompliance with technology based permit effluent limitations because of factors beyond the reasonable control of the permittee. An upset does not include noncompliance to the extent caused by operation error, improperly designed treatment facilities, inadequate treatment facilities, lack of preventative maintenance, or careless or improper operation.

b. Effect of an upset. An upset constitutes an affirmative defense to an action brought for noncompliance with such technology-based permit effluent limitations if the requirements of General Condition B.4.c are met. No determination made during administrative review of claims that noncompliance was caused by upset, and before an action for noncompliance, is final administrative action subject to judicial review.

c. Conditions necessary for a demonstration of upset. A permittee who wishes to establish the affirmative defense of upset must demonstrate, through properly signed, contemporaneous operating logs, or other relevant evidence that:
(1) An upset occurred and that the permittee can identify the causes(s) of the upset;
(2) The permitted facility was at the time being properly operated;
(3) The permittee submitted notice of the upset as required in General Condition D.5, hereof (24-hour notice); and
(4) The permittee complied with any remedial measures required under General Condition A.3 hereof.

d. Burden of proof. In any enforcement proceeding the permittee seeking to establish the occurrence of an upset has the burden of proof.

5. <u>Treatment of Single Operational Upset</u>

For purposes of this permit, A Single Operational Upset that leads to simultaneous violations of more than one pollutant parameter will be treated as a single violation. A single operational upset is an exceptional incident that causes simultaneous, unintentional, unknowing (not the result of a knowing act or omission), temporary noncompliance with more than one Clean Water Act effluent discharge pollutant parameter. A single operational upset does not include Clean Water Act violations involving discharge without a NPDES permit or noncompliance to the extent caused by improperly designed or inadequate treatment facilities. Each day of a single operational upset is a violation.

6. Overflows from Wastewater Conveyance Systems and Associated Pump Stations
   a. Definitions
      (1) "Overflow" means any spill, release or diversion of sewage including:
         i. An overflow that results in a discharge to waters of the United States; and
         ii. An overflow of wastewater, including a wastewater backup into a building (other than a backup caused solely by a blockage or other malfunction in a privately owned sewer or building lateral), even if that overflow does not reach waters of the United States.
   b. Prohibition of overflows. Overflows are prohibited. The Department may exercise enforcement discretion regarding overflow events. In exercising its enforcement discretion, the Department may consider various factors, including the adequacy of the conveyance system's capacity and the magnitude, duration and return frequency of storm events.
   c. Reporting required. All overflows must be reported orally to the Department within 24 hours from the time the permittee becomes aware of the overflow. Reporting procedures are described in more detail in General Condition D.5.

7. Public Notification of Effluent Violation or Overflow

If effluent limitations specified in this permit are exceeded or an overflow occurs that threatens public health, the permittee must take such steps as are necessary to alert the public, health agencies and other affected entities (e.g., public water systems) about the extent and nature of the discharge in accordance with the notification procedures developed under General Condition B.8. Such steps may include, but are not limited to, posting of the river at access points and other places, news releases, and paid announcements on radio and television.

8. Emergency Response and Public Notification Plan

The permittee must develop and implement an emergency response and public notification plan that identifies measures to protect public health from overflows, bypasses or upsets that may endanger public health. At a minimum the plan must include mechanisms to:

a. Ensure that the permittee is aware (to the greatest extent possible) of such events;
b. Ensure notification of appropriate personnel and ensure that they are immediately dispatched for investigation and response;
c. Ensure immediate notification to the public, health agencies, and other affected public entities (including public water systems). The overflow response plan must identify the public health and other officials who will receive immediate notification;
d. Ensure that appropriate personnel are aware of and follow the plan and are appropriately trained;
e. Provide emergency operations; and
f. Ensure that DEQ is notified of the public notification steps taken.

9. Removed Substances

Solids, sludges, filter backwash, or other pollutants removed in the course of treatment or control of wastewaters must be disposed of in such a manner as to prevent any pollutant from such materials from entering waters of the state, causing nuisance conditions, or creating a public health hazard.

SECTION-C, MONITORING AND RECORDS

1. Representative Sampling

Sampling and measurements taken as required herein shall be representative of the volume and nature of the monitored discharge. All samples must be taken at the monitoring points specified in this permit, and shall be taken, unless otherwise specified, before the effluent joins or is diluted by any other waste stream, body of water, or substance. Monitoring points may not be changed without notification to and the approval of the Department.

2. Flow Measurements

Appropriate flow measurement devices and methods consistent with accepted scientific practices must be selected and used to ensure the accuracy and reliability of measurements of the volume of monitored discharges. The devices must be installed, calibrated and maintained to insure that the accuracy of the measurements is consistent with the accepted capability of that type of device. Devices selected must be capable of measuring flows with a maximum deviation of less than ± 10 percent from true discharge rates throughout the range of expected discharge volumes.

3. Monitoring Procedures

Monitoring must be conducted according to test procedures approved under 40 CFR part 136, or in the case of sludge use and disposal, under 40 CFR part 503, unless other test procedures have been specified in this permit.

4. Penalties of Tampering

The Clean Water Act provides that any person who falsifies, tampers with, or knowingly renders inaccurate any monitoring device or method required to be maintained under this permit may, upon conviction, be punished by a fine of not more than $10,000 per violation, imprisonment for not more than two years, or both. If a conviction of a person is for a violation committed after a first conviction of such person, punishment is a fine not more than $20,000 per day of violation, or by imprisonment of not more than four years, or both.

5. Reporting of Monitoring Results

Monitoring results must be summarized each month on a Discharge Monitoring Report form approved by the Department. The reports must be submitted monthly and are to be mailed, delivered or otherwise transmitted by the 15th day of the following month unless specifically approved otherwise in Schedule B of this permit.

6. Additional Monitoring by the Permittee

If the permittee monitors any pollutant more frequently than required by this permit, using test procedures approved under 40 CFR part 136, or in the case of sludge use and disposal, under 40 CFR part 503, or as specified in this permit, the results of this monitoring must be included in the calculation and reporting of the data submitted in the Discharge Monitoring Report. Such increased frequency must also be indicated. For a pollutant parameter that may be sampled more than once per day (e.g., Total Chlorine Residual), only the average daily value must be recorded unless otherwise specified in this permit.

7. Averaging of Measurements

Calculations for all limitations that require averaging of measurements must utilize an arithmetic mean, except for bacteria which shall be averaged as specified in this permit.

8. Retention of Records

Records of monitoring information required by this permit related to the permittee's sewage sludge use and disposal activities shall be retained for a period of at least five years (or longer as required by 40 CFR part 503). Records of all monitoring information including all calibration and maintenance records, all original strip chart recordings for continuous monitoring instrumentation, copies of all reports required by this permit and records of all data used to complete the application for this permit shall be retained for a period of at least 3 years from the date of the sample, measurement, report, or application. This period may be extended by request of the Department at any time.

9. Records Contents

Records of monitoring information must include:

a. The date, exact place, time, and methods of sampling or measurements;
b. The individual(s) who performed the sampling or measurements;
c. The date(s) analyses were performed;
d. The individual(s) who performed the analyses;
e. The analytical techniques or methods used; and
f. The results of such analyses.

10. Inspection and Entry

The permittee must allow the Department or EPA upon the presentation of credentials to:

a. Enter upon the permittee's premises where a regulated facility or activity is located or conducted, or where records must be kept under the conditions of this permit;
b. Have access to and copy, at reasonable times, any records that must be kept under the conditions of this permit;
c. Inspect at reasonable times any facilities, equipment (including monitoring and control equipment), practices, or operations regulated or required under this permit, and
d. Sample or monitor at reasonable times, for the purpose of assuring permit compliance or as otherwise authorized by state law, any substances or parameters at any location.

11. Confidentiality of Information

Any information relating to this permit that is submitted to or obtained by DEQ is available to the public unless classified as confidential by the Director of DEQ under ORS 468.095. The Permittee may request that information be classified as confidential if it is a trade secret as defined by that statute. The name and address of the permittee, permit applications, permits, effluent data, and information required by NPDES application forms under 40 CFR 122.21 will not be classified as confidential (40 CFR 122.7(b)).

SECTION-D, REPORTING REQUIREMENTS

1. Planned Changes

The permittee must comply with OAR chapter 340, division 52, "Review of Plans and Specifications" and 40 CFR Section 122.41(l) (1). Except where exempted under OAR chapter 340, division 52, no construction, installation, or modification involving disposal systems, treatment works, sewerage systems, or common sewers may be commenced until the plans and specifications are submitted to and approved by the Department. The permittee must give notice to the Department as soon as possible of any planned physical alternations or additions to the permitted facility.

2. Anticipated Noncompliance
The permittee must give advance notice to the Department of any planned changes in the permitted facility or activity that may result in noncompliance with permit requirements.

3. Transfers
This permit may be transferred to a new permittee provided the transferee acquires a property interest in the permitted activity and agrees in writing to fully comply with all the terms and conditions of the permit and the rules of the Commission. No permit may be transferred to a third party without prior written approval from the Department. The Department may require modification, revocation, and reissuance of the permit to change the name of the permittee and incorporate such other requirements as may be necessary under 40 CFR Section 122.61. The permittee must notify the Department when a transfer of property interest takes place.

4. Compliance Schedule
Reports of compliance or noncompliance with, or any progress reports on interim and final requirements contained in any compliance schedule of this permit must be submitted no later than 14 days following each schedule date. Any reports of noncompliance must include the cause of noncompliance, any remedial actions taken, and the probability of meeting the next scheduled requirements.

5. Twenty-Four Hour Reporting
The permittee must report any noncompliance that may endanger health or the environment. Any information must be provided orally (by telephone) to DEQ or to the Oregon Emergency Response System (1-800-452-0311) as specified below within 24 hours from the time the permittee becomes aware of the circumstances.

a. Overflows.

(1) Oral Reporting within 24 hours.

i. For overflows other than basement backups, the following information must be reported to the Oregon Emergency Response System (OERS) at 1-800-452-0311. For basement backups, this information should be reported directly to DEQ.

a) The location of the overflow;
b) The receiving water (if there is one);
c) An estimate of the volume of the overflow;
d) A description of the sewer system component from which the release occurred (e.g., manhole, constructed overflow pipe, crack in pipe); and
e) The estimated date and time when the overflow began and stopped or will be stopped.

ii. The following information must be reported to the Department's Regional office within 24 hours, or during normal business hours, whichever is first:

a) The OERS incident number (if applicable) along with a brief description of the event.

(2) Written reporting within 5 days.

i. The following information must be provided in writing to the Department's Regional office within 5 days of the time the permittee becomes aware of the overflow:
   a) The OERS incident number (if applicable);
   b) The cause or suspected cause of the overflow;
   c) Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the overflow and a schedule of major milestones for those steps;
   d) Steps taken or planned to mitigate the impact(s) of the overflow and a schedule of major milestones for those steps; and
   e) (for storm-related overflows) The rainfall intensity (inches/hour) and duration of the storm associated with the overflow.

The Department may waive the written report on a case-by-case basis if the oral report has been received within 24 hours.

b. Other instances of noncompliance.
   (1) The following instances of noncompliance must be reported:
      i. Any unanticipated bypass that exceeds any effluent limitation in this permit;
      ii. Any upset that exceeds any effluent limitation in this permit;
      iii. Violation of maximum daily discharge limitation for any of the pollutants listed by the Department in this permit; and
      iv. Any noncompliance that may endanger human health or the environment.
   (2) During normal business hours, the Department's Regional office must be called. Outside of normal business hours, the Department must be contacted at **1-800-452-0311** (Oregon Emergency Response System).
   (3) A written submission must be provided within 5 days of the time the permittee becomes aware of the circumstances. The written submission must contain:
      i. A description of the noncompliance and its cause;
      ii. The period of noncompliance, including exact dates and times;
      iii. The estimated time noncompliance is expected to continue if it has not been corrected;
      iv. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance; and
      v. Public notification steps taken, pursuant to General Condition B.**7**
   (4) The Department may waive the written report on a case-by-case basis if the oral report has been received within 24 hours.

6. Other Noncompliance

The permittee must report all instances of noncompliance not reported under General Condition D.4 or D.5, at the time monitoring reports are submitted. The reports must contain:

a. A description of the noncompliance and its cause;
b. The period of noncompliance, including exact dates and times;
c. The estimated time noncompliance is expected to continue if it has not been corrected; and
d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance.

7. Duty to Provide Information

The permittee must furnish to the Department within a reasonable time any information that the Department may request to determine compliance with the permit or to determine whether cause exists for modifying, revoking and reissuing, or terminating this permit. The permittee must also furnish to the Department, upon request, copies of records required to be kept by this permit.

Other Information: When the permittee becomes aware that it has failed to submit any relevant facts or has submitted incorrect information in a permit application or any report to the Department, it must promptly submit such facts or information.

8. Signatory Requirements
All applications, reports or information submitted to the Department must be signed and certified in accordance with 40 CFR Section 122.22.

9. Falsification of Information
Under ORS 468.953, any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit, including monitoring reports or reports of compliance or noncompliance, is subject to a Class C felony punishable by a fine not to exceed $100,000 per violation and up to 5 years in prison. Additionally, according to 40 CFR 122.41(k)(2), any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit including monitoring reports or reports of compliance or non-compliance shall, upon conviction, be punished by a federal civil penalty not to exceed $10,000 per violation, or by imprisonment for not more than 6 months per violation, or by both.

10. Changes to Indirect Dischargers
The permittee must provide adequate notice to the Department of the following:
a. Any new introduction of pollutants into the POTW from an indirect discharger which would be subject to section 301 or 306 of the Clean Water Act if it were directly discharging those pollutants and;
b. Any substantial change in the volume or character of pollutants being introduced into the POTW by a source introducing pollutants into the POTW at the time of issuance of the permit.
c. For the purposes of this paragraph, adequate notice shall include information on (i) the quality and quantity of effluent introduced into the POTW, and (ii) any anticipated impact of the change on the quantity or quality of effluent to be discharged from the POTW.

SECTION-E, DEFINITIONS

1. *BOD* means five-day biochemical oxygen demand.
2. *CBOD* means five day carbonaceous biochemical oxygen demand
3. *TSS* means total suspended solids.
4. "*Bacteria*" includes but is not limited to fecal coliform bacteria, total coliform bacteria, and E. coli bacteria.
5. *FC* means fecal coliform bacteria.
6. *Total residual chlorine* means combined chlorine forms plus free residual chlorine
7. *Technology based permit effluent limitations* means technology-based treatment requirements as defined in 40 CFR Section 125.3, and concentration and mass load effluent limitations that are based on minimum design criteria specified in OAR Chapter 340, Division 41.

8. *mg/l* means milligrams per liter.
9. *kg* means kilograms.
10. *m^3/d* means cubic meters per day.
11. *MGD* means million gallons per day.
12. 24-hour *Composite sample* means a sample formed by collecting and mixing discrete samples taken periodically and based on time or flow. The sample must be collected and stored in accordance with 40 CFR Part 136.
13. *Grab sample* means an individual discrete sample collected over a period of time not to exceed 15 minutes.
14. *Quarter* means January through March, April through June, July through September, or October through December.
15. *Month* means calendar month.
16. *Week* means a calendar week of Sunday through Saturday.
17. *POTW* means a publicly owned treatment works.

(Schedule-F, last update 9.18.2009)

GLS: Scappoose PermitDoc 20Apr2010.docx
Revised: May 3, 2010

Appendix B
COST ESTIMATES

Spring Lake Lift Station						
SPEC. NO.	**DESCRIPTION**	**QUANTITY**	**UNIT**	**UNIT COST**	**SUBTOTAL**	**TOTAL**
Pump Station						
	Division 02 - Site Construction					
02000	Demolition of Existing Lift Station	1	LS	$5,000.00	$5,000	
02900	Landscaping, Medium Visual Impact, Medium Areas	1	LS	$5,000.00	$5,000	
	Total					**$10,000**
	Division 11 - Equipment					
11312	5Hp Submersible Sump Pump	2	EA	$15,000.00	$30,000	
	Total					**$30,000**
	Division 15 - Mechanical					
15251	4" Flg Cldi Pipe In Bldg	60	LF	$44.89	$2,693	
	Total					**$2,693**
Valve Vault						
	Division 02 - Site Construction					
02000	Maintenance Pad	1	EA	$10,000.00	$10,000	
	Total					**$10,000**
	Division 03 - Concrete					
03000	Ladder and Hatches	1	EA	$15,000.00	$15,000	
	Total					**$15,000**
	Division 15 - Mechanical					
15000	Allowance	1	LS	$10,000.00	$10,000	
15000	Hose Bib	1	EA	$5,000.00	$5,000	
15114	4"- 200 Psi Ci Fxf Swing Check Valve	2	EA	$1,207.19	$2,414	
15116	4" 150# Flanged Cast Steel Plug Valve	2	EA	$1,161.16	$2,322	
15119	Air/Vac Valve, 150# Flange, 4"	2	EA	$2,357.76	$4,716	
15251	4" Flg Cldi Pipe In Open Trench	40	LF	$37.11	$1,485	
	Total					**$25,937**
16000/17000	**Electrical EIC Allowance**	1	LS	$30,000.00	$30,000	
						$38,522
	Grand Total					**$132,152**
	Mobilization/Demobilization @ 10%	10%				$13,215
	Subtotal					$145,367.74
	Contractor Overhead and Profit at 15%	15%				$21,805
	Subtotal					$167,172.90
	Contingencies @ 30%	30%				$50,152
	Subtotal					$217,325
	ELA @ 25%	25%				$54,331
	Total Estimated Project Cost					**$271,656**

UV Disinfection Equipment Replacement

SPEC. NO.	DESCRIPTION	QUANTITY	UNIT	UNIT COST	SUBTOTAL	TOTAL
	Division 02 - Site Construction					
02000	UV System Demolition Adder	1.00	LS	$10,000.00	$10,000	
02300	Disposal	1.00	LS	$15,000.00	$15,000	
	Total					**$25,000**
	Division 11 - Equipment					
11000	UV Disinfection System	1	EA	$200,000.00	$200,000	
	Total					**$200,000**
	Division 16 - Electrical					
16000	Electrical and Instrumentation Work	1	LS	$75,000.00	$75,000	
	Total					**$75,000**
	Grand Total					**$300,000**
	Mobilization/Demobilization @ 10%	10%				$30,000
	Subtotal					$330,000.00
	Contractor Overhead and Profit at 15%	15%				$49,500
	Subtotal					$379,500.00
	Contingencies @ 30%	30%				$113,850
	Subtotal					$493,350
	ELA @ 25%	25%				$123,338
	Total Estimated Project Cost					**$616,688**

Hydraulic Improvements						
SPEC. NO.	**DESCRIPTION**	**QUANTITY**	**UNIT**	**UNIT COST**	**SUBTOTAL**	**TOTAL**
	Division 02 - Site Construction					
02000	Demolition of Existing Splitter Box	1	LS	$20,000.00	$20,000	
02000	Cat 235 Trackhoe 1.50CY Bucket, Class C (Hard Digging)	977	CY	$3.88	$3,789	
02000	Dispose Spoils - 20 CY Dump Truck, 30 Miles/Round Trip	1009	CY	$10.88	$10,978	
02000	ABC	13	CY	$74.00	$962	
02000	Confined Structure Backfill - Import Material	679	CY	$58.02	$39,395	
02000	Drain Rock	81	CY	$50.00	$4,050	
02000	Stabilization Fabric	1271	SF	$1.00	$1,271	
	Total					**$80,445**
	Division 03 - Concrete					
03000	12" STRUCTURAL FLAT MAT ON GRADE	32	CY	$527.69	$16,886	
03000	12" EDGE FORMS, SLAB ON GRADE, ADD	170	LF	$11.93	$2,027	
03000	12" STRAIGHT WALL >8' HIGH	77	CY	$965.18	$74,319	
03000	12" ELEVATED SLAB	2.3	CY	$657.71	$1,513	
03000	CLASS 'C' CONCRETE	13	CY	$500.00	$6,500	
	Total					**$101,245**
	Division 11 - Equipment					
11000	Downward Opening SST Weir Gates - Manual Operator	2	EA	$10,560.00	$21,120	
	Total					**$21,120**
	Division 15 - Mechanical					
15000	Piping Allowance	1	LS	$50,000.00	$50,000	
	Total					**$50,000**
	Grand Total					**$252,810**
	Mobilization/Demobilization @ 10%	10%				$25,281
	Subtotal					$278,090.51
	Contractor Overhead and Profit at 15%	15%				$41,714
	Subtotal					$319,804.09
	Contingencies @ 30%	30%				$95,941
	Subtotal					$415,745
	ELA @ 25%	25%				$103,936
	Total Estimated Project Cost					**$519,682**

Secondary Clarifier

SPEC. NO.	DESCRIPTION	QUANTITY	UNIT	UNIT COST	SUBTOTAL	DIVISION TOTAL
02220	Excavation	14908	CY	$ 10	$149,081	
02220	Excavation - Disposal	7454	CY	$ 8	$59,632	
02220	12" ABC under slab	391	CY	$ 40	$15,657	
02220	Structural Backfill	7869	CY	$ 50	$393,427	
02220	Piles					
02220	Pile Mob	1	EA	$ 50,000	$50,000	
						$667,797
03300	External Walls	53	CY	$ 1,000	$53,315	
03000	FLOOR GROUT	58	CY	$ 400	$23,312	
03000	STAIRS	1	LS	$ 25,000	$16,750	
03300	CLARIFIER FOOTING	102	CY	$ 600	$61,282	
03300	CLARIFIER FLOOR SLAB	300	CY	$ 700	$210,269	
03300	CENTER PIER	9	CY	$ 1,800	$17,067	
03300	Concrete Forms	1169	CY	$ 50	$58,450	
03300	SIDEWALL	290	CY	$ 1,000	$290,117	
03300	LAUNDER SLAB	32	CY	$ 1,800	$57,573	
03300	LAUNDER WALL	26	CY	$ 1,400	$35,855	
03300	EXTERNAL WALLS	28	CY	$ 1,000	$27,704	
03300	ELEVATED SLAB	0.4	CY	$ 1,800	$667	
03300	30" FLOOR SLAB	300	CY	$ 700	$210,269	
03300	EXTERNAL WALLS	28	CY	$ 1,000	$27,704	
						$1,090,334
05500	Aluminum Handrail	300	LF	$ 76	$22,800	
05500	Alum Stairs	40	Risers	$ 560	$22,400	
05500	1 1/2" Galv Grating	300	SQ FT	$ 42	$12,600	
						$57,800
06000	FRP WEIR PLATE	173	SF	$ 30	$5,184	
06000	FRP SCUM BAFFLE	1728	SF	$ 30	$51,836	
	Total					$57,020
	Basin					
11381	Clarifier Mechanism	1	EA	$ 400,000	$400,000	
Div 15	Piping Allowance	1	LS	$ 100,000	$100,000	
Div 15	Valve Allowance	1	LS	$ 60,000	$60,000	
Div 15	RAS/WAS Pumping Allowance	1	LS	$ 200,000	$200,000	
						$360,000

Div 16	Electrical Allowance	15%	LS	$ 2,232,951	$334,943	
Div 17	Instrumentation Allowance	1	EA	$ 50,000	$50,000	
	Grand Total					**$2,232,951**
	Mobilization/Demobilization @ 10%	10%				$223,295
	Subtotal					$2,456,245.68
	Contractor Overhead and Profit at 15%	15%				$368,437
	Subtotal					$2,824,682.54
	Contingencies @ 30%	30%				$847,405
	Subtotal					$3,672,087
	ELA @ 25%	25%				$918,022
	Total Estimated Project Cost					**$4,590,109**

Aerobic Digester Life Safety

SPEC. NO.	DESCRIPTION	QUANTITY	UNIT	UNIT COST	SUBTOTAL	TOTAL
	Division 02 - Site Construction					
02000	Demolition of existing diffusers, blowers	1	LS	$10,000.00	$10,000	
	Railing fixing	1	LS	$20,000.00	$20,000	
	Total					**$30,000**
	Division 11 - Equipment					
11000	Replace coarse bubble diffusers	1	LS	$35,000.00	$35,000	
11000	Repalce 25HP blowers	2	EA	$40,000.00	$80,000	
11000	Sludge Pump	2	EA	$15,000.00	$30,000	
	Total					**$145,000**
	Division 15 - Mechanical					
15000	Replace sludge flow meter	1	EA	$8,000.00	$8,000	
	Total					**$8,000**
16000	**Electrical EIC Allowance**	15%	LS	$183,000.00	$27,450	
						$27,450
	Grand Total					**$210,450**
	Mobilization/Demobilization @ 10%	10%				$21,045
	Subtotal					$231,495.00
	Contractor Overhead and Profit at 15%	15%				$34,724
	Subtotal					$266,219.25
	Contingencies @ 30%	30%				$79,866
	Subtotal					$346,085
	ELA @ 25%	25%				$86,521
	Total Estimated Project Cost					**$432,606**

Headworks and Influent Pumping Station

SPEC. NO.	DESCRIPTION	QUANTITY	UNIT	UNIT COST	SUBTOTAL	TOTAL
	Division 02 - Site Construction					
02000	Piles					
02000	Pile Mobs					
02000	Excavation	2167	CY	$ 10	$ 21,674	
02000	Soil offsite disposal	2384	CY	$ 8	$ 19,073	
02000	12" ABC Under Slab	223	CY	$ 40	$ 8,920	
0'2000	Demolish Existing headworks	1	LS	$ 150,000	$ 150,000	
0'2000	Refurbish existing headworks	1	LS	$ 500,000	$ 500,000	
02000	Structural Backfill	1084	CY	$ 50	$ 54,185	
	Total					$ 753,853
	Division 03 - Concrete					
03300	Forms	591	CY	$ 50	$ 29,550	
03300	18" Straight Wall >8' High	55	CY	$ 1,200	$ 66,132	
03300	30" Straight Wall >8' High	114	CY	$ 1,000	$ 113,670	
03300	18" Straight Wall >8' High	18	CY	$ 1,200	$ 21,696	
03300	18" Straight Wall >8' High	14	CY	$ 1,200	$ 16,536	
03300	18" Straight Wall >8' High	9	CY	$ 1,200	$ 10,332	
03300	24" Structural Flat Mat On Grade	72	CY	$ 800	$ 57,248	
03300	18" Straight Wall >8' High	27	CY	$ 1,200	$ 32,004	
03300	18" Straight Wall >8' High	61	CY	$ 1,200	$ 73,788	
03300	12" Straight Wall >8' High	9	CY	$ 1,200	$ 10,668	
03300	12" Straight Wall, To 8' High	4	CY	$ 1,200	$ 4,800	
03300	24" Structural Flat Mat On Grade	37	CY	$ 800		
03300	18" Straight Wall, To 8' High	14	CY	$ 1,200		
03300	18" Straight Wall, To 8' High	1	CY	$ 1,200		
03300	12" Flat Non-Formed S.O.G.	13	CY	$ 600		
03300	18" Straight Wall >8' High	5	CY	$ 1,200		
03300	18" Straight Wall, To 8' High	3	CY	$ 1,200		
03300	18" Straight Wall, To 8' High	3	CY	$ 1,200		
03300	12" Curved Wall, 16'-30' Dia, To 8' High	19	CY	$ 1,400		
03300	12" Curved Wall, 7' Dia., To 8' High	5	CY	$ 1,400		
03300	12" Structural Flat Mat On Grade	9	CY	$ 600	$ 5,118	
03300	24" Structural Flat Mat On Grade	22	CY	$ 600	$ 13,332	
03300	24" Structural Flat Mat On Grade	3	CY	$ 600	$ 1,710	
03300	24" Structural Flat Mat On Grade	74	CY	$ 600	$ 44,442	
	Total					$ 501,026
	Division 05 - Metals					
05000	Miscellaneous Metals Allowance	1	LS	$ 50,000	$ 50,000	
	Total					$ 50,000
	Division 13 - Building					
130000	Electrical Room Allowance	1	LS	$ 60,000	$ 60,000	
	Total					$ 60,000
	Division 09 - Finishes					
09000	PVC Lining Allowance	1	LS	$ 80,000	$ 80,000	
	Total					$ 80,000

	Division 11 - Equipment					
11000	Multi-Rake Screen	2	EA	$ 195,000	$ 390,000	
11000	Screenings Washer/Compactor	1	EA	$ 65,000	$ 65,000	
11000	Induced Vortex Grit Removal System	1	EA	$ 58,695		
11000	Manual Bar Screen	1	EA	$ 3,000		
11000	Recessed Impeller Pump	2	EA	$ 39,000		
11000	Grit Cyclone/Classifier	2	EA	$ 75,400		
11293	Slide Gate, 84" X 72"	1	EA	$ 21,190		
11293	Slide Gate, 48" X 48"	3	EA	$ 16,085		
11293	Slide Gate, 36" X 36"	6	EA	$ 11,063		
11294	Add For Motor Operator To 96"	1	EA	$ 5,781		
11294	Add For Motor Operator To 48"	8	EA	$ 4,225		
11312	20Hp Submersible Pump	4	EA	$ 33,269	$ 133,077	
	Total					$ 588,077
	Division 14 - Conveying Systems					
14555	Shaftless Screw Conveyor	1	EA	$ 98,470	$ 98,470	
	Total					$ 98,470
	Division 15 - Mechanical					
15000	Miscellaneous Mechanical	1	LS	$ 100,000	$ 100,000	
	Total					$ 100,000
	Division 16 - Electrical					
16000	EI&C Allowance	0.2	LS	$ 2,231,425.50	$ 446,285	
	Total					$ 446,285
	Grand Total					**$2,677,711**
	Mobilization/Demobilization @ 10%	10%				$267,771
	Subtotal					$2,945,481.66
	Contractor Overhead and Profit at 15%	15%				$441,822
	Subtotal					$3,387,303.91
	Contingencies @ 30%	30%				$1,016,191
	Subtotal					$4,403,495
	ELA @ 25%	25%				$1,100,874
	Total Estimated Project Cost					**$5,504,369**

UV Disinfection Channel Addition

SPEC. NO.	DESCRIPTION	QUANTITY	UNIT	UNIT COST	SUBTOTAL	TOTAL
	Division 02 - Site Construction					
02220	Demo Concrete Walls, Heavy Rebar, 12"	1418.3333	SF	$24.00	$34,046	
02220	Cut Concrete Walls	12288	INFT	$1.99	$24,422	
02220	Demo Concrete Walls, Heavy Rebar, 12"	180	SF	$24.00	$4,321	
02220	Cut Concrete Walls	1152	INFT	$1.99	$2,290	
02300	Controlled Density Fill (Cdf)	177.0825	CY	$91.26	$16,161	
02300	Cat 225 Trackhoe, 1Cy Bucket, Class B (Medium Digging), 0-16' D	180.555	CY	$4.46	$806	
02300	Imported Pipe Bed & Zone/Confined Structure Backfill, Class A Material	144.44	CY	$60.41	$8,725	
	Total					**$90,769**
	Division 03 - Concrete					
03000	Connect Concrete Columns to Existing Slab	21	EA	$598.26	$12,563	
03000	Repair Concrete Wall	1	LS	$1,216.80	$1,217	
03300	12" Straight Wall >8' High	1.78	CY	$847.76	$1,509	
03300	12" Straight Wall >8' High	4.44	CY	$847.76	$3,764	
03300	18" Straight Wall, To 8' High	18.42	CY	$653.74	$12,042	
03300	12" Straight Wall, To 8' High	20.78	CY	$841.03	$17,477	
03300	8" Edge Forms Slabs On Grade, Add	164	LF	$5.56	$912	
03300	8" Flat Non-Formed S.O.G.	27.92	CY	$310.71	$8,675	
03300	12" W X 18" D Conc Beam	2.83	CY	$1,181.91	$3,345	
03300	12" Sloped S.O.G. Edge Forms (To 30%), Add	275	LF	$10.67	$2,935	
03300	12" Sloped Slab On Grade (To 30%)	35.801667	CY	$347.18	$12,430	
03300	12" Edge Forms, Slab On Grade, Add	35	LF	$9.49	$332	
03300	12" Flat Non-Formed S.O.G.	2.78	CY	$262.58	$730	
03300	12" Straight Wall >8' High	7.33	CY	$847.76	$6,214	
03300	20" W X 34" D Conc Beam	5.69	CY	$803.68	$4,573	
03300	24" W X 54" D Conc Beam	4.41	CY	$689.40	$3,040	
03300	48" Edge Forms, Slab On Grade, Add	32	LF	$36.98	$1,183	
03300	48" Structural Flat Mat On Grade	47.125	CY	$256.80	$12,102	
03300	12" Straight Wall, To 8' High	8.43	CY	$841.03	$7,090	
	Total					**$112,133**
	Division 05 - Metals					
05000	SST Vertical Baffle Plate	1	EA	$3,062.28	$3,062	
05000	Alum Tread Plate	493.5	SF	$31.64	$15,613	
05000	SST Structural Steel Shapes and Plates	360	LB	$7.79	$2,804	
05140	Structural Aluminum Shapes & Plates - Gc	219.45	LB	$3.54	$777	
05500	Aluminum Osha Handrail	276	LF	$74.11	$20,454	
05500	Aluminum Grating, 1-1/2"	367.5	SF	$52.84	$19,418	
	Total					**$62,127**
	Division 11 - Equipment					
11000	UV Banks	1	LS	$200,000.00	$200,000	
11293	Slide Gate	2	EA	$20,000.00	$40,000	
11294	Add For Motor Operator	2	EA	$5,862.42	$11,725	
	Total					**$251,725**

	Division 14 - Conveying Systems					
14631	3 Ton 40' Span Floor Operated Overhead Traveling Bridge Crane	1	EA	$50,000.00	$50,000	
	Total					**$50,000**
	Division 15 - Mechanical					
15000	4" Mud Valve	1	EA	$1,094.40	$1,094	
15000	Spray Nozzels	4	EA	$35.49	$142	
15000	Hose Rack and Hose Bibb	2	EA	$294.06	$588	
15000	Pipe Suport Allowance	1	LS	$10,140.00	$10,140	
15000	Mechanical Allowance	1	LS	$20,000.00	$20,000	
	Total					**$31,964**
	Division 16 - Electrical					
16000	Electrical Allowance	20%	LS	$598,718.15	$119,744	
	Total					**$119,744**
	Division 17 - Instrumentation and Controls					
17000	SCADA Programming	1	LS	$75,000.00	$75,000	
17000	Ethernet Switch	1	LS	$25,000.00	$25,000	
17000	Ultrasonic Level Transmitter	1	EA	$1,723.80	$1,724	
	Total					**$101,724**
	Grand Total					**$820,186**
	Mobilization/Demobilization @ 10%	10%				$82,019
	Subtotal					$902,204.14
	Contractor Overhead and Profit at 15%	15%				$135,331
	Subtotal					$1,037,534.76
	Contingencies @ 30%	30%				$311,260
	Subtotal					$1,348,795
	ELA @ 25%	25%				$337,199
	Total Estimated Project Cost					**$1,685,994**

Effluent Pump Station						
SPEC. NO.	**DESCRIPTION**	**QUANTITY**	**UNIT**	**UNIT COST**	**SUBTOTAL**	**TOTAL**
	Division 02 - Site Construction					
02000	Demolition and disposal of existing pumps	1	LS	$10,000.00	$10,000	
	Total					**$10,000**
	Division 8 - Doors and Windows					
0'8000	Rework of skylights	4	EA	$8,000.00	$32,000	
	Total					**$32,000**
	Division 11 - Equipment					
11000	Effluent Pumps	4	EA	$35,000.00	$140,000	
	Total					**$140,000**
	Division 15 - Mechanical					
11000	Flow meter	1	EA	$20,000.00	$20,000	
11000	Piping allowance	1	LS	$25,000.00	$25,000	
	Total					**$45,000**
16000	**Electrical EIC Allowance**	15%	LS	$227,000.00	$34,050	
						$34,050
	Grand Total					**$261,050**
	Mobilization/Demobilization @ 10%	10%				$26,105
	Subtotal					$287,155.00
	Contractor Overhead and Profit at 15%	15%				$43,073
	Subtotal					$330,228.25
	Contingencies @ 30%	30%				$99,068
	Subtotal					$429,297
	ELA @ 25%	25%				$107,324
	Total Estimated Project Cost					**$536,621**

Parallel Outfall						
SPEC. NO.	**DESCRIPTION**	**QUANTITY**	**UNIT**	**UNIT COST**	**SUBTOTAL**	**TOTAL**
	Division 15 - Mechanical					
11000	16" outfall	5280	LF	$200.00	$1,056,000	
	Total					**$1,056,000**
	Grand Total					**$1,056,000**
	Mobilization/Demobilization @ 10%	10%				$105,600
	Subtotal					$1,161,600
	Contractor Overhead and Profit at 15%	15%				$174,240
	Subtotal					$1,335,840
	Contingencies @ 30%	30%				$400,752
	Subtotal					$1,736,592
	ELA @ 25%	25%				$434,148
	Total Estimated Project Cost					**$2,170,740**

Aerobic Digester

SPEC. NO.	DESCRIPTION	QUANTITY	UNIT	UNIT COST	SUBTOTAL	TOTAL
	Division 02 - Site Construction					
02000	Excavation	11860	CY	$.92	$10,911	
02000	Loading	11860	CY	$.86	$10,200	
02000	Hauling	11860	CY	$2.14	$25,380	
02000	Backfill (unconfined)	4581	CY	$12.55	$57,494	
02000	Backfill (confined)	509	CY	$53.02	$26,988	
	Total					**$130,973**
	Division 03 - Concrete					
03000	1-4" Walls	212.5	CY	$1,026.00	$218,025	
03000	12" Walls	21.75	CY	$1,219.00	$26,513	
03000	Footings (12'-0 x 18")	158	CY	$278.00	$43,924	
03000	Slab x 15"	32.75	CY	$296.00	$9,694	
03000	Slab 8"	105	CY	$337.00	$35,385	
03001	Stairs	2	CY	$410.00	$820	
	Total					**$334,361**
	Division 05 - Metals					
05000	Platform 2" alum. grating	36	SF	$28.20	$1,015	
05000	Platform Supports	240	LBS	$4.90	$1,176	
05000	Handrail and Kickplate	42	LF	$79.00	$3,318	
	Total					**$5,509**
	Division 11 - Equipment					
11000	Blower No. 5, 6 & 7 w/sound enclosure	2	EA	$47,333.33	$94,667	
11000	Diffusers (3,608 diffusers/4 dist. grids/four digesters)	1	LS	$67,000.00	$67,000	
11000	BFP Feed Pumps (300 gpm, 30-HP)	4	EA	$22,000.00	$88,000	
	Total					**$249,667**
	Division 15 - Mechanical					
15000	Pipe Supports	1	LS	$10,000.00	$10,000	
15000	DI Plug Valve (10")	4	EA	$2,500.00	$10,000	
15000	DI Plug Valve (8")	10	EA	$1,800.00	$18,000	
15000	DI Plug Valve (6")	19	EA	$1,100.00	$20,900	
15000	DI Check Valve (6")	4	EA	$1,400.00	$5,600	
15000	Process Piping (8" Grooved JT DIP)	100	LF	$95.00	$9,500	
15000	Process Piping (6" Grooved JT DIP)	160	LF	$70.00	$11,200	
15000	Process Fittings (Grooved JT DIP Fittings @50% of pipe)	0.5	LS	$85,200.00	$42,600	
15000	Digester Blower Air Piping (6" Sch. 10S, 304L SST)	160	LF	$76.00	$12,160	
15000	Digester Blower Air Piping (8" Sch. 10S, 304L SST)	160	LF	$106.00	$16,960	
15000	Digester Blower Air Dampers	4	EA	$150.00	$600	
15000	Digester Blower Air Piping Fittings	0.5	LS	$29,720.00	$14,860	
	Total					**$172,380**
	Division 16 - Electrical					
16000	Allowance	0.3	LS	$892,890.41	$267,867	
	Total					**$267,867**
	Division 17 - I&C					
17000	Pressure Sensor w/ Switch and Gauge	2	EA	$1,500.00	$3,000	
17000	pH Probes and Transmitters	2	EA	$7,000.00	$14,000	
17000	TSS Probes and Transmitters	2	EA	$7,000.00	$14,000	

17000	DO Probes and Transmitters	2	EA	$7,000.00	$14,000	
17000	Magnetic Flow Meter (6")	1	EA	$4,000.00	$4,000	
	Total					**$49,000**
	Grand Total					**$1,209,758**
	Mobilization/Demobilization @ 10%	10%				$120,976
	Subtotal					$1,330,733.29
	Contractor Overhead and Profit at 15%	15%				$199,610
	Subtotal					$1,530,343.28
	Contingencies @ 30%	30%				$459,103
	Subtotal					$1,989,446
	ELA @ 25%	25%				$497,362
	Total Estimated Project Cost					**$2,486,808**

Appendix C

KELLER ASSOCIATES DRAFT MASTER PLAN 2016

Scappoose Wastewater Facilities Planning Study

Draft

December 2016

December 2016

City of Scappoose, Oregon

Wastewater Facilities Planning Study

Draft

KELLER associates

Get there!

Get there!

KELLER associates

Keller Associates
707 13th St. SE. Suite 280
Salem, OR 97301

215107\5\S16-005

REGISTERED PROFESSIONAL ENGINEER
12/9/16
83510PE
OREGON
NOV. 10, 2009
PETER OLSEN

EXPIRES: 12/31/18

Signed by:
Peter Olsen, P.E.
Project Manager

TABLE OF CONTENTS

ACRONYMS, ABBREVIATIONS, AND SELECTED DEFINITIONS

ES. EXECUTIVE SUMMARY

1. PROJECT PLANNING

LIST OF TABLES:

LIST OF CHARTS:

LIST OF FIGURES INCLUDED IN REPORT:

APPENDIX A: FIGURES

APPENDIX B: DEQ FLOW CALCULATION

APPENDIX C: CLEAN WATER ACT DATA

APPENDIX D: PUMP INFORMATION

APPENDIX E: SMOKE TESTING & CCTV

APPENDIX F: MODEL DATA AND RESULTS

APPENDIX G: PROJECT SUMMARIES

APPENDIX H: FINANCIAL

Acronyms, Abbreviations, and Selected Definitions

AADF	average annual daily flow
AAF	annual average flow
ADWF	average dry-weather flow
AWWF	average wet-weather flow
BLM	Bureau of Land Management
BOD	biochemical oxygen demand
CCTV	closed circuit television
CDC	Community Development Center
CIP	Capital Improvement Plan
CMS	construction management services
CMU	concrete masonry unit
DAF	Dissolved Air Floatation
DDT	dichlorodiphenyltrichloroethane
DEQ	Oregon Department of Environmental Quality
DMR	discharge monitoring report
DO	dissolved oxygen
DWF	dry weather flow
EDU	equivalent dwelling unit
EOA	Economic Opportunities Analysis
EPA	Environmental Protection Agency
FEMA	Federal Emergency Management Agency
FOG	fats, oil, and grease
fps	feet per second
ft	feet (or) foot
HMI	human-machine interface
hp	horsepower
HVAC	heating, ventilation, and air conditioning
GIS	geographic information system
gpcd	gallons per capita per day
gpd	gallons per day
gph	gallons per hour
gpm	gallons per minute
HDPE	High-density polyethylene
HP	horsepower
hrs	hours
HRT	hydraulic retention time
hz	hertz
I/I	inflow and infiltration
in	inch
KW	kilowatt
lbs	pounds
LS	lift station
MGD	million gallons per day
mg/L	milligrams per liter
mL	milliliter
MLE	Modified Ludzack-Ettinger

MLSS	mixed liquor suspended solids
$MMDWF_{10}$	maximum monthly average dry-weather flow
$MMWWF_5$	maximum monthly average wet-weather flow
NFPA	National Fire Protection Association
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollution Discharge Elimination System
NRCS	Natural Resource Conservation Service
OAR	Oregon Administrative Rules
ODOT	Oregon Department of Transportation
ODSL	Oregon Department of State Lands
O&M	operation and maintenance
OH&P	overhead and profit
PAA	peracetic acid
PCB	polychlorinated biphenyls
PCP	personal care products
PDAF	peak daily average flow
PIF	peak instantaneous flow
pH	hydrogen ion concentration (measure of the acidity or basicity)
PLC	programmable logic controller
ppcd	pounds per capita per day
ppd	pounds per day
PSU	Portland State University
PVC	polyvinyl chloride plastic
PWkF	peak week flow
RAS	return activated sludge
RDII	rainfall-derived infiltration and inflow
RMZ	regulatory mixing zone
ROW	right-of-way
RPA	reasonable potential analysis
rpm	revolutions per minute
RWUP	recycled water use plan
SBR	sequencing batch reactor
SCADA	supervisory control and data acquisition
SCFM	standard cubic feet per minute
SDC	system development charge
sf	square feet
SHPO	State Historic Preservation Office
SRT	solids retention time
TDH	total dynamic head
TKN	total Kjeldahl nitrogen
TMDL	total maximum daily load
TSS	total suspended solids
UGB	urban growth boundary
US	United States
USGS	US Geological Survey
USDA	US Department of Agriculture
USDA-RUS	US Department of Agriculture, Rural Utilities Services
UV	ultraviolet radiation
VFD	variable frequency drive
WAS	waste activated sludge

WPCF	water pollution control facility
WTP	water treatment plant
WWAF	wet-weather average flow
WWFPS	wastewater facilities planning study
WWMM	wet-weather maximum month
WWTP	wastewater treatment plant
ZID	zone of immediate dilution

ES. EXECUTIVE SUMMARY

In 2015, the City of Scappoose, Oregon, contracted with Keller Associates, Inc. to complete a wastewater facilities planning study for the City's sanitary sewer collection system and wastewater treatment plant (WWTP). This section summarizes the major findings of the facilities plan, including brief discussions of alternatives considered and final recommendations.

ES.1 PLANNING CRITERIA

Regulatory requirements, engineering best practices, and City-defined goals and objective form the basis for planning and design. For the Scappoose WWTP facilities, applicable requirements include the National Pollutant Discharge Elimination System (NPDES) permit, total maximum daily loads (TMDLs), State Water Quality Standards, Recycled Water (Reuse) Regulations, and Land Use and Comprehensive Plan Requirements. The planning criteria for the collection system are set by the City using engineering best practices.

ES.2 DESIGN CONDITIONS

ES.2.1 Study Area and Land Use

The study area coincides with the updated urban growth boundary (UGB) and is shown in Figure ES1. The wastewater system currently serves only those areas within the UGB. Build-out of the vacant properties within the UGB is not expected to occur during the current 20-year planning period. Keller Associates recommends that future development within the UGB provide adequate conveyance for connection of the upstream sewer basins.

ES.2.2 Demographics

The City's population has been increasing at an unsteady rate over the past few decades. Historical populations were obtained from records of the County and Portland State University (PSU). The population estimate for 2015 (the most current provided by PSU) was 6,745 people. PSU analyzes historical trends and anticipated growth patterns to develop growth rates in 5-year increments. Based on this data, the City's population is projected to be 9,974 people in 2035. The overall estimated population growth rate from 2015 to 2035 is 2.0% (6,745 to 9,974 people).

The City's Community Development Center (CDC), which includes the planning and engineering departments, provided areas within the City UGB that are anticipated to be developed during the 20-year planning period. These future growth areas, with respective City zoning, are presented in Figure ES2. The estimated population growth corresponding to these areas was calculated using zoning, average lot size by zoning, and people per dwelling unit; the estimated population for 2035 was calculated to be 13,188 people. This population estimation process is presented in detail in Section 1.3.

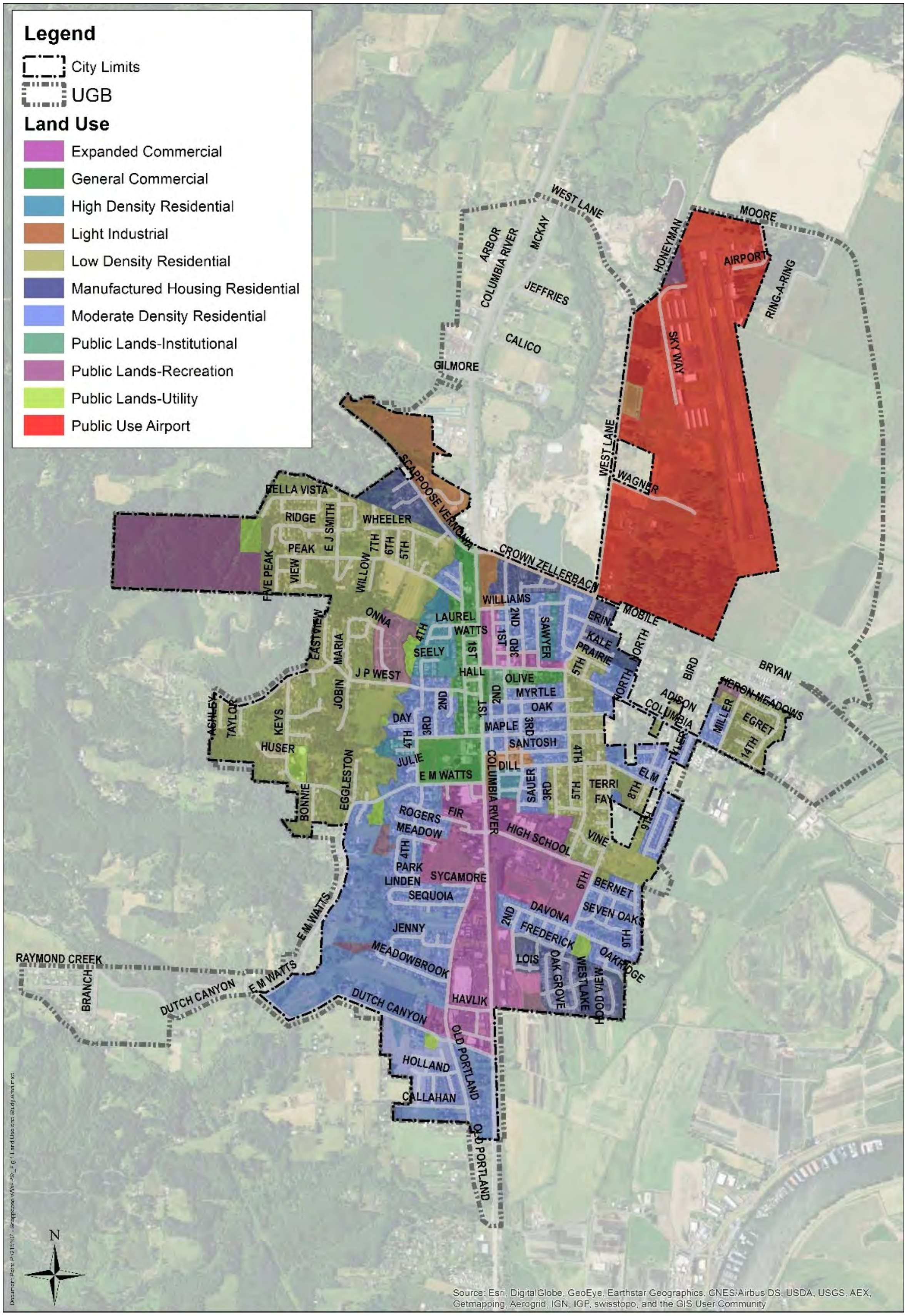


Figure ES1 – Land Use and Study Area

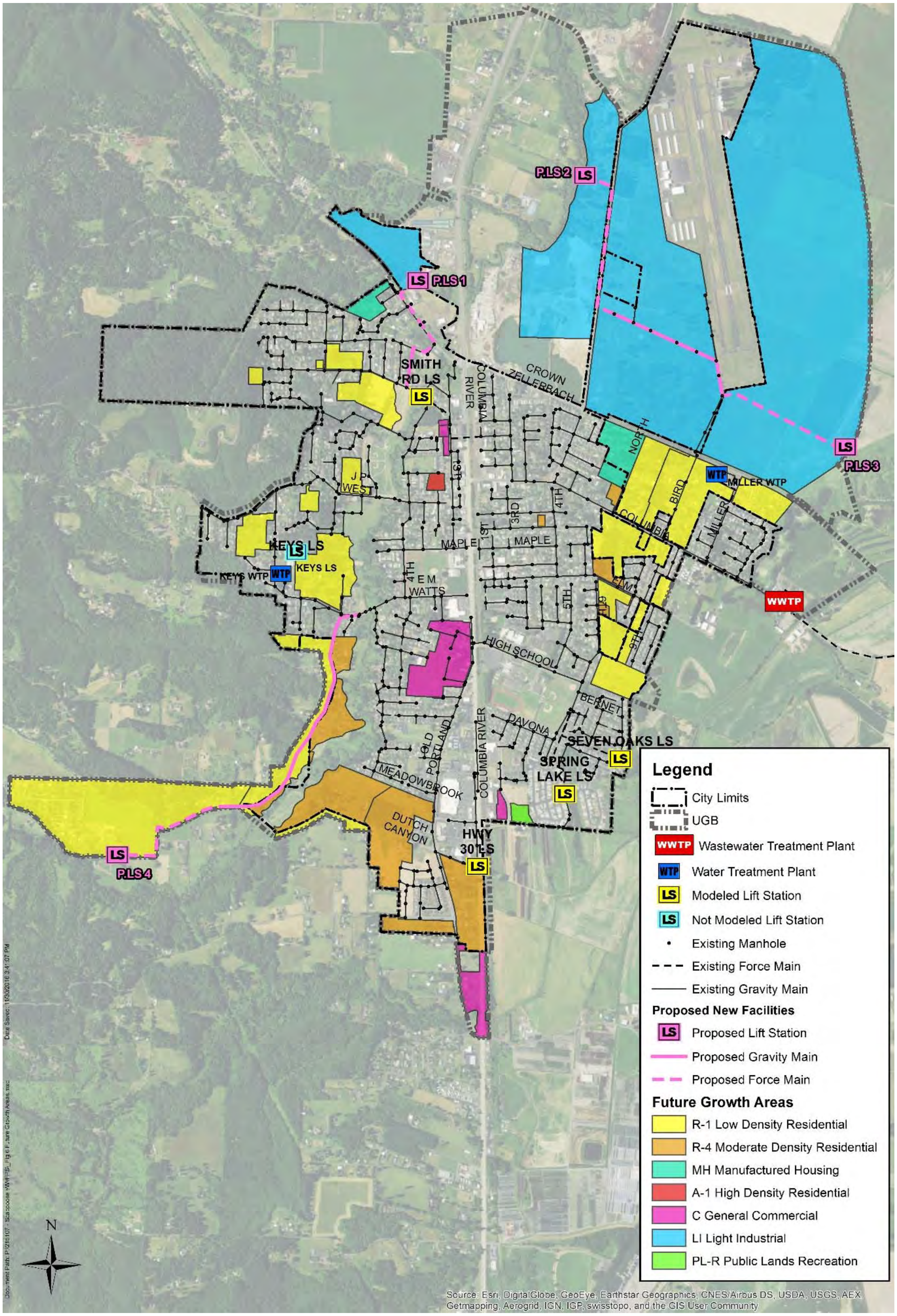


Figure ES2 – Future Growth Areas

The City Economic Opportunities Analysis (EOA) was adopted in 2011 and forecasts the creation of approximately 8,000 jobs (7.6% average annual growth rate) in Scappoose by 2030. The City's planning department is currently processing a number of residential development applications, and there are at least two large industries that have begun the application process. This EOA forecast, along with the influx of current residential and industrial development applications, support the City's projection of significant growth in Scappoose during the 20-year planning period.

As mentioned above, the 2035 PSU-projected population is 9,974, while the City-projected population is 13,188. The large difference between these two growth estimations has significant impacts on flow projections, and thus on the improvements and required capacity of the collection system and WWTP. As directed by the City, this study will use the higher, City-projected population. Further discussion of the revised projections is provided in Section 1.3.

ES.2.3 Wastewater Flows

Daily and monthly treatment plant flow data from January 2010 to September 2015 was provided by the City for analysis. Based on this data, design influent flows (Table ES-1) were calculated using methods recommended by Oregon DEQ (see Section 1.4 for further details).

Future domestic (residential and commercial) flow projections are based on historical per capita flows and City-projected population growth; while industrial flow estimates are based on development input provided by the City. The aggressive growth predictions result in 20-year future flows that are approximately three times that of existing flows. (See Section 1.4 for further discussion.) Table ES-1 shows the revised population and flow projections for the 20-year planning period.

TABLE ES-1: Design and Projected Flows

	2015 Design Flow (MGD)	Projected Unit Flow (gpcd)	Projected Flows [Domestic and Industrial] (MGD)			
Year	2015	2015	2020	2025	2030	2035
Population	6,745	6,745	9,943	10,924	12,003	13,188
AADF	**0.768**	114	1.176	1.777	2.166	2.390
ADWF	**0.660**	98	1.015	1.563	1.916	2.114
AWWF	**0.860**	127	1.315	1.960	2.382	2.628
$MMDWF_{10}$	**0.855**	127	1.308	1.952	2.372	2.616
$MMWWF_5$	**1.212**	180	1.845	2.663	3.207	3.537
PWkF	**1.397**	207	2.124	3.032	3.640	4.013
$PDAF_5$	**1.850**	274	2.805	3.935	4.700	5.181
PIF_5	**2.640**	391	3.995	5.510	6.549	7.218

Notes:

1. MGD = millions of gallons per day; gpcd = gallons per capita per day.
2. $PDAF_5$ modified based on actual peak day flow from DMR records. DEQ graphical $PDAF_5$ is 1.64 MGD.
3. PIF_5 modified based on City-provided flowcharts. DEQ graphical PIF_5 is 2.48 MGD.
4. Peaking factor of 2.5 used for industrial flows.
5. City-projected industrial growth is anticipated to be 30% complete in 5 years, 60% in 10 years, 90% in 15 years, and 100% in 20 years.

ES.3 COLLECTION SYSTEM EVALUATION

ES.3.1 Lift Station Evaluation

There are five lift stations in the Scappoose collection system and one that services the Miller WTP (see Figure ES3 for locations). Smith Road Lift Station is the largest; with a triplex system and an existing capacity of 1,100 gpm. It pumps through approximately 1,600 feet of 12-inch force main. The other four lift stations are each similarly-sized, smaller, duplex systems. Spring Lake has a firm capacity of 100 gpm; Keys Landing has a firm capacity of 120 gpm; and both Highway 30 and Seven Oaks have firm capacities of 150 gpm. Keller Associates visited all of the lift stations and completed a general inventory of facilities. Several issues were identified at each of the lift stations; recommendations to correct these issues are summarized in Section 2.

ES.3.2 Pipeline Condition and Capacity Evaluation

Scappoose's gravity collection system includes approximately 32.2 miles of gravity pipelines ranging from 4- to 21-inches in diameter (Figure ES3). Currently, approximately half or more of all flow goes through the Smith Road Lift Station to reach the WWTP.

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

Portions of the system were smoke tested to identify sources of infiltration and inflow (I/I). These portions tested, as well as problem locations found, are illustrated on Figure ES4.

A GIS-based computer model (XPSWMM 2016) of the collection system was built and exercised to evaluate capacities of the system's trunk lines. Model results (Figure ES5) were used to determine capacity-related problems that exist in the system for the design criteria and flow scenarios analyzed. Several areas of town experience flows that exceed the existing capacity of the pipelines during storm events.

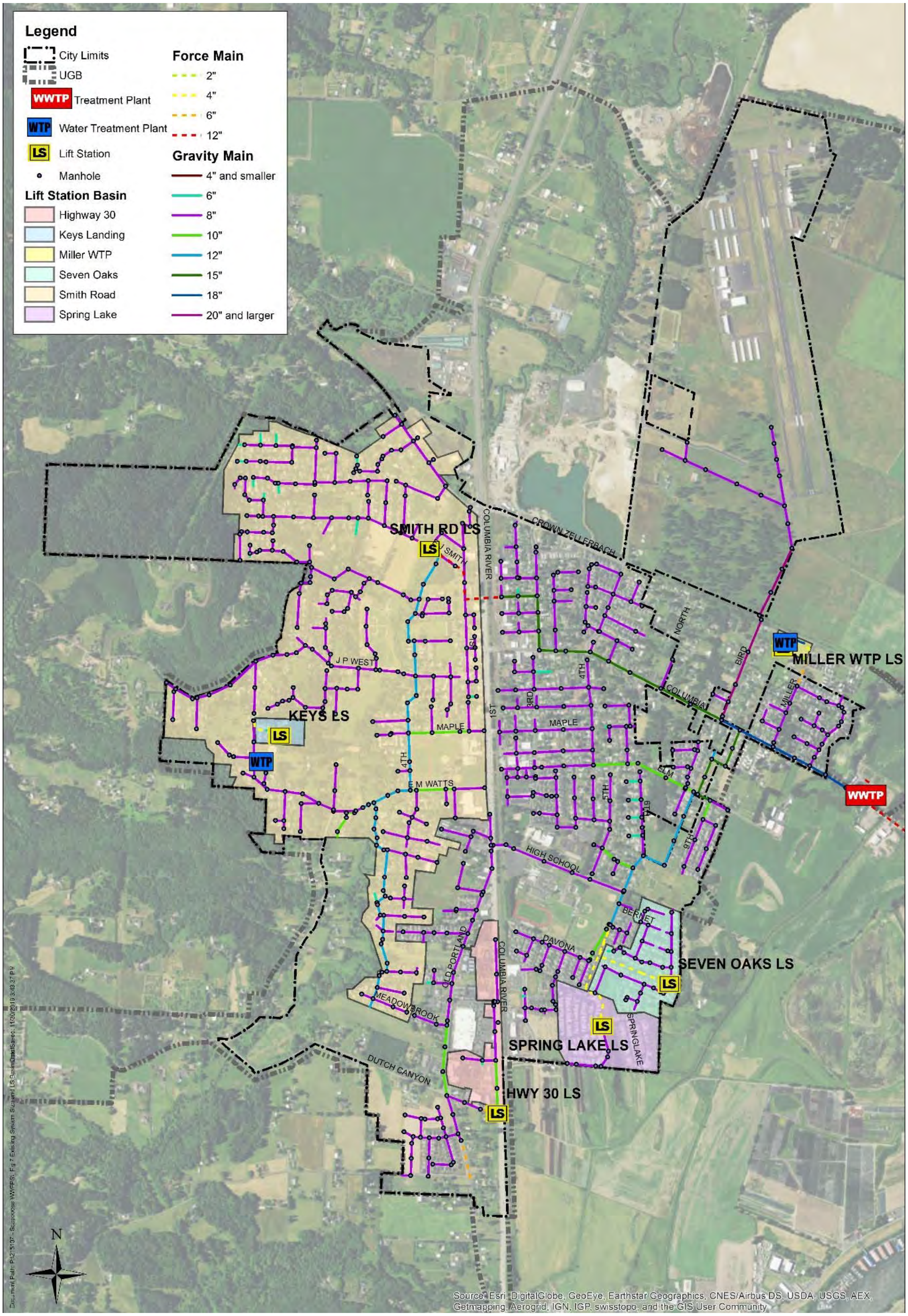


Figure ES3 – Existing System Pipe Size and Lift Station Basins

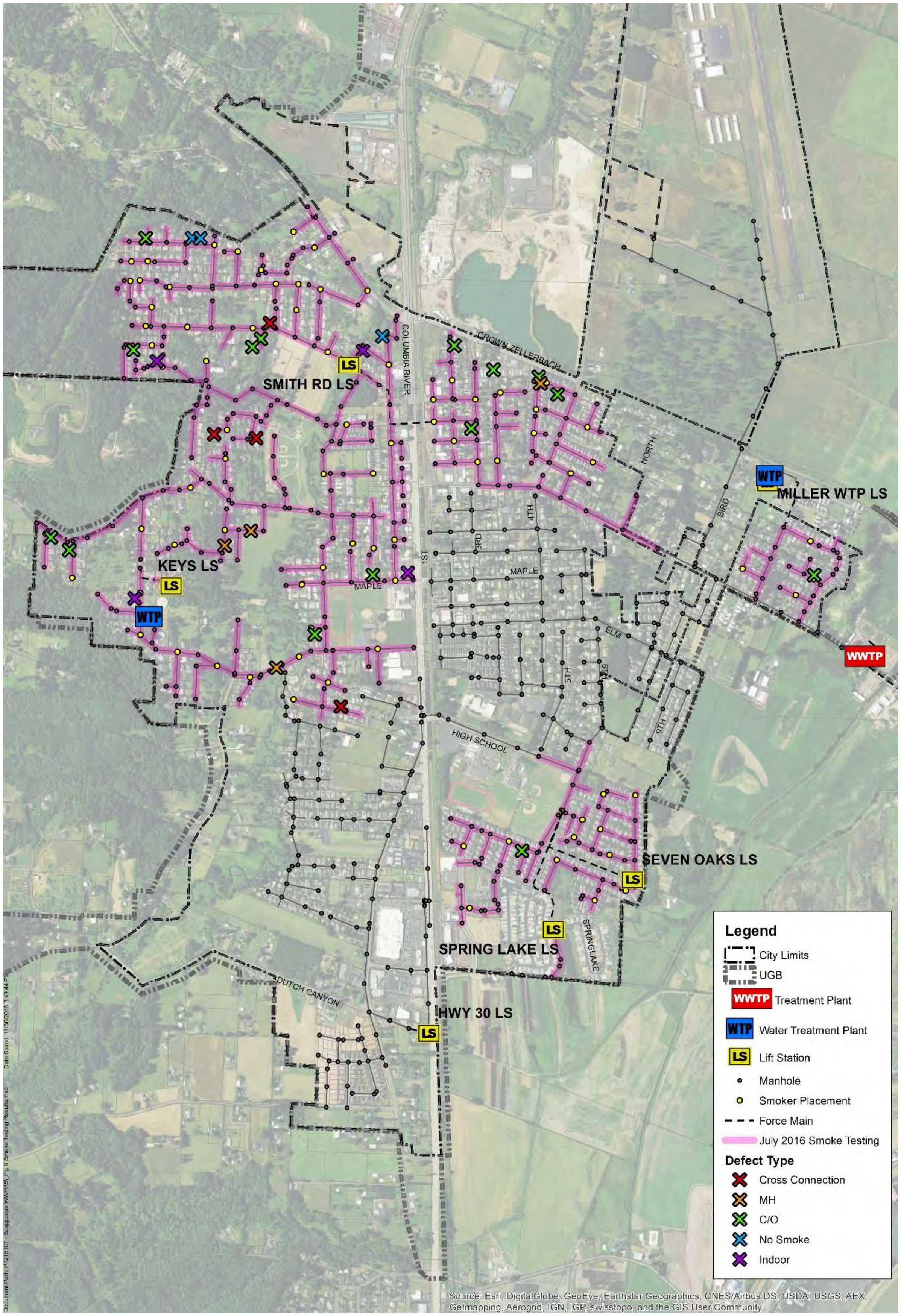


Figure ES4 – Smoke Testing

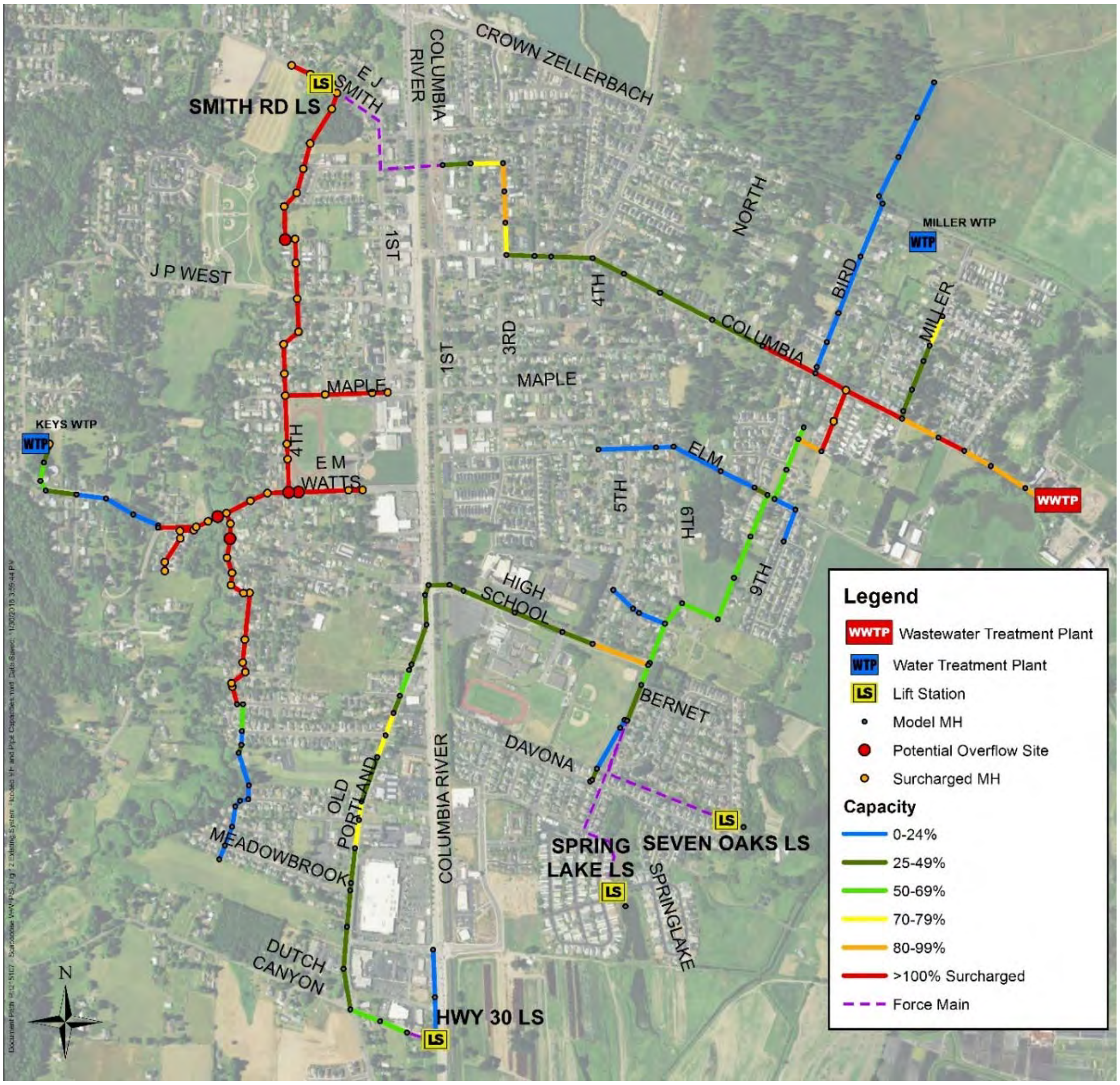


Figure ES5 – Existing System (Design Flow) Potential Overflow Sites and Pipe Capacity

ES.3.3 Collection System Alternatives

Two collection system alternatives were considered to address major capacity deficiencies along the western trunk line: Alternative A, which includes upsizing the existing trunk lines and Smith Road Lift Station; and Alternative B, which includes installing a new relief trunk line across town along SW Maple Street and SE Elm Street.

ES.3.4 Recommended Collection System Improvements

Alternative B was determined to have a lower cost and was selected by City staff as the preferred alternative. Recommended improvements included with Alternative B are summarized in the collection system Capital Improvement Plan (CIP) (Figure ES6). Priority 1 improvements are intended to correct existing deficiencies and are recommended to be completed as soon as funding allows; Priority 2 and 3 improvements address future growth; and Priority 4 is proposed infrastructure that would serve growth in areas currently unsewered.

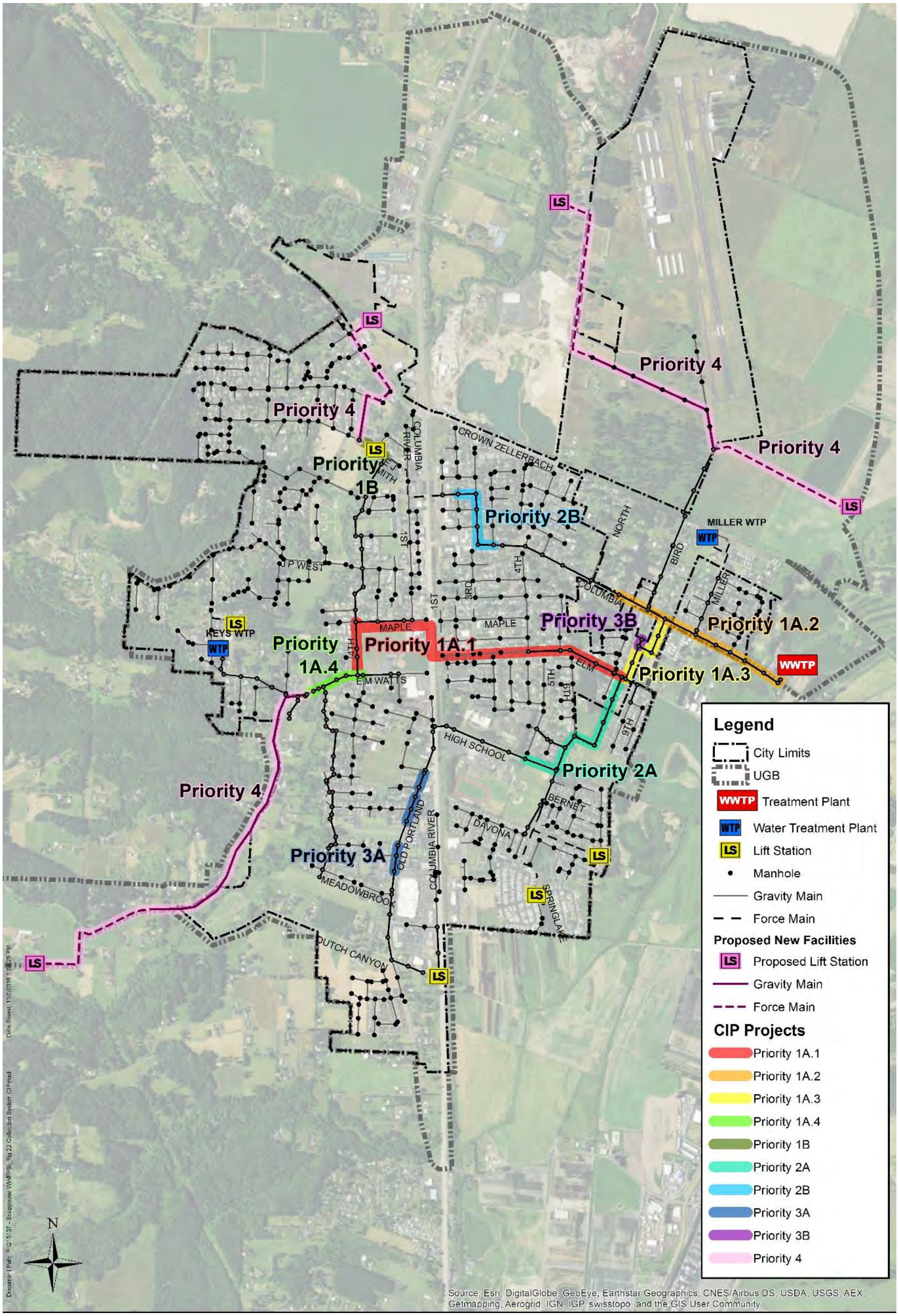


Figure ES6 – Collection System CIP

ES.4 EFFLUENT DISPOSAL

ES.4.1 Effluent Disposal Options

Currently, the WWTP's treated effluent is disinfected using ultraviolet light (UV). The disinfected effluent is then discharged to the Multnomah Channel under NPDES Permit No. 100677. Alternative disposal options were evaluated, including summer farmland application with winter storage and summer farmland application with winter surface water discharge.

ES.4.2 Effluent Disposal Recommendation

The recommendation is to maintain the current disposal method of year-round discharge to the Multnomah Channel.

ES.5 WASTEWATER TREATMENT

ES.5.1 Existing Facilities

The Scappoose WWTP is an extended aeration activated sludge system. The City's wastewater flows into the headworks where the influent is screened. (Screenings are placed in a dumpster and periodically taken to the landfill.) The wastewater is then pumped to a channel where influent samples are taken and the flow measured before entering the aeration basin. There, floating aerators provide oxygen to treat the wastewater. Upon departing the aeration basin, wastewater flows by gravity to secondary clarifiers for solids removal. From the clarifiers, the flow is pumped to cloth media filters for further solids removal prior to being disinfected in the channel UV system. The effluent is sampled and effluent flow is measured near the UV system. Figure ES7 illustrates the existing WWTP layout.

The treated wastewater then flows by gravity to the effluent pump station, where it is pumped approximately one mile to the Multnomah Channel. The effluent is discharged into the channel underwater through a single-port diffuser, which helps distribute and mix the effluent with the channel flow.

Solids removed in the clarifiers are either recycled to the aeration basin (return activated sludge [RAS]) or pumped to the aerobic digester (waste activated sludge [WAS]). The waste solids generated in the activated sludge process are treated in the aerobic digester basins (which are the old aeration basins constructed in the 1970s). Following digestion, the sludge is pumped (by a sludge pump located in the headworks) to the sludge storage lagoons. The digested sludge remains here until it can be applied to nearby agricultural fields. The City has negotiated contracts and received DEQ approval for the use of up to 200 acres for solids application.

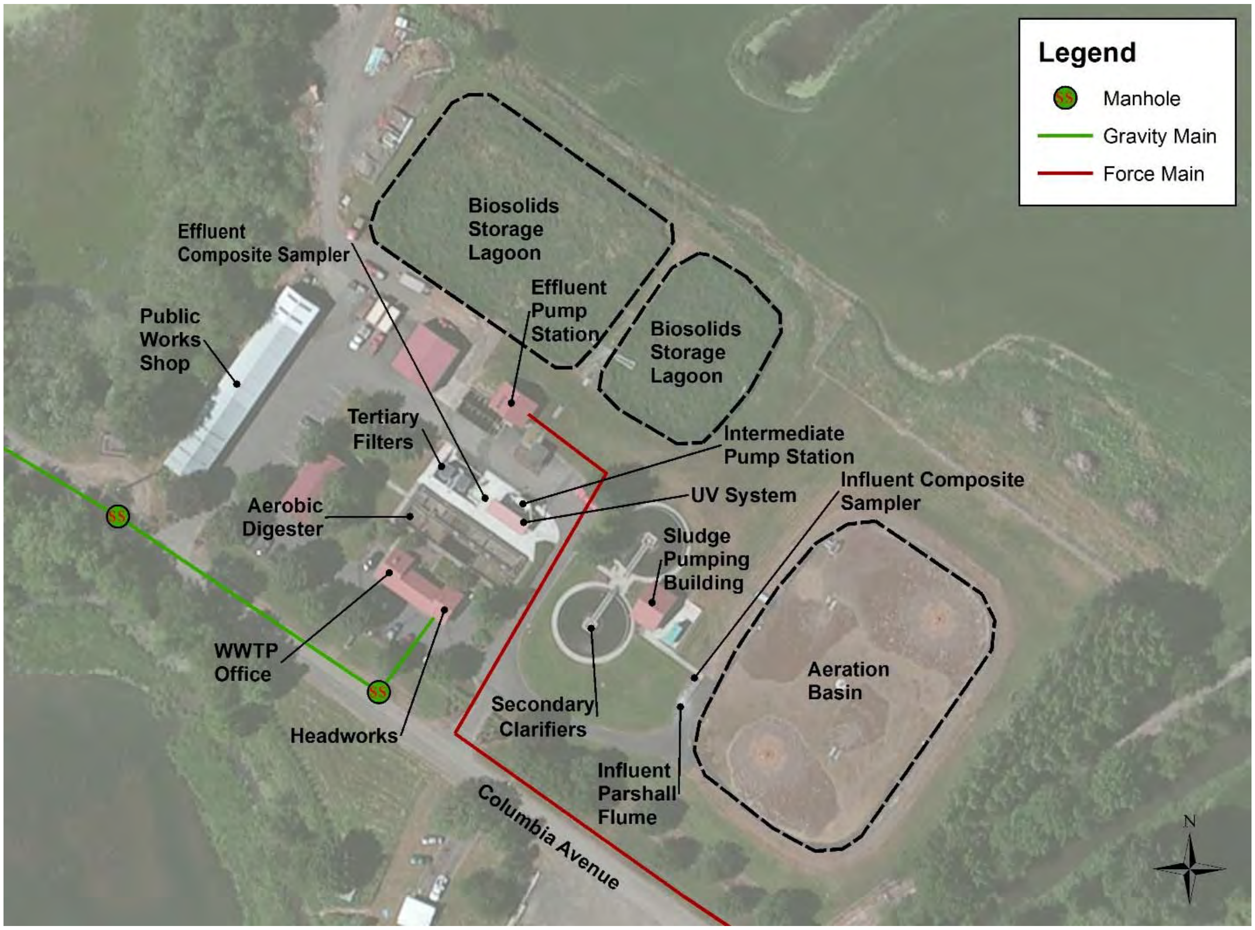


Figure ES7 – Existing WWTP Layout

Deficiencies of the existing wastewater treatment include:

- *Headworks* – The screen, influent pumps, influent pipe, and influent flume do not have sufficient capacity for the future peak instantaneous flows. Excessive grit is also accumulating in the plant, which decreases the plant capacity and also increases wear on the equipment. Some additional items include no screen on the bypass from the influent screen; no freeze protection on the influent screen; no bypass manhole/wet well inside the plant fence; a lack of accurate level measurement in the influent wet well; influent pumps that are not controlled as a system and are located in a wet and hazardous environment; a heating, ventilation, and air conditioning (HVAC) system that is not sized to meet regulatory requirements (NFPA 820); and a sump in the pump dry pit that does not have redundancy.
- *Aeration Basins* – There is only one aeration basin, requiring it to remain in service at all times. This does not allow for liner maintenance and makes equipment maintenance more hazardous. Additionally, the basin does not provide capacity for future flows, nor is it completely mixed; and the aeration system is not adequate and requires a lot of maintenance.
- *Clarifiers* – The secondary clarifiers do not have solids loading capacity for the current peak conditions. They also do not have hydraulic loading capacity for

future peak conditions. The flow split to the clarifiers is not equal, resulting in periodic additional overloading, and the wiring to the clarifier drives is in need of repair.

- *Sludge Pumping* – When a new clarifier is added, there may not be room for the new RAS and WAS pumps in the sludge pumping building. Additionally, the sludge pumping building likely does not meet Class I, Division II NFPA 820 requirements for six air exchanges per hour.
- *Intermediate Pump Station and Tertiary Filters* – The existing intermediate pump station does not have redundancy for the current flows. The tertiary filters and intermediate pump station do not have capacity for future flows. Additionally, the intermediate pump station human-machine interface (HMI) is too small, and the filter covers do not protect operators from the weather.
- *UV* – The UV seals have been leaking, and the system efficiency has been diminishing. Also, future peak flows will exceed the capacity of the UV system and V-notch effluent weir.
- *Effluent Pumping* – The effluent pumps and pipe do not have sufficient capacity for future flows. It is difficult to remove the pumps from the effluent pump station. Additionally, some of the effluent pumps are nearing the end of their expected lifespan.
- *Aerobic Digester* – The aerobic digester is not currently adequately sized to achieve Class B biosolids. Additionally, some of the coarse bubble diffusers are not working, thus some cells are not receiving adequate oxygen. The digester blowers are not rated per NFPA 820 requirements. There is no redundancy for the backup sludge pump, and the sludge flow meter is currently not working.
- *Biosolids Storage* – The biosolids are not mixed or aerated in the biosolids storage lagoons; therefore, the solids deposit in the lagoons. Grass begins to grow on the biosolids, which makes it more difficult to remove the solids without damaging the lagoons.
- *Controls, Utility Water, and Lab* – There is no formal supervisory control and data acquisition (SCADA) system to provide trending and process information. Also, some of the alarms are not being sent by the existing autodialer that could be sent through a SCADA system. Currently, the WWTP uses a groundwater well for washdown activities and plant seal water, as there is no provision for reusing plant effluent. The WWTP office does not have a separate lab area.

ES.5.2 Wastewater Composition

Wastewater flowing into the treatment plant from the City varies in concentration due to the infiltration and inflow diluting the wastewater. Otherwise, the composition of the wastewater is fairly consistent.

The influent BOD_5 and TSS data for the time period of January 2010 to September 2015 was evaluated to determine annual average, dry weather average, dry weather maximum month, wet weather average, and wet weather maximum month loads (pounds per day). The pounds per day BOD_5 and TSS loading data was used to calculate the pounds per capita per day (ppcd) for the various flows; these values were used to estimate the design year 2035 loadings using the 2035 population of 13,188.

ES.5.3 Treatment Alternatives

Process alternatives were considered to address WWTP deficiencies. Alternatives considered for headworks were constructing a new headworks, or repairing and expanding the existing headworks. Options to improve secondary treatment were new aeration basins, new oxidation ditches, and new sequencing batch reactors (SBRs). Disinfection alternatives included upgrading and expanding the existing UV system, chlorination and dechlorination, and peracetic acid (PAA). The interim biosolids plan was developed evaluating alternatives that included a new dewatering building, and repurpose of an existing metal shop to house dewatering equipment. Biosolids treatment options were to construct either new aerobic or anaerobic digesters. Biosolids disposal alternatives considered were continued Class B land application, and Class A composting using covered aerated static piles.

ES.5.4 Recommended Treatment Improvements

The recommended treatment processes include:

- *Headworks* – Upgrade and expand the existing headworks.
- *Secondary Treatment* – New aeration basins, as it is the most familiar technology to the City.
- *Disinfection* – Upgrade and expand the existing UV system.
- *Interim Biosolids Plan* – Construct a new dewatering building.
- *Biosolids Treatment* – New aerobic digesters.
- *Biosolids Disposal* – Continue Class B land application.

A proposed layout of the future treatment plant improvements is shown in Figure ES8.

The City-projected growth is significant, and initial WWTP improvements should not be sized to handle the full revised peak flows. Instead, a phased approach for WWTP improvements is presented in the CIP. Future recorded peak flows and updated flow projections for the WWTP will trigger subsequent phases of the improvements. The full 20-year CIP shown in Table ES-2 includes flow triggers for each priority project. These flow triggers will help City staff track and determine when improvements need to be completed as the City grows. Table ES-2 also includes an estimated number of additional EDUs that will increase system flows to meet each flow trigger. For these estimates, an EDU was assumed to be a low-density, single-dwelling unit with an associated $MMWWF_5$ flow of 450 gpd/EDU and PIF_5 flow of 978 gpd/EDU. Expandability will be a key component of the recommended WWTP improvements. This will allow the WWTP to operate efficiently while planning for future growth.

Figure ES8 – Proposed Future WWTP Layout

ES.6 CAPITAL IMPROVEMENT PLAN AND FINANCING

ES.6.1 Summary of Costs

Table ES-2 presents the 20-year CIP. Projects are organized by priority. The need for each improvement varies for reasons including compliance with the City's existing discharge permit, achieving capacity necessary to accommodate growth, improving operations, and replacing worn/old equipment. Costs reflect planning-level estimates and should be refined in the subsequent pre-design and design phases of implementation. Priority 1 improvement expenses are anticipated to occur over the next several years. Priority 2, 3, and 4 improvements are primarily triggered by growth.

TABLE ES-2: 20-Year Capital Improvement Plan

ID#	Item	WWTP Flow Trigger	Additional EDUs to Meet Flow Trigger	Total Estimated Cost (2016)	SDC Growth Portion		City's Estimated Portion
					%	Cost	
Priority 1 Improvements							
Wastewater Collection System							
1A.1	New, Relief Trunk Line	--	--	$ 1,720,000	30%	$ 516,000	$ 1,204,000
1A.2	E Columbia Ave Trunk Line	--	--	$ 1,290,000	59%	$ 761,000	$ 529,000
1A.3	SE Tyler St and SE Tussing Wy Trunk Line	--	--	$ 630,000	49%	$ 309,000	$ 321,000
1A.4	SW Em Watts Rd Trunk Line	--	--	$ 270,000	35%	$ 95,000	$ 175,000
1B	NW Smith Road Trunk Line	--	--	$ 160,000	6%	$ 10,000	$ 150,000
1C	Lift Station Improvements	--	--	$ 410,000	0%	$ -	$ 410,000
Wastewater Treatment							
1a	Interim Biosolids Plan	Beyond Capacity	--	$ 2,530,000	33%	$ 843,000	$ 1,687,000
1b	Rehabilitate UV System	--	--	$ 373,000	15%	$ 55,000	$ 318,000
1c.1	Add 3rd pump to Inter. Pump Station	Beyond Capacity	--	$ 35,000	70%	$ 25,000	$ 10,000
1c.2	Add disks to existing Tertiary Filters	At Capacity	--	$ 97,000	100%	$ 97,000	$ -
1d	SCADA System	--	--	$ 297,000	63%	$ 188,000	$ 109,000
1e.1	Aeration for Aeration Basin	Beyond Capacity	--	$ 341,000	33%	$ 114,000	$ 227,000
1e.2	Sec. Clarifier and Sludge Bldg. Exp.	Beyond Capacity	--	$ 2,190,000	100%	$ 2,190,000	$ -
Total Priority 1 Improvements (rounded)				**$ 10,340,000**		**$ 5,200,000**	**$ 5,140,000**
Rate Impact (20 yr, 1.6%)				**$ 18.77**			**$ 9.33**
Priority 2 Improvements							
Wastewater Collection System							
2A	SE 6th St Trunk Line	--	--	$ 610,000	100%	$ 610,000	$ -
2B	NE Laurel St and NE 3rd St Trunk Line	--	--	$ 370,000	100%	$ 370,000	$ -
2C	Lift Station Improvements	--	--	$ 240,000	26%	$ 62,000	$ 178,000
Wastewater Treatment							
2a.1	New Aeration Basins	1.9 MGD $MMWWF_5$	1,530	$ 7,750,000	54%	$ 4,173,000	$ 3,577,000
2a.2	New Aerobic Digester	1.8 MGD $MMWWF_5$	1,310	$ 2,020,000	48%	$ 966,000	$ 1,054,000
2b.1	Expand Headworks	4.1 MGD PIF_5	1,500	$ 3,410,000	63%	$ 2,163,000	$ 1,247,000
2b.2	Upgrade Influent Pumps	3.5 MGD PIF_5	880	$ 928,000	63%	$ 589,000	$ 339,000
2c.1	Upgrade Effluent Pumps	3.3 MGD PIF_5	675	$ 833,000	63%	$ 528,000	$ 305,000
2c.2	Increase Effluent Pipe	4.0 MGD PIF_5	1,400	$ 2,100,000	63%	$ 1,332,000	$ 768,000
2d.1	Upgrade Intermediate Pump Station	4.0 MGD PIF_5	1,400	$ 455,000	100%	$ 455,000	$ -
2d.2	Additional Tertiary Filter Unit	1.8 MGD $MMWWF_5$	1,310	$ 877,000	100%	$ 877,000	$ -
2e	Upgrade UV System	3.1 MGD PIF_5	480	$ 1,117,000	63%	$ 709,000	$ 408,000
Total Priority 2 Improvements (rounded)				**$ 20,710,000**		**$12,830,000**	**$ 7,880,000**
Priority 3 Improvements							
Wastewater Collection System							
3A	SW Old Portland Rd Trunk Line	--	--	$ 280,000	100%	$ 280,000	$ -
3B	SE Tussing Wy Trunk Line	--	--	$ 50,000	40%	$ 20,000	$ 30,000
Wastewater Treatment							
3a.1	Additional Aeration Basin	2.6 MGD $MMWWF_5$	3,090	$ 3,220,000	100%	$ 3,220,000	$ -
3a.2	Additional Secondary Clarifier	3.4 MGD $MMWWF_5$	4,870	$ 1,320,000	100%	$ 1,320,000	$ -
3b.1	Additional Aerobic Digester	2.3 MGD $MMWWF_5$	2,420	$ 1,830,000	100%	$ 1,830,000	$ -
3b.2	Additional Screw Presses	2.3 MGD $MMWWF_5$	2,420	$ 1,684,000	100%	$ 1,684,000	$ -
3c	Plant Water System	--	--	$ 208,000	63%	$ 132,000	$ 76,000
Total Priority 3 Improvements (rounded)				**$ 8,590,000**		**$ 8,490,000**	**$ 110,000**
Priority 4 Improvements							
Wastewater Collection System							
4A	P.LS 1, Force Main and Gravity Line	--		$ 660,000	100%	$ 660,000	$ -
4B	P.LS 2, Force Main and Gravity Line	--		$ 1,160,000	100%	$ 1,160,000	$ -
4C	P.LS 3 and Force Main	--		$ 750,000	100%	$ 750,000	$ -
4D	P.LS 4, Force Main and Gravity Line	--		$ 1,210,000	100%	$ 1,210,000	$ -
Total Priority 4 Improvements (rounded)				**$ 3,780,000**		**$ 3,780,000**	**$ -**
TOTAL WASTEWATER IMPROVEMENTS COSTS (rounded)				**$ 43,420,000**		**$30,300,000**	**$ 13,130,000**

* All costs in 2016 Dollars. Costs include mobilization (**10%**), contractor overhead and profit (**OH&P; 15%**), contingency (**30%**), engineering and construction management services (**CMS; 20-30%**), and legal, administrative, and permitting services (**2%**).

ES.6.2 Budget and Rate Impacts

Funding for the recommended system improvements may come from any number of sources. This section presents potential user rate impacts if priority improvements are funded through a low-interest loan with debt service payments (20 year, 1.6%) made through a user rate increase. Table ES-3 outlines the potential residential user rate impacts and assumes a flat rate increase to all 2,700 sewer EDUs. Actual rate impacts will vary depending on the City's rate structure, available system development charge (SDC) funds, existing budget surplus, funding source(s), potential grants, project phasing, and terms of the loan. A separate user rate study may be warranted to complete a more detailed evaluation of potential user rate impacts.

TABLE ES-3: Potential Monthly User Rate Impact to Fund Priority Improvements

	Annual Paymet (20 years, 1.6%)	User Rate Increase	User Rate Total
Existing User Rate (2016)	--	--	$ 43.31
Priority 1 Improvements[1,2]	$ 302,343	$ 9.33	$ 52.64

[1] Including capital improvements only.
[2] Assuming SDCs are providing funding portion attributed to growth.

ES.6.3 Other Annual Costs

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

- Collection system replacement/rehabilitation needs – City should work towards establishing a total budget of approximately $595,000/year.
- Collection system cleaning and CCTV needs – Following the timeline described in Section 7.1.2, the City should budget approximately $103,000/year.
- Other additional annual operation and maintenance costs and short-lived assets associated with Priority 1 WWTP Capital Improvement Plan (dewatering equipment, additional aeration, filters, pumping, disinfection, etc.) are anticipated to be approximately $200,000/year. Short-lived assets include pump replacements, motor replacements, etc. A large portion of this is associated with increased power usage.

ES.6.4 SDCs

The City's current sewer system development charge (SDC) for a single-family home is $4,276.04. The sewer SDC is typically divided into two components: reimbursement and growth. The scope of this study included estimating the SDC eligibility for each identified capital improvement. It is the intent that this information will be utilized by the City's financial consultant to update the City's SDCs. The estimated SDC eligibility for each identified capital improvement is shown in Table ES-2.

ES.6.5 Financing Options

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

1. PROJECT PLANNING

The City of Scappoose owns and operates a municipal sewage collection system and a wastewater treatment plant (WWTP). The purpose of this study is to determine the needs of the City for wastewater collection and treatment, evaluate if the existing pipe network and WWTP can meet those needs, and provide a long-term plan to implement improvements to the collection system and WWTP so the needs of the City can be met.

This facilities plan describes the conditions, flows, and problems in the existing system; analyzes the hydraulic and biologic flow data; and provides recommendations for improvements to the collection system and WWTP.

1.1 LOCATION

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

1.2 ENVIRONMENTAL RESOURCES PRESENT

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

1.2.1 Land Use / Prime Farmland / Formally Classified Lands

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

1.2.2 Earthquake Hazards

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

1.2.3 Floodplains

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

1.2.4 Wetlands

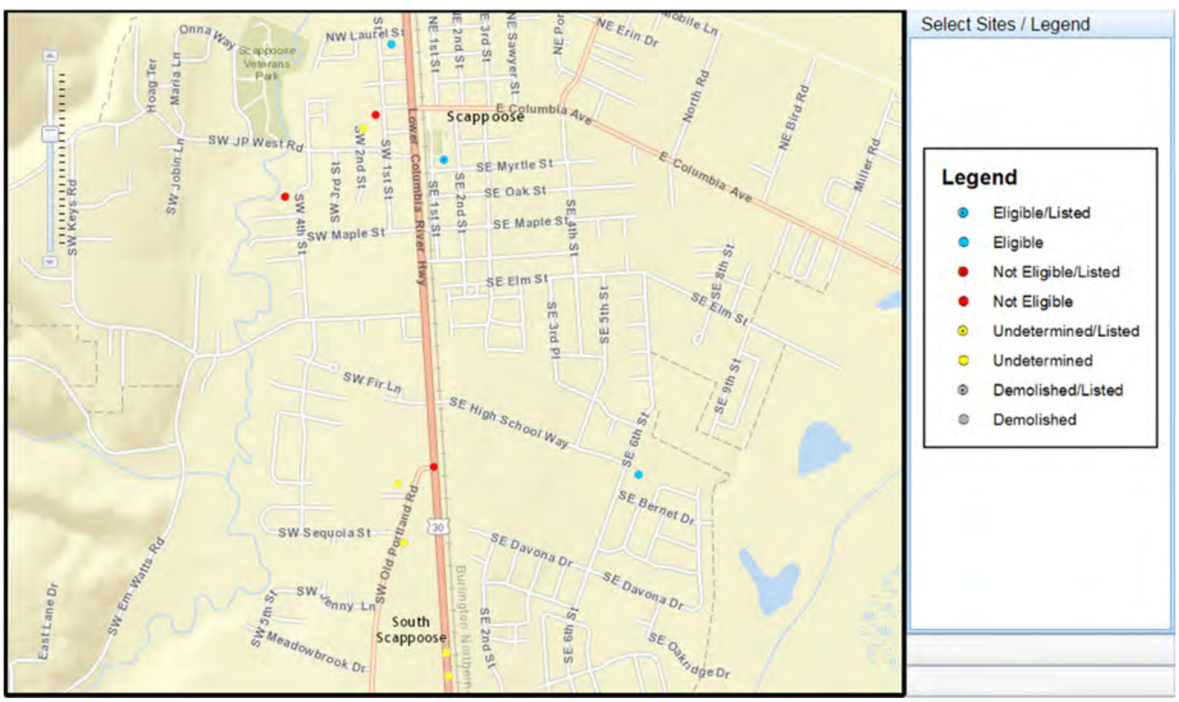
The Oregon Department of State Lands (ODSL) keeps an inventory of local wetland areas in Oregon. The City of Scappoose has a completed Local Wetland Inventory from December 1998. Wetland delineation was not within the scope of this project, so the ODLS Local Wetland Inventory and the U.S. Fish and Wildlife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

1.2.5 Cultural Resources

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

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

Chart 1-1: Above-Ground Cultural Resources



1.2.6 Biological Resources

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

1.2.7 Water Resources

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

1.2.8 Coastal Resources

There are no coastal areas within the study area.

1.2.9 Socio-Economic Conditions

The population in the area is primarily (91%) Caucasian, according to the 2010 census. Hispanics make up 5% of the population. The median household income is $62,244 (in 2014 dollars), which is 23% higher than the state average.

1.2.10 Miscellaneous Issues

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

1.3 POPULATION TRENDS

The official population projections and records of the City of Scappoose are currently coordinated by collaborative efforts of the County and Portland State University (PSU). The collaborating agencies published a document in 2008 establishing the official coordinated population projection rates for all of the cities in Columbia County. The document is titled "Population Forecasts for Columbia County Oregon, its Cities and Unincorporated Area 2010-2035", and also includes a summary of historical populations from the US Census.

The historical populations presented in the referenced document are shown in Table 1-1. Each year, PSU establishes a certified population estimate. The population shown for 2015 (Table 1-1) is the most recent certified population at the time of these projections. This population was used as the base starting point for population projections. The projections shown in Table 1-1 were calculated using the moderate growth rates presented in the referenced document.

Growth rates are not anticipated to be consistent for the entire 20-year planning period. The overall estimated population growth rate from 2015 to 2035 is 2.0% (from 6,745 to 9,974 people).

Table 1-1: Population History and Projections

Year	Population	Source
1970	1,859	U.S. Census referenced in PSU Report for Columbia County
1980	3,213	U.S. Census referenced in PSU Report for Columbia County
1990	3,529	U.S. Census referenced in PSU Report for Columbia County
2000	4,976	U.S. Census referenced in PSU Report for Columbia County
2010	6,592	U.S. Census referenced in PSU Report for Columbia County
2015	6,745	PSU Certified population
2020	7,520	Calculated using coordinated growth rate (2.2%)
2025	8,262	Calculated using coordinated growth rate (1.9%)
2030	9,078	Calculated using coordinated growth rate (1.9%)
2035	9,974	Calculated using coordinated growth rate (1.9%)

The area of land associated with the PSU projected population was estimated using average population densities and lot sizes. Assuming an average lot size of 0.2 acres with 20% additional area for right-of-way (ROW) development and a population density of 2.5 people per dwelling unit, the associated growth area for PSU population projections is approximately 310 acres.

The City's Community Development Center (CDC), which includes the planning and engineering departments, provided areas within the City UGB that are anticipated to be developed during the 20-year planning period (Figure 6 in Appendix A). This total area includes 440 acres of domestic development, and an additional 660 acres of light industrial development. Areas outside City limits are currently zoned in the City's Comprehensive Plan. The CDC provided City zoning designations that correspond with Comprehensive Plan zoning designations for future growth areas (Table 1-2).

Table 1-2: Comprehensive Plan Zoning and Corresponding City Zoning

Comp. Plan Zoning	City Zoning
SR	R-1
GR	R-4
MH	MH

The City of Scappoose Economic Opportunities Analysis (EOA) was prepared by Johnson Reid and adopted as an amendment to the Scappoose Comprehensive Plan in 2011. The EOA's employment forecast anticipates the creation of approximately 8,000 jobs (7.6% average annual growth rate) in Scappoose by 2030. Currently, the most prominent industries in the Scappoose area are aviation manufacturing and services, retail, and nursery services. The commercial and industrial land demand (including streets, utilities, etc.) projected in the EOA for the City's growth is 483 acres by 2030. The City's planning department is currently processing a number of applications for residential developments, and there are at least two large industrial developments that have begun the application process with the City. The EOA forecast, along with the influx of current residential and industrial development applications, support the City's projection of significant growth in Scappoose during the 20-year planning period.

Estimates made by the CDC and the City Development Code were used to establish average dwelling units per lot and average lot size by zoning category (Table 1-3).

Table 1-3: Average Dwellings per Lot and Lot Size

Zoning	Average Dwelling Units per Lot	Average Lot Size (sqft)
R-1	100% single dwelling (1.0 EDU/lot)	7,500
R-4[1]	70% single dwelling, 30% fourplex (1.9 EDU/lot)	7,800
MH[1]	70% single dwelling, 30% fourplex (1.9 EDU/lot)	7,800
A-1[2]	100% 8 dwelling units per lot (8.0 EDU/lot)	17,000
C[3]	65% commercial, 35% residential [equivalent to A-1] (2.8 EDU/lot)	12,450

1. Assume 70% are single dwellings and 30% are fourplexes.
2. Assume max. allowable units (8 units/lot).
3. Assume 65% commercial with flow rate of 1,250 gpd/lot and 35% residential (same density as A-1).

The last column of Table 1-4 shows the projected population growth associated with the growth areas by zoning category identified in Figure 6 (Appendix A). These populations were calculated using average lot size, population density, and area of future growth. A population density of 2.5 people per dwelling unit was provided by CDC and documented in the PSU population projection. The City recommended that 20% of future growth area be allocated for right-of-way (ROW) and other public dedication.

Table 1-4: City-Projected Population Growth

Zoning	Average Lot Size (sqft)	People/Lot	Pop. Density (people/ac)	Future Growth Area[4,5] (ac)	Pop. in Future Growth Area
R-1	7,500	2.5	15	277	3,218
R-4[1]	7,800	4.75	27	104	2,207
MH[1]	7,800	4.75	27	19.1	405
A-1[2]	17,000	20	51	1.93	79
C[3]	12,450	20 (if res.)	51	37.2	534
DOMESTIC TOTALS:				**440**	**6,443**
LI	N/A	N/A	N/A	658	-
DOMESTIC AND INDUSTRIAL TOTALS:				**1,098**	**6,443**

1. Assume 70% are single dwellings and 30% are fourplexes.
2. Assume max. allowable units (8 units/lot).
3. Assume 65% commercial with flow rate of 1250 gpd/lot and 35% residential (same density as A-1).
4. See Figure 6.
5. Allocates 20% of area for roads and other public dedications, except for industrial zoning.

Table 1-5 compares the predicted population growth in the 20-year planning period using the PSU moderate growth rate, the PSU high growth rate, and the City's projected growth areas. The City's projected growth is twice as high as the PSU moderate growth rate projections. The City has directed the higher, City-projected growth to be used for this study.

Table 1-5: 20-Year Population Growth Estimates

Estimated 20-yr Pop. Growth	Method
3,229	PSU Moderate Growth Rate
4,040	PSU High Growth Rate
6,443	City's Projected Growth Areas (Figure 6)

PSU is in the process of updating the population forecast for Columbia County; documents are anticipated to be completed in July 2017. A buildable lands study is also in progress for the City, which is due to be complete in May or June 2017. Once released, both of these documents could trigger an update to this facilities plan if they indicate population and future growth significantly different than their respective previous studies and City estimates.

1.4 FLOWS

The wastewater flow analysis reviews historical wastewater flows, develops design flows, and provides flow projections for the planning period. This section summarizes the results of the flow analysis. Keller Associates used the methods recommended by Oregon DEQ in "Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon" for determining design flows in the City's system (summarized in Table 1-8). Details of how each design flow was derived are discussed in the following paragraphs.

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. The years with a complete data set (2010-2014) were averaged to obtain the design AADF.

Average Dry-Weather Flow (ADWF)

The average dry-weather flow (ADWF) is the average daily flow for the period of May through October. An ADWF was calculated for each year of data. Data from January 2010 to September 2015 was averaged to obtain the design ADWF.

Average Wet-Weather Flow (AWWF)

The AWWF was calculated as the average daily flow for the period encompassing January through April, and November through December, for each year of data. Five years' worth of data (2010-2014) was averaged to obtain the AWWF.

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

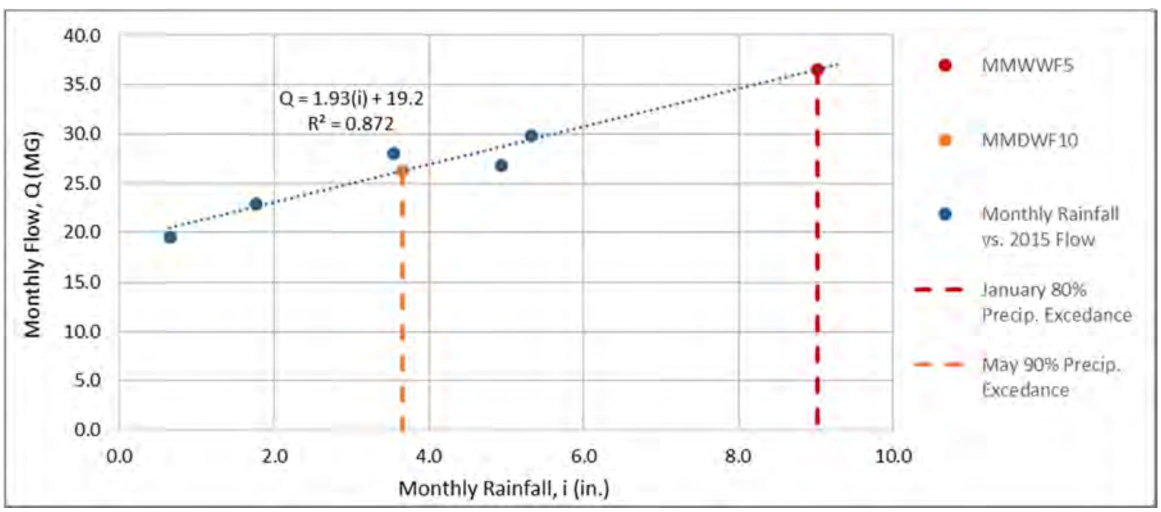
The $MMDWF_{10}$ represents the rainiest summer month of high groundwater. The DEQ method for calculating $MMDWF_{10}$ is to graph the January through May total monthly flows for each month of the most recent year against total precipitation for the month. A trend line is fitted to the data, and then $MMDWF_{10}$ is read from the trend line at a precipitation equal to the May 90% precipitation exceedance value (3.66 in.) interpolated from the 1981-2010 US Climate Normals[1]. Monthly normals for the City of Vernonia were used due to it being the closest station with precipitation probability data. Since Oregon DEQ states that May is typically the maximum month for the dry-weather period of May through October, selecting the May 90% precipitation exceedance most likely corresponds to the maximum month during the dry-weather period for a 10-year event.

DEQ recommends using the most recent year of valid data. As sufficient data from 2015 was available, it was selected. Chart 1-2 shows the graphical calculation of $MMDWF_{10}$ using the DEQ guidance methodology. Table 1-2 summarizes the data points illustrated in Chart 1-2.

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

The $MMWWF_5$ represents the highest monthly average during the winter period of high groundwater. The DEQ method for calculating $MMWWF_5$ is to enter the graph of January through May average daily flows versus monthly precipitation and read $MMWWF_5$ from the trend line at a precipitation equal to the January 80% precipitation exceedance value (9.02 in.) interpolated from US Climate Normals. Since Oregon DEQ states that January is typically the maximum month for the wet-weather period of January through April, selecting the January 80% precipitation exceedance most likely corresponds to the maximum month during the wet-weather period for a 5-year event. This result is illustrated in Chart 1-2 and broken down in Table 1-6.

Chart 1-2: Flow vs. Rainfall ($MMDWF_{10}$ and $MMWWF_5$)



[1] Produced by NOAA and the US Department of Commerce

Table 1-6: Flow vs. Rainfall ($MMDWF_{10}$ and $MMWWF_5$)

Month	Flow (MG/Month)	Rainfall (in./Month)
January	28.1	3.54
February	29.8	5.32
March	26.8	4.94
April	22.6	1.77
May	19.6	0.65
$MMDWF_{10}$	26.2	3.66
$MMDWF_5$	36.6	9.02

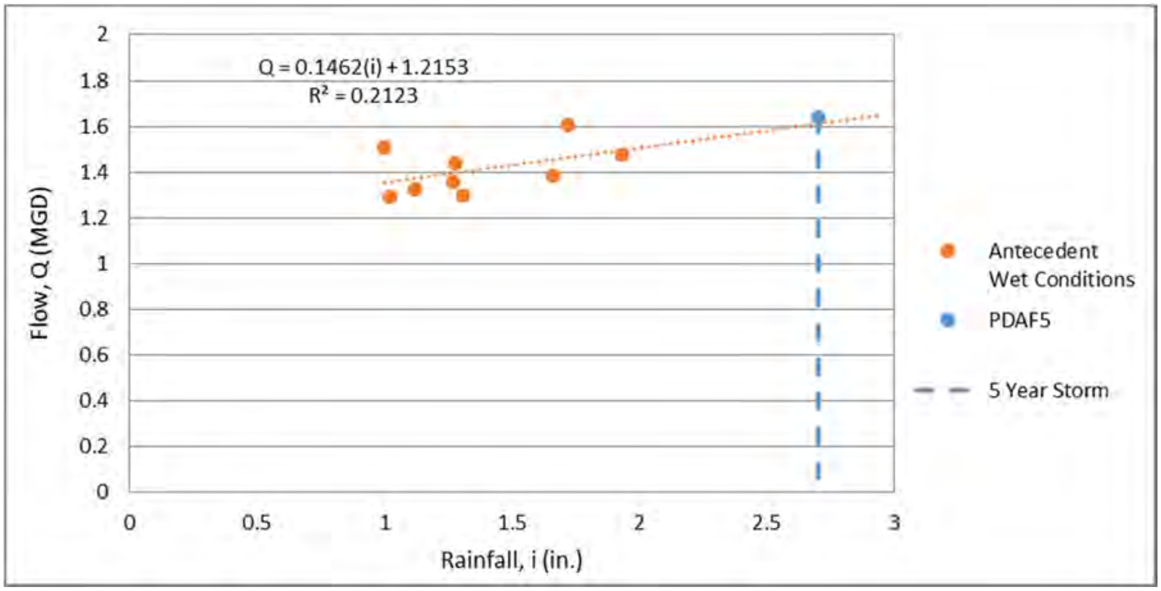
Peak Week Flow (PWkF)

A 7-day average flow was calculated for every day using the seven previous days of data (rolling average). Peak week flow (PWkF) for each year was then calculated as the maximum of all weekly (7-day) rolling averages in a given year. The maximum week was selected as the PWkF.

Peak Daily Average Flow ($PDAF_5$)

As outlined by Oregon DEQ, the $PDAF_5$ typically corresponds to the 5-year storm event, and therefore, is calculated as the flow resulting from a 5-year storm event during a period of likely high groundwater (January through April). The DEQ method for determining $PDAF_5$ is to plot daily plant flow against daily precipitation for large storm events over several years, only using data during wet-weather seasons when groundwater is high. A trend line is fitted to the data, and $PDAF_5$ is then read from the trend line at the 5-year, 24-hour storm event (2.7 inches per the NOAA isopluvial maps for Oregon). For the purpose of this analysis, a large storm event is considered more than 1 inch in 24 hours. Antecedent conditions are considered wet if any one of the preceding four days had a storm event of 0.5 inches or larger, as long as there were not two or more days in a row between storm events with 0.00 inches of precipitation. Those events meeting DEQ criteria were analyzed as shown in Chart 1-3.

Chart 1-3: Flow vs. Rainfall ($PDAF_5$)



After analyzing data, the peak flows for a storm event were determined to most frequently occur the following day after the precipitation reading was recorded. Since not all storm events had the highest associated flow occur exactly one day later, rainfall for a specific day was associated with the largest flow in the next two days following the rainfall. For large, multi-day rain events – where more than one consecutive day met the previously listed conditions for a high rainfall event – the association was chronological. The first large rainfall event for one day was associated with the chronologically first large daily flows within the two days, and so on. The DEQ method $PDAF_5$ was then compared to City DMR records of the previous five years. The peak day flow for the three highest events ranged from 1.65-1.85 MGD. The design $PDAF_5$ was modified to reflect the highest daily DMR flow in the previous five years.

Peak Instantaneous Flow (PIF_5)

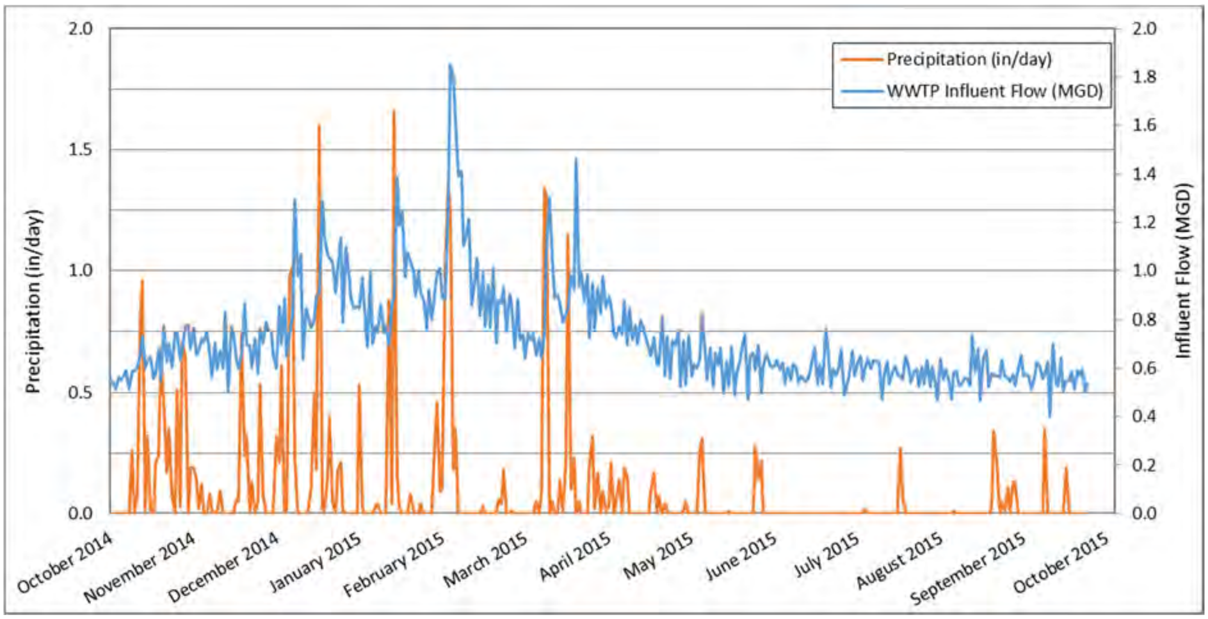
In the absence of hourly flow data, DEQ recommends obtaining the peak instantaneous flow (PIF_5) by extrapolation from their own chart titled Graph #3. On Graph #3, $PDAF_5$, PWkF, $MMWWF_5$, and AADF are all graphed (on specific log-probability graph paper provided by DEQ) versus their probability of occurrence as shown in Appendix B. A line of best fit is then drawn between the points. The PIF_5 is located where that best fit line crosses the 0.011% probability.

The City collects instantaneous flow data (in gpm) on graphical flow charts. Flow charts for the highest daily flows in the last five years were reviewed. Peak instantaneous flows for the three highest events ranged from 2.35-2.64 MGD. The design PIF_5 was based on the flow chart data as it is actual flow measurements recorded at the plant in the previous five years.

Infiltration and Inflow (I/I)

Infiltration and inflow (I/I) is storm water or groundwater that enters the sanitary sewer system. I/I can come from a variety of sources, such as storm sewers connected to the sanitary sewer, storm inflow through manhole lids, and groundwater infiltration into cracked/broken pipelines and services. Visual evidence of I/I can be seen in Chart 1-4, which shows the October 2014-October 2015 daily flows plus precipitation recordings at the WWTP. Peaks in the influent flow occur concurrently with peaks in the rainfall, which indicates rainfall influence on sewer flows. Flows in Chart 1-4 are representative of previous years, which follow the same pattern.

Chart 1-4: 2013 Daily Flow and Precipitation



The peak flow compared to the base flow is an indication of I/I influence in the system. Table 1-7 summarizes annual average base flow and the ratio of peak flow to the base flow for the last five years. During this time, the peak flow ranges from 1.6 to just over 3.0 times the base flow. I/I exists in the system, but is not excessive. Some communities experience peak flows in excess of 10 times the base flow.

Table 1-7: Annual Peak Day Flow / Average Base Flow

Year	Avg Base Flow (MGD)	Peak Flow/Avg Base Flow
2010	0.65	2.32
2011	0.70	2.20
2012	0.76	2.26
2013	0.70	1.60
2014	0.60	2.67
2015	0.60	3.08

The City does not have a formal program to remove I/I; instead, pipeline lining and replacement projects are completed as budget allows. Large defects discovered during CCTV inspection are given priority. New future construction should not experience these specific I/I problems due to the use of newer and more watertight sewer components.

Projected Flows

Future domestic (residential and commercial) flow projections are based on historical flow per capita and projected population growth. Communities often utilize PSU population forecasts to estimate population growth and project future flows. The City anticipates higher growth and population projections than those from PSU (Section 1.3). Table 1-8 compares the projected population and flows for the planning period based on PSU forecasts and City-projected growth.

Table 1-8: Comparison of Projected Flows

	Design Flow (MGD)	Projected Unit Flow (gpcd)	Projected Domestic Flows (MGD) (PSU Moderate Growth Rate Pop. Projections)				City Projected Flows (MGD) (City Estimated Pop. Projections)	
Year	2015	2015	2020	2025	2030	2035	2035 Domestic	2035 Domestic and Industrial
Population	6,745	6,745	7,520	8,262	9,078	9,974	13,188	13,188
AADF	**0.768**	114	0.856	0.940	1.033	1.135	1.501	2.390
ADWF	**0.660**	98	0.736	0.809	0.889	0.977	1.291	2.114
AWWF	**0.860**	128	0.959	1.053	1.157	1.271	1.681	2.628
$MMDWF_{10}$	**0.855**	127	0.954	1.048	1.151	1.265	1.672	2.616
$MMWWF_5$	**1.212**	180	1.352	1.485	1.631	1.792	2.370	3.537
PWkF	**1.397**	207	1.558	1.711	1.880	2.066	2.732	4.013
$PDAF_5$	**1.850**	274	2.063	2.266	2.490	2.736	3.617	5.181
PIF_5	**2.640**	391	2.943	3.234	3.553	3.904	5.162	7.218

Notes:

1. $PDAF_5$ modified based on actual peak day flow from DMR records. DEQ graphical $PDAF_5$ is 1.64 MGD.
2. PIF_5 modified based on City provided flow charts. DEQ graphical PIF_5 is 2.48 MGD.
3. Peaking factor of 2.5 used for industrial flows.

Flow projections for light industrial zoning were estimated separately from domestic flows, as there are no dwelling units or population to correlate with industrial development. A flow rate of 1,250 gpd/acre was used to project industrial flows, as well as strictly commercial areas (Metcaff and Eddie, 3rd Edition) since there is no population associated with commercial development.

Table 1-9 lists City-projected flows for the next 5, 10, 15, and 20 years. The City estimates industrial growth (shown in Figure 6, Appendix A) to be 30% complete in 5 years, 60% in 10 years, 90% in 15 years, and 100% in 20 years. These projected flows are used in this study for future analysis of the collection system and WWTP.

Table 1-9: Projected Flows

	2015 Design Flow (MGD)	Projected Unit Flow (gpcd)	Projected Flows [Domestic and Industrial] (MGD)			
Year	2015	2015	2020	2025	2030	2035
Population	6,745	6,745	9,943	10,924	12,003	13,188
AADF	**0.768**	114	1.176	1.777	2.166	2.390
ADWF	**0.660**	98	1.015	1.563	1.916	2.114
AWWF	**0.860**	127	1.315	1.960	2.382	2.628
$MMDWF_{10}$	**0.855**	127	1.308	1.952	2.372	2.616
$MMWWF_5$	**1.212**	180	1.845	2.663	3.207	3.537
PWkF	**1.397**	207	2.124	3.032	3.640	4.013
$PDAF_5$	**1.850**	274	2.805	3.935	4.700	5.181
PIF_5	**2.640**	391	3.995	5.510	6.549	7.218

Notes:

1. $PDAF_5$ modified based on actual peak day flow from DMR records. DEQ graphical $PDAF_5$ is 1.64 MGD.
2. PIF_5 modified based on City provided flow charts. DEQ graphical PIF_5 is 2.48 MGD.
3. Peaking factor of 2.5 used for industrial flows.
4. Industrial growth shown in Figure 6 projected to be 30% complete in 5 years, 60% in 10 years, 90% in 15 years, and 100% in 20 years.

The City-projected peak flow for 2035 is nearly twice the PSU population projected flows. This has significant impacts on the improvements and required capacity of the collection system and WWTP. The higher projected flows will result in conservative recommendations. It is unknown exactly where growth will occur in the system throughout the planning period. One entire side of town may develop before the other, or development may be more equally distributed. Conservatively, the collection system should be sized to provide adequate conveyance for connection of the upstream sewer basin within the UGB. The trunk lines have a long life expectancy and should be conservatively sized to the projected peak flows in Table 1-9.

The initial WWTP improvements should not be sized to handle the full revised peak flows. Instead, a phased approach for WWTP improvements is recommended (presented in Section 7). Future recorded peak flows and updated flow projections for the plant will trigger subsequent phases of the improvements. Expandability will be a key component, which will allow the WWTP to operate efficiently while planning for future growth.

1.5 NPDES PERMIT

The WWTP discharges treated effluent into the Multnomah Channel, which is a side channel of the Lower Willamette River. The designated beneficial uses for the Multnomah Channel at the outfall are:

- *Water Supply* – Irrigation and livestock watering.
- *Aquatic Life* – Including anadromous fish passage and rearing, salmonid passage and rearing, resident fish, and aquatic life.
- *Recreational* – Including wildlife, hunting, fishing, boating, and water contact recreation.
- *Other* – Aesthetic quality.

The Multnomah Channel in the vicinity of the Scappoose WWTP outfall was on Oregon's 2012 Integrated Report of water quality limited streams for dissolved oxygen, mercury, and temperature.

The City of Scappoose discharges treated effluent under NPDES Permit No. 100677 (Appendix C). Existing effluent limits are summarized in Table 1-10. The City's permit had an expiration date of October 31, 2014. However, even though the permit expired in 2014, it remains in effect as allowed by OAR 340-045-0040.

Table 1-10: Existing NPDES Permit Limits

Parameter	Average Monthly	Average Weekly	Maximum Daily
BOD_5 [1] (May 1 – October 31)	10 mg/L 125 ppd [2] 85% removal	15 mg/L 190 ppd	255 ppd
TSS [3] (May 1 – October 31)	10 mg/L 125 ppd 85% removal	15 mg/L 190 ppd	255 ppd
BOD_5 (November 1 – April 30)	25 mg/L 315 ppd 85% removal	37 mg/L 475 ppd	630 ppd
TSS (November 1 – April 30)	25 mg/L 315 ppd 85% removal	37 mg/L 475 ppd	630 ppd
pH	Daily minimum and maximum between 6.0 and 9.0		
E. coli Bacteria	126/100 mL	--	406/100 mL

[1] BOD_5 = 5-Day Biochemical Oxygen Demand

[2] ppd = pounds per day

[3] TSS = total suspended solids

Keller Associates has communicated with DEQ regarding future permit conditions. Though DEQ is unable to provide specifics at this time, there are a few wastewater constituents that may be included in future NPDES permits.

Ammonia is often found in sewage treatment plant effluent at levels that exceed the State of Oregon water quality standards for toxicity. The NPDES permit includes a defined Regulatory Mixing Zone (RMZ) within which some or all water quality standards can be exceeded. Acute and chronic standards can be exceeded within a Zone of Immediate Dilution (ZID), and RMZ, respectively. To evaluate compliance with the criteria, DEQ performs a Reasonable Potential Analysis (RPA) using a statistical procedure that takes into account the number of data points, alkalinity/pH, temperature, relative flows in the stream, and the available dilution at the ZID and RMZ. An RPA will be performed as part of the NPDES permit renewal. It may be prudent for the City to perform RPAs for ammonia ahead of the permit renewal so as to anticipate future treatment needs.

According to DEQ, no additional oxygen demand can be placed on the Multnomah Channel from January 1st through May 15th. Even though the future effluent flow will be more than is currently permitted (1.515 MGD), the current permitted monthly average BOD_5 load (125 lbs./day) is not expected to increase substantially. However, there is the possibility that the BOD_5 discharge load could be increased prior to January 1st. Also, since the WWTP was built before June 30, 1992, and the City has implemented the required I/I reduction program, the City may again qualify for a suspension of the load limit if the flow is twice the average dry weather flow.

Phosphorus and temperature are not likely to be included in the new NPDES permit. However, more stringent pH limits may be part of the new permit; although, based on previous sampling results, it's unlikely this will require additional treatment.

In the future (beyond the pending new NPDES permit), it is possible that nutrients (nitrogen and phosphorus), mercury, temperature, and low dissolved oxygen levels may be reflected in new limits of NPDES permits. Also, ongoing work on toxic substances, including additional heavy metals, polychlorinated biphenyls (PCBs), and DDT, could also have future effects on the WWTP.

1.6 COMMUNITY ENGAGEMENT

The Scappoose community had the opportunity to engage in the planning process by participating in City Council meetings and a public open house.

2. COLLECTION SYSTEM EXISTING FACILITIES

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

2.1 LOCATION MAP

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

2.2 HISTORY

The majority of the collection system was constructed in 1972 of concrete pipe. Most of the pipe installed since that time has been polyvinyl chloride (PVC). There are small segments of ductile iron and cast iron pipes in the system. Figure 8 (Appendix A) shows the existing system identified by pipe material.

2.3 SYSTEM DESCRIPTION

The wastewater collection system consists of approximately 32 miles of gravity mains, 1.4 miles of force mains, and five lift stations that discharge to the wastewater treatment plant (WWTP). The pipelines range from 4 to 21 inches in diameter. There are approximately 700 manholes in the City's collection system. Figure 7 (Appendix A) illustrates the pipe diameters and Figure 8 illustrates the pipe material in the City's collection system. The topography of the City and the elevation of the WWTP require more than half of the City's wastewater to be pumped to the WWTP.

2.4 CONDITION OF EXISTING LIFT STATIONS

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

Table 2-1: Lift Station Inventory

	Smith Road	Spring Lake	Keys Landing	Highway 30	Seven Oaks
PUMP STATION					
Type	Wet-well, Dry-well, triplex pump system	Wet-well, Surface-mounted, duplex pump system	Wet-well, Surface-mounted, duplex pump system	Wet-well, Surface-mounted, duplex pump system	Wet-well, Surface-mounted, duplex pump system
Pump Type	Vertical, variable speed, non-clog centrifugal (Crane Deming model 7196-4x4x12x3)	Horizontal, constant speed, self-priming centrifugal (Hydromatic model #181T)	Horizontal, constant speed, self-priming centrifugal (Hydromatic model #183)	Horizontal, constant speed, self-priming centrifugal (Hydromatic model #183)	Horizontal, constant speed, self-priming centrifugal (Hydromatic model #183)
Capacity* (gpm)	Each pump: 1,100 gpm @ approx. 28 ft. TDH	Each pump: 100 gpm @ approx. 24 ft. TDH (with 9 ft. suction lift)	Each pump: 120 gpm @ approx. 46 ft. TDH (with 13 ft. suction lift)	Each pump: 150 gpm @ approx. 42 ft. TDH (with 19 ft. suction lift)	Each pump: 150 gpm @ approx. 45 ft. TDH (with 16 ft. suction lift)
Pump HP (each)	15hp @ 1200 rpm (460V, 60 Hz, 3 ph)	3hp @ 1150 rpm (230V, 60 Hz, 1 ph)	5hp @ 1750 rpm (230V, 60 Hz, 3 ph)	7.5hp @ 1800 rpm (230V, 60 Hz, 3 ph)	7.5hp @ 1750 rpm (230V, 60 Hz, 3 ph)
Level Control Type	Pressure level transducer in wet well	Four (4) Mercury level sensors	Four (4) Mercury level sensors	Four (4) Mercury level sensors	Four (4) Mercury level sensors
Overflow Point	Existing manhole just west of pump station	At pump station	At pump station	At pump station	At MH just north of pump station
Overflow Discharge	To creek west of pump station	To Spring Lake	To residences near pump station	To storm system at HWY 30 and then into Jackson Creek	To residences near pump station
Auxiliary Power Type	Permanent diesel generator	Portable generator	Portable generator	Portable generator	Portable generator
Location	inside pump station	At WWTP	At WWTP	At WWTP	At WWTP
Output	60 kW	17 kW	17 kW	17 kW	17 kW
Fuel Tank Capacity (gal)	120	100	100	100	100
Transfer Switch	Automatic	Manual	Manual	Manual	Manual
Alarm Telemetry Type	Autodialer and Red Light Outside	Autodialer and Red Light Outside	Autodialer and Red Light Outside	Autodialer and Red Light Outside	Autodialer and Red Light Outside
Originally Constructed	1972	1993	1996	1996	1996
Year(s) Upgraded	2001	2001 and 2003	NA	NA	NA
Wet Well Diameter (ft)	6	6	6	8	7
Wet Well Net Storage (gal)	634	528	528	2,237	1,315
FORCE MAIN					
Length, Type	Approx. 1,600 ft. of 12-inch Ductile Iron	Approx. 1,726 ft of 4-inch PVC	Approx. 550 ft of 4-inch PVC	Approx. 250 ft of 6-inch C900	Approx. 1,565 ft. of 4-inch C900 PVC
Profile, Continuously Ascending (Yes/No)	Yes	Yes	Yes	Yes	No
Discharge Location	MH at intersection of Burlington Northern RR and Laurel Street	MH at 6th Street and Seven Oaks Drive	MH on SW Keys Road at end of SW Keys Landing Way	MH on SW Dutch Canyon Road off Columbia River HWY	MH at 6th Street and Seven Oaks Drive
Air Release Valves	None	None	None	Yes	Yes
Vacuum Release Valves	None	None	None	Yes	Yes
Sulfide Control System	None	None	Yes	Yes	Yes

** Capacity as reported in O&M Manuals.*

Smith Road Lift Station

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

Smith Road Lift Station

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

Keller Associates analyzed the pump run time data from the past three years to assess capacity. If the lift station runs two pumps for more than 20 hours per day, or if it cannot pass the peak instantaneous flow with one pump offline, the lift station is considered to be undersized. During the last three years, the maximum pump run time was 19 hours/day. Based on model results, the current peak flow through Smith LS during a 5-year, 24-hour storm is 2700 gpm. The firm capacity of the lift station should be able to handle the peak flow. Each pump in the lift station is rated for 1100 gpm, the firm capacity of the LS is stated as 2200 gpm in the Operation and Maintenance (O&M) manual. Pump tests should be completed to verify the firm capacity of the lift station. Based on the information provided, the lift station is undersized for current peak design flows.

The operational volume of the lift station is approximately 425 gallons. The duty pump runs for 1-2 minutes under normal flow conditions to empty the operational volume. During peak flow hours, the pump runs roughly every 10 minutes. The pump runs approximately once an hour during the lowest flows of the day.

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

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

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

Deficiencies

- Undersized for existing peak flows.
- Small operating volume.
- Backs up into gravity line.
- Access hatch is heavily corroded.
- Grease builds up.
  - Heavy grease ring in wet well.
- No pre-screening.
- Exterior red light and audible alarms have not been tested.
- One of the blowers for the fan in the dry well does not work.
- Bypass is missing cap for cam fitting.

Recommendations

- Increase pumping capacity or divert flow away from the lift station.
- Replace the existing Smith Road lift station wet well to accommodate higher inflows.
- Adjust operational set points to prevent backups into the collection system.
- Clean and coat the access hatch for corrosion control.
- Develop pre-treatment program to prevent grease from reaching the collection system.
- Install screen ahead of pumps in Smith Road lift station.
- Test red light and audible alarm. Perform necessary maintenance.
- Replace drywell blower.
- Replace bypass cap.
- Perform a review of the electrical system and assure all equipment meets compliance standards.

Spring Lake Lift Station

Spring Lake Lift Station

The Spring Lake Lift Station is located near the intersection of Westlake Drive and SE 6th Street. The lift station is a prefabricated package style unit and was put into service in 1993. The lift station consists of the wet well with four float switches, two surface-mounted pumps, and a control panel. In 2001, both pumps were replaced and in 2003, the power transfer switch was replaced. Pump 2 had the impeller, wear plate, and cutter replaced recently.

The gravity lines draining to the lift station are all private lines except one 8" line. The pumps are controlled by the float switches using a lead on, lag on, and pump off operational strategy. The top-most float switch is used to trigger a high level alarm. The floats have not been tested recently. It is recommended the floats are tested soon. Mission Controls sends alarms to the operator. A red alarm light is also located on the outside of the lift station, though it has not been tested recently. In the event of a power outage, an alarm is sent via Mission Controls. The lift station has a manual power transfer switch with receptacle. In the event of a power failure, a 17 kW portable generator, normally housed at the WWTP, can be transported to the site and connected to the receptacle for backup power.

The O&M Manual states that the pump capacity is 140 gpm. In the last three years, the maximum pump run time in one day was 13.8 hours. This indicates the pump station is not undersized for the current demand. The long run time could be a symptom of the poor performance of the pump and may decrease when the station is upgraded. There have been no known issues with the lift station overflowing or with pumps running continuously for an extended period of time. The lift station overflow discharges directly into Spring Lake. Spring Lake and any in-stream activities immediately downstream would be negatively affected, if an overflow occurred.

The lift station often does not operate well and alarms are consistently sent to City staff. Pump 1 runs poorly. Pump 2 does not stay primed and pumps poorly when it does run. Operators must go to the lift station often and prime the pump. Grease buildup in the wet well has prevented the level floats from operating properly. City staff also report heavy paper products in the wet well. Kennedy-Jenks Consultants completed a pump station upgrade pre-design report in 2009. The cost estimate for design and construction was not within the City budget at the time and the project was put on hold.

During this study, City staff noted that the force main has a high point prior to discharging to the gravity collection system. Keller Associates staff recommended testing the pump station for a potential air lock scenario occurring in the force main, thus causing the pumps to lose prime and/or operate outside of their normal efficiencies. Keller Associates recommends performing these tests prior to moving forward with design and construction of upgrades to the lift station. The City has purchased pressure gauges for installation to test for an air lock.

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

Deficiencies

- Pump 1 pumps poorly.
- Pump 2 does not stay primed; pumps poorly.
- Deflection on discharge pipe at coupling.
- Probable air lock in force main.
- Insufficient power supply.
- Heavy corrosion on wet well piping.
- Grease has prevented floats from working in past.
- No level readout or human machine interface (HMU) for modifying settings.

Recommendations

- Perform test to verify there is no air lock in the lift station force main. If there is an airlock, then install an air release valve. If there is no air lock, replace the pumps/pump station at Spring Lake.
- Provide adequate power supply with a 3-phase service.
- Clean and coat piping in wet well and valve vaults as necessary.
- Establish pre-treatment program to prevent grease from reaching the collection system.
- SCADA upgrades to include level readout and HMU at LS.
- Perform a review of the electrical system and assure all equipment meets compliance standards.

Keys Landing Lift Station

Keys Landing Lift Station

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

The operation and alarms for the lift station are identical to the Spring Lake Lift Station. The lift station has a manual power switch with a receptacle. In the event of a power failure, a 17 kW portable generator, normally housed at the WWTP, can be transported to the site and connected to the receptacle for backup power. During a power outage, City staff estimates they have approximately an hour before backup power is required to avoid overflows. The system model indicates there is much more time than this before an overflow would occur. There have been no known issues with the lift station overflowing or with pumps running continually for an extended period of time. In the three-year pump run time history analyzed, the maximum run time was 2 hours a day. This indicates the lift station has adequate capacity.

Currently, the lift station is only running one pump because the second pump loses prime. The O&M Manual states that the pump capacity is 120 gpm. There have been some odor complaints for this lift station. The lift station overflow, at the nearest gravity manhole, is currently plugged to prevent odors. City staff report that due to site conditions, it can be difficult to get to the lift station in the winter to hook up the portable generator in the case of a power outage. The lift station is located relatively close to the Keys Landing Water Treatment Plant (WTP). City staff have discussed using stand-by power available at the WTP for this lift station.

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

Deficiencies

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

Recommendations

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

Highway 30 Lift Station

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

Highway 30 Lift Station

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

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

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

<u>Deficiencies</u>

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

<u>Recommendations</u>

- Use pretreatment program to address grease buildup.
- Clean and coat piping in wet well and valve vaults as necessary.

- Install bollards or approved ODOT traffic barriers to protect station from traffic collisions.
- Perform a review of the electrical system and assure all equipment meets compliance standards.

Seven Oaks Lift Station

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

Seven Oaks Lift Station

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

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

Deficiencies

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

Recommendations

- Repair insulation on bottom of clam shell covering.
- Clean and coat piping in wet well and valve vaults as necessary.
- Use pretreatment program to address grease buildup.
- Perform a review of the electrical system and assure all equipment meets compliance standards.

2.5 CONDITION OF EXISTING COLLECTION SYSTEM PIPELINES

2.5.1 Gravity Mains

Table 2-2 summarizes the size, material, and length of each type of gravity sewer line. The type of sewer pipe is a good indicator of the age of the sewer lines. Most of the concrete pipes in the City were installed in the 1970's and the PVC pipes were installed in the 1990's and more recently.

Table 2-2: Pipe Type and Size Summary

Pipe Diameter (in)	Pipe Material Lengths (ft)					Total by Diameter (ft)	% of Total
	Concrete	PVC	Cast Iron	Ductile Iron	Unknown		
4"		250				*250*	*0%*
6"	1,272	1,847		89	303	*3,500*	*2%*
8"	62,086	69,104	223	983	4,658	*137,100*	*81%*
10"	3,593	3,295	100	20		*7,000*	*4%*
12"	7,436	3,095	92			*10,600*	*6%*
15"	5,611	201				*5,800*	*3%*
18"	2,314			387		*2,700*	*2%*
21"		2,707				*2,700*	*2%*
Total by Material (ft)	*82,300*	*80,500*	*400*	*1,500*	*5,000*	*170,000*	*100%*
% of Total	*48.41%*	*47.4%*	*0.2%*	*0.9%*	*2.9%*	*32.2*	*MILES*

Approximately 48% of Scappoose's wastewater pipelines are concrete, with the remainder PVC. Concrete pipe is susceptible to corrosion, while PVC is inert. Eventually, all concrete pipe should be replaced.

Inflow and infiltration

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

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

Smoke Testing

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

Keller Associates worked with City staff to smoke test sections of the gravity sanitary sewer pipelines in the City between July 20th - 27th, 2016. Areas of the City that were smoke tested are shown on Figure 9 in Appendix A. 96,300 linear feet of pipelines were smoke tested.

Smoke introduced into the sanitary sewer system should only be released from nearby manhole and cleanout pick holes and plumbing vents on buildings. Smoke found to be emitting from other locations is a sign that water may be entering the sanitary sewer system in these locations. Overall, there were not a lot of unexpected locations found to be smoking. The problem areas located during smoke testing are summarized in Appendix E, along with photographs for each identified problem area. Figure 9 shows where each problem identified is located. Roughly half of the problems found while smoke testing were related to open or broken cleanout caps. There were a few cross connections identified, including roof and driveway drains. In addition, a few manholes were smoking around the rims or elsewhere on the manhole collar. There were a handful of cases where smoke was found inside houses; most of these were likely dry P-traps. The issues found are summarized in a table in Appendix E, which includes recommended action to correct identified problems.

2.5.2 Steinfeld Plant's Gravity Line

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

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

2.6 COLLECTION SYSTEM COMPUTER MODEL

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

2.6.1 Model Selection

XPSWMM 2016, version 17.0 [12.0] build December 9, 2015, was selected as the modeling software for this project. XPSWMM is a fully dynamic model which allows for evaluation of complex hydraulic flow patterns.

2.6.2 Model Creation

GeoSolve, Inc. is a GIS consulting firmed hired by the City to maintain the Scappoose GIS database. Information from this database was used to populate pipe diameter and invert elevation data in the model. In places where the GIS was missing data, record drawings were consulted and, when necessary, field investigations were performed by the City or Keller staff. When discrepancies arose, field investigations were assumed to be more accurate than the GIS database, which is assumed to have been populated from record drawings.

Trunk lines with diameters of 10-inches and larger were modeled. Any pipes that connect the trunk lines together were also modeled regardless of their diameter. Figure 10 in Appendix A shows the modeled lines in the system. Once all manholes and pipes were created and data populated in the model, several queries were conducted to reveal anomalies in the data. These included reverse slope pipes, changes in pipe size, and uncommon configurations in the pipe network. Any anomaly was discussed with City personnel, additional field work was completed, and appropriate changes were made to the model.

Lift Stations

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

2.6.3 Model Calibration

Model loads refer to the wastewater flows that enter the sewer collection system. These loads are comprised of wastewater collected from individual services (base flows), plus groundwater infiltration and storm water inflows (I/I). As part of this study, flow monitoring was completed during the wet weather period from January 2016 into February 2016. Flow monitoring data was collected at various manholes throughout the system to help calibrate the model. Five monitoring sites were selected, dividing the system into basins. Figure 11 in Appendix A shows a map of flow meter locations and basins. The basins were used to characterize flows throughout the system. The collected data was analyzed along with continuous precipitation data to establish typical 24-hour patterns, average flows at each site, and gauge rainfall influence in the system. Both dry weather and wet weather periods were used for loading and calibration efforts. Loads for the model were developed and calibrated in several stages as described below.

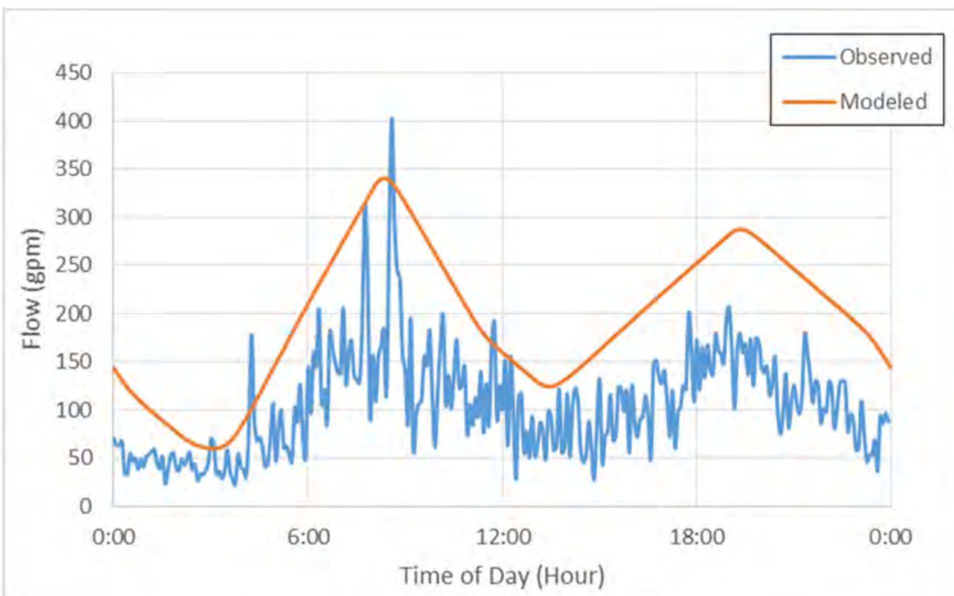
Dry Weather Calibration

For a starting point, base flows were estimated using winter potable water consumption data (winter water meter readings provided by the City were averaged). Individual water meter records for customers in Scappoose were linked to the sewer model using GIS to provide a highly accurate distribution of potable water demands. Dry weather wastewater flows were assumed to be 90% of winter potable water demands. Those properties that did not have consumption data were assigned a base flow based on the winter consumption data of similar properties in the surrounding area. A winter month average was used because it is most likely to exclude additional usage for irrigation that would not return to the sewer collection system. An average dry weather flow was assigned to each modeled manhole based on the consumption data.

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

In addition to calibrating the model at various locations within the collection system, total modeled influent flows at the Wastewater Treatment Plant (WWTP) were also compared to the targeted design average daily flow. Appendix F contains a summary of the data and analysis used for modeling purposes. An example of calibration results are shown in Chart 2-1.

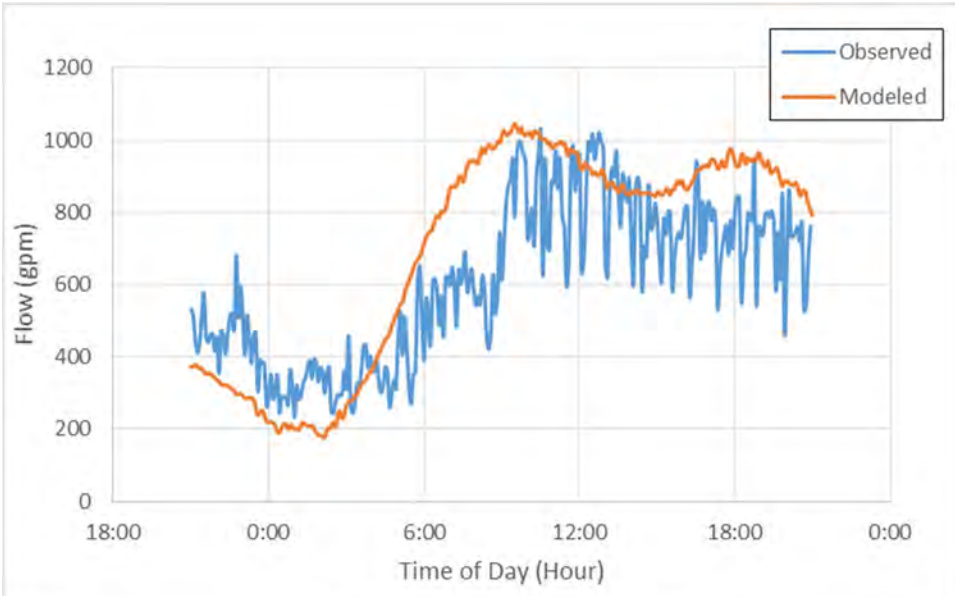
Chart 2-1: Sample Dry Calibration Site 1 – Modeled vs. Observed Flows (MH0484)



Wet Weather Calibration

The RTK method was used for rainfall-derived infiltration and inflow (RDII) prediction. Rainfall data for a 24-hour period with the highest cumulative and highest intensity rainfall during the period of flow monitoring was utilized to calibrate wet weather flows. The storm event was entered into XPSWMM. RTK parameters were then adjusted to match model results with flow monitoring data. At one monitoring location, the model dry weather flow (DWF) was higher than the wet weather flow (WWF) monitoring data because of adjustments made to DWF to match daily average flow to WWTP data. In this instance, the percent difference between peak DWF and peak WWF in the flow monitoring data was applied to the model peak DWF to estimate the target peak WWF. Again, total modeled influent flows at the WWTP were compared to the targeted design average daily flow in addition to calibrating the model at various locations within the collection system. Example calibration is shown in Chart 2-2.

Chart 2-2: Sample Wet Calibration Site 5 – Modeled vs. Observed Flows (MH0537)



Design Storm

The design storm for model evaluation was the 5-year, 24-hour storm event. A standard 24-hour NRCS rainfall distribution for a Type 1A storm was used. The rainfall for the 5-year, 24-hour storm event from NOAA isopluvial maps is 2.7 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 and peak flow at the WWTP was compared to the design PIF_5 (Table 1-9). The peak flows matched; no further calibration was performed on the model.

2.6.4 Existing System Capacity Limitations

The calibrated model was exercised to determine the effects of a 2015 peak day flow event on the system. Figure 12 in Appendix A illustrates the available capacity of the existing system. The figure is color-coded to show a gradation of pipes based on utilized capacity (e.g., red = flowing at >100% capacity, orange = flowing at 80-99% of capacity, yellow = flowing at 70-79% capacity, etc.). Those sections shown in red experience pipeline surcharging and represent the greatest risk for backing up services and possible overflow sites. The majority of pipes nearing or at capacity are located on the main trunk line on the western side of town. There are also a number of pipes nearing or at capacity on the Columbia Avenue trunk line near the WWTP and where the eastern trunk line connects to it.

It should be noted that some of the pipelines showing >100% capacity resulted in sanitary sewer overflows or surcharging at manholes. Those locations have been noted on Figure 12. Although present in the model, overflows at these locations have not been observed by City staff, potentially due to the extra storage available in lateral lines which were not modeled. Surcharging in these locations has been noted by City staff and Keller Associates recommends continued monitoring and investigations, especially during high flow events, to determine the actual extents of any flooding that occurs.

Low-Velocity Areas

The City's existing collection system model was used to evaluate the maximum velocity achieved during the peak hour flow for a 2015 design storm event. This flow represents the flushing velocity of a typical day. Figure 13 in Appendix A illustrates the resulting velocities for the existing gravity collection system. The recommended minimum velocity is 2 feet per second (fps). Approximately 37% of the linear footage for the existing system has velocities below 2 fps.

Low velocities can result in accumulation of material, increasing the risk of upstream surcharging and overflows. It will also increase generation of hydrogen sulfide in the collection system. This can lead to aggressive corrosion on the system and is a significant hazard to the system operators. It is recommended that the City monitor the accumulation of debris in these areas to determine if a more aggressive sewer line cleaning schedule is warranted. It should be noted again that the velocities provided in Figure 13 are for the 2015 design peak instantaneous flows. Most of the time, velocities are likely lower and more prone to accumulate material.

2.6.5 Pipeline Conditions

In-field pipeline material condition inspection and review were not included as part of this report. However, it is important to note that one of the basic assumptions of the hydraulic model is that all of the lines are free from physical obstructions such as roots and accumulated debris. Such maintenance issues, which certainly exist, must be discovered and addressed through consistent maintenance efforts. The modeled capacities discussed in this chapter represent the capacity assuming the sewer lines are in good working order.

2.7 FUTURE COLLECTION SYSTEM PERFORMANCE

This section summarizes future flow projections and the model evaluation of future system expansion, and documents anticipated future deficiencies. Improvements to address these deficiencies are presented in Chapter 5.

2.7.1 Future Flow Rate Projections & Model Scenarios

Future loads were distributed based on City projected future residential, commercial, and industrial growth. The projected growth areas are shown in Figure 6. Flows per capita for projected population growth were assumed to be similar to existing flows per capita. Domestic flows (residential and commercial flows) were projected using future growth area, average lot size, population density, and ADWF per capita. Additional flow was allocated for future large industrial users as directed by City staff. WTP flows (for both Miller Road Plant and Keys Landing Plant) were assumed to remain the same in the future. The flows used in the existing system are conservative and it would be recommended that the water treatment plants begin to recycle the wash water to reduce water use, which would decrease their flow contribution to the sewers. Projected flows are presented in Table 1-9 and Section 1.4 provides a detailed outline of the process used to calculate the projected flows.

Flow for each growth area identified in Figure 6 was added to the closest modeled manhole to allocate future flows. Additionally, four lift stations and two different gravity mains were added to the model to handle future flows. This infrastructure is identified in Figure 6. Table 2-3 summarizes the estimated peak influent flows calculated for each proposed lift station.

Table 2-3: Proposed Lift Station Summary

Proposed LS	Est. Peak Flow (gpm)
P.LS 1	90
P.LS 2	600
P.LS 3	630
P.LS 4	185

Proposed lift station P.LS 2 is located and sized to handle future industrial flows (Figure 6). The potential land use and development of the land within the UGB and to the west of the lift station should be considered during the design process of this lift station. The Buildable Lands Study that is in process now, should also be taken into account during the design of any of these proposed lift stations. The future model was run to analyze the effects of future growth on the system out to the year 2035.

2.7.2 Future System Capacity Limitations

Modeling results show that the majority of pipelines that do not have sufficient capacity for future growth flows are also undersized to meet current flow rates. The east trunk line, running from High School Way to just south of Columbia Ave, has sufficient capacity for current flows, but needs to be upsized for future flows.

There are three sections on SW Old Portland Road that are adequate for existing flows, but are at capacity for future flows. These pipes surcharge a few inches above pipe crown during the 2035 projected 5-year, 24-hour event. These sections should be monitored and be upsized if problems arise. The most recent CCTV inspection of this line show there are a number of sags along the line. If other City projects, roadway or otherwise, are performed on this section of road, then upsizing the sewer pipes should be added to the project to share costs. Figure 21 illustrates the available capacity of the future system after all improvements have been made, not including those on Old Portland Road.

2.8 CITY ORDINANCES & PRETREATMENT

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

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

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

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

2.9 FINANCIAL STATUS OF EXISTING FACILITIES

See Section 7.6 for the financial status of existing facilities.

2.10 WATER/ENERGY/WASTE AUDITS

No water, energy or waste audits have been created at this time.

3. WASTEWATER TREATMENT PLANT EXISTING FACILITIES

This section contains a description and evaluation of the existing wastewater treatment plant (WWTP) for the City of Scappoose.

3.1 LOCATION MAP

A map of the existing WWTP is included in Figure 14 (Appendix A). A schematic process layout of the WWTP is located in Figure 15 (Appendix A). The wastewater facilities are all located within the City limits.

3.2 HISTORY

The original treatment plant was constructed in the early 1970s with a design capacity of 0.5 MGD. In 1994, the WWTP was expanded to handle 1.515 MGD to accommodate population growth and wastewater discharged from the Steinfeld's Products Company factory. However, in 2001 the Steinfeld's factory closed, which opened up some capacity to the WWTP for future population growth. An influent screen and tertiary filters were added to the WWTP in 2010.

3.3 SYSTEM DESCRIPTION

The wastewater collection system discharges to the WWTP. The WWTP is an extended aeration activated sludge system. The City's wastewater flows into the Headworks where the influent is screened. (Screenings are placed in a dumpster and periodically taken to the landfill.) Following screening, the wastewater is pumped to a channel where influent samples are taken and the flow is measured before entering the aeration basin. Floating aerators provide oxygen to treat the wastewater in the aeration basin. Following the aeration basin, the wastewater flows by gravity to secondary clarifiers for solids removal. From the clarifiers, the flow is pumped to cloth media filters for further solids removal prior to being disinfected in the channel ultraviolet (UV) system. The effluent is sampled and effluent flow is measured near the UV system.

The treated wastewater then flows by gravity to the Effluent Pump Station where it is pumped approximately one mile to the Multnomah Channel. The effluent is discharged into the channel underwater through a single-port diffuser, which helps distribute and mix the effluent with the channel flow.

Solids removed in the clarifiers are either recycled to the aeration basin (return activated sludge [RAS]) or pumped to the aerobic digester (waste activated sludge [WAS]). The waste solids generated in the activated sludge process are treated in the aerobic digester basins (which are the old aeration basins that were constructed in the 1970s). Following digestion, the sludge is pumped (by a sludge pump located in the Headworks) to the sludge storage lagoons. The digested sludge remains in the storage lagoons until it can be applied to nearby agricultural fields. The City has negotiated contracts and received DEQ approval for up to 200 acres to be used for solids application.

3.4 CONDITION OF EXISTING FACILITIES

3.4.1 Headworks

The Headworks is located on the south side of the WWTP adjacent to the WWTP Office, and includes influent screening and pumping. Wastewater flows into the WWTP through an 18-inch sewer line. A new Huber Rotamat® RoK 4 influent screen with 0.25-inch mesh openings was installed in August 2010. The rated capacity of the unit is 4.1 MGD, which according to growth projections will be reached in approximately 5 years. Screenings from the unit are automatically washed, bagged and deposited into a dumpster adjacent to the screen. Since the screen was installed, the operating staff have noticed a significant decrease in solids deposited in the aeration basin. However, the WWTP does not have a grit removal system following the influent screen, which would provide additional solids removal. There is no second influent screen for redundancy. If the screen needs to be taken down for maintenance, temporary pumps and hoses are placed in a manhole outside of the WWTP and the wastewater is pumped directly to the aeration basin.

Influent Screen

The screen discharges wastewater directly into an influent wet well. The influent screen is covered, but there are two open sides on the Headworks building in this area. This protects the screen from snow and rain; however, there is potential for freezing. In addition to the screened influent, the wet well in the Headworks receives supernatant from the Aerobic Digester plus plant drain water (although not storm water). The WWTP does not accept septage. A small amount of ozone from a 3 HP DO_2E ozonator is introduced into the wet well in order to reduce the buildup of grease in the Headworks. Although there is less buildup of grease in the Headworks, operations staff have noticed that grease is still getting through and is now accumulating in the aeration basin and clarifiers.

Four (4) 15 HP vertical influent centrifugal pumps are located in a dry well in the Headworks. The influent pumps pull from the wet well and pump the wastewater to the aeration basin. The pumps activate based on the water level, measured by a bubbler in the wet well. The influent pump variable frequency drives (VFDs) increase the pumping speed in order to maintain a set water level in the wet well. A high level float is used to trigger an alarm. The air compressor used for the bubbler needs to be repaired.

According to the record drawings, each pump has a capacity of 750 gpm at 40 ft. total dynamic head (TDH). The firm capacity (the capacity with the largest unit offline) of the influent pumps is 2,250 gpm, which is expected to be exceeded in the next 5 years. One of the pumps (Pump #3) is used much more than the others (~80% of the time). This is due to the Hand/Off/Auto selector switches not all working, each pump having its own electrical control panel, and the pumps not automatically cycling to equalize run times.

All of the pumps were rebuilt within the last 5 years, but they are worn and likely need to be replaced.


Influent Pumps

An overhead crane is located above the influent pumps for maintenance and removal. A sump pump, located in the southwest corner of the dry well, is used to remove water and pumps it to the wet well. It has been replaced twice in the past 10 years. There was also a large amount of moisture near the control panels. It is also likely that the number of air exchanges in the Headworks do not meet NFPA 820 requirements for an unclassified environment.

A permanent 500 kW diesel generator with automatic transfer switch is located near the Sludge Pumping Building. This generator powers the Headworks equipment if there is a loss of power.

Deficiencies

- Grit is accumulating in the aeration basin.
- Grease is moving downstream to the secondary clarifiers.
- The bypass manhole is outside of the fenced WWTP grounds.
- There is no freeze protection on the influent screen.
- There is no second influent screen for redundancy and the existing influent screen may not be capable of screening the peak instantaneous flow in approximately 5 years.
- The bubbler air compressor needs to be repaired.
- The influent pump controls are not linked, so the pumps run unequally.
- In less than 5 years, the influent pumps may not be capable of providing redundancy at the peak instantaneous flows.
- Control panels are exposed to water and some components are no longer functioning.
- The sump pump does not have installed redundancy.
- Equipment and HVAC system in the Headworks likely do not meet NFPA 820 requirements for an unclassified environment.

Recommendations

- Add grit removal downstream of the influent screen.
- Use pretreatment program to address grease.
- Add freeze protection to the influent screen.
- Add a second influent screen for redundancy and a third influent screen to assist in handling the future peak instantaneous flow. This would also remove the need for a bypass manhole.

- Replace the bubbler with an ultrasonic or pressure transducer level sensor.
- Install a new combined control panel for the influent pumps with pump alternator to equalize pump utilization.
- Add capacity by changing out the influent pumps.
- Make repairs to Headworks to protect the electrical components from water.
- Upgrade the equipment and HVAC system to comply with current NFPA 820 requirements for an unclassified environment.

3.4.2 Aeration Basin

The wastewater is pumped by the influent pumps in the Headworks through a 12-inch pipe into a channel adjacent to the aeration basin, where the flow is measured and influent samples are collected. A 9-inch Parshall flume with an Ametek® Drexelbrook USonic-R™ ultrasonic level sensor is used to measure the influent flow. The 9-inch Parshall flume has a capacity of 5.73 MGD, which is not sufficient to measure the flows for the entire 20-year planning period. An ISCO Model 6712FR refrigerated composite sampler is programmed to collect influent samples. The composite sampler can be paced to collect samples based on the influent flow measurements. The ultrasonic level sensor and composite sampler are connected to a Mission Control system in the plant control panel, which notifies City staff of alarm conditions, including power failure and high/low levels.

RAS from the clarifiers enters at the end of the channel prior to the flow entering the aeration basin. A large amount of grit accumulates in the channel and must be removed so as not to affect the influent sample or the influent flow measurement.

Aeration Basin

The current aeration basin, constructed in 1994, is an HDPE-lined lagoon basin. The aeration basin is approximately 200 ft. long x 100 ft. wide x 12 ft. deep, and has a volume of approximately 1.9 million gallons. Three (3) 40 HP surface aerators and six (6) 10 HP DO_2E high volume floating aerator/mixers provide oxygen for the activated sludge process. One of the 10 HP DO_2E units recently failed and is being replaced with a 40 HP Aire-O_2 Triton® aerator/mixer with a 7.5 HP aspirator blower. Two (2) dissolved oxygen (DO) probes monitor the DO concentrations and data is sent to the WWTP

Office via Bluetooth® technology. The DO_2E floating aerators are manually turned off/on, depending on the DO measurements in the aeration basin. The remaining aerators, however, are left on continuously in order to provide basin mixing.

The DO_2E floating aerator/mixers were a relatively new technology at the time they were installed and claimed to reduce energy usage. Though the operators report that they have done a good job at decreasing the required aeration power, they have also required a significant amount of maintenance. The DO_2E floating aerators fill up with solids until the pressure is too great for them to work. A crane is required to lift them out of the basin while a crew on a boat hoses the solids out. Currently, the DO_2E floating aerators are cleaned every 3 months. This expensive maintenance has essentially offset the cost savings from using less electricity.

The aeration basin is operated as an extended aeration process, as the average hydraulic retention time (HRT) in the basin at current maximum month conditions is approximately 37 hours. The keys to a well-functioning aeration basin are the ability to maintain dissolved oxygen (DO), provide adequate solids retention time (SRT), and produce settleable solids. The SRT measures how long the mixed liquor remains in the basins and is also an indicator of the relative settleability of the mixed liquor and its ability to nitrify. In cold weather, an SRT near 12 days is normally necessary to consistently nitrify. The SRT at Scappoose typically varies between 50 and 70 days, which is sufficient for nitrification.

While Scappoose does not currently have a permit limit that requires nitrification, the plant is an extended aeration process, which should be able to nitrify. As discussed in Section 1.5, an ammonia limit may possibly be added in the future, requiring continuous nitrification. For these reasons, the ability of the Scappoose WWTP to continually achieve nitrification was evaluated. Nitrification requires a longer SRT and more aeration than is required for carbonaceous removal. Typically, a longer SRT leads to more settleable solids in the secondary clarifier and better effluent quality; however, a long SRT can also lead to filamentous bacteria growth (such as Microthrix parvicella), which can affect settling. It is normally desirable to maintain 2.0 mg/l DO in the aeration basins to ensure adequate oxygen is available for metabolism of the influent organic matter (BOD) by the microorganisms in the process and for nitrification (which is typically unavoidable in an extended aeration process).

The four (4) 40 HP surface aerators and the five (5) DO_2E floating aerator/mixers have a combined firm capacity (with one of the 40 HP surface aerator units out of service) of approximately 4,800 lbs. oxygen (O_2)/day. Assuming influent concentrations of BOD_5 of 194 mg/L and total Kjeldahl nitrogen (TKN) of 40 mg/L, a peaking factor of 1.25, and aeration requirements of 1.2 lbs. O_2/lb. BOD_5 and 4.6 lbs. O_2/lb. TKN, the existing aeration system has firm capacity to handle a maximum flow of approximately 1.1 MGD. This means that the aeration system is currently at capacity.

Since there is only one aeration basin, if maintenance is required or there is a process upset, the wastewater will be transferred directly from the influent pump wet well to the secondary clarifiers. This lack of redundancy was one of the reasons for installing the tertiary filters. The wastewater exits the aeration basin over an adjustable 5-foot long, sharp-crested weir on the north side of the basin.

If there is a loss of power, a permanent 500 kW diesel generator with automatic transfer switch near the Sludge Pumping Building can power the aerators. However, the aerators require a manual restart once power is restored (either main power or generator) and there is no automatic alarm notifying the operators of the power outage at the WWTP.

See Section 3.4.16 and Section 3.5 for a discussion on water quality and treatment aspects of the aeration basin.

Deficiencies

- With only one aeration basin, maintenance is extremely difficult as the basin needs to constantly be in operation.
- The basin is not being thoroughly mixed. There have been noticeable solids deposits in some areas of the basin, an indication that the entire basin volume is not available for treatment.
- The aeration system, particularly the DO_2E floating aerators, requires a lot of maintenance, which is costly and dangerous for the operators.
- There is not sufficient room around the basin for maintenance activities.

Recommendations

- Replace the aeration basin with at least a two-basin secondary treatment system to allow a basin to be taken offline for maintenance.
- The secondary treatment basins should have flat walls rather than slopes, so that the basins can be more easily mixed and equipment can be accessible.

3.4.3 Secondary Clarifiers

There are two (2) Eimco 50-foot diameter secondary clarifiers, consisting of a scraper and skimmer mechanism in a circular concrete tank. The effluent weir is steel and a concrete effluent channel is located on the outer edge of the clarifier tank. Wastewater from the aeration basin flows through two (2) 18-inch pipes, one to each clarifier. There are sluice gates to control the flow, but these require manual adjustment. Both clarifiers are center feed, with mixed liquor from the aeration basin distributed to the clarifier via a series of distribution ports on the center column. The clarifiers have a 15-foot side water depth and were constructed in 1994. According to the operators, the clarifiers are in good mechanical condition.

The scum that is removed from the surface of both clarifiers moves by gravity to a combined Scum Pump Station (60-inch precast manhole). The scum is then pumped by a 5 HP chopper pump into the 4-inch WAS pipe going to the Aerobic Digester. There are two (2) float switches in the Scum Pump Station to trigger the scum pump to turn on and off.

Secondary Clarifiers

If there is a power loss, the clarifiers will be powered by the 500 kW generator near the Sludge Pumping Building. The wiring to the clarifiers is a concern as the WWTP has experienced several shorts.

The hydraulic capacity of the secondary clarifiers is based on overflow rates of 400-700 gpd/sf for average conditions and 1,400 gpd/sf for the peak hour (Metcalf & Eddy, Wastewater Engineering, 4th Edition). At current flows, secondary clarifier overflow rates average 310 gpd/sf during the maximum month and 680 gpd/sf during the peak hour, which are within normal ranges. Considering surface overflow rates only, the maximum firm capacity of the secondary clarifiers (when a clarifier is offline, the remaining clarifier can handle 75% of the design flow) is 2.0 MGD for maximum month flows and 4.1 MGD for peak flows. Current maximum month flows are 1.21 MGD and current peak flows are 2.64 MGD. The maximum month and peak flows are expected to reach 2.0 MGD and 4.1 MGD, respectively, in approximately 5 years. The peak flow is expected to reach the hydraulic capacity of three (3) 50-foot diameter secondary clarifiers in approximately 20 years.

The solids loading capacity of the clarifiers depends on the operation of the aeration basin with regard to mixed liquor suspended solids (MLSS), RAS concentration, flow, and wasting rate. The recommended solids loading capacities of secondary clarifiers are 19.2 - 28.8 lbs/sqft/day for average conditions and 40.0 lbs/sqft/day for peak hour (Metcalf & Eddy, Wastewater Engineering, 4th edition). At current loads, the clarifiers are at these limits for solids capacity. An additional secondary clarifier is needed to provide redundancy for the current loads. A fourth secondary clarifier will be needed within the 20-year planning period.

<u>Deficiencies</u>

- The flow split to current clarifiers is by trial and error and is not equal.
- Wiring to the drive mechanisms has experienced several shorts.
- The existing clarifiers do not have enough solids capacity for the current peak flows and are nearing their hydraulic capacity.

Recommendations

- Construct a new secondary effluent box to provide an equal flow split to the secondary clarifiers.
- Replace the wiring to the clarifiers.
- Add two (2) 50-foot secondary clarifiers to handle the current and future solids loading and hydraulic flows.

3.4.4 Sludge Pumping Building

There are three (3) 7.5 HP vertical centrifugal RAS pumps and two (2) 2 HP vertical centrifugal WAS pumps in the Sludge Pumping Building. The RAS pumps are each designed to pump 700 gpm at 17 ft. TDH, with one of the RAS pumps as a standby. Each of the RAS pumps is operated by a VFD, so the operator can manually adjust the pump flow rate. A 6-inch electromagnetic flow meter (located in a vault south of the Sludge Pumping Building) is used to measure the RAS flow rate. The RAS pumps were last rebuilt in 2010-2011.

The WAS pumps have a combined capacity of approximately 265 gpm at 20 ft. TDH and a firm capacity (with one pump operating) of approximately 135 gpm. Each of the WAS pumps is operated by a VFD, so the operator can manually adjust the pump flow rate. A 4-inch electromagnetic flow meter (located inside the Sludge Pumping Building near the valves on the northwest corner) is used to measure the WAS flow rate. The time that the WAS pumps are operated is manually adjusted to control the wasting. Currently, the WAS pumps are set to operate for approximately 1.7 minutes every hour. The WAS pumps need to be rebuilt.


Sludge Pumping Building

An overhead crane is located above the pumps for maintenance and removal. A duplex 1/3 HP sump pump system, located near the WAS flow meter, is used to remove water that accumulates in the Sludge Pumping Building by pumping it to the Headworks. The number of air exchanges must be at least 6 per hour, or else the area would become a NFPA 820, Class I, Division 2 classified area.

Each additional secondary clarifier would require an additional RAS pump and a WAS pump; there is not space in the Sludge Pumping Building for these additional pumps.

Deficiencies

- Space in the existing Sludge Pumping Building is limited and not sufficient to accommodate additional RAS and WAS pumps.
- The WAS pumps need to be rebuilt.

- The HVAC system for the building may not provide the minimum 6 air exchanges per hour needed for Class 1, Division 1 classification, which would require that the electrical equipment comply with the more stringent Class I, Division 2 requirements.

Recommendations

- Add on to the Sludge Pumping Building to provide space for additional pumps.
- Rebuild the WAS pumps.
- Add an HVAC system that provides at least 6 air exchanges per hour.

3.4.5 Tertiary Filters

Following the secondary clarifiers, the wastewater flows through an 18-inch line to an intermediate pump station. The pump station was constructed by Hydronix in 2010 to transfer the wastewater to the tertiary filters. Each of the two (2) pumps is operated by a VFD and has a design capacity of 2.0 MGD. The current instantaneous peak flow is 2.64 MGD, so the intermediate pump station does not meet the current design requirement with one pump out of service. A Davit crane is located near the transfer pumps for maintenance and removal. Fall safety protection is provided. The human machine interface (HMI) for the pump station is very small, which makes it difficult to modify the settings. It is also difficult to see in the sunlight.


Tertiary Filters

The tertiary filters are two (2) Aqua-Aerobic Systems' AquaDisk package units that provide solids polishing prior to the UV system. Each package filter unit has a design capacity of approximately 1.2 MGD (maximum month flow). Two (2) additional disks can be added to each package filter unit, which would bring the design capacity for each package filter unit up to 1.8 MGD. The current maximum month flow is 1.2 MGD, so the package filter units are near the current design requirement with one unit out of service. The package filter units have a fabric cover, but a permanent roof would be helpful to provide weather protection for the operators.

Deficiencies

- The intermediate pump station HMI is too small.
- The intermediate pump station does not have redundancy for the current peak flow.
- The tertiary filters are at capacity and will not have redundancy for future flows.
- The filter covers do not protect the operators from the weather.

Recommendations

- Increase the size of the HMI on the intermediate pump station.
- Add a third pump in the intermediate pump station to provide redundancy for the current peak flow. Add capacity by changing out the pumps.
- Add disks to the existing filter units to provide redundancy in the near term with one package filter unit out of service. Add a third filter to meet future flows.
- Add a permanent cover over the filters.

3.4.6 UV System


UV System

The outlet piping from the filters discharges into a UV disinfection channel. The UV system disinfects the wastewater through UV radiation, which initiates a photochemical reaction that destroys the genetic information contained in the DNA of bacteria. The bacteria lose their reproductive capability and thus are inactivated.

The existing UV system, constructed in 1994, consists of two (2) banks each with 11 modules at 8 bulbs per module, for a total of 88 bulbs per bank. The banks were replaced in 2010; however, the seals have been leaking and need to be replaced. According to the Plant O&M Manual, the firm capacity (with one module out of service) is 3.1 MGD. Although the current peak hour flow is below 3.1 MGD (currently 2.64 MGD), the projected peak flows will exceed this in less than 5 years. The operators have also noticed that the UV system efficiency has been diminishing, likely due to its age.

A 90° V-notch weir and Ametek® Drexelbrook USonic-R™ ultrasonic level sensor upstream of the UV system is used to measure the effluent flow. The V-notch is approximately 1.33 ft. deep, with a capacity of approximately 3.30 MGD. The projected peak flows will exceed this value in less than 5 years. A 4-20 mA signal is sent from the ultrasonic level sensor to the WWTP Office to control a chart reader for monitoring purposes. UV intensity readings are sent to the WWTP Office via Bluetooth® technology. As effluent leaves the UV system, an ISCO Model 6712FR refrigerated composite sampler is used to capture effluent samples. The composite sampler can also be paced to collect samples based on the effluent flow measurements.

A permanent 150 kW diesel generator with automatic transfer switch is located in the Effluent Pump Station. This generator powers the effluent pumps, UV system, and selected emergency lighting if there is a loss of power.

Deficiencies

- The UV seals have been leaking.
- Projected future peak flows will exceed the capacity of the UV system and the V-notch weir within the next 5 years.

Recommendations

- Replace the seals and expand the UV system for future peak flows.
- Replace the V-notch weir to measure future peak flows.

3.4.7 Effluent Pump Station

Four (4) 40 HP vertical turbine effluent pumps are located in dry well in the Effluent Pump Station. Each effluent pump is rated for a capacity of 760 gpm at 120 ft. TDH. The firm capacity of the effluent pumps (with one pump out of service) is approximately 2,280 gpm (3.3 MGD), which is less than the 2020 future peak instantaneous flow. The effluent pumps pull from an adjacent wet well based on the water level in the wet well as measured using a bubbler. A high level float is used to trigger an alarm. Effluent Pump #2 was rebuilt recently, but the remaining pumps are in need of being rebuilt and/or replaced. An overhead crane is located above the pumps for maintenance and removal. Although there are access ports on the roof, the roof pitch makes it hazardous to unfasten the ports when the weather is poor.


Effluent Pump Station

Two surge tanks are used to maintain consistent pressure and minimize pump cycling at low flows. A groundwater well adjacent to the Effluent Pump Station is used to provide water for these surge tanks. A 5 HP groundwater pump is located in the well.

The wastewater is pumped through a 12-inch diameter pipe approximately one mile to the Multnomah Channel. A single-port diffuser under the channel is used to mix the discharged effluent with the channel flow. A permanent 150 kW diesel generator with automatic transfer switch is located in the Effluent Pump Station. The generator powers the effluent pumps, UV system, and selected emergency lighting if there is a loss of power.

Deficiencies

- It is difficult to remove the pumps when the weather is poor.
- Some of the pumps are old and nearing the end of their expected lifespan.
- The effluent pumps are expected to exceed their firm capacity in the next 5 years.

Recommendations

- Upgrade the roof access ports to make them easier to open.
- Add capacity by changing out the effluent pumps.

3.4.8 Aerobic Digester

Aerobic Digester

A 192,000-gallon, six-cell aerobic digester is used to treat the solids generated in the activated sludge process. Solids being sent to the Aerobic Digester are either from the WAS pumps in the Sludge Pumping Building or from the Scum Pump Station near the secondary clarifiers. The solids flow by gravity through the six cells. The goal is to achieve Class B biosolids as set forth in EPA Part 503-Standards for the Use or Disposal of Sewage Sludge; however, the digester does not have sufficient volume.

Two (2) 25 HP blowers, located on the main floor in the Headworks, are used to provide air to the digester. The combined air flow capacity is 1,200 SCFM, but the firm capacity is 700 SCFM. The blowers draw air in from the building through their inlet filters. A review of the electrical system should be a part of any upgrade to ensure compliance with the Standard for Fire Protection in Wastewater Treatment and Collection Facilities (NFPA 820). The area is likely a NFPA 820, Class I, Division 2 classified area since it is connected to the Headworks dry well, but the equipment does not appear to be rated for this area. Although there are access ports on the roof, the roof pitch makes it hazardous to unfasten the ports when the weather is poor.

Coarse bubble diffusers in the bottom of the digester cells distribute the air from the blowers. However, some of the diffusers are plugged and it is difficult to take down a cell to clean and still meet Class B requirements. Additionally one of the “swingfusers” is broken, which doesn’t allow air to reach the cell. The basin is not being adequately mixed, which is reducing the volatile solids destruction. Also, some of the safety railing near the basin is corroded or missing.

A suction line in the bottom of the last cell is connected to a 5 HP Wemco Model EVM sludge transfer pump located in the Headworks. The sludge transfer pump is operated manually and pumps the digested sludge to the Biosolids Storage Lagoons. A 6-inch electromagnetic flow meter is normally used to measure the sludge flow to the lagoons, but it is currently malfunctioning. Supernatant from the digester flows by gravity through an 8-inch drain line to the Headworks wet well.

Deficiencies

- The digester does not have sufficient volume to achieve Class B biosolids through maintaining a minimum solids retention time.
- Some of the coarse bubble diffusers are plugged and the cells cannot easily be taken down to perform maintenance. At least one of the “swingfusers” is broken, which means that the cell is not aerated.
- The blowers are likely located in a classified area.

- There is no backup sludge pump.
- The sludge flow meter is currently not working.

Recommendations

- Due to the digester's size and the issues with the equipment, it is recommended that the digester be replaced and either abandoned or repurposed.

3.4.9 Biosolids Storage Lagoons

Aerobically digested biosolids are stored on the north side of the WWTP in two clay-lined storage lagoons, (originally built in 1972 and modified in 1994), with a combined storage volume of approximately 1.15 million gallons. The sludge is pumped from the Aerobic Digester to either of the lagoons. From 1994 to 1997, the western lagoon was aerated with five (5) 7.5 HP surface aerators. The aerators were used to keep the lagoon mixed and to provide oxygen for further treatment. The aerators have since been removed. The eastern lagoon was used only for storage.

Biosolids Storage Lagoons

The biosolids storage lagoons are currently emptied during alternate years by a contract hauler. The biosolids are typically 11% solids, so rather than pumping the solids to the hauler, the solids are removed by a backhoe and loaded onto a manure spreader. A 20 HP sludge spray irrigation pump has also been used in the past to drain the lagoons, but it is no longer functioning.

The City recently increased the approved biosolids application area to approximately 200 acres. The land is used primarily for cattle grazing.

Deficiencies

- The lagoons are no longer mixed and aerated by aerators, so the volatile solids destruction is less consistent.

Recommendations

- Add back aerators into the western lagoon to increase the volatile solids destruction.
- The aerators will keep the sludge in suspension and may decrease the solids concentrations; therefore, it is recommended to add dewatering equipment. Dewatering equipment would increase the solids concentration (15-20% solids) and decrease the hauling costs.

3.4.10 SCADA

There is currently no SCADA system, which makes trending and process monitoring and control difficult. Each of the buildings has its own control panel and motor control centers. The control panel in the WWTP Office controls the influent screen, influent pumps, digester blowers, and sludge transfer pump. It also provides status for all of the pump and motors throughout the plant and displays flow measurements. The autodialer (Mission Control) is also located in this panel. The control panel in the Sludge Pumping Building controls the lagoon aerators, RAS and WAS pumps, scum pump, and clarifiers. The control panel in the Effluent Pump Station controls the effluent pumps and sludge spray irrigation pump. The UV System, Intermediate Pump Station, and the Tertiary Filters each contain their own control panel and motor starters.

The autodialer (Mission Control) provides information on the influent and effluent pump runtimes, influent and effluent flow rates, and alarms for high level in the influent or effluent pump wet well or if the influent pump, effluent pump, UV, clarifier, influent screen, or control power fails. Since the control power is normally backed up with a battery, the operators do not get an alarm when there is a brief power outage.

3.4.11 Electricity

All of the electricity at the WWTP is provided by Portland General Electric. 277/480-volt power is supplied to the motor control centers located throughout the plant, and 120/208-volt loads are served through step-down transformers or 120/208 volt panel boards. Permanent generators located near the Sludge Pumping Building and in the Effluent Pump Station power the WWTP equipment whenever the power goes out. The generators are exercised periodically. The 500 kW diesel generator near the Sludge Pumping Building was purchased in 2006. The 150 kW diesel generator in the Effluent Pump Station is older than the other generator. It has an automatic transfer switch and powers the effluent pumps, UV system, and selected emergency lighting.

3.4.12 Plant Water

The groundwater well near the Effluent Pump Station is used to provide plant seal and washdown water. Potable water from the City system is used in the laboratory. There is currently no use of WWTP effluent for plant seal and washwater. It is recommended that the City investigate installing a plant water system using treated and disinfected effluent rather than groundwater. Pumps, and an additional water distribution system separate from the potable water, will be necessary.

3.4.13 WWTP Office

The WWTP Office was constructed in 1994. It currently houses both the laboratory equipment and kitchen equipment in the same room. The laboratory equipment should be separate from the food preparation and eating area, as the wastewater samples and chemicals used in the laboratory pose health hazards and should not be ingested. It is recommended that the oven and refrigerator be removed from the WWTP Office and that all eating take place in the Shop Office rather than the WWTP Office.

3.4.14 Site Security and Maintenance

The WWTP is fully fenced. The gate must be left open during business hours. There are maintenance shops on the west side of the WWTP, which provide for essential storage.

3.4.15 Influent Quality

Wastewater flowing into the WWTP varies in concentration due to the I/I diluting the wastewater during wet weather. Wastewater flows can also vary throughout the day. During dry weather when infiltration and inflow are low, the composition of the wastewater is fairly consistent.

The WWTP does not accept septage; however, there is one RV dump at a gas station in town. The flow from this RV dump is currently not measured or recorded. The City's building code requires grease traps for most establishments, but this has not been enforced, which has resulted in grease problems at the WWTP. The establishment owner is responsible for efficient operation and maintenance of grease traps, but there is no inspection program to check or enforce O&M of the traps. The City would like improve the pretreatment program and operation to prevent as much grease as possible from entering the WWTP. The City is in the process of updating their pretreatment ordinance. Additional discussion of pretreatment and grease prevention is in Section 2.8.

Analysis of Plant Records

The plant data taken from the Discharge Monitoring Reports (DMRs) was analyzed from January 2010 to September 2015. The influent parameters monitored by the City include flow, BOD_5, total suspended solids (TSS), pH, and temperature. The effluent parameters include BOD_5, TSS, pH, *E. coli*, ammonia, DO, UV radiation intensity, and temperature (including temperature in the Multnomah Channel). The biosolids parameters include total solids, volatile solids, ammonia, nitrate, total Kjeldahl nitrogen, phosphorus, potassium, pH, metals (As, Cd, Cu, Hg, Mo, Ni, Pb, Se, and Zn), and fecal coliform or salmonella.

The City collects composite samples of both the influent and effluent at least twice a week and tests for BOD_5 and TSS. The City collects grab samples of the influent and effluent for pH three times per week. During the summer months (May 1 through October 31) the City monitors the temperature in the influent, effluent, and Multnomah Channel three times per week. The City collects grab samples of the effluent for ammonia and DO once a week, UV radiation intensity daily, and *E. coli* twice per week. The City also records influent flow daily and calibrates the influent flow measurement semi-annually. When land applying, the City collects at least seven individual samples for fecal coliform or salmonella every month, plus a composite sample once a year for total solids, volatile solids, ammonia, nitrate, total Kjeldahl nitrogen, phosphorus, potassium, pH, and metals.

BOD_5 Loading

The average monthly influent BOD_5 concentrations and loads into the WWTP from January 2010 through September 2015 are provided in Charts 3-1 and 3-2. The influent BOD_5 concentrations generally vary from approximately 100 to 350 mg/L, which are within the range of typical wastewater values. For Scappoose, these concentrations equate to BOD_5 loadings of approximately 1,000 to 2,000 lbs./day. The waste strength has been increasing over the reporting period.

Chart 3-1: WWTP Influent BOD_5 Concentrations

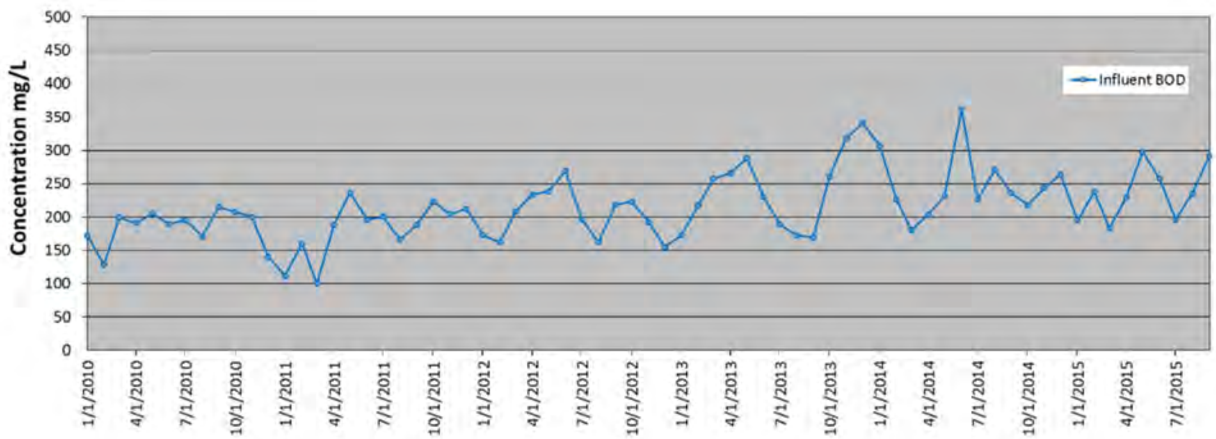
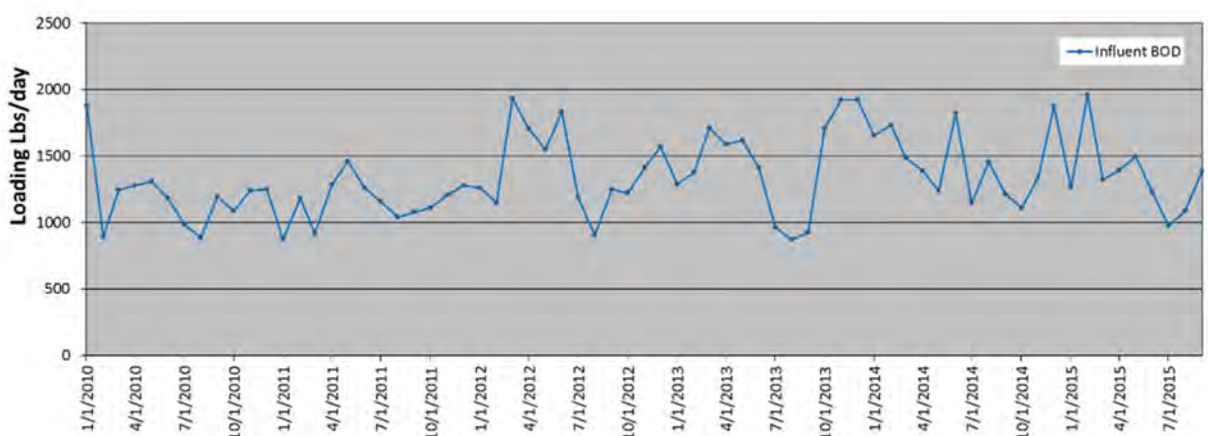


Chart 3-2: WWTP Influent BOD_5 Loading



The BOD_5 loading rates are shown in Table 3-1. The BOD_5 loading rates are normalized for the population to provide units of BOD_5 in pounds per capita per day (ppcd) using the Table 1-1 population estimates. The BOD_5 ppcd loading rates have remained fairly steady over the reporting period. The typical range for BOD_5 is shown in the table footnote. The design values for this study are also shown in Table 3-1. Since the loading rates have remained fairly constant, the maximum values were used as the design values, which is conservative.

Table 3-1: Summary of Influent BOD_5 Data

	2010	2011	2012	2013	2014	Jan. - Sept. 2015	Design
Population	6,592	6,622	6,652	6,683	6,714	6,745	6,745
AADF (PPD)	1,329	1,149	1,394	1,439	1,453	1,344	1,453
ADWF (PPD)	1,114	1,179	1,295	1,249	1,309	1,230	1,309
MMDWF (PPD)	1,309	1,460	1,835	1,708	1,825	1,496	1,835
AWWF (PPD)	1,553	1,119	1,494	1,635	1,596	1,486	1,635
MMWWF (PPD)	1,881	1,285	1,933	1,922	1,875	1,963	1,963
AADF (ppcd)	0.202	0.173	0.209	0.215	0.216	0.199	0.216
ADWF (ppcd)	0.169	0.178	0.195	0.187	0.195	0.182	0.195
MMDWF (ppcd)	0.199	0.221	0.276	0.256	0.272	0.222	0.276
AWWF (ppcd)	0.236	0.169	0.225	0.245	0.238	0.220	0.245
MMWWF (ppcd)	0.285	0.194	0.291	0.288	0.279	0.291	0.291

*Typical values BOD_5 (Metcalf & Eddy): 0.130 - 0.260 ppcd

TSS Loading

Influent TSS concentrations from January 2010 through September 2015 are provided in Charts 3-3 and 3-4. The TSS concentrations generally vary between approximately 100 and 400 mg/L, which are within the range of typical wastewater values. These concentrations equate to TSS loadings between approximately 1,000 and 2,500 lbs./day. The waste strength has been increasing during the reporting period.

Chart 3-3: WWTP Influent TSS Concentrations

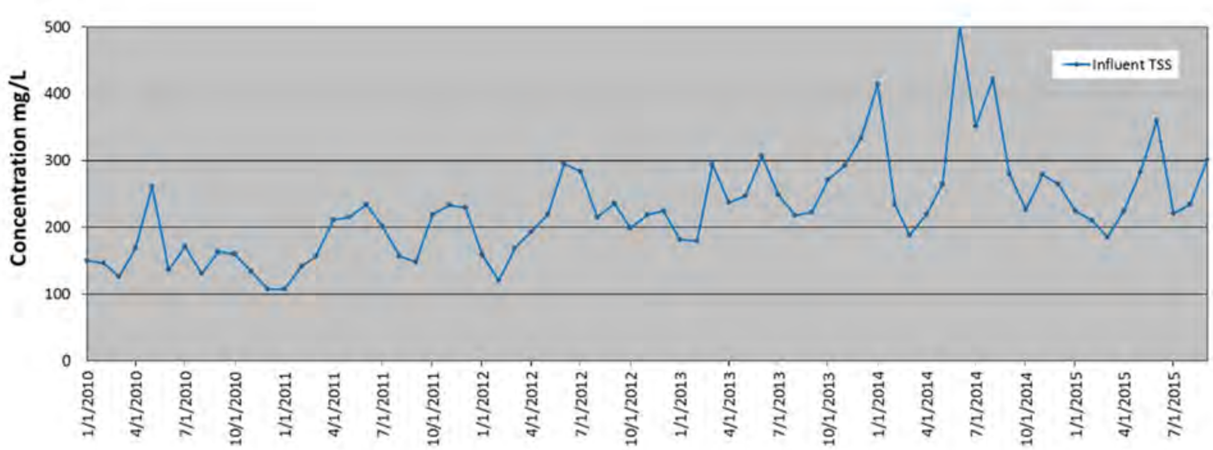


Chart 3-4: WWTP Influent TSS Loading

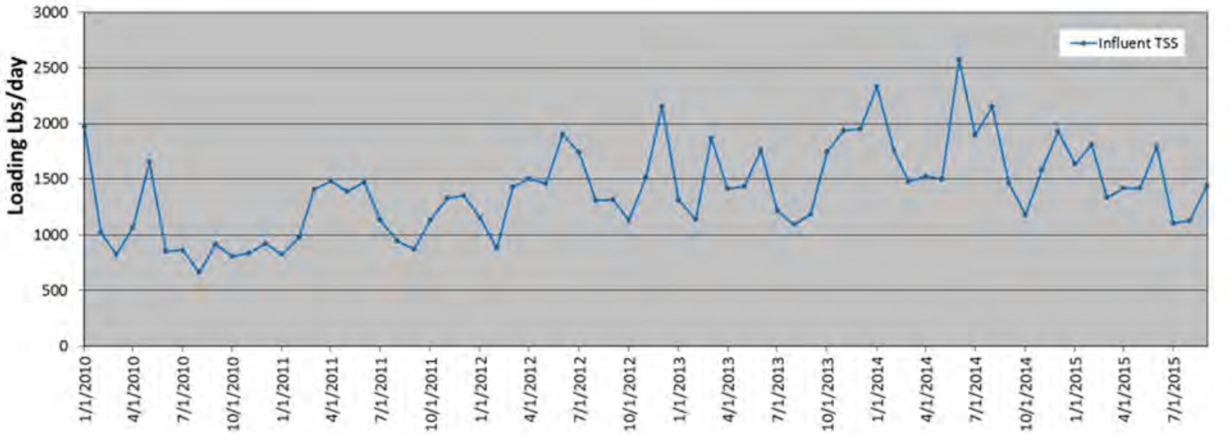


Table 3-2 shows the TSS ppcd summary. The TSS ppcd appears to be fairly constant. The typical range for TSS is shown in the table footnote. The design values for this study are also shown in Table 3-2. Since the loading rates appear to be fairly constant, the maximum year loads from 2010-September 2015 were used as the design values.

Table 3-2: Summary of Influent TSS Data

	2010	2011	2012	2013	2014	Jan. - Sept. 2015	Design
Population	6,592	6,622	6,652	6,683	6,714	6,745	6,745
AADF (PPD)	1,023	1,200	1,426	1,510	1,764	1,446	1,764
ADWF (PPD)	958	1,166	1,443	1,405	1,742	1,370	1,742
MMDWF (PPD)	1,656	1,468	1,902	1,759	2,571	1,791	2,571
AWWF (PPD)	1,090	1,236	1,409	1,612	1,788	1,540	1,788
MMWWF (PPD)	1,972	1,485	2,153	1,951	2,330	1,811	2,330
AADF (ppcd)	0.155	0.181	0.214	0.226	0.263	0.214	0.263
ADWF (ppcd)	0.145	0.176	0.217	0.210	0.259	0.203	0.259
MMDWF (ppcd)	0.251	0.222	0.286	0.263	0.383	0.266	0.383
AWWF (ppcd)	0.165	0.187	0.212	0.241	0.266	0.228	0.266
MMWWF (ppcd)	0.299	0.224	0.324	0.292	0.347	0.269	0.347

* Typical values TSS (Metcalf & Eddy): 0.130 - 0.330 ppcd

The same design ppcd values in Tables 3-1 and 3-2 were also used to estimate the design pounds per day based on the population projections in Table 3-3 (from Table 1-9). Table 3-3 shows the BOD_5 and TSS plant loadings for these design years. For the industrial portion of the future load, concentrations of 300 mg/L BOD and 300 mg/L TSS were used, which are commonly adopted pretreatment limits for these constituents.

Table 3-3: Influent Loading Projections

	Planning Criteria (ppcd*)	Revised Projections (Populations and Industrial Flows From City)								
	Year	2015	2020 Domestic	2020 Domestic and Industrial	2025 Domestic	2025 Domestic and Industrial	2030 Domestic	2030 Domestic and Industrial	2035 Domestic	2035 Domestic and Industrial
	Est. Population	6,745	9,943	9,943	10,924	10,924	12,003	12,003	13,188	13,188
BOD_5										
AADF	0.216	1,459	2,151	2,261	2,363	3,699	2,597	4,598	2,853	5,077
ADWF	0.195	1,315	1,939	2,041	2,130	3,364	2,340	4,194	2,571	4,631
MMDWF	0.276	1,860	2,742	2,860	3,013	4,431	3,310	5,437	3,637	5,999
AWWF	0.245	1,651	2,433	2,553	2,673	4,094	2,937	5,069	3,227	5,597
MMWWF	0.291	1,963	2,894	3,039	3,179	4,931	3,493	6,120	3,838	6,758
TSS										
AADF	0.263	1,773	2,613	2,723	2,871	4,207	3,154	5,156	3,466	5,690
ADWF	0.259	1,750	2,579	2,682	2,834	4,067	3,114	4,968	3,421	5,480
MMDWF	0.383	2,583	3,807	3,925	4,183	5,601	4,596	6,723	5,050	7,412
AWWF	0.266	1,797	2,649	2,769	2,910	4,331	3,197	5,329	3,513	5,882
MMWWF	0.347	2,341	3,451	3,596	3,792	5,543	4,166	6,793	4,578	7,497

3.4.16 WWTP Operations

WWTP Performance

This section evaluates the effluent quality from the existing plant relative to current effluent limits for BOD_5, TSS, pH, and E. coli bacteria.

<u>BOD_5</u>

Monthly and weekly effluent BOD_5 data from January 2010 through September 2015 are shown in Charts 3-5 and 3-6, along with discharge limits per the current permit. No violations were noted during this period. In addition, the plant met the current 85% BOD_5 removal requirement for the entire period, as shown in Chart 3-7. The effluent BOD_5 load was consistently lower than the permitted maximum average monthly, average weekly, and daily loads, as shown in Charts 3-8 through 3-10.

Chart 3-5: WWTP Effluent BOD5 Concentrations (Monthly)

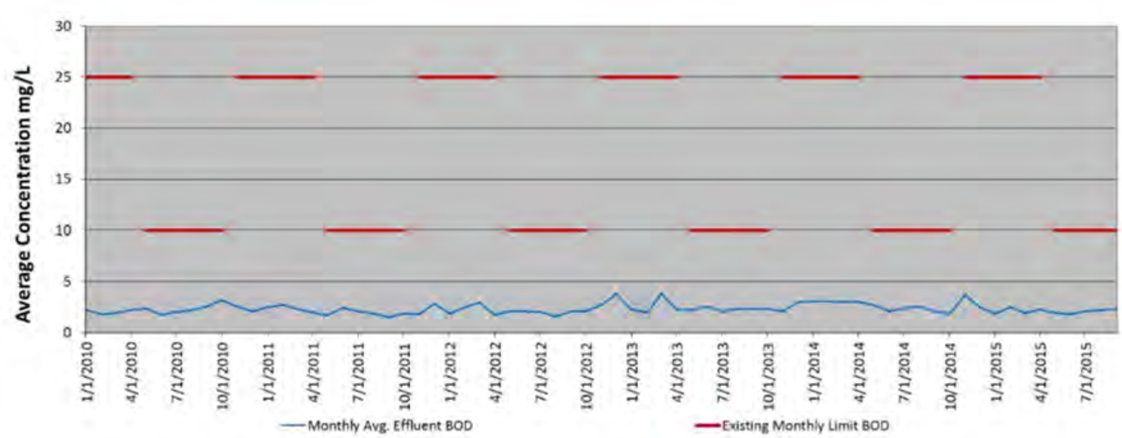


Chart 3-6: WWTP Effluent BOD_5 Concentrations (Weekly)

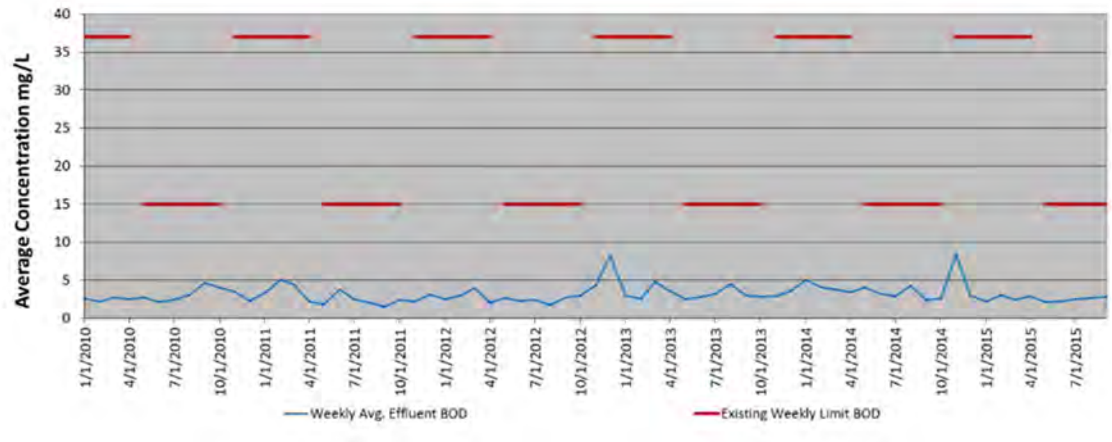


Chart 3-7: WWTP Effluent BOD_5 Percent Removal (Monthly)

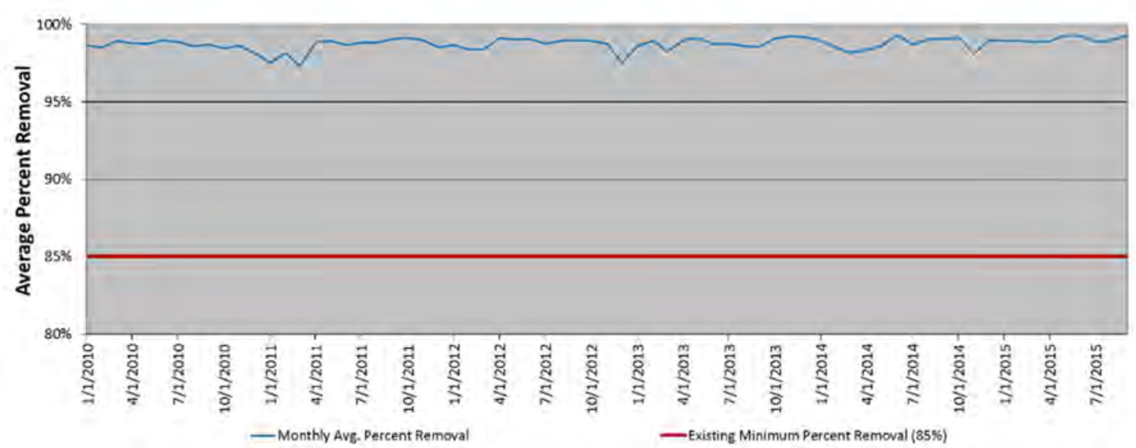


Chart 3-8: WWTP Effluent BOD_5 Loading (Average Monthly)

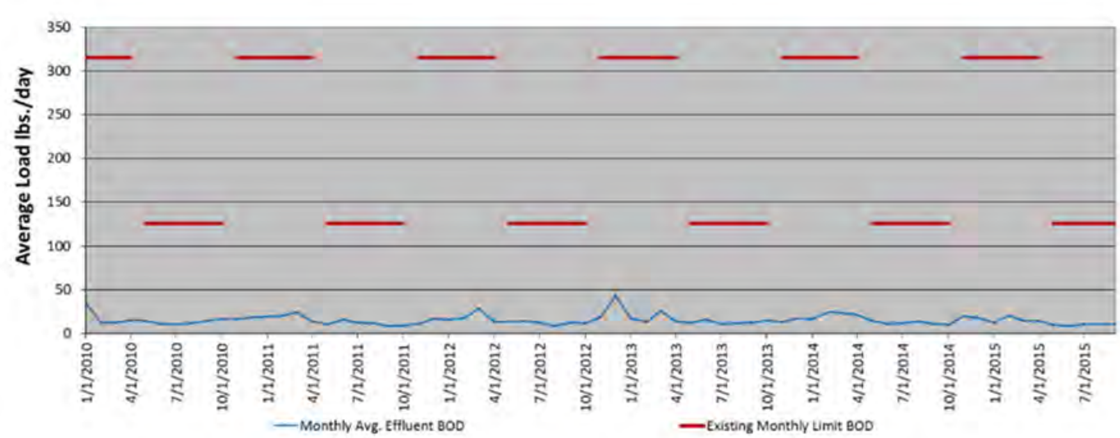


Chart 3-9: WWTP Effluent BOD_5 Loading (Average Weekly)

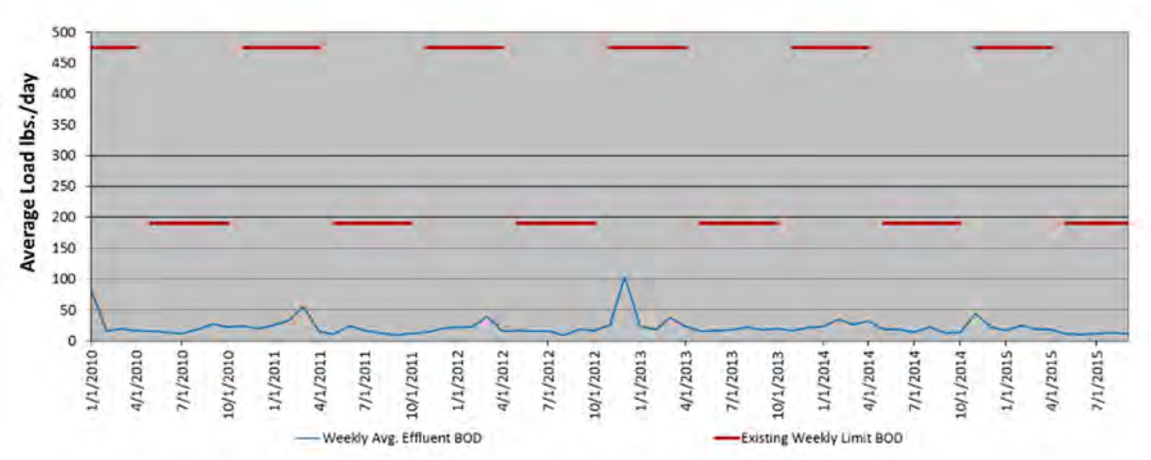
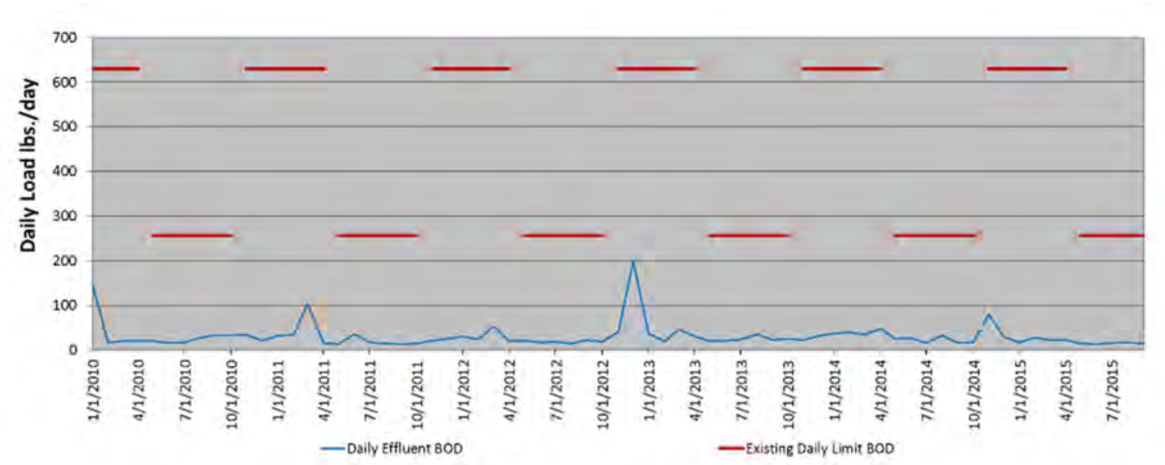


Chart 3-10: WWTP Effluent BOD_5 Loading (Maximum Daily)



TSS

Monthly and weekly effluent TSS data from January 2010 through September 2015 are shown in Charts 3-11 and 3-12 with discharge limits per the current permit. The wastewater treatment plant has not experienced TSS permit violations. Similar to the BOD_5 results, TSS removals have consistently been above the anticipated permit requirement of 85% (Chart 3-13). The effluent TSS load has been consistently lower than the permitted maximum average monthly, average weekly, and daily loads as shown in Charts 3-14 through 3-16.

Chart 3-11: WWTP Effluent TSS Concentrations (Monthly)

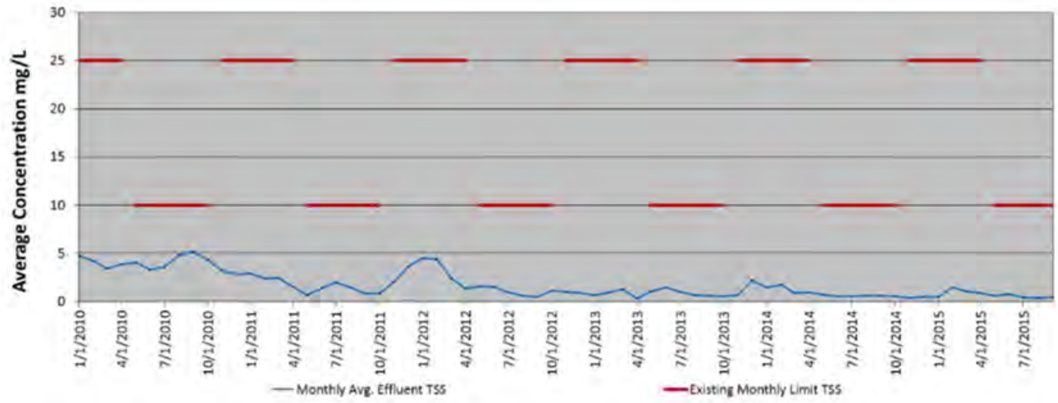


Chart 3-12: WWTP Effluent TSS Concentrations (Weekly)

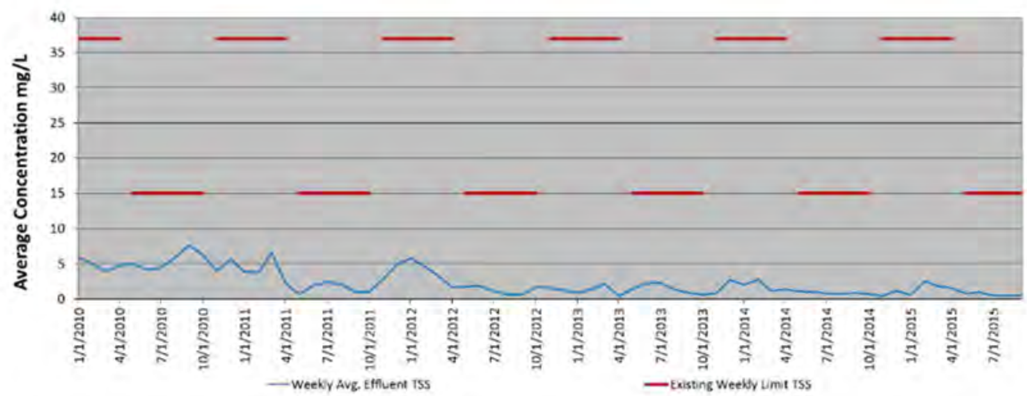


Chart 3-13: WWTP Effluent TSS Percent Removal (Monthly)

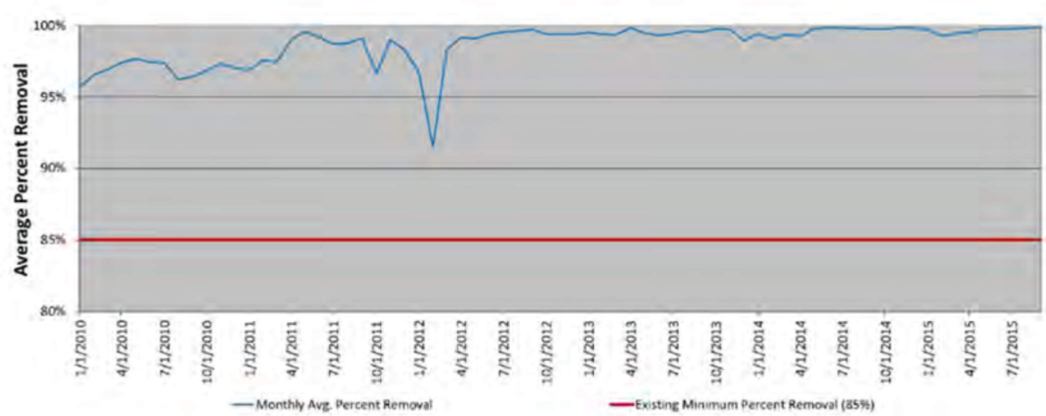


Chart 3-14: WWTP Effluent TSS Loading (Average Monthly)

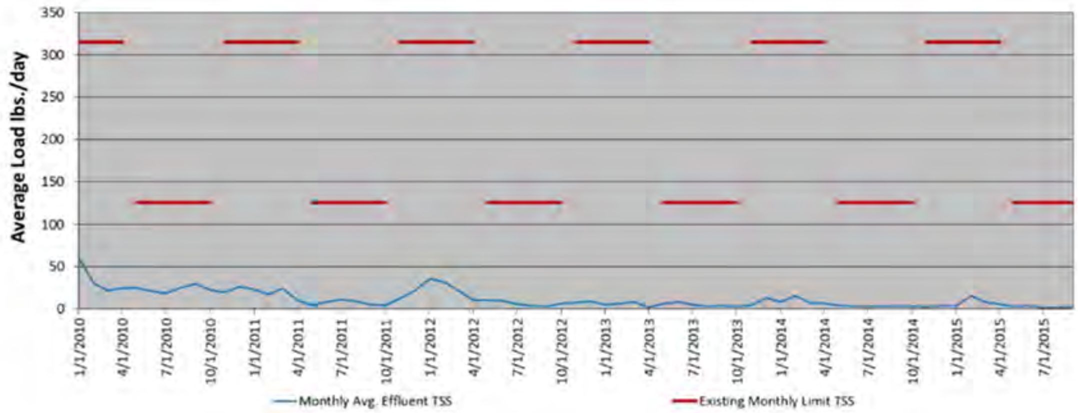


Chart 3-15: WWTP Effluent TSS Loading (Average Weekly)

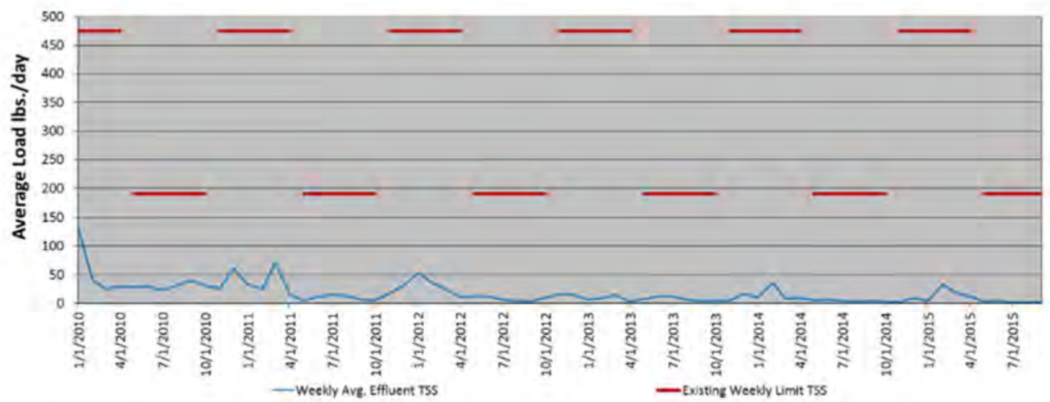
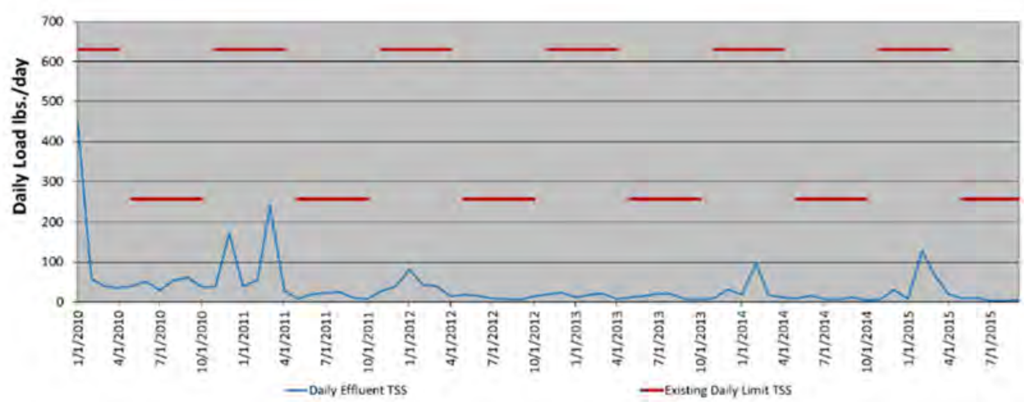


Chart 3-16: WWTP Effluent TSS Loading (Maximum Daily)



E. coli

Bacteria

E. coli bacteria effluent data from January 2010 through September 2015 are shown in Charts 3-17 and 3-18. One daily E. coli violation occurred in September 2015. The City staff found that a duck had died and was decomposing in the Effluent Pump Station. No previous daily or monthly violations were noted during this period.

Chart 3-17: WWTP Effluent E. coli Bacteria (Monthly)

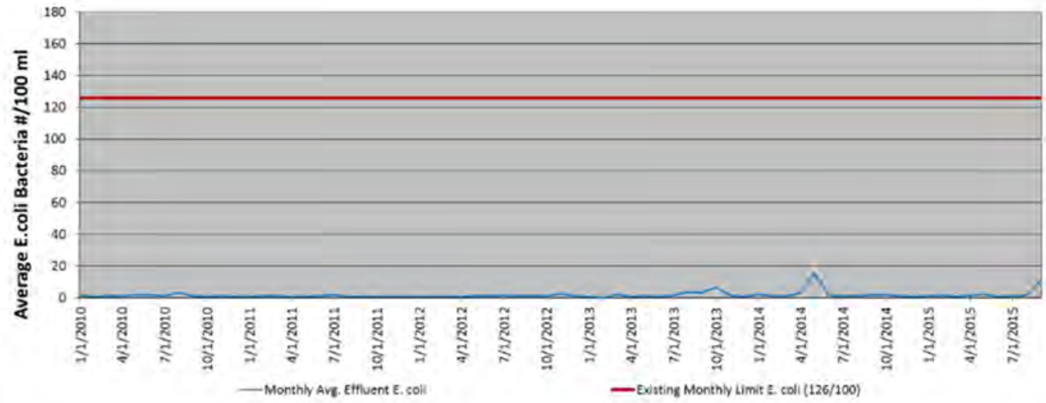
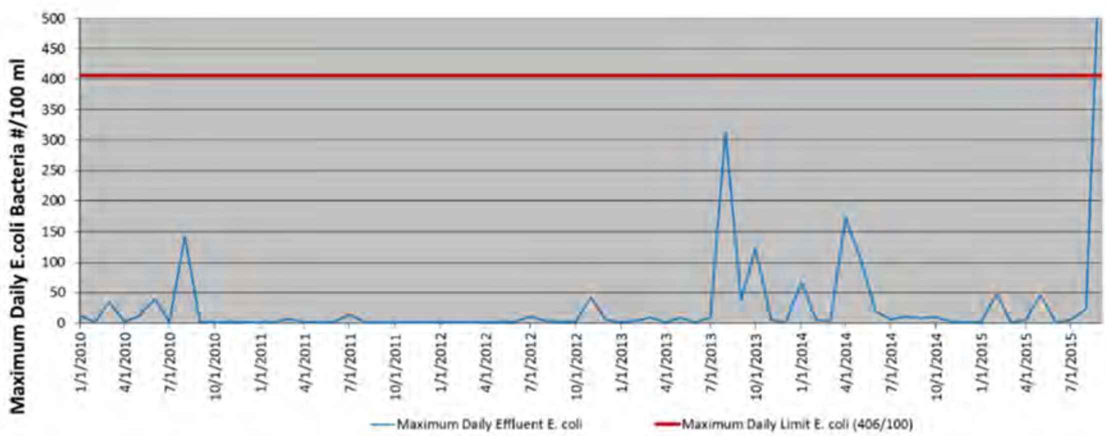


Chart 3-18: WWTP Effluent E. coli Bacteria (Daily)



<u>pH</u>

The daily maximum and minimum pH effluent data from January 2010 through September 2015 are shown in Charts 3-19 and 3-20. One daily pH violation occurred in February 2010. This was due to a probe malfunction. The probe was recalibrated that day. No other daily pH violations were noted during this period.

Chart 3-19: WWTP Effluent pH (Maximum Daily)

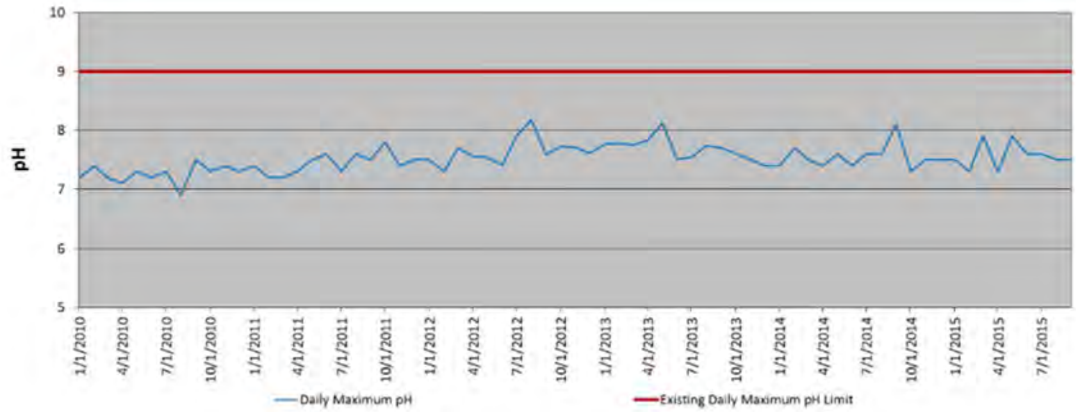
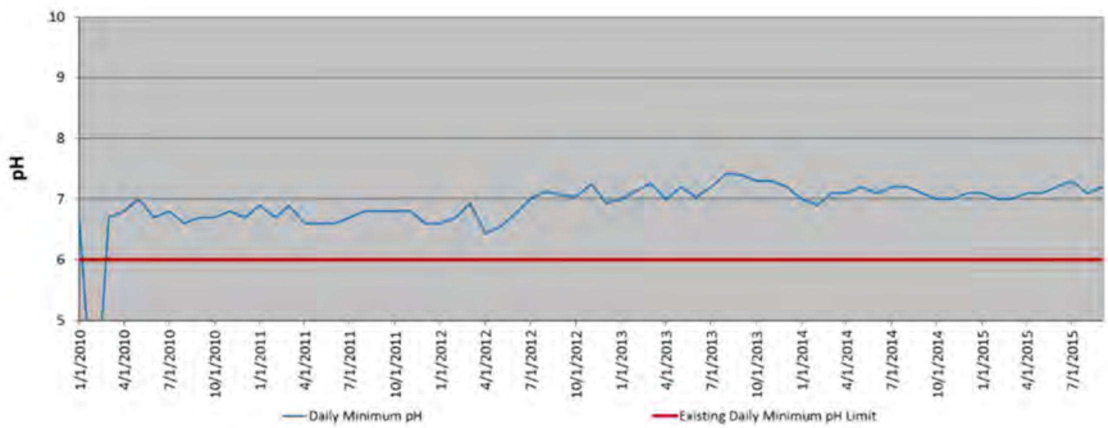


Chart 3-20: WWTP Effluent pH (Minimum Daily)



Reliability Evaluation

A summary of the reliability evaluation based on the current WWTP is provided in Table 3-4. This includes ratings for redundancy, criticality, and equipment condition.

Table 3-4: Unit Process Reliability Evaluation

Equipment	Backup Rating	Criticality Rating	Equipment Condition Rating
Influent Screen	4	S/H, EQ, PF, CC	LN
Influent Pumps	1	S/H, EQ, PF, CC	W/R
Aeration Basin	5	EQ, PF	W/R
Aeration Basin Aerators	1	S/H, EQ, PF, CC	M
Secondary Clarifiers	5	EQ, PF	W
RAS Pumps	1	EQ, PF	M
WAS Pumps	4	PF	M
Intermediate Pump Station	4	EQ, PF	LN
Tertiary Filters	1	EQ, PF	LN
UV Disinfection	1	S/H, EQ, PF	W/R
Effluent Pumps	1	S/H, EQ, PF, CC	W/R
Aerobic Digester	5	EQ, PF	W/R
Biosolids Storage Lagoons	5	PF	W/R

Backup Rating	
1	**One level of "in kind" redundancy** (Identical piece of equipment is available to replace primary unit)
2	**Two or more levels of "in kind" redundancy** (More than one piece of equipment is available for replacement)
3	**Equipment alternative** (An alternative piece of equipment is provided)
4	**Procedural alternative** (An alternative operating procedure is required to provide redundancy)
5	**No Backup** (Failure of equipment will shut entire process down)
Criticality Rating	
S/H	**Safety and Health Risk** (Loss would create risk to safety and health of plant personnel and others)
EQ	**Effluent Quality Risk** (Loss would create risk to WWTP effluent quality and could result in NPDES permit violations)
PF	**Process Functionality Risk** (Loss would affect the function and/or efficiency of the affected processes)
CC	**Cost Critical** (Loss would have a significant cost impact in short term or long term)
Equipment Condition Rating	
N	**New** (Equipment is new, or replaced in last 12 months)
LN	**Like New** (Equipment is operated very little or recently overhauled to a condition like new)
M	**Used But Maintained** (Equipment showing expected wear, but is adequately maintained and functions well)
W	**Heavily Worn** (Equipment close to end of useful life, needs overhaul, difficulty in performing intended functions)
R	**Needs Replacement** (Equipment does not acceptably perform, beyond cost-effective repair)

3.5 CAPACITY LIMITATIONS

Headworks

The capacity of the new influent screen (according to the screen manufacturer) is approximately 4.1 MGD. According to the current growth projections, the peak instantaneous flow rate will reach the screen capacity in approximately 5 years. The required 2035 peak instantaneous flow rate is 7.22 MGD. Also, there is only one automatic mechanical influent screen (no redundancy).

The Oregon DEQ standards require that pump stations have a firm capacity (aggregate pump capacity with the largest pump offline) to pump the peak instantaneous flows. The firm capacity of the influent pumps is 2,250 gpm (3.5 MGD), which is much less than the 2035 future peak instantaneous flow of approximately 5,000 gpm (7.22 MGD). Based on the current growth projections the influent pump capacity will be exceeded within the next 5 years.

The water is pumped from the influent pumps to the Aeration Basin through a 12-inch pipe. *Oregon Standards for Design and Construction of Wastewater Pump Stations* specify a maximum force main velocity of 8 feet per second (fps), which for a clean 12-inch pipeline represents a capacity of approximately 2,800 gpm (4.0 MGD), so the pipeline only has sufficient capacity, based on the projected flows, for the next 5 years. The future 2035 peak instantaneous flow rate is approximately 5,000 gpm.

Aeration Basin

The capacity of the City's 9-inch influent Parshall flume is 5.73 MGD. Based on the current projected flows, it is estimated that the peak flow may approach this value in approximately 10 years. The projected 2035 peak instantaneous flow rate is 7.22 MGD.

The surface aerators and DO2E floating aerators have a combined firm capacity of approximately 4,800 lbs. O2/day (with the largest unit out of service). Assuming typical industry standard oxygen requirements of 1.2 lbs. O2/lb. BOD_5 and 4.6 lbs. O2/lb. TKN, influent concentrations of BOD_5 of 194 mg/L and TKN of 40 mg/L based on the 2010-2015 plant data, and a conservative peaking factor of 1.25, the existing aeration system has capacity to handle a maximum flow of approximately 1.1 MGD, which means that it is currently at capacity. The aeration is also much less than the 3.54 MGD needed for the 20-year planning period.

There is only one aeration basin and since this is where the majority of the plant treatment takes place, it is very difficult to perform any maintenance on the aeration basin structure. A minimum of two aeration basins is recommended to enable a basin to be taken down for maintenance. The approximate maximum capacity of the aeration basin, (assuming adequate aeration), is 1.9 MGD, which means it will reach its capacity in approximately 5 years based on current growth projections.

Secondary Clarifiers

There are two (2) Eimco, 50 ft. diameter secondary clarifiers at the WWTP. The firm capacities of the secondary clarifiers (when a clarifier is offline, the remaining clarifier can handle 75% of the design flow), considering surface overflow rates only, is approximately 2.0 MGD for maximum month flows and 4.1 MGD for peak hour flows. Additionally, the secondary clarifiers are currently at their peak capacity based on the solids loading rates (firm capacity based on solids loading of 1.0 MGD). An additional secondary clarifier is needed to provide redundancy for the current loads. Even if the solids loading is diminished, based on the current growth projections, the two secondary clarifiers will reach their capacity in approximately 5 years.

Sludge Pumping Building

The RAS and WAS pumps currently have a firm capacity of 1,400 gpm (2.0 MGD) and 135 gpm (0.2 MGD), respectively. An additional RAS pump and WAS pump should be added for each additional clarifier added.

The RAS is pumped through a 6-inch pipe. Oregon Standards for Design and Construction of Wastewater Pump Stations specify a maximum force main velocity of 8 fps, which for a clean 6-inch pipeline represents a capacity of approximately 700 gpm (1.0 MGD). The RAS pipe size should be increased to match the RAS pump capacity. The WAS is pumped through a 4-inch pipe. A clean 4-inch pipeline represents a capacity of approximately 310 gpm (0.45 MGD), so the existing pipe is sufficient for the current and 2035 future flows.

Tertiary Filters

The intermediate pump station between the secondary clarifiers and the tertiary filters has a firm capacity (with the largest pump offline) of approximately 1,400 gpm (2.0 MGD), which is less than the current peak instantaneous flow of approximately 1,800 gpm (2.6 MGD). The 2035 peak instantaneous flow is 7.22 MGD.

There is an 18-inch pipe between the intermediate pump station and the tertiary filters. A clean 18-inch pipeline represents a capacity of approximately 6,350 gpm (9.2 MGD), which is sufficient for the 20-year planning period.

The firm capacity of the tertiary filters (according to the filter manufacturer with one filter offline) is approximately 1,400 gpm (2.0 MGD) at peak day flow and 840 gpm (1.2 MGD) at maximum month flow. The current maximum month flow is approximately 840 gpm (1.2 MGD), so although the filters currently provide redundancy, they will not provide redundancy in the near future. The 2035 peak day flow is approximately 3,600 gpm (5.18 MGD) and the 2035 maximum month flow is approximately 2,450 gpm (3.54 MGD).

UV Disinfection

According to the Plant O&M Manual, the firm peak flow capacity of the UV system (with one module out of service) is 3.1 MGD. Although the current peak flow is 2.64 MGD, the peak flow is expected to exceed 3.1 MGD within the next 5 years. The projected 2035 peak flow is 7.22 MGD.

The effluent 90° V-notch weir is approximately 1.33 ft. deep, and has the capacity of approximately 3.30 MGD. As mentioned above, the current peak flow is 2.64 MGD, it is expected that 3.3 MGD will be exceeded in the next 5 years. The projected 2035 peak flow is 7.22 MGD.

Effluent Pump Station

The firm capacity of the effluent pumps is approximately 2,280 gpm (3.3 MGD), which although adequate for the current peak flow (2.64 MGD), will be surpassed in the next 5 years.

The treated effluent is pumped from the WWTP to the Multnomah Channel. The outfall piping was installed in 1994 and consists of approximately one mile of 12-inch pipeline. The piping extends into the river and the effluent exits through one (1) elastomeric diffuser. A clean 12-inch pipeline represents a capacity of approximately 2,800 gpm (4.0 MGD), so the pipeline only has sufficient capacity, based on the projected flows, for the next 5 years. The future 2035 peak instantaneous flow rate is approximately 5,000 gpm (7.22 MGD).

Summary

A summary of the existing and required treatment capacity at the plant is provided in Table 3-5.

Table 3-5: Plant Capacity Summary

Component	Capacity[1] (MGD)	2015 Cap'y Needed (MGD)	2035 Cap'y Needed (MGD)	Comments
Influent Screen	4.1 (PIF_5)	2.6	7.2	No redundancy
Influent Pumps	3.5 (PIF_5)	2.6	7.2	3 pumps in service (4th is redundant)
Influent Pipe	4.0 (PIF_5)	2.6	7.2	--
Influent Measurement	5.7 (PIF_5)	2.6	7.2	--
Aeration Basin	1.9 ($MMWWF_5$)	1.2	3.5	Basin Integrity (no redundancy)
Aeration Basin Aerators	1.1 ($MMWWF_5$)	1.2	3.5	One aerator is redundant
Secondary Clarifiers	1.0 ($MMWWF_5$)	1.2	3.5	No Redundancy with Solids Loading
RAS Pumps	2.0	1.3	2.6	2 pumps in service (3rd is redundant)
RAS Pipe	1.0	1.3	2.6	--
WAS Pumps	0.2	0.04	0.08	Open/close valves for redundancy
WAS Pipe	0.2	0.04	0.08	--
Tertiary Pump Station	2.0 (PIF_5)	2.6	7.2	Second pump is redundant
Tertiary Pipe	9.2 (PIF_5)	2.6	7.2	--
Tertiary Filters	1.2 ($MMWWF_5$)	1.2	3.5	Second filter is redundant
UV Disinfection	3.1 (PIF_5)	2.6	7.2	One module redundant
Effluent Measurement	3.3 (PIF_5)	2.6	7.2	--
Effluent Pumps	3.3 (PIF_5)	2.6	7.2	3 pumps in service (4th is redundant)
Effluent Pipe	4.0 (PIF_5)	2.6	7.2	--

1- Capacity flow numbers are used only for comparative purposes. MGD – million gallons per day, PIF_5 – Peak Instantaneous Flow, $MMWWF_5$ – Max Month Wet Weather Flow.

3.6 FINANCIAL STATUS OF EXISTING FACILITIES

See Section 7.6 for the financial status of existing facilities.

3.7 WATER/ENERGY/WASTE AUDITS

No water, energy or waste audits have been created at this time.

4. NEED FOR PROJECT

4.1 HEALTH, SANITATION, AND SECURITY

The Clean Water Act of 1972 provides the primary regulations for water quality in the waters of the United States. It requires that point source contributions to surface waters obtain a discharge permit (currently permits are issued from Oregon DEQ as NPDES permits). These permits determine the conditions for discharge into surface waters.

Compliance with the NPDES permit for Scappoose was discussed in Section 1.5 of this report. The City of Scappoose's WWTP has been in compliance with the NPDES effluent limits, with a few exceptions, since at least 2010 according to the records provided. The City reports that there has not been a lasting compliance issue in a long time.

Oregon DEQ provided information about other Clean Water Act items, including the status of receiving streams, beneficial uses, and waste load allocations from the TMDL in the NPDES Fact Sheet for Scappoose. The Fact Sheet can be found in Appendix C. DEQ will update the fact sheet during the process of renewing the discharge permit.

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

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

4.2 AGING INFRASTRUCTURE

Aging infrastructure is an issue for Scappoose. Approximately half of the collection system is concrete pipe, which is often susceptible to wear damage, cracking, root intrusion, and other problems associated with older, brittle materials. Infiltration will increase due to aging pipes, and is already an issue (Section 2.5.1 provides more information on I/I).

Many of the components of the WWTP are nearing the end of their useful life and are beginning to show signs of failure. Details of the system deficiencies are discussed in Section 3 of this report.

4.3 SYSTEM DEFICIENCIES

Collection system deficiencies, including lift stations, are discussed in detail in Section 2. WWTP deficiencies are discussed in detail in Section 3. A summary of system-wide deficiencies is included below.

Smith Lift Station

Smith Lift Station has a small operating volume and backs flow up into the gravity mains. The lift station is currently undersized for the existing system flows coming into the wet well. The access hatch is heavily corroded, there is grease build up in the wet well, and one of the blowers for the fan in the dry well does not work.

Spring Lake Lift Station

Pump 1 pumps poorly and pump 2 does not stay primed. There is a probable air lock in the force main and the station has insufficient power supply. There is heavy corrosion on the wet well piping and grease has been an issue in the past. The station does not have a level readout or human machine interface (HMU) to modify settings at the station.

Keys Landing Lift Station

Only one pump is operating because the second pump does not stay primed. There have been odor problems at the station. The wet well piping has heavy corrosion. It is difficult to get the portable generator on site; particularly in winter. The clam shell door does not stay open on its own.

Highway 30 Lift Station

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

Seven Oaks Lift Station

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

Collection System

The western trunk line is significantly undersized for current and future design flows and causes potential overflows sites. Parts of the trunk line on Columbia Avenue and SE Tyler Street are currently at or above capacity. There are a few segments on the trunk line on SW Old Portland Road and SE High School Way that are undersized for future flows. Much of the east trunk line from High School Way to just south of Columbia Ave is undersized for projected flows.

Headworks

The screen, influent pumps, influent pipe, and influent flume do not have sufficient capacity for future peak instantaneous flows. Excessive grit is also accumulating in the plant, which decreases the plant capacity and increases the wear on the equipment. Some additional deficiencies include no screen on the bypass from the influent screen, no freeze protection on the influent screen, no bypass manhole/wet well inside the plant fence, a lack of accurate measurement of level in the influent wet well, influent pumps that are not controlled as a system and are located in a wet and hazardous environment, an HVAC system that is not sized for the NFPA 820 requirements, and a sump in the pump dry pit that does not have redundancy.

Aeration Basin

There is only one aeration basin, so the basin must remain in service at all times. This does not allow for liner maintenance and makes equipment maintenance more hazardous. Additionally, the basin does not provide capacity for future flows, it is not completely mixed, the aeration system is not adequate, and requires a lot of maintenance.

Secondary Clarifiers

The secondary clarifiers do not have solids loading capacity for current peak conditions. They also do not have hydraulic loading capacity for future peak conditions. The flow split to the clarifiers is not equal resulting in periodic additional overloading and the wiring to the clarifier drives is in need of repair.

Sludge Pumping Building

As mentioned above, the clarifiers are inadequate. When a new clarifier is added, there may not be room for the new RAS and WAS pumps in the Sludge Pumping Building. Additionally, the Sludge Pumping Building likely does not meet the Class I, Division II NFPA 820 requirements for 6 air exchanges per hour.

Tertiary Filters

The existing intermediate pump station does not have redundancy for current flows. The tertiary filters and intermediate pump station do not have capacity for future flows. Additionally, the intermediate pump station HMI is too small and the filter covers do not protect the operators from the weather.

UV Disinfection

The UV seals have been leaking and the system efficiency has been diminishing. Also, future peak flows will exceed the capacity of the UV system and V-notch effluent weir.

Effluent Pump Station

The effluent pumps and effluent pipe will not have sufficient capacity for future flows. It is difficult to remove the pumps from the Effluent Pump Station. Additionally, some of the effluent pumps are nearing the end of their expected lifespan.

Aerobic Digester

The Aerobic Digester is currently not adequately sized to achieve Class B biosolids. Additionally, some of the coarse bubble diffusers are not working, which means that some cells are not receiving adequate oxygen. The digester blowers are not rated per NFPA 820 requirements. There is no redundancy for the backup sludge pump and the sludge flow meter is currently not working.

Biosolids Storage Lagoons

The biosolids are not mixed or aerated in the Biosolids Storage Lagoons, so the solids deposit in the lagoons. Grass begins to grow on the biosolids, which makes it more difficult to remove the solids without damaging the lagoons.

Other

There is no formal SCADA system to provide trending and process information. Some of the alarms are not being sent by the existing autodialer that could be sent through a SCADA system. Currently the WWTP uses a groundwater well for washdown activities and plant seal water, as there is no provision for reusing plant effluent. The WWTP Office does not have a separate lab area.

4.4 REASONABLE GROWTH

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

The City expects that at the end of the planning period there will be limited land within the UGB for development. At this time there are several property owners that are pursuing annexation. A buildable lands inventory is underway and is anticipated to be completed in 2017. This inventory is anticipated to support a UGB expansion effort.

5. ALTERNATIVES CONSIDERED

This section describes the alternatives considered to meet the wastewater facilities deficiencies. It also includes design criteria and environmental and constructability considerations.

5.1 PLANNING CRITERIA

The planning criteria used for this facilities planning effort are summarized below.

5.1.1 Collection System

The City's conveyance system will be sized for the projected 2035 peak instantaneous flow rates associated with the 5-year, 24-hour storm event, which is 7.2 MGD (2035 PIF_5 in Table 5-1). Keller Associates sized all new lines one nominal pipe size larger than what is needed for the 20-year planning period to allow for additional growth since the life of the pipe is anticipated to be greater than 20 years. Additionally, it should be noted that efforts to reduce I/I in the collection system could further extend the service population.

In evaluating the gravity collection system, pipelines should be sized to carry design flows without surcharging, and sewage lift stations should be designed to handle these flows with the largest pump out of service. While surcharging of pipelines presently occurs, this introduces a risk of overflows, backing up into homes, and exfiltration. For this evaluation, pipes are considered surcharged if they are 80% or more full, which is consistent with industry standards for collection system evaluations.

5.1.2 Wastewater Treatment Plant Facilities

The characteristics of the influent and effluent that form the basis for sizing the treatment plant facilities are summarized in Table 5-1. Flow criteria that will be used for sizing various potential treatment components are summarized in Table 5-2.

Table 5-1: 20-Year (2035) Design Criteria for Scappoose WWTP

Parameter	Influent	Average Monthly Limit	Average Weekly Limit	Maximum Daily Limit
Average Annual Daily Flow (AADF)	2.390 MGD			
Max Month Wet-Weather Flow ($MMWWF_5$)	3.537 MGD			
Peak Instantaneous Flow (PIF_5)	7.218 MGD			
BOD_5 [1] (May 1 – October 31)	274 mg/L 5,999 lbs./day -	10 mg/L 125 ppd [2] 85% removal	15 mg/L 190 ppd -	255 ppd
TSS [3] (May 1 – October 31)	339 mg/L 7,412 lbs./day -	10 mg/L 125 ppd 85% removal	15 mg/L 190 ppd -	255 ppd
BOD_5 (November 1 – April 30)	229 mg/L 6,758 lbs./day -	25 mg/L 315 ppd 85% removal	37 mg/L 475 ppd -	630 ppd
TSS (November 1 – April 30)	254 mg/L 7,497 lbs./day -	25 mg/L 315 ppd 85% removal	37 mg/L 475 ppd -	630 ppd
pH		Daily minimum and maximum between 6.0 and 9.0		
E. coli Bacteria		126/100 mL	-	406/100 mL
Ammonia	29.7 mg/L	-	-	-

[1] BOD_5 = 5-day biochemical oxygen demand

[2] ppd = pounds per day

[3] TSS = total suspended solids

Table 5-2: Design Criteria for Component Sizing

Treatment Component	Sizing Criteria	Flow (MGD)
Headworks (Influent Screens, Pumps, etc.)	PIF_5	7.2
Secondary Treatment (Basins, Aeration, Clarifiers, etc.)	$MMWWF_5$	3.5
Intermediate Pump Station	PIF_5	7.2
Tertiary Filters	$MMWWF_5$	3.5
Disinfection	PIF_5	7.2
Effluent Pump Station	PIF_5	7.2

5.2 DESCRIPTION

The alternatives considered were based on the following goals:

- Provide facilities capable of reliably meeting current permit limits into the future.
- Maximize use of existing facilities.
- Find solutions that are practical and cost-effective.
- Utilize equipment and materials that are readily available.
- Construct facilities without unacceptably impacting effluent quality.

5.2.1 Regionalization

Due to the political complexity and physical distance between Scappoose and a city with larger wastewater facilities, developing a partnership with another community to share wastewater facilities was not investigated.

5.2.2 Conveyance System Alternatives

This section discusses alternatives that were considered to address the collection system deficiencies mentioned in Sections 2.4 through 2.7. For most of the pipeline deficiencies highlighted in Section 2, the recommended improvement includes pipeline replacement with a larger diameter line. For all pipeline upsize projects, there is the option to construct a second, parallel line, rather than upsize the existing line. There are also options to use trenchless or open cut technologies for pipeline replacement/rehabilitation. These options should be vetted during the pre-design process for each pipeline project. For planning purposes, replacement with larger pipelines has been assumed for the capital improvement plan. Similarly, the majority of the lift station recommendations are relatively minimal and straightforward. Recommended improvements and associated capital costs are summarized in Section 7.

For the collection system, two alternatives were evaluated to correct existing flooding and surcharging on the west trunk line that runs next to Scappoose Creek. The first option includes upsizing the existing trunk line and the capacity of Smith Road Lift Station. This alternative also includes upsizing downstream trunk lines because upsizing Smith Road Lift Station triggers downstream capacity issues. The second option is to install a new relief trunk line from the west line at SW Maple Street that bypasses the west trunk line and Smith Road Lift Station and diverts flow to the east side of town. Both alternatives are discussed in further detail below. The alternatives are presented as full system options because the pipelines are connected, and required capacities throughout the system depend on how flow in the west line is directed. System-wide improvements for each alternative are shown in Figures 16 and 17 (Appendix A). The recommended alternative is discussed in Section 6.

Alternative A – Smith Road Lift Station and Scappoose Creek Line Upsize

The 12-inch pipeline parallel to Scappoose Creek and along SW 4th Street from SW Em Watts Road to the Smith Road Lift Station is undersized for existing flows (Figure 12, Appendix A). The trunk line becomes surcharged at existing peak design flows, causing a number of manholes along the west trunk line to overflow. This is a result of undersized pipelines.

Alternative A (Figure 16, Appendix A) corrects the existing flooding problems on the trunk line along Scappoose Creek and SW 4th Street by upsizing both the existing trunk line and the Smith Road Lift Station. Figure 16 shows the location and required pipe sizes of system-wide improvements associated with this alternative. Projects are summarized briefly below.

Project A.1

The line along Scappoose Creek and SW 4th Street would be upsized to eliminate flooding and relieve surcharging.

Project A.2

The Smith Road Lift Station is undersized for current peak flows. The lift station would be upsized to have a firm capacity of 3,700 gpm. Additionally, the wet well size would be increased to provide more operating volume for the lift station. Miscellaneous lift station improvements recommended in Section 2 should also be completed during this project. In order to prevent the Smith Road Lift Station wet well from backing up the pipelines, it is recommended that the pump on/off set points be adjusted.

Project A.3

The line on NE Laurel Street and NE 3rd Street would be upsized to manage current flows. The rest of this trunk line (along Columbia Avenue) is addressed in Projects A.4 and A.8. All three of these projects should be considered collectively when going through the pre-design process to determine if there would be cost savings in completing the projects simultaneously. Should these projects be completed together, regrading of the line should be discussed in pre-design.

Project A.4

This section of the Columbia Avenue trunk line would require upsizing to handle current flows. Sections down SE Tyler Street also would be upsized for current flows. This project should be considered during the pre-design phase for Projects A.3 and A.8 to determine any cost savings by combining projects.

Project A.5

Sections on SW Em Watts Road and SW Maple Street would be upsized to handle current flows.

Project A.6

The existing line along Smith Road, extending northwest from the Smith Road Lift Station, is undersized for current design flows. This study does not include modeling lines smaller than 10-inches in diameter; however, placement of the flow meter required part of the existing 8-inch line along Smith Road to be modeled. Before this project is designed, it is recommended that additional flow monitoring and modeling of this area be performed to determine if other lines in the area are undersized for current and future flows.

Project A.7

The trunk line on SE 6th Street, between SE 8th and 9th Streets, is adequately sized for existing flows, but will would be upsized to handle future flows.

Project A.8

This section along Columbia Avenue handles existing flows, but would be upsized to handle future flow demands. This project should be considered during pre-design phases of Projects A.3 and A.4 to determine if combining projects would provide cost savings.

Project A.9

There are three sections along SW Old Portland Road from Meadowbrook Road to SW Sycamore Street that will reach capacity at future flows. Modeling shows this line causes slight surcharging in two manholes during the 5-year, 24-hour event in 2035, but not near overflow levels. These could be upsized or regraded to meet future capacity demands. Because future flows indicate marginal surcharging, Keller Associates recommends that the City monitor flows in this area before making any capital investment.

Alternative B – New Relief Trunk Line Across Town

Alternative B (Figure 17, Appendix A) addresses existing flooding problems on the trunk line along Scappoose Creek and SW 4th Street by constructing a new, 18-inch relief pipeline from the intersection of SW 4th Street and SW Maple Street, following SW Maple Street to the Columbia River Highway, turning south, eventually turning east to cross Columbia River Highway, and connecting to the existing line on SE Elm Street. This would divert flow from the Scappoose Creek/SW 4th Street trunk line and Smith Road Lift Station. The existing line from SE Elm Street and SE Tussing Way to the WWTP could be upsized, or a parallel line constructed to accommodate the additional flow. Figure 17 (Appendix A) shows the location and pipe size details of system-wide improvements for this alternative. The projects are summarized briefly below.

Project B.1

The new, 18-inch relief line would be constructed from SW Maple Street to SE Elm Street to relieve flooding and surcharging on the west side of the City. The line will require being bored under the Columbia River Highway and railroad tracks. The capacity of the existing lines on SW 4th Street and SE Elm Street would be increased. Projects B.2, B.3, and B.4 should be considered during the pre-design phase of this project to determine if combining projects could provide cost savings.

Project B.2

This section of the Columbia Avenue trunk line requires upsizing to manage current flow demands. This project should be considered during the pre-design phase for Project B.1, along with Projects B.3 and B.4, to determine if costs could be saved by combining projects.

Project B.3

The pipeline down SE Tyler Street and SE Tussing Way would be upsized to handle current flows. This project should be considered during the pre-design phase for Project B.1, along with Projects B.2 and B.4, to determine if cost savings could be obtained by combining projects. In addition, Project B.9 should be considered during pre-design of this project for cost savings.

Project B.4

Sections on SW Em Watts Road need to be upsized to handle current flows. This project should be considered during the pre-design phase for Projects B.1, B.2, and B.3 to determine if combining projects would produce cost savings.

Project B.5

The line along Smith Road to the east of the wet well is undersized for current flows. The existing line along Smith Road, extending northeast from the Smith Road Lift Station, is undersized for current design flows. This study does not include modeling lines smaller than 10-inches; however, placement of the flow meter required part of the existing 8-inch line along Smith Road to be modeled. Before this project is designed, it is recommended that additional flow monitoring and modeling of this area be performed to determine if other lines in the area are undersized for current and future flows as well.

Project B.6

This is the same as Project A.7 described previously.

Project B.7

The line downstream of the Smith Road force main discharge handles current flows, but would need to be upsized to handle future flows. Regrading of the line should be discussed during pre-design.

Project B.8

This is the same as Project A.9 described previously.

Project B.9

The segment at the north end of SE Tussing Way reaches capacity at future flows. The segment could be upsized or regraded to meet future capacity demands. Because future flows indicate marginal surcharging, Keller Associates recommends that the City monitor flows in this area before making the capital investment recommended.

5.2.3 WWTP Alternatives

WWTP Disposal Alternatives

There are three main alternatives for wastewater disposal:

1. Continue Year-Round Surface Water Discharge (No Action): As discussed in Section 1.5, future discharge permit limits to the Multnomah Channel may be more stringent than current requirements. It is possible that nutrients (phosphorus and nitrogen [including ammonia]), mercury, temperature, and low dissolved oxygen (DO) levels may be reflected in future NPDES permits. Also, ongoing work on toxic substances, including additional heavy metals, polychlorinated biphenyls (PCBs), personal care products (PCPs), pharmaceuticals, and DDT could also result in future limits.

2. Farmland Application: The City could look at agricultural reuse, which could involve working with farmers to use reclaimed water for irrigation. Alternatively, the City could purchase their own land for wastewater land application and growing water-intensive crops. The treatment requirements for reuse may be less stringent than for continued discharge to the Multnomah Channel. Use of treated wastewater outside of the WWTP typically requires an NPDES or water pollution control facility (WPCF) permit and a recycled water use plan (RWUP). It is governed by recycled water regulations, as outlined in Oregon Administrative Rules (OAR) 340-055. The April 2008 revisions to Oregon's Recycled Water Use Rules allow the use of recycled water for beneficial purposes if the use provides a resource value, and protects public health and the environment. Replacing another water source that would be used under the same circumstances, or supplying nutrients to a growing crop, are considered as resource values and beneficial purposes.

   OAR 340-055 defines five categories of effluent; identifies allowable uses for each category; and provides requirements for treatment, monitoring, public access, and setback distances. Irrigation of fodder, fiber, and seed crops not for human consumption is allowed for any class of effluent. Fewer restrictions are imposed for higher-quality effluent, as shown in Table 5-3.

Table 5-3: Requirements for Reuse of Effluent by Category

	Class A	Class B	Class C	Class D	Non-Disinfected
Treatment[1]	O,D,F	O,D	O,D	O,D	O
Total Coliform, 7-day Median #/100 mL	2.2	2.2	23	-[2]	Per Permit
Turbidity, NTU	2	-	-	-	
Public Access[3]		Limited	Limited	Controlled	Prevented
Setback to Property Line[4]		10 feet	70 feet	100 feet	Per RWUP[1]
Setback to Water Supply Source		50 feet	100 feet	100 feet	150 feet

[1] O = oxidized, D = disinfection, F = filtration, RWUP = Recycle Water Use Permit

[2] Rather than total coliform, Class D Recycled Water is required to sample for E. coli. E. coli is a subgroup of the total coliform organisms, so a total coliform analysis includes the E. coli organisms. For Class D Recycled Water, the 100 ml sample must not exceed a 30-day log mean of 126 E. coli organisms per 100 ml; and must not exceed 406 E. coli organisms per 100 ml in a single sample.

[3] Limited public access: no direct contact during irrigation cycle.

[4] Sprinkler irrigation assumed.

Scappoose's current effluent could potentially meet Class A requirements since it is oxidized, disinfected, and filtered (total coliform and turbidity monitoring would be needed).

For recycled water use, groundwater must be protected in accordance with the requirements of OAR 340-040. For agricultural use, this typically translates to irrigating at agronomic rates to match the net irrigation requirements of the crops. Water application can take place during the growing season at a rate of approximately 18.5 inches per acre per year on a grass seed crop (Oregon Crop Water Use and Irrigation Requirements, 1992, OSU ext. Pub. 8530). The theoretical irrigated farmland needed during the growing season (based on the 2035 AADF) and average monthly precipitation data from the Western Regional Climate Center is approximately 2,500 acres. It should be noted that if the farmland used for effluent disposal is privately owned, the City may have limited control over when the effluent is used. Many farmers in the area grow crops without irrigation. In order to have control over the irrigation, the City may need to own the land. This alternative would also require storage during the winter (non-growing season).

3. Summer Farmland Application and Winter Surface Water Discharge: This option would discharge to the Multnomah Channel during the winter (wet weather) and utilize agricultural reuse during the summer. This alternative would not require a large winter storage facility and would decrease the overall discharge loading to the Multnomah Channel. However, there would still be the cost of the land and maintaining the agricultural reuse operation.

WWTP Treatment Alternatives

Options for addressing certain deficiencies of the existing wastewater treatment are shown below. If a WWTP deficiency had only a single solution (such as the effluent pumps, increasing the effluent pipeline, etc.), then the solution is discussed in individual project summary sheets found in Appendix G.

1. *Headworks* – Issues at the headworks include: influent screens, influent pipe, influent flow measurement, and influent pumps that do not have sufficient capacity for the future peak instantaneous flow; lack of freeze protection on the influent screen; no grit removal; inaccurate level measurement in the influent wet well; influent pumps that are not controlled as a system and are located in a wet and hazardous environment; an HVAC system that is not sized to create an unclassified environment according to NFPA 820 requirements for the influent pumps; and a sump pump in the pump dry pit that does not have redundancy.

   There are two main options to address these deficiencies:

   i. Construct a new headworks. The new headworks would be constructed without the issues of the existing headworks. It may be possible for the existing influent screen to be relocated and reused inside the new headworks. The remainder of the headworks equipment would be new. The existing headworks could be abandoned or used for bypass pumping.

   ii. Repair and expand the headworks. The headworks would be expanded with new influent screens and an influent flow meter, and the influent pumps would be replaced to provide capacity for the future peak instantaneous flow. Additional heat tracing would be added to the influent screen to address the freezing concerns. The bubbler would be replaced with an ultrasonic level sensor or pressure transducer. Electrical controls for the pumps would be replaced and the HVAC system would be upgraded. The existing digester blowers and digester pump would be removed from the headworks. An additional sump pump would be installed and piped into the existing sump piping.

2. *Primary Treatment* – Primary clarifiers are sometimes used to decrease the size of the secondary treatment system. The solids removed in primary clarifiers are typically treated using anaerobic digesters, which are discussed later in this section.

3. *Secondary Treatment* – Since there is only one aeration basin, the basin must remain in service at all times, making it difficult to perform maintenance. Additionally, the aeration basin is not completely mixed and is not large enough for future loads. The floating aerators do not provide adequate aeration and require expensive maintenance. Due to these deficiencies and limited space for expansion, it is recommended that the aeration basin either be demolished or modified and repurposed.

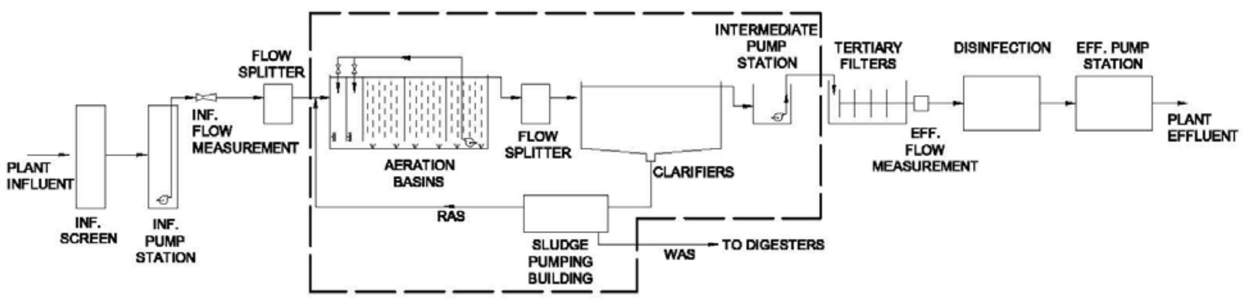
   The secondary clarifiers do not have solids loading capacity for the current peak conditions and are not sized for future flows. Additionally, the flow split to the clarifiers is not equal (resulting in periodic overloading of clarifiers), and the wiring to the clarifier drives is in need of repair. When a new clarifier is added, there is not adequate room for the new RAS and WAS pumps in the sludge pumping building. Additionally, the building likely does not meet NFPA 820 requirements.

There are three main options to address these deficiencies.

i. New aeration basins. This option involves constructing three new aeration basins. These would include diffused aeration for increased aeration efficiency (energy savings compared to surface aerators), divider walls (to avoid backmixing for process control), and submersible mixers in anaerobic/anoxic cells (to create selectors for filamentous control and potentially biological phosphorus release). A mixed liquor recycle pump for internal recycle would also be included, with valves to provide the operators flexibility to use the basins in either an A^2O (anaerobic, anoxic [denitrification], and aerobic) or Modified Ludzack-Ettinger (MLE) process.

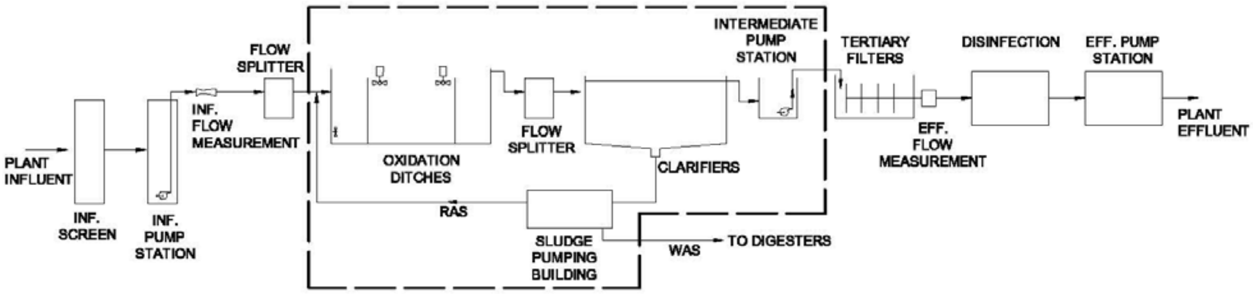
This alternative would also include a blower building, a secondary flow splitter box going to the clarifiers, two (2) additional secondary clarifiers, fixing the clarifier wiring on the existing clarifiers, upgrading the intermediate pump station to handle future peak flows, and expanding and upgrading the existing sludge pumping building. A process flow diagram of the option is shown below, and a site layout of the option is shown on Figure 18 in Appendix A.

Chart 5-1: New Aeration Basins – Process Flow Diagram



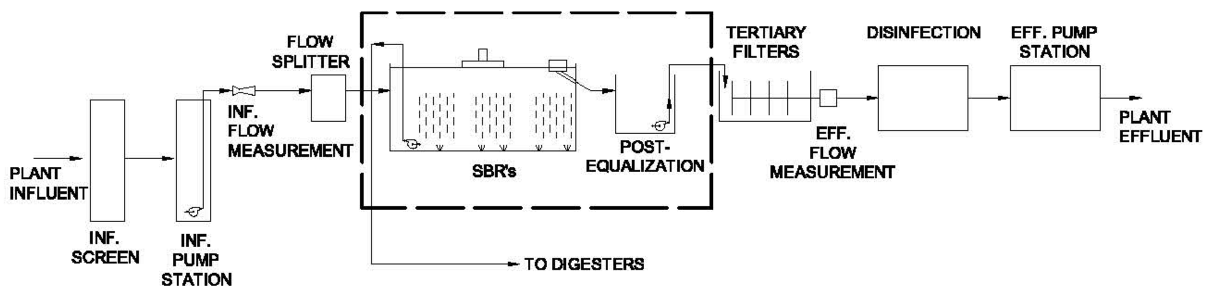
ii. Oxidation ditches. This option would include construction of two new oxidation ditches with aerators and submersible mixers. The oxidation ditches would include selector basins (anaerobic and anoxic) for filamentous control, denitrification, and potentially biological phosphorus release. This alternative would also include adding a secondary flow splitter box, two (2) additional secondary clarifiers, fixing the clarifier wiring, upgrading the intermediate pump station to handle future peak flows, and expanding and upgrading the existing sludge pumping building. A process flow diagram of the option is shown below, and a site layout of the option is shown on Figure 19 in Appendix A.

Chart 5-2: Oxidation Ditches – Process Flow Diagram



iii. Sequencing batch reactor (SBR). This option would include construction of three new SBR basins with diffused aeration, mixers, decanters, and sludge pumps. The SBR basins would be sized to allow selection in the basins (filamentous control and potentially biological phosphorus release), as well as denitrification (longer SRT). This alternative would include an equalization basin downstream of the SBR to dampen decant flows. The equalized flows would be pumped directly to the tertiary filters. This alternative would also include a blower building. An SBR system does not require secondary clarifiers, the intermediate pump station, or the sludge pumping building. These structures would likely be abandoned. A process flow diagram of the option is shown below, and a site layout of the option is shown on Figure 20 in Appendix A.

Chart 5-3: SBR – Process Flow Diagram



4. *Filtration* – Although the existing filters do not have capacity for future flows, it is fairly straightforward to add disks to the existing units and an additional filter unit to meet future flows; therefore, expanding the existing filters was the only option considered.

5. *Disinfection* – The UV system has been having problems with seals, and the system efficiency has been diminishing (likely due to its age). Additionally, the system is not sized for future capacity.

   There are three main options to address these deficiencies:

   i. Upgrade and expand the existing UV system. The new UV system would be sized to handle future flows.

ii. Rehabilitate the old chlorine contact basin for chlorination and dechlorination. This alternative would include increasing the basin size for current and future flows. A well-functioning chemical dosing system (both for chlorination and dechlorination) is necessary for this system to be acceptable, as chlorine discharge is highly regulated.

iii. Rehabilitate the old chlorine contact basin to be used for peracetic acid (PAA) disinfection. Although PAA has been approved for use by the Environmental Protection Agency (EPA), it is still a fairly new technology and would require pilot testing.

6. *Biosolids Treatment (Aerobic Digester / Biosolids Storage Lagoons)* – The existing aerobic digester is not adequately sized to achieve Class B biosolids for current flows. Additionally, some of the coarse bubble diffusers are not working, the digester blowers are not rated for the NFPA 820 classified environment where they are currently, there is no redundancy for the backup sludge pump, and the sludge flow meter is currently not working.

   The existing biosolids storage lagoons are not mixed or aerated. Not only does this mean that additional biosolids treatment does not occur, but grass begins to grow on the biosolids, making it more difficult to remove the solids without damaging the lagoons.

   Due to the size, condition, and age of the existing aerobic digester, upgrading it is not recommended. Thickening the sludge prior to the digester is not a good option, as it would be more difficult to aerate/mix in the digester and for the existing sludge pump to pump the higher solids concentration to the biosolids storage lagoons.

   The alternatives that were considered for biosolids treatment were the following:

   i. New aerobic digesters. The new aerobic digesters would be large enough to achieve Class B biosolids. The digester blowers would be located in a blower building, while the sludge pumps and flow meter would be located in a dewatering building.

   ii. New anaerobic digesters. New anaerobic digesters would be sized to achieve Class B biosolids. To decrease the size of the anaerobic digesters, gravity belt thickeners would be included to increase the sludge concentration from approximately 1% to 4% solids. A control building would be constructed to house the pumps, heat exchangers, and boilers. The biogas produced by the anaerobic digesters would be used to heat the digesters.

7. *Biosolids Disposal Alternatives* – There are two biosolids disposal alternatives that were considered. Each of these options assumes that the biosolids are dewatered first.

   i. Continue Class B land application on farm fields. This alternative continues the current practice of having a farmer apply the solids.

   ii. Add composting to create Class A biosolids. With this alternative, Class A biosolids would be applied to farmers' fields.

8. *Interim Biosolids Handling Plan* – Due to the urgent nature of the biosolids treatment and storage deficiencies, an interim biosolids handling plan was created in September 2016. The interim improvements recommended to meet Class B requirements for current sludge flows, maximize existing resources, and position the City to implement future improvements outlined in this facility planning study. The interim biosolids handling plan included the following:

    i. Install four (4) 7.5-horsepower and two (2) 15-horsepower surface aerators in the western biosolids storage lagoon to provide aeration and mixing to achieve Class B treatment for current sludge flows.

    ii. Modify the existing gates/pipes between the biosolids storage lagoons to cut off the interconnection of flow.

    iii. Construct a new dewatering building to house a new screw press, sludge transfer pump, polymer dosing system, and dewatered biosolids conveyor.

    iv. Install piping from the existing wet well between the biosolids storage lagoons to the new dewatering building.

    v. Convert the eastern biosolids storage lagoon into a dewatered biosolids storage area by installing concrete in the lagoon, along with a drain system to remove water back to the headworks. A frontend loader would be used to move the biosolids around the lagoon, and to load it into trucks for transport. A cover (such as a clear span) would be added over the area to prevent rain and snow from wetting the dewatered sludge. Access to the storage area would be provided from the existing plant road.

    vi. The WWTP is experimenting with a probiotic solution to decrease the sludge volume. However, if the western biosolids storage lagoon reaches capacity, temporary dewatering equipment may be needed. This equipment (e.g. geotextile dewatering containers, rented screw press, etc.) would require a sludge pump, polymer system, and insulated and heat-traced pipe (if the piping is outside and above-grade). It would also need an asphalt or concrete surface that drains to a sump or pipe that returns to the headworks.

    Two dewatering building alternatives were considered:

    i. Repurpose an existing (approximately 40 ft. x 30 ft.) metal shop to house the dewatering equipment (new screw press, sludge transfer pump, polymer dosing system, and dewatered sludge conveyor).

    ii. Construct a new dewatering building (approximately 40 ft. x 50 ft.) to store the dewatering equipment.

5.3 MAP

Maps of the existing sewershed and WWTP are provided in Figures 7 and 8 (Appendix A). A flow schematic of the existing WWTP is in Figure 15 (Appendix A).

5.4 ENVIRONMENTAL IMPACTS

Potential impacts of the alternatives to the environmental resources presented in Section 1 are described below. A comparison of the potential impacts is summarized in Table 5-4.

5.4.1 Land Use / Prime Farmland / Formally Classified Lands

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

5.4.2 Floodplains

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

5.4.3 Wetlands

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

5.4.4 Cultural Resources

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

5.4.5 Biological Resources

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

5.4.6 Water Resources

Modifications to the WWTP to improve treatment reliability should have a beneficial impact on the Multnomah Channel. There are no alternatives that involve stream crossings.

5.4.7 Socio-Economic Conditions

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

TABLE 5-4: Affected Environment / Environmental Consequences Summary for Alternatives

Environmental Criteria	WWTP Alternatives							
	WWTP Disposal			WWTP Headworks		Secondary Treatment		
	No Action	Farmland Application	Farmland and River	New Headworks	Repair Headworks	New Aeration Basins	New Oxidation Ditches	SBRs
Land Use/ Important Farmland/Formally Classified Lands	No Impact	City purchase / irrigate prime farmland. Force main crossings.	City purchase / irrigate prime farmland. Force main crossings.	No Impact	No Impact	No Impact	No Impact	No Impact
Floodplains	No Impact	In 100-year floodplain	In 100-year floodplain	In 100-year floodplain	In 100-year floodplain	In 100-year floodplain	In 100-year floodplain	In 100-year floodplain
Wetlands	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact
Cultural Resources	No Impact	None known	None known	No Impact	No Impact	No Impact	No Impact	No Impact
Biological Resources	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact
Water Quality Issues	No Impact	Less volume (loading)	Less volume (loading)	Improved effluent quality	Improved effluent quality	Improved effluent quality	Improved effluent quality	Improved effluent quality
Coastal Resources	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Socio-Economic/ Environmental Justice Issues	No Impact	Less volume (loading)	Less volume (loading)	No Impact	No Impact	More energy used	More energy used	More energy used
Miscellaneous Issues	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact

TABLE 5-4, Cont'd: Affected Environment / Environmental Consequences Summary for Alternatives

Environmental Criteria	WWTP Alternatives Cont'd.							Collection System
	WWTP Disinfection			Biosolids Treatment		Disposal Alternatives		
	UV	Chlorine	PAA	Aerobic Digester	Anaerobic Digesters	No Action	Class A - Composting	All
Land Use/ Important Farmland/Formally Classified Lands	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact
Floodplains	In 100-year floodplain	In 100-year floodplain	In 100-year floodplain	In 100-year floodplain	In 100-year floodplain	In 100-year floodplain	In 100-year floodplain	Varies
Wetlands	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact
Cultural Resources	No Impact	No Impact	No Impact	No Impact	No Impact	None known	None known	No Impact
Biological Resources	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact
Water Quality Issues	Improved effluent quality	Improved effluent quality	Improved effluent quality	None known	None known	None known	None known	Less O&M
Coastal Resources	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Socio-Economic/ Environmental Justice Issues	More energy used	More chemicals used	More energy used	More energy used	More energy used	No Impact	More energy used	No Impact
Miscellaneous Issues	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact

5.5 LAND REQUIREMENTS

The City would need to purchase land if farmland application is chosen as a WWTP disposal alternative.

5.6 POTENTIAL CONSTRUCTION PROBLEMS

The depth of the water table and subsurface rock may affect construction of the alternatives. However, subsurface investigations were not within the scope of this project.

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

5.7 SUSTAINABILITY CONSIDERATIONS

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

5.7.1 Water and Energy Efficiency

The new relief trunk line alternative would require less energy because Smith Road Lift Station would not be increased in size.

The farmland disposal alternatives would decrease the volume of WWTP effluent discharged to the river. The effluent, because of the nutrients, would also be beneficial to the farmland.

Further treatment options such as mixing, filtration/DAF, aeration, or UV disinfection, would require additional energy. Adding a chlorination/dechlorination or PAA disinfection system would increase the use of chemicals.

5.7.2 Green Infrastructure

Using WWTP effluent for farmland irrigation helps protect the river and uses the nutrients for crop growth.

5.7.3 Other

Replacement of mains smaller than 8-inch will facilitate improved maintenance.

5.8 COST ESTIMATES

Cost estimates for this report were prepared using estimated construction costs with 10% mobilization; 15% contractor overhead and profit, plus a contingency of 30%; and engineering services including construction of 20-30% (based on total construction cost). Legal, administrative, and permitting costs of 2% are included for the selected alternatives. Present worth analyses are based on a real discount rate of 1.2% and a 20-year time period. An average rate of $0.085 per kWh was used for estimating power costs. Cost estimates for each alternative are presented in Section 6.

6. SELECTION OF AN ALTERNATIVE

Alternatives were considered to address deficiencies noted in the previous sections. Advantages, disadvantages, and comparative costs (where applicable) are presented for evaluating each process alternative. The comparative cost estimates do not include costs common to all alternatives. For the treatment plant improvement options, annual operation and maintenance (O&M) costs are included in cost estimates to arrive at a present value for comparison of alternatives. The present value analysis was conducted using a real discount rate of 1.2% and a 20-year time period. The equipment (unless a short-lived asset) is assumed to have a 20-year useful life, thus no salvage value is included in comparing the alternatives.

6.1 COMPARATIVE ANALYSIS (COSTS AND NON-MONETARY FACTORS)

6.1.1 Collection System West Side Flow Alternatives

1. Alternative A – Smith Road Lift Station and Scappoose Creek Line Upsize

   Alternative A is described in detail in Section 5.2.2. This alternative consists of upsizing existing trunk lines on the west side of town and Columbia Avenue. Figure 16 (Appendix A) highlights all collection system improvements that would be completed if this alternative was chosen. The Smith Road Lift Station capacity and wet well size would also be increased. A preliminary cost estimate is summarized in Table 6-1, which does not include additional O&M cost increases. There is approximately 450 feet of additional pipe in this alternative compared to Alternative B, and Smith Road Lift Station nearly doubles in capacity. Based on these items, the O&M for Alternative A would be higher than that of Alternative B, making Alternative B more favorable.

Table 6-1: Smith Road Lift Station & Scappoose Creek Line Upsize Cost Estimate

Item	Cost
Pipeline Improvements	$ 2,123,500
Manhole Replacement	$ 565,600
Surface Restoration	$ 497,800
Lift Station Improvements	$ 350,000
Mobilization (5%)	$ 176,900
Contingency (30%)	$ 1,114,200
Construction Subtotal	**$ 4,828,000**
Engineering and CMS (25%)	$ 1,207,000
Legal, Admin, and Permitting (2%)	$ 96,600
Total Project Cost	**$ 6,140,000**

2. Alternative B – New Relief Trunk Line Across Town

   Alternative B is described in detail in Section 5.2.2. This alternative consists of placing a new, 18-inch gravity main across town from SW Maple Street and connecting with existing lines on SE Elm Street. The new line would require being bored under the Columbia River Highway and railroad tracks. Figure 17 (Appendix A) highlights all collection system improvements that would be completed with this alternative. A preliminary cost estimate is summarized in Table 6-2, which does not include O&M costs; however, the O&M for this option would be less than that of Alternative A.

Table 6-2: New Relief Trunk Line Across Town Cost Estimate

Item	Cost
Pipeline Improvements	$ 1,818,900
Manhole Replacement	$ 498,300
Surface Restoration	$ 494,300
Bore and Jacking	$ 117,000
Mobilization (5%)	$ 146,500
Contingency (30%)	$ 922,500
Construction Subtotal	**$ 3,998,000**
Engineering and CMS (25%)	$ 999,500
Legal, Admin, and Permitting (2%)	$ 80,000
Total Project Cost	**$ 5,080,000**

West Side Flow Recommendation

The recommended option is Alternative B – the new relief line across town on SW Maple and SE Elm Streets – based on construction costs.

6.1.2 WWTP Disposal Alternatives

1. Continue Year-Round Surface Water Discharge (No Action):

   As discussed in Sections 1 and 5, with continued year-round discharge to the Multnomah Channel there may be additional permit limits in the future.

2. Farmland Application:

   This alternative involves land application of all treated effluent. This would likely require the purchase of approximately 2,500 acres of farmland, in addition to costs for pipeline, and winter storage reservoirs when the treated effluent cannot be land-applied. The feasibility of finding sufficient land close enough to the City to be economically feasible is unknown; however, the cost to purchase the required land would likely be significant. Future treatment requirements for reuse may be less stringent than for continued discharge to the Multnomah Channel.

3. Summer Farmland Application and Winter Surface Water Discharge:

   The third alternative includes discharge to the Multnomah Channel on a seasonal (wet-weather) basis during the winter, and agricultural reuse during the summer. Summer farmland application of the treated wastewater would decrease the discharge loading to the Multnomah Channel and provide operational flexibility to the WWTP. This alternative would require the acquisition of the same amount of farmland (approximately 2,500 acres) as Alternative 2 and pipeline installation, but would not require large winter storage reservoirs.

Disposal Recommendation

Due to the likely significant cost to purchase the required land, and the unlikelihood of finding sufficient land close enough to the WWTP, the recommendation is Alternative 1 – Continue Year-Round Surface Water Discharge (No Action), as this option does not currently require any additional disposal expenses. It is possible that recycled water may have more appeal to the City in the future, and storage/piping costs can be phased into future plans.

6.1.3 Headworks Alternatives

Two options were evaluated to correct issues at the headworks.

1. Construct a New Headworks:

   A new headworks would be constructed. For the purpose of this comparison, it is assumed that the existing influent screen could be relocated and reused inside the new headworks. A preliminary cost estimate for this option is summarized in Table 6-3.

Table 6-3: New Headworks Cost Estimate

Item	Cost (2016)	
Site Work	$	50,000
New Building	$	700,000
New Influent Screens and Channels	$	470,000
Relocate Existing Screen	$	20,000
Grit Chambers and Classifiers	$	900,000
New Influent Pumps	$	210,000
Piping/Valves and Instrumentation	$	30,000
Electrical/Controls	$	360,000
Mobilization (10%)	$	280,000
Overhead and Profit (15%)	$	420,000
Contingency (30%)	$	830,000
Construction Subtotal	**$**	**4,270,000**
Engineering & CMS (20%)	$	860,000
Total Project Cost	**$**	**5,130,000**
Estimated Annual O&M	$	42,000
Total Present Value	**$**	**5,880,000**

2. Upgrade and Expand Existing Headworks:

   The existing headworks issues listed in Section 3 would be addressed with this alternative. The headworks would also be expanded with additional influent screens, grit removal, a new influent flow meter, and new influent pumps to provide capacity for future flows. A preliminary cost estimate is summarized in Table 6-4.

Table 6-4: Repair and Expand Headworks Cost Estimate

Item	Cost (2016)
Demolition and Heat Tape	$ 70,000
Expand Cover and Channels	$ 60,000
New Influent Screens and Channels	$ 470,000
Grit Chambers and Classifiers	$ 900,000
New Influent Pumps	$ 210,000
Piping/Valves and Instrumentation	$ 30,000
HVAC	$ 90,000
Electrical/Controls	$ 460,000
Mobilization (10%)	$ 230,000
Overhead and Profit (15%)	$ 350,000
Contingency (30%)	$ 690,000
Construction Subtotal	**$ 3,560,000**
Engineering & CMS (20%)	$ 720,000
Total Project Cost	**$ 4,280,000**
Estimated Annual O&M	$ 44,000
Total Present Value	**$ 5,060,000**

Headworks Recommendation

Based on the construction cost, the recommended option is Alternative 2 – Upgrade and Expand Existing Headworks.

6.1.4 Primary Treatment Alternatives

Primary clarifiers are sometimes used to decrease the size of the secondary treatment system. However, the solids removed in primary clarifiers are typically treated using anaerobic digesters. As discussed later in this section, aerobic digesters are recommended for biosolids treatment rather than anaerobic digesters; therefore, primary clarifiers were not evaluated further.

6.1.5 Secondary Treatment Alternatives

As mentioned in the previous sections, there are several issues with the existing secondary treatment process that need to be addressed; it is recommended that the existing aeration basin either be demolished or modified and repurposed. Three alternatives were evaluated for secondary treatment, which have included considerations for a future ammonia limit (additional aeration capacity), as well as future total nitrogen and total phosphorus removal (larger basins).

1. Conventional Activated Sludge (New Aeration Basins):

The new aeration basins would include diffused aeration, divider walls, and submersible mixers. A mixed liquor recycle pump is also included for internal recycle to create an A^2O or Modified Ludzack-Ettinger (MLE) process. This alternative also includes constructing a new blower building, two (2) additional secondary clarifiers, a secondary flow splitter box, fixing the clarifier wiring, upgrading the intermediate pump station to handle future peak flows, and expanding and upgrading the existing sludge pumping building.

A preliminary cost estimate for this alternative is shown in Table 6-5.

Table 6-5: New Aeration Basins Cost Estimate

Item	Cost (2016)
Demolition and Site Work	$ 380,000
Influent Splitter Box	$ 70,000
New Aeration Basins	$ 1,920,000
Mixers/Pumps	$ 210,000
Fine Bubble Diffusers	$ 110,000
Blowers	$ 900,000
Misc. Metals (guardrail, grating)	$ 360,000
Piping/Valves and Instrumentation	$ 600,000
Blower Building	$ 380,000
Effluent Splitter Box	$ 70,000
Secondary Clarifiers	$ 840,000
Existing Clarifier Wiring	$ 20,000
Upgrade Sludge Pump Building (including pumps)	$ 440,000
Upgrade Intermediate Pump Station	$ 200,000
Electrical/Controls	$ 1,340,000
Mobilization (10%)	$ 790,000
Overhead and Profit (15%)	$ 1,180,000
Contingency (30%)	$ 2,360,000
Construction Subtotal	**$ 12,170,000**
Engineering & CMS (20%)	$ 2,440,000
Total Project Cost	**$ 14,610,000**
Estimated Annual O&M	$ 198,000
Total Present Value	**$ 18,120,000**

2. Oxidation Ditches:

   New oxidation ditches would include aerators and mixers. This alternative also includes adding two (2) secondary clarifiers and a secondary flow splitter box, fixing the clarifier wiring, and expanding and upgrading the existing sludge pumping building.

   A preliminary cost estimate is shown in Table 6-6.

Table 6-6: Oxidation Ditches Cost Estimate

Item	Cost (2016)	
Demolition and Site Work	$	380,000
Influent Splitter Box	$	70,000
Oxidation Ditches	$	2,120,000
Oxidation Ditch Equipment	$	1,260,000
Misc. Metals (guardrail, grating)	$	360,000
Piping/Valves and Instrumentation	$	500,000
Effluent Splitter Box	$	70,000
Secondary Clarifiers (including equipment)	$	840,000
Existing Clarifier Wiring	$	20,000
Upgrade Sludge Pump Building (including pumps)	$	440,000
Upgrade Intermediate Pump Station	$	200,000
Electrical/Controls	$	1,440,000
Mobilization (10%)	$	770,000
Overhead and Profit (15%)	$	1,160,000
Contingency (30%)	$	2,310,000
Construction Subtotal	**$**	**11,940,000**
Engineering & CMS (20%)	$	2,390,000
Total Project Cost	**$**	**14,330,000**
Estimated Annual O&M	$	210,000
Total Present Value	**$**	**18,050,000**

3. Sequencing Batch Reactor (SBR) Facility:

   A new SBR facility would include SBR and post-equalization basins, diffused aeration mixers, decanters, sludge pumps, transfer pumps, and a blower building. The existing secondary clarifiers, sludge pump building, and intermediate pump station would be abandoned in place and could be repurposed or demolished.

   A preliminary cost estimate is shown in Table 6-7.

Table 6-7: SBR Facility Cost Estimate

Item	Cost (2016)
Site Work	$ 300,000
Influent Splitter Box	$ 70,000
SBR and Post-Equalization Basins	$ 1,920,000
Equipment (including Blowers and Pumps)	$ 1,760,000
Misc. Metals (guardrail, grating)	$ 360,000
Piping/Valves and Instrumentation	$ 550,000
Blower Building	$ 380,000
Electrical/Controls	$ 1,610,000
Mobilization (10%)	$ 700,000
Overhead and Profit (15%)	$ 1,050,000
Contingency (30%)	$ 2,090,000
Construction Subtotal	**$ 10,790,000**
Engineering & CMS (20%)	$ 2,160,000
Total Project Cost	**$ 12,950,000**
Estimated Annual O&M	$ 202,000
Total Present Value	**$ 16,530,000**

A summary of the advantages and disadvantages of each secondary treatment technology is provided in Table 6-8.

TABLE 6-8: Summary of Secondary Treatment Advantages and Disadvantages

Technology	Advantages	Disadvantages
Conventional Activated Sludge (New Aeration Basins)	• Same technology as used currently at WWTP. • Effluent flow equilization is not required.	• Requires construction of additional clarifiers. • Requires more equipment than other alternatives. • Can be sensitive to peak flows or loads.
Oxidation Ditch	• Resilient to peak flows. • Effluent flow equilization is not required.	• Requires construction of additional clarifiers. • Effluent suspended solids concentrations can be higher than the other two alternatives. • The aeration system may create more aerosols. • Largest footprint of the alternatives.
Sequencing Batch Reactor (SBR) Facility	• Lowest capital and 20 year life-cycle costs. • Changes can be quickly made based on cycle time. • Smallest footprint. • Does not require clarifiers.	• Most automated technology of the three. • Effluent equalization is recommended so downstream processes can handle the decant flow. • Peak flows can disrupt process. Influent equalization may be desired. • Due to controls, expansion phasing can be difficult.

Secondary Treatment Recommendation

Based on the advantages and disadvantages, and since the Conventional Activated Sludge technology is the most familiar to the City and it utilizes existing equipment, Alternative 1 – Conventional Activated Sludge (New Aeration Basins) is the recommended alternative.

6.1.6 Filtration Alternatives

Since it is fairly straightforward to add disks to the existing units and an additional filter to meet the future flows, expanding the existing filters was the only option considered for upgrading the filtration system.

6.1.7 Disinfection Alternatives

The UV system has been having problems with seals and system efficiency, likely due to its age. Additionally the system is not sized for future flows. Three alternatives were evaluated to address the UV system deficiencies.

1. Upgrade and Expand UV System:

   The existing UV system could be upgraded and expanded. A preliminary cost estimate is summarized in Table 6-9.

Table 6-9: Upgrade and Expand UV Cost Estimate

Item	Cost (2016)
Expand Channels	$ 50,000
UV Equipment	$ 600,000
Electrical/Controls	$ 90,000
Mobilization (10%)	$ 74,000
Overhead and Profit (15%)	$ 111,000
Contingency (30%)	$ 222,000
Construction Subtotal	**$ 1,147,000**
Engineering & CMS (20%)	$ 230,000
Total Project Cost	**$ 1,377,000**
Estimated Annual O&M	$ 49,000
Total Present Value	**$ 2,250,000**

2. Implement Chlorination and Dechlorination:

   This alternative would include increasing the size of the old chlorine contact basin for future flows. A well-functioning chemical dosing system (both for chlorination and dechlorination) is necessary for this system to be reliable.

   A preliminary cost estimate is summarized in Table 6-10.

Table 6-10: Chlorination and Dechlorination Cost Estimate

Item	Cost (2016)
Demolish and Expand Basin	$ 180,000
Chemical Building	$ 80,000
Chlorination/Dechlorination Equipment	$ 80,000
Chlorine Monitoring Equipment	$ 10,000
Electrical/Controls	$ 70,000
Mobilization (10%)	$ 50,000
Overhead and Profit (15%)	$ 70,000
Contingency (30%)	$ 130,000
Construction Subtotal	**$ 670,000**
Engineering & CMS (20%)	$ 140,000
Total Project Cost	**$ 810,000**
Estimated Annual O&M	$ 48,000
Total Present Value	**$ 1,660,000**

3. Implement Disinfection with Peracetic Acid (PAA):

   This alternative would include reusing the old chlorine contact basin. Although PAA has been approved for use by the EPA, it is still a fairly new technology and may not have full approval by DEQ.

   A preliminary cost estimate is summarized in Table 6-11.

Table 6-11: Peracetic Acid (PAA) Cost Estimate

Item	Cost (2016)
Demolish and Expand Basin	$ 180,000
Chemical Building	$ 80,000
PAA Equipment	$ 90,000
Electrical/Controls	$ 70,000
Mobilization (10%)	$ 50,000
Overhead and Profit (15%)	$ 70,000
Contingency (30%)	$ 130,000
Construction Subtotal	**$ 670,000**
Engineering & CMS (20%)	$ 140,000
Total Project Cost	**$ 810,000**
Estimated Annual O&M	$ 78,000
Total Present Value	**$ 2,190,000**

A summary of the advantages and disadvantages of each disinfection technology is provided in Table 6-12.

Table 6-12: Summary of Disinfection Advantages and Disadvantages

Technology	Advantages	Disadvantages
Upgrade and Expand UV System	• Same technology as used currently at WWTP • Eliminates the need to generate, handle, transport, or store toxic/hazardous or corrosive chemicals. • No residual effect that can be harmful to humans or aquatic life. • Requires shorter contact time compared to other disinfectants (approximately 20 to 30 seconds with low-pressure lamps). • Requires less space than other methods.	• Low dosage may not effectively inactivate some viruses, spores, and cysts. • Organisms can sometimes repair and reverse the destructive effects of UV. • A preventive maintenance program is necessary to control fouling of tubes. • Turbidity and total suspended solids (TSS) in the wastewater can render UV disinfection ineffective. Low-pressure lamps are not as effective for secondary effluent with TSS levels above 30 mg/L. • Not as cost-effective as chlorination, but costs are competitive when chlorination and de-chlorination is used and fire codes are met.
Implement Chlorination and Dechlorination	• Well-established technology. • Can be more cost-effective than UV disinfection (dechlorination and fire code requirements can make it cost more than UV disinfection). • Chlorine residual remaining in the effluent can prolong disinfection even after initial treatment, and can be measured to evaluate effectiveness. • Reliable and effective against a wide spectrum of pathogenic organisms. • Effective in oxidizing certain organic and inorganic compounds. • Beneficial for recycled water to have a chlorine residual for pipeline maintenance. • Flexible dosing control. • Can eliminate certain noxious odors during disinfection.	• Chlorine residual, even at low concentrations, is toxic to aquatic life and will require a well-controlled de-chlorination system. • All forms of chlorine are highly corrosive and toxic, so storage, shipping, and handling pose a risk, requiring increased safety regulations. • Oxidizes some organic matter in wastewater to create more hazardous compounds (disinfection byproducts such as trihalomethanes [THMs] are regulated and would require additional treatment). • Level of total dissolved solids is increased in the treated effluent. • Chlorine residual is unstable in the presence of high concentrations of chlorine-demanding materials, thus requiring higher doses to effect adequate disinfection. • Some parasitic species have shown resistance to low doses of chlorine. • Long-term effect of discharging de-chlorinated compounds into the environment is unknown.
Implement Disinfection with Peracetic Acid (PAA)	• Newer technology for wastewater disinfection in the US. • Lower dose and less contact time is needed for PAA when compared to chlorination/dechlorination. • Not as prone to freezing and more stable than chlorine. • Enhances UV effectiveness and reduces cleaning frequency when combined with UV.	• Less corrosive and toxic than chlorine, so storage, shipping, and handling are less hazardous. • Less likely to form hazardous byproducts than chlorine. • Although it has been approved by EPA, it may not have full approval by the DEQ. • Does not maintain a residual in the effluent. • Increases effluent BOD concentration. • Piloting is recommended.

Disinfection Recommendation

Although it is not the lowest-cost option, because of its advantages and the disadvantages of the other alternatives, Alternative 1 – Upgrade and Expand UV System is recommended.

6.1.8 Biosolids Treatment Alternatives

The existing biosolids treatment process is undersized to produce Class B biosolids for the future projected waste sludge volumes. Two options were evaluated for biosolids treatment.

1. New Aerobic Digesters:

   The new aerobic digesters would be large enough to achieve Class B biosolids (60-day SRT in the winter). The digester blowers would be located in a blower building, while the sludge pumps and flow meter would be located in a dewatering building.

   A preliminary cost estimate for this alternative is shown in Table 6-13.

Table 6-13: New Aerobic Digester Cost Estimate

Item	Cost (2016)
Site Work	$ 180,000
Digester Basins	$ 760,000
Misc. Metals (guardrail, grating)	$ 200,000
Piping/Valves and Instrumentation	$ 120,000
Digester Equipment (including Blowers)	$ 490,000
Digester Blower Building	$ 230,000
Electrical/Controls	$ 300,000
Mobilization (10%)	$ 230,000
Overhead and Profit (15%)	$ 350,000
Contingency (30%)	$ 690,000
Construction Subtotal	**$ 3,550,000**
Engineering & CMS (20%)	$ 710,000
Total Project Cost	**$ 4,260,000**
Estimated Annual O&M	$ 97,000
Total Present Value	**$ 5,980,000**

2. New Anaerobic Digesters:

   The new anaerobic digesters would be sized to achieve Class B biosolids. To decrease the size of the digesters, gravity belt thickeners would be included, increasing the sludge concentration from approximately 1% to 4% solids. A control building would be constructed to house the pumps, heat exchangers, and boilers.

   A preliminary cost estimate is shown in Table 6-14.

Table 6-14: Anaerobic Digesters Cost Estimate

Item	Cost (2016)
Demolition and Site Work	$ 180,000
Gravity Belt Thickener (including polymer)	$ 350,000
Gravity Belt Thickener Building	$ 200,000
Anaerobic Digesters (including equipment)	$ 2,200,000
Misc. Metals (guardrail, grating)	$ 200,000
Digester Control Building	$ 900,000
Electrical/Controls	$ 610,000
Mobilization (10%)	$ 470,000
Overhead and Profit (15%)	$ 700,000
Contingency (30%)	$ 1,400,000
Construction Subtotal	**$ 7,210,000**
Engineering & CMS (20%)	$ 1,450,000
Total Project Cost	**$ 8,660,000**
Estimated Annual O&M	$ 60,000
Total Present Value	**$ 9,730,000**

A summary of the advantages and disadvantages of aerobic and anaerobic digestion is provided in Table 6-15.

Table 6-15: Summary of Biosolids Treatment Advantages and Disadvantages

Technology	Advantages	Disadvantages
Aerobic Digestion	• Lower capital cost and 20 year life-cycle cost. • The process is easier to start-up and control. • Lower ammonia and BOD in the return stream. • Lower potential for odors. Explosive gases do not have to be dealt with. • Same technology as used currently at WWTP.	• More energy/power required to aerobically treat sludge than to anaerobically treat sludge. • Aerobically digested solids are more difficult to dewater than anaerobically digested solids. • Temperatures are not controlled, so the process fluctuates with changes in temperature.
Anaerobic Digestion	• The biogas produced by the anaerobic digesters potentially can be used to heat the digesters. • Less energy/power required to treat sludge. • Sludge is more easily dewatered. • More sludge digestion occurs (lower sludge disposal costs).	• Higher capital cost and 20 year life-cycle cost. • More difficult to start-up and control. • Higher ammonia and BOD in the return stream. • Greater safety risk due to explosive gases.

Biosolids Treatment Recommendation

The recommendation is Alternative 1 – Aerobic Digesters, as it has the lowest total present value.

6.1.9 Biosolids Disposal Alternatives

There are only two options presented below for biosolids disposal. Since the interim biosolids handling plan includes the dewatering building, initial screw press, and dewatered solids storage facility, these costs were not used in this evaluation.

1. Class B – Dewatered Biosolids:

   This alternative continues the current practice of having a farmer apply the solids. The solids would be dewatered using screw presses, and the dewatered biosolids stored under a cover in the eastern lagoon area.

   A preliminary cost estimate for this alternative is shown in Table 6-16.

Table 6-16: Class B – Dewatered Biosolids Cost Estimate

Item	Cost (2016)
Addl. Equipment (Screw Presses, Polymer, Pump, Conveyor)	$ 770,000
Electrical/Controls	$ 120,000
Mobilization (10%)	$ 89,000
Overhead and Profit (15%)	$ 134,000
Contingency (30%)	$ 267,000
Construction Subtotal	**$ 1,380,000**
Engineering & CMS (20%)	$ 276,000
Total Project Cost	**$ 1,656,000**
Estimated Annual O&M	$ 80,000
Total Present Value	**$ 3,080,000**

2. Class A – Compost (Covered Aerated Static Pile):

   This alternative would produce Class A biosolids to be applied to farmers' fields. Composting also requires dewatering equipment (such as screw presses) to provide a consistent solids concentration. Costs are higher with this alternative since it also includes all costs of the first alternative; however, there are advantages with regards to disposal.

   A preliminary cost estimate is shown in Table 6-17.

Table 6-17: Class A – Compost (Covered Aerated Static Pile) Cost Estimate

Item	Cost (2016)
Addl. Screw Presses, Polymer, Pump, Conveyor	$ 770,000
Composting Equipment	$ 1,800,000
Laydown Area	$ 450,000
Electrical/Controls	$ 460,000
Mobilization (10%)	$ 350,000
Overhead and Profit (15%)	$ 530,000
Contingency (30%)	$ 1,050,000
Construction Subtotal	**$ 5,410,000**
Engineering & CMS (20%)	$ 1,090,000
Total Project Cost	**$ 6,500,000**
Estimated Annual O&M	$ 500,000
Total Present Value	**$ 15,350,000**

Disposal costs or sales are not included, but it is assumed that ultimate disposal would continue to be agricultural land application. A summary of the advantages and disadvantages of the alternatives is provided in Table 6-18.

Table 6-18: Summary of Biosolids Disposal Advantages and Disadvantages

Technology	Advantages	Disadvantages
Class B	• Lower capital and 20 year life-cycle costs. • Does not require additional treatment from what is currently used at the WWTP.	• Requires prior land approval and more extensive record keeping regarding application. • Land application or disposal of Class B biosolids is becoming more difficult.
Class A - Compost (Covered Aerated Static Pile)	• Less restrictive with regard to application area.	• Higher capital and 20 year life-cycle costs. • Requires bulking agent with acceptable and uniform characteristics. • Requires additional monitoring. • More energy and space is required.

Biosolids Disposal Recommendation

The recommended alternative is to continue Class B land application until there is a driver to make up for the additional costs to produce Class A biosolids.

6.1.10 Dewatering Building Alternatives

Two building options were evaluated to house the biosolids dewatering equipment.

1. Repurpose Metal Shop:

   This alternative would install the biosolids dewatering equipment (new screw press, sludge transfer pump, polymer dosing system, and dewatered sludge conveyor) into an existing (approximately 40 ft. x 30 ft.) metal shop. This option also includes a covering for a dumpster adjacent to the shop. It is assumed that all other capital improvements for the interim biosolids plan would be incorporated into this option, which includes the aerators and covered storage area. Dewatered biosolids would be moved from the dumpster to the storage area. The existing metal shop is not large enough to house three screw presses, so an addition to the shop is included. The metal shop has an existing heater, which is assumed to be adequate. Heating for the shop would be more expensive than with the new building option.

A preliminary cost estimate for this alternative is shown in Table 6-19.

Table 6-19: Repurpose Metal Shop Cost Estimate

Item	Cost (2016)	
Site Work	$	50,000
Equipment (screw press, polymer, pump, conveyor)	$	440,000
Piping/Valves and Instrumentation	$	50,000
Cover for Dumpster	$	20,000
Expand Shop	$	100,000
Aerobic Digester Aerators	$	70,000
Concrete for Sludge Storage Area	$	50,000
Cover for Sludge Storage Area	$	100,000
Electrical/Controls	$	130,000
Mobilization (10%)	$	110,000
Overhead and Profit (15%)	$	160,000
Contingency (30%)	$	310,000
Construction Subtotal	**$**	**1,590,000**
Engineering & CMS (20%)	$	320,000
Total Project Cost	**$**	**1,910,000**
Estimated Annual O&M	$	153,000
Total Present Value	**$**	**4,620,000**

2. New Dewatering Building:

This alternative would construct a new dewatering building (40 ft. x 50 ft.), which could be used to house current and future dewatering equipment. It is assumed that the building would be constructed using concrete masonry unit (CMU) blocks, which would retain heat better than the existing metal shop. The life expectancy is also longer, and the maintenance less, with a CMU building.

A preliminary cost estimate is shown in Table 6-20.

Table 6-20: New Dewatering Building Cost Estimate

Item	Cost (2016)
Site Work	$ 50,000
Dewatering Building	$ 400,000
Equipment (screw press, polymer, pump, conveyor)	$ 440,000
Piping/Valves and Instrumentation	$ 50,000
Aerobic Digester Aerators	$ 70,000
Concrete for Sludge Storage Area	$ 50,000
Cover for Sludge Storage Area	$ 100,000
Electrical/Controls	$ 160,000
Mobilization (10%)	$ 140,000
Overhead and Profit (15%)	$ 200,000
Contingency (30%)	$ 400,000
Construction Subtotal	**$ 2,060,000**
Engineering & CMS (20%)	$ 420,000
Total Project Cost	**$ 2,480,000**
Estimated Annual O&M	$ 126,000
Total Present Value	**$ 4,710,000**

Dewatering Building Recommendation

Although over a 20-year period, repurposing the shop has the lowest net present value, to avoid the cost of rehabbing the shop in the future, Alternative 2 – New Dewatering Building is recommended.

7. PROPOSED PROJECT (RECOMMENDED ALTERNATIVES)

This section consists of the recommended plan to address wastewater system deficiencies. A location map showing the changes to the collection system and wastewater treatment plant are shown in Figures 22 and 23 (Appendix A), respectively.

7.1 PRELIMINARY PROJECT DESIGN

7.1.1 Collection System

Detailed project summary sheets for the collection system are located in Appendix G. Each project sheet provides the objective, key issues, cost estimate, and project location map. The recommended alternative for the collection system is Alternative B.

7.1.2 Pipeline Cleaning and CCTV

Cleaning and CCTV inspection work has been subcontracted out in the past. Pipelines should be cleaned approximately every three years (frequency can be adjusted based on pipe material plus scour conditions and observations by City staff), because a scum buildup will typically form within two years of operation and is a precursor to corrosion. Approximately 56,600 feet/year should be cleaned to cover the entire system every three years. As a general recommendation, concrete pipelines should be CCTV inspected about every 5 years, as they are more susceptible to corrosion. PVC pipelines should be CCTV inspected about every 10 years, primarily to check for any bellies or sags that may have formed or, pipeline joints that may have separated. Problematic areas may be cleaned and inspected every year or two, or more regularly as required. Areas with adverse grades or large sags may require more frequent attention.

7.1.3 Service Lines

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

7.1.4 Flow Monitoring

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

7.1.5 Pipeline Replacement Program

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

Keller Associates recommends the City begin budgeting for replacement/rehabilitation of an average of 2,300 feet of the collection pipeline system each year. This amount would allow replacement of the entire system within about 75 years, the estimated useful life of pipelines. Any concrete pipes in the system should be replaced first. The linear feet of pipeline or number of manholes replaced each year is an average and should be adjusted based on future CCTV and other maintenance records. The costs associated with funding an on-going replacement/rehabilitation program are summarized in Table 7-1 and discussed further in Section 7.6.

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

Table 7-1: Replacement Budgets

Item	Lifespan	Cost/Year
Pipelines	75 year	$ 408,000
Manholes	50 year	$ 102,000
Cleanouts	50 year	$ 4,000
Laterals/Cleanouts	50 year	$ 81,000
	Total	$ 595,000

7.1.6 Lift Station Improvements

Lift station improvements are included in the detailed project sheets (Appendix G). Each project summary sheet provides the objective, key issues, and cost estimate. Both Spring Lake and Highway 30 Lift Stations' modifications are in Priority 1C. The Smith Road, Keys Landing, and Seven Oaks Lift Stations' modifications are part of Priority 2C.

7.1.7 WWTP Improvements

WWTP improvements are included in the detailed project sheets (Appendix G). Each project summary sheet provides the objective, key issues, cost estimate, and project location map. The recommended improvements for the WWTP are summarized below.

- *Headworks* – The headworks should be upgraded to include influent screens, an influent pipe, influent flow measurement, and influent pumps that have sufficient capacity for the future peak instantaneous flow. Freeze protection should be added to the influent screens. Grit removal should also be included to protect downstream equipment from wear due to grit. Accurate level measurement and enhanced influent pump controls should be provided. The HVAC system should be upgraded to create an unclassified environment in accordance with NFPA 820 requirements for influent pumps. Also, a sump pump should be added to the pump dry pit to provide redundancy.

- *Secondary Treatment* – New aeration basins should replace the existing aeration basin. These new basins should include diffused aeration, divider walls, and submersible mixers. A mixed liquor recycle pump should be included for internal recycle to create an A^2O or Modified Ludzack-Ettinger (MLE) process. This upgrade should also include constructing a blower building, an influent flow splitter box, two (2) additional secondary clarifiers, and a secondary effluent flow splitter box; fixing the clarifier wiring; upgrading the intermediate pump station to handle the future peak flows; and expanding and upgrading the existing sludge pumping building.
- *Filtration* – Disks should be added to the existing filter units, and an additional filter should be added to meet future flows.
- *Disinfection* – The existing UV system should be upgraded and expanded.
- *Effluent Pump Station* – The effluent pumps should be upgraded, and the effluent pipe size should be increased to meet future flows.
- *Biosolids Treatment* – New aerobic digesters should be added to achieve Class B biosolids (60-day SRT in the winter). The digester blowers should be located in the blower building, while sludge pumps and a sludge flow meter should be located in a dewatering building.
- *Biosolids Disposal* – The current practice of having a farmer apply the solids should be continued until there is a driver to make up the additional cost difference to produce Class A biosolids. The solids should be dewatered using screw presses, and the dewatered biosolids should be stored under a cover in a refurbished eastern solids lagoon.
- *Interim Biosolids Plan* – Due to the urgent nature of the biosolids treatment and storage, an interim biosolids handling plan was created in September 2016. The interim improvements were recommended to meet Class B requirements for current sludge flows, maximize existing resources, and position the City to implement future improvements outlined in this facility planning study. The interim biosolids handling plan included the following:

  i. Installing four (4) 7.5-horsepower and two (2) 15-horsepower surface aerators in the western biosolids storage lagoon to provide aeration and mixing in order to achieve Class B treatment for current sludge flows.

  ii. Modifying the existing gates/pipes between the biosolids storage lagoons to cut off the interconnection of flow.

  iii. Constructing a new dewatering building to house a new screw press, sludge transfer pump, polymer dosing system, and dewatered sludge conveyor.

  iv. Installing piping from the existing wet well between biosolids storage lagoons to the new dewatering building.

  v. Converting the eastern biosolids storage lagoon into a dewatered sludge storage area by installing concrete in the lagoon, along with a drain system to remove water back to the headworks. A frontend loader would be used to move the sludge around the lagoon, and to load it into trucks for transport. A cover (such as a clear span) would be added over the area to prevent rain and snow from wetting the dewatered sludge. Access to the storage area would be provided from the existing plant road.

vi. The WWTP operating staff are experimenting with a probiotic solution to decrease the sludge volume. However, if the western biosolids storage lagoon reaches capacity, temporary dewatering equipment may be needed. This equipment (e.g., geotextile dewatering containers, rented screw press, etc.) would require a sludge pump, polymer system, and insulated and heat-traced pipe (if the piping is outside and above-grade). It would also need an asphalt or concrete surface that drains to a sump, or pipe that returns to the headworks.

7.1.8 Other

A formal SCADA system should be added to provide process trending, as well as alarms, to the City staff. The existing control panels and new SCADA system should not be located in the lab area.

Currently, the plant uses groundwater for seal water and washdown activities. By installing a chlorination system and pumps, the City could use treated, non-potable, wastewater from the chlorine contact chamber for plant water.

7.2 PROJECT SCHEDULE

An estimated schedule for Priority 1 improvements is shown in Table 7-2. Individual schedules for each project will be further refined at a later date by the City during the pre-design phase for each proposed improvement. Costs presented here are planning-level estimates. Actual costs may vary depending on market conditions and shall be updated as projects are further refined in the pre-design and design phases.

Table 7-2: Priority 1 Capital Improvement Plan

ID#	Item	Cost	Opinion of Probable Costs (2016 Dollars)		
			2017	2018	2019
Wastewater Collection System					
1A.1	New, Relief Trunk Line	$ 1,720,000	$ 206,400	$ 1,513,600	
1A.2	E Columbia Ave Trunk Line	$ 1,290,000	$ 154,800	$ 1,135,200	
1A.3	SE Tyler St and SE Tussing Wy Trunk Line	$ 630,000	$ 75,600	$ 554,400	
1A.4	SW Em Watts Rd Trunk Line	$ 270,000	$ 32,400	$ 237,600	
1B	NW Smith Road Trunk Line	$ 160,000		$ 19,200	$ 140,800
1C	Lift Station Improvements	$ 410,000			$ 410,000
Wastewater Treatment					
1a	Interim Biosolids Plan	$ 2,530,000	$ 2,530,000		
1b	Rehabilitate UV System	$ 373,000	$ 373,000		
1c.1	Add 3rd pump to Inter. Pump Station	$ 35,000	$ 35,000		
1c.2	Add disks to existing Tertiary Filters	$ 97,000	$ 97,000		
1d	SCADA System	$ 297,000	$ 148,500	$ 148,500	
1e.1	Aeration for Aeration Basin	$ 341,000			$ 341,000
1e.2	Sec. Clarifier and Sludge Bldg. Exp.	$ 2,190,000		$ 262,800	$ 1,927,200
Total		**$ 10,343,000**	**$ 3,652,700**	**$ 3,871,300**	**$ 2,819,000**

* All costs in 2016 Dollars. Costs include mobilization (**10%**), contractor overhead and profit (**OH&P; 15%**), contingency (**30%**), engineering and construction management services (**CMS; 20-30%**), and legal, administrative, and permitting services (**2%**).

The City-projected growth is significant, and the initial WWTP improvements should not be sized to handle the full revised peak flows. Instead, a phased approach for WWTP improvements is presented in the CIP. Future recorded peak flows and updated flow projections for the plant will trigger subsequent phases of the improvements. The full 20-year CIP shown in Table 7-3 includes flow triggers for each priority project, which will help City staff track and determine when improvements need to be completed as the City grows. Table 7-3 also includes an estimated number of additional EDUs that will increase system flows to meet each flow trigger. For these estimates, an EDU was assumed to be a low-density, single-dwelling unit with an associated $MMWWF_5$ flow of 450 gpd/EDU and PIF_5 flow of 978 gpd/EDU. Expandability will be a key component of the recommended WWTP improvements. This will allow the plant to operate efficiently while planning for future growth.

7.3 PERMIT REQUIREMENTS

The City's NPDES discharge permit is in DEQ's queue for renewal, which may affect certain parameters of treatment. In communications with DEQ, they were unable to provide specifics at this time. Possible future permit requirements were discussed in Section 1. The recommendations set forth in the CIP could allow the plant upgrades to be flexible to meet the anticipated permit requirements.

7.4 SUSTAINABILITY CONSIDERATIONS

7.4.1 Water and Energy Efficiency

The proposed Priority 2 improvements incorporate diffused aeration, which saves energy versus surface aerators. Upgrading the UV system increases the efficiency, which allows for better treatment using less power. The use of treated wastewater for plant washdown water decreases the demand/use of groundwater.

7.4.2 Green Infrastructure

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

7.4.3 Other

The proposed alternatives incorporate the use of SCADA into many aspects of the treatment system. This allows for better system resiliency and operation simplicity, as well as improved system optimization.

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

The summary of the Scappoose wastewater facility improvement costs is in Table 7-3 (Capital Improvement Plan). The percent SDC eligibility factored in the existing design flow, existing capacity, and improved capacity. The amount of capacity that can be utilized for future connections is divided by the future capacity in 2035. For projects that did not have an increase in flows, the percent SDC eligible is derived from the percent growth in population over the 20-year planning period. Costs shown are planning-level estimates and can vary depending on market conditions; they shall be updated as the project is further refined in the pre-design and design phases.

Table 7-3: 20-Year Capital Improvement Plan

ID#	Item	WWTP Flow Trigger	Additional EDUs to Meet Flow Trigger	Total Estimated Cost (2016)	SDC Growth Portion		City's Estimated Portion
					%	Cost	
Priority 1 Improvements							
Wastewater Collection System							
1A.1	New, Relief Trunk Line	--	--	$ 1,720,000	30%	$ 516,000	$ 1,204,000
1A.2	E Columbia Ave Trunk Line	--	--	$ 1,290,000	59%	$ 761,000	$ 529,000
1A.3	SE Tyler St and SE Tussing Wy Trunk Line	--	--	$ 630,000	49%	$ 309,000	$ 321,000
1A.4	SW Em Watts Rd Trunk Line	--	--	$ 270,000	35%	$ 95,000	$ 175,000
1B	NW Smith Road Trunk Line	--	--	$ 160,000	6%	$ 10,000	$ 150,000
1C	Lift Station Improvements	--	--	$ 410,000	0%	$ -	$ 410,000
Wastewater Treatment							
1a	Interim Biosolids Plan	Beyond Capacity	--	$ 2,530,000	33%	$ 843,000	$ 1,687,000
1b	Rehabilitate UV System	--	--	$ 373,000	15%	$ 55,000	$ 318,000
1c.1	Add 3rd pump to Inter. Pump Station	Beyond Capacity	--	$ 35,000	70%	$ 25,000	$ 10,000
1c.2	Add disks to existing Tertiary Filters	At Capacity	--	$ 97,000	100%	$ 97,000	$ -
1d	SCADA System	--	--	$ 297,000	63%	$ 188,000	$ 109,000
1e.1	Aeration for Aeration Basin	Beyond Capacity	--	$ 341,000	33%	$ 114,000	$ 227,000
1e.2	Sec. Clarifier and Sludge Bldg. Exp.	Beyond Capacity	--	$ 2,190,000	100%	$ 2,190,000	$ -
Total Priority 1 Improvements (rounded)				**$ 10,340,000**		**$ 5,200,000**	**$ 5,140,000**
Rate Impact (20 yr, 1.6%)				**$ 18.77**			**$ 9.33**
Priority 2 Improvements							
Wastewater Collection System							
2A	SE 6th St Trunk Line	--	--	$ 610,000	100%	$ 610,000	$ -
2B	NE Laurel St and NE 3rd St Trunk Line	--	--	$ 370,000	100%	$ 370,000	$ -
2C	Lift Station Improvements	--	--	$ 240,000	26%	$ 62,000	$ 178,000
Wastewater Treatment							
2a.1	New Aeration Basins	1.9 MGD MMWWF$_5$	1,530	$ 7,750,000	54%	$ 4,173,000	$ 3,577,000
2a.2	New Aerobic Digester	1.8 MGD MMWWF$_5$	1,310	$ 2,020,000	48%	$ 966,000	$ 1,054,000
2b.1	Expand Headworks	4.1 MGD PIF$_5$	1,500	$ 3,410,000	63%	$ 2,163,000	$ 1,247,000
2b.2	Upgrade Influent Pumps	3.5 MGD PIF$_5$	880	$ 928,000	63%	$ 589,000	$ 339,000
2c.1	Upgrade Effluent Pumps	3.3 MGD PIF$_5$	675	$ 833,000	63%	$ 528,000	$ 305,000
2c.2	Increase Effluent Pipe	4.0 MGD PIF$_5$	1,400	$ 2,100,000	63%	$ 1,332,000	$ 768,000
2d.1	Upgrade Intermediate Pump Station	4.0 MGD PIF$_5$	1,400	$ 455,000	100%	$ 455,000	$ -
2d.2	Additional Tertiary Filter Unit	1.8 MGD MMWWF$_5$	1,310	$ 877,000	100%	$ 877,000	$ -
2e	Upgrade UV System	3.1 MGD PIF$_5$	480	$ 1,117,000	63%	$ 709,000	$ 408,000
Total Priority 2 Improvements (rounded)				**$ 20,710,000**		**$ 12,830,000**	**$ 7,880,000**
Priority 3 Improvements							
Wastewater Collection System							
3A	SW Old Portland Rd Trunk Line	--	--	$ 280,000	100%	$ 280,000	$ -
3B	SE Tussing Wy Trunk Line	--	--	$ 50,000	40%	$ 20,000	$ 30,000
Wastewater Treatment							
3a.1	Additional Aeration Basin	2.6 MGD MMWWF$_5$	3,090	$ 3,220,000	100%	$ 3,220,000	$ -
3a.2	Additional Secondary Clarifier	3.4 MGD MMWWF$_5$	4,870	$ 1,320,000	100%	$ 1,320,000	$ -
3b.1	Additional Aerobic Digester	2.3 MGD MMWWF$_5$	2,420	$ 1,830,000	100%	$ 1,830,000	$ -
3b.2	Additional Screw Presses	2.3 MGD MMWWF$_5$	2,420	$ 1,684,000	100%	$ 1,684,000	$ -
3c	Plant Water System	--	--	$ 208,000	63%	$ 132,000	$ 76,000
Total Priority 3 Improvements (rounded)				**$ 8,590,000**		**$ 8,490,000**	**$ 110,000**
Priority 4 Improvements							
Wastewater Collection System							
4A	P.LS 1, Force Main and Gravity Line	--		$ 660,000	100%	$ 660,000	$ -
4B	P.LS 2, Force Main and Gravity Line	--		$ 1,160,000	100%	$ 1,160,000	$ -
4C	P.LS 3 and Force Main	--		$ 750,000	100%	$ 750,000	$ -
4D	P.LS 4, Force Main and Gravity Line	--		$ 1,210,000	100%	$ 1,210,000	$ -
Total Priority 4 Improvements (rounded)				**$ 3,780,000**		**$ 3,780,000**	**$ -**
TOTAL WASTEWATER IMPROVEMENTS COSTS (rounded)				**$ 43,420,000**		**$ 30,300,000**	**$ 13,130,000**

* All costs in 2016 Dollars. Costs include mobilization (**10%**), contractor overhead and profit (**OH&P; 15%**), contingency (**30%**), engineering and construction management services (**CMS; 20-30%**), and legal, administrative, and permitting services (**2%**).

7.6 FINANCIAL STATUS OF EXISTING FACILITIES

The financial information statement for the City of Scappoose sewer utility is located in Appendix H. This information includes the latest two resolutions regarding sewer rates. The rate was increased to $30.21 per EDU in 2006, and then to $43.31 per EDU in 2015 (took effect in 2016). Estimated total wastewater fund resources for the 2015-2016 fiscal year are approximately $1,838,516. The annual charges for services are estimated to be $1,069,500 of the $1,838,516.

Annual O&M costs for the wastewater system, separated by type of expense, are also shown in Appendix H. For the 2015-2016 fiscal year, the estimated total expenditures for the wastewater fund is approximately $1,838,516.

The City took out construction loans for the sewer utility in 2011 and 2014. The 2011 loan is through DEQ with 0.00% interest that will mature in 2031. Its balance as of June 30, 2015, was $273,441. The 2014 loan is a $700,000 loan from US Bank, with an interest rate of 2.47%. That loan will mature in 2020 and has a balance (as of June 30, 2014) of $644,460. The annual debt service of both loans is $132,594. The DEQ loan has a holding or reserve requirement of one half of the average annual debt service – $8,821. The US Bank loan does not have any holding requirements.

7.7 ANNUAL OPERATING BUDGET

An itemized annual operating budget for the fiscal year 2015-2016 is provided in Appendix H. Additional information on budget specifics can be found in the following sections.

7.7.1 Potential User Rate Impacts

The existing sewer rate schedule consists of a monthly flat rate fee of $43.31 per EDU. The most recent sewer rate increase was in 2015 (took effect in 2016). The current number of EDUs is estimated at 2,700.

The rate impacts assume, as directed by City staff, that none of the existing revenue/budget can be used annually to offset future capital improvements or sewer rehabilitation/replacement budgets.

Table 7-4 shows the existing and potential monthly charge for sewer services for one EDU. Funding for the recommended system improvements may come from any number of sources. This section presents potential user rate impacts if priority improvements are funded only through a low-interest loan with debt service payments (20-year, 1.6%) made through a user rate increase. Calculations for the user rate impact are referenced in Appendix H. The user rate impacts assume that SDC funds or commercial/industrial development contributions will be available for Priority 1 (urgent need). The user rate impacts also assume, as directed by City staff, there is no surplus in the annual budget contributing to the annual debt service payment.

In the event that grant funds, low-interest loans, or principal forgiveness could be obtained, then the user rate impacts would be less than those illustrated in Table 7-4.

Keller Associates recommends that the City actively pursue these opportunities that would mitigate user rate impacts.

Table 7-4: User Rate Impact

	Annual Paymet (20 years, 1.6%)	User Rate Increase	User Rate Total
Existing User Rate (2016)	--	--	$ 43.31
Priority 1 Improvements[1,2]	$ 302,343	$ 9.33	$ 52.64

[1] Including capital improvements only.
[2] Assuming SDCs are funding portion attributed to growth.

The anticipated user rate impacts presented in Table 7-4 do not account for potential O&M costs associated with the recommended treatment improvements. The rate increases also do not account for recommended replacement/rehabilitation or short-lived asset replacement budgets. These budget items will require further analyzing by the City's financial consultant as part of a full-rate study. The rate impacts for pipeline replacement/rehabilitation are usually delayed until after the capital improvement plan for the collection system is completed. This is due to the fact that the CIP should be replacing more than the average linear footage recommended for the annual replacement/rehabilitation. Keller Associates does not anticipate that future collection system improvements will increase the City's O&M requirements. In fact, with new facilities and pipelines, it may be the case that maintenance demands for the collection system could decrease slightly. It should be noted that all costs are in 2016 dollars; the City should plan on annual increases in user rates of 2-5% to account for inflation.

7.7.2 System Development Charges

The City's current sewer System Development Charge (SDC) for a single-family home is $4,276.04. The sewer SDC is typically divided into two components: reimbursement and growth. The scope of this study included estimating the SDC eligibility for each identified capital improvement. It is the intent that this information will be utilized by the City's financial consultant to update the City's SDCs. The estimated SDC eligibility for each identified capital improvement is shown in Table 7-3.

7.7.3 Annual O&M Costs

In addition to the capital improvement costs presented in Table 7-3, the following expected annual operating costs are recommended for consideration in setting annual budgets:

- Additional collection system replacement/rehabilitation needs: City should eventually budget an additional $595,000/year (to be either contracted out or completed using City crews).
- Collection system cleaning and CCTV needs: following the timeline described in Section 7.1.2, the City should budget approximately $103,000/year.

- Other additional annual O&M costs and short-lived assets associated with Priority 1 WWTP CIP (dewatering equipment, additional aeration, filters, pumping, disinfection, etc.) are anticipated to be approximately $200,000/year. Short-lived assets include pump replacements, motor replacements, etc. A large portion of this is associated with increased power usage.


APPENDIX A - Figures

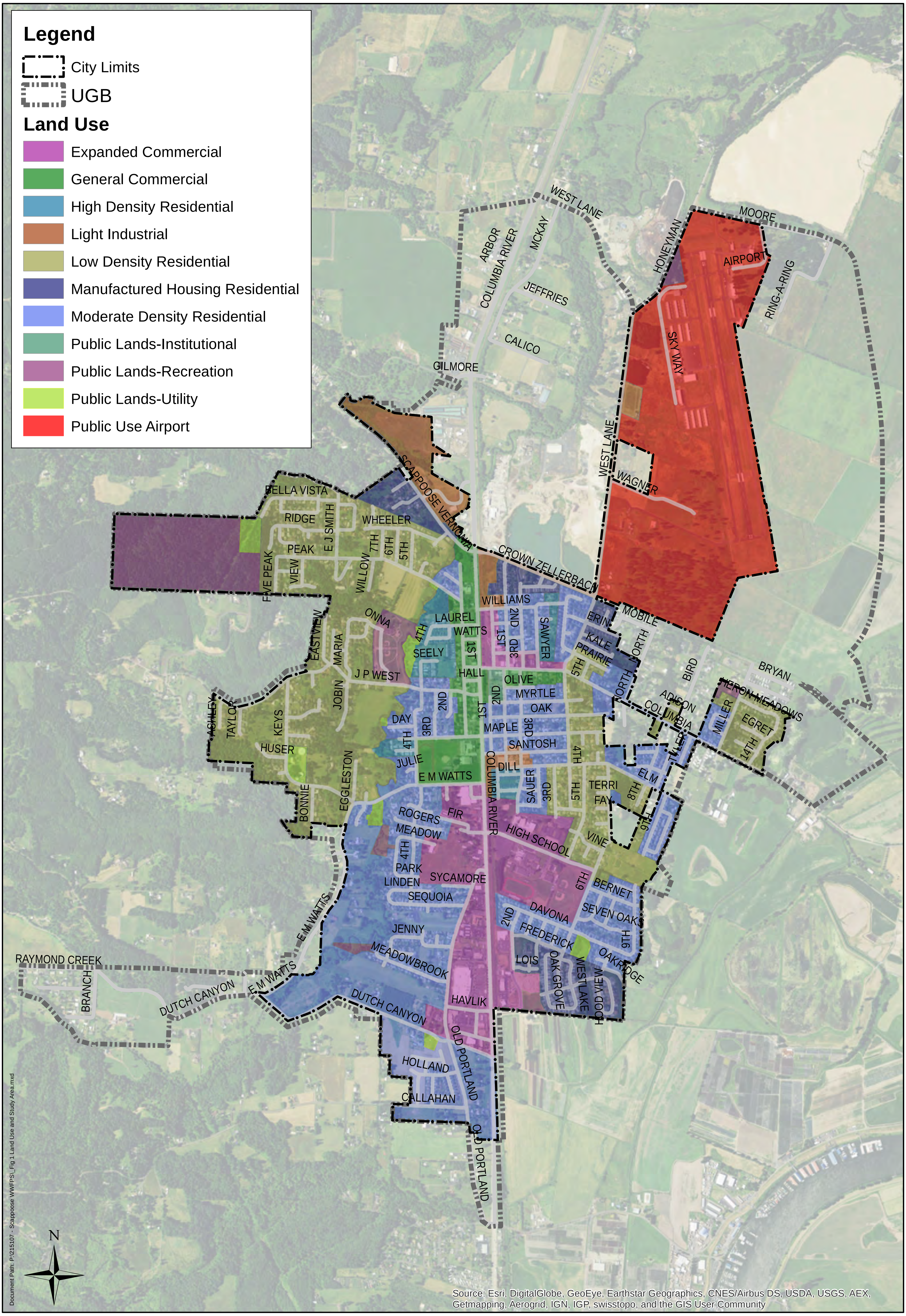


Figure:	Title:	Project:	Prepared for:
1	LAND USE AND STUDY AREA	WWFPS	CITY OF SCAPPOOSE, OR

KELLER associates

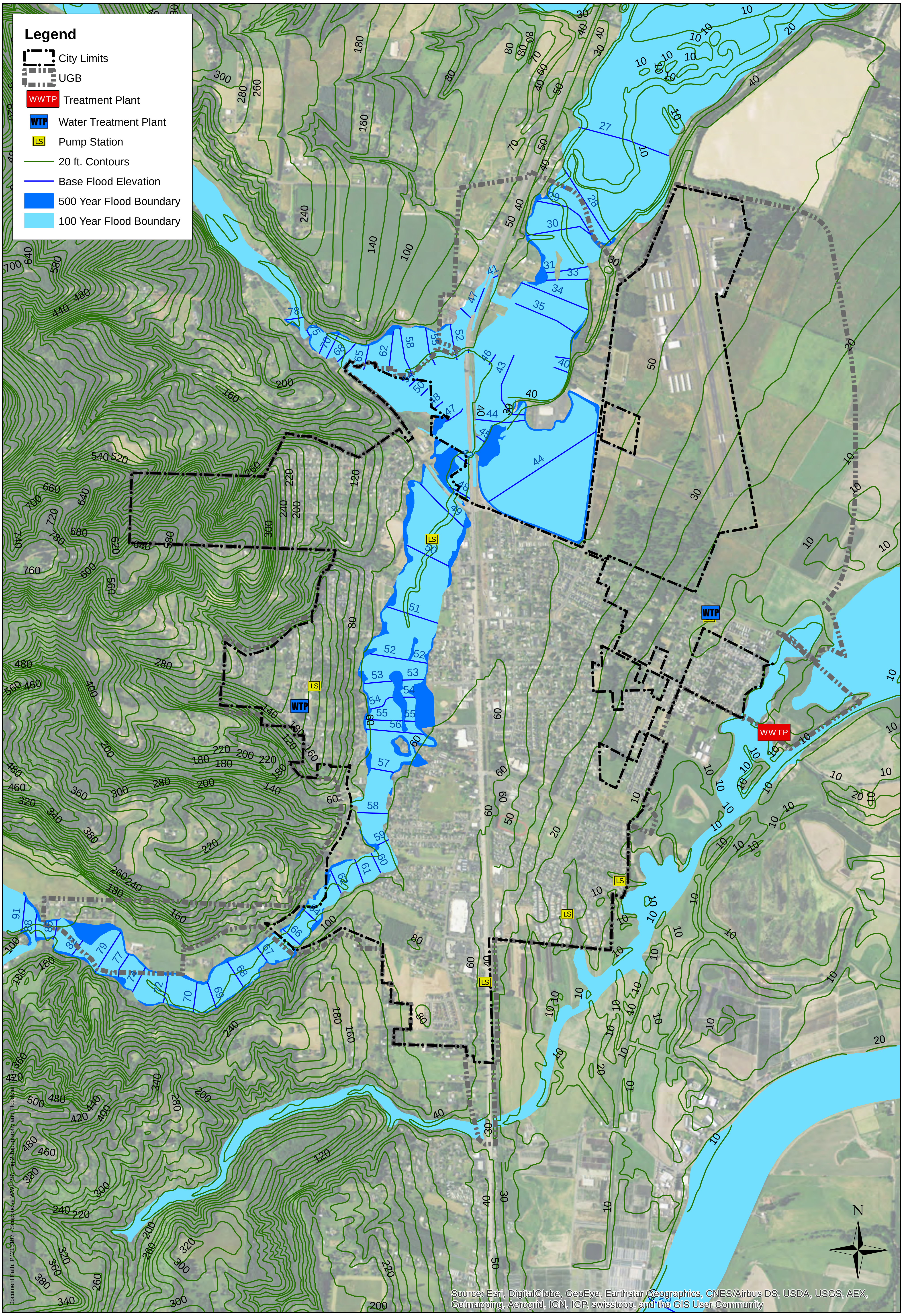


Figure:	Title:	Project:	Prepared for:
2	TOPOGRAPHY AND FLOOD PLAIN	WWFPS	CITY OF SCAPPOOSE, OR

Legend

- City Limits
- UGB

Farmland

- Prime
- Statewide Importance
- If Drained
- Aloha silt loam
- Bacona silt loam
- Cascade silt loam
- Cloquato silt loam
- Cornelius silt loam
- Crims silt loam
- Dayton silt loam
- Dowde silt loam
- Goble silt loam
- Hapludalfs-Udifluvents complex
- Latourell silt loam
- McBee silt loam
- Quafeno loam
- Quatama silt loam
- Rafton silt loam
- Rafton-Sauvie-Moag complex
- Sauvie silt loam
- Sauvie silty clay loam
- Scaponia-Braun silt loams
- Sifton gravelly loam
- Sifton loam
- Vernonia silt loam
- Wapato silt loam
- Water
- Wauld very gravelly loam
- Xerochrepts, steep

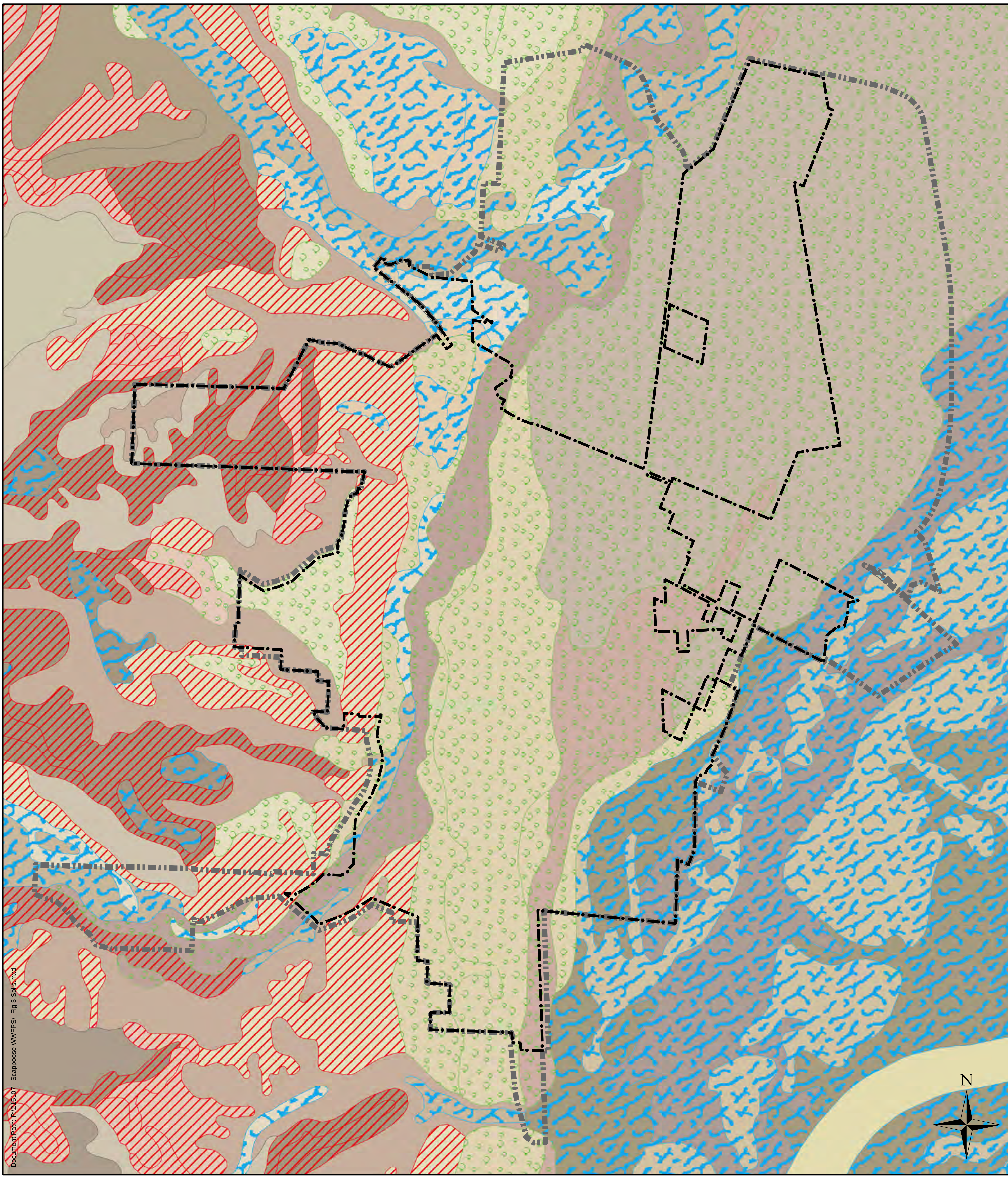
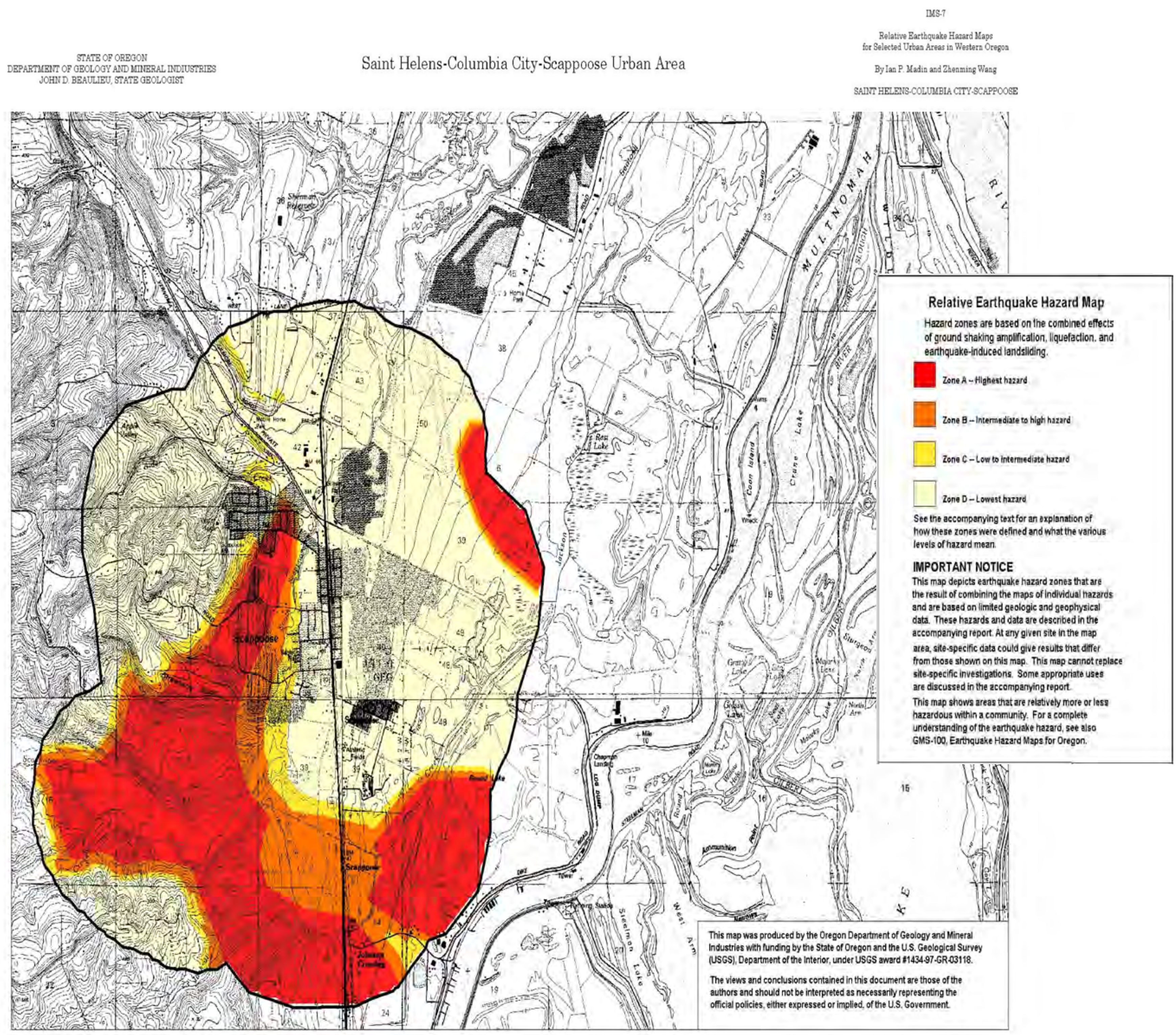


Figure:	Title:	Project:	Prepared for:
3	NRCS SOIL SURVEY	WWFPS	CITY OF SCAPPOOSE, OR

KELLER associates



Document Path: P:\215107 - Scappoose WWFPS_Fig 4 Relative Earthquake Hazard.mxd

Figure:	Title:	Project:	Prepared for:
4	RELATIVE EARTHQUAKE HAZARD MAP	WWFPS	CITY OF SCAPPOOSE, OR

KELLER associates

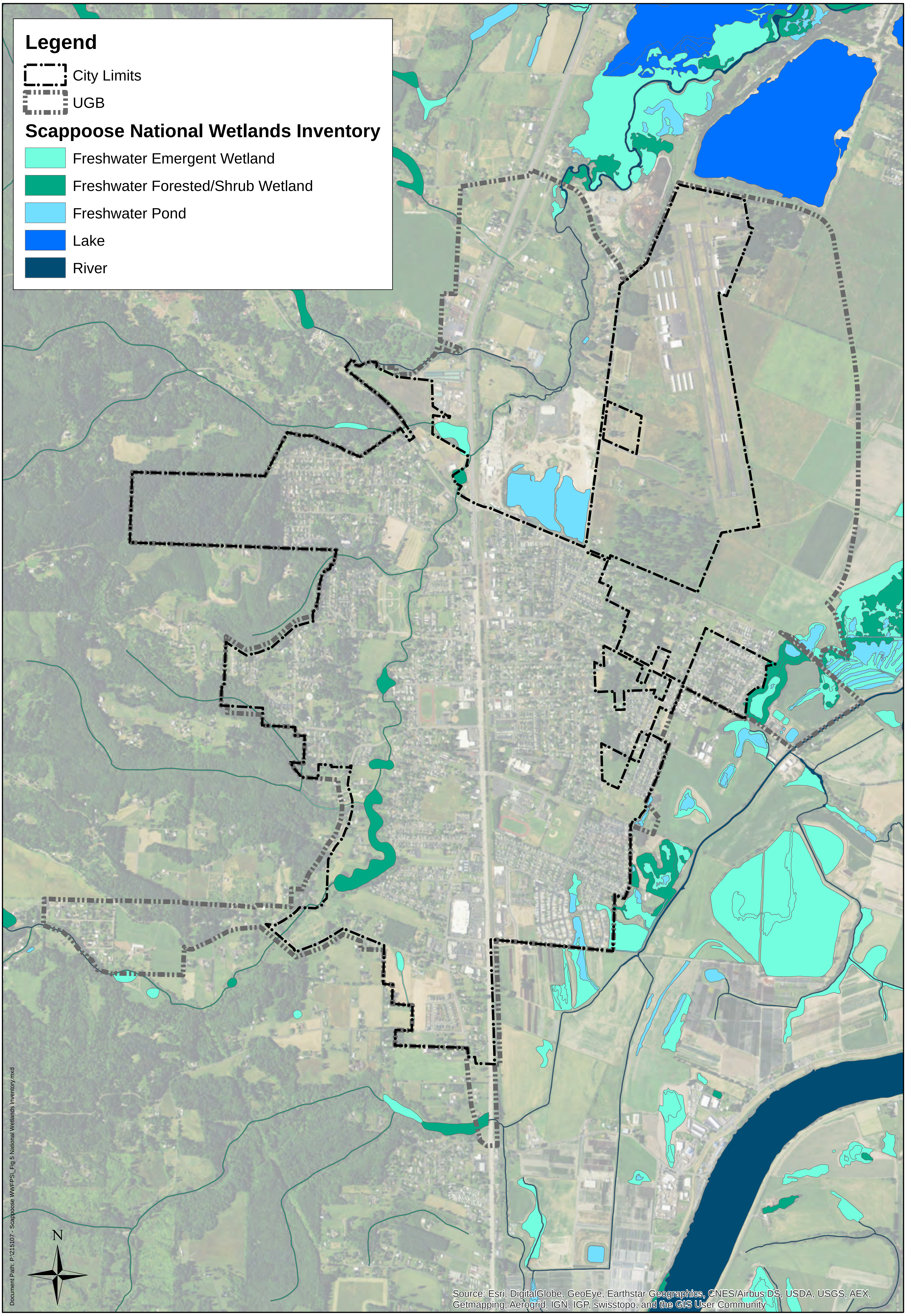


Figure:	Title:	Project:	Prepared for:		
5	NATIONAL WETLANDS INVENTORY	WWFPS	CITY OF SCAPPOOSE, OR		KELLER associates

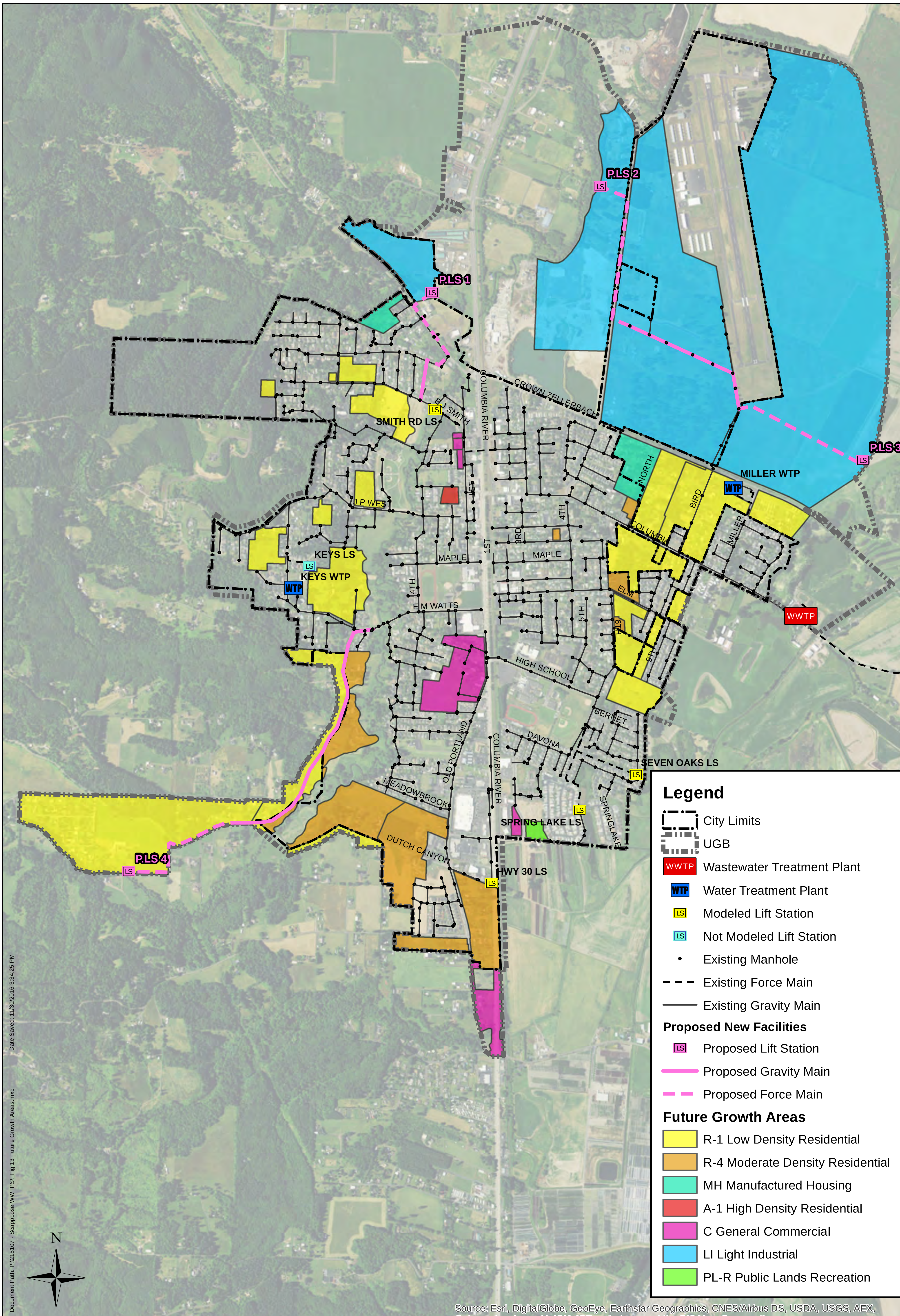


Figure:	Title:	Project:	Prepared for:
6	FUTURE GROWTH AREAS	WWFPS	CITY OF SCAPPOOSE, OR

KELLER associates

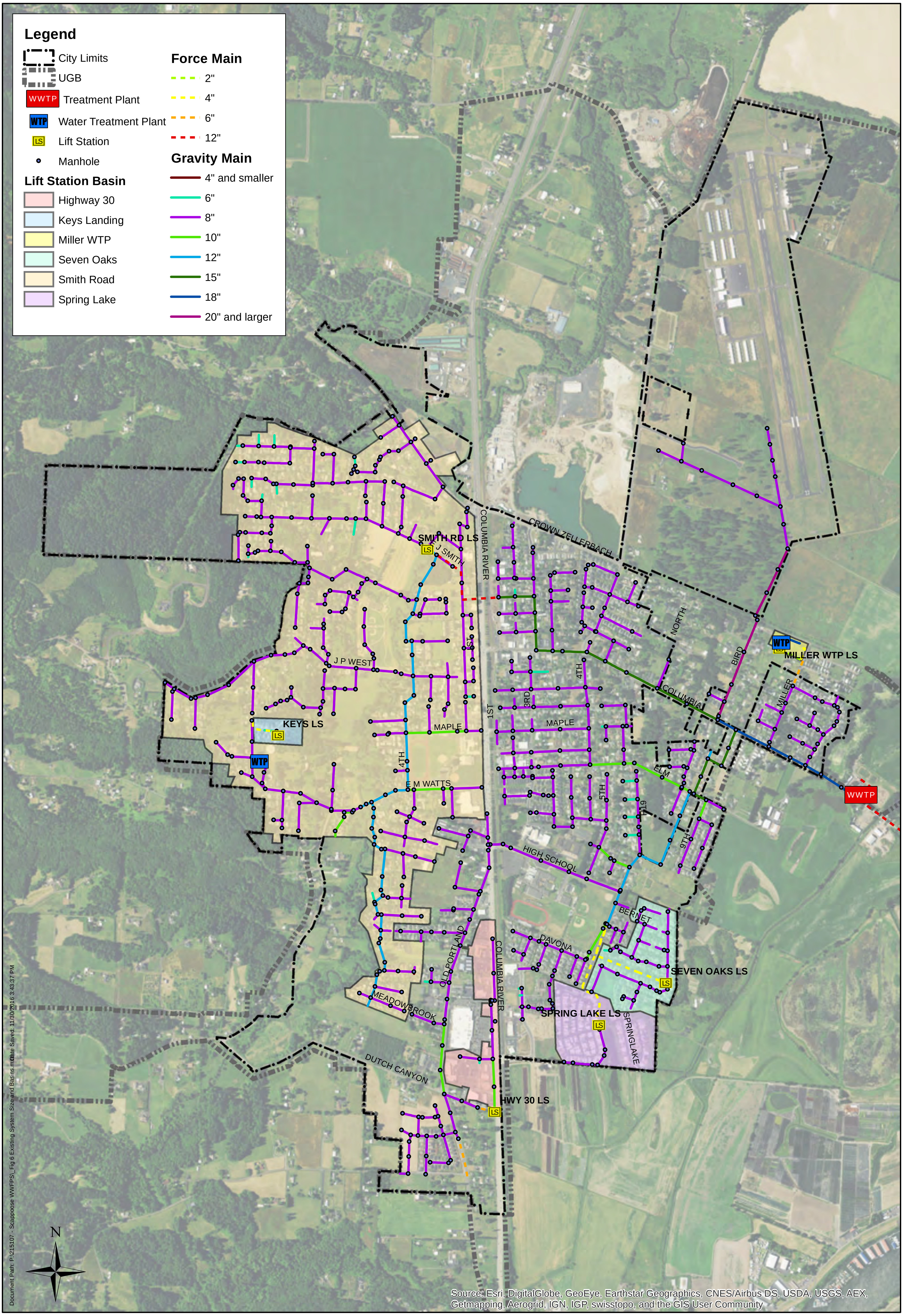


Figure:	Title:	Project:	Prepared for:
7	EXISITING SYSTEM PIPE DIAMETER & LIFT STATION BASINS	WWFPS	CITY OF SCAPPOOSE, OR

KELLER associates

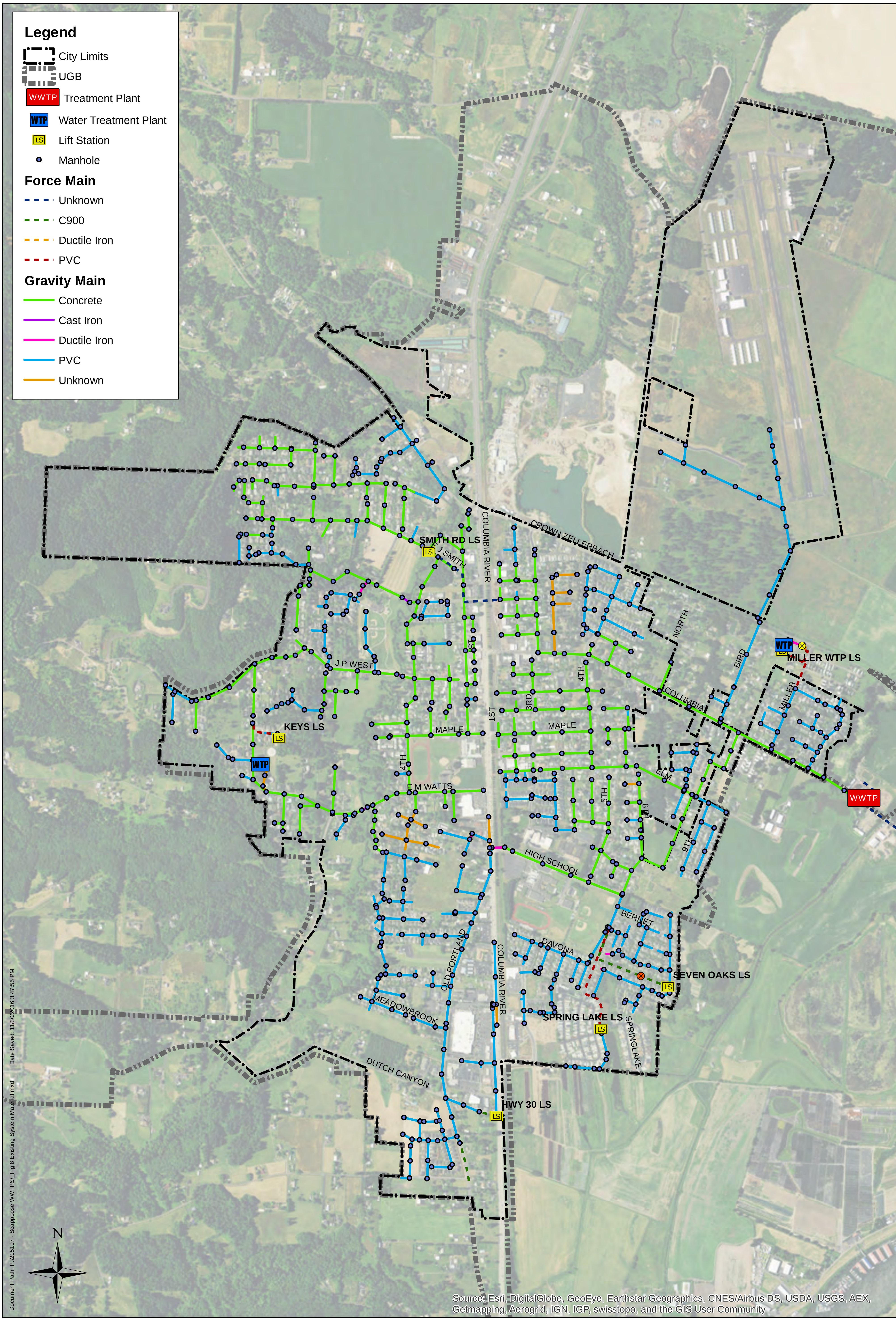


Figure:	Title:	Project:	Prepared for:
8	EXISTING SYSTEM PIPE MATERIAL	WWFPS	CITY OF SCAPPOOSE, OR

KELLER associates

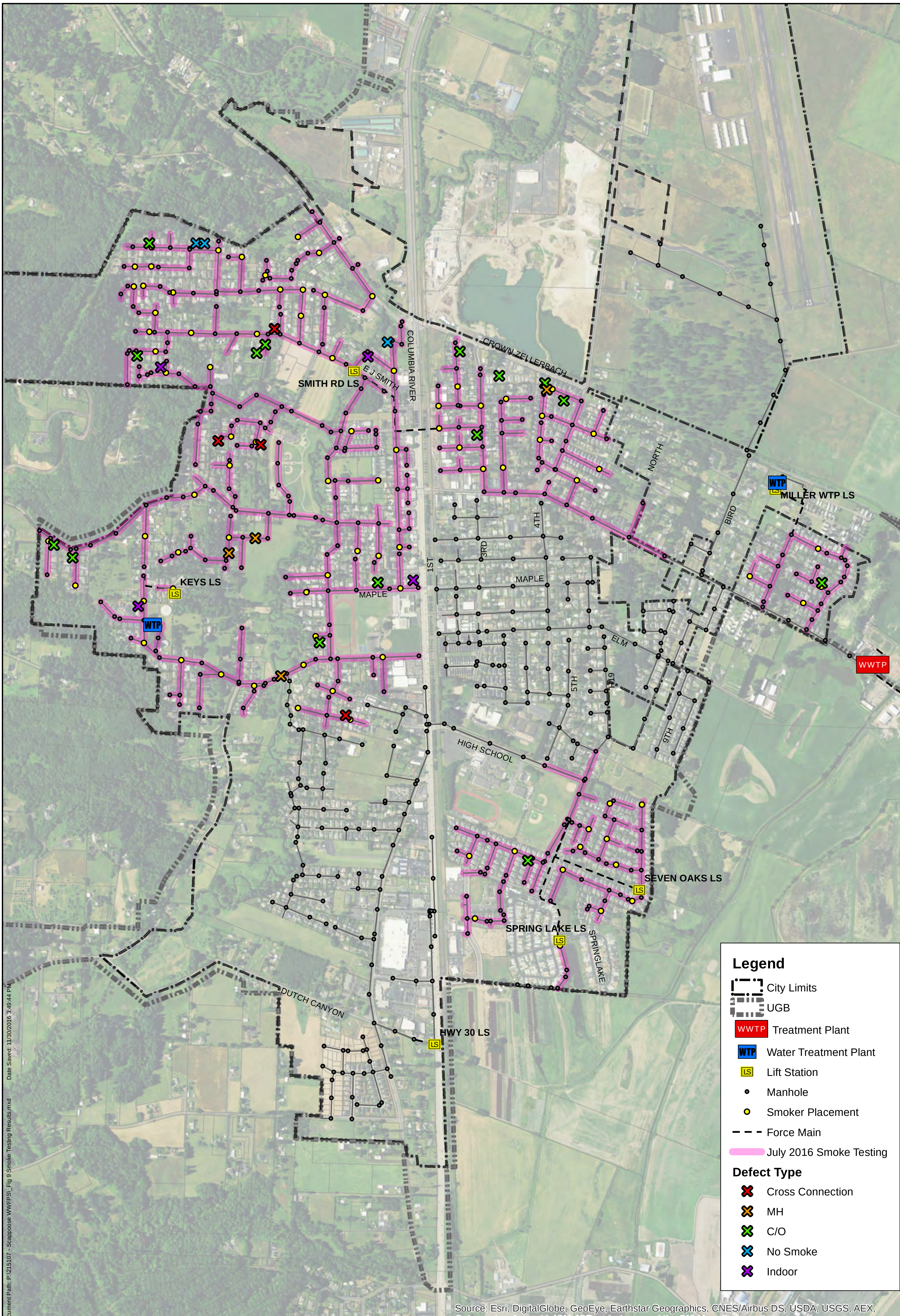


Figure:	Title:	Project:	Prepared for:		
9	SMOKE TESTING	WWFPS	CITY OF SCAPPOOSE, OR		KELLER associates

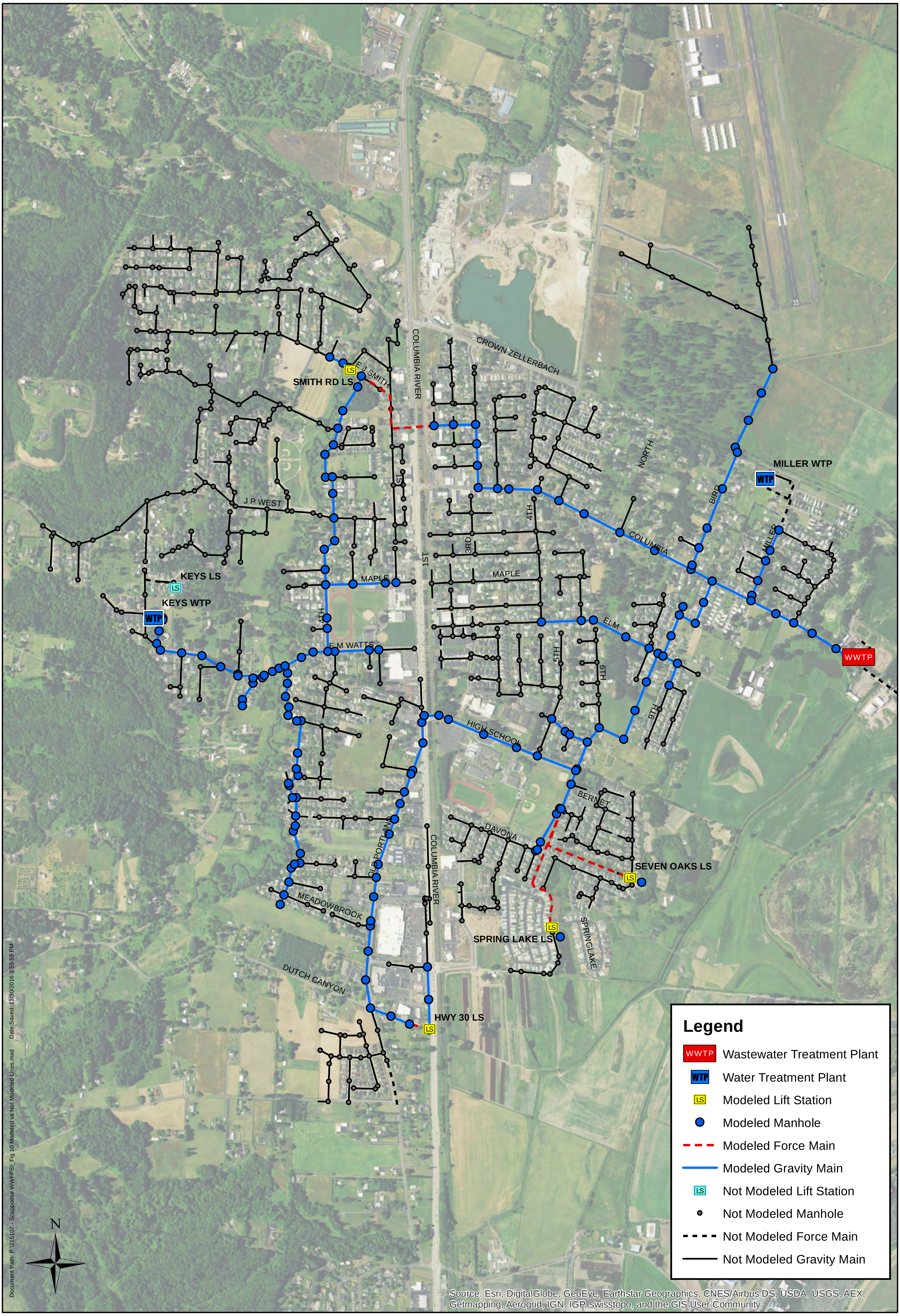


Figure:	Title:	Project:	Prepared for:		
10	MODELED VS NOT MODELED MH AND LINES	WWFPS	CITY OF SCAPPOOSE, OR		KELLER associates

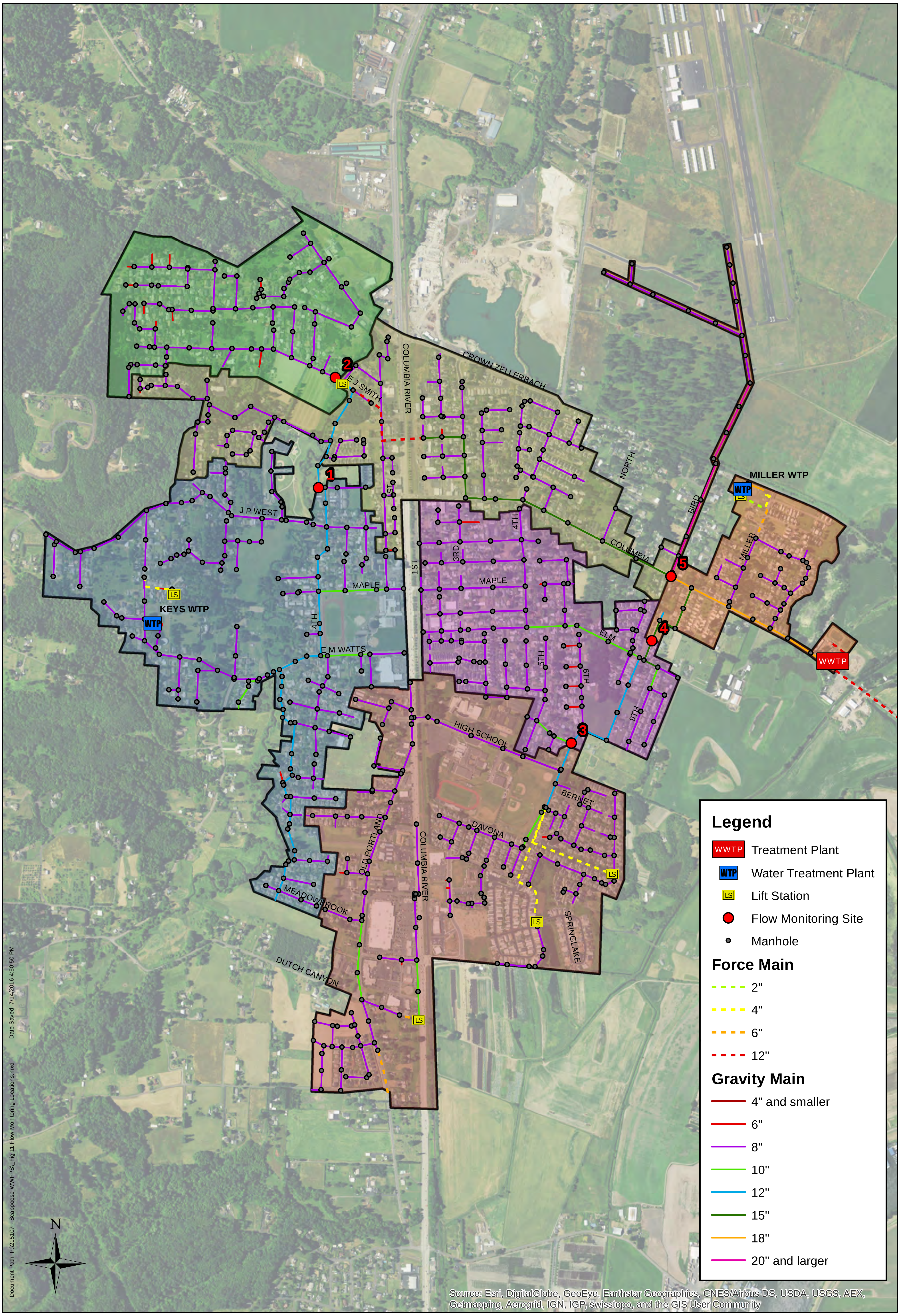


Figure:	Title:	Project:	Prepared for:
11	FLOW MONITORING LOCATIONS/BASINS	WWFPS	CITY OF SCAPPOOSE, OR

KELLER associates

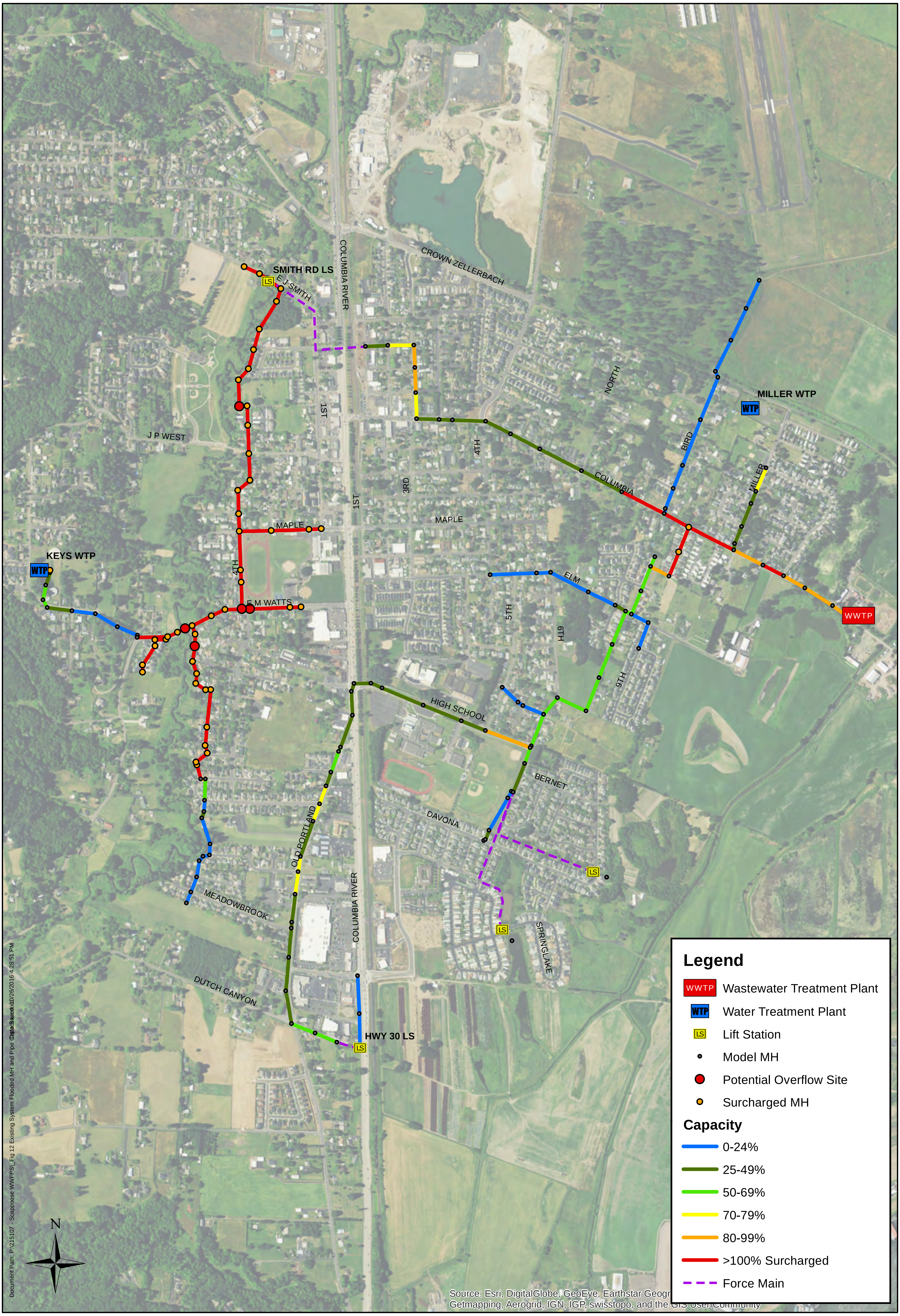


Figure:	Title:	Project:	Prepared for:
12	EXISTING SYSTEM (DESIGN FLOW) POTENTIAL OVERFLOW SITES AND PIPE CAPACITY	WWFPS	CITY OF SCAPPOOSE, OR

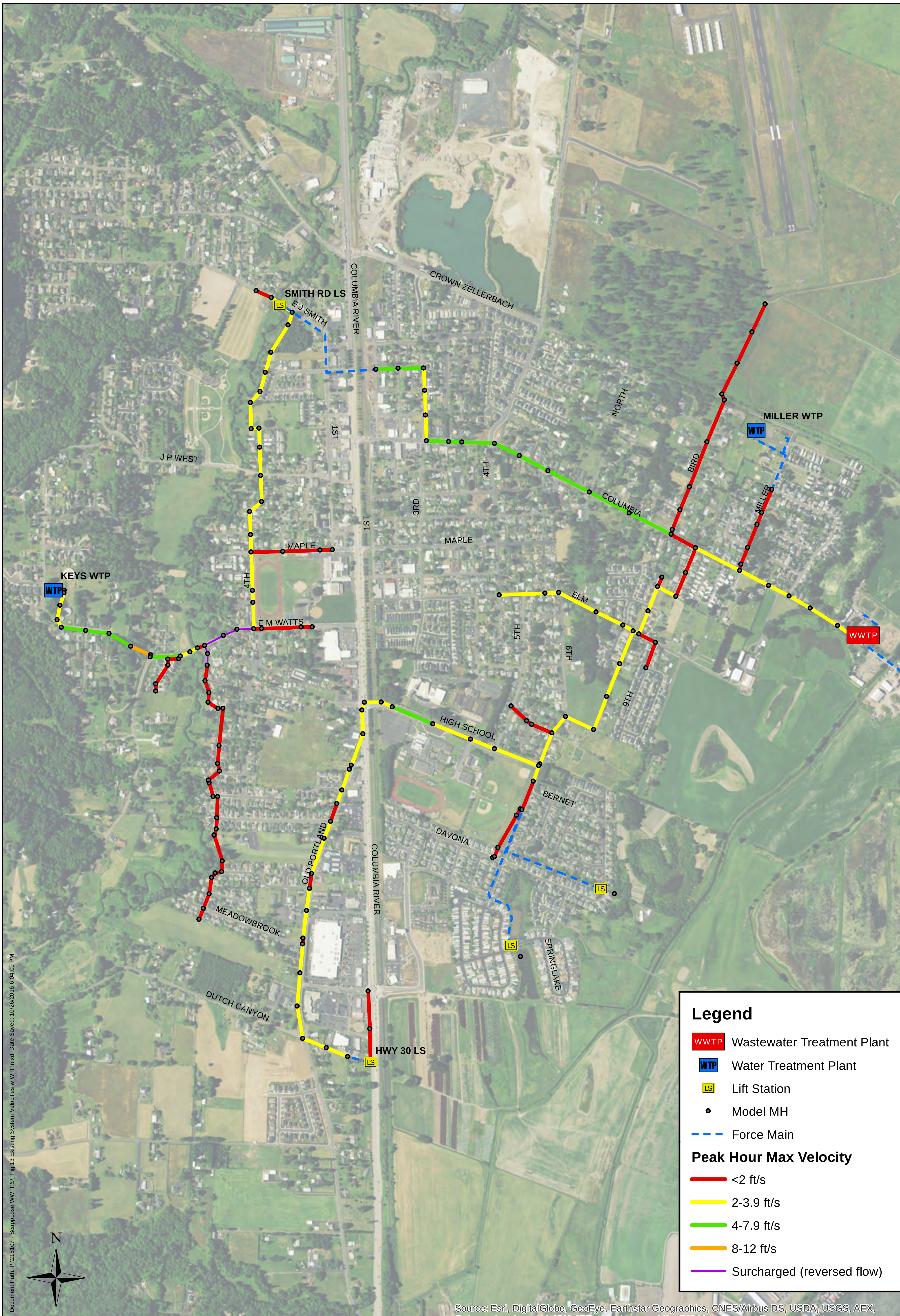
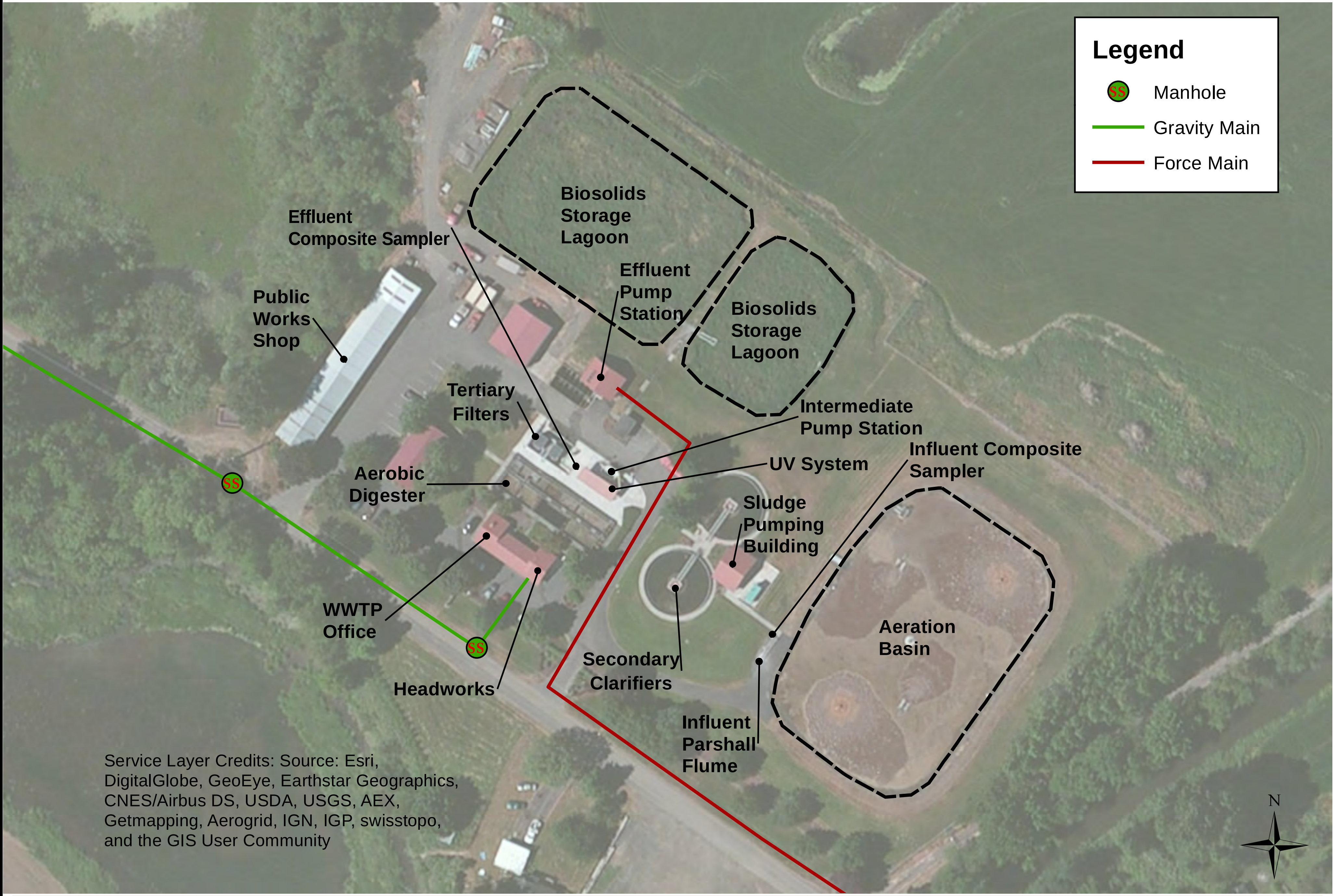


Figure:	Title:	Project:	Prepared for:		
13	EXISTING SYSTEM MAXIMUM VELOCITY (PEAK HOUR)	WWFPS	CITY OF SCAPPOOSE, OR		KELLER associates



Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

Figure:	Title:	Project:	Prepared for:		
14	EXISTING WWTP LAYOUT	WWFPS	CITY OF SCAPPOOSE, OR	SCAPPOOSE	KELLER associates

Document Path: J:\215107 Scappoose WWFPS\GIS\Existing WWTP Layout.mxd

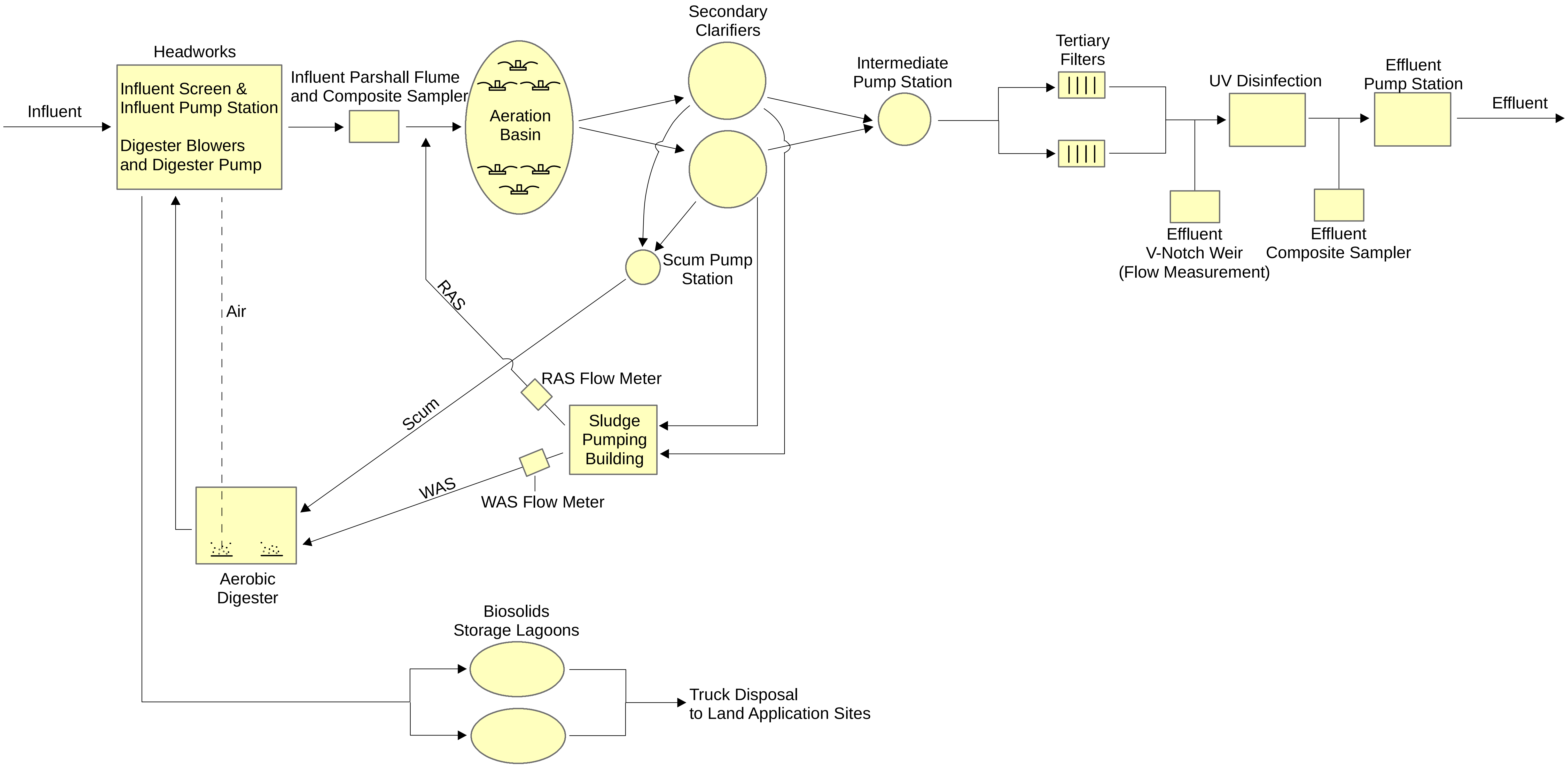


Figure:	Title:	Project:	Prepared for:
15	PROCESS FLOW DIAGRAM	WWFPS	CITY OF SCAPPOOSE, OR

KELLER associates

Document Path: J:\215107 Scappoose WWFPS\GIS\Process Flow Diagram.mxd

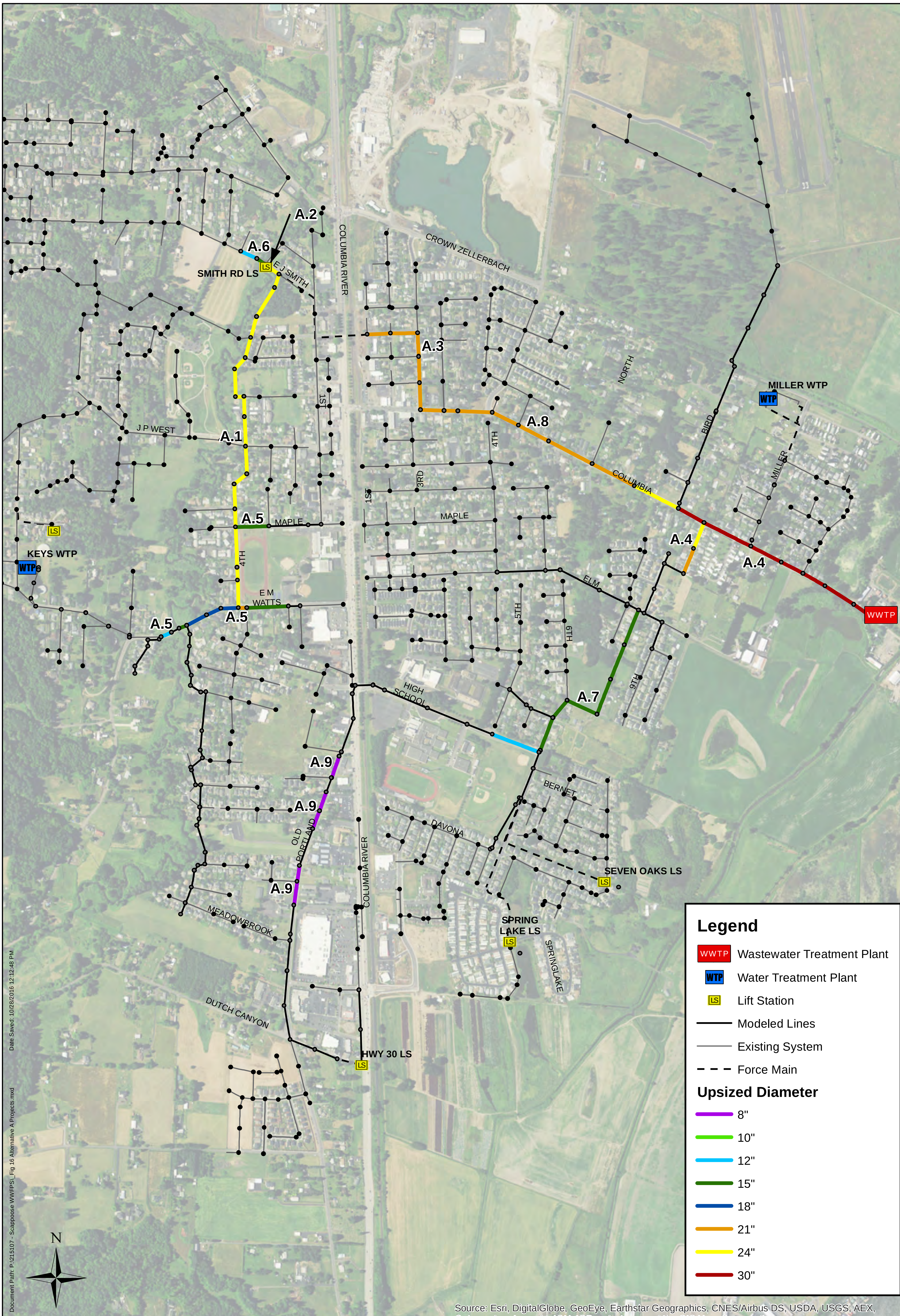


Figure:	Title:	Project:	Prepared for:
16	ALTERNATIVE A PROJECTS	WWFPS	CITY OF SCAPPOOSE, OR

KELLER associates

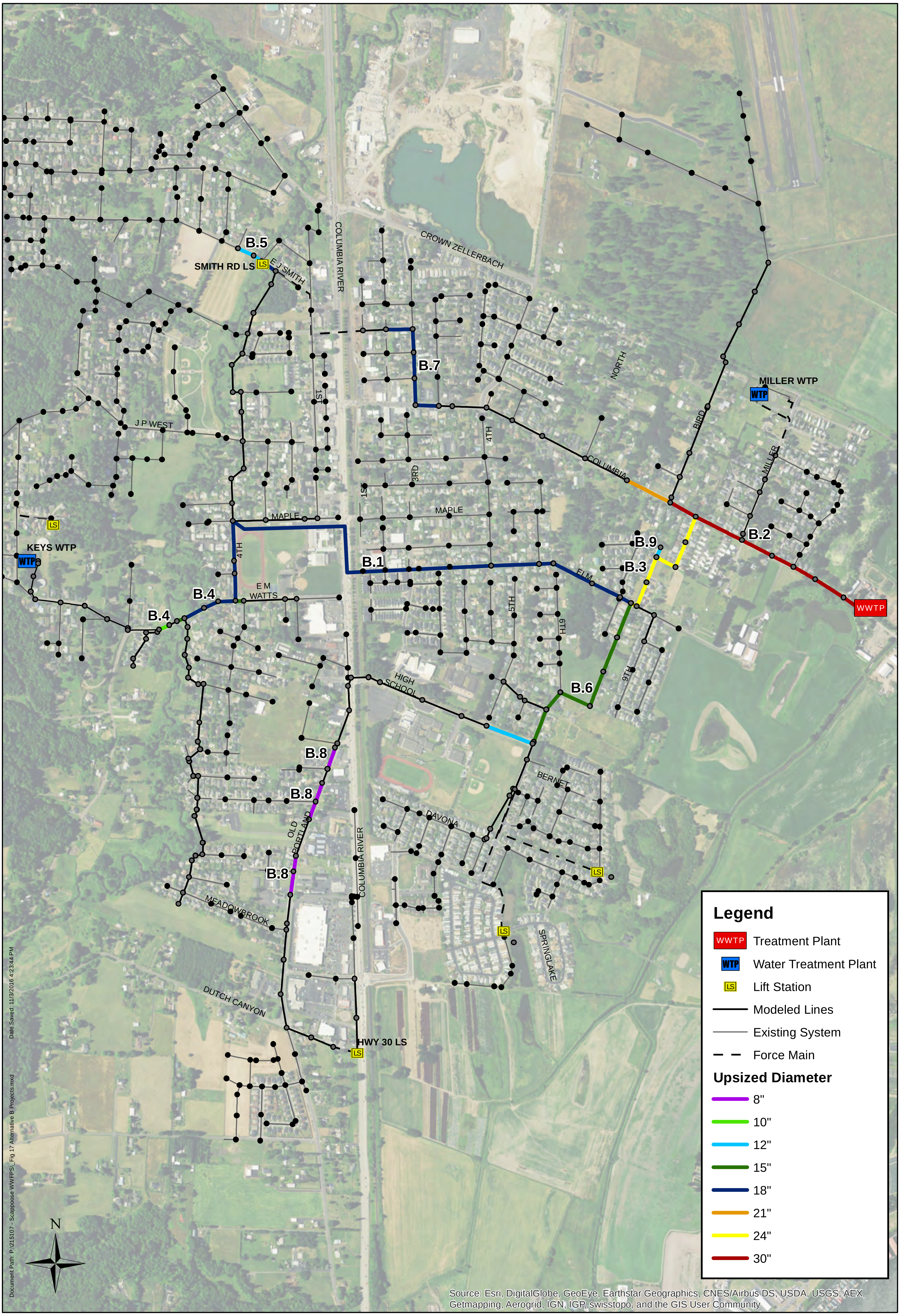
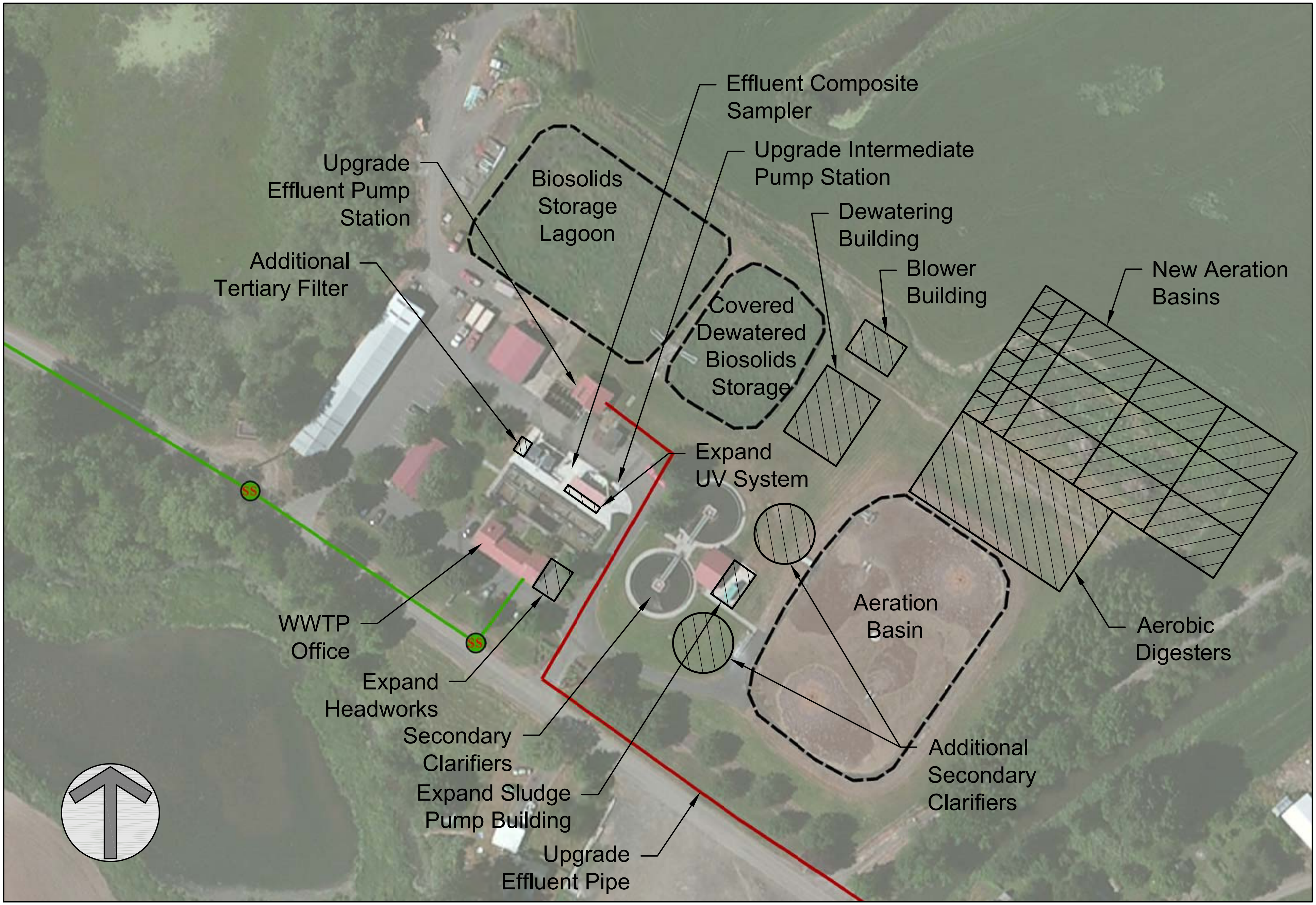
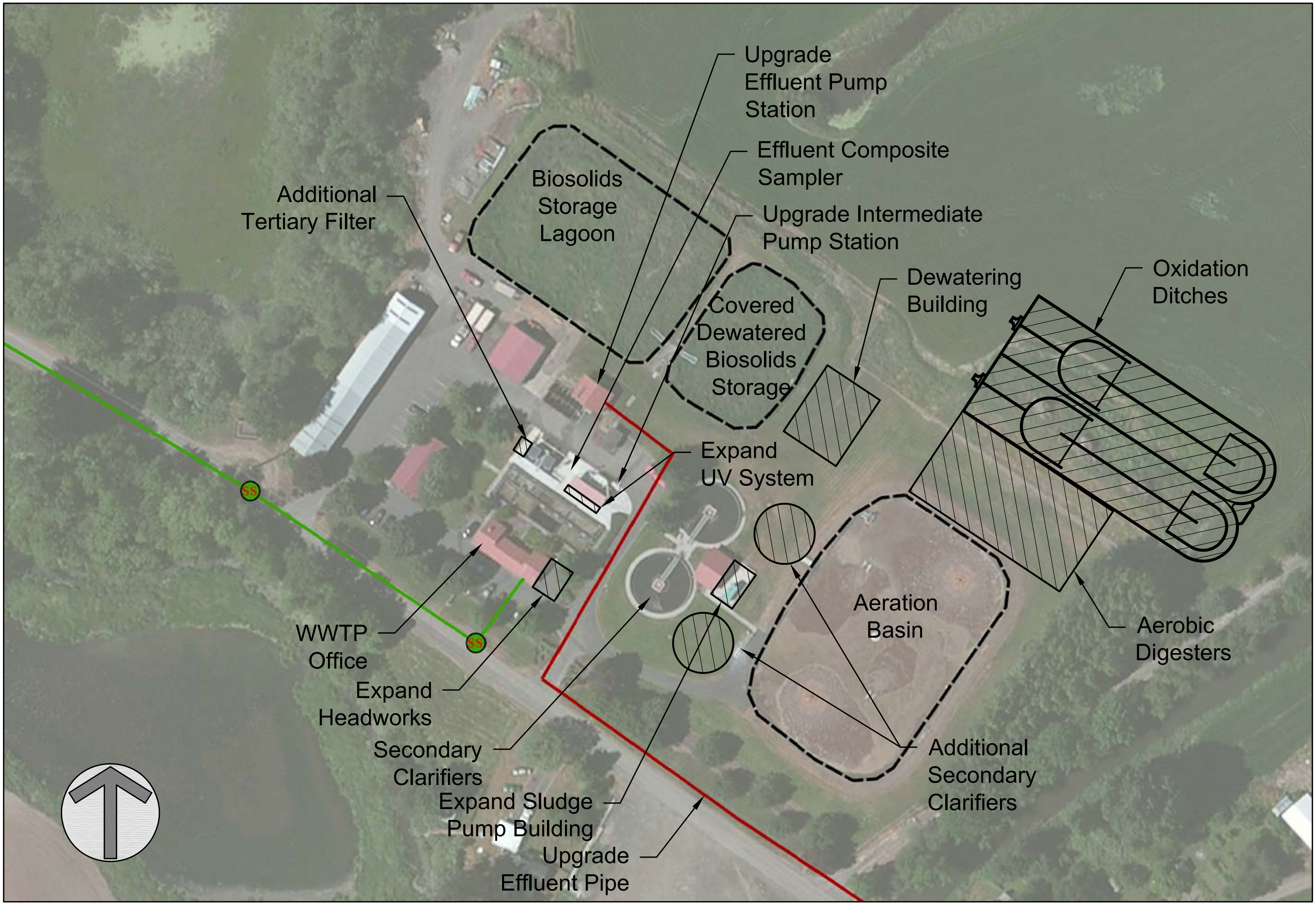


Figure:	Title:	Project:	Prepared for:
17	ALTERNATIVE B PROJECTS	WWFPS	CITY OF SCAPPOOSE, OR



KELLER associates	WWFPS SCAPPOOSE, OR	NEW AERATION BASINS	PROJECT NO: 215107 FIGURE NO: 18



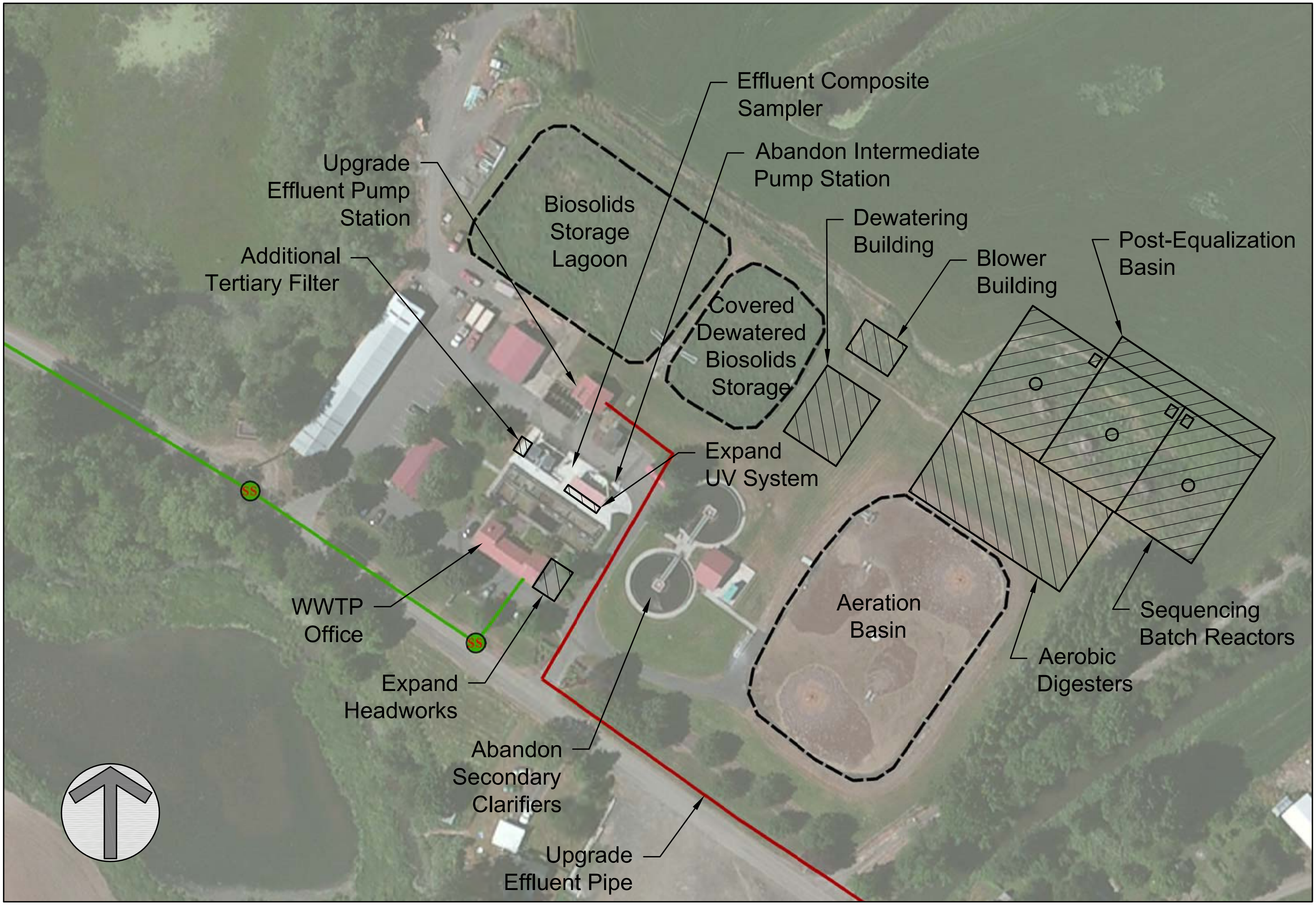
KELLER associates

WWFPS
SCAPPOOSE, OR

OXIDATION DITCHES

PROJECT NO: 215107

FIGURE NO: 19



KELLER associates

WWFPS
SCAPPOOSE, OR

SEQUENCING BATCH REACTOR (SBR)

PROJECT NO: 215107

FIGURE NO: 20

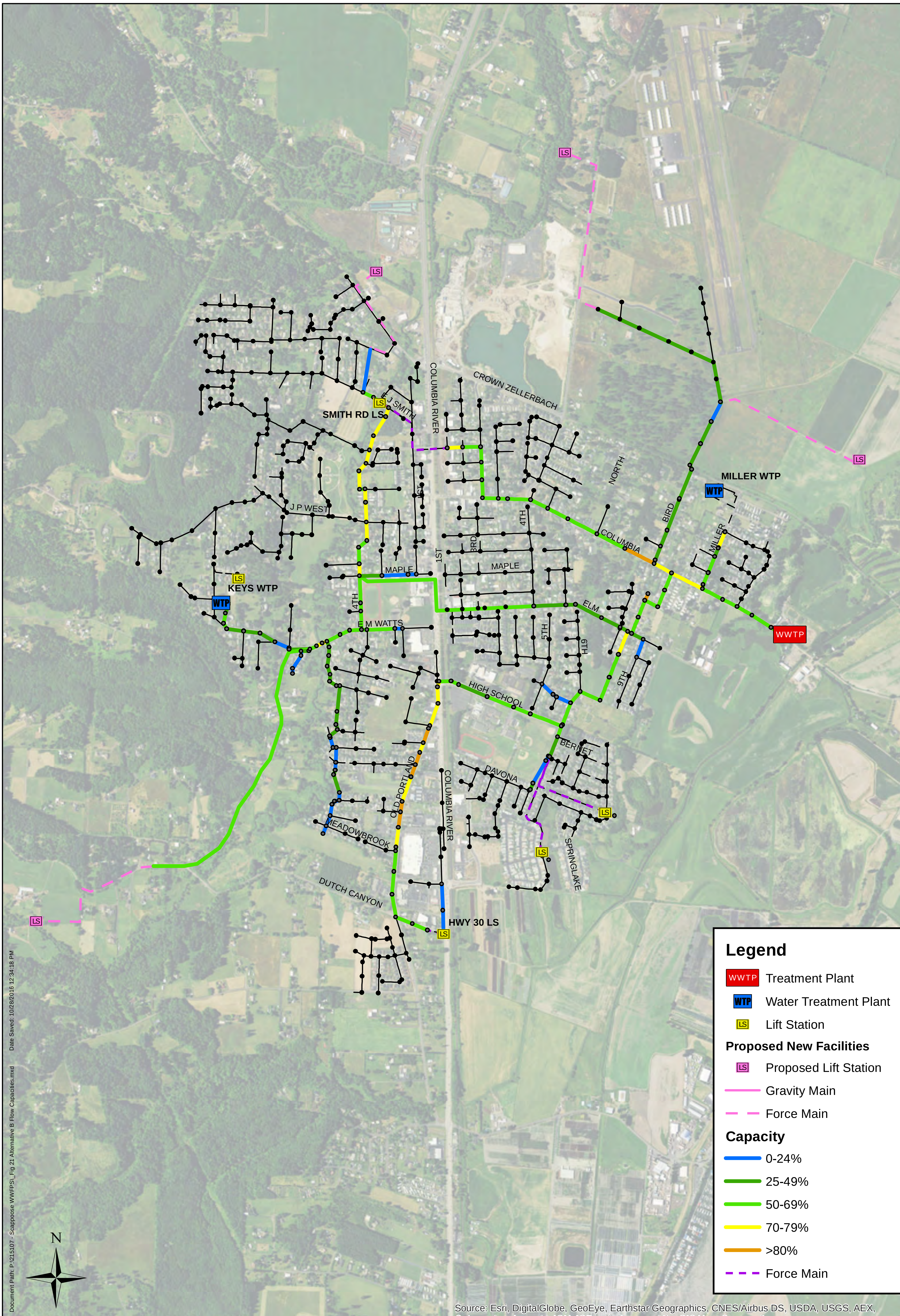


Figure:	Title:	Project:	Prepared for:
21	RECOMMENDED ALTERNATIVE FUTURE PIPE CAPACITIES	WWFPS	CITY OF SCAPPOOSE, OR

KELLER associates

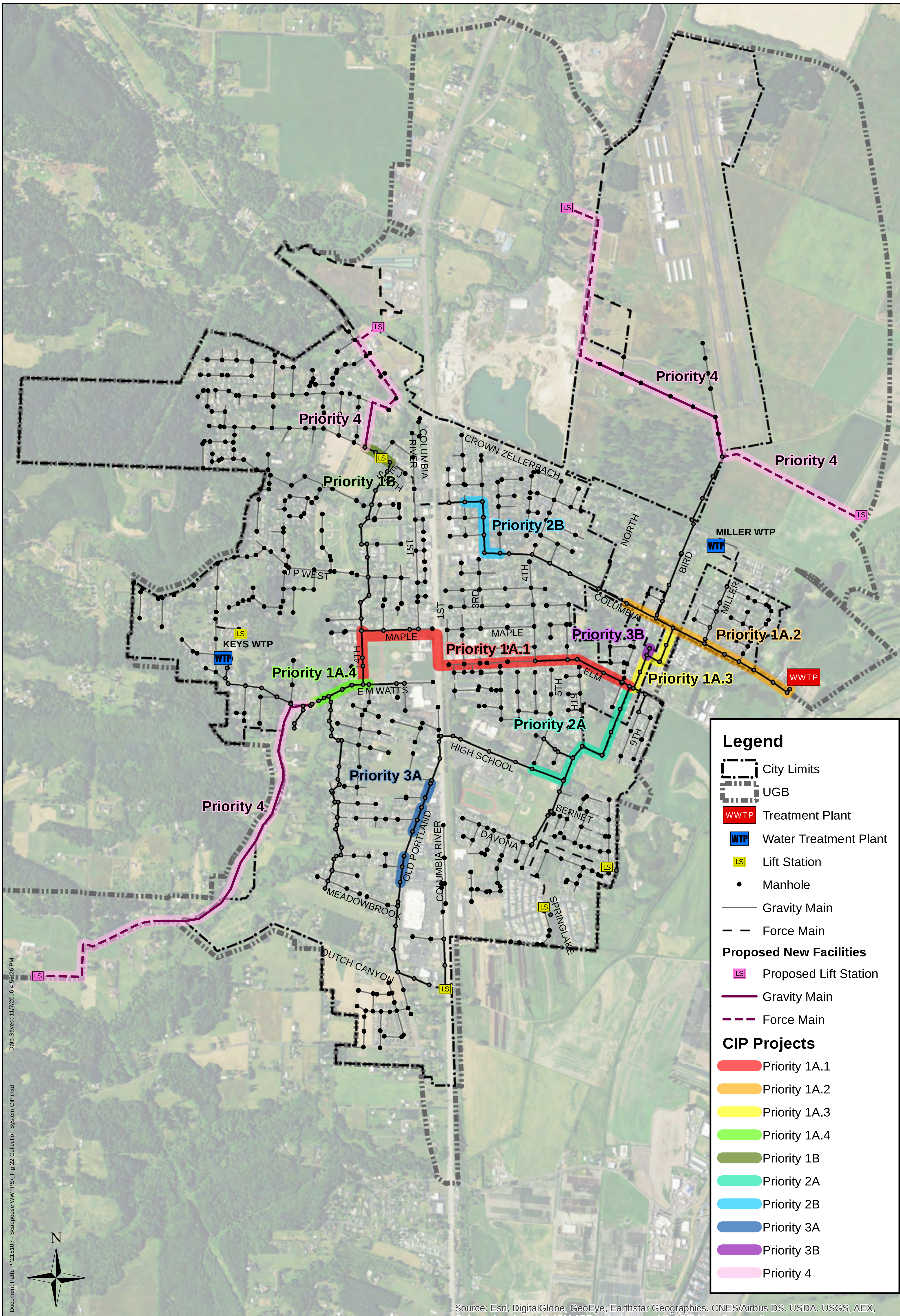
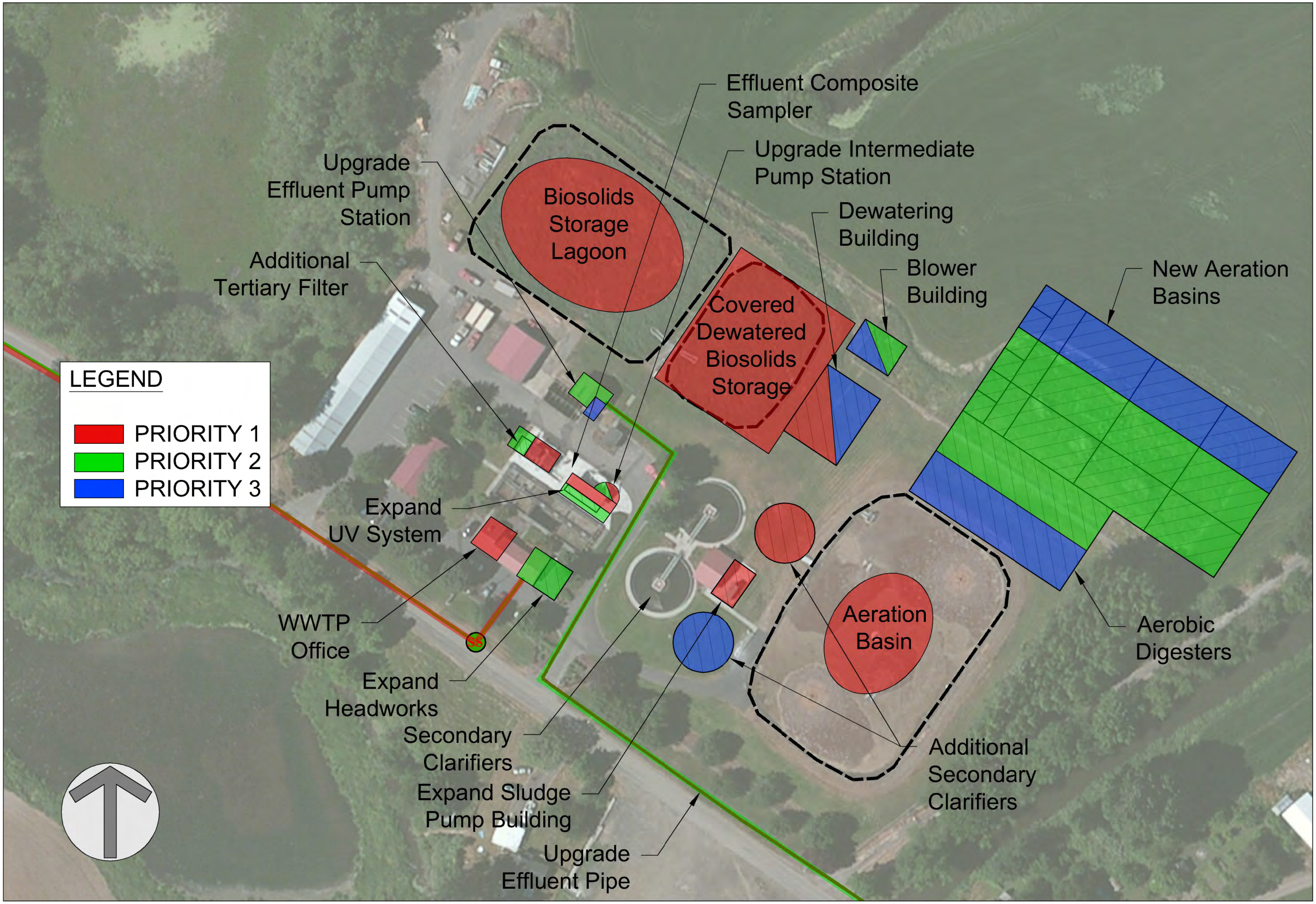


Figure:	Title:	Project:	Prepared for:
22	COLLECTION SYSTEM CAPITAL IMPROVEMENT PLAN	WWFPS	CITY OF SCAPPOOSE, OR



KELLER associates

WWFPS
SCAPPOOSE, OR

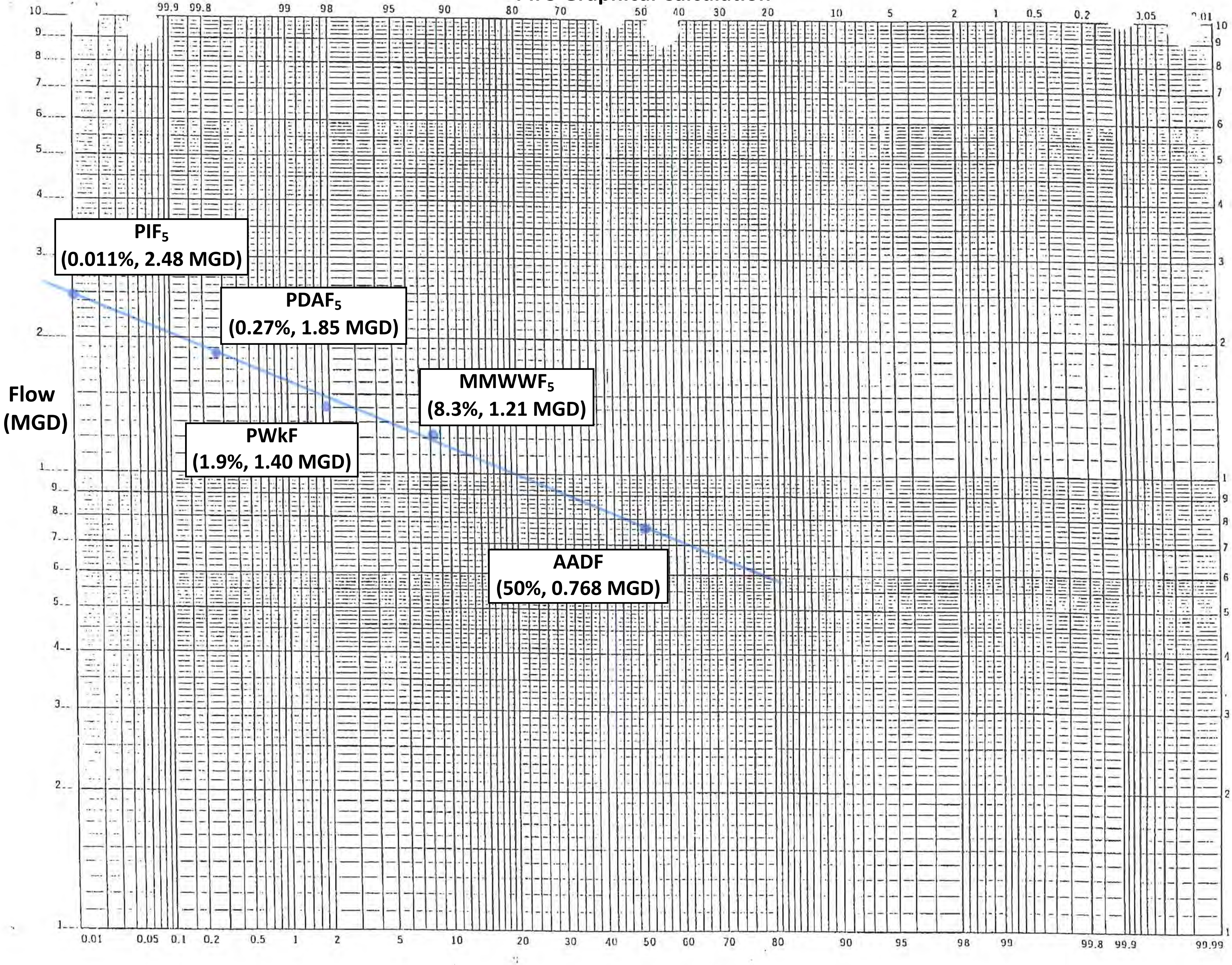
PRIORITY IMPROVEMENTS

PROJECT NO: 215107

FIGURE NO: 23

APPENDIX B – DEQ Flow Calculation

PIF5 Graphical Calculation




APPENDIX C – Clean Water Act Data

C-1:
NPDES 100677

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM
WASTE DISCHARGE PERMIT

Department of Environmental Quality
Northwest Region – Portland Office
2020 SW 4th Ave., Suite 400, Portland, OR 97201
Telephone: (503) 229-5263

Issued pursuant to ORS 468B.050 and The Federal Clean Water Act

ISSUED TO:

City of Scappoose
33568 East Columbia Avenue
Scappoose, OR 97056

SOURCES COVERED BY THIS PERMIT:

Type of Waste	Outfall Number	Outfall Location
Treated Wastewater	001	R.M. 10.6

FACILITY TYPE AND LOCATION:

Activated Sludge/Extended Aeration
Scappoose STP

34485 East Columbia Avenue
Scappoose OR 97056

RECEIVING STREAM INFORMATION:

Basin: Willamette
Sub-Basin: Lower Willamette

Receiving Stream: Multnomah Channel
LLID: 1227863458618-10.5796 D
County: Columbia

Treatment System Class: Level III
Collection System Class: Level II

EPA REFERENCE NO: OR0022420

This permit is issued in response to Application No. 971614 received June 9, 2009.

This permit is issued based on the land use findings in the permit record.

Gregory L. Geist, Manager, Water Quality Source Control Section — Date
Northwest Region

PERMITTED ACTIVITIES

Until this permit expires or is modified or revoked, the permittee is authorized to construct, install, modify, or operate a wastewater collection, treatment, control and disposal system and discharge to public waters adequately treated wastewaters only from the authorized discharge point or points established in Schedule A and only in conformance with all the requirements, limitations, and conditions set forth in the attached schedules as follows:

Unless specifically authorized by this permit, by another NPDES or WPCF permit, or by Oregon Administrative Rule, any other direct or indirect discharge of waste is prohibited, including discharge to waters of the state or an underground injection control system.

SCHEDULE-A

1. Waste Discharge Limitations not to be exceeded after permit issuance.

a. Treated Effluent Outfall 001

(1) May 1 - October 31:

Parameter	Average Effluent Concentrations Monthly	Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
BOD_5	10 mg/L	15 mg/L	125	190	255
TSS	10 mg/L	15 mg/L	125	190	255

(2) November 1 - April 30:

Parameter	Average Effluent Concentrations Monthly	Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
BOD_5	25 mg/L	37 mg/L	315	475	630
TSS	25 mg/L	37 mg/L	315	475	630

* Average dry weather design flow to the facility equals **1.515 MGD**. Mass load limits are based upon average dry weather design flow to the facility.

(3)

Other parameters (year-round)	Limitations
E. coli Bacteria	Shall not exceed 126 organisms per 100 mL monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL. (See Note 1)
pH	Shall be within the range of 6.0 - 9.0
BOD_5 and TSS Removal Efficiency	Shall not be less than 85% monthly average for BOD_5 and 85% monthly for TSS.

(4) Mixing Zone: Except as provided for in OAR 340-045-0080, no wastes shall be discharged and no activities shall be conducted which violate Water Quality Standards, as adopted in OAR 340-041; except in the following defined mixing zone:

> The allowable mixing zone is that portion of Multnomah Channel contained within a band extending out 100 feet from the shore side of the outfall, and 200 feet downstream and 200 feet upstream from the outfall. The Zone of Immediate Dilution (ZID) shall be defined as that portion of the allowable mixing zone that is within a 20 foot radius of the discharge point.

(5) Chlorine: Ultra-Violet Disinfection of effluent is required at this facility. Chlorine and chlorine compounds must not be used as a disinfecting agent of the treated effluent, and no

chlorine residual is allowed in the discharged effluent due to chlorine used for maintenance purposes.

b. Groundwater: No activities shall be conducted that could cause an adverse impact on existing or potential beneficial uses of groundwater. All wastewater and process related residuals shall be managed and disposed in a manner that will prevent a violation of the Groundwater Quality Protection Rules (OAR 340-040).

NOTES:

1. If a single sample exceeds 406 organisms per 100 mL, then five consecutive re-samples may be taken at four-hour intervals beginning within 28 hours after the original sample was taken. If the log mean of the five re-samples is less than or equal to 126 organisms per 100 mL, a violation shall not be triggered.

SCHEDULE-B

1. Minimum Monitoring and Reporting Requirements

The permittee shall monitor the parameters as specified below at the locations indicated. The laboratory used by the permittee to analyze samples shall have a quality assurance/quality control (QA/QC) program to verify the accuracy of sample analysis. If QA/QC requirements are not met for any analysis, the results shall be included in the report, but not used in calculations required by this permit. When possible, the permittee shall re-sample in a timely manner for parameters failing the QA/QC requirements, analyze the samples, and report the results.

a. Influent

The facility influent sampling locations are the following: Influent grab samples and measurements and composite samples are taken at the inlet to the influent Flow Measurement Structure (Parshall flume). All samples for toxics are taken in the same location.

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD)	Daily	Measurement
Flow Meter Calibration	Semi-Annual	Verification
BOD_5	2/Week	Composite
TSS	2/Week	Composite
pH	3/Week	Grab

b. Treated Effluent Outfall 001

The facility effluent sampling locations are the following: Effluent grab samples, measurements, and composite samples are taken at the outlet to the Ultra-Violet (UV) disinfection unit. All samples for toxics are taken at the same location.

Item or Parameter	Minimum Frequency	Type of Sample
BOD_5	2/Week	Composite
TSS	2/Week	Composite
pH	3/Week	Grab
E. coli	2/Week	Grab
Ammonia (Measured as N)	Weekly	Grab
Dissolved Oxygen	Weekly	Measurement
UV Radiation Intensity	Daily	Reading (See Note 1)
Pounds Discharged (BOD_5 and TSS)	2/Week	Calculation of Daily Mass Load
Pounds Discharged (BOD_5 and TSS)	Weekly	Calculation of Weekly Average Mass Load
Pounds Discharged (BOD_5 and TSS)	Monthly	Calculation of Monthly Average Mass Load
Average Percent Removed (BOD_5 and TSS)	Monthly	Calculation

c. Biosolids Management

Item or Parameter	Minimum Frequency	Type of Sample
Biosolids analysis including: Total Solids (% dry wt.) Volatile solids (% dry wt.) Biosolids nitrogen for: NH_3-N; NO_3-N; & TKN (% dry wt.) Phosphorus (% dry wt.) Potassium (% dry wt.) pH (standard units) Biosolids metals content for: As, Cd, Cu, Hg, Mo, Ni, Pb, Se & Zn, measured as total in mg/kg	Annually	Composite sample to be representative of the product to be land applied from the Storage lagoon or pond (See Note 2).
Record of locations where biosolids are applied on each DEQ approved site. (Site location maps to be maintained at treatment facility for review upon request by DEQ)	Each Occurrence	Date, volume, & locations where biosolids were applied recorded on site location map.
Record of % volatile solids reduction accomplished through stabilization	Annually	Calculation (See Note 3).
Fecal coliform per gram total solids (dry weight basis) or Salmonella sp. bacteria per four grams total solids (dry weight basis)	Monthly (when land applying biosolids)	At least seven (7) individual samples representative of the product to be land applied from the storage lagoon or pond (See Note 4).

d. Temperature Monitoring (Monitored only required May 1 - October 31)

Item or Parameter	Minimum Frequency	Type of Sample
Influent Temperature (ºC), Daily Maximum	3/Week	Measurement between 3 and 5 PM.
Effluent Temperature (ºC), Daily Maximum	3/Week	Measurement between 3 and 5 PM.
Multnomah Channel Temperature (ºC), Daily Max	3/Week	Measurement between 3 and 5 PM.
Effluent Temperature (ºC), Average of Daily Maximums	Weekly	Calculation

2. Monitoring Report Procedures

a. Monitoring results must be reported on approved forms. The reporting period is the calendar month. Reports must be submitted to the Department's Northwest Region - Portland office by the 15th day of the following month.

b. State monitoring reports must identify the name, certificate classification and grade level of each principal operator designated by the permittee as responsible for supervising the wastewater collection and treatment systems during the reporting period. Monitoring reports must also identify each system classification as found on page one of this permit.

c. Monitoring reports must include a record of the quantity and method of use of all sewage sludge removed from the treatment facility; and must record all applicable equipment breakdowns and bypassing.

3. Biosolids Report Submittals

Biosolids Report: For any year in which biosolids are land applied, a report shall be submitted to the Department by February 19 of the following year that describes biosolids handling activities for the previous year and includes, but is not limited to, the required information outlined in OAR 340-050-0035(6)(a)-(e).

NOTES:

1. UV Intensity Measurement. The intensity of UV radiation passing through the water column affects the system's ability to kill organisms. To track the reduction in intensity, the UV disinfection system must include a UV intensity meter with a sensor located in the water column at a specified distance from the UV bulbs. This meter will measure the intensity of UV radiation in mWatts-seconds/cm^2. The daily UV radiation intensity shall be determined by reading the meter each day. If more than one meter is used, the daily recording will be an average of all meter readings each day. Intensity meter(s) must be calibrated at a frequency recommended by the manufacturer. The manufacturer's UV intensity curves shall be used to determine when UV bulbs must be replaced or cleaned. Record all daily UV intensity readings in the treatment facility's log book. Record any change of UV bulbs. **Daily UV intensity readings are required for at least 5 days per week**.

2. Biosolids Sampling. Biosolids composite samples from the storage lagoon or pond shall be taken from reference areas in the storage lagoon or pond pursuant to Test Methods for Evaluating Solid Waste, Volume 2; Field Manual, Physical/Chemical Methods, November 1986, Third Edition, Chapter 9.

   Inorganic pollutant monitoring must be conducted according to Test Methods for Evaluating Solid Waste, Physical/Chemical Methods, Second Edition (1982) with Updates I and II and Third Edition (1986) with Revision I.

3. Volatile Solids Reduction. Calculation of the % volatile solids reduction is to be based on comparison of a representative grab sample of total and volatile solids entering each digester (a weighted blend of the primary and secondary clarifier solids) and a representative composite sample of solids exiting each digester withdrawal line.

4. Fecal Coliform Sampling. Analyze and report a fecal coliform result for each sample separately. Calculate and report the geometric mean of all the samples.

5. Temperature Monitoring. After two years of temperature monitoring, and if approved in writing by the Department; monitoring may be waived for those months when the 7-day average of effluent temperature does

not exceed the stream temperature standard of 18.0 °C (64.4 °F). Temperature monitoring results must be reported on the monthly DMR.

SCHEDULE-C

<u>Permit Compliance Conditions</u>

There are no compliance conditions.

SCHEDULE-D

Special Conditions

1. Sewer Cleaning Report: Permittee is required to inspect via television camera and clean 20% of its sanitary sewer system each year. A Sewer Cleaning Report shall be submitted by February 19 of the following year. The Report shall identify the sewer lines cleaned, structural defects noted, and any repairs made to correct identified structural defects.

2. Biosolids Management: All biosolids shall be managed in accordance with the current, DEQ approved Biosolids Management Plan, and the site authorization letters issued by the DEQ. Any changes in biosolids management activities that significantly differ from operations specified under the approved plan require the prior written approval of the DEQ.

   All new biosolids application sites shall meet the site selection criteria set forth in OAR 340-050-0070 and must be located within DEQ approved application sites in Columbia County. All currently approved sites are located adjacent to the wastewater treatment facility on 35 acres, where 28.1 acres are available for actual disposal. No new Public Notice is required for continued use of the currently approved sites; however, a copy of the latest approved Biosolids Management Plan is included for review with renewal permit documentation during the Public Comment Period. Property owners adjacent to any newly approved application sites shall be notified, in writing or by any method approved by DEQ, of the proposed activity prior to the start of application. Proposed new application sites that are deemed by the DEQ to be sensitive with respect to residential housing, runoff potential, or threat to groundwater are subject to public comment in accordance with OAR 340-050-0030.

3. Changes in Biosolids Standards: This permit may be modified to incorporate any applicable standard for biosolids use or disposal promulgated under section 405(d) of the Clean Water Act, if the standard for biosolids use or disposal is more stringent than any requirements for biosolids use or disposal in the permit, or controls a pollutant or practice not limited in this permit.

4. Operator Certification: The permittee shall comply with Oregon Administrative Rules (OAR), Chapter 340, Division 049, "Regulations Pertaining To Certification of Wastewater System Operator Personnel" and accordingly:

   a. The permittee shall have its wastewater system supervised by one or more operators who are certified in a classification and grade level (equal to or greater) that corresponds with the classification (collection and/or treatment) of the system to be supervised as specified on page one of this permit.

      Note: A "supervisor" is defined as the person exercising authority for establishing and executing the specific practice and procedures of operating the system in accordance with the policies of the permittee and requirements of the waste discharge permit. "Supervise" means responsible for the technical operation of a system, which may affect its performance or the quality of the effluent produced. Supervisors are not required to be on-site at all times.

   b. The permittee’s wastewater system may not be without supervision (as required by Special Condition-4.a. above) for more than thirty (30) days. During this period, and at any time that the supervisor is not available to respond on-site (i.e. vacation, sick leave or off-call), the permittee must make available another person who is certified at no less than one grade lower then the system classification.

   c. If the wastewater system has more than one daily shift, the permittee shall have the shift supervisor, if any, certified at no less than one grade lower than the system classification.

d. The permittee is responsible for ensuring the wastewater system has a properly certified supervisor available at all times to respond on-site at the request of the permittee and to any other operator.

e. The permittee shall notify the Department of Environmental Quality in writing within thirty (30) days of replacement or redesignation of certified operators responsible for supervising wastewater system operation. The notice shall be filed with the Water Quality Division, Operator Certification Program, 811 SW 6th Ave, Portland, OR 97204. This requirement is in addition to the reporting requirements contained under Schedule B of this permit.

f. Upon written request, the Department may grant the permittee reasonable time, not to exceed 120 days, to obtain the services of a qualified person to supervise the wastewater system. The written request must include: (1) Justification for the time needed, (2) A schedule for recruiting and hiring, (3) The date the system supervisor's availability ceased, and (4) The name of the alternate system supervisor(s) as required by 4.b. above.

5. Groundwater: The permittee shall not be required to perform a hydro-geologic characterization or groundwater monitoring during the term of this permit provided:

a. The facilities are operated in accordance with the permit conditions; and

b. There are no adverse groundwater quality impacts (complaints or other indirect evidence) resulting from the facility's operation.

If warranted at permit renewal, the Department may evaluate the need for a full assessment of the facilities impact on groundwater quality.

6. Notification: The permittee shall notify the DEQ Northwest Region - Portland Office (phone: (503) 229-5263) in accordance with the response times noted in the General Conditions of this permit, of any malfunction so that corrective action can be coordinated between the permittee and the Department.

SCHEDULE-E

NOT APPLICABLE

(Pretreatment is not required)

SCHEDULE-F

NPDES GENERAL CONDITIONS – DOMESTIC FACILITIES

(Schedule-F, last update 9.18.2009)

SECTION-A, STANDARD CONDITIONS

1. Duty to Comply with Permit
   The permittee must comply with all conditions of this permit. Failure to comply with any permit condition is a violation of Oregon Revised Statutes (ORS) 468B.025 and the federal Clean Water Act and is grounds for an enforcement action. Failure to comply is also grounds for the Department to terminate, modify and reissue, revoke, or deny renewal of a permit.

2. Penalties for Water Pollution and Permit Condition Violations
   The permit is enforceable by DEQ or EPA, and in some circumstances also by third-parties under the citizen suit provisions 33 USC §1365. DEQ enforcement is generally based on provisions of state statutes and EQC rules, and EPA enforcement is generally based on provisions of federal statutes and EPA regulations.

   ORS 468.140 allows the Department to impose civil penalties up to $10,000 per day for violation of a term, condition or requirement of a permit. The federal Clean Water Act provides for civil penalties not to exceed $32,500 and administrative penalties not to exceed $11,000 per day for each violation of any condition or limitation of this permit.

   Under ORS 468.943, unlawful water pollution, if committed by a person with criminal negligence, is punishable by a fine of up to $25,000, imprisonment for not more than one year, or both. Each day on which a violation occurs or continues is a separately punishable offense. The federal Clean Water Act provides for criminal penalties of not more than $50,000 per day of violation, or imprisonment of not more than 2 years, or both for second or subsequent negligent violations of this permit.

   Under ORS 468.946, a person who knowingly discharges, places, or causes to be placed any waste into the waters of the state or in a location where the waste is likely to escape into the waters of the state is subject to a Class B felony punishable by a fine not to exceed $200,000 and up to 10 years in prison. The federal Clean Water Act provides for criminal penalties of $5,000 to $50,000 per day of violation, or imprisonment of not more than 3 years, or both for knowing violations of the permit. In the case of a second or subsequent conviction for knowing violation, a person shall be subject to criminal penalties of not more than $100,000 per day of violation, or imprisonment of not more than 6 years, or both.

3. Duty to Mitigate
   The permittee must take all reasonable steps to minimize or prevent any discharge or sludge use or disposal in violation of this permit that has a reasonable likelihood of adversely affecting human health or the environment. In addition, upon request of the Department, the permittee must correct any adverse impact on the environment or human health resulting from noncompliance with this permit, including such accelerated or additional monitoring as necessary to determine the nature and impact of the noncomplying discharge.

4. Duty to Reapply

If the permittee wishes to continue an activity regulated by this permit after the expiration date of this permit, the permittee must apply for and have the permit renewed. The application must be submitted at least 180 days before the expiration date of this permit.

The Department may grant permission to submit an application less than 180 days in advance but no later than the permit expiration date.

5. Permit Actions

This permit may be modified, revoked and reissued, or terminated for cause including, but not limited to, the following:

a. Violation of any term, condition, or requirement of this permit, a rule, or a statute;
b. Obtaining this permit by misrepresentation or failure to disclose fully all material facts;
c. A change in any condition that requires either a temporary or permanent reduction or elimination of the authorized discharge;
d. The permittee is identified as a Designated Management Agency or allocated a wasteload under a Total Maximum Daily Load (TMDL);
e. New information or regulations;
f. Modification of compliance schedules;
g. Requirements of permit reopener conditions;
h. Correction of technical mistakes made in determining permit conditions;
i. Determination that the permitted activity endangers human health or the environment;
j. Other causes as specified in 40 CFR 122.62, 122.64, and 124.5;
k. For communities with combined sewer overflows (CSOs);
   (1) To comply with any state or federal law regulation that addresses CSOs that is adopted or promulgated subsequent to the effective date of this permit;
   (2) If new information, not available at the time of permit issuance, indicates that CSO controls imposed under this permit have failed to ensure attainment of water quality standards, including protection of designated uses;
   (3) Resulting from implementation of the Permittee’s Long-Term Control Plan and/or permit conditions related to CSOs.

The filing of a request by the permittee for a permit modification, revocation or reissuance, termination, or a notification of planned changes or anticipated noncompliance, does not stay any permit condition.

6. Toxic Pollutants

The permittee must comply with any applicable effluent standards or prohibitions established under Oregon Administrative Rules (OAR) 340-041-0033 and 307(a) of the federal Clean Water Act for toxic pollutants, and with standards for sewage sludge use or disposal established under Section 405(d) of the Clean Water Act, within the time provided in the regulations that establish those standards or prohibitions, even if the permit has not yet been modified to incorporate the requirement.

7. Property Rights and Other Legal Requirements

The issuance of this permit does not convey any property rights of any sort, or any exclusive privilege, or authorize any injury to persons or property or invasion of any other private rights, or any infringement of federal, tribal, state, or local laws or regulations.

8. Permit References

Except for effluent standards or prohibitions established under Section 307(a) of the federal Clean Water Act and OAR 340-041-0033 for toxic pollutants, and standards for sewage sludge use or disposal established under Section 405(d) of the Clean Water Act, all rules and statutes referred to in this permit are those in effect on the date this permit is issued.

9. Permit Fees

The permittee must pay the fees required by Oregon Administrative Rules.

SECTION-B, OPERATION AND MAINTENANCE OF POLLUTION CONTROLS

1. Proper Operation and Maintenance

The permittee must at all times properly operate and maintain all facilities and systems of treatment and control (and related appurtenances) that are installed or used by the permittee to achieve compliance with the conditions of this permit. Proper operation and maintenance also includes adequate laboratory controls and appropriate quality assurance procedures. This provision requires the operation of back-up or auxiliary facilities or similar systems that are installed by a permittee only when the operation is necessary to achieve compliance with the conditions of the permit.

2. Need to Halt or Reduce Activity Not a Defense

For industrial or commercial facilities, upon reduction, loss, or failure of the treatment facility, the permittee must, to the extent necessary to maintain compliance with its permit, control production or all discharges or both until the facility is restored or an alternative method of treatment is provided. This requirement applies, for example, when the primary source of power of the treatment facility fails or is reduced or lost. It is not a defense for a permittee in an enforcement action that it would have been necessary to halt or reduce the permitted activity in order to maintain compliance with the conditions of this permit.

3. Bypass of Treatment Facilities

a. Definitions

(1) "Bypass" means intentional diversion of waste streams from any portion of the treatment facility. The permittee may allow any bypass to occur which does not cause effluent limitations to be exceeded, provided the diversion is to allow essential maintenance to assure efficient operation. These bypasses are not subject to the provisions of paragraphs b. and c. of this section.

(2) "Severe property damage" means substantial physical damage to property, damage to the treatment facilities which causes them to become inoperable, or substantial and permanent loss of natural resources that can reasonably be expected to occur in the absence of a bypass. Severe property damage does not mean economic loss caused by delays in production.

b. Prohibition of bypass.

(1) Bypass is prohibited and the Department may take enforcement action against a permittee for bypass unless:

i. Bypass was unavoidable to prevent loss of life, personal injury, or severe property damage;
ii. There were no feasible alternatives to the bypass, such as the use of auxiliary treatment facilities, retention of untreated wastes, or maintenance during normal periods of equipment downtime. This condition is not satisfied if adequate backup equipment should have been installed in the exercise of reasonable engineering judgment to prevent a bypass that occurred during normal periods of equipment downtime or preventative maintenance; and
iii. The permittee submitted notices and requests as required under General Condition B.3.c.

(2) The Department may approve an anticipated bypass, after considering its adverse effects and any alternatives to bypassing, when the Department determines that it will meet the three conditions listed above in General Condition B.3.b.(1).

c. Notice and request for bypass.
(1) Anticipated bypass. If the permittee knows in advance of the need for a bypass, a written notice must be submitted to the Department at least ten days before the date of the bypass.
(2) Unanticipated bypass. The permittee must submit notice of an unanticipated bypass as required in General Condition D.5.

4. Upset

a. Definition. "Upset" means an exceptional incident in which there is unintentional and temporary noncompliance with technology based permit effluent limitations because of factors beyond the reasonable control of the permittee. An upset does not include noncompliance to the extent caused by operation error, improperly designed treatment facilities, inadequate treatment facilities, lack of preventative maintenance, or careless or improper operation.

b. Effect of an upset. An upset constitutes an affirmative defense to an action brought for noncompliance with such technology-based permit effluent limitations if the requirements of General Condition B.4.c are met. No determination made during administrative review of claims that noncompliance was caused by upset, and before an action for noncompliance, is final administrative action subject to judicial review.

c. Conditions necessary for a demonstration of upset. A permittee who wishes to establish the affirmative defense of upset must demonstrate, through properly signed, contemporaneous operating logs, or other relevant evidence that:
(1) An upset occurred and that the permittee can identify the causes(s) of the upset;
(2) The permitted facility was at the time being properly operated;
(3) The permittee submitted notice of the upset as required in General Condition D.5, hereof (24-hour notice); and
(4) The permittee complied with any remedial measures required under General Condition A.3 hereof.

d. Burden of proof. In any enforcement proceeding the permittee seeking to establish the occurrence of an upset has the burden of proof.

5. Treatment of Single Operational Upset

For purposes of this permit, A Single Operational Upset that leads to simultaneous violations of more than one pollutant parameter will be treated as a single violation. A single operational upset is an exceptional incident that causes simultaneous, unintentional, unknowing (not the result of a knowing act or omission), temporary noncompliance with more than one Clean Water Act effluent discharge pollutant parameter. A single operational upset does not include Clean Water Act violations involving discharge without a NPDES permit or noncompliance to the extent caused by improperly designed or inadequate treatment facilities. Each day of a single operational upset is a violation.

6. Overflows from Wastewater Conveyance Systems and Associated Pump Stations

a. Definitions

(1) "Overflow" means any spill, release or diversion of sewage including:

i. An overflow that results in a discharge to waters of the United States; and

ii. An overflow of wastewater, including a wastewater backup into a building (other than a backup caused solely by a blockage or other malfunction in a privately owned sewer or building lateral), even if that overflow does not reach waters of the United States.

b. Prohibition of overflows. Overflows are prohibited. The Department may exercise enforcement discretion regarding overflow events. In exercising its enforcement discretion, the Department may consider various factors, including the adequacy of the conveyance system's capacity and the magnitude, duration and return frequency of storm events.

c. Reporting required. All overflows must be reported orally to the Department within 24 hours from the time the permittee becomes aware of the overflow. Reporting procedures are described in more detail in General Condition D.5.

7. Public Notification of Effluent Violation or Overflow

If effluent limitations specified in this permit are exceeded or an overflow occurs that threatens public health, the permittee must take such steps as are necessary to alert the public, health agencies and other affected entities (e.g., public water systems) about the extent and nature of the discharge in accordance with the notification procedures developed under General Condition B.8. Such steps may include, but are not limited to, posting of the river at access points and other places, news releases, and paid announcements on radio and television.

8. Emergency Response and Public Notification Plan

The permittee must develop and implement an emergency response and public notification plan that identifies measures to protect public health from overflows, bypasses or upsets that may endanger public health. At a minimum the plan must include mechanisms to:

a. Ensure that the permittee is aware (to the greatest extent possible) of such events;

b. Ensure notification of appropriate personnel and ensure that they are immediately dispatched for investigation and response;

c. Ensure immediate notification to the public, health agencies, and other affected public entities (including public water systems). The overflow response plan must identify the public health and other officials who will receive immediate notification;

d. Ensure that appropriate personnel are aware of and follow the plan and are appropriately trained;

e. Provide emergency operations; and

f. Ensure that DEQ is notified of the public notification steps taken.

9. Removed Substances

Solids, sludges, filter backwash, or other pollutants removed in the course of treatment or control of wastewaters must be disposed of in such a manner as to prevent any pollutant from such materials from entering waters of the state, causing nuisance conditions, or creating a public health hazard.

SECTION-C, MONITORING AND RECORDS

1. Representative Sampling

Sampling and measurements taken as required herein shall be representative of the volume and nature of the monitored discharge. All samples must be taken at the monitoring points specified in this permit, and shall be taken, unless otherwise specified, before the effluent joins or is diluted by any other waste stream, body of water, or substance. Monitoring points may not be changed without notification to and the approval of the Department.

2. Flow Measurements

Appropriate flow measurement devices and methods consistent with accepted scientific practices must be selected and used to ensure the accuracy and reliability of measurements of the volume of monitored discharges. The devices must be installed, calibrated and maintained to insure that the accuracy of the measurements is consistent with the accepted capability of that type of device. Devices selected must be capable of measuring flows with a maximum deviation of less than ± 10 percent from true discharge rates throughout the range of expected discharge volumes.

3. Monitoring Procedures

Monitoring must be conducted according to test procedures approved under 40 CFR part 136, or in the case of sludge use and disposal, under 40 CFR part 503, unless other test procedures have been specified in this permit.

4. Penalties of Tampering

The Clean Water Act provides that any person who falsifies, tampers with, or knowingly renders inaccurate any monitoring device or method required to be maintained under this permit may, upon conviction, be punished by a fine of not more than $10,000 per violation, imprisonment for not more than two years, or both. If a conviction of a person is for a violation committed after a first conviction of such person, punishment is a fine not more than $20,000 per day of violation, or by imprisonment of not more than four years, or both.

5. Reporting of Monitoring Results

Monitoring results must be summarized each month on a Discharge Monitoring Report form approved by the Department. The reports must be submitted monthly and are to be mailed, delivered or otherwise transmitted by the 15th day of the following month unless specifically approved otherwise in Schedule B of this permit.

6. Additional Monitoring by the Permittee

If the permittee monitors any pollutant more frequently than required by this permit, using test procedures approved under 40 CFR part 136, or in the case of sludge use and disposal, under 40 CFR part 503, or as specified in this permit, the results of this monitoring must be included in the calculation and reporting of the data submitted in the Discharge Monitoring Report. Such increased frequency must also be indicated. For a pollutant parameter that may be sampled more than once per day (e.g., Total Chlorine Residual), only the average daily value must be recorded unless otherwise specified in this permit.

7. Averaging of Measurements

Calculations for all limitations that require averaging of measurements must utilize an arithmetic mean, except for bacteria which shall be averaged as specified in this permit.

8. Retention of Records

Records of monitoring information required by this permit related to the permittee's sewage sludge use and disposal activities shall be retained for a period of at least five years (or longer as required by 40 CFR part 503). Records of all monitoring information including all calibration and maintenance records, all original strip chart recordings for continuous monitoring instrumentation, copies of all reports required by this permit and records of all data used to complete the application for this permit shall be retained for a period of at least 3 years from the date of the sample, measurement, report, or application. This period may be extended by request of the Department at any time.

9. Records Contents

Records of monitoring information must include:

a. The date, exact place, time, and methods of sampling or measurements;
b. The individual(s) who performed the sampling or measurements;
c. The date(s) analyses were performed;
d. The individual(s) who performed the analyses;
e. The analytical techniques or methods used; and
f. The results of such analyses.

10. Inspection and Entry

The permittee must allow the Department or EPA upon the presentation of credentials to:

a. Enter upon the permittee's premises where a regulated facility or activity is located or conducted, or where records must be kept under the conditions of this permit;
b. Have access to and copy, at reasonable times, any records that must be kept under the conditions of this permit;
c. Inspect at reasonable times any facilities, equipment (including monitoring and control equipment), practices, or operations regulated or required under this permit, and
d. Sample or monitor at reasonable times, for the purpose of assuring permit compliance or as otherwise authorized by state law, any substances or parameters at any location.

11. Confidentiality of Information

Any information relating to this permit that is submitted to or obtained by DEQ is available to the public unless classified as confidential by the Director of DEQ under ORS 468.095. The Permittee may request that information be classified as confidential if it is a trade secret as defined by that statute. The name and address of the permittee, permit applications, permits, effluent data, and information required by NPDES application forms under 40 CFR 122.21 will not be classified as confidential (40 CFR 122.7(b)).

SECTION-D, REPORTING REQUIREMENTS

1. Planned Changes

The permittee must comply with OAR chapter 340, division 52, "Review of Plans and Specifications" and 40 CFR Section 122.41(l) (1). Except where exempted under OAR chapter 340, division 52, no construction, installation, or modification involving disposal systems, treatment works, sewerage systems, or common sewers may be commenced until the plans and specifications are submitted to and approved by the Department. The permittee must give notice to the Department as soon as possible of any planned physical alternations or additions to the permitted facility.

2. <u>Anticipated Noncompliance</u>
The permittee must give advance notice to the Department of any planned changes in the permitted facility or activity that may result in noncompliance with permit requirements.

3. <u>Transfers</u>
This permit may be transferred to a new permittee provided the transferee acquires a property interest in the permitted activity and agrees in writing to fully comply with all the terms and conditions of the permit and the rules of the Commission. No permit may be transferred to a third party without prior written approval from the Department. The Department may require modification, revocation, and reissuance of the permit to change the name of the permittee and incorporate such other requirements as may be necessary under 40 CFR Section 122.61. The permittee must notify the Department when a transfer of property interest takes place.

4. <u>Compliance Schedule</u>
Reports of compliance or noncompliance with, or any progress reports on interim and final requirements contained in any compliance schedule of this permit must be submitted no later than 14 days following each schedule date. Any reports of noncompliance must include the cause of noncompliance, any remedial actions taken, and the probability of meeting the next scheduled requirements.

5. <u>Twenty-Four Hour Reporting</u>
The permittee must report any noncompliance that may endanger health or the environment. Any information must be provided orally (by telephone) to DEQ or to the Oregon Emergency Response System (1-800-452-0311) as specified below within 24 hours from the time the permittee becomes aware of the circumstances.

a. Overflows.
   (1) Oral Reporting within 24 hours.
      i. For overflows other than basement backups, the following information must be reported to the Oregon Emergency Response System (OERS) at 1-800-452-0311. For basement backups, this information should be reported directly to DEQ.
         a) The location of the overflow;
         b) The receiving water (if there is one);
         c) An estimate of the volume of the overflow;
         d) A description of the sewer system component from which the release occurred (e.g., manhole, constructed overflow pipe, crack in pipe); and
         e) The estimated date and time when the overflow began and stopped or will be stopped.
      ii. The following information must be reported to the Department’s Regional office within 24 hours, or during normal business hours, whichever is first:
         a) The OERS incident number (if applicable) along with a brief description of the event.
   (2) Written reporting within 5 days.

i. The following information must be provided in writing to the Department's Regional office within 5 days of the time the permittee becomes aware of the overflow:
   a) The OERS incident number (if applicable);
   b) The cause or suspected cause of the overflow;
   c) Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the overflow and a schedule of major milestones for those steps;
   d) Steps taken or planned to mitigate the impact(s) of the overflow and a schedule of major milestones for those steps; and
   e) (for storm-related overflows) The rainfall intensity (inches/hour) and duration of the storm associated with the overflow.

The Department may waive the written report on a case-by-case basis if the oral report has been received within 24 hours.

b. Other instances of noncompliance.
   (1) The following instances of noncompliance must be reported:
      i. Any unanticipated bypass that exceeds any effluent limitation in this permit;
      ii. Any upset that exceeds any effluent limitation in this permit;
      iii. Violation of maximum daily discharge limitation for any of the pollutants listed by the Department in this permit; and
      iv. Any noncompliance that may endanger human health or the environment.
   (2) During normal business hours, the Department's Regional office must be called. Outside of normal business hours, the Department must be contacted at **1-800-452-0311** (Oregon Emergency Response System).
   (3) A written submission must be provided within 5 days of the time the permittee becomes aware of the circumstances. The written submission must contain:
      i. A description of the noncompliance and its cause;
      ii. The period of noncompliance, including exact dates and times;
      iii. The estimated time noncompliance is expected to continue if it has not been corrected;
      iv. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance; and
      v. Public notification steps taken, pursuant to General Condition B.**7**
   (4) The Department may waive the written report on a case-by-case basis if the oral report has been received within 24 hours.

6. <u>Other Noncompliance</u>

The permittee must report all instances of noncompliance not reported under General Condition D.4 or D.5, at the time monitoring reports are submitted. The reports must contain:

a. A description of the noncompliance and its cause;
b. The period of noncompliance, including exact dates and times;
c. The estimated time noncompliance is expected to continue if it has not been corrected; and
d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance.

7. <u>Duty to Provide Information</u>

The permittee must furnish to the Department within a reasonable time any information that the Department may request to determine compliance with the permit or to determine whether cause exists for modifying, revoking and reissuing, or terminating this permit. The permittee must also furnish to the Department, upon request, copies of records required to be kept by this permit.

Other Information: When the permittee becomes aware that it has failed to submit any relevant facts or has submitted incorrect information in a permit application or any report to the Department, it must promptly submit such facts or information.

8. <u>Signatory Requirements</u>
All applications, reports or information submitted to the Department must be signed and certified in accordance with 40 CFR Section 122.22.

9. <u>Falsification of Information</u>
Under ORS 468.953, any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit, including monitoring reports or reports of compliance or noncompliance, is subject to a Class C felony punishable by a fine not to exceed $100,000 per violation and up to 5 years in prison. Additionally, according to 40 CFR 122.41(k)(2), any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit including monitoring reports or reports of compliance or non-compliance shall, upon conviction, be punished by a federal civil penalty not to exceed $10,000 per violation, or by imprisonment for not more than 6 months per violation, or by both.

10. <u>Changes to Indirect Dischargers</u>
The permittee must provide adequate notice to the Department of the following:
   a. Any new introduction of pollutants into the POTW from an indirect discharger which would be subject to section 301 or 306 of the Clean Water Act if it were directly discharging those pollutants and;
   b. Any substantial change in the volume or character of pollutants being introduced into the POTW by a source introducing pollutants into the POTW at the time of issuance of the permit.
   c. For the purposes of this paragraph, adequate notice shall include information on (i) the quality and quantity of effluent introduced into the POTW, and (ii) any anticipated impact of the change on the quantity or quality of effluent to be discharged from the POTW.

<u>SECTION-E, DEFINITIONS</u>

1. *BOD* means five-day biochemical oxygen demand.
2. *CBOD* means five day carbonaceous biochemical oxygen demand
3. *TSS* means total suspended solids.
4. "*Bacteria*" includes but is not limited to fecal coliform bacteria, total coliform bacteria, and E. coli bacteria.
5. *FC* means fecal coliform bacteria.
6. *Total residual chlorine* means combined chlorine forms plus free residual chlorine
7. *Technology based permit effluent limitations* means technology-based treatment requirements as defined in 40 CFR Section 125.3, and concentration and mass load effluent limitations that are based on minimum design criteria specified in OAR Chapter 340, Division 41.

8. *mg/l* means milligrams per liter.
9. *kg* means kilograms.
10. *m^3/d* means cubic meters per day.
11. *MGD* means million gallons per day.
12. 24-hour *Composite sample* means a sample formed by collecting and mixing discrete samples taken periodically and based on time or flow. The sample must be collected and stored in accordance with 40 CFR Part 136.
13. *Grab sample* means an individual discrete sample collected over a period of time not to exceed 15 minutes.
14. *Quarter* means January through March, April through June, July through September, or October through December.
15. *Month* means calendar month.
16. *Week* means a calendar week of Sunday through Saturday.
17. *POTW* means a publicly owned treatment works.

(Schedule-F, last update 9.18.2009)

GLS: Scappoose PermitDoc 20Apr2010.docx
Revised: May 3, 2010

C-2:
NPDES Fact Sheet

DEQ FILE

FACT SHEET
And
NPDES WASTEWATER DISCHARGE PERMIT EVALUATION

Department of Environmental Quality
Northwest Region – Portland Office
2020 SW 4th Ave., Suite 400, Portland, OR 97201
Telephone: (503) 229-5263

PERMITTEE:
City of Scappoose
33568 East Columbia Avenue
Scappoose, OR 97056

SOURCE LOCATION:
Scappoose Wastewater Treatment Facility
34485 East Columbia Avenue
Scappoose OR 97056

SOURCE CONTACT:
Steve Wabschall — Telephone Number: 503-543-7184

PERMIT WRITER:
Garry L. Sage, EIT — Telephone Number: 503-229-5690

PROPOSED ACTION: Issuance of a renewal National Pollutant Discharge Elimination System (NPDES) wastewater discharge permit

SOURCE CATEGORY: Minor Domestic

TREATMENT SYSTEM CLASS: Level III

COLLECTION SYSTEM CLASS: Level II

PERMIT APPLICATION DATE: June 9, 2009

PERMIT APPLICATION NUMBER: 971614

Table of Contents **Page**

Attachments

Summary of Proposed Permit Changes

Only minor changes are proposed for this renewal permit. Effluent and Multnomah Channel water quality data collected over the last 5-year permit cycle indicates that the facility is protective of all beneficial uses in the receiving stream. Effluent filters will soon be installed at the sewage treatment plant (STP) to improve summer season effluent quality by reducing total suspended solids (TSS). A TSS reduction should help reduce total metals concentrations in final effluent.

Documentation on File for Public Review

The following documents are on file for public review:
- DEQ's Water Quality Permit Checklist,
- STP aerial view and treatment process schematic,
- Notice of Non-compliance database printout,
- Oregon's 2004/2006 Integrated Report for water quality in Multnomah Channel,
- A copy of the Scappoose Biosolids Management Plan dated February 2003,

- Fish use/rearing maps for Multnomah Channel,
- Whole effluent toxicity (WET) test data (Years 2004 through 2007),
- Anti-degradation Review Sheet, and
- Groundwater Review Sheet.

Background

Introduction

The City of Scappoose operates a sewage treatment plant (STP) located on the southeast edge of the City. Wastewater is treated and discharged to Multnomah Channel (Outfall 001) in accordance with NPDES Permit number 100677. The current facility permit was issued on March 18, 2005 and expired on December 31, 2009. This permit is one of several NPDES permits in the Willamette River Basin permitting cycle.

The Department of Environmental Quality (DEQ or Department) received a renewal application on June 9, 2009. A renewal permit is necessary to discharge to state waters pursuant to provisions of Oregon Revised Statutes (ORS) 468B.050 and the Federal Clean Water Act. The Department proposes to renew the permit.

Wastewater Treatment Plant History

The STP was originally placed into operation in the early 1970's. The last major expansion occurred in 1994 when the design average dry weather flow (ADWF) was increased from 0.5 million gallons/day (mgd) to ADWF = 1.515 mgd. The facility upgrade was required to accommodate population growth in the service area and to treat a high strength waste stream from the Steinfeld's Products Company. Steinfeld's, however, permanently ceased operation on November 30, 2001. Consequently, the loading on the treatment facility was significantly reduced.

List of Wastewater Treatment and Collection System Facilities

STP

- Influent pump station (PS) with 4 pumps in a wet-well/dry-well configuration,
- Influent 24-hour composite sampler,
- Influent flow measurement (Parshall flume),
- Plastic-lined aeration basin equipped with 5 mixing pumps and 2 aspirator pumps (pond is 11 feet deep, 200 feet long, and100 feet wide),
- Two clarifiers,
- Ultra-violet (UV) disinfection (2 banks each with 11 bulb racks at 8 bulbs per rack = 88 bulbs per bank),
- 24-hour composite effluent sampler,
- Six-cell aerobic sludge digestion structure,

- Four pump effluent PS with each pump intake screened with a 1 centimeter grid screen basket,
- 12-inch diameter outfall pipe approximately 1-mile long ending underwater in the Multnomah Channel (single-port diffuser design),
- Two clay-lined biosolids storage lagoons (Currently emptied during alternate years over a 2-3 day interval by using a manure spreader. Biosolids are typically 11% solids and approximately 80 dry tons of biosolids are applied each year),
- 28.1 acre biosolids application area (currently grass covered and used for cattle grazing),
- Two emergency diesel-electric generators (one installed at the influent PS and the other at the effluent PS).
- Emergency power at the treatment plant was upgraded in May 2007 to provide full facility coverage during a power outage/emergency, per current permit Schedule-C, Condition #2.

<u>Collection System</u>

- Two trailer-mounted portable generators for use at the 5 collection system PSs, and
- Four Hydronics package PSs and the Smith Road PS for a total of 5 PSs in the sewage collection system.

<u>STP Flow and Treatment Status Discussion</u>

Dry weather flows do not include the high levels of infiltration and inflow (I&I) that are associated with the winter in Oregon. The design ADWF estimates how much treatment capacity is available to treat organic loads. For this facility, the design <u>ADWF = 1.515 mgd</u>. The actual ADWF (May 1 through October 31) for the past two years was <u>0.592 mgd</u> <u>(Attachment-1)</u>. Current flow data indicates that this facility is at 39% of its organic treatment capacity. Based on current low flows and the lack of recurring effluent violations, no major expansion of the facility is needed at this time.

The actual average wet weather flow (AWWF, November 1 through April 30) for the past two years was <u>0.747 mgd</u> <u>(Attachment-1)</u>. The peak day winter season flow over the past two years was <u>1.905 mgd</u> (January 2009 discharge monitoring report (DMR)). The design peak wet weather flow is <u>3.24 mgd</u>. The STP currently does not have significant inflow and infiltration (I&I) and there is adequate hydraulic capacity.

<u>STP Improvements Proposed for the Upcoming 5-Year Permit Cycle</u>

In December 2009 the City received DEQ approval for the following STP additions:

- Replace an existing comminuter at the headworks with a fine screen (0.25-inch mesh) rated at a design peak flow = 4 mgd.
- Add disk filters in two new basins. Filters will be used during summer season to improve effluent quality. Installation includes two parallel installations of four disks each

with a separate capacity of 2 mgd and a combined capacity of 4 mgd. Additional filter disks can be added to increase the total capacity to 6 mgd.

- Add a new in-plant PS to provide additional head for the disk filter assemblies. The PS will have two, 2 mgd variable speed pumps.
- Replace UV disinfection ballasts and bulbs.
- Replace sampling equipment.

Receiving Stream Water Quality

Multnomah Channel, the receiving stream for STP effluent at Outfall 001, is a side channel of the Lower Willamette River. Currently, the Lower Willamette River Sub-basin is water quality limited (WQL) for: organic and metallic toxics, biological criteria, bacteria, and temperature. A Willamette Basin total maximum daily load (TMDL) was issued in September 2006 to address bacteria, mercury, and temperature. Provisions of this TMDL apply to Multnomah Channel at Outfall 001.

Proposed Permit Complies With Willamette Basin TMDL

Willamette Basin TMDL requirements for temperature, mercury, and bacteria are completely addressed by the proposed renewal permit. The effluent bacteria limit in Schedule-A of the current and proposed permit complies with the TMDL *E. coli* bacteria waste load allocation (WLA) for this source. Effluent temperature has no reasonable potential (NRP) to violate TMDL constraints (Attachments-1 & -2). Effluent mercury monitoring (Attachment-3), per Schedule C, Compliance Condition-1 of the current permit demonstrates that effluent mercury levels are below the quantification limit (QL) established in DEQ's Internal Management Document, Reasonable Potential Analysis for Toxic Pollutants.

Recent Facility Inspections

The last STP inspection by DEQ Source Control staff was held on September 26, 2007; and reported on October 30, 2007. On the day of the inspection, the treatment facility was found to be in compliance. A subsequent mixing zone (MZ) inspection was conducted by DEQ's Laboratory staff on September 15, 2008. The MZ study/report was published on April 3, 2009.

ZID and MZ 7Q10 Dilutions

The Department conducted a MZ study for Scappoose Outfall 001 on September 15, 2008. DEQ's MZ report dated April 3, 2009 (summarized in Attachment-4) determined that dilutions for the zone of immediate dilution (**ZID**) and the **MZ** are **1.2** and > **40**, respectively. These dilutions are based on the worst-case, 7-day average summer low-flow expected during a 10-year interval (7Q10 low flow).

Biosolids Management and Utilization

A Biosolids Management Plan (BMP) was submitted by the permittee on June 9, 2009 for DEQ review. This document is identical to the February 2003 edition that was submitted for Public Notice during the last permit renewal process. The BMP has not changed in scope or content. It is still applicable to the current production and use of biosolids at this treatment facility. A copy is on file for public review by interested citizens.

All biosolids must be managed in accordance with the Department approved BMP to ensure compliance with the federal biosolids regulations (40 Code of Federal Regulations (CFR) Part 503). After treatment necessary to comply with vector attraction and pathogen reduction requirements, the Class-B biosolids are beneficially land applied on fields adjacent to the treatment facility. Any future land application sites must conform to the site selection criteria in the BMP and must be in Columbia County.

The wastewater facility currently sends 3,000-4,000 gallons/day of sewage solids to aerobic digestion. After digestion, settling, and decanting are completed, biosolids are transferred to two clay lined facultative lagoons which were built in 1972. The permittee typically applies biosolids over a 2-3 day period each year when one lagoon is emptied onto 35 acres (28.1 usable acres) of City owned pasture land by using a manure spreader. This land is adjacent to the STP. Biosolids are typically 11% solids, and about 82 dry tons are applied to the authorized site each year. Grazing by cattle is used to control the height of the grass cover crop. At current biosolids application rates, the available 28.1 acres has about a 32% reserve capacity, and additional lands will not be needed within the next 5-years.

On February 15, 1995 the City of Scappoose signed an agreement with the City of St. Helens to send up to 5,000 gallons/week of biosolids to the St. Helens STP in the event that a biosolids disposal emergency occurred at the Scappoose STP. This agreement is still in affect, but has never been used.

The proposed permit requires annual reporting of percent volatile solids reduction accomplished through stabilization. It also requires the submittal of an annual report by February 19 each year, whenever biosolids are land applied.

Inflow and Infiltration (I&I)

Based on an evaluation of summer and winter period flows (see discussion above), I&I does not appear to be an extreme problem for this treatment facility. As a matter of infrastructure protection policy, the Department recommends a long-term program that will completely replace the collection system based on a life expectancy of 60 to 100 years. The replacement program should be directed at the oldest sub-basins, or those in the worst condition. At this time the City has a continuing, aggressive program for sewer maintenance and repair.

Pretreatment

The permittee does not have a formal pretreatment program, nor is one required for this source.

Pollutants Discharged

Permitted Pollutants

The existing permit allows the City of Scappoose to discharge daily, year-around from its STP. The existing permit sets limits on the following pollutants: (1) Five-day Biochemical Oxygen Demand (BOD_5), (2) Total Suspended Solids (TSS), (3) *E. coli* bacteria, (4) pH, and (5) Pollutant (BOD_5 & TSS) removal efficiency. The proposed renewal permit regulates these same pollutants without change.

The renewal permit proposes continued monitoring for effluent ammonia and DO, influent and effluent temperature, and Multnomah Channel temperature. This monitoring data is necessary to further document effluent and background water quality trends for the next permit renewal in five years.

Effluent Metals Discussion

Under the current permit, the permittee was required by Schedule-C, Compliance Condition #1 to collect and sample effluent for total mercury. Eight quarterly ultra-clean, total mercury tests were required. Attachment-3 lists the mercury sampling data. As discussed above, test results showed "non-detect" (at 0.005 micro-grams per liter (µg/L)) for all effluent and Multnomah Channel samples. This testing indicates that the Scappoose STP complies fully with the Willamette Basin TMDL for mercury reduction.

Additional metals data was required by Schedule-D, Special Condition #5 of the current permit. With the exceptions of copper and zinc, these tests showed relatively low levels of metals present in STP effluent (Attachment-5). No effluent copper or zinc sample exceeded the acute limit set for these metals (copper=18 µg/L and zinc=120 µg/L). Only one copper test (16.1 µg/L) showed the effluent copper concentration above the chronic criterion = 12 µg/L. Since the minimum 7Q10 dilution in the ZID = 1.2 and MZ > 40 (see Page 5 of Attachment-4), the highest measured copper concentration after ZID dilution is (16.1 µg/L)/1.2 = 13.4 µg/L and after MZ dilution = (16.1 µg/L)/40 = 0.4 µg/L). This would not cause acute copper toxicity outside of the ZID or chronic toxicity outside of the MZ given worst-case dilution conditions.

When evaluating effluent metals toxicity, it is important to note that the effluent PS operates intermittently. As the STP discharge flow decreases, the pause between pump cycles increases. This means that effluent is not constantly discharged to Multnomah Channel; the outfall discharges only when the PS runs. At the PS a wetwell level control is tripped by rising water. One or more pumps run until a second level control for "low water" is reached and the pumps shut down. The process repeats once the wetwell fills to the "pump on" level. The PS inflow rate determines the pump cycle time. During summer season, calculations show that the effluent PS operates only about 20% - 25% of the time (Page 4, Attachment-4). Intermittent PS operation favors ZID and MZ dilution by increasing Channel mixing. Therefore, the intermittent effluent discharge should reduce ZID and MZ metals concentrations below those found under steady-state conditions. DEQ's MZ study dated April 3, 2009 (Attachment-4) was conducted under steady-state conditions; i.e. only when pumps were discharging at Outfall 001.

Whole Effluent Toxicity (WET) Tests

WET tests are summarized in Attachment-4. Summer season WET tests were completed for the Years 2004 through 2007 (Table-1, Attachment-4). All WET tests showed no acute toxicity, i.e. no statistically significant reduction (NSSR) in survival of the test organisms. All flathead minnow tests for chronic toxicity showed NSSR in survival. With the exception of Years 2004 & 2005, WET chronic testing showed NSSR in survival. For the Years 2004 and 2005, chronic testing with *Ceriodaphnia dubia* showed limited impairment.

Based on the WET testing summarized above, no special WET testing is proposed for inclusion in the renewal permit. However, prior to the next permit renewal 4 additional WET tests are required on Page 15 (Supplemental Application Information, Part E) of EPA Form 3510-2A (Rev. 1-99). This level of WET testing appears to be sufficient given the worst-case 7Q10 ZID and MZ dilutions, and past WET test performance (Table-1, Attachment-4) discussed above.

pH Reasonable Potential Analysis (RPA)

A pH RPA model was run for treatment facility effluent, per Schedule-A limits (Attachment-6). Modeling used the most recent DEQ MZ modeling results and DMR data. The RPA determined that there is no reasonable potential for a pH violation, given the proposed permit limits listed in Schedule-A.

Ammonia

Attachment-1 shows weekly average effluent ammonia levels to be ≤ 0.4 mg/L, since February 2006 (except for 1.6 mg/L during Week-2 of March 2008). Non-detect ammonia values are shown at the detection limit of 0.1 mg/L in Attachment-1. Over a total of 223 weeks, the maximum ammonia reading was 2.5 mg/L (3rd week in October 2005). These consistently low ammonia concentrations indicate that effluent does not create toxicity or significant oxygen reduction in the ZID, MZ, or Multnomah Channel. An ammonia reasonable potential analysis (RPA) was run for effluent (Attachment-7). There is no reasonable potential (NRP) for effluent induced ammonia toxicity in Multnomah Channel.

Outfall

Treated wastewater is discharged to Multnomah Channel at River Mile 10.6. The outfall is 12-inches in diameter and approximately 1 mile long. The outfall pipe is single-port and extends 15 feet from the bank. The port points 73 degrees downstream relative to the bank and downward at 10 degrees below the horizontal. At the outfall, the channel is approximately 600 feet wide and has a maximum water depth of 42 feet. During summer low water conditions, the outfall typically has only a few feet of cover, and it discharges in shallow water near the channel edge. The outfall location has some good and bad aspects. Discharges tend to hug the bank edge and mixing is reduced; however, warm water discharges during the summer season do not disrupt deeper, cool water in the channel (cold water refugia). As discussed above effluent discharges are not continuous.

Receiving Stream Impacts

Beneficial Uses

The designated beneficial uses for Multnomah Channel at Outfall 001 are: irrigation, livestock watering, anadromous fish passage and rearing, salmonid passage and rearing, resident fish and aquatic life, wildlife and hunting, fishing, boating, water contact recreation, and aesthetic quality.

Water Quality Standards

Water quality standards for the Willamette Basin and Multnomah Channel are listed in the Oregon Administrative Rules (OAR), Chapter 340, Division 041-0340. These standards were developed to protect the beneficial uses for the basin, as described above. At this location, Multnomah Channel is WQL for a number of parameters, as listed under the Receiving Stream Water Quality section above. For this discharge, summer season effluent temperature was the most problematic.

Willamette Basin TMDL Summary

As discussed above, the Willamette Basin TMDL was approved on September 2006 to correct temperature, bacteria, and mercury problems. Multnomah Channel is part of the Willamette Basin, i.e. the Lower Willamette River. The TMDL assigns pollutant Waste Load Allocations (WLAs) to the Scappoose point source discharge for bacteria and thermal load. The proposed renewal permit complies with all aspects of the TMDL and with OAR 340-041 water quality criteria. Disinfected effluent discharges comply with the instream WLA for bacteria listed in the TMDL. Mercury testing of effluent shows "non-detect" concentrations (see above). Thermal modeling for this source indicates that criteria for "small source bubble allocation" are met.

TMDL and Bacteria

The TMDL lists the Lower Willamette River (River mile 0 to 25) as being WQL for elevated fecal bacteria concentrations (Table 2.2). The Scappoose outfall is located on this reach of the Willamette. Table 2.6 of the TMDL sets WLAs for STPs. For the Scappoose discharge, the WLA is:

"126 *E. coli* organisms/100 ml as a log-mean based on a minimum of 5 samples in a 30-day period, and not to exceed 406 *E. coli* organisms/100 ml in any single sample."

The proposed renewal permit complies with the draft TMDL for bacteria. The permit lists the TMDL's WLA for *E. coli* in Schedule A, as the applicable bacteria limit; and specifies the minimum *E. coli* sampling frequency to be 2/week or 8/month (Schedule-B).

TMDL and Mercury

The Willamette Basin TMDL for mercury is designed to restore the beneficial use of fish consumption to the Willamette River and its tributaries. This TMDL focuses on the bioaccumulation of mercury in edible fish tissue. DEQ acknowledges the current limitations to understanding the fate, transport, bioaccumulation, loading and sources of mercury in the Willamette Basin. For this reason DEQ proposes a two-step process for addressing mercury bioaccumulation. Step I establishes interim water column guidance values and sector-specific allocations based on the collected body of information currently available. Step II, as proposed by the draft TMDL, entails source specific mercury reductions beginning in Year-2009.

Step I interim guidance values, per the TMDL, are not site-specific numeric criteria (standards) but rather system-wide average annual concentrations that will allow a restoration of the beneficial uses of fish consumption and the protection of public health. The interim guidance values and allocations are based on total mercury, as opposed to methyl mercury. Methyl mercury is more difficult to measure in trace amounts, and is the form of mercury most likely to be found in fish tissue (bio-accumulated). The preliminary sector-specific allocations for total mercury will not be translated into numeric water quality based effluent limits for individual point sources at this time. The interim targets and allocations will be used to define the extent of the problem and to identify the level of effort needed to address the bio-accumulation of mercury in fish.

Under Step II, general permit requirements and/or a TMDL implementation rule will be drafted to establish monitoring requirements for mercury, to develop mercury reduction strategies, and to identify and implement best management practices for likely mercury point sources. Levels of toxic substances, including mercury, may not exceed the criteria listed in OAR 340-041, Table 20 which is based on criteria established by the United States Environmental Protection Agency (USEPA) and published in Quality Criteria for Water (1986) and subsequent revisions, unless otherwise noted.

The TMDL utilizes a fish tissue criterion of 0.30 mg/kg to establish an interim water column guidance value. Oregon's fish consumption advisories are issued by the Division of Health Services (DHS) when the average fish tissue concentrations of mercury exceed the threshold of 0.35 mg/kg. The use of 0.30 mg/kg in the TMDL, as opposed to 0.35 mg/kg, represents a conservative margin of safety on the order of 15% and is consistent with recently developed guidance from the USEPA.

As discussed above, DEQ sees no benefit in developing wasteload allocations at a level of detail finer than the sector-wide allocations presented in the TMDL; or individual NPDES permit limits at this point in time. In the future point sources within a sector may be required to develop Mercury Minimization Plans and to monitor their effluent to better characterize their contribution of mercury and the effectiveness of management measures. The implementation of best management practices should allow point sources to meet the overall allocation for the specific sector.

The proposed City of Scappoose discharge permit complies with TMDL mercury reduction. As discussed above, the *current permit* required a minimum of 8 "ultra-clean" total mercury tests (Schedule C, Compliance Condition #1). Condition #1, also, required two "ultra-clean, total mercury scans for the receiving water in Multnomah Channel to define the exact background conditions present at the discharge point. The permittee submitted the required mercury test data. Mercury testing shows non-detect for total mercury based on a quantitation limit = 0.005 μg/L (Attachment-3) for both effluent and Multnomah Channel. This source does not need a **mercury reduction strategy at this time.**

TMDL and Temperature

Discharges at Outfall 001 must comply with the 7-day-average maximum temperature criterion for a stream identified as having salmon and trout rearing and migration use (may not exceed 18.0 °C or 64.4 °F), per OAR 340-041-0028 (4) (c); unless "natural thermal potential" is applicable [OAR 340-041-0028 (8)]. Under the rule natural thermal potential (NTP) applies when a water body's 7-day-average maximum temperature exceeds the applicable criterion (in this case 18.0 °C). Natural Conditions Criteria [OAR 340-041-0028 (8)] are defined as follows:

> Where the department determines that the natural thermal potential of all or a portion of a water body exceeds the biologically-based criteria in section (4) of this rule, the natural thermal potential temperatures supersede the biologically-based criteria, and are deemed to be the applicable temperature criteria for that water body.

During the summer months of July, August, and September Multnomah Channel is naturally much warmer than 18.0 °C. The table below summarizes summer season, monthly average temperatures for effluent and Multnomah Channel (Years 2005 through 2009). Monthly average temperatures are derived by averaging the weekly maximum temperatures for that month (see Attachment-1 for weekly maximum temperatures). Attachment-2 contains 3 temperature modeling runs (Thermal Calculator Runs 2a, 2b, and 2c) using 18.0 °C, 19.4 °C, and 21.7 °C, respectively. Run 2a is the criterion run at 18.0 °C. Run 2b uses the NTP temperature for September 2008, and Run 2C uses the NTP temperature for July 2009. All three runs result in a "No Reasonable Potential" to exceed the allowable temperature. This means effluent does not exert a significant thermal effect on the Waters of Multnomah Channel during summer season. Several conservative assumptions go into this temperature analysis:

- The flow estimate for Multnomah Channel is 1% of the main-stem Willamette River 7Q10 low flow,
- Effluent cooling does not occur in the 1-mile long outfall pipe,
- Temperature measurements in the range of 0.01 °C are practical and reliable, and
- The full design ADWF (1.515 mgd) is used for the Thermal Calculator runs when current STP flow averages 0.6 mgd during summer season.

The table below shows those months (rose color) when effluent temperature exceeds Channel temperature. Note that these exceedances are all ≤ 0.2 °C. Eleven months out of 15 or 73% of the time (table below), the monthly average Channel temperature exceeds the monthly average effluent temperature:

Year/Month	Effluent Monthly Average Temperature (°C)	Channel Natural Thermal Potential, Monthly Average (°C)
2009		
September	20.3	20.5
August	21.3	23.1
July	21.8	21.7
2008		
September	19.6	19.4
August	20.9	21.8
July	20.6	20.5
2007		
September	19.6	20.5
August	20.9	21.9
July	21.1	22.0
2006		
September	19.9	19.8
August	20.7	22.2
July	21.6	22.5
2005		
September	19.5	19.7
August	22.0	23.9
July	22.1	22.6

Furthermore, all effluent temperatures are measured at the ultra-violet (UV) disinfection unit discharge pipe. This location is 1-mile from the discharge point. The outfall pipe is buried underground where temperatures are typically 50-55 °F, and groundwater is high. It is likely that the actual effluent discharge temperature is < the measured effluent temperature at the UV disinfection unit. Given these conditions and the data above, it appears that effluent has little or no effect on Channel water temperature during the critical summer months of July, August, and September.

<u>TMDL, Small Source Bubble Allocation</u>. The Scappoose discharge is included under the "Small Source Bubble Allocation" for thermal contribution, per the Willamette Basin temperature TMDL (Pages 4-70 & 71) as follows (*note italics below*):

> *<u>Bubble Allocations for Small Point Sources</u>*
> Many small point sources that discharge heat into the Willamette River system individually have an effect on the overall temperature of the river. The cumulative effects of these sources on main stem temperature are also very small; but because our knowledge of these sources is incomplete, small portions of the human use allowance are allocated as aggregate loads to small point sources. These WLAs represent a small portion of the total point source allocation at the point of maximum impact in each of the upper river, middle river, and lower river and *<u>together account for approximately 0.01°C of the 0.30°C HUA. The small point source bubble allocations represent heat loads from a dynamic set of individual and general NPDES sources</u>*. The number of sources, their locations and heat load characteristics will change as new sources are permitted, old sources discontinue operations, or waste treatment processes change. It is the intent of this WLA to address all point sources that are operating or have applied to operate under a NPDES general permit. The small point source sector allocation was based on a conservative treatment of point source data. Impacts were estimated by *<u>using available data where possible</u>*, or by assuming an average Willamette Basin TMDL: Temperature September 2006 OREGON DEPARTMENT OF ENVIRONMENTAL QUALITY, 4-71flow of 0.5 MGD and effluent temperatures of 22°C. Effluent temperatures from a number of the non-contact cooling sources included in sector allocation are substantially warmer than 22°C, but their effluent flow rates are generally very low. Some sources do not discharge throughout the critical period but were tallied in these initial WLAs. Finally, WLAs apply at the point of maximum impact of all point source loads (for example river mile 115 near Albany) when in fact sources are distributed throughout each reach. Heat loss from river of these small source loads was not factored into the WLAs. *<u>ODEQ will not assign individual effluent limits to each source within the small point source bubble allocation. Instead ODEQ will track the number of small sources within each river reach and estimate cumulative heat loads based on discharge monitoring reports or other effluent characterization approaches. To assist with this effort, some small sources, such as municipal treatment plants, may be required to collect additional effluent temperature data following issuance of the TMDL</u>*. However effluent monitoring is not required of most general permit sources and heat loads for each category such as non-contact cooling water, are assumed. Available reserve capacity will be drawn upon as the small source heat load approaches the bubble allocation limit.

Based on the above TMDL paragraphs, it is clear that the temperature TMDL, as it applies to the City of Scappoose, must be implemented by site-specific temperature measurements of effluent and the receiving water. As documented and discussed above, summer season temperature monitoring was performed for the last 5-years (2005 - 2009). RPA temperature modeling runs (Attachment-2, Runs 2b and 2c) show that the worst-case heating of effluent does not exceed 0.03 C for 25%-channel mixing when natural thermal potential temperatures are assigned.

Thermal Plume Limitations [OAR 340-041-0053 (2) (d) (A-D)]. Temperature TMDL compliance for this source also includes compliance with thermal plume limitations. These limits are established to protect salmonids inside the MZ. They are listed as follows:

- Impairment of an active salmonid spawning area where spawning redds are located or likely to be located (is prevented);
- Acute impairment or instantaneous lethality is prevented or minimized by limiting fish exposure to temperatures of 32.0 °C or more to less than 2 seconds;
- Thermal shock caused by a sudden increase in water temperature is prevented or minimized limiting potential fish exposure to temperatures of 25.0 °C or more to less than 5% of the cross section of 100% of the 7Q10 low flow of the water body; and
- Unless the ambient temperature is 21.0 °C or greater, migration blockage is prevented or minimized by limiting potential fish exposure to temperatures of 21.0 °C or more to less than 25% of the cross section of 100% of the 7Q10 low flow of the water body.

This source does not violate any of the above thermal plume limitations based on water quality data and the modeling discussed above.

Groundwater

DEQ's Groundwater Prioritization Worksheet was completed for the proposed permit and is on file. The Worksheet must be completed and attached, per the NPDES permit renewal; since the facility includes a sewage aeration basin, two biosolids storage ponds, and biosolids are agronomically applied to lands adjacent to the STP.

For purposes of groundwater protection, the Scappoose STP has a large sewage aeration basin and two biosolids storage ponds. The aeration basin has a plastic liner and the biosolids ponds are both protected from excessive leakage by 24-inch engineered clay liners. There are 16 irrigation and/or drinking water wells within 0.25 miles of the STP (attached to NPDES renewal application). During the last permit renewal (current permit), the City sampled irrigation well #04355/12 (100 feet deep), situated between the aeration basin and the biosolids ponds, for nitrates and *E. coli* bacteria. The testing showed a low nitrate level (0.9 mg/L) and *E. coli* $\leq$ 1/100 ml the detection limit. Three additional wells adjacent to the treatment facility were sampled for nitrate and bacteria. These wells roughly encircle the STP and are on private property. Sampling of the three wells indicated that STP ponds are not contaminating groundwater.

Based on the Department's current information, this facility has a low potential for adversely impacting groundwater quality. Therefore, Schedule-D of the proposed permit states that no groundwater evaluations will be required during this permit cycle. The permit also includes a condition in Schedule-A that prohibits any adverse impact on groundwater quality.

Stormwater

The Scappoose STP has a design flow greater than 1 mgd. Given this criterion, a General NPDES permit for storm water is required for a facility, if storm water is discharged from the plant site. The Scappoose STP does not discharge storm water; therefore, no storm water permit is necessary.

Permit History

The current permit was renewed on March 18, 2005 for a 5-year term and expired on December 31, 2009. It is currently under administrative continuation, pending renewal. The proposed renewal permit maintains summer BOD5/TSS limits at 10/10 mg/L (Basin standards) and winter BOD5/TSS limits at 25/25 mg/L. Mass loads are unchanged. Ammonia limits are not included in Schedule-A, and a thermal load limit is not required. Recent water quality monitoring data (Attachement-1) shows low effluent ammonia levels and summer season temperature within established TMDL constraints (see above).

An Antidegradation Review was completed with a recommendation to proceed with this permit action. A copy of the review sheet can be found in the file.

Compliance History

The monitoring reports for this facility were reviewed for the period since the current permit was issued, including any actions taken relating to effluent violations. The permit compliance conditions and all inspection reports for the same period were reviewed. Based on this review, no permit violations were documented at this facility during the term of the current permit.

PERMIT DISCUSSION

Face Page

The permittee is authorized to construct, install, modify, or operate a wastewater collection, treatment, control, and disposal system. Permits discharge of treated effluent to the Multnomah Channel within limits set by Schedule A and the following schedules. All other discharges are prohibited.

Schedule-A, Waste Discharge Limitations

BOD and TSS concentration and mass limits

Based on the Willamette Basin minimum design criteria, wastewater treatment resulting in a monthly average effluent concentration of 10 mg/L for BOD_5 and TSS must be provided from May

1 - October 31. From November 1 - April 30, a minimum of secondary treatment or equivalent control is required. Secondary treatment for this facility is defined as monthly average concentration limit of 30 mg/L for both BOD_5 and TSS. Winter limits in the renewal permit are more stringent than secondary, i.e. 25 mg/L for both BOD_5 and TSS. A previous permit set the winter TSS limit at 25 mg/L... a limit that applied after treatment facility upgrade and the incorporation of process waste water from Steinfeld's Products. To comply with anti-backsliding doctrine, the Department proposes to keep the winter TSS and BOD_5 limits at 25 mg/L.

The monthly average summer BOD_5 and TSS concentration limits are 10 mg/L with a weekly average limit of 15 mg/L for each.

The monthly average winter BOD_5 and TSS concentration limits are 25 mg/L with a weekly average limit of 37 mg/L for each.

The summer mass limits for biochemical oxygen demand (BOD_5) and suspended solids (TSS) are based on the design ADWF = 1.515 mgd, and the monthly average BOD_5 and TSS concentration limits of 10 mg/L and 10 mg/L, respectively.

The winter mass load limits for the facility are based on the design ADWF = 1.515 mgd, and the monthly average BOD_5 or TSS concentration limits of 25 mg/L and 25 mg/L, respectively. The limits are in accordance with OAR 340-041-0061 (10). All mass load limitations are rounded to two significant figures and/or nearest 5 pounds.

<u>BOD_5 and TSS</u>

The limits are:

(1) May 1 - October 31:

Parameter	Average Effluent Concentrations		Monthly Average lb/day	Weekly Average lb/day	Daily Maximum Lbs
	Monthly	Weekly			
BOD_5	10 mg/L	15 mg/L	125	190	255
TSS	10 mg/L	15 mg/L	125	190	255

(2) November 1 - April 30:

Parameter	Average Effluent Concentrations		Monthly Average lb/day	Weekly Average lb/day	Daily Maximum Lbs
	Monthly	Weekly			
BOD_5	25 mg/L	37 mg/L	315	475	630
TSS	25 mg/L	37 mg/L	315	475	630

Calculations:

(1) <u>Summer BOD_5 & TSS</u>

(a) 1.515 mgd x 8.34 #/gal x 10 mg/L monthly avg. = 125 lbs/day
(b) 125 lbs/day monthly avg. x 1.5 = 190 lbs/day weekly avg.
(c) 125 lbs/day monthly avg. x 2.0 = 255 lbs/day daily max.

(2) Winter BOD_5 and TSS

(a) 1.515 mgd x 8.34 #/gal x 25 mg/L monthly avg. = 315 lbs/day
(b) 315 lbs/day monthly avg. x 1.5 = 475 lbs/day weekly avg.
(c) 315 lbs/day monthly avg. x 2.0 = 630 lbs/day daily max.

A review of recent monitoring data indicates the City should generally be able to comply with the above permit limits. No changes from the previous permit are proposed.

BOD_5 and TSS Percent Removal Efficiency

A minimum level of percent removal for BOD_5 and TSS for municipal dischargers is required by the Code of Federal Regulations (CFR) secondary treatment standards (40 CFR, Part 133). An 85% removal efficiency limit is included in the proposed permit to comply with federal requirements. An examination of the DMR data indicates the permittee will have little difficulty meeting the limit with the current facilities.

pH

The Willamette Basin Water Quality Standard for pH is found in OAR 340-041-0345 (1) (a). The allowed range is 6.5 to 8.5. The proposed permit limits pH to the range 6.0 to 9.0. This limit is based on Federal wastewater treatment guidelines for sewage treatment facilities, and is applied to the majority of NPDES permittees in the state. Within the permittee's mixing zone, the water quality standard for pH does not have to be met. It is the Department's belief that mixing with ambient water within the mixing zone will ensure that the pH at the edge of the mixing zone meets the standard, and the Department considers the proposed permit limits to be protective of the water quality standard (see Attachment-6).

Bacteria

The proposed permit limits are based on an *E. coli* standard approved in January 1996. The proposed limits are a monthly geometric mean of 126 *E. coli* per 100 mL, with no single sample exceeding 406 *E. coli* per 100 mL. The new bacteria standard allows that if a single sample exceeds 406 *E. coli* per 100 mL, then the permittee may take five consecutive re-samples. If the log mean of the five re-samples is less than or equal to 126, a violation is not triggered. The re-sampling must be taken at four hour intervals beginning within 28 hours after the original sample was taken. The proposed bacteria limits are achievable and they comply with the draft TMDL, as previously discussed.

Ammonia

Ammonia limits are not listed in the renewal permit because modeling indicates (See Attachments-1 and -7) that effluent discharges have no reasonable potential to cause toxicity in the receiving stream under worst-case conditions. During the last 5-years (228 weekly samples), the highest weekly average maximum ammonia concentration was 2.5 mg/L. The long-term weekly average for all 228 weeks is 0.2 mg/L. For the last two years, ammonia samples were generally non-detect (at or below the detection limit of 0.1 mg/L). Attachment-7 (Freshwater Ammonia Criteria Calculator) indicates that ammonia limits are not necessary for this source; since weekly average and monthly average, maximum effluent ammonia levels are consistently near 0.2 mg/l.

Chlorine Residual

The treatment facility uses Ultra-Violet (UV) light to disinfect the treated wastewater. No chlorine or chlorine compounds may be used for disinfection purposes and no chlorine residual will be allowed in the effluent due to chlorine used for maintenance purposes.

Temperature

As discussed above, the Willamette Basin temperature TMDL provides a collective temperature waste load allocation (WLA) for small dischargers (small point source bubble allocation, Page 4-70) of 0.01 °C of the available 0.30 °C human use allowance (HUA). The TMDL assumes that these small dischargers have flows of approximately 0.5 MGD and effluent temperatures of 22 °C. Additionally, the TMDL states the following:

> ODEQ will not assign individual effluent limits to each source within the small point source bubble allocation. Instead ODEQ will track the number of small sources within each river reach and estimate cumulative heat loads based on discharge monitoring reports or other effluent characterization approaches.

At Scappoose, the Department modeled worst-case temperature impacts on the receiving stream (Attachment-2) relative to the collective small source bubble allocation for temperature. Modeling and subsequent analysis used the:

- Temperature WLA;
- 7Q10 worst-case low flow in Multnomah Channel;
- STP design ADWF;
- Criterion stream temperature (18 °C as 7-day-average maximum allowed temperature for salmon and trout rearing and migration);
- Natural thermal potential (NTP) temperature for the receiving water;
- Worst-case, 7-day-average maximum effluent temperature (24.0 °C); and
- 25% and 100% mix of effluent with the 7Q10 low flow.

For the criterion temperature of 18.0 °C, modeling indicates the facility has NRP to violate at 25% or 100% of the stream cross section. Using Channel NTP (Attachment-1) and the corresponding effluent temperature (average weekly maximum), modeling shows NRP for violation; since river temperature is affected ≤ 0.03 °C for 25% full-channel mixing. Therefore, no 7-day-average maximum temperature limit, or thermal load limit must be listed in Schedule-A of the renewal permit. The proposed renewal permit complies with the provisions of the TMDL for temperature reduction in the Willamette Basin.

Mixing Zone and Zone of Immediate Dilution

The allowable mixing zone is that portion of Multnomah Channel contained within a band extending out 100 feet from the shore side of the outfall, and 200 feet downstream and 200 feet upstream from the outfall. The Zone of Immediate Dilution (ZID) shall be defined as that portion of the allowable mixing zone that is within a 20 foot radius of the discharge point.

The Department believes that the beneficial uses of the receiving stream will not be affected by the discharge, and that the defined MZ meets the criteria in the rule.

Other Parameters

Emergency Overflows

Emergency overflows do not occur at the STP or in the sewage collection system; unless there is a catastrophe at a pump station or at the treatment plant, or a blockage occurs in a sewer mainline. Currently, all pumping stations have emergency back-up power, or are covered by portable generators stored off-site. The permittee has installed emergency power at the treatment facility to ensure that all essential equipment runs during a major electrical power outage.

Groundwater

The Department proposes to state in Schedule-A that no activities shall be conducted at the STP that could cause an adverse impact on existing or potential beneficial uses of groundwater. All wastewater and process related residuals shall be managed and disposed in a manner that will prevent a violation of the Groundwater Quality Rules (OAR 340-040).

Schedule-B, Minimum Monitoring and Reporting Requirements

In 1988, the Department developed a monitoring matrix for commonly monitored parameters. Proposed monitoring frequencies for all parameters are based on this matrix and, in some cases, may have changed from the current permit. The proposed monitoring frequencies for all parameters correspond to those of facilities of similar size and complexity in the state.

The permittee is required to have a laboratory Quality Assurance/Quality Control program. The Department recognizes that some tests do not accurately reflect the performance of a treatment facility due to quality assurance/quality control problems. These tests should not be considered when evaluating the compliance of the facility with the permit limitations. Thus, the Department is also proposing to include in the opening paragraph of Schedule-B a statement recognizing that some test results may be inaccurate or invalid (do not adequately represent the facility's performance) and should not be used in calculations required by the permit.

Influent Monitoring

(1) Influent flow is monitored daily and the flow meter must be calibrated on a semi-annual basis,
(2) BOD_5 and TSS are monitored by composite sampler twice per week, and
(3) pH is grab-sampled three times per week.

Effluent Monitoring

(1) BOD_5 and TSS must be monitored by composite sampler twice per week,
(2) pH must be grab-sampled three times per week,
(3) *E. coli* must be grab-sampled twice per week,
(4) Ammonia (grab-sample) and dissolved oxygen (measurement) must be monitored on a weekly frequency,
(5) UV radiation intensity must be read daily,
(6) BOD_5 and TSS mass loads must be calculated daily (twice per week frequency),
(7) BOD_5 and TSS average mass loads must be calculated weekly and monthly, and
(8) BOD_5 and TSS monthly average percent removal must be calculated.

Biosolids Sampling and Tracking

(1) Biosolids must be analyzed annually for nutrients,
(2) DEQ approved biosolids application sites must be used and each application must be documented,
(3) % volatile solids reduction must be recorded annually, and
(4) Fecal coliform must be sampled monthly when land applying biosolids.

Temperature Monitoring (May 1 through October 31)

(1) Daily maximum influent and effluent temperatures must be measured three times per week between 1500 and 1700 hours,
(2) The daily maximum temperature for Multnomah Channel must be measured three times per week between 1500 and 1700 hours, and
(3) The average of daily maximum effluent temperature must be calculated weekly.

Discharge Monitoring Reports

Discharge monitoring reports must be submitted to the Department monthly by the 15th day of the following month. The monitoring reports must identify the principal operators designated by the permittee to supervise the treatment and collection systems. The reports must also include records concerning application of biosolids and all applicable equipment breakdowns and bypassing.

Annual Biosolids Report

An annual biosolids report is required by February 19 of the following year, when biosolids were land applied.

Note 1 to Schedule-B: The UV disinfection process must be monitored on a daily basis for UV intensity. Daily UV intensity readings are required for at least 5 days per week.

Note 2 to Schedule-B: Biosolids sampling and testing must be as outlined.

Note 3 to Schedule-B: Calculation of % volatile reduction for biosolids must be as outlined.

Note 4 to Schedule-B: Fecal coliform sample analysis and reporting must comply with these requirements.

Note 5 to Schedule-B: After two years of temperature monitoring, and if approved in writing by the Department; monitoring may be waived for those months when the 7-day average of effluent temperature does not exceed the stream temperature standard of 18.0 °C.

Schedule-C, Compliance Conditions

There are no compliance conditions.

Schedule-D, Special Conditions

The proposed permit includes 6 special conditions. The requirements include:

(1) Sewer Cleaning Report. Permittee is required to annually inspect and clean approximately 20% of its sanitary sewer system. An annual Sewer Cleaning Report is due by February 19 of the following year.

(2) Biosolids Management. Biosolids must be managed per the approved Biosolids Management Plan and DEQ site authorization letters. No changes are allowed without prior DEQ approval. All new biosolids application sites shall meet the site selection criteria and be located in Columbia County.

(3) Changes in Biosolids Standards. It is proposed that this permit may be modified to incorporate any applicable standard for biosolids use or disposal promulgated under Section 405(d) of the Clean water Act, if the standard for biosolids use or disposal is more stringent than any requirements for biosolids use or disposal in the permit, or controls a pollutant or practice not limited in this permit.

(4) Operator Certification. The permittee must have its facilities supervised by personnel certified by the Department in the operation of treatment and/or collection systems.

(5) Groundwater. The proposed permit states that the permittee shall not be required to perform a hydro-geologic characterization or groundwater monitoring during the term of the permit, provided that the facility is operated to preclude adverse impacts on groundwater.

(6) Notification. Permittee shall notify DEQ, per response conditions listed in the permit General Conditions of any malfunction so that corrective action can be coordinated by DEQ and the permittee.

Schedule-E, Pretreatment Program

NOT APPLICABLE

Schedule-F, NPDES General Conditions

These standard conditions are attached to the draft permit. The latest version (September 18, 2009) of Schedule-F is used with the renewal permit.

GLS: Scappoose FactSheet 20Apr2010.docx
Revised: 20May2010

Attachments

1. Scappoose - DMR Summary
2. Scappoose - Thermal Calculator Runs
3. Scappoose - Effluent Mercury Data Summary
4. Scappoose - Permit Milestones
5. Scappoose - Effluent Metals Data Summary
6. Scappoose - STP MZ pH Study
7. Scappoose - Freshwater Ammonia Criteria Calculations

Scappoose FactSht Attachments 30April2010.docx

1. Scappoose - DMR Summary

1 – 1/10

SCAPPOOSE - DMR DATA SUMMARY

Temperature Criterion : 7-day-average maximum temperature of a stream identified as having salmon and trout rearing and migration use may not exceed 18.0 °C (64.4 °F), per OAR 340-041-0028 (4) (c) .

Effluent 7-day-average maximum temperature > 18.0 C & > channel temperature (May through October).

*Ammonia non-detect (N/D) readings were converted to 0.1 mg/L for subsequent calculations.

Month/Yr	STP Flow (mgd)	EFFLUENT pH (s.u.)	7-Day Average	Temp (°C)	BOD (mg/L)	TSS (mg/L)	D.O. (mg/L)	* Ammonia (mg/L)	Multnomah Channel Temp (°C)	Effluent Exceeds Channel Temp By (°C)	Estimate Natural Thermal Potential Monthly Average (°C)
Jan-10			*Week-1*	*	2.3	4.0	5.2	0.1	*	*	
Average	0.979		*Week-2*	*	1.9	5.9	5.0	0.1	*	*	
Minimum	0.765	6.6	*Week-3*	*	2.7	4.2	4.4	0.1	*	*	
Maximum	1.290	7.2	*Week-4*	*	2.1	4.6	5.1	0.1	*	*	
Dec-09			*Week-1*	*	3.6	1.6	5.1	0.1	*	*	
Average	0.726		*Week-2*	*	1.6	1.8	6.9	0.1	*	*	
Minimum	0.562	6.6	*Week-3*	*	4.3	4.5	4.8	0.1	*	*	
Maximum	1.000	7.3	*Week-4*	*	3.0	7.3	5.0	0.1	*	*	
Nov-09			*Week-1*	*	2.7	3.5	5.5	0.1	*	*	
Average	0.691		*Week-2*	*	2.1	3.0	4.8	0.1	*	*	
Minimum	0.495	6.8	*Week-3*	*	2.2	3.6	5.2	0.1	*	*	
Maximum	0.918	7.3	*Week-4*	*	2.4	1.5	5.0	0.1	*	*	
Oct-09			*Week-1*	17.1	1.9	2.5	4.7	0.1	15.8	1.3	13.9
Average	0.609		*Week-2*	15.9	2.0	4.3	4.2	0.1	13.1	2.8	
Minimum	0.474	6.5	*Week-3*	17.3	2.4	2.3	4.9	0.1	13.2	4.1	
Maximum	0.760	7.0	*Week-4*	16.7	1.4	3.6	6.7	0.1	13.4	3.3	
Sep-09			*Week-1*	20.9	2.7	3.8	4.2	0.1	21.7	-0.8	20.5
Average	0.609		*Week-2*	20.4	2.9	4.3	4.6	0.1	20.4	0.0	

1 – 2/10

Minimum	0.437	6.7	*Week-3*	20.6	2.7	3.9	4.2	0.1	20.5	0.1	
Maximum	0.786	7.0	*Week-4*	19.3	2.0	3.0	3.7	0.1	19.3	0.0	
Aug-09			*Week-1*	21.3	2.1	3.7	4.6	0.1	24.6	-3.3	23.1
Average	0.578		*Week-2*	20.8	1.5	6.5	3.9	0.1	23.9	-3.1	
Minimum	0.446	6.6	*Week-3*	22.4	2.4	5.7	4.4	0.1	22.2	0.2	
Maximum	0.765	7.9	*Week-4*	20.6	2.6	5.1	5.1	0.1	21.6	-1.0	
Jul-09			*Week-1*	20.0	2.8	1.8	4.6	0.1	20.0	0.0	21.7
erage	0.549		*Week-2*	21.2	2.3	2.5	5.1	0.1	21.2	0.0	
Minimum	0.459	6.6	*Week-3*	22.5	1.5	3.2	4.0	0.1	21.9	0.6	
Maximum	0.671	7.6	*Week-4*	23.5	2.2	3.3	4.8	0.1	23.7	-0.2	
Jun-09			*Week-1*	20.0	2.1	3.3	3.9	0.1	17.4	2.6	17.7
Average	0.567		*Week-2*	19.6	1.8	1.2	4.1	0.1	17.1	2.5	
Minimum	0.458	6.7	*Week-3*	20.1	2.3	2.5	4.1	0.1	17.7	2.4	
Maximum	0.712	7.2	*Week-4*	19.3	1.5	2.4	4.4	0.1	18.4	0.9	
May-09			*Week-1*	15.8	2.1	1.4	6.4	0.1	11.8	4.0	13.1
Average	0.619		*Week-2*	16.7	2.6	0.9	4.7	0.1	11.8	4.9	
Minimum	0.466	6.7	*Week-3*	17.7	1.7	1.6	4.0	0.1	13.7	4.0	
Maximum	0.781	7.3	*Week-4*	18.9	2.2	3.5	4.2	0.1	15.2	3.7	
Apr-09			*Week-1*	*	2.0	2.4	5.7	0.1	*	*	
Average	0.629		*Week-2*	*	1.8	0.9	4.5	0.1	*	*	
Minimum	0.444	6.6	*Week-3*	*	2.5	1.8	4.6	0.1	*	*	
Maximum	0.772	7.3	*Week-4*	*	1.5	1.9	4.6	0.1	*	*	
Mar-09			*Week-1*	*	1.2	1.7	5.8	0.1	*	*	
Average	0.672		*Week-2*	*	1.7	1.9	5.9	0.1	*	*	
Minimum	0.366	6.6	*Week-3*	*	1.9	1.5	5.2	0.1	*	*	
Maximum	0.887	7.1	*Week-4*	*	1.9	1.4	5.0	0.1	*	*	

1-3/10

Feb-09			*Week-1*	*	1.5	1.0	5.6	0.1	*	*	
Average	0.675		*Week-2*	*	3.3	3.8	5.4	0.1	*	*	
Minimum	0.514	6.8	*Week-3*	*	2.3	2.8	5.4	0.1	*	*	
Maximum	0.976	7.0	*Week-4*	*	1.9	1.8	4.2	0.1	*	*	
Jan-09			*Week-1*	*	4.1	7.0	4.8	0.1	*	*	
Average	0.886		*Week-2*	*	1.9	2.9	5.1	0.1	*	*	
Minimum	0.588	6.8	*Week-3*	*	1.5	2.6	5.2	0.1	*	*	
Maximum	1.905	7.4	*Week-4*	*	1.8	1.7	5.8	0.1	*	*	
Dec-08			*Week-1*	*	2.7	4.2	5.6	0.1	*	*	
Average	0.665		*Week-2*	*	8.5	7.1	5.7	0.1	*	*	
Minimum	0.530	6.6	*Week-3*	*	1.6	3.1	6.4	0.1	*	*	
Maximum	1.000	7.3	*Week-4*	*	4.9	3.5	6.2	0.1	*	*	
Nov-08			*Week-1*	*	2.2	3.4	5.8	0.1	*	*	
Average	0.632		*Week-2*	*	2.8	3.2	5.4	0.1	*	*	
Minimum	0.490	6.9	*Week-3*	*	1.9	1.5	5.5	0.1	*	*	
Maximum	0.840	7.1	*Week-4*	*	2.4	3.0	5.7	0.1	*	*	
Oct-08			*Week-1*	18.0	3.0	1.8	5.3	0.1	15.7	2.3	13.6
Average	0.605		*Week-2*	16.5	2.7	3.0	6.0	0.2	13.6	2.9	
Minimum	0.495	6.9	*Week-3*	14.6	2.5	1.8	6.8	0.2	12.6	2.0	
Maximum	0.843	7.5	*Week-4*	14.9	2.7	4.2	6.8	0.2	12.4	2.5	
Sep-08			*Week-1*	19.4	2.5	2.2	5.0	0.1	19.8	-0.4	19.4
Average	0.588		*Week-2*	20.4	1.5	2.6	5.5	0.2	19.9	0.5	
Minimum	0.466	6.5	*Week-3*	20.1	1.5	2.8	5.0	0.1	19.7	0.4	
Maximum	1.150	8.2	*Week-4*	18.5	1.4	4.3	5.1	0.1	18.1	0.4	
Aug-08			*Week-1*	20.8	1.7	1.7	4.7	0.1	21.0	-0.2	21.8
Average	0.591		*Week-2*	21.7	1.7	2.6	4.8	0.1	21.9	-0.2	
Minimum	0.407	6.5	*Week-3*	20.9	1.9	2.7	5.1	0.1	22.2	-1.3	

1-4/10

Maximum	0.759	7.2	*Week-4*	20.1	2.0	3.0	5.0	0.1	22.1	-2.0	
Jul-08			*Week-1*	20.6	1.7	3.6	3.4	*	18.6	2.0	20.5
Average	0.595		*Week-2*	21.5	1.5	3.6	4.7	*	20.7	0.8	
Minimum	0.403	6.9	*Week-3*	20.2	3.7	1.7	5.1	*	21.2	-1.0	
Maximum	0.786	7.8	*Week-4*	20.1	1.9	1.1	4.7	*	21.4	-1.3	
Jun-08			*Week-1*	16.9	1.9	2.5	5.6	0.1	12.4	4.5	13.5
Average	0.579		*Week-2*	16.1	1.7	3.3	4.7	0.2	11.2	4.9	
inimum	NA	6.9	*Week-3*	18.3	2.4	1.5	4.5	0.1	14.1	4.2	
Maximum	0.722	7.8	*Week-4*	19.5	1.5	3.7	4.5	0.1	16.1	3.4	
May-08			*Week-1*	15.0	4.3	5.6	5.1	0.1	11.7	3.3	11.7
Average	0.610		*Week-2*	16.4	4.0	3.9	4.7	0.2	11.2	5.2	
Minimum	0.489	6.9	*Week-3*	18.1	4.9	2.6	4.3	0.1	12.8	5.3	
Maximum	0.793	7.5	*Week-4*	17.2	3.9	3.5	4.8	0.2	11.2	6.0	
Apr-08			*Week-1*	*	2.0	7.0	6.0	0.1	*	*	
Average	0.662		*Week-2*	*	3.7	6.6	6.0	0.1	*	*	
Minimum	NA	6.8	*Week-3*	*	3.1	7.6	6.5	0.1	*	*	
Maximum	0.903	7.1	*Week-4*	*	4.9	7.7	5.7	0.1	*	*	
Mar-08			*Week-1*	*	2.7	5.1	6.2	0.2	*	*	
verage	0.628		*Week-2*	*	10.2	2.7	5.1	1.6	*	*	
Minimum	0.436	6.9	*Week-3*	*	2.0	4.9	6.3	0.1	*	*	
Maximum	0.981	7.7	*Week-4*	*	5.1	4.3	6.1	0.1	*	*	
Feb-08			*Week-1*	*	1.7	4.2	6.3	0.1	*	*	
Average	0.700		*Week-2*	*	2.4	2.9	8.2	0.1	*	*	
Minimum	0.535	6.5	*Week-3*	*	2.3	2.1	6.6	0.1	*	*	
Maximum	1.056	7.8	*Week-4*	*	2.0	2.7	6.4	0.1	*	*	
Jan-08			*Week-1*	*	2.8	4.8	6.1	0.1	*	*	

1 – 5/10

Average	0.890		*Week-2*	*	1.7	3.3	6.6	0.1	*	*	
Minimum	0.356	6.6	*Week-3*	*	3.2	6.8	7.8	0.1	*	*	
Maximum	1.311	7.3	*Week-4*	*	3.9	5.7	7.4	0.1	*	*	
Dec-07			*Week-1*	*	2.1	8.3	5.8	0.1	*	*	
Average	0.906		*Week-2*	*	2.0	5.5	6.0	0.1	*	*	
Minimum	0.459	6.7	*Week-3*	*	1.5	3.8	6.2	0.1	*	*	
Maximum	1.842	7.3	*Week-4*	*	2.4	2.9	6.5	0.1	*	*	
Nov-07			*Week-1*	*	1.9	6.2	5.2	0.1	*	*	
Average	0.478		*Week-2*	*	1.5	8.0	5.1	0.2	*	*	
Minimum	0.263	6.8	*Week-3*	*	1.5	5.2	5.2	0.1	*	*	
Maximum	0.714	7.3	*Week-4*	*	1.4	5.9	5.6	0.1	*	*	
Oct-07			*Week-1*	17.2	1.7	3.5	5.0	0.3	16.2	1.0	14.1
Average	0.494		*Week-2*	16.8	2.6	3.7	4.7	0.1	14.6	2.2	
Minimum	0.285	6.8	*Week-3*	16.6	1.5	3.6	5.4	0.1	13.6	3.0	
Maximum	0.691	7.4	*Week-4*	15.5	4.0	9.7	5.2	0.1	12.1	3.4	
Sep-07			*Week-1*	20.9	2.2	1.1	5.3	0.1	21.9	-1.0	20.5
Average	0.570		*Week-2*	20.4	1.9	1.7	4.6	0.3	21.8	-1.4	
Minimum	0.400	6.8	*Week-3*	18.6	2.4	3.7	5.3	0.2	19.8	-1.2	
Maximum	0.873	7.3	*Week-4*	18.3	1.6	2.7	4.9	0.2	18.5	-0.2	
Aug-07			*Week-1*	20.6	2.0	3.6	5.1	0.2	22.0	-1.4	21.9
Average	0.620		*Week-2*	21.0	2.0	2.8	4.4	0.2	22.1	-1.1	
Minimum	0.317	6.8	*Week-3*	21.2	2.1	2.5	4.9	0.1	21.9	-0.7	
Maximum	0.781	7.2	*Week-4*	20.6	1.7	1.7	4.4	0.2	21.6	-1.0	
Jul-07			*Week-1*	20.7	2.0	2.3	4.5	0.3	20.5	0.2	22.0
Average	0.655		*Week-2*	21.7	2.1	2.9	4.6	0.2	22.0	-0.3	
Minimum	0.429	6.7	*Week-3*	21.0	3.5	5.9	4.9	0.1	22.7	-1.7	
Maximum	0.864	7.5	*Week-4*	20.8	2.1	5.6	4.9	0.2	22.7	-1.9	

1- 6/10

Jun-07			*Week-1*	19.0	2.2	1.9	4.7	0.1	18.0	1.0	18.3
Average	0.622		*Week-2*	18.0	2.2	2.4	4.9	0.1	18.0	0.0	
Minimum	0.487	6.8	*Week-3*	19.0	2.7	1.7	4.6	0.2	18.0	1.0	
Maximum	0.749	7.4	*Week-4*	20.0	1.6	2.3	4.7	0.1	19.0	1.0	
May-07			*Week-1*	15.0	1.9	0.6	4.7	0.2	14.0	1.0	14.8
Average	0.615		*Week-2*	17.0	2.3	0.7	4.8	0.1	14.0	3.0	
Minimum	0.288	6.4	*Week-3*	17.0	1.4	2.1	5.0	0.2	15.0	2.0	
aximum	0.833	7.1	*Week-4*	18.0	1.7	1.9	4.6	0.1	16.0	2.0	
Apr-07			*Week-1*	*	2.9	1.3	6.9	0.1	*	*	
Average	0.602		*Week-2*	*	1.7	1.2	5.8	0.1	*	*	
Minimum	0.363	6.7	*Week-3*	*	3.2	1.1	4.8	0.1	*	*	
Maximum	0.969	7.0	*Week-4*	*	2.3	0.5	4.6	0.1	*	*	
Mar-07			*Week-1*	*	2.7	2.4	4.5	0.1	*	*	
Average	0.764		*Week-2*	*	2.5	2.8	4.8	0.1	*	*	
Minimum	0.506	6.7	*Week-3*	*	2.0	2.2	4.5	0.1	*	*	
Maximum	1.207	7.4	*Week-4*	*	1.5	3.2	4.5	0.2	*	*	
Feb-07			*Week-1*	*	2.8	0.8	4.1	0.1	*	*	
Average	0.666		*Week-2*	*	2.9	1.3	4.7	0.1	*	*	
linimum	0.447	6.9	*Week-3*	*	2.8	2.7	4.6	0.1	*	*	
Maximum	0.927	7.2	*Week-4*	*	2.9	1.3	4.8	0.1	*	*	
Jan-07			*Week-1*	*	2.0	1.9	4.5	0.1	*	*	
Average	0.838		*Week-2*	*	1.8	1.2	4.8	0.1	*	*	
Minimum	0.545	6.8	*Week-3*	*	1.5	1.3	5.2	0.1	*	*	
Maximum	1.268	7.1	*Week-4*	*	1.5	1.1	4.9	0.1	*	*	
Dec-06			*Week-1*	*	2.0	2.1	4.5	0.2	*	*	
Average	0.914		*Week-2*	*	1.6	3.5	4.4	0.2	*	*	

1 – 7/10

Minimum	0.614	6.8	*Week-3*	*	1.5	1.8	5.3	0.1	*	*	
Maximum	1.419	7.1	*Week-4*	*	2.0	2.5	4.9	0.1	*	*	
Nov-06			*Week-1*	*	1.7	5.0	4.6	0.1	*	*	
Average	0.725		*Week-2*	*	1.5	3.1	4.7	0.1	*	*	
Minimum	0.317	6.9	*Week-3*	*	3.0	5.5	4.9	0.1	*	*	
Maximum	1.246	7.1	*Week-4*	*	2.0	4.8	4.8	0.1	*	*	
Oct-06			*Week-1*	18.1	2.4	4.8	4.8	0.2	17.2	0.9	15.5
Average	0.461		*Week-2*	17.7	2.6	7.1	5.2	0.2	16.5	1.2	
Minimum	0.328	6.7	*Week-3*	16.6	1.9	7.2	5.1	0.1	14.6	2.0	
Maximum	0.705	7.1	*Week-4*	15.6	1.7	5.7	5.4	0.4	13.5	2.1	
Sep-06			*Week-1*	20.8	2.5	6.3	4.5	0.1	21.5	-0.7	19.8
Average	0.623		*Week-2*	19.5	1.9	6.2	4.4	0.2	20.5	-1.0	
Minimum	0.420	7.0	*Week-3*	18.9	3.3	5.7	4.5	0.3	19.2	-0.3	
Maximum	1.085	7.6	*Week-4*	20.2	2.4	4.9	4.1	0.2	18.0	2.2	
Aug-06			*Week-1*	21.0	1.9	5.0	4.5	0.1	23.3	-2.3	22.2
Average	0.801		*Week-2*	21.0	2.2	7.3	4.4	0.1	22.2	-1.2	
Minimum	NA	7.0	*Week-3*	20.0	2.0	4.0	5.1	0.1	21.3	-1.3	
Maximum	1.115	7.4	*Week-4*	21.0	2.1	5.8	5.1	0.1	21.8	-0.8	
Jul-06			*Week-1*	20.1	2.0	4.9	4.7	0.1	21.8	-1.7	22.5
Average	0.831		*Week-2*	20.9	1.5	4.9	4.5	0.1	21.5	-0.6	
Minimum	0.454	7.0	*Week-3*	21.4	1.7	5.7	4.4	0.2	22.7	-1.3	
Maximum	1.240	7.3	*Week-4*	24.0	1.7	4.4	4.5	0.1	24.0	0.0	
Jun-06			*Week-1*	18.5	2.1	3.8	4.8	0.2	15.9	2.6	17.7
Average	0.837		*Week-2*	18.8	2.8	4.5	4.8	0.2	16.5	2.3	
Minimum	0.631	6.8	*Week-3*	19.5	2.6	4.1	5.0	0.2	17.9	1.6	
Maximum	1.033	7.3	*Week-4*	21.8	2.2	5.0	4.8	0.3	20.6	1.2	

May-06			*Week-1*	15.4	6.0	5.1	5.6	0.2	14.2	1.2	15.3
Average	0.808		*Week-2*	16.3	2.6	6.1	5.8	0.3	14.6	1.7	
Minimum	0.555	6.8	*Week-3*	18.3	4.0	5.4	5.6	0.1	16.7	1.6	
Maximum	1.070	7.1	*Week-4*	17.1	3.7	6.7	5.4	0.2	15.7	1.4	
Apr-06			*Week-1*	*	2	3.3	6.1	0.1	*	*	
Average	0.827		*Week-2*	*	3.2	3.2	6.2	0.1	*	*	
Minimum	0.674	6.8	*Week-3*	*	2.5	2.5	5.8	0.1	*	*	
Maximum	1.110	7.2	*Week-4*	*	2.2	3.3	5.9	0.1	*	*	
Mar-06			*Week-1*	*	1.7	4.2	6.0	0.1	*	*	
Average	0.817		*Week-2*	*	2.5	2.6	6.3	0.1	*	*	
Minimum	0.674	6.8	*Week-3*	*	1.0	2.9	6.1	0.1	*	*	
Maximum	0.986	7.1	*Week-4*	*	2.0	2.4	6.0	0.15	*	*	
Feb-06			*Week-1*	*	1.9	3.4	5.8	0.80	*	*	
Average	0.980		*Week-2*	*	1.9	3.9	6.0	1.50	*	*	
Minimum	0.620	6.8	*Week-3*	*	4.4	3.6	6.0	0.52	*	*	
Maximum	1.560	7	*Week-4*	*	2.9	5.3	5.7	1.00	*	*	
Jan-06			*Week-1*	*	1.3	1.7	5.8	1.80	*	*	
Average	1.395		*Week-2*	*	1.2	4.7	5.8	1.10	*	*	
Minimum	1.028	6.8	*Week-3*	*	2.8	2.3	5.8	0.50	*	*	
laximum	1.937	7.1	*Week-4*	*	1.9	2.2	5.3	1.20	*	*	
Dec-05			*Week-1*	*	1.6	1.4	6.4	0.68	*	*	
Average	0.938		*Week-2*	*	1.9	1.6	6.2	0.15	*	*	
Minimum	0.515	6.9	*Week-3*	*	1.8	1.8	6.1	0.85	*	*	
Maximum	1.850	7.3	*Week-4*	*	2.0	2.6	6.1	0.10	*	*	
Nov-05			*Week-1*	*	2.3	3.2	5.9	0.30	*	*	
Average	0.798		*Week-2*	*	2.3	3.1	5.8	0.17	*	*	
Minimum	0.564	6.9	*Week-3*	*	2.6	4.2	5.4	0.31	*	*	

1–8/10

1-3/10

Maximum	0.990	7.2	*Week-4*	*	2.0	2.1	5.9	0.78	*	*	
Oct-05			*Week-1*	18.0	2.1	4.9	5.9	0.11	16.9	1.1	15.5
Average	0.748		*Week-2*	17.7	2.1	5.1	5.4	0.71	15.4	2.3	
Minimum	0.501	6.9	*Week-3*	18.0	2.2	3.4	5.9	2.50	15.2	2.8	
Maximum	0.951	7.2	*Week-4*	17.3	4.4	6.1	5.2	0.50	14.5	2.8	
Sep-05			*Week-1*	21.3	2.7	6.2	5.3	0.1	21.5	-0.2	19.7
Average	0.778		*Week-2*	20.3	1.7	4.6	5.6	0.11	20.5	-0.2	
Minimum	0.508	6.8	*Week-3*	18.4	2.2	4.6	5.5	0.15	18.7	-0.3	
Maximum	1.020	7.2	*Week-4*	18.1	2.3	5.5	5.8	0.20	18.0	0.1	
Aug-05			*Week-1*	22.4	3.1	4.8	5.7	0.11	24.2	-1.8	23.9
Average	0.817		*Week-2*	23.0	2.7	4.8	5.9	0.1	25.0	-2.0	
Minimum	0.626	6.7	*Week-3*	21.1	2.6	6.0	5.8	0.14	23.7	-2.6	
Maximum	0.992	7.1	*Week-4*	21.4	1.9	4.6	5.8	0.12	22.8	-1.4	
Jul-05			*Week-1*	21.0	2.4	4.5	6.3	0.10	21.8	-0.8	22.6
Average	0.876		*Week-2*	21.0	3.2	7.7	6.4	0.1	21.9	-0.9	
Minimum	0.542	6.6	*Week-3*	22.7	4.9	7.2	6.3	0.45	23.3	-0.6	
Maximum	1.198	7.1	*Week-4*	23.8	5.9	4.7	5.8	0.14	23.5	0.3	
Jun-05			*Week-1*	18.6	4.1	6.3	5.6	0.19	18.1	0.5	19.0
Average	0.839		*Week-2*	18.7	2.9	2.7	5.7	0.12	18.1	0.6	
Minimum	0.527	6.6	*Week-3*	20.3	2.5	3.6	5.8	0.11	19.5	0.8	
Maximum	1.080	7.4	*Week-4*	20.7	4.0	3.1	6.3	0.1	20.2	0.5	
May-05			*Week-1*	17.1	2.7	3.4	6.5	0.51	16.9	0.2	16.4
Average	0.897		*Week-2*	17.5	3.0	3.5	6.0	0.38	16.1	1.4	
Minimum	0.596	6.9	*Week-3*	17.6	2.7	3.0	5.5	0.13	16.1	1.5	
Maximum	1.110	7.3	*Week-4*	18.8	4.2	2.9	5.4	0.11	16.3	2.5	
Apr-05			*Week-1*	*	4.0	7.1	6.9	0.14	*	*	

1–10/10

Average	0.986		*Week-2*	*	4.4	4.6	6.6	0.20	*	*
Minimum	0.650	6.8	*Week-3*	*	4.3	4.4	6.7	0.1	*	*
Maximum	1.466	7.3	*Week-4*	*	3.5	5.8	6.5	0.38	*	*
Mar-05			*Week-1*	*	10.5	15.3	6.6	*	*	*
Average	0.801		*Week-2*	*	6.0	12.4	6.7	*	*	*
Minimum	0.522	6.8	*Week-3*	*	7.3	8.9	6.7	*	*	*
Maximum	1.300	7.2	*Week-4*	*	5.3	9.9	6.9	*	*	*

fluent 7-day-average maximum temperature > 18.0 C & > channel temperature (May through October).

										Effluent
									Multnomah	Exceeds
	STP	EFFLUENT							Channel	Channel
	Flow	pH	7-Day	Temp	BOD	TSS	D.O.	Ammonia	Temp	Temp By
Month/Yr	(mgd)	(s.u.)	Average	(°C)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(°C)	(°C)
MAXIMUM	1.937	8.2		24.0	10.5	15.3	8.2	2.5	25.0	6.0
MINIMUM	0.263	6.4		14.6	1.0	0.5	3.4	0.1	11.2	-3.3
SAMPLE NO.				120	236	236	236	228	120	
AVERAGE				19.2	2.6	3.8	5.3	0.2	18.4	
STD DEVTN								0.280		
CV								1.432		

Total weeks with effluent 7-day-average maximum temperature > 18.0 C & > channel temperature (May 2005 through Jan 2010).

	SUM	COMMENTS
May Weeks =	4	
June Weeks =	17	
July Weeks =	5	
August Weeks =	1	
September Weeks =	5	
October Weeks =	0	
Total Weeks =	32	

2. Scappoose - Thermal Calculator Runs

Attachment 2a

Facility Name: Scappoose STP **Date:** 4/19/2010

Enter data into white cells below:			
7Q10 =	**62.9**	cfs	1)
Ambient Temperature or Criterion	**18**	°C	2)
Effluent Flow =	**1.515**	mgd	3)
Effluent Temperature	**24**	°C	4)
			5)

25% of 7Q10 = **15.7** cfs

25% dilution = **8** dilution = (Qe+Qr)/Qe

Temperature at 25% cross section = 18.78 °C **No Reasonable Potential**

1) 10% of Willamette R. 7Q10 as measured at river mile 12.0 (Gage 14211720 @ Portland)
0.10(6290 cfs) = **629 cfs**. 7Q10 flow from DEQ Willamette Basin TMDL-Temperature, P. 4-131, Table 4.38.

1) 0.05% of Willamette R. 7Q10 as measured at river mile 12.0 (Gage 14211720 @ Portland)
0.05(6290 cfs) = **314.5 cfs**. 7Q10 flow from DEQ Willamette Basin TMDL-Temperature, P. 4-131, Table 4.38.

1) 0.01% of Willamette R. 7Q10 as measured at river mile 12.0 (Gage 14211720 @ Portland)
0.01(6290 cfs) = **62.9 cfs**. 7Q10 flow from DEQ Willamette Basin TMDL-Temperature, P. 4-131, Table 4.38.

2) 7-day-average maximum temperature of a stream identified as having salmon and trout rearing and migration use may not exceed 18.0 °C, per OAR 340-041-0028 (4) (c).

3) Design average dry weather flow (ADWF) for STP = 1.515 mgd (taken from current permit, Schedule-A).

4) Highest weekly average maximum effluent temperature recorded in DMRs (March 2005 through January 2010 DMR High temperature value was taken from 4th week of July 2006. During this week Multnomah Channel's high temperature was also 24.0 °C.

5) Notes:

a) Effluent temperature is measured at STP's UV disinfection system discharge.

b) Effluent is pumped from the STP to Multnomah Channel via the outfall pipe. Pumping is a discontinuous process; i.e. wet well fills until a preset level is reached and pumps come on. Pumps stop at preset level and process is repeated.

c) The outfall pipe is one mile long. It is reasonable to assume that considerable cooling occurs in the buried pipe prior to discharge.

Attachment - 2b

Facility Name: Scappoose STP **Date:** 4/22/2010

Enter data into white cells below:

7Q10 =	**62.9**	cfs	1)
Ambient Temperature or Criterion	**19.4**	°C	2)
Effluent Flow =	**1.515**	mgd	3)
Effluent Temperature	**19.6**	°C	4)
			5)

25% of 7Q10 = **15.7** cfs

25% dilution = **8** dilution = (Qe+Qr)/Qe

Temperature at 25% cross section = 19.43 °C No Reasonable Potential

1) 10% of Willamette R. 7Q10 as measured at river mile 12.0 (Gage 14211720 @ Portland)
0.10(6290 cfs) = **629 cfs**. 7Q10 flow from DEQ Willamette Basin TMDL-Temperature, P. 4-131, Table 4.38.

1) 0.05% of Willamette R. 7Q10 as measured at river mile 12.0 (Gage 14211720 @ Portland)
0.05(6290 cfs) = **314.5 cfs**. 7Q10 flow from DEQ Willamette Basin TMDL-Temperature, P. 4-131, Table 4.38.

1) 0.01% of Willamette R. 7Q10 as measured at river mile 12.0 (Gage 14211720 @ Portland)
0.01(6290 cfs) = **62.9 cfs**. 7Q10 flow from DEQ Willamette Basin TMDL-Temperature, P. 4-131, Table 4.38.

2) 7-day-average maximum weekly temperature of Multnomah Channel for month of September 2008 (natural thermal potential this month).

3) Design average dry weather flow (ADWF) for STP = 1.515 mgd (taken from current permit, Schedule-A).

4) 7-day-average maximum weekly effluent temperature recorded for month of September 2008.

5) Notes:

a) Effluent temperature is measured at STP's UV disinfection system discharge.

b) Effluent is pumped from the STP to Multnomah Channel via the outfall pipe. Pumping is a discontinuous process; i.e. wet well fills until a preset level is reached and pumps come on. Pumps stop at preset level and process is repeated.

c) The outfall pipe is one mile long. It is reasonable to assume that considerable cooling occurs in the buried pipe prior to discharge.

Attachment - 2c

Facility Name: Scappoose STP **Date:** 4/22/2010

Enter data into white cells below:

7Q10 =	**62.9**	cfs	1)
Ambient Temperature or Criterion	**21.7**	°C	2)
Effluent Flow =	**1.515**	mgd	3)
Effluent Temperature	**21.8**	°C	4)
			5)

25% of 7Q10 = 15.7 cfs

25% dilution = 8 dilution = (Qe+Qr)/Qe

Temperature at 25% cross section = 21.71 °C Reasonable Potential

1) 10% of Willamette R. 7Q10 as measured at river mile 12.0 (Gage 14211720 @ Portland)
0.10(6290 cfs) = **629 cfs**. 7Q10 flow from DEQ Willamette Basin TMDL-Temperature, P. 4-131, Table 4.38.

1) 0.05% of Willamette R. 7Q10 as measured at river mile 12.0 (Gage 14211720 @ Portland)
0.05(6290 cfs) = **314.5 cfs**. 7Q10 flow from DEQ Willamette Basin TMDL-Temperature, P. 4-131, Table 4.38.

1) 0.01% of Willamette R. 7Q10 as measured at river mile 12.0 (Gage 14211720 @ Portland)
0.01(6290 cfs) = **62.9 cfs**. 7Q10 flow from DEQ Willamette Basin TMDL-Temperature, P. 4-131, Table 4.38.

2) 7-day-average maximum weekly temperature of Multnomah Channel for month of July 2009 (natural thermal potential this month).

3) Design average dry weather flow (ADWF) for STP = 1.515 mgd (taken from current permit, Schedule-A).

4) 7-day-average maximum weekly effluent temperature recorded for month of July 2009.

5) Notes:

   a) Effluent temperature is measured at STP's UV disinfection system discharge.
   b) Effluent is pumped from the STP to Multnomah Channel via the outfall pipe. Pumping is a discontinuous process; i.e. wet well fills until a preset level is reached and pumps come on. Pumps stop at preset level and process is repeated.
   c) The outfall pipe is one mile long. It is reasonable to assume that considerable cooling occurs in the buried pipe prior to discharge.

3. Scappoose - Effluent Mercury Data Summary

SCAPPOOSE - EFFLUENT MERCURY DATA SUMMARY

Effluent Sample Date Month/Yr	Measured Concen-tration (µg/L)	Lab MRL (µg/L)
27-Jul-09	ND	0.005
28-Apr-09	ND	0.005
1-Dec-08	ND	0.005
26-Aug-08	ND	0.005
21-Apr-08	ND	0.005
20-Dec-07	ND	0.005
15-Aug-07	ND	0.005
12-Jun-07	ND	0.005
13-Dec-06	ND	0.005
5-Apr-06	ND	0.005
7-Apr-06	ND	0.200
2-Aug-05	ND	0.200
3-Aug-05	ND	0.200

SCAPPOOSE - MULTNOMAH CHANNEL MERCURY DATA SUMMARY

Sample Date Month/Yr	Measured Concen-tration (µg/L)	Lab MRL (µg/L)
12-Jun-07	ND	0.005
13-Dec-06	ND	0.005
5-Apr-06	ND	0.005

CITY OF SCAPPOOSE
PERMIT MILESTONES

PERMIT DATA
Permit No: 100677
File No: 78980
Application No: 971614
Permit issued: March 18, 2005
Permit expired: December 31, 2009
Design average dry weather flow (DADWF) = **1.515 mgd** (current permit), & 1.58 mgd (from permit renewal application.).

LEGEND
Whole effluent toxicity (WET) testing nomenclature
- No observed effect concentration (NOEC).
- Lowest observed effect concentration (LOEC).
- Concentration of sample causing a 50% reduction in survival (LC_{50}).
- The concentration of sample causing a 25% reduction in biological measurement, e.g. growth (IC_{25} value).

Mixing zone (MZ) and zone of initial dilution (ZID) study data.

Requirements taken from the current permit.

Date	Action/Event
04Jan2010	Sewer Cleaning Report submitted for Year-2009. • 20,596 feet of sewer mainlines were jet cleaned and TV'd, and • 14,083 feet were flushed for total of 34,679 feet of sewer pipe.
01Dec2009	DEQ gave conditional approval to a Sewage Treatment Plant (STP) Improvements Plan. Project does the following. • Replace an existing comminuter at the headworks with a fine screen (0.25-inch mesh) rated at a design peak flow = 4 MGD. • Add disk filters in two new basins. Filters will be used during summer season to meet lower effluent limits. Two parallel installations of four disks each with a separate capacity of 2 MGD and a combined capacity of 4 MGD. Additional filter disks can be added to increase total capacity to 6 MGD. • Add new in-plant pump station (PS) to provide additional head for disk filter assemblies; PS will have two 2 MGD variable speed pumps. • Replace UV equipment ballasts and bulbs. • Replace sampling equipment. • Provide O&M manual updates and as-built plans.
13Nov2009	Gordon Munro, PE, of Kennedy Jenks Consultants submitted plans to DEQ for STP Improvement Project.
11Jun2009	Permit renewal application received (Form NPDES-R & Form 2A).

09Jun2009 CH2M-Hill Bioassay Report for whole effluent toxicity (WET) testing was received. Testing occurred 31Jul – 07Aug2007. Acute and chronic tests were completed at 6.25, 12.5, 25, 50, & 100% effluent strength. Schedule-D of permit does not specify effluent dilutions; only requires 4 WET tests, either quarterly in one year or over a 4.5 year period.

- Acute testing (*Ceriodaphnia dubia & Pimephales promelas*, i.e. flathead minnow) resulted in no statistically significant reduction in survival at any of the effluent concentrations tested when compared to the control.
- Chronic testing (*Ceriodaphnia dubia & Pimephales promelas*, i.e. flathead minnow) showed no statistically significant chronic response.

09Jun2009 CH2M-Hill Bioassay Report for whole effluent toxicity (WET) testing was received. Testing occurred 26Sep – 03Oct2006. Acute and chronic tests were completed at 6.25, 12.5, 25, 50, & 100% effluent strength. Schedule-D of permit does not specify effluent dilutions; only requires 4 WET tests, either quarterly in one year or over a 4.5 year period.

- Acute testing (*Ceriodaphnia dubia*) resulted in no statistically significant reduction in survival at any of the effluent concentrations tested when compared to the control.
- Chronic testing (flathead minnow) showed no statistically significant reduction in survival at any of the effluent concentrations tested and a statistically significant reduction in growth at the 25% effluent concentration compared to the control. NOEC and the LOEC were 100 & 25% effluent, respectively. ***This test was determined to be a false positive and NOEC was determined to be 100% sample.***
- Chronic testing (*Ceriodaphnia dubia*) showed no statistically significant reduction in survival or reproduction at any effluent concentration tested when compared to the control. The NOEC and LOEC were 100% and > 100% effluent concentration, respectively. Control survival was 90%. The IC_{25} value calculated on *Ceriodaphnia dubia* reproduction was > 100% effluent.

09Jun2009 CH2M-Hill Bioassay Report for whole effluent toxicity (WET) testing was received. Testing occurred 30Aug - 05Sep2005. Acute and chronic tests were completed at 6.25, 12.5, 25, 50, & 100% effluent strength. Schedule-D of permit does not specify effluent dilutions; only requires 4 WET tests, either quarterly in one year or over a 4.5 year period.

- Acute testing (*Ceriodaphnia dubia*) resulted in no statistically significant reduction in survival at any of the effluent concentrations tested when compared to the control.
- Chronic testing (flathead minnow) resulted in no statistically significant reduction in survival at any of the effluent concentrations tested when compared to the control.
- ***Chronic testing (Ceriodaphnia dubia) indicated a statistically significant reduction in reproduction at the 100% effluent concentration when compared to the control. The NOEC and LOEC were 25% and 50% effluent concentration, respectively. The IC_{25} value calculated on Ceriodaphnia dubia reproduction was 25% effluent.***

09Jun2009 CH2M-Hill Bioassay Report for whole effluent toxicity (WET) testing was received. Testing occurred 03Aug – 10Aug2004. Acute and chronic tests were completed at 6.25, 12.5, 25, 50, & 100% effluent strength. Schedule-D of permit does not specify effluent dilutions; only requires 4 WET tests, either quarterly in one year or over a 4.5 year period.

- Acute testing (*Ceriodaphnia dubia*) resulted in no statistically significant reduction in survival at any of the effluent concentrations tested when compared to the control.
- Chronic testing (flathead minnow) showed a statistically significant reduction in survival at the 6.25% effluent concentration when compared to the control (NOEC = 100% and LOEC = 6.25%). The IC_{25} value was > 100% effluent. ***This test was determined to be a false positive and NOEC was determined to be 100% sample. Fungus was noted in the 6.25% replicate.***
- Chronic testing (*Ceriodaphnia dubia*) ***indicated a statistically significant reduction in reproduction at the 50% and 100% effluent concentration when compared to the control. The NOEC and LOEC were 25% and 50% effluent concentration, respectively. Control survival was 90%. The IC_{25} value calculated on Ceriodaphnia dubia reproduction was 31.4% effluent.***

TABLE-1: WET Testing Summary.

Date	Acute Testing	Chronic Testing				
		Flathead minnow	Ceriodaphnia dubia	NOEC	LOEC	IC25
31Jul-07Aug2007	NSSR	NSSR	NSSR			
26Sep-03Oct2006	NSSR	NSSR	NSSR			
30Aug-05Sep2005	NSSR	NSSR	SSR @ 100%	25%	50%	25%
03Aug-10Aug2004	NSSR	NSSR	SSR @ 50% & 100%	25%	50%	31.4%

* NSSR = No statistically significant reduction in survival.

09Jun2009 Biosolids Management Plan dated February 2003 was received. City says this draft is still current. City applies biosolids on its own 28.4 acre property adjacent to the wastewater treatment facility.

03Apr2009 DEQ's Lab submitted DEQ MZ Study/Report for Scappoose's STP outfall.

- Field work was conducted on 15Sep2008.
- Outfall was discharging during the MZ study. Sampling was done only when an effluent discharge occurred.
- Sampling done on outgoing low tide.
- MZ defined in permit as that portion of Multnomah Channel contained within a band extending out 100 feet from the shore side of the outfall and 200 feet downstream and 200 feet upstream from the outfall. The zone of initial dilution (ZID) is defined as that portion of the allowable MZ that is within a 20 foot radius of the discharge point.
- Outfall is located at river mile (RM) 10.6 (measured from Columbia River upstream on Multnomah Channel).
- Purpose of study was to estimate available ZID & MZ dilutions.

- A Level-2 study was performed per DEQ's MZ internal management directive (IMD).
- Conductivity mapping was used to determine dilutions in the ZID & MZ. Background specific conductance of Multnomah Channel = 103 µmhos/cm during this test.
- Effluent specific conductance (measured at discharge before any dilution) = 492 µmhos/cm.
- Top of outfall pipe was submerged 8-inches below the water's surface during testing.
- Effluent discharge (measured on study day) = 0.609 MGD.
- Used mid-depth measurements throughout MZ for conductivity mapping.

....

Table-2: Conductivity Measurements (taken on 15Sep2008)

Location #	Distance to Outfall (Feet)	Specific Conductance (µmhos/cm)	Station Notes*
-	0	492	100% effluent
-	-	103	Multnomah Channel background
1	287	104	DS, Outside MZ (Dilution > 40)
2	218	105	DS Edge of MZ (Dilution > 40)
3	143	104	DS (Dilution > 40)
4	104	105	DS (Dilution > 40)
5	41	105	DS (Dilution > 40)
6	25	244	Edge of ZID (Dilution = 1.8)
7	25	378	Edge of ZID (Dilution = 0.35)
8	48	108	US (Dilution > 40)
9	97	105	US (Dilution > 40)
10	154	115	US (Dilution > 40)
11	253	105	US, Outside MZ (Dilution > 40)
12	327	105	US, Outside MZ (Dilution > 40)
13	9	480	Within ZID (Dilution < 0.10)
14	29	239	Edge of ZID (Dilution = 1.9)
15	45	105	Within MZ (Dilution > 40)

*Dilution Equation: [effluent conductance + N X (river background conductance)]/ (N+1) = Table of conductance values for effluent/river mixing.

Where: N= number of equal volumes of background river water added to one volume of effluent = dilution value.

Table-2 Notes:

1. Pump Cycles and Effluent Discharge. The outfall discharge is batched; i.e. the pump station (PS) runs periodically to force effluent to the river through a 1-mile long forcemain. Each PS discharge creates turbulence/mixing in the ZID & MZ, especially when pumping begins. When the PS switches off (rests) discharged effluent tends to exit the ZID & MZ, as in-channel mixing proceeds. The PS batch-type discharge is thought to enhance mixing and reduce over-all effluent toxicity (acute and chronic).

2. WET Testing Discussion. Acute and chronic whole effluent toxicity (WET) tests are steady-state tests that are done in a lab (see Table-1). WET tests for Scappoose probably overstate the actual effluent toxicity for the batch discharges discussed above. ***On the MZ sampling day above (15Sep2008), effluent pumps ran a total of approximately 5 hours (DEQ calculation based on pump performance measured on 29Mar2010). This means that the PS operated about 21% of the time. As a consequence, ZID & MZ dilutions vary throughout the day, as the actual effluent concentration in the ZID and MZ falls between pump cycles. These non-steady-state discharge conditions suggest that WET tests are more stringent than actual MZ conditions warrant when evaluating acute and chronic effluent toxicity.***

3. STP Upgrades Likely To Improve Effluent Quality. On 01Dec2009 (see above) DEQ approved plans for STP upgrades including the following:
   - Add disk filters in two new basins;
   - Disk filters will be used during summer season to meet lower effluent limits;
   - Two parallel installations of four disks, each with a separate capacity of 2 MGD and a combined capacity of 4 MGD; and
   - Additional filter disks can be added to increase total capacity to 6 MGD.

   The disk filter addition above will help remove fine effluent particles prior to effluent discharge. Filtration will reduce TSS and BOD_5 concentrations. Increased TSS removal should lower effluent total metals concentrations. Effluent filtration will reduce over-all effluent toxicity and BOD_5 oxygen demand.

4. Table-2 dilution values (given non-steady-state discharge conditions).
   - MZ dilution is > 40 (both upstream and downstream at MZ boundary),
   - ZID dilution is approximately ≈ 1.2 at ZID boundary [average conductance = (244+378+239)/3 = 287], and
   - MZ dilution is > 40 at 50-feet from bank (measured on a line that starts at the outfall discharge point and runs perpendicular to the channel bank).

....

- Environmental mapping identified critical habitats, resources, and beneficial uses:
  - Boating, fishing, and swimming are common near the outfall,
  - Multnomah Channel is a salmon and steelhead migration corridor, and
  - The channel is tidally influenced.
- Multnomah Channel (RM 0-21.7) is listed as being water quality limited (WQL) for temperature year-around. The applicable criterion is the 7-day-average maximum temperature (18.0 °C (64.4 °F)) of a stream identified as having salmon and trout rearing and migration use [OAR 340-041-0028 (4) (c)].
- DEQ's MZ study at Scappoose was conducted during the low-flow, summer season at low tide to mirror worst-case conditions.

TABLE-3: Water Quality Sampling Summary (15Sep2008 DEQ MZ Study)

Parameter	Units	Effluent	Channel US of Outfall 001	MZ Boundary 200 ft DS	25 ft DS of Outfall @ ZID Edge
pH	s.u.	7.1	7.5	7.5	7.3
Specific Conductance	µmhos/cm	492	103	104	370
Dissolved Oxy	mg/L	5.7	7.0	8.0	8.4
DO Saturation	%	60	75	86	92
Temperature	°C	19.6	19.4	19.4	20.4
Turbidity	NTU	2	6	6	9
E. coli	MPN/100 mL	1	28	13	47
Alkalinity	mg/L	53	28	28	41
BOD_5	mg/L	0.8	void	0.4	0.8
Ammonia as N	mg/L	< 0.02	0.06	0.06	0.05
Nitrite/nitrate as N	mg/L	23.0	0.291	0.306	11.2
Total suspended solids	mg/L	2	8	14	41

TABLE-4: Summary of Priority Pollutant Metals Testing (15Sep2008 DEQ MZ Study)

Parameter	Units	Effluent	Channel US of Outfall 001	MZ Boundary 200 ft DS	25 ft DS of Outfall @ ZID Edge
Barium	µg/L	7.4	8.5	9.4	25.5
Chromium	µg/L	< 1.0	3.5	< 1.0	1.9
Cobalt	µg/L	< 0.20	0.23	0.30	1.23
Copper	**µg/L**	**9.2**	**< 1.5**	**< 1.5**	**8.1**
Lead	µg/L	< 0.20	4.15	0.23	1.09
Nickel	**µg/L**	**1.2**	**< 1.0**	**< 1.0**	**2.5**
Vanadium	µg/L	< 4.0	4.0	4.3	7.7
Zinc	**µg/L**	**41.3**	**4.3**	**< 3.0**	**31.2**

- Metals & toxics data were taken during this MZ study. Organics were estimates. Metals found at level above Channel background = boron, calcium, magnesium, potassium, silicon, sodium, copper, nickel, and zinc.
- ***Cu detected in effluent at levels above both the acute & chronic WQ criteria.***
- ***Cu exceeded the acute WQ criterion at edge of ZID.***
- ***Cu was not detected at edge of MZ.***
- High values shown for edge of ZID above are thought to result from associated sediments at this location; i.e. the high level of suspended solids present in samples taken at this location.

Proposed Metals & BOD_5 Testing in Renewal Permit. The proposed permit should require

- Continued WET testing.
- Continued testing for copper (Cu). One half of the Cu tests should be conducted before filter installation and the remainder after. Filtration should reduce total Cu in effluent discharges by reducing total suspended solids.
- May need to investigate lengthening the outfall pipe to clear current mud/sandbar, and increase minimum water depth over the outfall pipe.
- STP staff has indicated that they can increase the volume of each effluent discharge by perhaps 30,000 gallons based on the storage volume of the old, unused chlorine contact basin. An increase in the batch discharge volume will consequently reduce the discharge frequency. This should improve both ZID and MZ mixing.

11Mar2009 DEQ lab submitted WQ data for City of Scappoose STP MZ Evaluation that occurred on 15Sep2008.

02Mar2009 DEQ completed a predesign report review for Springdale Pump Station. Proposes chopper-style, 5 HP Vaughan suction-lift, duplex pumps rated at 150 gpm at 24 feet TDH; and a 500-foot long, 4-inch diameter PVC forcemain.

11Feb2009 Sewer cleaning report for Year 2008: Repair 11 manholes, clean & TV (1,850 + 8,155) feet of sanitary sewer, and treat 977 feet of sanitary sewer for root intrusion.

28Jan2009 DEQ received City's Temperature & Dissolved Oxygen (DO) Evaluation Study Report.

- DEQ approved study results on 29Jan2009.
- Study was prepared by Kennedy/Jenks Consultants in response to DEQ's letter dated 08Aug2007.
- Study measured temperature and dissolved oxygen in vicinity of outfall to investigate potential effluent effects on in-stream DO and cold water refugia.
- Study was conducted during summer low-flow in Multnomah Channel.
- Study concluded that maximum DO sag caused by STP effluent = 0.02 mg/L.
- Cold water refugia areas were not located within the MZ.
- ***Used estimated Multnomah Channel 7Q10 low flow = 17.8 m^3/sec; which is 10% of summer low flow for the Willamette River per Willamette Basin TMDL.***

15Sep2008 DEQ conducted a MZ study for the Scappoose STP outfall.

31Jul2008 DEQ's review letter to City concerning its Temperature & Dissolved Oxygen Evaluation Work Plan prepared by Kennedy/Jenks. DEQ listed/verified WQ criteria to be used in completing the study.

24Jul2008 Kennedy/Jenks Consultants submitted City's Temperature & Dissolved Oxygen Evaluation Work Plan for DEQ review.

29Jan2008 City submitted its Years-2007 & 2008 Sewer Cleaning Report.

- Clean & TV sanitary sewers = 1,400 + 17,200 + 2,699 feet.

11Jan2008 DEQ reviewed and approved Kenney/Jenks revised Mixing Zone Study Report dated 07Dec2007. The parameters under investigation

- Temperature (allowed increase per TMDL waste load allocation),
- Ammonia toxicity (acute & chronic), and
- Allowed dissolved oxygen (DO) reduction in the receiving stream.

DEQ concluded that MZ study report shows that... *no outfall or facility modifications are required at this time to address the parameters listed above.* DEQ attached its own independent review of the above parameters to this approval letter.

24Dec2007 Kennedy/Jenks letter concerning Mixing Zone Study. City will use revised Multnomah Channel low-flow for revised study, per DEQ's letter dated 20Aug2007.

11Dec2007 The City submitted a revised Mixing Zone Study Report.

07Dec2007 Kenney/Jenks submitted a revised Mixing Zone Study Report to DEQ. The study determined that worst-case summer season low-flow dilutions for ZID and MZ are

- ZID dilution = 4
- MZ dilution = 14

30Oct2007 DEQ inspection report for STP. Inspection was completed on 26Sep2007. No faults noted.

24Sep2007 DEQ approved Elm Street Subdivision sanitary sewer plans.

29Aug2007 A predesign report for proposed STP improvements was approved by DEQ. DEQ listed WQ criteria and assumptions to be used to complete the study.

20Aug2007 DEQ sent City a letter concerning MZ study parameters.

- DEQ's 7Q10 low flow for Multnomah Channel (Thermal Calculator run) = 8,537 cfs.
- Draft TMDL sets temperature [migration corridor, OAR 340-041-0028 (4) (d)] for Lower Columbia = 20 °C.
- Draft TMDL "bubble allocation" for a Scappoose-like source = 0.0081 °C after complete mix with 25% of Multnomah Channel 7Q10 flow.
- Until the temperature TMDL is finalized, the allowable temperature increase for Scappoose is 0.3 °C after complete mix with 25% of the 7Q10 flow or with the temperature MZ, whichever is more stringent, per OAR 340-041-0028 (12) (a) (A).
- City was asked to provide MZ dilution update, and to study DO reduction and the existence of cold-water refugia.

29May2007 City letter to DEQ stating that emergency power upgrade at STP is complete, per Schedule C, Condition 2 of the permit.

20Mar2007 DEQ granted City an exemption for gravity sanitary sewer plan review, per OAR-340-052-045.

29Jan2007 City's Sewer Cleaning Report for Year-2006.

- Cleaned & TV'd 2,987 feet of 8-inch and 716 feet of 12-inch sanitary sewer.
- Cleaned & TV's 18,000 feet of sanitary sewer pipe.

Year-2007 Revised MZ study for STP outfall at RM 10.6.

15Feb2006 City's Sewer Cleaning Report for Year-2005.
- Clean 27,560 feet of sanitary sewer and of this amount TV 16,400 feet.
- Clean 5 pump stations and treatment plant wetwell.

28Dec2005 DEQ approved Springlake Meadows-2 Subdivision's sanitary sewer plans.

22Mar2005 DEQ review letter for draft predesign report for Headworks & Filtration Improvements at STP.

18Mar2005 Current permit was issued. The current permit requires the following:

1. Schedule-B, annual Biosolids Report (for previous year) due each year by February 19 (if land application occurred).
2. Schedule-C, effluent Hg testing requirements:
- Begin Hg testing within 6-months of permit issuance;
- Do a minimum of 8 quarterly, total mercury tests;
- Use 24-hour daily composite samples; and
- All tests per EPA Method 1631, ultra-clean.
3. Schedule-C, Multnomah Channel Hg testing:
- Begin Hg testing within 6-months of permit issuance, and complete testing within 18 months of permit issuance;
- Provide 2 Hg tests using grab samples (one winter & one summer);
- All tests per EPA Method 1631, ultra-clean.
4. Schedule-C, worst-case dilutions are listed as follows:
- **ZID = 4**, and
- **MZ = 23.**
5. Schedule-C, Sewer Cleaning Report is required each year by February 19 for past year. Each year permittee must TV inspect & clean ≥ 20% of its sanitary sewer system.
6. Schedule-C, Install emergency power at treatment facility within 12-months of permit issuance.
7. Schedule-D, biosolids site next to treatment facility is 35 acres with 28.1 acres available for application.
8. Schedule-D, testing required for next permit renewal = testing listed in Parts D & E of EPA Form 3510-2A (Rev. 1-99).
- All testing per 40 CFR Part 136.
- Part-D testing for metals, hardness, and 85 pollutants: 3 scans during 4.5 years after permit issuance with scan between 4 & 8 months apart, 24-hour composite samples for non-volatiles, and mercury tests from Schedule-C can be used to meet this goal.
9. Part-E whole effluent toxicity (WET) testing: for acute and chronic toxicity, and 4 tests performed at least annually in the 4.5 years prior to permit renewal application.

Year-2004 MZ study completed for STP outfall.

GLS: Scappoose Permitmilestones 16Mar2010.docx
Revised: 20May2010

5-1/8

SCAPPOOSE - EFFLUENT METALS DATA SUMMARY

(All samples are grab samples taken at discharge to UV disinfection unit)

Effluent Sample Date Month/Yr	Metal	Measured Concen-tration (µg/L)	Lab MRL (µg/L)		Measured Concen-tration (mg/L)	Lab MRL (mg/L)		Effluent Hardness CaCO3 (mg/L)	Lab Hardness MRL (mg/L)	Effluent Alkalinity CaCO3 (mg/L)
15-Sep-08	Calcium				28.9	0.1		94.7	0.70	53
	Magnesium				5.48	0.1				
	Antimony	< 2.0	2.0							
	Arsenic	< 2.0	2.0							
	Beryllium	< 0.30	0.30							
	Cadmium	< 0.30	0.30							
	Chromium	< 1.0	1.0							
	Copper	**9.2**	1.5							
	Lead	< 0.20	0.20							
	Nickel	**1.2**	1.0							
	Selenium	< 2.0	2.0							
	Silver	< 0.10	0.10							
	Thallium	< 0.10	0.10							
	Zinc	**41.3**	3.0							
	Cyanide	*	*							
	Phenolics	*	*							
	Bis(2-ethylhexyl)phthalate	< 0.500	0.500							
12-Jun-07	Calcium	*	*		29.9	0.100		102	0.662	
	Magnesium	*	*		6.73	0.100				
	Antimony	ND	1.00							
	Arsenic	ND	1.00							
	Beryllium	ND	1.00							

5-2/8

	Cadmium	ND	1.00							
	Chromium	ND	1.00							
	Copper	**11.4**	2.00							
	Lead	ND	1.00							
	Nickel	ND	2.00							
	Selenium	ND	2.00							
	Silver	ND	1.00							
	Thallium	ND	1.00							
	Zinc	**36.2**	5.00							
	Cyanide	ND	5.00							
	Phenolics	ND	*		0.100					
	Bis(2-ethylhexyl)phthalate	ND	9.52							
7-Apr-06	Calcium	*	*		25.3	0.100		83.4	0.662	
	Magnesium	*	*		4.90	0.100				
	Antimony	ND	1.00							
	Arsenic	ND	1.00							
	Beryllium	ND	1.00							
	Cadmium	ND	1.00							
	Chromium	ND	1.00							
	Copper	**7.7**	2.00							
	Lead	ND	1.00							
	Nickel	ND	2.00							
	Selenium	ND	2.00							
	Silver	ND	1.00							
	Thallium	ND	1.00							
	Zinc	**34.9**	5.00							
	Cyanide	ND	5.00							
	Phenolics	ND	*		0.100					
	Bis(2-ethylhexyl)phthalate	**31.9**	9.62							

5-3/8

2-Aug-05	Calcium	*	*		20.4	0.250		82.7	1.00	
	Magnesium	*	*		4.43	0.500				
	Antimony	ND	3.00							
	Arsenic	ND	1.00							
	Beryllium	ND	1.00							
	Cadmium	ND	1.00							
	Chromium	ND	1.00							
	Copper	**16.1**	1.00							
	Lead	ND	1.00							
	Nickel	**2.66**	1.00							
	Selenium	ND	1.00							
	Silver	ND	1.00							
	Thallium	ND	1.00							
	Zinc	47.2	10.0							
	Cyanide	ND	5.00							
	Phenolics	ND	*		0.100					
	Bis(2-ethylhexyl)phthalate	**12.0**	10.0							

EFFLUENT METALS TOXICITY SUMMARY

Metal	**Sample Location Relative to Outfall**	**Concentration µg/L**	**MRL µg/L**	**Comments**
	UV Disinfection Unit Outlet			
COPPER	SAMPLE COUNT =	4	*	Freshwater acute criterion = 18 µg/L.
	MAXIMUM =	16.1	1.5	Freshwater chronic criterion = 12 µg/L.
	MINIMUM =	7.7	1.5	***Conclusion: no reasonable potential for***
	AVERAGE =	11.1	1.5	***efffluent Cu toxicity.***

5-4/8

ZINC	SAMPLE COUNT =	4	*	Freshwater acute criterion = 120 µg/L.			
	MAXIMUM =	47.2	1.5	Freshwater chronic criterion = 110 µg/L.			
	MINIMUM =	34.9	1.5	***Conclusion: no reasonable potential for***			
	AVERAGE =	39.9	1.5	***effluent Zn toxicity.***			

5-5/8

SCAPPOOSE - MULTNOMAH CHANNEL METALS DATA SUMMARY (DEQ LAB REPORT FOR OUTFALL)									
(All samples are grab samples taken when effluent is activily being discharged to Multnomah Channel)									
Sample taken upstream (US) of RMZ (Sample 002)									
		Total Recoverable							
		Measured			Channel	Lab	Channel		
Sample		Concen-	Lab		Hardness	Hardness	Alkalinity	Channel	Ammonia
Date	Metal	tration	MRL		CaCO3	MRL	CaCO3	pH	as N
Month/Yr	(total recoverable)	(µg/L)	(µg/L)		(mg/L)	(mg/L)	(mg/L)	s.u.	mg/L
15-Sep-08	Calcium	**6750**	100		27.0	0.70	28	7.5	0.06
	Magnesium	**2470**	100						
	Antimony	< 2.0	2.0						
	Arsenic	< 2.0	2.0						
	Barium	**8.5**	2.0						
	Beryllium	< 3.0	0.30						
	Cadmium	< 3.0	0.30						
	Chromium	**3.5**	1.0						
	Cobalt	**0.23**	0.2						
	Copper	< 1.5	1.5						
	Lead	**4.15**	0.20						
	Molybdenum	< 3.0	0.30						
	Nickel	< 1.0	0.10						
	Selenium	< 2.0	2.0						
	Silver	< 1.0	0.10						
	Thallium	< 1.0	0.10						
	Zinc	**4.3**	3.0						
Sample taken 200 ft DS of Scappoose Outfall (edge of RMZ, Sample 003)									
15-Sep-08	Calcium	**6780**	100		27.4	0.70	28	7.5	0.06
	Magnesium	**2530**	100						
	Antimony	< 2.0	2.0						
	Arsenic	< 2.0	2.0						

5 - 6/8

	Barium	**9.4**	2.0						
	Beryllium	< 3.0	0.30						
	Cadmium	< 3.0	0.30						
	Chromium	< 1.0	1.0						
	Cobalt	**0.30**	0.2						
	Copper	**< 1.5**	1.5						
	Lead	**0.23**	0.20						
	Molybdenum	< 3.0	0.30						
	Nickel	< 1.0	0.10						
	Selenium	< 2.0	2.0						
	Silver	< 1.0	0.10						
	Thallium	< 1.0	0.10						
	Zinc	< 3.0	3.0						
Sample taken 25 ft DS of Scappoose Outfall (within RMZ, Sample 004)					69.5	0.7	41	7.3	0.05
15-Sep-08	Calcium	**20200**	100						
	Magnesium	**4640**	100						
	Antimony	< 2.0	2.0						
	Arsenic	< 2.0	2.0						
	Barium	**25.5**	2.0						
	Beryllium	< 3.0	0.30						
	Cadmium	< 3.0	0.30						
	Chromium	**1.9**	1.0						
	Cobalt	**1.23**	0.2						
	Copper	**8.1**	1.5						
	Lead	**1.09**	0.20						
	Molybdenum	< 3.0	0.30						
	Nickel	**2.5**	0.10						
	Selenium	< 2.0	2.0						
	Silver	< 1.0	0.10						
	Thallium	< 1.0	0.10						
	Zinc	**31.2**	3.0						

5-7/8

Sample taken 200 ft DS of Scappoose Outfall (RMZ edge, Sample 005)					27.2	0.7	28	7.5	0.05
15-Sep-08	Calcium	**6730**	100						
	Magnesium	**2520**	100						
	Antimony	< 2.0	2.0						
	Arsenic	< 2.0	2.0						
	Barium	**11.3**	2.0						
	Beryllium	< 3.0	0.30						
	Cadmium	< 3.0	0.30						
	Chromium	< 1.0	1.0						
	Cobalt	**0.35**	0.2						
	Copper	**1.60**	1.5						
	Lead	**0.41**	0.20						
	Molybdenum	< 3.0	0.30						
	Nickel	< 1.0	0.10						
	Selenium	< 2.0	2.0						
	Silver	< 1.0	0.10						
	Thallium	< 1.0	0.10						
	Zinc	**3.9**	3.0						

INSTREAM METALS TOXICITY SUMMARY

Metal	Sample Location Relative to Outfall	Concentration µg/L	MRL µg/L	Comments
Barium	US	**8.5**	2.0	All in-stream values exceed effluent concentration.
	200 feet DS	**9.4**	2.0	Suspended instream sediment appears to be source.
	25 feet DS	**25.5**	2.0	***Conclusion: no reasonable potential for***
	200 feet DS	**11.3**	2.0	***efffluent Ba toxicity.***
	UV Disinfection Unit Outlet	**7.4**	2.0	
Chromium	US	**3.5**	1.0	All in-stream values exceed effluent concentration.
	200 feet DS	< 1.0	1.0	Suspended instream sediment appears to be source.
	25 feet DS	**1.9**	1.0	***Conclusion: no reasonable potential for***

5- 8/8

	200 feet DS	< 1.0	1.0	***effluent Cr toxicity.***
	UV Disinfection Unit Outlet	< 1.0	1.0	
Copper	US	< 1.5	1.5	Effluent appears to be Cu source.
	200 feet DS	< 1.5	1.5	Freshwater acute criterion = 18 µg/L.
	25 feet DS	**8.1**	1.5	Freshwater chronic criterion = 12 µg/L.
	200 feet DS	**1.6**	1.5	***Conclusion: no reasonable potential for***
	UV Disinfection Unit Outlet	**9.2**	1.5	***efffluent Cu toxicity.***
Lead	US	**4.15**	0.20	All in-stream values exceed effluent concentration.
	200 feet DS	**0.23**	0.20	Suspended instream sediment appears to be source.
	25 feet DS	**1.09**	0.20	***Conclusion: no reasonable potential for***
	200 feet DS	**0.41**	0.20	***effluent Pb toxicity.***
	UV Disinfection Unit Outlet	< 0.20	0.20	
Zinc	US	**4.3**	3.0	Effluent appears to be Zn source.
	200 feet DS	< 3.0	3.0	Freshwater acute criterion = 120 µg/L.
	25 feet DS	**31.2**	3.0	Freshwater chronic criterion = 110 µg/L.
	200 feet DS	**3.9**	3.0	***Conclusion: no reasonable potential for***
	UV Disinfection Unit Outlet	**41.3**	3.0	***effluent Zn toxicity.***

6. Scappoose - STP MZ pH Study

Calculation of pH of a mixture of two flows.
Based on the procedure in EPA's DESCON program (EPA, 1988. Technical Guidance on Supplementary Stream Design Conditions for Steady State Modeling. USEPA Office of Water, Washington D.C.)

Scappoose STP MZ pH Study	RPA for pH		
INPUT	Lower pH	Upper pH	
	Criteria	Criteria	
1. DILUTION FACTOR AT MZ BOUNDARY - (Qe+Qr)/Qe	5	5	DEQ MZ Study *
2. UPSTREAM/BACKGROUND CHARACTERISTICS			
Temperature (deg C):	25.0	25.0	DMR temp max
pH:	7.5	7.5	DEQ MZ Study
Alkalinity (mg CaCO3/L):	28.0	28.0	DEQ MZ Study
3. EFFLUENT CHARACTERISTICS			
Temperature (deg C):	24.0	24.0	DMR temp max
pH:	6.4	8.2	DMR pH max/min
Alkalinity (mg CaCO3/L):	53.0	53.0	DEQ MZ Study
4. APPLICABLE PH CRITERIA	6.0	9.0	Current permit
OUTPUT			
1. IONIZATION CONSTANTS			
Upstream/Background pKa:	6.35	6.35	
Effluent pKa:	6.36	6.36	
2. IONIZATION FRACTIONS			
Upstream/Background Ionization Fraction:	0.93	0.93	
Effluent Ionization Fraction:	0.53	0.99	
3. TOTAL INORGANIC CARBON			
Upstream/Background Total Inorganic Carbon (mg CaC	29.98	29.98	
Effluent Total Inorganic Carbon (mg CaCO3/L):	100.88	53.76	
4. CONDITIONS AT MIXING ZONE BOUNDARY			
Temperature (deg C):	24.79	24.79	
Alkalinity (mg CaCO3/L):	33.32	33.32	
Total Inorganic Carbon (mg CaCO3/L):	45.07	35.04	
pKa:	6.35	6.35	
pH at Mixing Zone Boundary:	**6.8**	**7.6**	
Is there Reasonable Potential?	**No**	**No**	

* DILUTION = 5 IS VERY CONSERVATIVE. DEQ MZ STUDY FOUND DILUTION > 40 @ MZ BOUNDARY.

7. Freshwater Ammonia Criteria Calcul: Scappoose 20-May-10

Dilution Values? (Y/N)	Y	calculated
Dilution @ ZID =	1.2	*
Dilution @ MZ =	40	*
Enter data below if no dilution data is available		
Data to estimate dilution		
7Q10 (CFS) =	*	
1Q10 (CFS) =	*	
% dilution at MZ =	25	
% dilution at ZID =	10	
Effluent Flow (mgd) =	1	

*** Notes:**
pH must be between 6.5 and 9.0
Temperature must be between 0 C and 30 C

Water Chemistry	Effluent	Stream	Mixed		
			ZID	MZ	
pH * =	7.1	7.5	7.1	7.5	(6.5-9)
Temp * =	19.6	19.4	19.6	19.4	° C
Alkalinity =	53	28			
Salmonids Present? (Y/N)		y			

WQ CRITERIA	
1 Hour	4 Day
(CMC)	(CCC)
mg/l	mg/l
17.66	**1.29**

al	cl	la	lc	Apal	Apcl
20	15	2.30	1.48	1.03	1.41

Calculation of pH of a mixture of two flows.
Based on the procedure in EPA's DESCON program (EPA, 1988. Technical
Guidance on Supplementary Stream Design Conditions for Steady State

7-1/2

Facility Name: <u>Scappoose</u> **Date:** 5/20/2010

Dilution Values? (Y/N)	Y	calculated
w Flow Dilution @ ZID (1Q10)	1.2	*
ow Flow Dilution @ MZ (7Q10)	40	*
ow Flow Dilution @ MZ (30Q5)	40	*
h Flow Dilution @ ZID (1Q10)	1.2	*
gh Flow Dilution @ MZ (7Q10)	40	*
gh Flow Dilution @ MZ (30Q5)	40	*
If no dilution values Enter flow rates here	Summer	Winter
Effluent Flow (MGD)	*	*
1Q10 (CFS)	*	*
7Q10 (CFS)	*	*
30Q5 (CFS)	*	*
% dilution at MZ	*	*
% dilution at ZID	*	*
probability basis (WLA multipliers)	99%	

Summer data	Effluent	Stream	Mixed			
			ZID	MZ	MZ	
			1Q10	7Q10	30Q5	
pH * =	7.1	7.5	7.1	7.5	7.5	(6.5-9)
Temp * =	19.6	19.4	19.6	19.4	19.4	° C
Alkalinity =	53	28				
Salmonids Present? (Y/N)	n/a	Y				
Salmonid Spawning? (Y/N)	n/a	N				
Fresh Water ? (Y/N)	n/a	Y				
Salinity	*	*	*	*		
Winter data						
pH * =	7	7	7.0	7.0	7.0	(6.5-9)
Temp * =	18	18	18.0	18.0	18.0	° C
Alkalinity =	53	28				
Salmonids Present? (Y/N)	n/a	Y				
Salmonid Sapwning (Y/N)	n/a	N				
Fresh Water ? (Y/N)	n/a	Y				
Salinity	*	*	*	*		

	WATER QUALITY CRITERIA				Allocations									Concentration Limits	
PARAMETER	1 Hour (CMC)	4 Day (CCC)	30 Day (CCC)	Back-ground	Acute	4 Day	30 Day	CV	# Samples	Acute LTA	4 day LTA	30 day LTA	Min LTA	95% Monthly	99% Daily
	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/L		/Mo	mg/l	mg/l	mg/L	mg/l	mg/l	mg/l
Low Flow Season															
CHLORINE	0.019	0.011	n/a	*	*	*	n/a	*	*	*	*	n/a	*	*	*
AMMONIA - Freshwater	17.7	1.3	n/a	0.06	21.18	49.31	n/a	1.432	228	3.17	13.58	n/a	3.17	**3.7**	**21.2**
AMMONIA - Saltwater	n/a	n/a	n/a	*	*	*	n/a	*	*	*	*	n/a	*	*	*
AMMONIA - Proposed	21.4	8.2	3.3	*	*	*	*	*	*	*	*	*	*	*	*
High Flow Season															
CHLORINE	0.019	0.011	n/a	*	*	*	n/a	*	*	*	*	n/a	*	*	*
AMMONIA - Freshwater	19.5	1.4	n/a	0.06	23.34	54.51	n/a	1.432	228	3.49	15.01	n/a	3.49	**4.1**	**23.3**
AMMONIA - Saltwater	n/a	n/a	n/a	*	*	*	n/a	*	*	*	*	n/a	*	*	*
AMMONIA - Proposed	24.1	11.8	4.7	*	*	*	*	*	*	*	*	*	*	*	*

NOTES :

Temperature must be between 0 and 30 ° C
pH must be between 6.5 and 9
Ammonia is mg/l ammonia as N.

7-2/2

C-3:
Oregon 2012 Integrated Report

Oregon Department of Environmental Quality

Oregon DEQ: Water Quality - Water Quality Assessment - Oregon's 2012 Integrated Report Database

11/18/2015 8:43:28 AM (Page 1 of 1)

Oregon's 2012 Integrated Report

To select new search criteria click here - DO NOT USE THE BACK ARROW

Refresh Report Show All Records Records per page: 100

Lookup LASAR Station data Link to LASAR Web

Basin Name Subbasin 4th Field HUC Record ID	Water Body LLID River Miles Segment Miles Beach Name Beach ID	Pollutant	Season	Criteria	Beneficial Uses	Status	2012 Assessment Action	[Data Source] Supporting Data
Willamette Lower Willamette 17090012 16569	Multnomah Channel 1227863458618 0 to 21.7 21.7	Alkalinity	Year Round	Table 20 Toxic Substances	Aquatic life	Cat 3: Insufficient data	No action	Previous Data: [DEQ] LASAR 30756 River Mile 21: From 9/9/2003 to 9/9/2003, 0 out of 1 samples < 20 mg/L (Table 20 criterion). [DEQ] LASAR 30757 River Mile 18.7: From 9/9/2003 to 9/9/2003, 0 out of 1 samples < 20 mg/L (Table 20 criterion). Previous Status: Cat 3: Insufficient data Previous Action: Added to database

								Previous Assessment Year: 2004
Willamette Lower Willamette 17090012 16570	Multnomah Channel 1227863458618 0 to 21.7 21.7	Ammonia	Year Round	Table 20 Toxic Substances	Aquatic life	Cat 3: Insufficient data	No action	Previous Data: [DEQ] LASAR 30757 River Mile 18.7: From 9/9/2003 to 9/9/2003, 0 out of 1 samples > applicable Table 20 criterion. [DEQ] LASAR 12935 River Mile 18.6: From 8/5/1997 to 8/5/1997, 0 out of 1 samples > applicable Table 20 criterion. [DEQ] LASAR 12936 River Mile 14.3: From 8/5/1997 to 8/5/1997, 0 out of 1 samples > applicable Table 20 criterion. [DEQ] LASAR 12937 River Mile 14.2: From 8/5/1997 to 8/5/1997, 0 out of 1 samples > applicable Table 20 criterion. [DEQ] LASAR 12938 River Mile 18.6: From 8/5/1997 to 8/5/1997, 0 out of 1 samples > applicable Table 20 criterion. [DEQ] LASAR 30756 River Mile 21: From 9/9/2003 to 9/9/2003, 0 out of 1

								samples > applicable Table 20 criterion. Previous Status: Cat 3: Insufficient data Previous Action: Added to database Previous Assessment Year: 2004
Willamette Lower Willamette 17090012 25229	Multnomah Channel 1227863458618 0 to 21.7 21.7	Dissolved Oxygen	January 1 - May 15	Spawning: Not less than 11.0 mg/L or 95% of saturation		Cat 5: Water quality limited, 303(d) list, TMDL needed	Added to database	2012 Data: [DEQ] STATION 36160 at RM 6.4 from 03/14/2006 to 05/09/2011, 4 of 11 (36%) samples < 11.0 mg/l and < 95% saturation. [DEQ] STATION 33748 at RM 14.7 from 03/28/2006 to 04/14/2011, 1 of 9 (11%) samples < 11.0 mg/l and < 95% saturation
Willamette Lower Willamette 17090012 12075	Multnomah Channel 1227863458618 0 to 21.7 21.7	Dissolved Oxygen	Year Round (Non-spawning)	Cold water: Not less than 8.0 mg/l or 90% of saturation	Cold-water aquatic life	Cat 2: Attaining some criteria/uses	Status modification - Attaining criteria/uses	2012 Data: [DEQ] STATION 36160 at RM 6.4 from 06/02/2006 to 12/12/2011, 0 of 22 (0%) samples < 6.5 mg/L. [DEQ] STATION 35110 at RM 10.6 from 09/15/2008 to 09/15/2008, 0 of 1 (0%) samples <

6.5 mg/L. [DEQ] STATION 33748 at RM 14.7 from 05/21/2006 to 10/18/2011, 1 of 20 (5%) samples < 6.5 mg/L. [DEQ] STATION 30757 at RM 18.7 from 09/09/2003 to 09/09/2003, 0 of 1 (0%) samples < 6.5 mg/L. [DEQ] STATION 30756 at RM 21 from 09/09/2003 to 09/09/2003, 0 of 1 (0%) samples < 6.5 mg/L

Previous Data: [DEQ] LASAR 12935 River Mile 18.6: From 8/5/1997 to 8/5/1997, 0 out of 1 samples (0%) < 8 mg/l and applicable % saturation. [DEQ] LASAR 12938 River Mile 18.6: From 8/5/1997 to 8/5/1997, 0 out of 1 samples (0%) < 8 mg/l and applicable % saturation. Previous Status: Cat 3: Insufficient data Previous Action: Added to database Previous Assessment Year: 2004

Willamette Lower Willamette 17090012 16571	Multnomah Channel 1227863458618 0 to 21.7 21.7	E. Coli	Summer	30-day log mean of 126 E. coli organisms per 100 ml; no single sample > 406 organisms per 100 ml	Water contact recreation	Cat 3: Insufficient data	No action	Previous Data: [DEQ] LASAR 12938 River Mile 18.6: From 8/5/1997 to 8/5/1997, 0 out of 1 samples (0%) > 406 organisms; maximum 30-day log mean of 0 [DEQ] LASAR 12937 River Mile 14.2: From 8/5/1997 to 8/5/1997, 0 out of 1 samples (0%) > 406 organisms; maximum 30-day log mean of 0 [DEQ] LASAR 12935 River Mile 18.6: From 8/5/1997 to 8/5/1997, 0 out of 1 samples (0%) > 406 organisms; maximum 30-day log mean of 0 [DEQ] LASAR 12936 River Mile 14.3: From 8/5/1997 to 8/5/1997, 0 out of 1 samples (0%) > 406 organisms; maximum 30-day log mean of 0 Previous Status: Cat 3: Insufficient data Previous Action: Added to database Previous Assessment Year: 2004
Willamette	Multnomah	Mercury	Year	Table 40	Human	Cat 5:	Added to	

Lower Willamette 17090012 25228	Channel 1227863458618 0 to 21.7 21.7		Round	Human Health Criteria for Toxic Pollutants	health	Water quality limited, 303(d) list, TMDL needed	database	2012 Data: [ODEQ] STATION 10549 at RM 1.3 from 9/9/2008 to 9/9/2008, the geometric mean of 0.221 mg/Kg from 5 valid individual fish tissue samples exceeds the 0.040 mg/kg criteria. [ODEQ] STATION 10550 at RM 7.1 from 9/26/2009 to 9/26/2009, the geometric mean of 0.12 mg/Kg from 1 valid individual fish tissue samples exceeds the 0.040 mg/kg criteria
Willamette Lower Willamette 17090012 16572	Multnomah Channel 1227863458618 0 to 21.7 21.7	pH	Summer	pH 6.5 to 8.5	Water contact recreation; Resident fish and aquatic life	Cat 3: Insufficient data	No action	Previous Data: [DEQ] LASAR 12935 River Mile 18.6: From 8/5/1997 to 8/5/1997, 0 out of 1 samples (0%) outside pH criteria range 6.5 to 8.5. [DEQ] LASAR 12936 River Mile 14.3: From 8/5/1997 to 8/5/1997, 0 out of 1 samples (0%) outside pH criteria range 6.5 to 8.5. [DEQ] LASAR 12937 River Mile 14.2: From

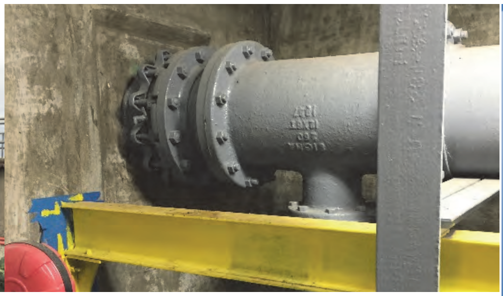
								8/5/1997 to 8/5/1997, 0 out of 1 samples (0%) outside pH criteria range 6.5 to 8.5. [DEQ] LASAR 12938 River Mile 18.6: From 8/5/1997 to 8/5/1997, 0 out of 1 samples (0%) outside pH criteria range 6.5 to 8.5. [DEQ] LASAR 30756 River Mile 21: From 9/9/2003 to 9/9/2003, 0 out of 1 samples (0%) outside pH criteria range 6.5 to 8.5. [DEQ] LASAR 30757 River Mile 18.7: From 9/9/2003 to 9/9/2003, 0 out of 1 samples (0%) outside pH criteria range 6.5 to 8.5. Previous Status: Cat 3: Insufficient data Previous Action: Added to database Previous Assessment Year: 2004
Willamette Lower Willamette 17090012 21504	Multnomah Channel 1227863458618 0 to 21.7 21.7	Phosphate Phosphorus	Summer	Total phosphates as phosphorus (P): Benchmark 50 ug/L in streams to control excessive aquatic growths	Aquatic life	Cat 3: Insufficient data	No action	Previous Data: [DEQ] LASAR 12936 River Mile 14.3: From 8/5/1997 to 8/5/1997, 1 out of 1 samples > 50 ug/L

benchmark criterion. [DEQ] LASAR 12935 River Mile 18.6: From 8/5/1997 to 8/5/1997, 1 out of 1 samples > 50 ug/L benchmark criterion. [DEQ] LASAR 12937 River Mile 14.2: From 8/5/1997 to 8/5/1997, 1 out of 1 samples > 50 ug/L benchmark criterion. [DEQ] LASAR 30756 River Mile 21: From 9/9/2003 to 9/9/2003, 1 out of 1 samples > 50 ug/L benchmark criterion. [DEQ] LASAR 12938 River Mile 18.6: From 8/5/1997 to 8/5/1997, 1 out of 1 samples > 50 ug/L benchmark criterion. [DEQ] LASAR 30757 River Mile 18.7: From 9/9/2003 to 9/9/2003, 1 out of 1 samples > 50 ug/L benchmark criterion. Previous Status: Cat 3: Insufficient data Previous Action: Added to database Previous Assessment Year: 2004

Willamette Lower Willamette 17090012 6758	Multnomah Channel 1227863458618 0 to 1.6 1.6	Sedimentation	Undefined	The formation of appreciable bottom or sludge deposits or the formation of any organic or inorganic deposits deleterious to fish or other aquatic life or injurious to public health, recreation, or industry may not be allowed.	Resident fish and aquatic life; Salmonid fish rearing; Salmonid fish spawning	Cat 3: Insufficient data	No action	Previous Status: Insufficient data Previous Action: Added to database Previous Assessment Year: 1998
Willamette Lower Willamette 17090012 12988	Multnomah Channel 1227863458618 0 to 21.7 21.7	Temperature	Year Round (Non-spawning)	Salmon and trout rearing and migration: 18.0 degrees Celsius 7-day-average maximum	Salmon and trout rearing and migration	Cat 4A: Water quality limited, TMDL approved	No action	Need to correct 4th field HUC. TMDL Approved: 09/29/2006 Willamette Basin TMDL Willamette Basin TMDL 2004 Data: [DEQ/SECOR] LASAR 26760 River Mile 11.5: From 9/28/2001 to 10/4/2002, 107 days with 7-day-average maximum > 18 degrees Celsius. Previous Status: Cat 4A: Water quality limited, TMDL approved Previous Action: Delisted - TMDL approved Previous Assessment Year: 2010

Download the spreadsheet (.csv) file: wqaclient955.csv

To select new search criteria click here - DO NOT USE THE BACK ARROW.



APPENDIX D – Pump Information

D-1:
Smith Road Pump Station

TRIANGLE PUMP AND EQUIPMENT INC.

Equipment Approval Data Date 7/13/99

Project Smith Rd Wastewater Pump Station/Scappoose

Customer Emerald Construction Order No

Engineer KCM

Equipment Ref 15140 Pumps

We request your approval to supply the following described equipment for the above referenced project:

Three (3) **Crane Deming** figure 7196, size 4 x 4 x 12 x 3, vertical, frame mounted, flexible coupled, dry pit, non clog sewage pumps with floor mounting stands, 6" x 4" suction elbows, motor mounting stands, couplings, 15 HP, 1150 RPM, 3/60/460 VAC TEFC invertor duty rated motors, factory certified tests, spare parts & start up service.

Proposed pumps will have the standard double mechanical seal (with carbon verses ceramic faces) with seal flush/lubrication system including tap from pump discharge through filter to feed stuffing box.

May we have your approval?

Cordially,

Harold R Clayton
President

attachments

/lc

14940 S.E. 82nd Drive / PO Box 950
Clackamas, Oregon 97015
(503) 656-1473
FAX (503) 656-2037

269 S.W. 41st Street
Renton, Washington 98055
(425) 251-9666
FAX (425) 251-9667

Eng: KCM	Proj: SMITH RD PUMP STA	No:	CRANE DEMING PUMP DIV.
Contr: EMERALD CONST.	Tag:	S.O.: ***	
Owner: CITY OF SCAPPOOSE	Rep.: TRIANGLE PUMP	P.O.: 19396	Salem, Ohio, U.S.A.

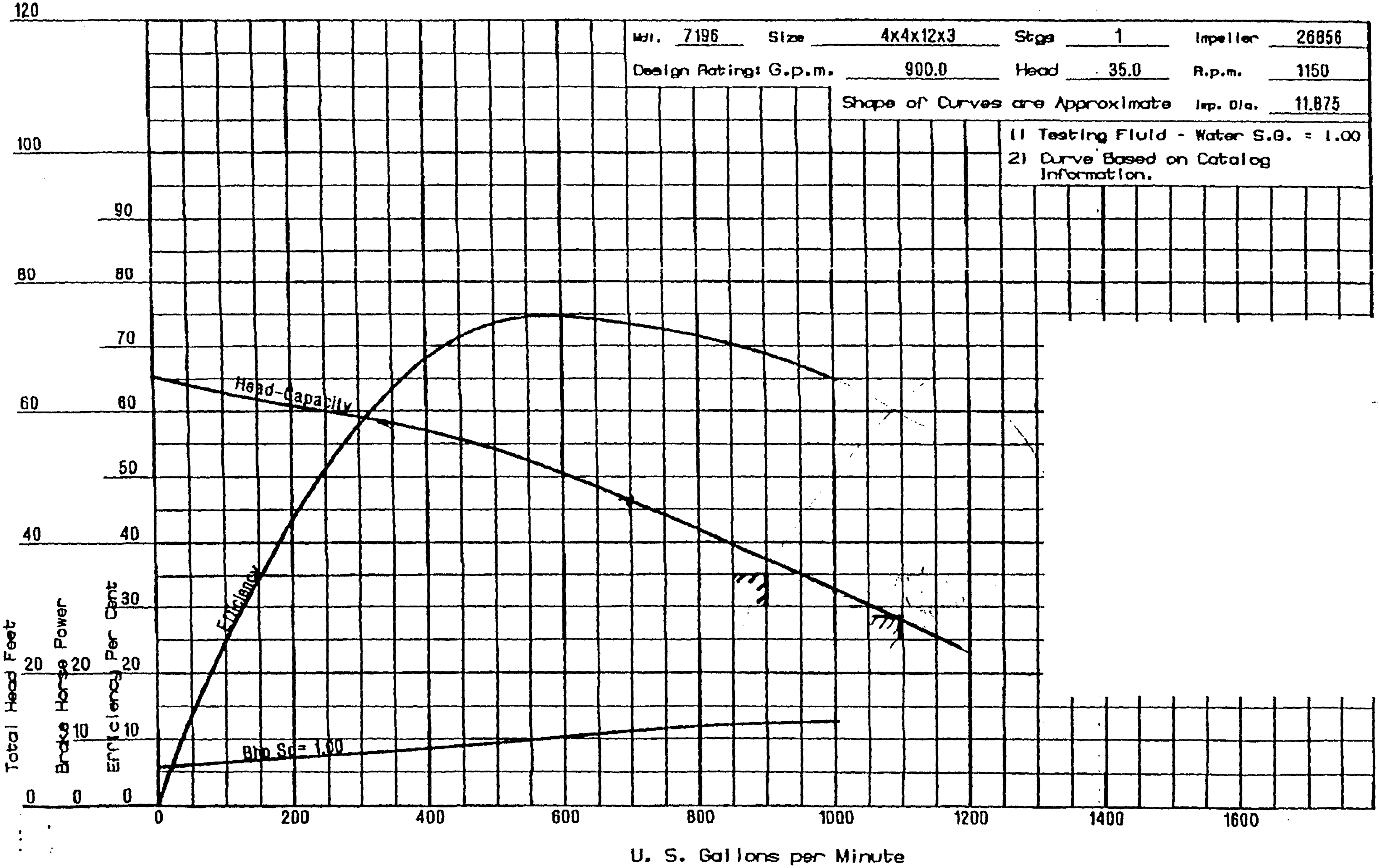
Mdl. 7196 Size 4x4x12x3 Stge 1 Impeller 26856

Design Rating: G.p.m. 900.0 Head 35.0 R.p.m. 1150

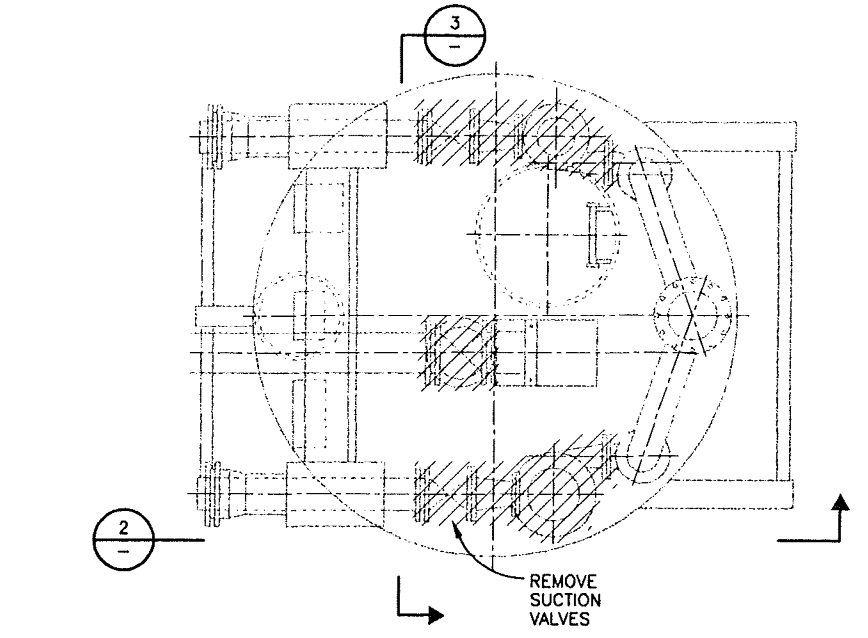
Shape of Curves are Approximate Imp. Dia. 11.875

1) Testing Fluid - Water S.G. = 1.00

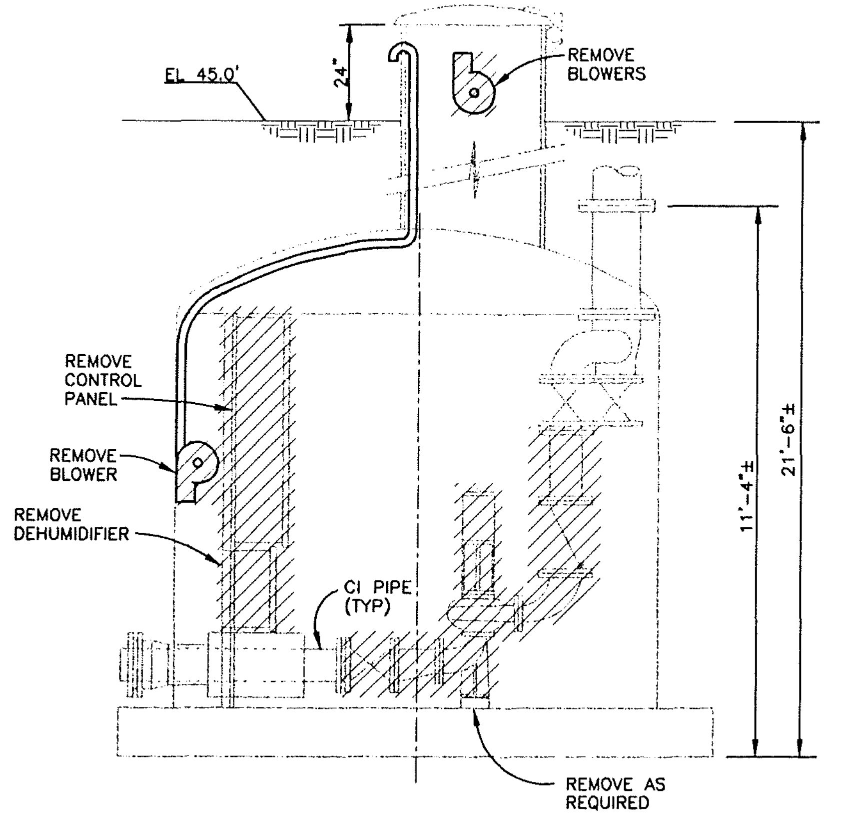
2) Curve Based on Catalog Information.



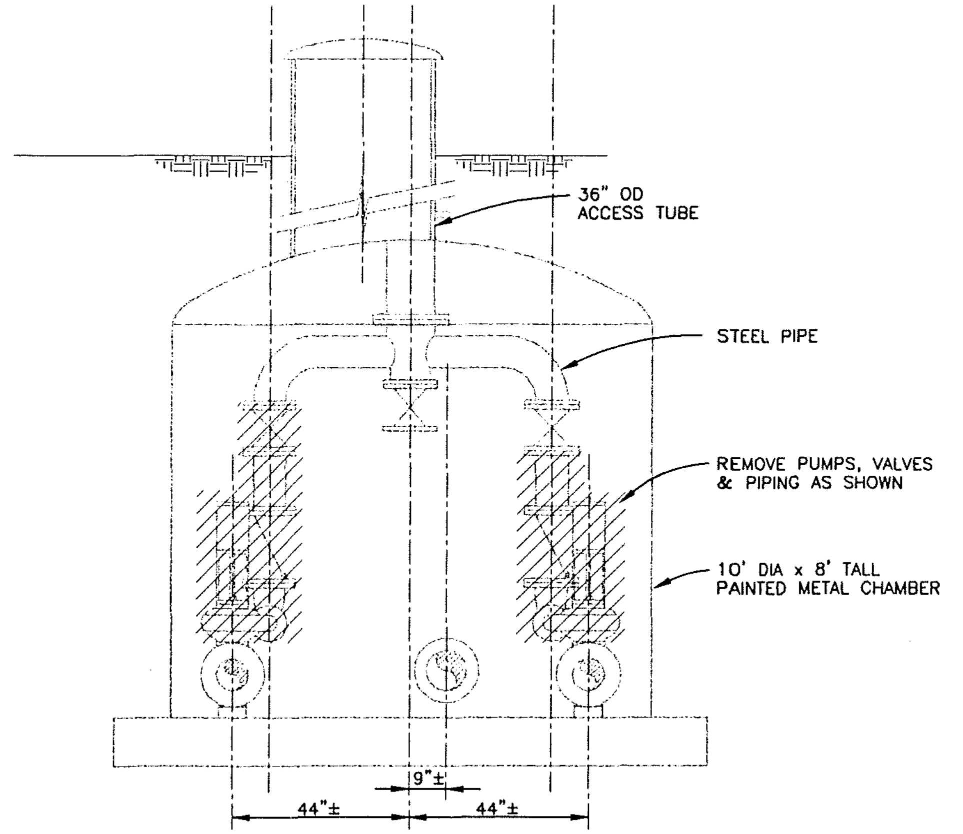
Submitted By:	Date:	Submittal ! **	Date: 07/02/99



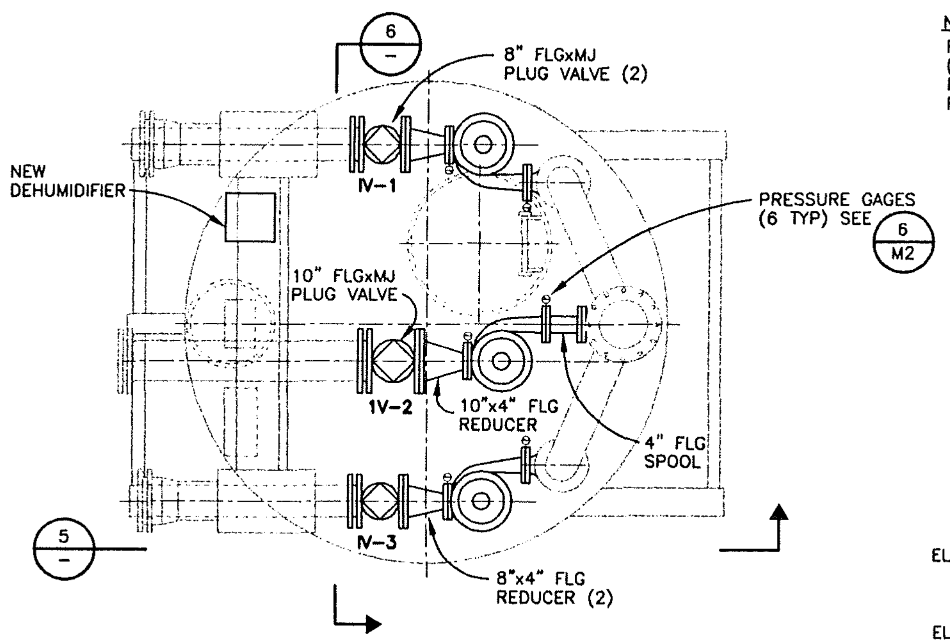
EXISTING PUMP STATION LAYOUT
1/2"=1'-0"
1



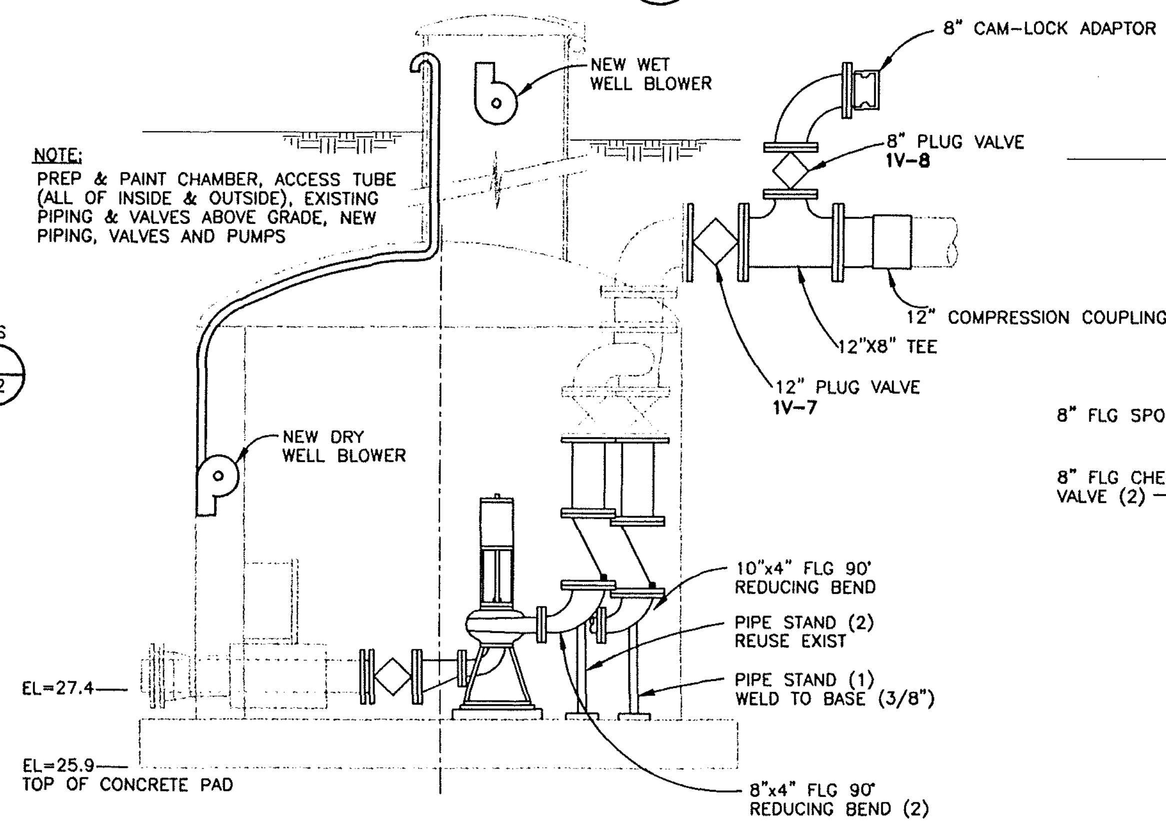
EXISTING PUMP STATION ELEVATION
1/2"=1'-0"
2



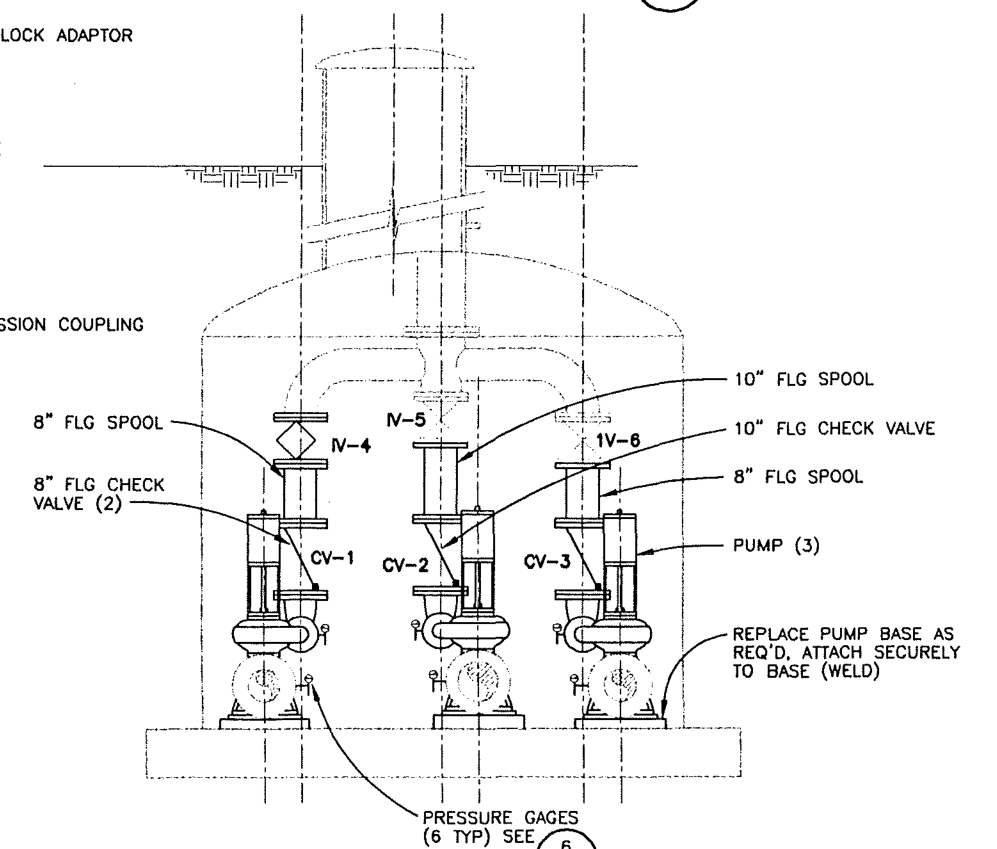
EXISTING PUMP STATION SECTION
1/2"=1'-0"
3



PROPOSED PUMP STATION LAYOUT PLAN
1/2"=1'-0"
4



PROPOSED PUMP STATION ELEVATION
1/2"=1'-0"
5



PROPOSED PUMP STATION SECTION
1/2"=1'-0"
6

2-20/CAD/DESIGN/M1.dwg @1=24 04/22/02

This drawing is full size when 22" x 34" or is reduced to half size when 11"x17"

KCM

KCM, Inc.
7080 SW Fir Loop
Portland, Oregon 97223

date MAY 1999	designed by
	drawn by
scale AS NOTED	checked by
	approved by

RECORD DRAWING

ORIGINAL DRAWING REVISED BY KCM BASED ON INFORMATION PROVIDED BY EMERALD CONSTRUCTION KCM CANNOT VERIFY THE ACCURACY OR COMPLETENESS OF THIS INFORMATION AND SHALL NOT BE RESPONSIBLE FOR ANY ERRORS OR OMISSIONS INCORPORATED HEREIN AS A RESULT

revisions

KCM NO. 1882-020

City of Scappoose, Oregon
SMITH ROAD
PUMP STATION

MECHANICAL

EXISTING DRY WELL PIPING MODIFICATIONS

drawing number M1

sheet number 9 of 14

D-2:
Spring Lake Pump Station

HYDRONIX Inc. 2425 S. E. OCHOCO STREET • PORTLAND, OREGON 97202 • 503·659·6230

PUMP STATION DATA SHEET

LOCATION SPRING LAKE PARK SERIAL NO. #34558-S

OWNER CITY OF SCAPPOOSE ENGINEER ECM ENGINEERING CO.

CONTRACTOR SCHARF PROPERTIES DISTRIBUTOR HYDRONIX, INC.

STATION TYPE 181-T Wet Well 72" Dia. X 9' 6" Deep Wet Well Cover steel

______ DATE INSTALLED ______

PUMP STATION

PIPING: Suction 4", Disch. 4"; Suction 4", Disch. 4".

CONDITIONS OF SERVICE

Design Duty: 100 GPM, 24' T.D.H., 9' Suction Lift; Liquid ______

Solids 3, N.P.S.H. (AVL.) ______, (REQ.) ______.

PUMP DATA

Pump Model: 40MPC Imp. Dia. 8 5/32 Priming: (XX)Self, ~~( )Vacuum,~~ ~~( )Flooded.~~ Stuffing Box Type Mech. Seal, Lubrication H^2O.

Rotation of Pump #1 C, #2 C, #3 ------.

MOTOR DATA

Brand GE, Enclosure HBP, Horsepower 3,

RPM 1150, Phase 1, Cycle 60, Volt 230, Starting Code ______,

Modification Close coupled Frame ______.

ELECTRICAL DATA

1 Phase, 60 Cycle, 230 Volts, 4 Wire, 110 Control Voltage.

Transformer Required: ( )YES, (X)NO. Size ______.

CONTROL PANEL

Location In pump station NEMA Type 1

Service Entrance Size ______ AMPS. Pump #1 Pump #2 Pump #3

Circuit Breaker--Trip Rating/AMPS ______ ______ ______

Magnetic Starter--NEMA size ______ ______ ______

Overload Heater size ______ ______ ______

Alternator Type Electrical MFG. Deversified Coil Voltage 110

Control Panel Components ________________________________

____

Alarm Functions: High Water

Level Control: Mercury Level Controls

ELEVATIONS:

Ground Level	18.4	Low Water	10.3
Station Disch.	13.6	Pump Off	10.3
Station Floor	18.4	Lead Pump On	11.8
W.W. Invert	13.3	2 Pump Overload	12.8
W.W. Floor	8.8	High Water	12.3

STATION OPTIONS

(X) 1500 Watt Heater, ~~( )Station Blower, ( )Wet Well Blower, ( )_____P.P.D. Dehumidifier, ( )Station Light,~~ (X)Trouble Light, (X)Convenience Outlet, ~~( )_____KVA Transformer, ( )Ladder, ( )Sump Pump,~~ (XX)Elapsed Time Meters, ~~( )_____VDC Battery and Charger,~~ (X)Light~~/Horn/Bell/Dim Glow Alarm, ( )Dry Alarm Contacts, ( )Telemetry Included/By Others, ( )Vacuum Pumps, ( )Air Compressor~~, (X)Spare Parts Kit, (X) 2 O & M Instruction Manuals.

NOTES: ________________________________

____

____

____

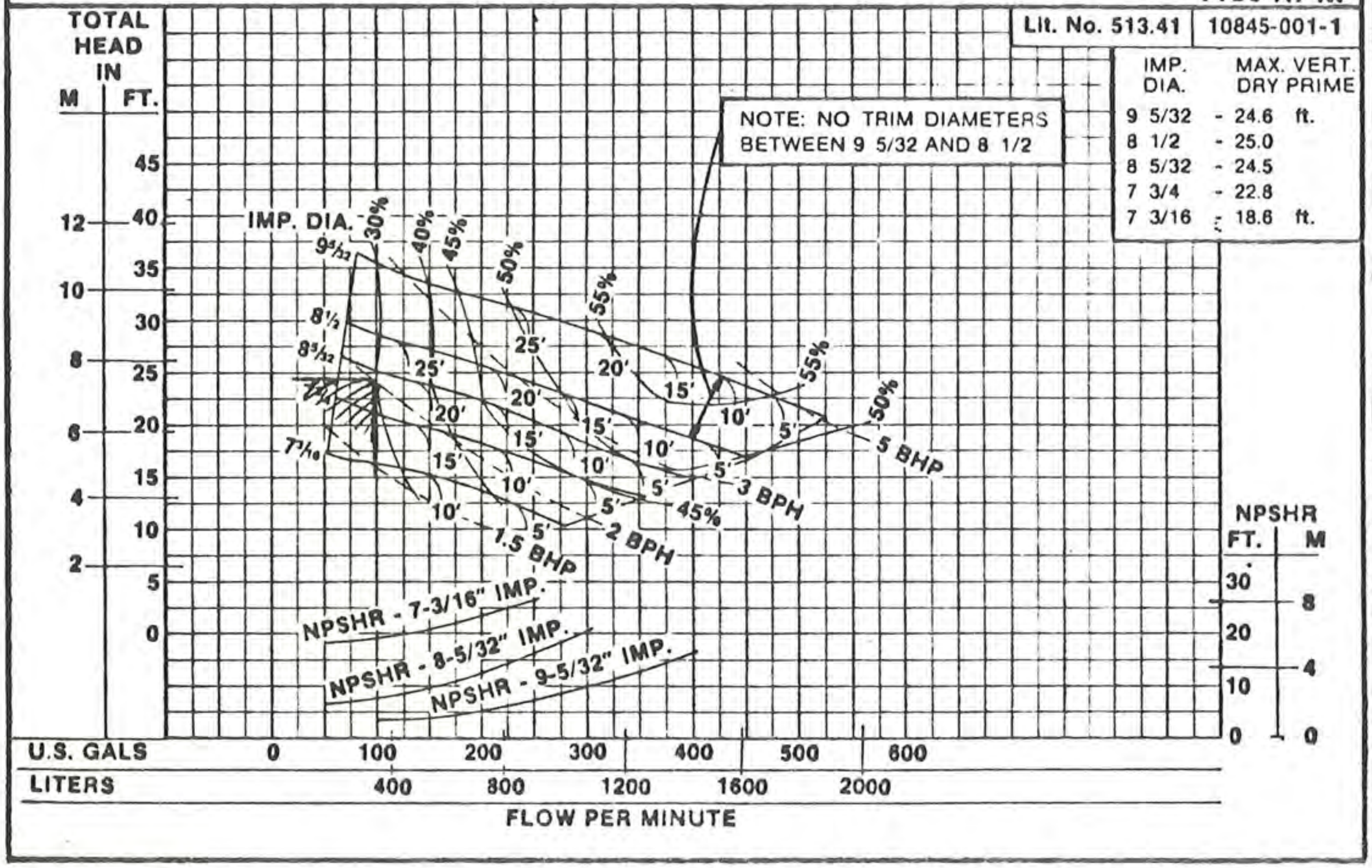
Application Engineering

Prepared By LARRY R. CHAPMAN/sf Date: 5-23-86

HYDRONIX Inc. 2425 S.E. OCHOCO STREET • PORTLAND, OREGON 97222 • 503-659-6230

HYDROMATIC PUMPS

MODEL: 40MPD - 40MPC - 40MPSF SELF PRIMER - MAX. SOLIDS: 3" SPHERE - 1150 RPM



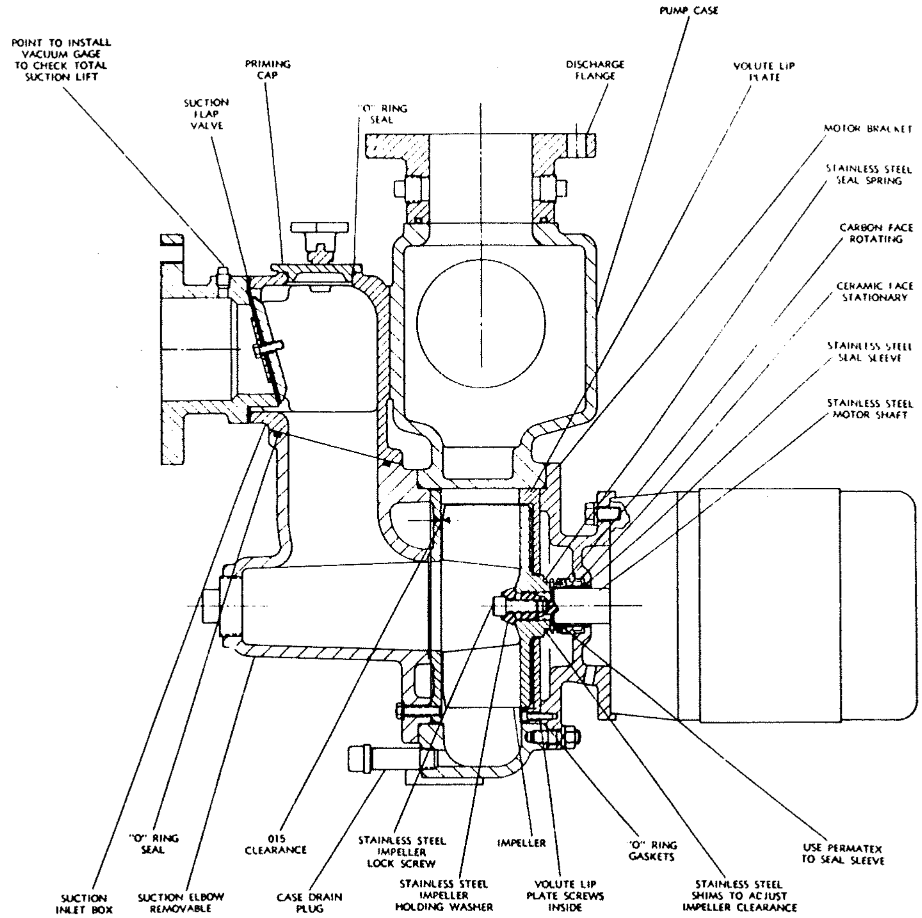
GPM 100 TDH 24'

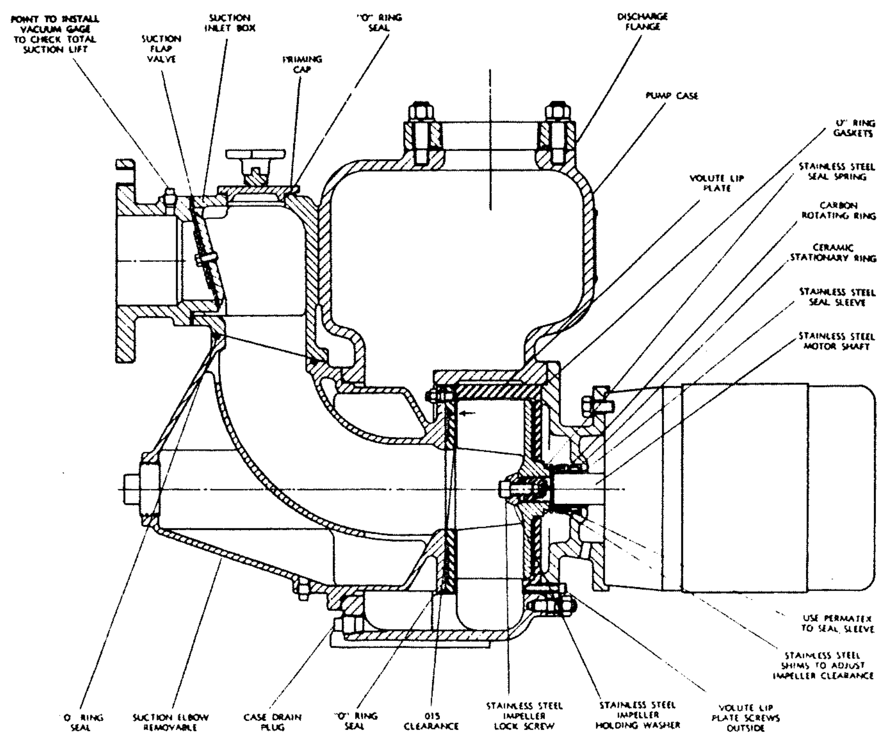
EFF. 30 BHP 3

NPSHR 8'

TDH + PSI X 2.31

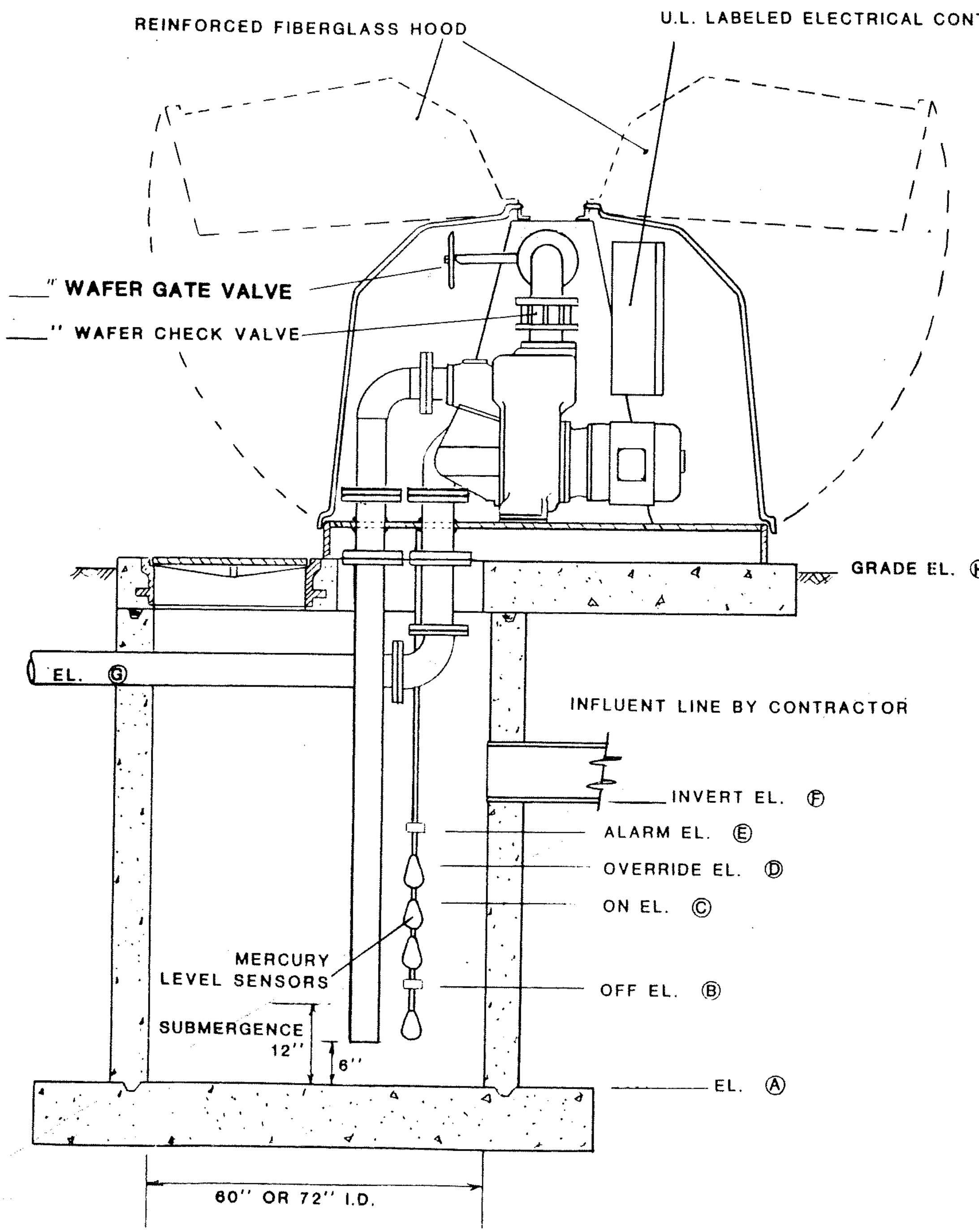
SECTIONAL DRAWINGS OF SELF PRIMING, CLOSE COUPLED SEWAGE PUMPS

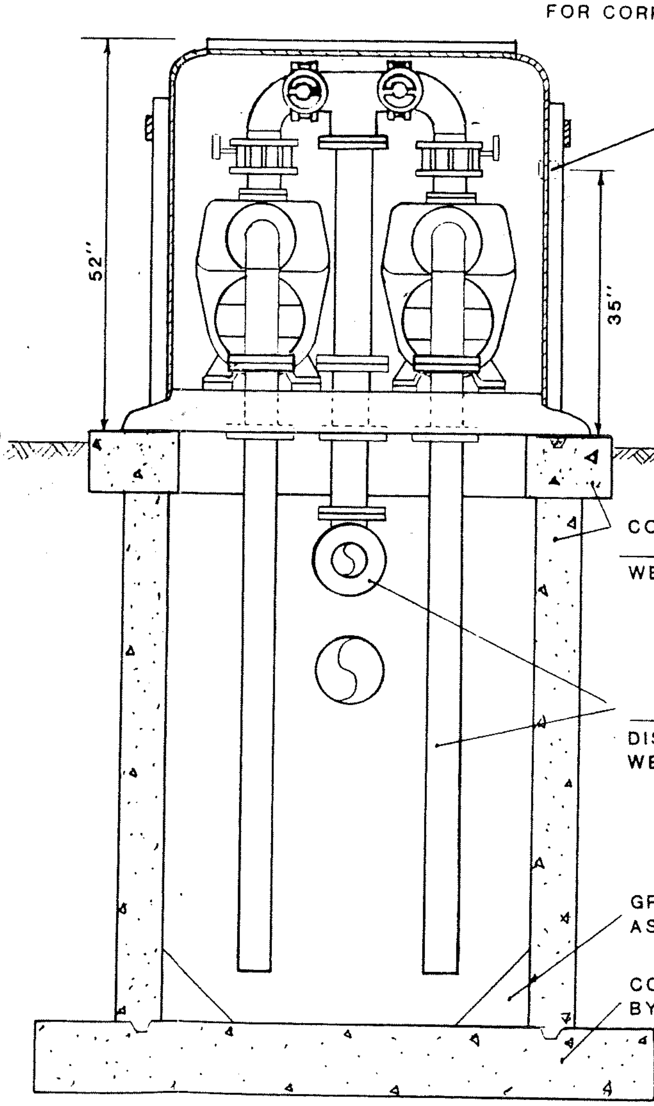


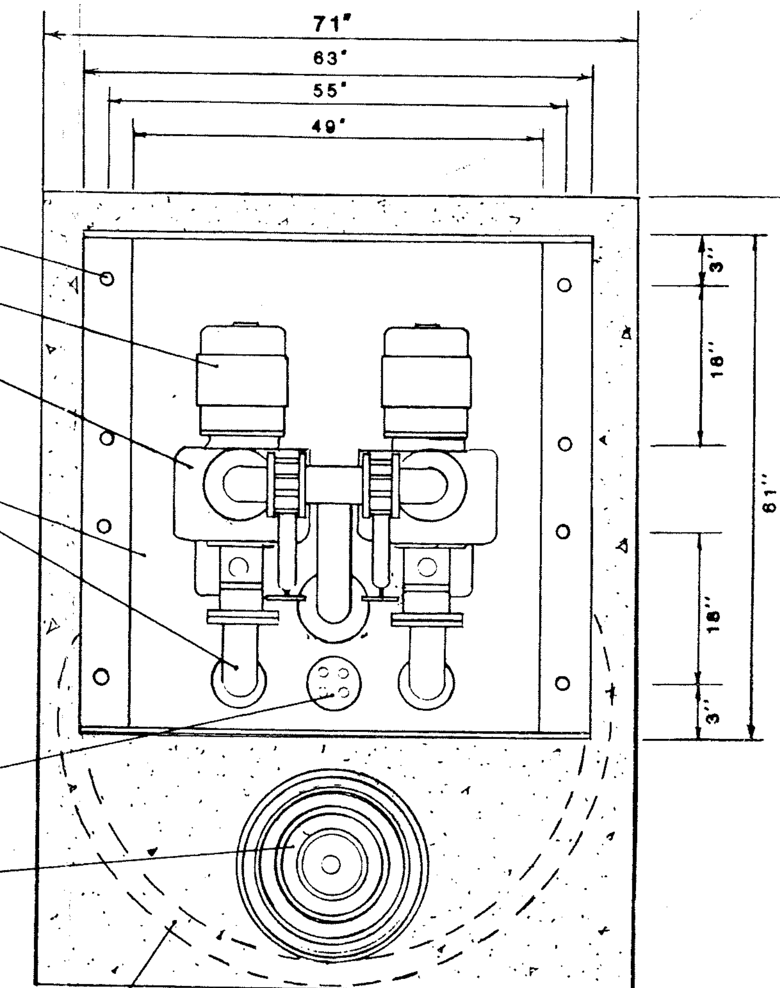


HYDRONIX

STATION	G.P.M.	T.D.H.	H.P.	R.P.M.	PHASE	VOLTS	ELEVATIONS A	B	C	D	E	F	G	H
Scappoose Trailer Park	100	24'	3	1150	1	230								







ACCESSORIES NOT SHOWN FOR CLARITY

- ___ KVA TRANSFORMER
- 4 CIRCUIT LIGHTING PANEL
- CONVENIENCE OUTLET
- 60 WATT DROP LIGHT
- 1300 WATT HEATER WITH THERMOSTAT
- BLOWER WITH THERMOSTAT

HYDRONIX INC. PORTLAND, OREG.		
MODEL 181 CLOSE COUPLED SELF-PRIMING SEWAGE PUMPING STATION		
DATE: 8/83		DWG. BY: JWR
SCALE: NONE	REVISIONS:	DWG# 31-01-3570

D-3:
Keys Landing Pump Station

HYDRONIX Inc. 2425 S. E. OCHOCO STREET • PORTLAND, OREGON 97202 • 503-659-6230

PUMP STATION DATA SHEET

LOCATION KAYS LANDING SERIAL NO.

OWNER SCAPOOSE, OR ENGINEER DAVE WINSHIP

CONTRACTOR MERRILL STROBEL CONST DISTRIBUTOR HYDRONIX

STATION TYPE 183 Wet Well 6' Dia. X 13' Deep ~~Wet Well Cover~~

DATE INSTALLED

PUMP STATION

PIPING: Suction 4, Disch. 4; Suction 4, Disch. 4.

CONDITIONS OF SERVICE

Design Duty: 120 GPM, 46 T.D.H., 13' Suction Lift; Liquid RAW

Solids 3", N.P.S.H. (AVL.) ______, (REQ.) ______.

PUMP DATA

Pump Model: 40MPV Imp. Dia. 9 5/32 Priming: (X) Self, ~~( ) Vacuum,~~ ~~( ) Flooded~~. Stuffing Box Type MECH., Lubrication OIL.

Rotation of Pump #1 CW, #2 CW, #3 ______.

MOTOR DATA

Brand CHOICE, Enclosure ODP, Horsepower 5,

RPM 1750, Phase 3, Cycle 60, Volt 230, Starting Code ______,

Modification ______ Frame ______.

ELECTRICAL DATA

3 Phase, 60 Cycle, 230 Volts, 4 Wire, 120 V Control Voltage.

Transformer Required: ( ) YES, (X) NO. Size ______.

HYDRONIX Inc.

RECEIVED
- 5 1992

QUOTATION # 91172
Date 5-28-92 Page # 1 of 2 pages
2425 SE Ochoco
Portland, OR 97222
503-659-6230 FAX 503-659-8718
1-800-547-9708

Project Name: ~~Keeps~~ Keys Landing
Contact Person: Merrill Strobel
Phone: H-543-2581 M-936-4303 FAX:

TO: Merrill Strobel Construction
54444 Kalberer Rd.
Scappoose, OR 97056

QTY.	DESCRIPTION	[redacted]
1	HYDRONIX Model 183, 4" x 4", prefabricated, hot-dipped, galvanized, surface mounted, self-priming sewage lift station with fiberglass access hoods. Installed, in the chamber, will be pumps, motors, valving, piping, control panel, and appurtenances. The equipment will be factory assembled, tested, and ready for operation, for mounting over a 7'0" diameter x 13'0" deep wet well, by others.	

MAIN CHAMBER SIZE: 78"W x 67"L x 51" high
ENTRANCE: Fiberglass hinged hoods
PUMPS: Hydromatic Model 40 MPV
MOTORS: 5 HP, 1750 RPM
V-BELT DRIVE RPM: 1750 / 1370
POWER SERVICE: 3 phase, 60 hertz, 230 volts, 4 wire
PUMP CAPACITY: 120 GPM @ 46' T.D.H.
CONTROLLER: Nema 1, duplex, alternating
LEVEL SENSOR: Mercury float switch
ALARM: Local light

ACCESSORIES

1. Suction pipes
2. 4 circuit lighting panel
3. Duplex GFI receptacle
4. 60 watt drop light
5. 13/15 M watt heaters
6. Spare parts kit
7. O & M manuals
8. T.D. no flow limit switches
9. 156 CFM blower

Estimated shipping weight is 2,500 pounds
Start up included

Total Price [redacted]

Above quoted price good for: 30 days
Delivery in Approximately 6 WKS • Terms C.O.D. • FOB:Portland, Oregon FFA

THIS QUOTATION AND ANY AGREEMENT TO SELL RESULTING FROM THE QUOTATION IS SUBJECT TO HYDRONIX GENERAL TERMS AND CONDITIONS OF SALE SUBMITTED WITH THIS QUOTE. TERMS AND CONDITIONS ON BACK SIDE.

Co. Name Merril Strobel Construction

Accepted by: Merril Strobel Date 6/1/92
SIGNATURE

Please print name Merril Strobel

Hydronix, Inc.
Proposed by: M. Watterson
SIGNATURE

Michael Watterson, Sales Engineer

HYDRONIX Inc. 2425 S.E. OCHOCO STREET • PORTLAND, OREGON 97222 • 503-659-6230

HYDROMATIC PUMPS

HYDROMATIC PUMPS

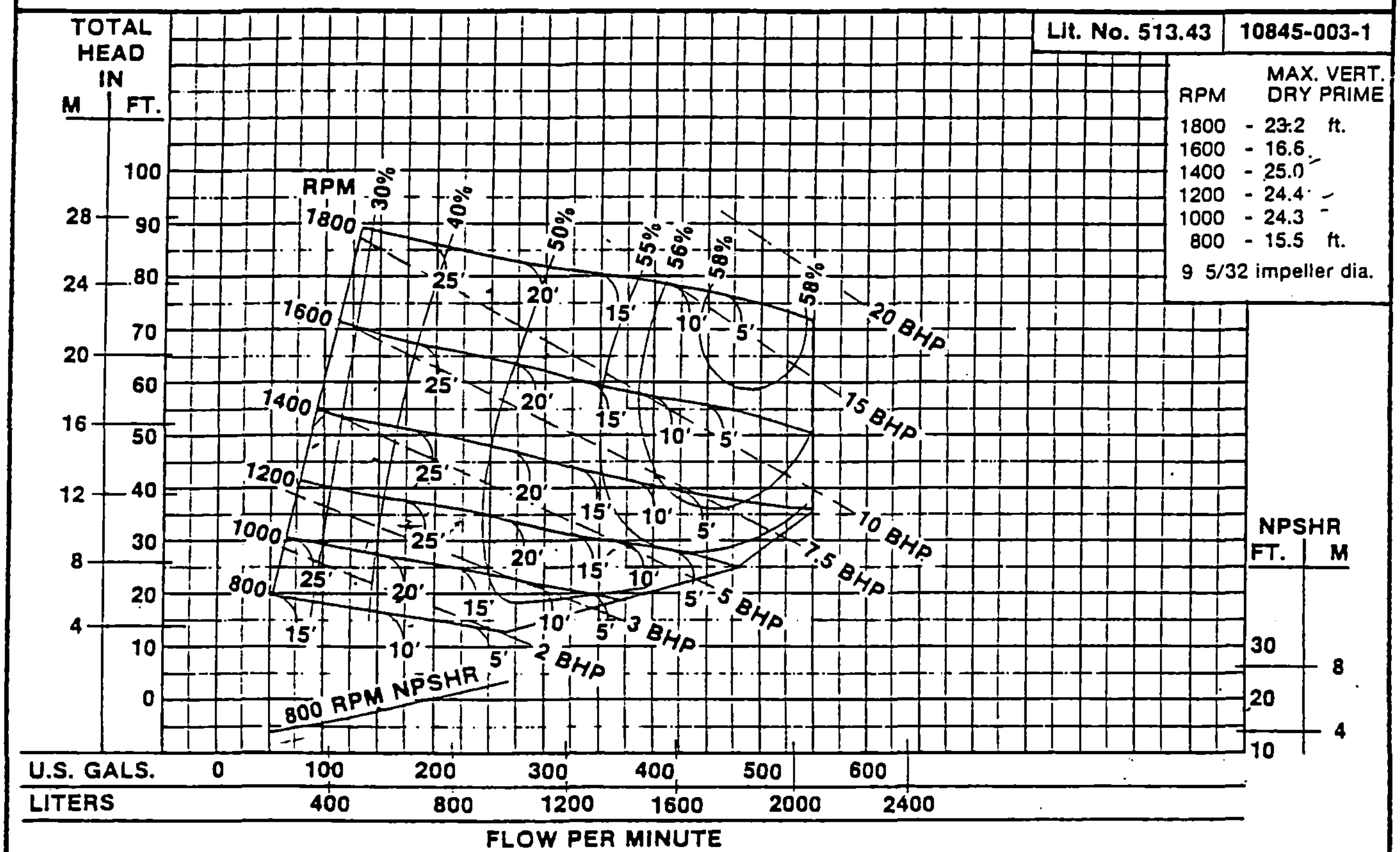
SECTION 510
PERFORMANCE DATA

MODEL: 40MPV - 40MPSFV SELF PRIMER - MAX. SOLIDS: 3" SPHERE - VAR. RPM

Lit. No. 513.43 | 10845-003-1

RPM	MAX. VERT. DRY PRIME
1800	- 23.2 ft.
1600	- 16.6
1400	- 25.0
1200	- 24.4
1000	- 24.3
800	- 15.5 ft.

9 5/32 impeller dia.

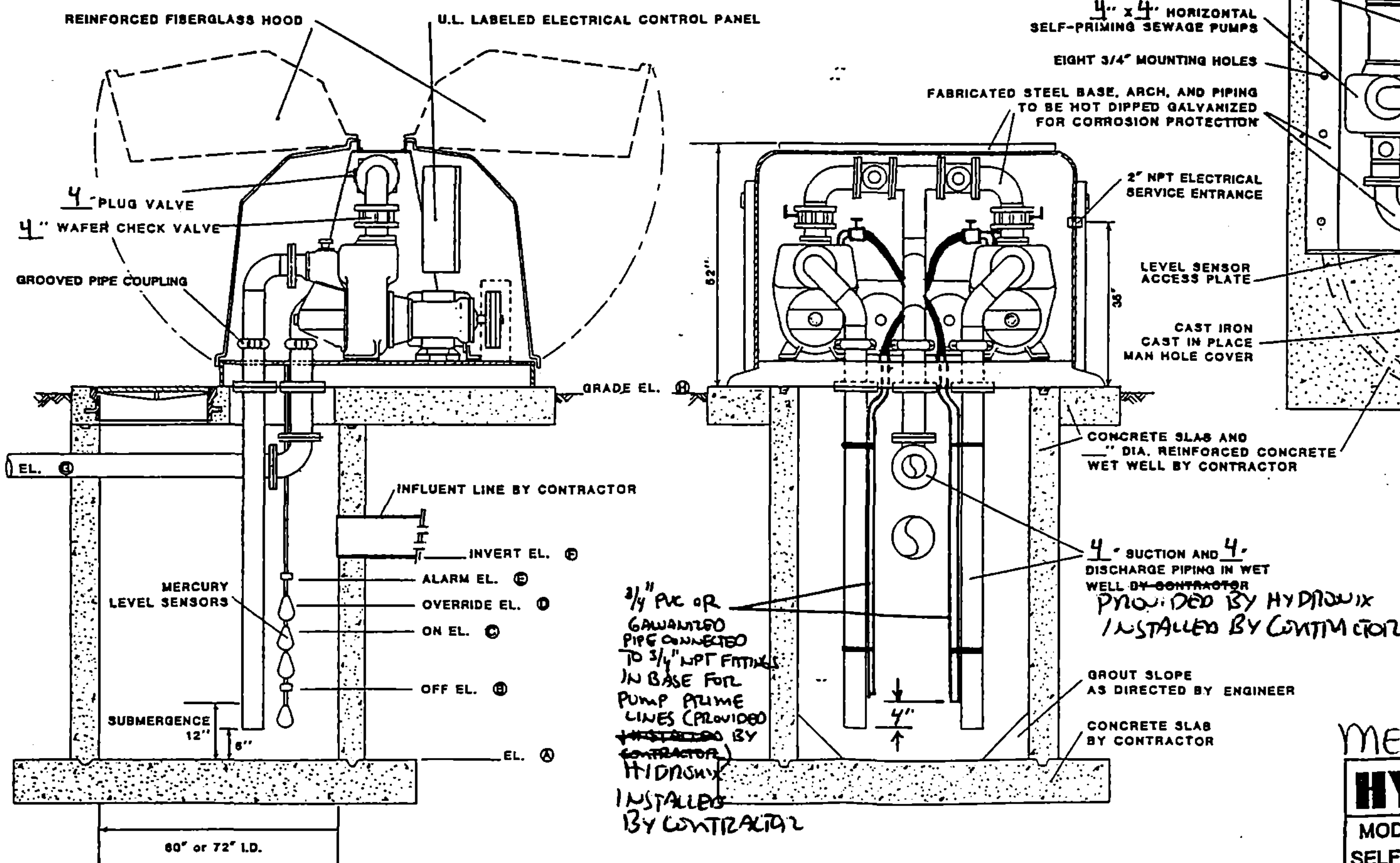


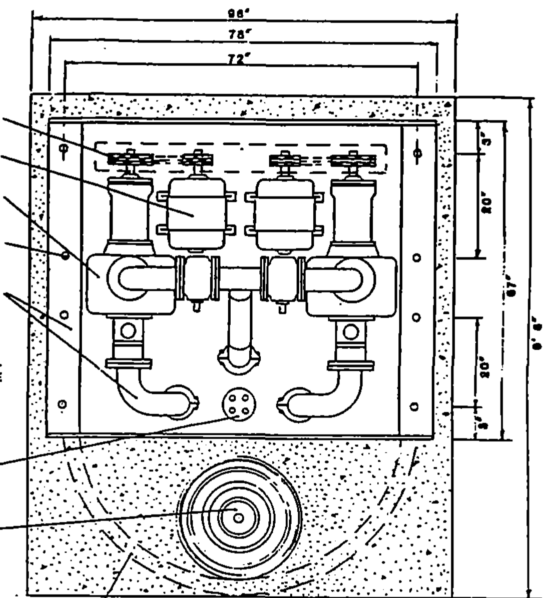
GPM 120 TDH 46'

EFF. 35 BHP 3.9

NPSHR 10'

STATION	G.P.M.	T.D.H.	H.P.	R.P.M.	PHASE	VOLTS	ELEVATIONS A	B	C	D	E	F	G	H
183-T	120	46	5	1370	3	230								





ACCESSORIES NOT SHOWN FOR CLARITY

- ~~KVA TRANSFORMER~~
- 4 CIRCUIT LIGHTING PANEL
- CONVENIENCE OUTLET
- 60 WATT DROP LIGHT
- 1300 WATT HEATER WITH THERMOSTAT
- BLOWER WITH THERMOSTAT
- SUCTION & PRIMING LINE PIPES

MERRILL STROBEL / KEBBS LANDING

HYDRONIX INC. PORTLAND, OREG.

MODEL 183 V-BELT DRIVEN
SELF PRIMING SEWAGE PUMPING STATION

DATE: 2/83		DWG. BY: JWR
SCALE: NONE	REVISIONS:	DWG# 31-01-3684

D-4:
Highway 30 Pump Station

Pump Station Data Sheet

Location: Scappoose, Oregon **Serial No.** 48698

Owner: City of Scappoose **Engineer:**

Contractor: Clearwater Construction **Distributor**: Hydronix, Inc.

Station Type 183 **Wet Well** 8' **Dia. x** 19' **Deep Wet Well Cover**

Date Installed June, 1996

Pump Station

Piping: Suction 4" **,Disch.** 4" ; **Suction** 4" **,Disch.** 4"

Conditions of Service

Design Duty: 300 **GPM,** 42' **TDH,**__**Suction Lift; Liquid** Sewage

Solids 3" **, NPSH (avl)**_______, **(req)**_______.

Driven RPM 1400

Sheave 4.75x2 Groove **,Bushing** SH 1-3/8 **,Belt** 3VX475

Pump Data:

Serial No.:____________

Pump Model: 40MP **,Imp.Dia.** 9-5/32" **Priming**:(X)**self,**( )**vacuum**

( )**flooded. Stuffing Box Type**_____, **Lubrication**: Media

Rotation of Pump #1 CW **,#2** CW **,#3**__.

Motor Data

Brand_ Baldor ___, **Enclosure**_ODP_, **Horsepower**_7.5__

RPM_1800_, **Phase**_3_, **Cycle**_60_, **Volt**_230_, **Starting Code**__

Modification__________, **Frame**_213T__

Sheave_3V6.0x2 Groove_, **Bushing**_SH 1-1/2_, **Belt**_3VX475___

Electrical Data

Phase_3_, **Cycle**_60_, **Volts**_230_, **Wire**_4_, **Control Voltage**_120__

Transformer Required: (X)Yes , () No. Size_E180-3PB__

Control Panel

Location_Inside Station____ **NEMA Type**_4__

Service Entrance Size___Amps	**Pump #1**	**Pump #2**	**Pump #3**
Circuit Breaker-Trip Rating/AMPS	30	30	
Magnetic Starter-NEMA size	1	1	
Overload Heater size	2013B-3	2013B-3	

Alternator Type_67C-1COA_ **Mfg**_Warrick_ **Coil Voltage**_110_

Alarm Functions:_Pump 1 & 2 fail, High wet well, Compressor fail_______

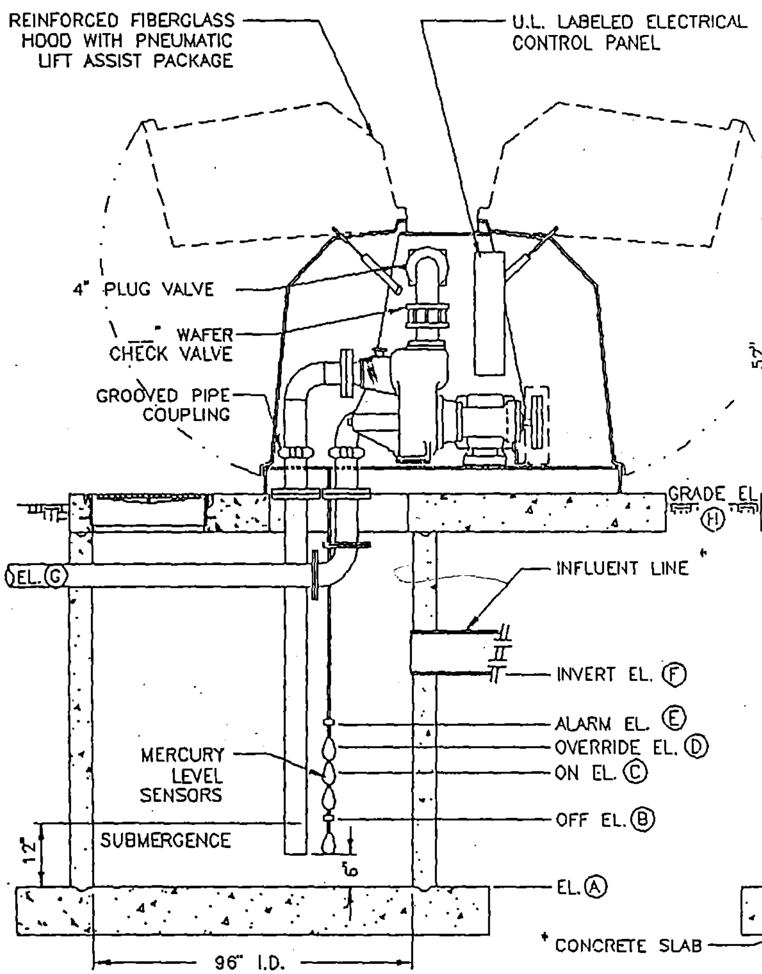
Level Control:_Intrinsically Safe Floats_____

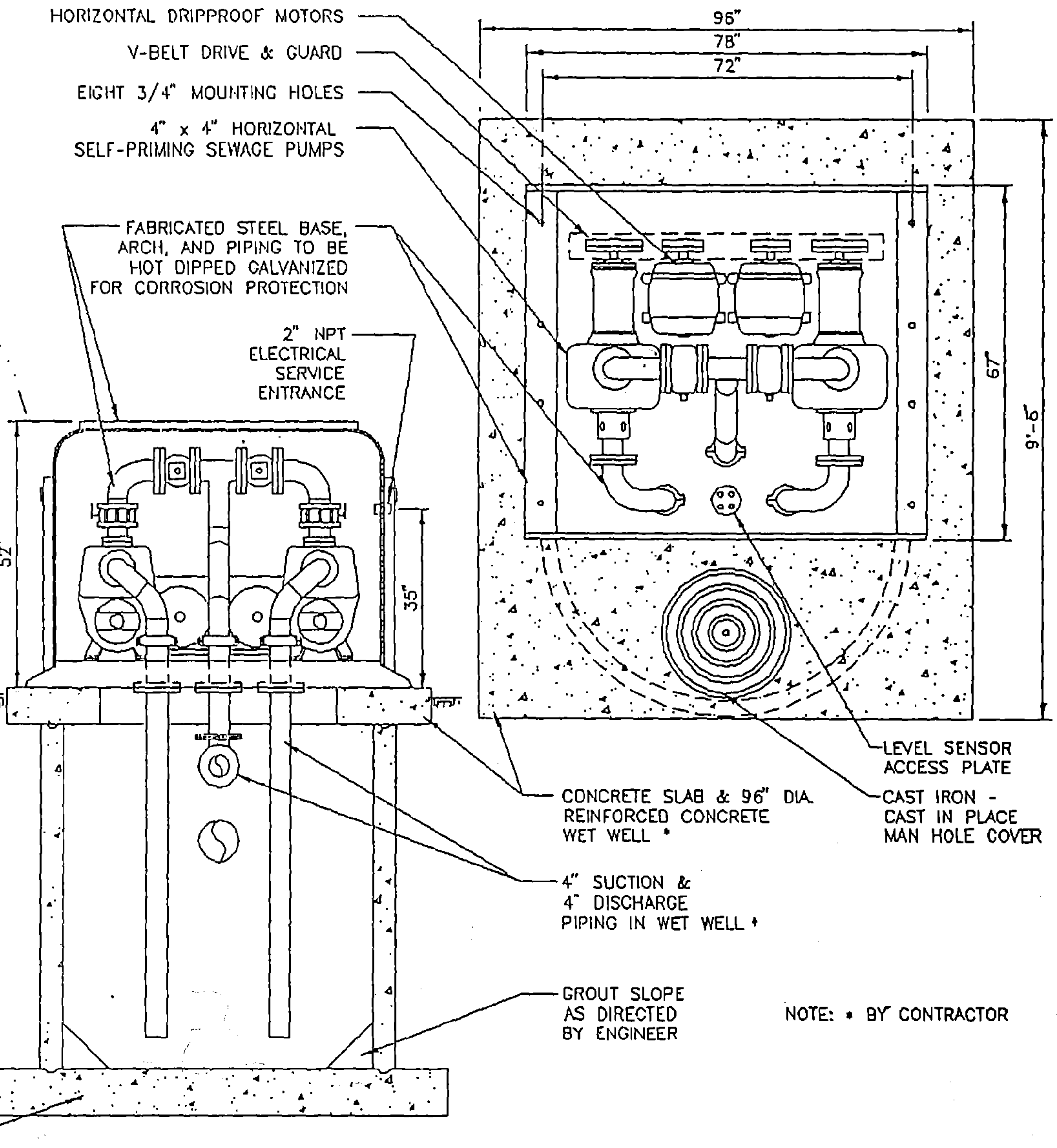
Elevations:

Ground Level_46.6___	**Low Water**__________
Station Disch._42.0___	**Pump Off**_28.5_____
Station Floor_26.0___	**Lead Pump On**_33.45__
WW Invert_34.45___	**2 Pump Overload**_33.95_
WW Floor__________	**High Water**_34.45____

ACCESSORIES NOT SHOWN FOR CLARITY

- DRAIN BACK SYSTEM
- 4 CIRCUIT LIGHTING PANEL
- CONVENIENCE OUTLET
- 60 WATT DROP LIGHT
- 1300 WATT HEATER WITH THERMOSTAT
- BLOWER WITH THERMOSTAT
- AUTO DIALER



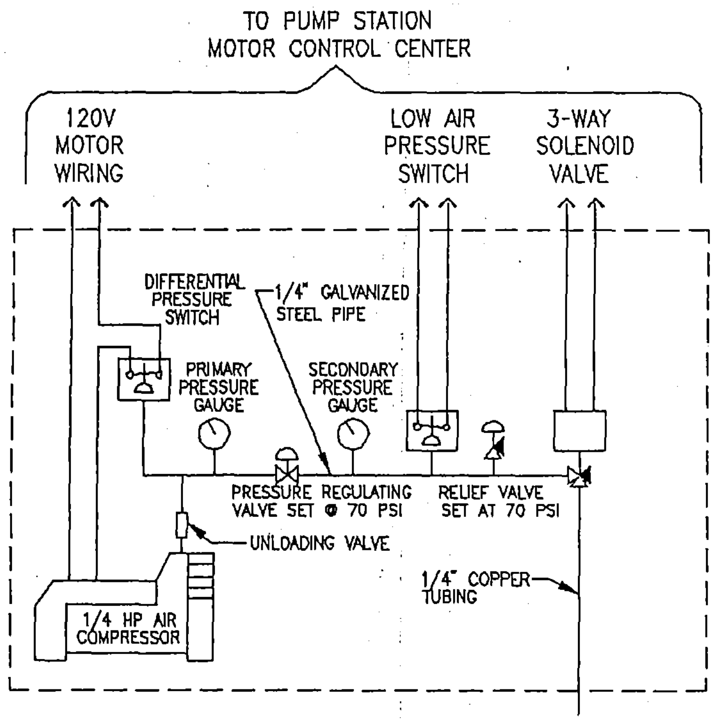


SCALES & DIMENSIONS ARE FOR REFERENCE ONLY, DRAWINGS MUST BE CERTIFIED CORRECT FOR CONSTRUCTION PURPOSES

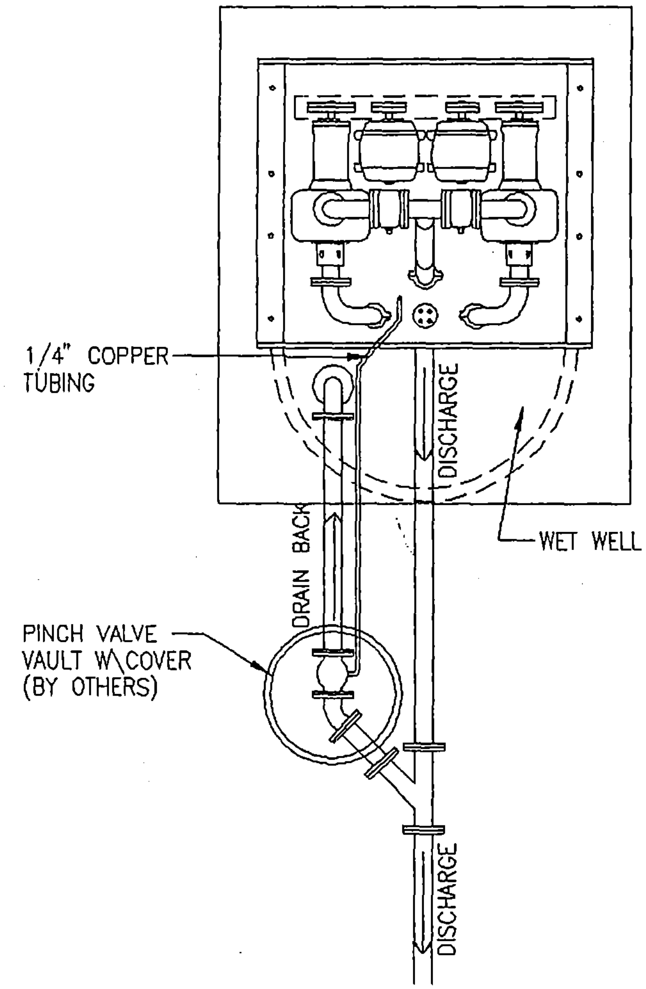
STATION	G.P.M.	T.D.H.	H.P.	R.P.M.	PHASE	VOLTS	ELEVATIONS A	B	C	D	E	F	G	H
183	300	42	7.5	1400	3	230	26.00	28.5	[illegible]3.45	[illegible].95	[illegible]5		4[illegible]	46.[illegible]

SCAPPOOSE FRED MEYER
SEWAGE PUMP STATION
SCAPPOOSE, OREGON

MODEL # 183-017	Des By	MW	Date	2-5-96
	Drn By	BT		
DWG # 183-017	Chk By	XX	Job #	48698
[illegible]el 1	Rev:	[illegible] Dat	[illegible]	X

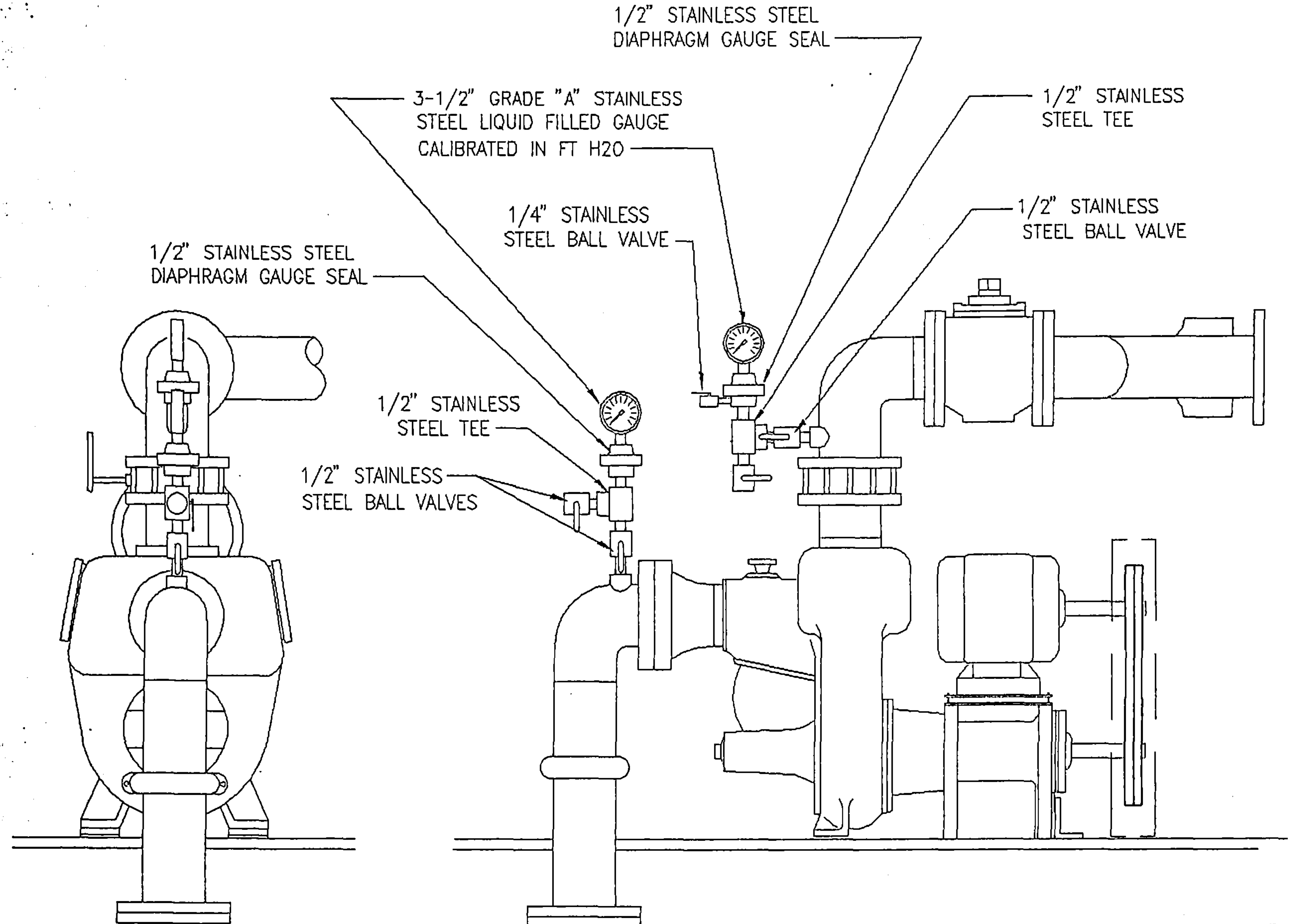


H2S DRAINBACK CONTROL SCHEMATIC



H2S TREATMENT DRAINBACK SYSTEM

HYDRONIX	JOB #	N/A	DATE	3-10-95	
	Des By	MW	Dwg #	DEQ-0002	
	Drn By	SM	Sheet	1 of 1	
	Chk By	MW	Rev: 1	2-5-96	Chk: MW



SUCTION AND DISCHARGE GAUGE KIT

HYDRONIX

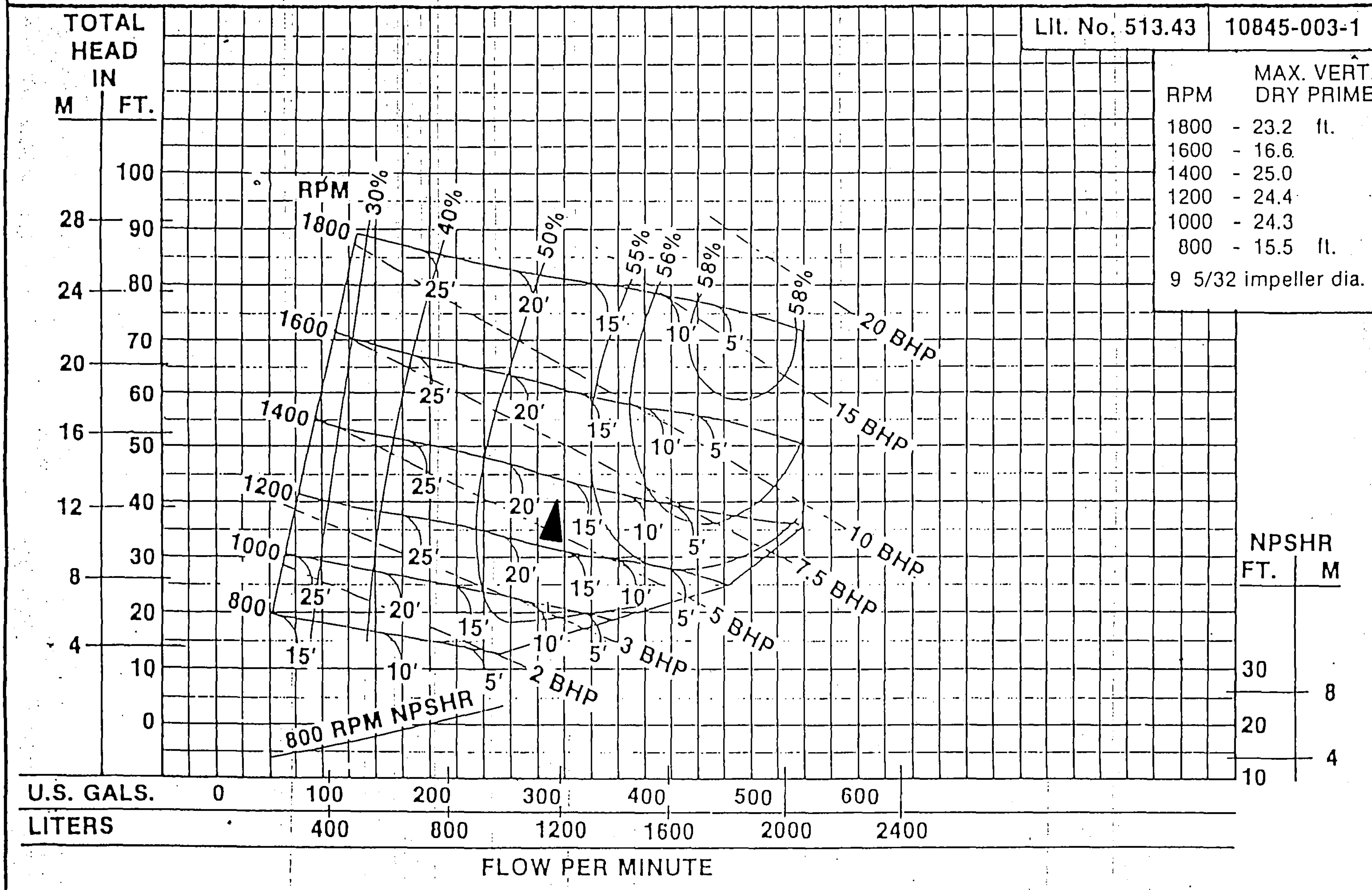
JOB # N/A		DATE 9-13-95	
Des By	MW	Dwg #	DEQ-0007
Drn By	SM	Sheet	1 of 1
Chk By	MW	Rev: 1	2-5-96 Chk: MW

HYDR-O-MATIC PUMPS

SECTION 510
PERFORMANCE DATA

300 G.P.M. @ 42' T.D.H.

MODEL: 40MPV - 40MPSFV SELF PRIMER - MAX. SOLIDS: 3" SPHERE - VAR. RPM



D-5:
Seven Oaks Pump Station

Pump Station Data Sheet

Location: Scappoose, Oregon **Serial No.** 48526

Owner: City of Scappoose **Engineer:** Koss-Brod-Goodrich & Assoc.

Contractor: Bones Construction **Distributor:** Hydronix

Station Type 183 **Wet Well** 84 **Dia. x** 16' **Deep Wet Well Cover**

Date Installed 4/96

Pump Station

Piping: Suction 4" **,Disch.** 4" **; Suction** 4" **,Disch.** 4"

Conditions of Service

Design Duty: 150**GPM,** 45 **TDH,__Suction Lift; Liquid** Sewage

Solids 3", **NPSH (avl)**_______, **(req)**_______.

Driven RPM 1350

Sheave 3V5.3x2 Gr. **,Bushing** SH 1-1/2 **,Belt** 3VX400

Pump Data:

Serial No.:____________

Pump Model: 40MP **,Imp.Dia.** 9 5/32 **Priming:**(X)**self,**( )**vacuum**

( )**flooded. Stuffing Box Type**_____, **Lubrication:** Media

Rotation of Pump #1 CW **,#2** CW **,#3**__.

Motor Data

Brand_Baldor_,Enclosure_ODP_,Horsepower_7.5_

RPM_1750_,Phase_3_,Cycle_60_,Volt_230_,Starting Code__

Modification__________,Frame___

Sheave_3V4.12x2 Gr._,Bushing_SH 1 3/8_,Belt_3VX400_

Electrical Data

Phase_3_,Cycle_60_,Volts_230_,Wire_4_,Control Voltage_120_

Transformer Required: (X)Yes , () No. Size_E180-3PB_

Control Panel

Location_Inside Station_NEMA Type_1_

Service Entrance Size___Amps	Pump #1	Pump #2
Circuit Breaker-Trip Rating/AMPS	50	50
Magnetic Starter-NEMA size	AE16FNSOAC	AE16FNSOAC
Overload Heater size	H2013B-3	H2013B-3

Alternator Type_67C1COA_Mfg_Warrick_Coil Voltage___

Alarm Functions:_Controlled by CB-4 AutoDialer_

Level Control:_Mercury Float Switches_

Elevations:

Ground Level_13.91_	Low Water_________
Station Disch._8.91_	Pump Off_-0.5_
Station Floor_-3.0_	Lead Pump On_3.07_
WW Invert_4.56_	2 Pump Overload_3.57_
WW Floor__________	High Water_4.07_
	OVERFLOW 10.96

Station Options

(X) 1500 Watt Heater,(X)Station Blower,( )Wet Well Blower,()P.P.D. Dehumidifier,(X)Station Light,(X)Trouble Light,( )Vacuum Pumps, (X)Convenience Outlet,( )__KVA Transformer,()Sump Pump, ()Ladder,(X)Elapsed Time Meters,( )__VDC Battery and Charger, (X)Light/Horn/Bell/Dim Glow Alarm,( )Dry Alarm Contacts, ( )Telemetry Included/By Others,(X)Air Compressor, (X)Pinch Valve, (X) 4 O&M Manual, (X)Spare Parts Kit.

Notes: System has DEQ approved 4" Pinch Valve on discharge line

Application Engineering

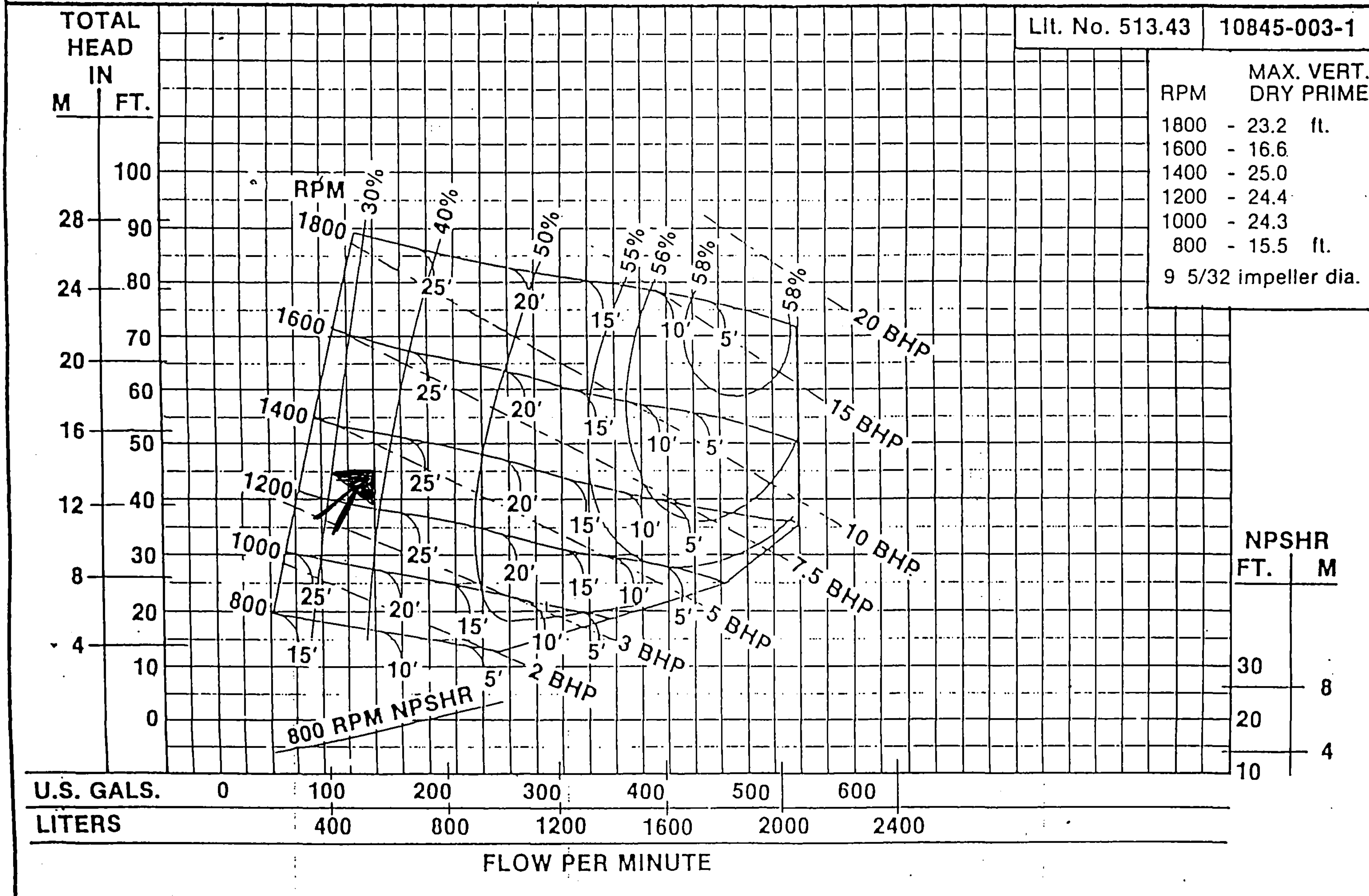
Prepared By Butch Kline **Date** 4/22/96

HYC -O-MATIC PUMPS

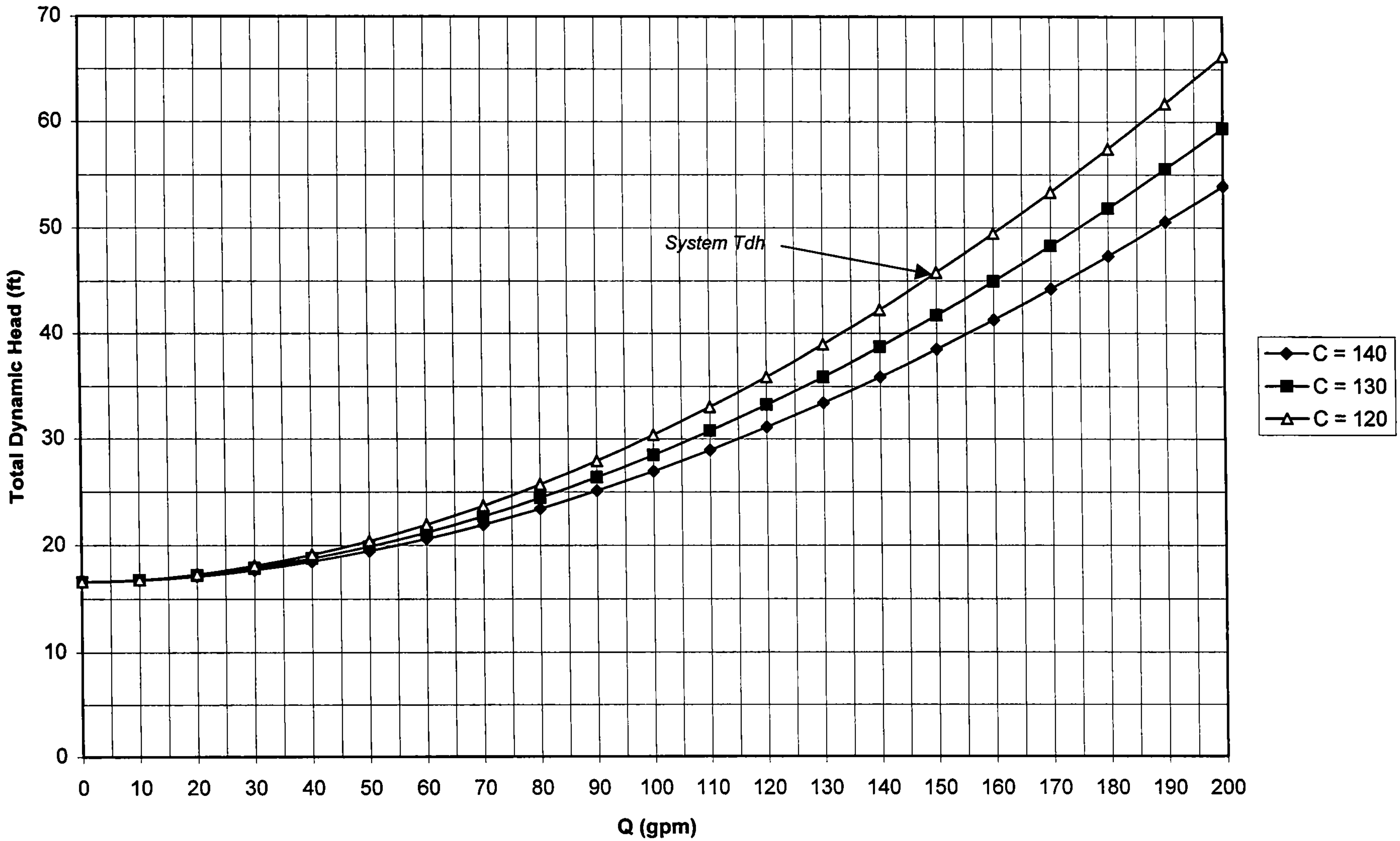
SEC ON 510
PERFORMAN : DATA

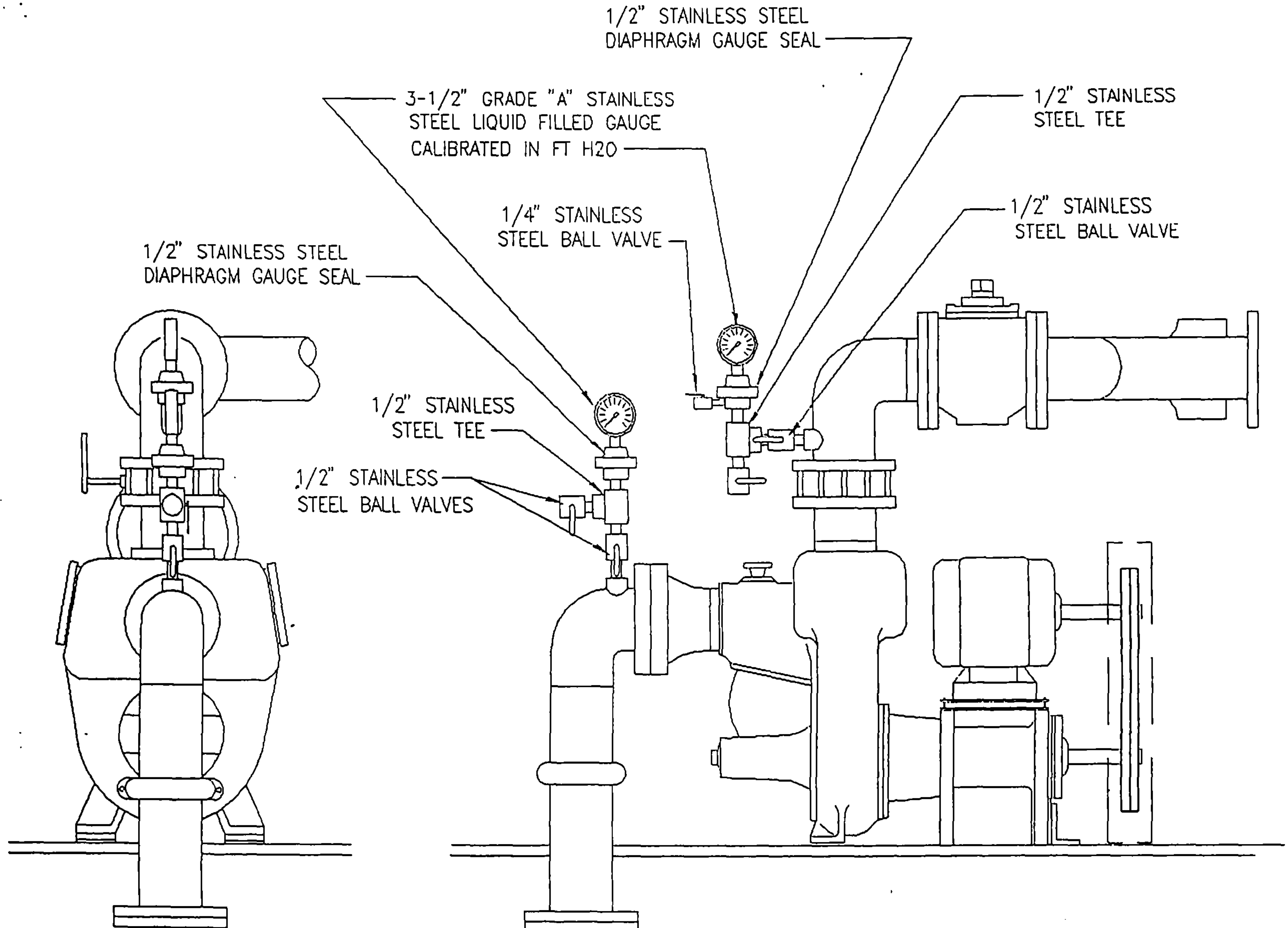
145 G.P.M. @ 45 T.D.H.

MODEL: 40MPV - ~~40MPSFV~~ SELF PRIMER - MAX. SOLIDS: 3" SPHERE - VAR. RPM



System Curves for Varies C Values





SUCTION AND DISCHARGE GAUGE KIT

JOB #	N/A	DATE	9-13-95
Des By	MW	Dwg #	DEQ-[illegible]07
Drn By	SM	Sheet	1
Chk By	MW	Rev: 1	2-5-96 · Chk: MW

CONTROL PANEL NOTES:

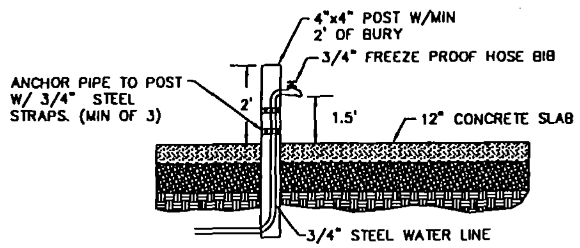
1. ALL ELECTRICAL INSTALLATIONS WILL BE IN CONFORMANCE WITH THE NATIONAL ELECTRICAL CODE AND STATE AND LOCAL REQUIREMENTS.
2. ELECTRIC COMPONENTS SHOULD BE SIZED TO ACCOMMODATE ACTUAL HORSEPOWER REQUIREMENTS.

CLEARLY LABEL ALL CONTACTS AND COMPONENTS.

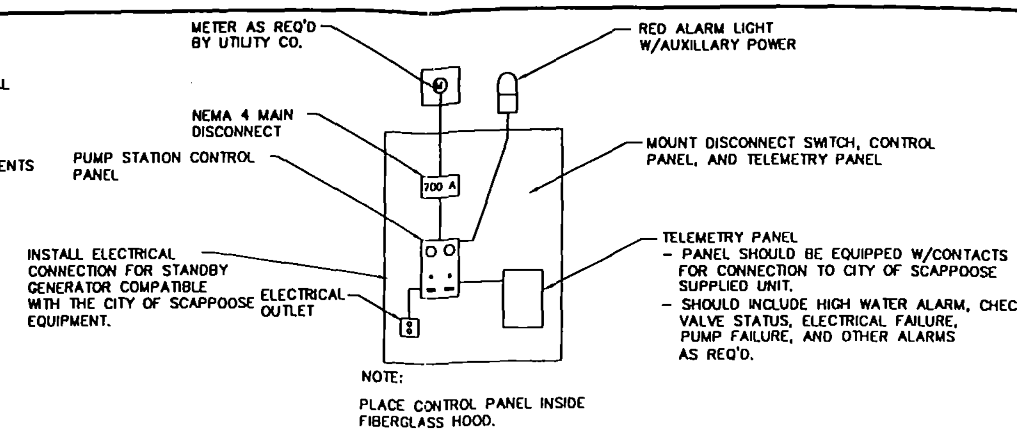
THE ELECTRICAL CONTROL PANEL ENCLOSURE SHALL BE NEMA FOUR WITH MATERIAL BEING FIBERGLASS TO PREVENT PREMATURE CORROSION.

5. THE PANEL WILL CONTAIN TWO CIRCUIT BREAKERS (ONE FOR EACH PUMP), ELECTRIC ALTERNATOR, CONTROL POWER TRANSFORMER, TERMINAL STRIP FOR CONNECTING FOUR (4) FLOAT SWITCHES, REMOTE ALARM AND TELEMETERY HOUR METERS FOR EACH PUMP EXTERNAL FLASHING ALARM LIGHT TO BE LOCATED ON OUTSIDE ON ELECTRICAL PANEL.
6. ALARM SYSTEM SHALL INCLUDE A SILENCING RESET ON THE PANEL.
7. THE PANEL SHALL HAVE A LOCKABLE DEAD FRONT DOOR WITH RUN LIGHTS AND H.O.A. SWITCHES INSIDE FOR EASY OPERATION.
8. PROVIDE CONNECTION FOR PORTABLE GENERATOR.
9. PUMP CONTROL SYSTEM SHALL BE FLOATS UNLESS OTHERWISE APPROVED BY CITY OF SCAPPOOSE.

NOTES:
- ALL ELECTRICAL INSTALLATIONS WILL BE IN CONFORMANCE WITH THE NATIONAL ELECTRICAL CODE AND STATE AND LOCAL REQUIRMENTS.
- ELECTRICAL COMPONENTS SHOULD BE SIZED TO ACCOMMODATE ACTUAL HORSEPOWER REQUIREMENTS.
- CLEARLY LABEL ALL CONTACTS AND COMPONENTS



WATER SERVICE
N.T.S.



CONTROL PANEL
N.T.S.

PUMP STATION NOTES

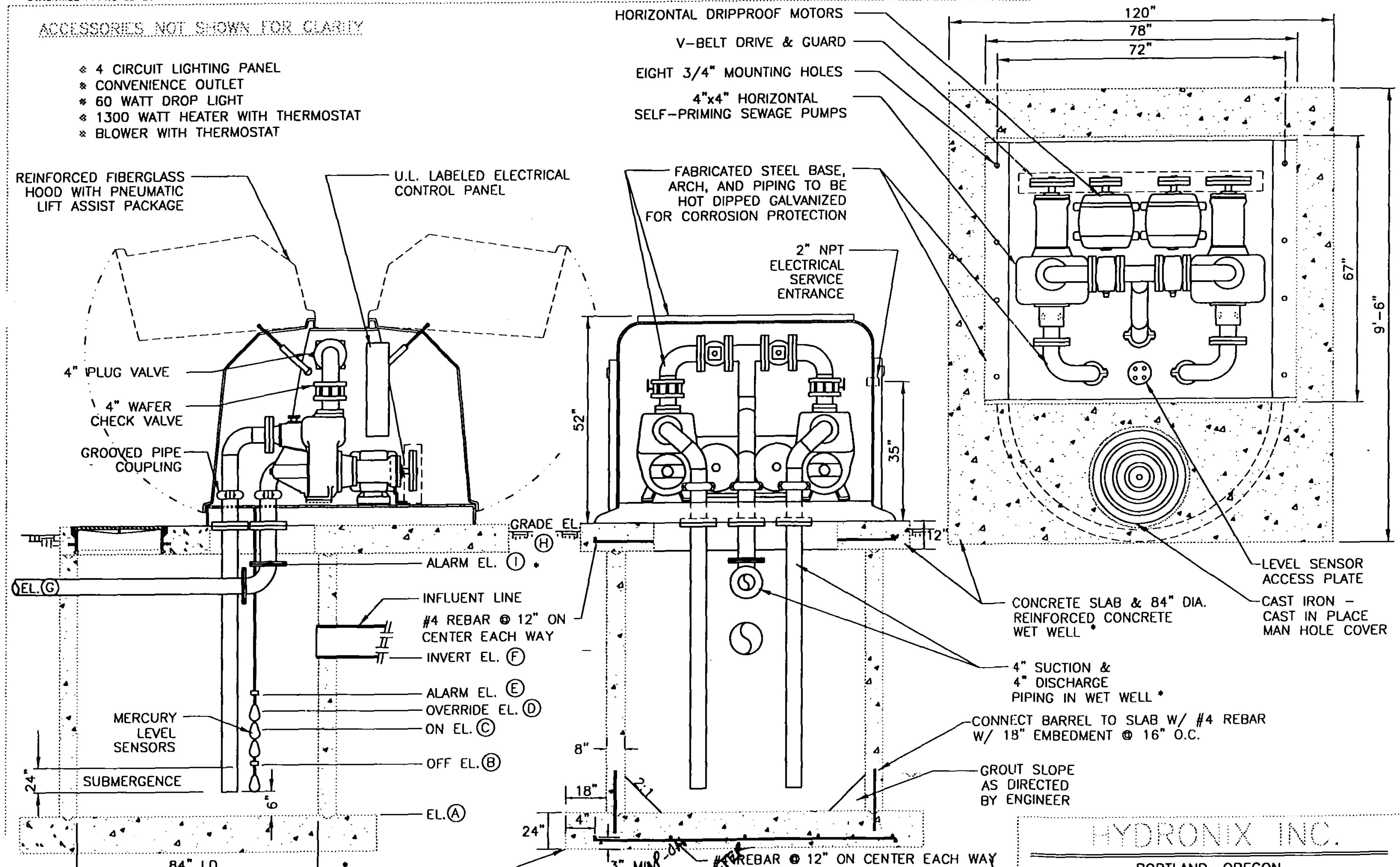
1. THE CONTRACTOR SHALL FURNISH AND INSTALL TWO SURFRACE MOUNTED SELF PRIMING SEWAGE PUMPS AS INDICATED ON THE DRAWINGS AND SPECIFIED HEREIN. PUMP SYSTEM SHALL BE SUPPLIED BY HYDRONIX PUMP SERVICE OR APPROVED EQUAL.
2. PUMP DESIGN: THE PUMPS SHALL BE V-BELT DRIVEN SELF PRIMING SEWAGE PUMPS FOR SURFACE MOUNTED INSTALLATION.
3. THE PUMP MOTOR SHALL BE HOUSED IN AN AIR-FILLED WATER-TIGHT CASING WITH CLASS F INSULATION AND A 1.20 SERVICE FACTOR. BUILT IN MOTOR PROTECTION SHALL CONSIST OF ONE BY METALLIC MICRO SWITCH IN EACH PHASE OF THE WINDING.

   MOTORS SHALL BE NON-OVERLOADING THROUGHOUT THE ENTIRE PUMP OPERATING RANGE. THE MOTOR MUST BE SUITABLE FOR USE IN CLASS 1, DIVISION 1, GROUPS C&D AREAS. OIL FILLED MOTORS SHALL NOT BE CONSIDERED EQUAL OR ACCEPTABLE.
4. ALL CABLE ENTRY JUNCTION BOXES SHALL BE SEPARATED FROM THE MOTOR BY A TERMINAL BOARD AND THE CABLE ENTRY WATER SEAL SHALL NOT REQUIRE EPOXIES, SILICONES, OR OTHER SECONDARY SEALING SYSTEMS.
5. SYSTEM REQUIREMENTS: EACH PUMP SHALL HAVE THE FOLLOWING CHARACTERISTICS:

   CAPACITY 150 (GPM) ~~TDH 38 (FT)~~ 45
6. PUMP WARRANTY: THE PUMPS ARE TO HAVE A ONE-YEAR NON-PRORATED WARRANTY WHICH COVERS DEFECTS IN MATERIALS AND WORKMANSHIP.
7. SEE ATTACHED SPECIFICATION PACKET FOR MORE INFORMATION.
8. PIPING AROUND STRUCTURES AND MACHINERY SHOULD BE D.I. PIPE. PVC IS NOT ACCEPTABLE.
9. A DELAY TIMER WILL BE SET TO OPEN THE PINCH VALVE ON THE DRAIN BACK LINE. THE TIME WILL BE CHANGEABLE BY THE CITY WITH A 30 MINUTE MAXIMUM TIME.
10. THE DISCHARGE MANHOLE WILL BE COATED WITH A LINER TO PROTECT FROM DAMAGE FROM HYDROGEN SULFIDE.
11. A 12' WIDE GRAVEL ROAD ALONG THE FUTURE RIGHT-OF-WAY WILL PROVIDE ALL-WEATHER ACCESS TO THE PUMP STATION UNTIL PHASE II HAS BEEN COMPLETED.
12. PRESSURE GAGES WILL BE PROVIDED ON THE SUCTION AND DISCHARGE SIDE OF THE EACH PUMP WITH THE APPROPRIATE VALVING TO ISOLATE EACH GAGE.
13. THE WETWELL NEEDS TO BE PLACED ON 12" OF 3/4"-0 CRUSHED ROCK COMPACTED TO 95% OF AASHTO T-180. AND THE WETWELL WILL BE BACKFILLED WITH 3/4"-0 CRUSHED ROCK.
14. PROVIDE A POWER METER, MAIN DISCONNECT AND MANUAL TRANSFER SWITCH.
15. EQUIP LIFT STATION WITH A 60W CAGE LIGHT ON BOTH SIDES OF THE CENTRAL SUPPORT, A 1300W HEATER AND A DUAL WEATHER PROOF DUPLEX OUTLET.
16. BASE OF WETWELL WILL BE TIED TO SIDES OF WETWELL WITH #4 REBAR AT 12" ON CENTER. TO KEEP THE BASE FROM SEPERATING FOR THE SIDES.

ACCESSORIES NOT SHOWN FOR CLARITY

- 4 CIRCUIT LIGHTING PANEL
- CONVENIENCE OUTLET
- 60 WATT DROP LIGHT
- 1300 WATT HEATER WITH THERMOSTAT
- BLOWER WITH THERMOSTAT



SCALES & DIMENSIONS ARE FOR REFERENCE ONLY. DRAWINGS MUST BE CERTIFIED CORRECT FOR CONSTRUCTION PURPOSES

STATION	G.P.M.	T.D.H.	H.P.	R.P.M.	PHASE	VOLTS	ELEVATIONS								
							A	B	C	D LAG PUMP ON	E HIGH WATER	F	G	H	I OVERFLOW
7 OAKS	150	45	7.5	1300	3ø	120-240	-3.00	-0.50	3.07	3.57	4.07	4.56	8.91	13.91	10.96

HYDRONIX INC.
PORTLAND, OREGON

MODEL 183 V-BELT DRIVEN SELF PRIMING SEWAGE PUMPING STATION

SCALE: N.T.S.
DRAWING NUMBER: 183V01.DWG

MNWR|P
PLANNERS
ENGINEERS
LANDSCAPE ARCHITECTS
233 SW FRONT
PORTLAND, OR
97204
☎ 503|225|0822
FAX: 503|273|8353

REGISTERED PROFESSIONAL ENGINEER 15,190
OREGON
OCT 2, 1990
DARREN MARK WELBORN

SEVEN OAKS SUBDIVISION
PUMP STATION DETAILS

KOSS-BROD-GOODRICH AND ASSOCIATES
22595 DAY RD. WEST LINN, OREGON 97068

REV.	DATE	BY
1	11/1/95	SR

PROJECT NO.
KBG001

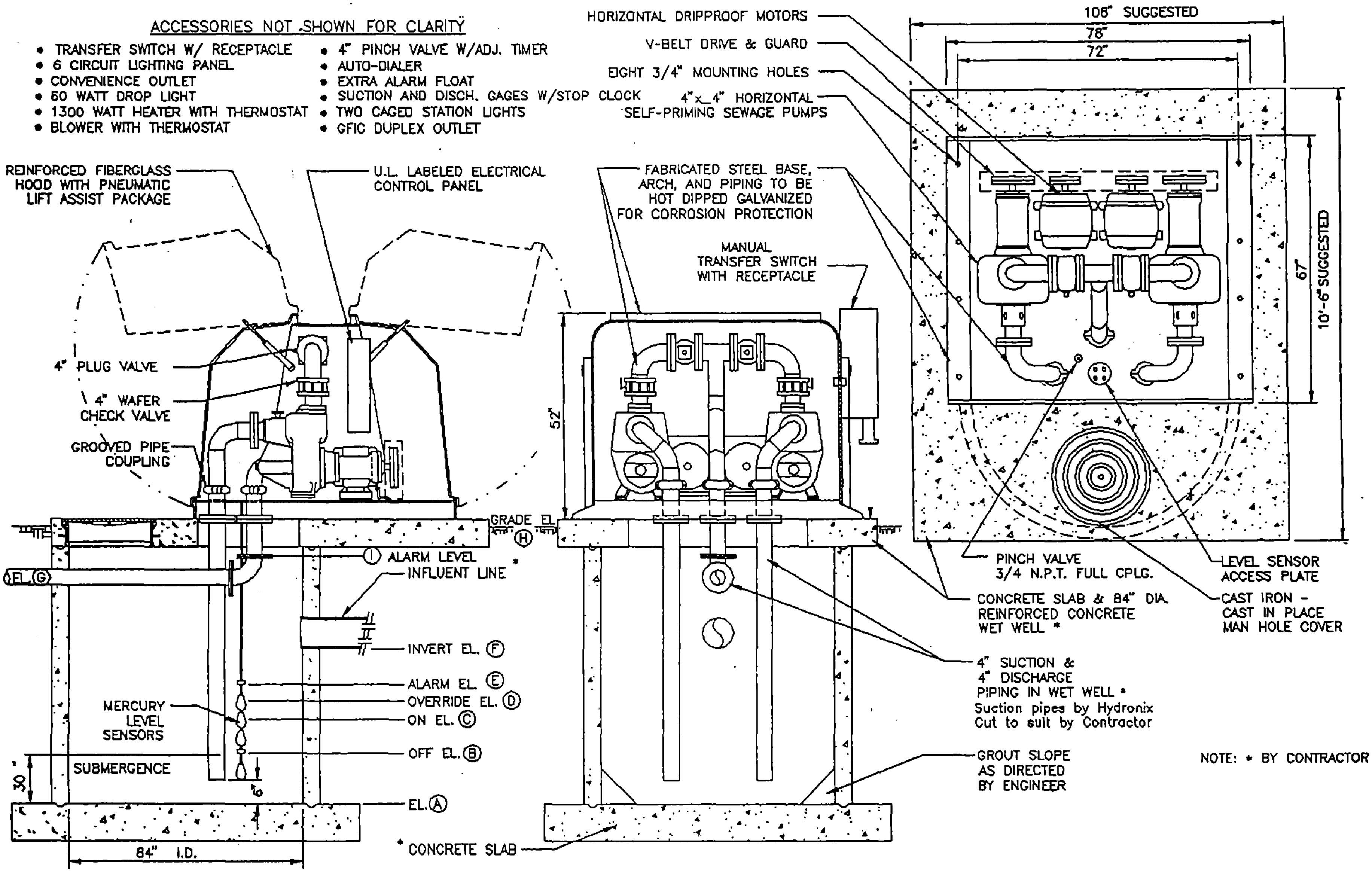
DATE 10/19/95
DESIGNED K. ACKERMAN
ENGINEER D. WELBORN
CHECKED J. REIMANN

SHEET TITLE
DETAILS

SHEET NUMBER
C13A

ACCESSORIES NOT SHOWN FOR CLARITY

- TRANSFER SWITCH W/ RECEPTACLE
- 6 CIRCUIT LIGHTING PANEL
- CONVENIENCE OUTLET
- 60 WATT DROP LIGHT
- 1300 WATT HEATER WITH THERMOSTAT
- BLOWER WITH THERMOSTAT
- 4" PINCH VALVE W/ADJ. TIMER
- AUTO-DIALER
- EXTRA ALARM FLOAT
- SUCTION AND DISCH. GAGES W/STOP CLOCK
- TWO CAGED STATION LIGHTS
- GFIC DUPLEX OUTLET

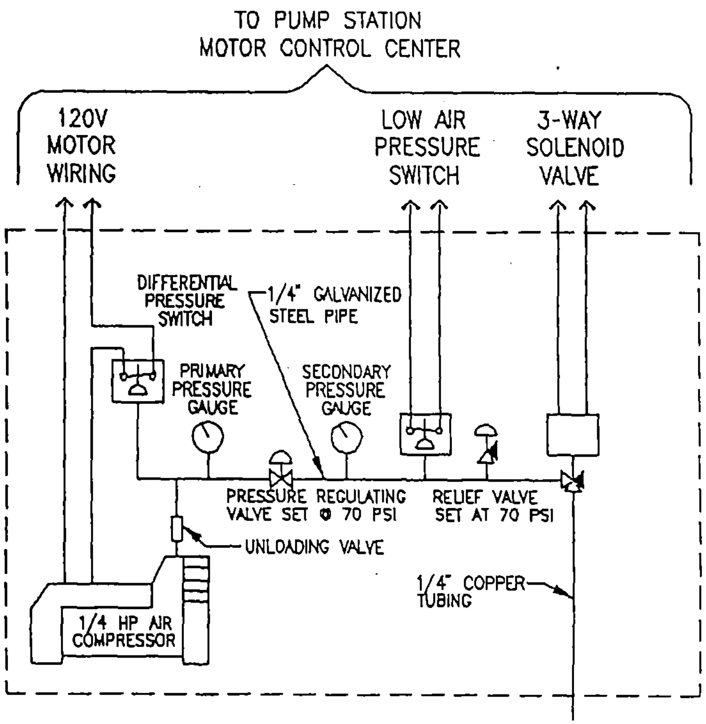


SCALES & ¹NSIONS ARE FOR REFERENCE ONLY, DRAWINGS MUST BE CERTIFIED CORRECT FOI ˙TRUCTION PURPOSES

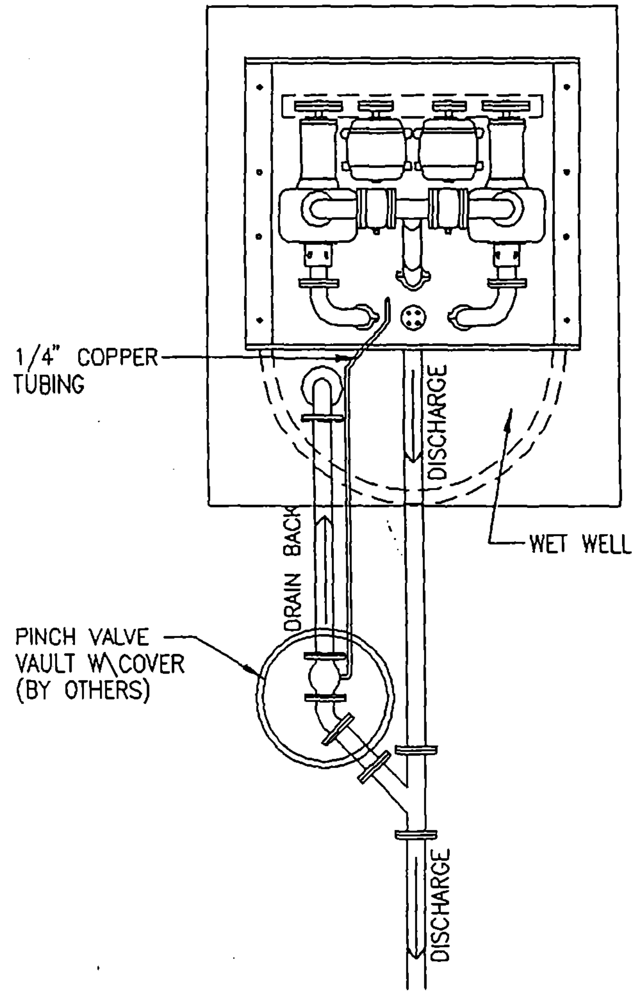
STATION	.	T.D.H.	H.P.	R.P.M.	PHASE	VOLTS	ELEVATIONS A	B	C	D	E		G	H	I
7 OAKS	150	45	7.5	1750	3	230	-3.0	-0.5	+3.07	+3.57	+4.07	+4.56	+8.91	+13.91	+10.91

SEVEN OAKS PUMP STATION
SCAPPOOSE, OREGON

MODEL # 183-010	JOB # 48526		
	Des By	JH	D· 2-6-95
DWG # 183-010	Drn By	BT	
	Chk By	JH	Sheu. 1 of 1



H2S DRAINBACK CONTROL SCHEMATIC



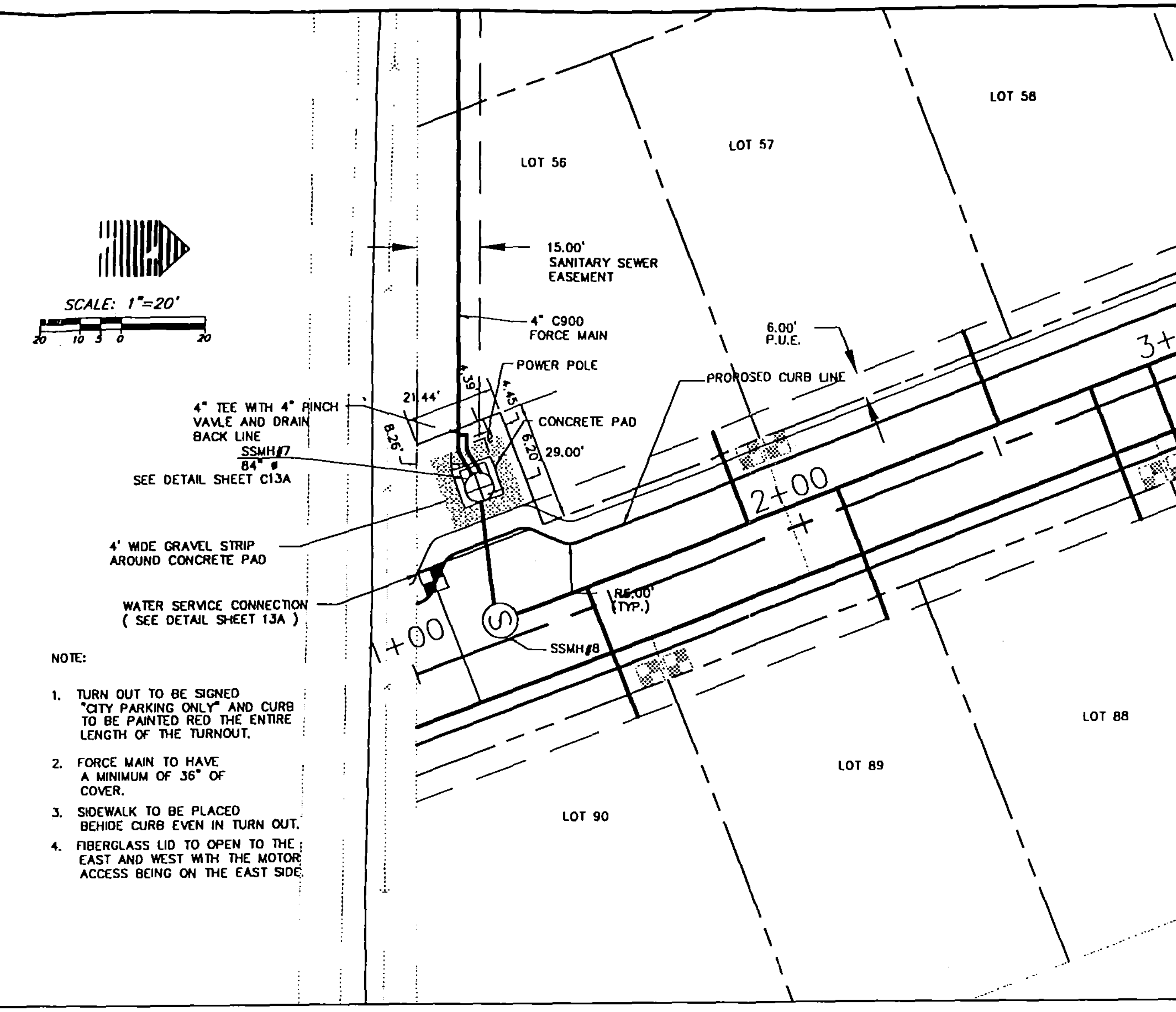
H2S TREATMENT DRAINBACK SYSTEM

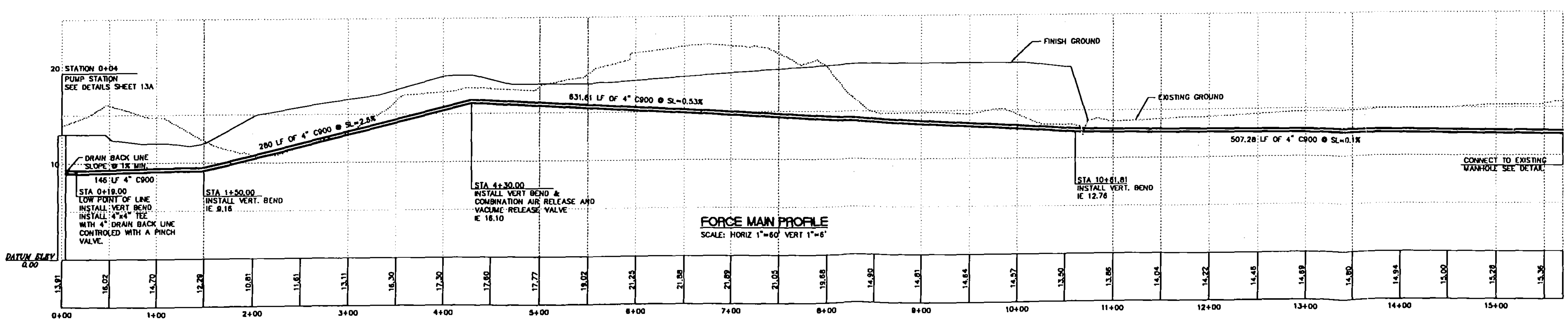
HYDRONIX	JOB #	N/A	DATE	3-10
	Des By	MW	Dwg #	DEQ-0
	Drn By	SM	Sheet	1 of

SEWAGE PUMP STATION DESIGN DATA

PUMP STATION	**LOCATED • SOUTH END OF 9TH STREET**
TYPE	SURFACE MOUNTED SELF PRIMING
PUMP TYPE	CONSTANT-SPEED NON-CLOG
CAPACITY	150 GPM @ 45FT TDH
PUMP HP	7.5 HP
LEVEL CONTROL	FLOATS
OVERFLOW POINT	SSMH#9 ELEV=10.96 (INT. OF SEVEN OAKS DR. & 9TH STREET)
OVERFLOW DISCHARGE	STORM SYSTEM ON SITE
AVG. TIME TO OVERFLOW	15.75 HRS
AUXILIARY POWER TYPE	3ø GENERATOR
LOCATION	CITY SHOPS
OUTPUT	N/A
FUEL CAPACITY	N/A
TRANSFER SWITCH	MANUAL
ALARM TELEMETRY TYPE	AUTODIALER
EPA RELIABILITY CLASS	II
FORCE MAIN	
LENGTH, TYPE	1,565LF OF C900
PROFILE	ASCENDING TO STA. 4+30 THEN CONTINUOUS DECENDING
DISCHARGE MANHOLE	SIXTH STREET
VACUUM RELEASE VALVE	ONE AT HIGH POINT (STA 4+30)
AVERAGE DETENTION	85 MIN
SULFIDE CONTROL SYSTEM	BACKDRAIN
BACKDRAIN SYSTEM	
CONTROL VALVE TYPE	PNUMATIC PINCH
VALVE SIZE	4"

NOTE:
BEFORE INSTALLATION, THE SIZE AND MATERIAL OF THE PINCH VALVE VAULT AND COVER NEEDS TO BE REVIEWED BY THE CITY OF SCAPPOOSE.





FORCE MAIN PROFILE
SCALE: HORIZ 1"=60' VERT 1"=6'

MNWR|P
PLANNERS
ENGINEERS
LANDSCAPE ARCHITECTS
233 SW FRONT
PORTLAND, OR
97204
☎ 503|225|0822
FAX: 503|273|8353

REGISTERED PROFESSIONAL ENGINEER 15,190
OREGON
OCT 2, 1990
DARREN MARK WELBORN
EXP 6/30/95

SEVEN OAKS SUBDIVISION
FORCE MAIN PROFILE \ PUMP STATION PLAN

KOSS-BROD-GOODRICH AND ASSOCIATES
22595 DAY RD. WEST LINN, OREGON 97068

REV.	DATE	BY

PROJECT NO.
KBG001

DATE	10/26/95
DESIGNED	K. ACKERMAN
ENGINEER	D. WELBORN
CHECKED	J. REIMANN

SHEET TITLE
PROFILE
SHEET NUMBER
C13

APPENDIX E – Smoke Testing & CCTV

E-1:
Smoke Testing Results Table

Picture ID	Date	MH Tested	Address	Defect Type	Recommended Action	Photo
1	7/20/2016	0355	32849 NW Bella Vista Dr	open C/O	Notify property owner, seal C/O	Y
2	7/20/2016	0386	53094 NW 11th St	no smoke	Investigate, notify property owner	N
3	7/20/2016	0386	32969 NW Bella Vista Dr	no smoke	Investigate, notify property owner	N
4	7/20/2016	0438	52828 NW Five Peak Ter	open C/O	Notify property owner, seal C/O	Y
5	7/20/2016	0225	52811 NE View Ter	indoor (toilet)	Notify property owner	N
6	7/20/2016	0410	52780 NW Willow Ln	open C/O	Notify property owner, seal C/O	Y
7	7/20/2016	0410	52770 NW Willow Ln	leaking C/O	Notify property owner, seal C/O	Y
8	7/20/2016	0410	52895 NE 7th St	cross connection (downspout)	Notify property owner, remove cross connection	Y
9	7/21/2016	0391	52859 NW 1st St	no smoke	Investigate, notify property owner	N
10	7/21/2016	0391	33349 NW Wickstrom St	indoor (dry P-trap)	Notify property owner	N
11	7/21/2016	0108	52122 Hoag Ter	cross connection (downspout)	Notify property owner, remove cross connection	Y
12	7/21/2016	0109	33114 Felisha Wy	cross connection (driveway drain)	Notify property owner, remove cross connection	Y
13	7/21/2016	0050	Ashley Ct and JP West Rd	open C/O	Under construction, in future check is sealed	Y
14	7/21/2016	0462	32676 JP West Rd	open C/O	Notify property owner, seal C/O	Y
15	7/21/2016	0472	MH 0469	MH	Re-grout or replace MH rim	Y
16	7/21/2016	0472	MH 0479	MH	Re-grout or replace MH rim	N
17	7/26/2016	0632	52239 SW Keys Rd	indoor (dry P-trap)	Notify property owner	N
18	7/26/2016	0346	33343 SW Rogers Rd	cross connection (downspout)	Notify property owner, remove cross connection	Y
19	7/26/2016	0428	52844 NE 2nd St	open C/O	Notify property owner, seal C/O	Y
20	7/26/2016	0425	52657 NE 3rd St	open C/O	Notify property owner, seal C/O	Y
21	7/26/2016	0425	33318 Royal Dr	open C/O	Notify property owner, seal C/O	Y
22	7/26/2016	0607	MH 0614	MH	Re-grout or replace MH rim	Y
23	7/26/2016	0292	33264 Julie Ct	broken C/O	Notify property owner, seal C/O	Y
24	7/26/2016	0594	52313 Columbia River Hwy	indoor (bathroom)	Notify property owner	N
25	7/26/2016	0594	334019 SW Maple St	open C/O (and indoor, washer)	Notify property owner, seal C/O	N
26	7/27/2016	0042	52753 NE Kern Ct	open C/O	Notify property owner, seal C/O	Y
27	7/27/2016	0042	33790 NE Kern Ct	open C/O	Notify property owner, seal C/O	Y
28	7/27/2016	0042	MH 0041	MH	Re-grout or replace MH rim	Y
29	7/27/2016	0174	34371 Egret Ln	open C/O	Notify property owner, seal C/O	Y
30	7/27/2016	0140	51704 SE 4th St	broken C/O	Notify property owner, seal C/O	Y

E-2:
Smoke Testing Results

1. 32849 NW Bella Vista Dr
Open C/O

2. 53094 NW 11th St
No smoke out of vents

No Picture

3. 32969 NW Bella Vista Dr
No smoke out of vents

No Picture

4. 52828 NW Five Peak Ter
Open C/O (around back of house)

5. 52811 NE View Ter
Indoor; toilet smoking from at connection with floor

No Picture

6. 52780 NW Willow Ln
Open C/O

7. 52770 NW Willow Ln
Leaking C/O cap

8. 52895 NE 7th St

Cross connection; downspout on front porch smoking

9. 52859 NW 1st St

No smoke out of vents

No Picture

10. 33349 NW Wickstrom St
Indoor; dry P-trap in bathroom

No Picture

11. 52122 Hoag Ter

Cross connection; downspout, driveway drain smoking

12. 33114 Felisha Way
Cross connection; driveway drain connected to sewer line

13. Ashley Ct and JP West Rd (SE corner house)
Open C/O in front; under construction, will most likely be sealed later on

14. 32676 JP West Rd (SW Taylor St side of house)
Open C/O

15. MH0469
MH rim smoking

16. MH0479
MH rim looks to be smoking; in field

No Picture

17. 52239 SW Keys Rd
Indoor; dry P-trap

No Picture

18. 33343 SW Rogers Rd

Cross connection; downspout over garage smoking

19. 52844 NE 2nd St
Open C/O

20. 52657 NE 3rd St (around back side of house)
Open C/O

21. 33318 Royal Dr
Open C/O

22. MH0614
MH rim smoking

23. 33264 Julie Ct
Broken C/O

24. 52313 Columbia River Hwy (US Bank)
Indoor; light smoke in the bathroom

No Picture

25. 334019 SW Maple St
Open C/O; indoor, washer drain smoking

No Picture

26. 52753 NE Kern Ct
Open C/O (in metal valve can, but uncapped inside can)

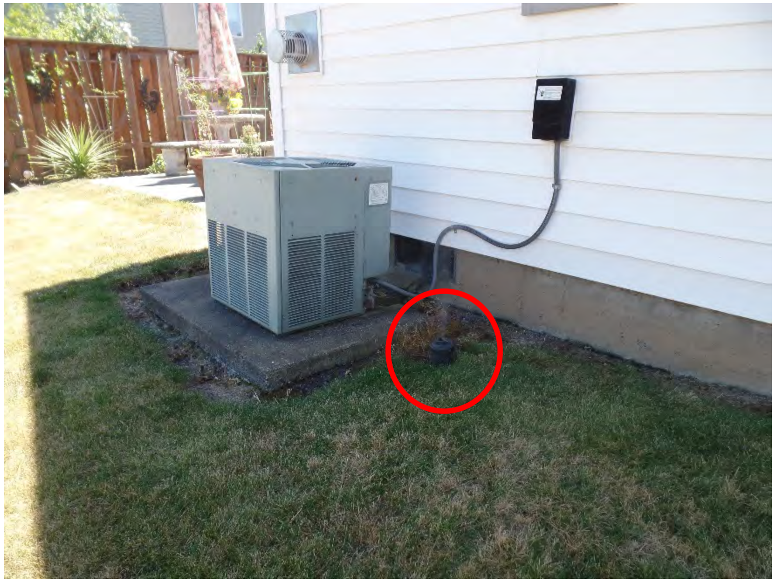
27. 33790 NE Kern Ct
Open C/O

28. MH0041

MH rim smoking (in addition to smoking from holes in lid)

29. 34371 Egret Ln
Open C/O

30. 51704 SE 4th St
Broken C/O



APPENDIX F – Model Data & Results

F-1:
Existing Link Data

Existing System Flows (2015; 5-year, 24-hour storm event)

Link Name	US Invert (ft)	DS Invert (ft)	Installation Year	Material	Roughness	Length (ft)	Diameter (ft)	Slope	Max Velocity (ft/s)	Min Velocity (ft/s)	Max Flow (cfs)	Design Full Flow (cfs)	Max d/D
GM0048	6.6	5.8	2006	PVC	0.011	201.21	1.25	0.4	1.08	0.76	0.035	4.81	0.063
GM0050	7.9	6.6	2006	PVC	0.011	292.23	0.833	0.44	1.17	0.8	0.028	1.73	0.094
GM00554	34.4	34.2			0.014	180.5	0.667	0.11	1.75	0	0.615	0.37	2.469
GM0062	22.11	19.77	2007	PVC	0.011	331.89	1.75	0.71	0	0	0	15.72	0
GM0063	19.52	18.09	2007	PVC	0.011	375.15	1.75	0.38	0	0	0	11.56	0
GM0064	17.89	16.16	2007	PVC	0.011	370.91	1.75	0.47	0	0	0	12.79	0
GM0140	48.94	48.62	2001	PVC	0.011	76	0.833	0.42	0.56	0	-0.113	1.68	3.433
GM0141	48.42	47.45	2001	PVC	0.011	242.29	0.833	0.4	1.27	0	-0.18	1.64	4.837
GM0142	47.25	46.97	2001	PVC	0.011	64.95	0.833	0.43	1.41	0	-0.187	1.7	5.413
GM0143	46.1	45.97	2001	PVC	0.011	33.12	0.833	0.39	2	0	0.193	1.62	6.611
GM0144	46.77	46.5	2001	PVC	0.011	118.56	0.833	0.23	1.63	0	0.193	1.23	5.975
GM0161	14.13	14.01	2000	PVC	0.011	23.02	0.833	0.52	1.32	0.23	0.04	1.87	0.13
GM0162	13.91	13.86	2000	PVC	0.011	104	0.833	0.05	0.92	0.14	0.094	0.57	0.25
GM0163	13.76	12.76	2000	PVC	0.011	400	0.833	0.25	1.35	0.25	0.105	1.29	0.204
GM0164	12.66	12.46	2000	PVC	0.011	79.17	0.833	0.25	1.43	0.26	0.115	1.3	0.208
GM0165	12.35	12.3	2000	DI	0.012	20	0.833	0.25	1.45	0.26	0.115	1.19	0.206
GM0224	35.42	33.93	1996	PVC	0.011	362.58	0.833	0.41	1.44	0.27	0.065	1.66	0.138
GM0225	37.22	35.62	1996	PVC	0.011	405.28	0.833	0.39	1.37	0.24	0.058	1.63	0.132
GM0237	51.7	51.52	1995	PVC	0.011	71.1	1	0.25	0.4	0.1	0.074	2.12	0.443
GM0238	52.53	51.8	1995	PVC	0.011	290.21	1	0.25	1.2	0.65	0.074	2.11	0.166
GM0239	52.93	52.63	1995	PVC	0.011	118.2	1	0.25	1.16	0.65	0.066	2.12	0.128
GM0241	53.21	53.03	1995	PVC	0.011	71.22	1	0.25	1.06	0.59	0.048	2.12	0.109
GM0242	53.47	53.31	1995	PVC	0.011	61.98	1	0.26	1.07	0.6	0.048	2.14	0.108
GM0243	54.05	53.57	1995	PVC	0.011	175.44	1	0.27	1.05	0.58	0.045	2.2	0.104
GM0244	54.48	54.05	1995	PVC	0.011	170.77	1	0.25	0.96	0.54	0.032	2.11	0.104
GM0245	54.83	54.51	1995	PVC	0.011	126.75	1	0.25	0.78	0.41	0.019	2.12	0.071
GM0276	36.76	36.36	1996	C	0.013	206.3	1	0.19	3.24	1.59	2.642	1.57	6.899
GM0374	51.84	51.34	1995	PVC	0.011	120.32	1	0.42	1.55	0.86	0.085	2.71	0.123
GM0375	51.14	50	1995	PVC	0.011	228.43	1	0.5	1.75	0.38	0.1	2.97	0.577
GM0376	49.4	48.75	1995	PVC	0.011	155.28	1	0.42	1.77	0.03	0.229	2.72	1.818
GM0377	49.8	49.6	1995	PVC	0.011	49.03	1	0.41	1.72	0	0.187	2.69	0.974
GM0409	60.55	59.55	1988	PVC	0.011	250	0.667	0.4	2.76	0.16	0.599	0.9	0.628
GM0410	59.55	58.53	1988	PVC	0.011	272.67	0.667	0.37	2.59	0.2	0.549	0.87	0.596
GM0411	58.36	57.4	1988	PVC	0.011	354.18	0.833	0.27	2.35	0.23	0.561	1.35	0.461
GM0412	57.4	56.35	1988	PVC	0.011	358.33	0.833	0.29	2.34	0.26	0.533	1.4	0.458
GM0413	55.25	55.2	1988	PVC	0.011	63.77	0.833	0.08	1.84	0.19	0.461	0.72	0.48
GM0414	56.35	55.35	1988	PVC	0.011	312.25	0.833	0.32	2.33	0.26	0.457	1.46	0.396
GM0417	12.19	11.54	1986	PVC	0.011	329.67	1	0.2	1.62	0.5	0.276	1.87	0.275
GM0418	11.48	10.54	1986	PVC	0.011	179.05	1	0.52	2.18	0.3	0.287	3.05	0.588
GM0419	10.54	10.49	1986	C	0.013	21.37	1	0.23	2.52	0.62	1.202	1.72	0.588
GM0446	44.24	43.86	1981	PVC	0.011	88.82	0.667	0.43	2.52	0.55	0.379	0.94	0.447
GM0447	45.35	44.34	1981	PVC	0.011	254.46	0.667	0.4	2.41	0.51	0.375	0.9	0.46

Link Name	US Invert (ft)	DS Invert (ft)	Installation Year	Material	Roughness	Length (ft)	Diameter (ft)	Slope	Max Velocity (ft/s)	Min Velocity (ft/s)	Max Flow (cfs)	Design Full Flow (cfs)	Max d/D
GM0448	46.91	45.45	1981	PVC	0.011	362.8	0.667	0.4	2.44	0.51	0.382	0.91	0.463
GM0449	47.07	46.93	1981	PVC	0.011	49.68	0.667	0.28	2.27	0.45	0.397	0.76	0.508
GM0450	48.77	47.83	1981	PVC	0.011	155.71	0.667	0.6	2.93	0.58	0.403	1.11	0.417
GM0451	49	48.87	1981	PVC	0.011	203.49	0.667	0.06	1.51	0.25	0.391	0.36	0.728
GM0452	50.47	49	1981	PVC	0.011	197.28	0.667	0.75	2.84	0.5	0.396	1.23	0.728
GM0453	52.17	50.57	1981	PVC	0.011	400.48	0.667	0.4	2.43	0.43	0.384	0.9	0.466
GM0454	52.36	52.27	1981	PVC	0.011	162.4	0.667	0.06	1.56	0.21	0.407	0.34	0.73
GM0455	55.19	53.99	1981	PVC	0.011	300.01	0.667	0.4	2.49	0.34	0.422	0.9	0.492
GM0456	53.89	52.36	1981	PVC	0.011	242.53	0.667	0.63	2.78	0.39	0.422	1.14	0.73
GM0466	44.2	43.99	1980	C	0.013	96.48	1	0.22	-2.71	0	-2.21	1.66	7.21
GM0467	44.48	44.2	1980	C	0.013	126.21	1	0.22	-2.72	0	-2.211	1.68	6.655
GM0468	44.85	44.48	1980	C	0.013	166.26	1	0.22	1.46	0	0.362	1.68	5.92
GM0469	45.16	44.85	1980	C	0.013	136.37	1	0.23	1.47	0	0.357	1.7	5.577
GM0470	45.4	45.16	1980	C	0.013	104	1	0.23	1.46	0.01	0.349	1.71	5.289
GM0471	45.68	45.4	1980	C	0.013	126.21	1	0.22	1.44	0	0.35	1.68	5.064
GM0472	45.8	45.68	1980	C	0.013	53.65	1	0.22	1.44	0	0.348	1.68	4.801
GM0473	46.78	45.9	1980	PVC	0.011	402.76	1	0.22	1.51	0.01	0.317	1.97	4.588
GM0474	47.22	46.78	1980	PVC	0.011	195.6	1	0.22	1.46	-0.01	0.291	2	3.755
GM0475	47.39	47.22	1980	PVC	0.011	84.85	1	0.2	1.41	0	0.275	1.88	3.332
GM0476	47.73	47.39	1980	PVC	0.011	153.11	1	0.22	1.42	0.01	0.254	1.98	3.168
GM0477	47.8	47.73	1980	C	0.013	29.87	1	0.23	1.31	0	0.242	1.72	2.837
GM0503	0.1	0	1975	DI	0.012	65.75	1.5	0.15	3.77	1.9	4.448	4.44	0.645
GM0531	43.76	41.12	1972	DI	0.012	185	0.667	1.43	3.72	0.82	0.39	1.57	0.34
GM0555	33.4	32.4	1973	CI	0.015	100	0.833	1	2.04	0	1.114	1.9	4.122
GM0556	32.47	32.07	1973	C	0.013	178	1	0.22	3.15	0	2.509	1.69	4.111
GM0563	34.22	33.73	1973	C	0.013	225	1	0.22	3.11	0	2.501	1.66	5.315
GM0564	33.73	32.96	1973	C	0.013	348	1	0.22	3.12	0	2.503	1.68	4.884
GM0565	32.88	32.57	1973	C	0.013	143	1	0.22	3.14	0	2.505	1.66	4.296
GM0646	34.8	34.22	1973	C	0.013	213.42	1	0.27	3.09	0.07	2.495	1.86	5.598
GM0647	35.25	34.9	1973	C	0.013	158.5	1	0.22	3.08	1.24	2.49	1.67	5.799
GM0648	35.97	35.35	1973	C	0.013	282.01	1	0.22	3.04	1.45	2.47	1.67	6.2
GM0649	36.26	36.07	1973	C	0.013	83.71	1	0.23	3.26	1.55	2.643	1.7	6.342
GM0650	37.47	36.86	1973	C	0.013	303.75	1	0.2	3.21	1.57	2.626	1.6	7.713
GM0652	38.1	37.47	1973	C	0.013	282.39	1	0.22	3.17	1.68	2.606	1.68	8.465
GM0653	38.56	38.2	1973	C	0.013	166.62	1	0.22	3.15	1.64	2.593	1.66	8.808
GM0654	39.2	38.66	1973	C	0.013	251.66	1	0.21	3.11	1.59	2.566	1.65	9.349
GM0675	50.55	48.71	1973	C	0.013	276.17	1.25	0.67	4.04	1.34	2.571	5.27	0.759
GM0676	52.19	50.55	1973	C	0.013	238.88	1.25	0.69	4.31	1.14	2.564	5.35	0.493
GM0678	48.61	48.24	1973	C	0.013	239.91	1.25	0.15	2.43	0.86	2.589	2.54	0.839
GM0682	48.24	47.87	1973	C	0.013	268.29	1.25	0.14	2.38	0.95	2.593	2.4	0.838
GM0686	47.87	47.46	1973	C	0.013	276.3	1.25	0.15	2.65	1.06	2.595	2.49	0.78
GM0687	47.46	45.3	1973	C	0.013	241.55	1.25	0.89	4.76	2.17	2.608	6.11	0.469
GM0708	11.19	3.29	1973	C	0.013	506.38	1.25	1.56	5.36	3.06	2.66	8.07	1.14

Link Name	US Invert (ft)	DS Invert (ft)	Installation Year	Material	Roughness	Length (ft)	Diameter (ft)	Slope	Max Velocity (ft/s)	Min Velocity (ft/s)	Max Flow (cfs)	Design Full Flow (cfs)	Max d/D
GM0709	19.11	11.19	1973	C	0.013	486.49	1.25	1.63	5.98	2.97	2.659	8.24	0.395
GM0710	29.26	19.11	1973	C	0.013	528.42	1.25	1.92	6.31	3.11	2.653	8.95	0.391
GM0711	32.47	29.26	1973	C	0.013	349.15	1.25	0.92	4.56	2.24	2.644	6.19	0.49
GM0712	40.43	32.47	1973	C	0.013	298.88	1.25	2.66	6.71	3.2	2.63	10.54	0.49
GM0713	44.12	40.8	1973	C	0.013	353.62	1.25	0.94	4.87	2.34	2.614	6.26	0.451
GM0714	45.3	44.12	1973	C	0.013	140.75	1.25	0.84	4.64	2.13	2.611	5.91	0.469
GM0724	30.96	20.91	1973	C	0.013	494.46	0.833	2.03	2.83	2.04	0.143	3.12	0.17
GM0725	20.91	18.4	1973	C	0.013	149.79	0.833	1.68	2.9	2.08	0.179	2.83	0.17
GM0726	18.4	10.03	1973	C	0.013	452.82	0.833	1.85	2.93	2.11	0.188	2.98	0.215
GM0727	10.03	7.3	1973	C	0.013	320.07	0.833	0.85	2.38	1.72	0.205	2.02	0.215
GM0728	7.24	5.65	1973	C	0.013	130.72	0.833	1.22	2.72	1.97	0.211	2.41	0.27
GM0729	5.24	5.14	1973	C	0.013	66.1	1.25	0.15	2.32	1.1	1.435	2.51	0.508
GM0730	4.64	4.22	1973	C	0.013	280.43	1.25	0.15	2.14	1.03	1.471	2.5	0.581
GM0731	4.11	3.78	1973	C	0.013	222.56	1.25	0.15	1.98	1.05	1.451	2.49	0.809
GM0732	3.68	3.26	1973	C	0.013	277.68	1.25	0.15	1.81	1.12	1.442	2.51	1.127
GM0733	3.26	2.83	1973	C	0.013	285.5	1.25	0.15	1.5	0.67	1.443	2.51	1.363
GM0734	2.62	2.36	1973	C	0.013	536.48	1.5	0.05	2.32	1.09	4.095	2.31	1.276
GM0735	2.95	2.62	1973	C	0.013	299.72	1.5	0.11	1.77	0.97	2.672	3.49	1.276
GM0747	7.16	6.33	1973	C	0.013	380.5	1	0.22	2.31	0.9	1.262	1.66	0.682
GM0748	6.33	5.49	1973	C	0.013	382.43	1	0.22	2.28	0.86	1.257	1.67	0.682
GM0752	5.04	4.64	1973	C	0.013	268.42	1.25	0.15	2.11	1.07	1.463	2.49	0.556
GM0757	0.94	0.52	1973	C	0.013	350	1.5	0.12	2.63	1.5	4.448	3.64	0.924
GM0758	1.33	0.94	1973	C	0.013	261.64	1.5	0.15	2.59	1.6	4.448	4.06	0.936
GM0759	1.54	1.43	1973	C	0.013	248.65	1.5	0.04	2.46	1.28	4.442	2.21	1.079
GM0760	2.36	1.97	1973	C	0.013	349.89	1.5	0.11	2.68	1.44	4.438	3.51	0.924
GM0761	0.52	0.2	1973	C	0.013	268	1.5	0.12	2.97	1.52	4.448	3.63	0.819
GM0778	42.11	42.05	1973	C	0.013	180.46	1	0.03	-2.55	0	-2.126	0.65	10.98
GM0779	42.05	41.81	1973	C	0.013	85.69	0.833	0.28	0.97	0	-0.121	1.16	13.469
GM0782	43.61	43.25	1973	C	0.013	119.48	0.833	0.3	0.8	0	-0.056	1.2	11.746
GM0783	43.25	42.15	1973	C	0.013	426.68	0.833	0.26	0.83	0	-0.122	1.11	13.061
GM0785	41.23	40.7	1973	C	0.013	129.73	1	0.41	3.03	1.82	2.523	2.28	10.569
GM0786	41.61	41.23	1973	C	0.013	283.85	1	0.13	3.01	1.35	2.521	1.3	11.42
GM0787	40.7	39.79	1973	C	0.013	413.76	1	0.22	3.03	1.63	2.525	1.67	10.532
GM0792	39.79	39.38	1973	C	0.013	186.15	1	0.22	3.09	1.62	2.559	1.67	9.621
GM0793	41.05	39.8	1973	C	0.013	126.8	0.667	0.99	3.12	0.76	0.414	1.2	0.405
GM0798	46	45.46	1973	C	0.013	131.83	0.833	0.41	1.25	0	-0.063	1.4	4.744
GM0799	45.46	43.86	1973	C	0.013	402.44	0.833	0.4	1.35	0	-0.119	1.38	6.662
GM0800	43.86	40.11	1973	C	0.013	341.39	1	1.1	1.88	0	-0.125	3.73	9.301
GM0805	45.12	44.6	1973	C	0.013	113.72	0.667	0.46	2.52	0.46	0.593	0.82	9.942
GM0806	44.2	43.2	1973	CI	0.015	91.5	1	1.09	2.89	0.18	0.596	3.23	8
GM0807	43.2	43.08	1973	C	0.013	79.98	1	0.15	1.71	0.09	0.595	1.38	8.12
GM0808	43.08	42.57	1973	C	0.013	229.8	1	0.22	-2.58	0	-2.133	1.68	9.377
GM0809	42.57	42.21	1973	C	0.013	160.35	1	0.22	-2.56	0	-2.131	1.69	10.251

Link Name	US Invert (ft)	DS Invert (ft)	Installation Year	Material	Roughness	Length (ft)	Diameter (ft)	Slope	Max Velocity (ft/s)	Min Velocity (ft/s)	Max Flow (cfs)	Design Full Flow (cfs)	Max d/D
GM0823	26.77	19.58	1973	C	0.013	438.75	0.667	1.64	3.75	0.92	0.415	1.55	0.354
GM0824	19.43	16.65	1973	C	0.013	277.63	0.667	1	3.13	0.79	0.412	1.21	0.402
GM0825	16.3	10.54	1973	C	0.013	512.2	0.667	1.12	3.02	0.23	0.411	1.28	0.882
GM0829	14.01	13.11	1973	C	0.013	232.8	0.833	0.39	1.05	0.75	0.036	1.36	0.129
GM0830	13.11	12.85	1973	C	0.013	63.89	0.833	0.41	1.09	0.78	0.044	1.4	0.129
GM0831	12.85	10.28	1973	C	0.013	240.49	0.833	1.07	1.62	1.14	0.045	2.26	0.099
GM0832	10.49	9.45	1973	C	0.013	359.73	1	0.29	2.52	0.59	1.2	1.92	0.65
GM0833	9.45	8.95	1973	C	0.013	230.1	1	0.22	2.35	0.73	1.247	1.66	0.65
GM0836	8.85	8.1	1973	C	0.013	335.24	1	0.22	2.34	0.77	1.254	1.69	0.658
GM0837	8	7.16	1973	C	0.013	381.13	1	0.22	2.34	0.88	1.261	1.67	0.655
GM0838	39.31	26.93	1973	C	0.013	475.22	0.667	2.61	4.44	1	0.416	1.95	0.314
GM0848	16.11	15.64	2007	PVC	0.011	69.16	1.75	0.68	0	0	0	15.44	0
GM0849	47.73	47.17	1981	PVC	0.011	236.09	0.667	0.24	1.99	0.41	0.395	0.7	0.566
GM0878	4.99	4.4	2009	PVC	0.011	111.16	1	0.53	0.46	0.04	0.003	3.07	0.546
GM0880	15.29	13.06	2007	PVC	0.011	483.76	1.75	0.46	0	0	0	12.71	0
GM0881	12.91	10.88	2007	PVC	0.011	526.75	1.75	0.39	0	0	0	11.62	0
GM0882	8.24	7.86	2007	PVC	0.011	53.79	1.75	0.71	0.58	0.32	0.005	15.74	0.011
GM0883	10.88	10.68	2007	PVC	0.011	265.82	1.75	0.08	0	0	0	5.14	0
GM0893	10.58	8.44	2007	PVC	0.011	502.69	1.75	0.43	0.48	0.26	0.005	12.22	0.014
Link295_SmithLS_FM	32	52.19	0		0.014	1531.04	1	-1.32	2.89	0	2.557	3.8	20.8
Link297 (FM)									0	0	2.674	0	0
Link297 (FM)									0	0	0	0	0
Link297 (FM)									0	0	0	0	0
Link298	32.07	32			0.014	28.87	1.5	0.24	1.53	0	2.721	4.8	2.543
Link299 (FM)									0	0	0.607	0	0
Link300_HWY30_FM	33.93	60.55	0		0.014	256.52	0.5	-10.38	2.25	0	0.578	1.68	54.078
Link303	8.91	12.5			0.014	1565	0.33	-0.23	2.1	0	0.128	0.08	46.458
Link304_SevenOaks									0	0	1.192	0	0
Link305_SpringLake									0	0	1.014	0	0
Link306	8	15.5			0.014	1726	0.33	-0.43	1.57	0	0.07	0.11	35.587
Link307	185	181.77			0.014	24.53	0.208	13.17	11.03	3.68	0.501	0.18	59.345
Link308	181.77	179.87			0.014	142.41	0.667	1.33	3.45	1.46	0.501	1.3	0.451
Link309	179.87	178.05			0.014	159.08	0.667	1.14	3.23	1.38	0.501	1.2	0.509
Link310	178.05	176.97			0.014	94.06	0.667	1.15	2.99	1.25	0.501	1.2	0.509
Link311	176.97	154.4			0.014	264.94	0.667	8.52	6.78	2.78	0.501	3.28	0.265
Link312	153.99	124.48			0.014	251.95	0.667	11.71	7.58	3.14	0.501	3.84	0.245
Link313	124.48	89.83			0.014	271.69	0.667	12.75	7.71	3.27	0.501	4.01	0.243
Link314	89.83	56			0.014	233.67	0.667	14.48	8.18	3.34	0.501	4.27	0.232
Link315	54.96	54.33			0.014	21.54	0.667	2.92	4.6	1.96	0.501	1.92	0.431
Link316	54.33	45.97			0.014	323.11	0.667	2.59	4.43	0.58	0.501	1.81	8.26
Link317	6.28	6.21			0.014	268.81	0.667	0.03	0.99	0.01	0.276	0.18	0.78
Link318	6.21	5.63			0.014	146.6	0.667	0.4	1.82	0	0.276	0.71	0.457
Link319	5.51	4.59			0.014	264	0.667	0.35	1.71	0.04	0.276	0.66	0.482

Link Name	US Invert (ft)	DS Invert (ft)	Installation Year	Material	Roughness	Length (ft)	Diameter (ft)	Slope	Max Velocity (ft/s)	Min Velocity (ft/s)	Max Flow (cfs)	Design Full Flow (cfs)	Max d/D
Link320	4.49	3.84			0.014	202	0.667	0.32	1.67	0.04	0.276	0.64	0.491
Link321	3.74	3.54			0.014	64.73	0.667	0.31	1.76	0	0.276	0.62	0.472

F-2:
Existing Manhole Data

Existing System Flows (2015; 5-year, 24-hour storm event)

Node Name	Ground Elevation (ft)	Invert Elevation (ft)	Constant Inflow (cfs)	Dry Weather Flow Flag	Temporal Variation	Peaking Factor	Volume of Const. Inflow (ft^3)	Volume of DWF Inflow (ft^3)	Time Flooded (min)	Time Surcharged (min)	Max Depth (ft)
MH0019	22.5	16.11	0	1	Scappoose_Diurnal	0	0	0	0	0	0
MH0020	23.7	17.89	0	1	Scappoose_Diurnal	0	0	0	0	0	0
MH0021	25	19.52	0	1	Scappoose_Diurnal	0	0	0	0	0	0
MH0022	26.2	22.11	0	1	Scappoose_Diurnal	1	0	0	0	0	0
MH0048	62.73	55.13	0.02	1	Site_3_DWF	0	0	2030.05	0	0	0.52
MH0049	67.27	56.35	0.002	1	Site_3_DWF	1	0	0	0	0	0.33
MH0059	15.34	6.6	0.012	1	Site_4_DWF	1	0	1746.14	0	0	0.078
MH0061	15.78	7.9	0.042	1	Site_4_DWF	0	0	6226.56	0	0	0.073
MH0098	62.41	55.11	0.012	1	Site_3_DWF	0	0	1212.39	0	0	0.408
MH0099	62.35	53.89	0.015	1	Site_3_DWF	0	0	1212.39	0	0	0.282
MH0125	52.5	46.1	0.001	1	Site_1_DWF	0	0	160.27	0	1647.7	5.377
MH0126	55.2	46.77	0.159	1	Site_1_DWF	1	0	26170.13	0	1617.958	4.709
MH0127	56.5	47.25	0.012	1	Site_1_DWF	0	0	1980.5	0	1588.214	4.229
MH0128	55.6	48.42	0.049	1	Site_1_DWF	0	0	8059.39	0	1514.237	3.059
MH0130	18.18	12.35	0	1	Site_3_DWF	0	0	0	0	0	0.172
MH0131	20.62	12.66	0.016	1	Site_3_DWF	0	0	1635.32	0	0	0.173
MH0132	21.5	13.76	0.02	1	Site_3_DWF	0	0	2030.05	0	0	0.17
MH0136	17.27	13.91	0.093	1	Site_3_DWF	0	0	8909.68	0	0	0.208
MH0137	17.76	14.13	0.158	1	Site_3_DWF	1	0	6682.26	0	0	0.084
MH0154	17.7	7.24	0.009	1	Site_4_DWF	0	0	1367.14	0	0	0.167
MH0173	12.7	1.33	0.044	1	Site_4_DWF	0	0	2138.69	0	0	1.404
MH0182	46.9	35.42	0.013	1	Site_3_DWF	1	0	1212.39	0	0	0.115
MH0184	47.01	37.22	0.1	1	Site_3_DWF	1	0	9727.34	0	0	0.11
MH0198	65.09	54.48	0.015	1	Site_1_DWF	0	0	2518.56	0	0	0.086
MH0202	60.8	54.05	0.015	1	Site_1_DWF	0	0	2472.77	0	0	0.104
MH0203	59.5	53.47	0.004	1	Site_1_DWF	0	0	663.98	0	0	0.108
MH0204	58.5	52.93	0.021	1	Site_1_DWF	0	0	3457.3	0	0	0.128
MH0205	59.4	53.21	0	1	Site_1_DWF	0	0	0	0	0	0.109
MH0206	60.07	52.53	0.009	1	Site_1_DWF	0	0	1476.79	0	0	0.128
MH0207	59.44	51.7	0	1	Site_1_DWF	0	0	0	0	0	0.266
MH0212	22.2	15.29	0.009	1	Scappoose_Diurnal	0	0	0	0	0	0
MH0214	46.96	36.76	0.035	1	Site_1_DWF	1	0	5758.34	0	2414.038	6.899
MH0215	16.84	11.48	0.045	1	Site_3_DWF	1	0	2058.25	0	0	0.207
MH0251	61.66	52.36	0.032	1	Site_3_DWF	0	0	2847.72	0	0	0.487
MH0254	57.4	49.8	0.027	1	Site_1_DWF	0	0	4441.82	0	0	0.777
MH0255	58	49.4	0.009	1	Site_1_DWF	0	0	1476.79	0	0	1.173
MH0256	62	51.14	0.018	1	Site_1_DWF	0	0	2965.03	0	0	0.126
MH0257	61.4	51.52	0.013	1	Site_1_DWF	0	0	2175.12	0	0	0.443
MH0262	61.6	49	0.032	1	Site_3_DWF	0	0	2847.72	0	0	0.485
MH0284	69.8	57.4	0.035	1	Site_3_DWF	0	0	3242.45	0	0	0.381
MH0285	68.49	58.36	0.03	1	Site_3_DWF	0	0	2847.72	0	0	0.384
MH0286	64.6	59.55	0.033	1	Site_3_DWF	0	0	2847.72	0	0	0.397
MH0287	63	60.55	0.024	1	Site_3_DWF	0	0	2030.05	0	0	0.419
MH0290	17.3	10.54	0	1	Site_3_DWF	0	0	0	0	0	0.588

Node Name	Ground Elevation (ft)	Invert Elevation (ft)	Constant Inflow (cfs)	Dry Weather Flow Flag	Temporal Variation	Peaking Factor	Volume of Const. Inflow (ft^3)	Volume of DWF Inflow (ft^3)	Time Flooded (min)	Time Surcharged (min)	Max Depth (ft)
MH0291	16.5	12.19	0.088	1	Site_3_DWF	1	0	0	0	0	0.275
MH0293	53.03	41.23	0.005	1	Site_1_DWF	0	0	824.26	0	2169.467	10.569
MH0304	56.93	45.35	0.002	1	Site_3_DWF	0	0	0	0	0	0.307
MH0305	58.93	43.76	0.024	1	Site_3_DWF	0	0	2030.05	0	0	0.227
MH0306	58.93	44.24	0.01	1	Site_3_DWF	0	0	817.66	0	0	0.298
MH0307	59.81	46.91	0.002	1	Site_3_DWF	0	0	0	0	0	0.309
MH0308	59.77	47.07	0.004	1	Site_3_DWF	0	0	394.73	0	0	0.339
MH0309	61.37	48.77	0.025	1	Site_3_DWF	0	0	2424.79	0	0	0.278
MH0310	61.37	50.47	0.025	1	Site_3_DWF	1	0	2424.79	0	0	0.26
MH0311	61.77	52.17	0.026	1	Site_3_DWF	1	0	2424.79	0	0	0.311
MH0318	51.9	44.2	0.003	1	Site_1_DWF	0	0	492.26	0	1752.95	6.655
MH0319	50.4	44.48	0.01	1	Site_1_DWF	0	0	1648.51	1264.7	1736.817	5.92
MH0320	53	44.85	0.003	1	Site_1_DWF	0	0	492.26	0	1716.571	5.577
MH0321	53	45.16	0.005	1	Site_1_DWF	0	0	824.26	0	1700.398	5.288
MH0322	51.3	45.4	0.002	1	Site_1_DWF	0	0	331.99	0	1687.75	5.062
MH0323	57.3	45.68	0.003	1	Site_1_DWF	0	0	492.26	0	1672.75	4.8
MH0324	57.8	45.8	0.04	1	Site_1_DWF	0	0	6582.6	0	1661.311	4.686
MH0325	59.2	46.78	0.022	1	Site_1_DWF	0	0	3617.57	0	1607.429	3.751
MH0326	58	47.22	0.007	1	Site_1_DWF	0	0	1156.25	0	1579.949	3.331
MH0327	58.2	47.39	0.013	1	Site_1_DWF	0	0	2140.78	0	1568.733	3.168
MH0328	60.9	47.73	0.003	1	Site_1_DWF	0	0	492.26	0	1547.517	2.837
MH0329	61	47.8	0	1	Site_1_DWF	0	0	0	0	1472.481	2.768
MH0373	56.81	41.05	0.046	1	Site_3_DWF	1	0	4454.84	0	0	0.27
MH0376	53.72	45.3	0.022	1	Site_5_DWF	0	0	817.34	0	0	0.586
MH0393	44.96	32.07	0.002	1	Site_5_DWF	1	0	266.11	0	5355.402	3.765
MH0395	44.63	32.47	0.018	1	Site_5_DWF	1	0	1634.69	0	5760	4.111
MH0402MTR2	46.2	33.4	0.415	1	Site_2_DWF	1	0	0	0	2577.207	2.446
MH0421	40.38	34.22	0.031	1	Site_5_DWF	0	0	2737.15	0	3883.897	5.315
MH0422	41.96	33.73	0.01	1	Site_5_DWF	0	0	817.34	0	5555.849	4.884
MH0423	39.96	32.88	0.007	1	Site_5_DWF	0	0	551.23	0	5760	4.296
MH0431	57.33	48.61	0.113	1	Site_5_DWF	0	0	4371.84	0	0	1.048
MH0473	47.96	37.47	0.042	1	Site_1_DWF	0	0	6914.59	0	2406.075	7.713
MH0483	46.96	36.26	0.003	1	Site_1_DWF	1	0	492.26	0	2436.069	6.342
MH0484MTR1	42.17	35.97	0.04	1	Site_1_DWF	1	0	6582.6	763.1	2460.046	6.2
MH0485	44.96	35.25	0.099	1	Site_5_DWF	0	0	8477.57	0	2512.156	5.799
MH0486	44.96	34.8	0.025	1	Site_5_DWF	0	0	2185.92	0	2565.775	5.598
MH0487	49.16	39.2	0.025	1	Site_1_DWF	0	0	4109.83	0	2335.11	9.349
MH0488	48.5	38.56	0.059	1	Site_1_DWF	0	0	9707.9	0	2364.572	8.808
MH0489	48.5	38.1	0.028	1	Site_1_DWF	0	0	4613.54	0	2376.806	8.465
MH0506	57.1	47.46	0.069	1	Site_5_DWF	1	0	2737.15	0	0	0.571
MH0507	57.17	47.87	0.017	1	Site_5_DWF	0	0	551.23	0	0	0.974
MH0510	56.83	48.24	0.025	1	Site_5_DWF	0	0	817.34	0	0	1.047
MH0513	57.97	50.55	0.051	1	Site_5_DWF	1	0	1919.81	0	0	0.616
MH0514	61.43	52.19	0.046	1	Site_5_DWF	0	0	1634.69	0	0	0.61
MH0525	53.3	44.12	0.018	1	Site_5_DWF	0	0	551.23	0	0	0.564

Node Name	Ground Elevation (ft)	Invert Elevation (ft)	Constant Inflow (cfs)	Dry Weather Flow Flag	Temporal Variation	Peaking Factor	Volume of Const. Inflow (ft^3)	Volume of DWF Inflow (ft^3)	Time Flooded (min)	Time Surcharged (min)	Max Depth (ft)
MH0526	50.2	40.43	0.09	1	Site_5_DWF	0	0	3554.5	0	0	0.425
MH0530	46.37	32.47	0.081	1	Site_5_DWF	0	0	3003.26	0	0	0.613
MH0533	40	29.26	0.053	1	Site_5_DWF	1	0	1919.81	0	0	0.466
MH0534	32.37	19.11	0.03	1	Site_5_DWF	0	0	1102.46	0	0	0.488
MH0536	20.49	11.19	0.009	1	Site_5_DWF	1	0	266.11	0	0	0.494
MH0537MTR5	15.5	2.95	0.004	1	Site_5_DWF	1	0	0	0	0	1.764
MH0549	41.05	30.96	0.22	1	Site_4_DWF	0	0	32310.43	0	0	0.121
MH0561	22.83	14.01	0.056	1	Site_4_DWF	0	0	8175.74	0	0	0.093
MH0565	30.81	20.91	0.056	1	Site_4_DWF	0	0	8175.74	0	0	0.142
MH0566	30.5	18.4	0.015	1	Site_4_DWF	1	0	2138.69	0	0	0.142
MH0567	19.96	10.03	0.026	1	Site_4_DWF	1	0	3898.37	0	0	0.179
MH0568	16.66	5.24	0.006	1	Site_4_DWF	0	0	974.59	0	0	0.635
MH0569	15.78	5.04	0.003	1	Site_4_DWF	0	0	392.54	0	0	0.689
MH0570	16.71	6.33	0.015	1	Site_4_DWF	1	0	2138.69	0	0	0.682
MH0571	16.71	7.16	0.02	1	Site_4_DWF	1	0	2923.78	0	0	0.655
MH0572	15.92	4.11	0.02	1	Site_4_DWF	0	0	974.59	0	0	0.836
MH0573	13.48	3.68	0.018	1	Site_4_DWF	1	0	974.59	0	0	1.112
MH0574	14.5	2.62	0.02	1	Site_4_DWF	1	0	2720.74	0	1094.46	1.913
MH0575MTR4	15.6	4.64	0.016	1	Site_4_DWF	1	0	3113.28	0	0	0.695
MH0576	11.86	3.26	0.028	1	Site_4_DWF	0	0	1367.14	0	152.55	1.409
MH0577	14.17	2.36	0.052	1	Site_4_DWF	1	0	0	0	0	1.386
MH0578	12.34	0.94	0.009	1	Site_4_DWF	1	0	392.54	0	0	1.386
MH0579	13.82	0.52	0	1	Site_4_DWF	0	0	0	0	0	1.229
MH0580	12.82	0.1	0	1	Site_4_DWF	0	0	0	0	0	0.967
MH0581	14.95	1.54	0.048	1	Site_4_DWF	1	0	2341.73	0	0	1.619
MH0591	52.41	39.31	0.013	1	Site_3_DWF	0	0	1212.39	0	0	0.209
MH0594	57.96	46	0.069	1	Site_1_DWF	0	0	11356.42	0	1486.149	3.413
MH0595	58.02	45.46	0.014	1	Site_1_DWF	0	0	2301.05	0	1526.169	3.953
MH0597	54.12	43.86	0.016	1	Site_1_DWF	0	0	2633.04	0	1615.319	5.552
MH0598	50.94	39.79	0.017	1	Site_1_DWF	0	0	2793.31	0	2291.579	9.621
MH0603	52	40.7	0.008	1	Site_1_DWF	0	0	1316.52	0	2304.313	10.532
MH0604	53.03	41.61	0.009	1	Site_1_DWF	0	0	1476.79	199.9	1900.798	11.42
MH0605	53.03	42.05	0.021	1	Site_1_DWF	0	0	3457.3	206.5	1912.574	10.98
MH0606	52.71	42.11	0.018	1	Site_1_DWF	0	0	2965.03	0	1887.2	10.351
MH0607	52.83	42.57	0.004	1	Site_1_DWF	0	0	663.98	0	1850.517	9.377
MH0608	51.78	43.08	0	1	Site_1_DWF	0	0	0	0	1764.911	8.12
MH0609	53.73	43.25	0.014	1	Site_1_DWF	0	0	2301.05	0	1803.15	9.784
MH0610	56.03	43.61	0.026	1	Site_1_DWF	0	0	4281.55	0	1784.058	9.425
MH0614	51.2	43.2	0.001	1	Site_1_DWF	0	0	160.27	410.4	1806.061	8
MH0615	52.7	44.2	0.006	1	Site_1_DWF	0	0	984.53	0	1751.092	7.031
MH0616	52.6	45.12	0	1	Keys WTP	1	0	0	0	1677.511	6.357
MH0634	39.43	26.77	0.01	1	Site_3_DWF	1	0	817.66	0	0	0.236
MH0635	15.71	8	0.026	1	Site_4_DWF	0	0	3898.37	0	0	0.649
MH0636	20.71	8.85	0.019	1	Site_4_DWF	1	0	2720.74	0	0	0.658
MH0637MTR3	20.49	9.45	0.011	1	Site_3_DWF	1	0	4454.84	0	0	0.65

Node Name	Ground Elevation (ft)	Invert Elevation (ft)	Constant Inflow (cfs)	Dry Weather Flow Flag	Temporal Variation	Peaking Factor	Volume of Const. Inflow (ft^3)	Volume of DWF Inflow (ft^3)	Time Flooded (min)	Time Surcharged (min)	Max Depth (ft)
MH0638	17.78	10.49	0.024	1	Site_3_DWF	1	0	394.73	0	0	0.577
MH0639	21.91	12.85	0.003	1	Site_4_DWF	0	0	392.54	0	0	0.082
MH0640	21.91	13.11	0.012	1	Site_4_DWF	0	0	1746.14	0	0	0.108
MH0642	22.79	16.3	0.006	1	Site_3_DWF	0	0	394.73	0	0	0.26
MH0643	30.44	19.43	0	1	Site_3_DWF	0	0	0	0	0	0.268
MH0133	59.93	47.73	0.011	1	Site_3_DWF	0	0	817.66	0	0	0.377
MH0665	19	8.24	0.007	1	Scappoose_Diurnal	1	0	0	0	0	0.02
MH0663	18.5	10.88	0.017	1	Scappoose_Diurnal	0	0	0	0	0	0
MH0664	20.2	12.91	0.019	1	Scappoose_Diurnal	0	0	0	0	0	0
MH0674	18.5	10.58	0.011	1	Scappoose_Diurnal	0	0	1093.28	0	0	0.024
MMH0001	15	4.99	0.01	1	Site_4_DWF	0	0	582.05	0	0	0.018
MMH0002	10	0	0.001	1	Site_4_DWF	0	0	0	0	0	0.809
MMH0003	63	54.83	0.022	1	Site_1_DWF	0	0	3663.36	0	0	0.071
MMH0004	56.6	48.94	0.004	1	Site_1_DWF	0	0	663.98	0	1489.877	2.539
Node449	46.6	24	0.021	1	Site_3_DWF	1	0	2030.05	0	0	8.5
Node450	45	24.5	0	1	Site_5_DWF	1	0	0	0	5523.024	11.315
Node451	45	32	0	1	Site_5_DWF	0	0	0	5760	5760	29.46
Node452	52.93	33.93	0	1	Scappoose_Diurnal	0	0	0	5760	5760	28.996
MH0401	48.2	34.4	0	1	Site_2_DWF	1	0	34020	0	1913.262	1.506
Node455	13.91	8.91	0	0	0	0	0	0	5282.9	5760	18.905
Node456	18.4	8	0	0	0	0	0	0	5046.1	5760	19.24
Node457	18.4	8.8	0	1	Site_3_DWF	1	0	8712.32	0	5760	3.003
Node458	10.96	-3	0	1	Site_3_DWF	1	0	10911.54	0	5760	6.069
Node461	67.5	54.33	0	0	0	0	0	0	0	0	0.287
Node462	69.5	54.96	0	0	0	0	0	0	0	0	0.233
Node463	105.5	89.83	0	0	0	0	0	0	0	0	0.154
Node464	153.5	124.48	0	0	0	0	0	0	0	0	0.162
Node465	177.5	153.99	0	0	0	0	0	0	0	0	0.163
Node466	184	176.97	0	0	0	0	0	0	0	0	0.176
Node467	192	178.05	0	0	0	0	0	0	0	0	0.339
Node468	194	179.87	0	0	0	0	0	0	0	0	0.301
Node469	192	181.77	0	0	0	0	0	0	0	0	0.288
Keys_WTP	202	185	0	1	Keys WTP	1	0	116581	0	5524.611	12.361
Node471	12.63	3.74	0	0	0	0	0	0	0	0	0.314
Node472	13.51	4.49	0	0	0	0	0	0	0	0	0.327
Node473	13.01	5.51	0	0	0	0	0	0	0	0	0.321
Node474	13.26	6.21	0	0	0	0	0	0	0	0	0.305
Miller_WTP_Inflow	10.4	6.28	0	1	Miller WTP	1	0	27820.8	0	0	0.52

F-3:
Future Link Data

Alternative B Future Flows (2035; 5-year, 24-hour storm event)

Link Name	US Invert (ft)	DS Invert (ft)	Installation Year	Material	Roughness	Length (ft)	Diameter (ft)	Slope	Max Velocity (ft/s)	Min Velocity (ft/s)	Max Flow (cfs)	Design Full Flow (cfs)	Max d/D
GM0048	6.6	5.8	2006	PVC	0.011	201.21	1.25	0.4	1.1	0	0.035	4.81	0.371
GM0050	7.9	6.6	2006	PVC	0.011	292.23	0.833	0.44	1.17	0	0.028	1.73	0.09
GM00554	34.4	33.5			0.014	180.5	1	0.499	3.14	0	1.574	2.34	0.619
GM0062	22.11	19.77	2007	PVC	0.011	331.89	1.75	0.71	26.85	0	4.381	172.96	0.206
GM0063	19.52	18.09	2007	PVC	0.011	375.15	1.75	0.38	4	0	2.955	11.56	0.349
GM0064	17.89	16.16	2007	PVC	0.011	370.91	1.75	0.47	4.21	0	2.694	12.79	0.314
GM0140	48.94	48.62	2001	PVC	0.011	76	0.833	0.42	0.55	0	0.003	1.68	0.036
GM0141	48.42	47.45	2001	PVC	0.011	242.29	0.833	0.4	1.27	0	0.045	1.64	0.118
GM0142	47.25	46.97	2001	PVC	0.011	64.95	0.833	0.43	1.41	0	0.056	1.7	0.125
GM0143	46.1	45.97	2001	PVC	0.011	33.12	0.833	0.39	2	0	0.193	1.62	0.234
GM0144	46.77	46.5	2001	PVC	0.011	118.56	0.833	0.23	1.6	0	0.192	1.23	0.277
GM0161	14.13	14.01	2000	PVC	0.011	23.02	0.833	0.52	1.4	0	0.049	1.87	0.14
GM0162	13.91	13.86	2000	PVC	0.011	104	0.833	0.05	0.95	0	0.102	0.57	0.26
GM0163	13.76	12.76	2000	PVC	0.011	400	0.833	0.25	1.38	0	0.114	1.29	0.212
GM0164	12.66	12.46	2000	PVC	0.011	79.17	0.833	0.25	1.46	0	0.123	1.3	0.215
GM0165	12.35	12.3	2000	DI	0.012	20	0.833	0.25	1.49	0	0.123	1.19	0.213
GM0224	35.42	33.93	1996	PVC	0.011	362.58	0.833	0.41	1.44	0	0.065	1.66	0.138
GM0225	37.22	35.62	1996	PVC	0.011	405.28	0.833	0.39	1.37	0	0.058	1.63	0.132
GM0237	51.7	51.52	1995	PVC	0.011	71.1	1	0.25	0.88	0	0.238	2.12	0.525
GM0238	52.53	51.8	1995	PVC	0.011	290.21	1	0.25	1.74	0	0.238	2.11	0.258
GM0239	52.93	52.63	1995	PVC	0.011	118.2	1	0.25	1.72	0	0.231	2.12	0.23
GM0241	53.21	53.03	1995	PVC	0.011	71.22	1	0.25	1.7	0	0.213	2.12	0.219
GM0242	53.47	53.31	1995	PVC	0.011	61.98	1	0.26	1.72	0	0.213	2.14	0.217
GM0243	54.05	53.57	1995	PVC	0.011	175.44	1	0.27	1.7	0	0.209	2.2	0.216
GM0244	54.48	54.05	1995	PVC	0.011	170.77	1	0.25	1.68	0	0.197	2.11	0.216
GM0245	54.83	54.51	1995	PVC	0.011	126.75	1	0.25	1.6	0	0.184	2.12	0.206
GM0276	36.76	36.36	1996	C	0.013	206.3	1	0.19	2.38	0	1.412	1.57	0.723
GM0374	51.84	51.34	1995	PVC	0.011	120.32	1	0.42	2.15	0	0.249	2.71	0.205
GM0375	51.14	50	1995	PVC	0.011	228.43	1	0.5	2.34	0	0.264	2.97	0.201
GM0376	49.4	48.75	1995	PVC	0.011	155.28	1	0.42	2.26	0	0.294	2.72	0.222
GM0377	49.8	49.6	1995	PVC	0.011	49.03	1	0.41	2.23	0	0.287	2.69	0.221
GM0409	60.55	59.55	1988	PVC	0.011	250	0.667	0.4	2.77	0	0.614	0.9	0.635
GM0410	59.55	58.53	1988	PVC	0.011	272.67	0.667	0.37	2.61	0	0.582	0.87	0.618
GM0411	58.36	57.4	1988	PVC	0.011	354.18	0.833	0.27	2.58	0	0.802	1.35	0.562
GM0412	57.4	56.35	1988	PVC	0.011	358.33	0.833	0.29	2.67	0	0.826	1.4	0.562
GM0413	55.25	55.2	1988	PVC	0.011	63.77	0.833	0.08	2.18	0	0.812	0.72	0.665
GM0414	56.35	55.35	1988	PVC	0.011	312.25	0.833	0.32	2.76	0	0.822	1.46	0.545
GM0417	12.19	11.54	1986	PVC	0.011	329.67	1	0.2	1.63	0	0.285	1.87	0.28
GM0418	11.48	10.54	1986	PVC	0.011	179.05	1	0.52	2.03	0	0.296	3.05	0.624
GM0419	10.54	10.49	1986	C	0.013	21.37	1.25	0.23	2.73	0	1.667	3.12	0.499
GM0446	44.24	43.86	1981	PVC	0.011	88.82	0.667	0.43	3.05	0	0.816	0.94	0.725
GM0447	45.35	44.34	1981	PVC	0.011	254.46	0.667	0.4	2.86	0	0.812	0.9	0.772
GM0448	46.91	45.45	1981	PVC	0.011	362.8	0.667	0.4	2.86	0	0.819	0.91	0.78

Link Name	US Invert (ft)	DS Invert (ft)	Installation Year	Material	Roughness	Length (ft)	Diameter (ft)	Slope	Max Velocity (ft/s)	Min Velocity (ft/s)	Max Flow (cfs)	Design Full Flow (cfs)	Max d/D
GM0449	47.07	46.93	1981	PVC	0.011	49.68	0.667	0.282	2.56	0	0.718	0.76	0.76
GM0450	48.77	47.83	1981	PVC	0.011	155.71	0.667	0.604	3.35	0	0.746	1.11	0.719
GM0451	49	48.87	1981	PVC	0.011	203.49	0.667	0.064	2.17	0	0.733	0.36	1.372
GM0452	50.47	49	1981	PVC	0.011	197.28	0.667	0.745	3.28	0	0.732	1.23	1.372
GM0453	52.17	50.57	1981	PVC	0.011	400.48	0.667	0.4	2.79	0	0.728	0.9	0.71
GM0454	52.36	52.27	1981	PVC	0.011	162.4	0.667	0.055	2.25	0	0.759	0.34	1.297
GM0455	55.19	53.99	1981	PVC	0.011	300.01	0.667	0.4	2.83	0	0.767	0.9	0.737
GM0456	53.89	52.36	1981	PVC	0.011	242.53	0.667	0.631	3.18	0	0.763	1.14	1.297
GM0466	44.2	43.99	1980	C	0.013	96.48	1	0.22	1.74	0	0.385	1.66	0.333
GM0467	44.48	44.2	1980	C	0.013	126.21	1	0.22	1.72	0	0.383	1.68	0.333
GM0468	44.85	44.48	1980	C	0.013	166.26	1	0.22	1.72	0	0.375	1.68	0.326
GM0469	45.16	44.85	1980	C	0.013	136.37	1	0.23	1.73	0	0.372	1.7	0.321
GM0470	45.4	45.16	1980	C	0.013	104	1	0.23	1.73	0	0.368	1.71	0.318
GM0471	45.68	45.4	1980	C	0.013	126.21	1	0.22	1.71	0	0.367	1.68	0.318
GM0472	45.8	45.68	1980	C	0.013	53.65	1	0.22	1.71	0	0.364	1.68	0.318
GM0473	46.78	45.9	1980	PVC	0.011	402.76	1	0.22	1.77	0	0.331	1.97	0.293
GM0474	47.22	46.78	1980	PVC	0.011	195.6	1	0.22	1.83	0	0.313	2	0.293
GM0475	47.39	47.22	1980	PVC	0.011	84.85	1	0.2	1.77	0	0.308	1.88	0.274
GM0476	47.73	47.39	1980	PVC	0.011	153.11	1	0.22	1.8	0	0.297	1.98	0.274
GM0477	47.8	47.73	1980	C	0.013	29.87	1	0.23	1.69	0	0.294	1.72	0.275
GM0503	0.1	0	1975	DI	0.012	65.75	2.5	0.15	4.58	0	10.954	17.33	0.495
GM0531	43.76	41.12	1972	DI	0.012	185	0.667	0.54	4.54	0	0.827	1.57	0.517
GM0555	33.4	32.4	1973	CI	0.015	100	1	1	3.92	0	1.574	3.09	0.554
GM0556	32.47	32.07	1973	C	0.013	178	1.5	0.225	2.01	0	1.491	4.98	0.589
GM0563	34.22	33.73	1973	C	0.013	225	1	0.22	2.38	0	1.465	1.66	0.734
GM0564	33.73	32.96	1973	C	0.013	348	1	0.22	2.42	0	1.467	1.68	0.734
GM0565	32.88	32.57	1973	C	0.013	143	1	0.22	2.57	0	1.469	1.66	0.701
GM0646	34.8	34.22	1973	C	0.013	213.42	1	0.27	2.54	0	1.459	1.86	0.731
GM0647	35.25	34.9	1973	C	0.013	158.5	1	0.22	2.52	0	1.454	1.67	0.701
GM0648	35.97	35.35	1973	C	0.013	282.01	1	0.22	2.42	0	1.435	1.67	0.719
GM0649	36.26	36.07	1973	C	0.013	83.71	1	0.23	2.56	0	1.423	1.7	0.671
GM0650	37.47	36.86	1973	C	0.013	303.75	1	0.2	2.33	0	1.401	1.6	0.728
GM0652	38.1	37.47	1973	C	0.013	282.39	1	0.22	2.35	0	1.377	1.68	0.728
GM0653	38.56	38.2	1973	C	0.013	166.62	1	0.22	2.44	0	1.369	1.66	0.681
GM0654	39.2	38.66	1973	C	0.013	251.66	1	0.21	1.17	0	0.131	1.65	0.597
GM0675	50.55	49.605	1973	C	0.013	276.17	1.5	0.342	3.68	0	4.033	6.14	0.604
GM0676	52.19	50.55	1973	C	0.013	238.88	1.25	0.69	4.71	0	4.023	5.35	0.725
GM0678	49.505	48.613	1973	C	0.013	239.91	1.5	0.372	3.81	0	4.048	6.41	0.587
GM0682	48.513	47.515	1973	C	0.013	268.29	1.5	0.372	3.8	0	4.051	6.41	0.589
GM0686	47.414	46.386	1973	C	0.013	276.3	1.5	0.372	3.79	0	4.038	6.41	0.589
GM0687	46.286	45.3	1973	C	0.013	241.55	1.5	0.408	3.93	0	4.046	6.71	0.571
GM0708	11.19	3.19	1973	C	0.013	506.38	1.75	1.58	5.54	0	4.196	19.92	0.795
GM0709	19.11	11.19	1973	C	0.013	486.49	1.25	1.63	6.38	0	4.156	8.24	0.534
GM0710	29.26	19.11	1973	C	0.013	528.42	1.25	1.92	7	0	4.068	8.95	0.534

Link Name	US Invert (ft)	DS Invert (ft)	Installation Year	Material	Roughness	Length (ft)	Diameter (ft)	Slope	Max Velocity (ft/s)	Min Velocity (ft/s)	Max Flow (cfs)	Design Full Flow (cfs)	Max d/D
GM0711	32.47	29.26	1973	C	0.013	349.15	1.25	0.92	5.09	0	4.065	6.19	0.637
GM0712	40.43	32.47	1973	C	0.013	298.88	1.25	2.66	7.55	0	4.056	10.54	0.637
GM0713	44.12	40.8	1973	C	0.013	353.62	1.25	0.94	5.42	0	4.046	6.26	0.586
GM0714	45.3	44.12	1973	C	0.013	140.75	1.25	0.84	5.18	0	4.049	5.91	0.612
GM0724	30.96	20.91	1973	C	0.013	494.46	1.5	2.03	6.95	0	3.695	14.98	0.361
GM0725	20.91	18.4	1973	C	0.013	149.79	1.5	1.68	6.49	0	3.715	13.6	0.361
GM0726	18.4	10.03	1973	C	0.013	452.82	1.5	1.85	6.57	0	3.72	14.28	0.431
GM0727	10.03	7.3	1973	C	0.013	320.07	1.5	0.85	5.13	0	3.739	9.7	0.431
GM0728	7.24	5.65	1973	C	0.013	130.72	1.5	1.22	5.76	0	3.742	11.59	0.475
GM0729	5.24	5.14	1973	C	0.013	66.1	2	0.15	2.94	0	5.336	8.8	0.562
GM0730	4.726	4.398	1973	C	0.013	280.43	2	0.117	2.68	0	5.369	7.74	0.61
GM0731	4.398	4.138	1973	C	0.013	222.56	2	0.117	2.7	0	5.372	7.73	0.606
GM0732	4.138	3.813	1973	C	0.013	277.68	2	0.117	2.76	0	5.381	7.74	0.598
GM0733	3.813	3.48	1973	C	0.013	285.5	2	0.117	3.02	0	5.387	7.73	0.57
GM0734	2.62	2.36	1973	C	0.013	536.48	2.5	0.048	2.8	0	10.828	9.03	0.747
GM0735	2.95	2.62	1973	C	0.013	299.72	2.5	0.11	1.85	0	6.184	13.61	0.747
GM0747	7.16	6.33	1973	C	0.013	380.5	1.25	0.22	2.54	0	1.78	3.02	0.572
GM0748	6.33	5.49	1973	C	0.013	382.43	1.25	0.22	2.46	0	1.797	3.03	0.699
GM0752	5.04	4.726	1973	C	0.013	268.42	2	0.117	2.67	0	5.362	7.74	0.612
GM0757	0.94	0.52	1973	C	0.013	350	2.5	0.12	3.39	0	10.963	14.21	0.631
GM0758	1.33	0.94	1973	C	0.013	261.64	2.5	0.149	3.42	0	10.975	15.84	0.631
GM0759	1.87	1.43	1973	C	0.013	248.65	2.5	0.177	3.72	0	10.981	17.25	0.582
GM0760	2.36	1.97	1973	C	0.013	349.89	2.5	0.11	3.45	0	10.992	13.69	0.628
GM0761	0.52	0.15	1973	C	0.013	268	2.5	0.138	3.78	0	10.954	15.24	0.582
GM0778	42.11	41.9	1973	C	0.013	180.46	1.5	0.116	2.79	0	3.258	3.58	0.645
GM0779	42.05	41.81	1973	C	0.013	85.69	1.25	0.28	0.68	0	0.054	3.42	0.646
GM0782	43.61	43.25	1973	C	0.013	119.48	0.833	0.3	0.82	0	0.022	1.2	0.124
GM0783	43.25	42.15	1973	C	0.013	426.68	0.833	0.26	0.92	0	0.034	1.11	0.562
GM0785	41.23	40.7	1973	C	0.013	129.73	1.5	0.41	3.54	0	3.596	6.71	0.653
GM0786	41.61	41.23	1973	C	0.013	283.85	1.5	0.13	2.65	0	3.275	3.84	0.672
GM0787	40.7	39.79	1973	C	0.013	413.76	1.5	0.22	3.03	0	3.595	4.93	0.653
GM0792	39.88	40.5	1973	C	0.013	186.15	1	-0.333	-0.85	0	-0.021	2.06	0.695
GM0793	41.05	39.8	1973	C	0.013	126.8	0.667	0.99	3.73	0	0.851	1.2	0.622
GM0798	46	45.46	1973	C	0.013	131.83	0.833	0.41	1.24	0	0.059	1.4	0.165
GM0799	45.46	43.86	1973	C	0.013	402.44	0.833	0.4	1.24	0	0.071	1.38	0.165
GM0800	43.86	40.11	1973	C	0.013	341.39	0.833	1.1	1.99	0	0.085	2.29	0.31
GM0805	45.12	44.6	1973	C	0.013	113.72	0.833	0.457	3.03	0	1.195	1.48	0.691
GM0806	44.2	43.2	1973	CI	0.015	91.5	1	1.093	3.6	0	1.199	3.23	0.782
GM0807	43.2	43.08	1973	C	0.013	79.98	1.25	0.15	2.1	0	1.2	2.5	0.702
GM0808	43.06	42.57	1973	C	0.013	229.8	1.5	0.213	2.95	0	3.254	4.85	0.599
GM0809	42.57	42.21	1973	C	0.013	160.35	1.5	0.225	3.02	0	3.254	4.98	0.588
GM0823	26.77	19.58	1973	C	0.013	438.75	0.667	1.64	4.54	0	0.853	1.55	0.53
GM0824	19.43	16.65	1973	C	0.013	277.63	0.667	1	3.75	0	0.849	1.21	0.618
GM0825	16.3	10.54	1973	C	0.013	512.2	1	1.125	3.62	0	0.848	3.78	0.624

Link Name	US Invert (ft)	DS Invert (ft)	Installation Year	Material	Roughness	Length (ft)	Diameter (ft)	Slope	Max Velocity (ft/s)	Min Velocity (ft/s)	Max Flow (cfs)	Design Full Flow (cfs)	Max d/D
GM0829	14.01	13.11	1973	C	0.013	232.8	0.833	0.39	1.05	0	0.036	1.36	0.129
GM0830	13.11	12.85	1973	C	0.013	63.89	0.833	0.41	1.09	0	0.044	1.4	0.129
GM0831	12.85	10.28	1973	C	0.013	240.49	0.833	1.07	1.62	0	0.045	2.26	0.099
GM0832	10.49	9.45	1973	C	0.013	359.73	1.25	0.29	2.75	0	1.667	3.47	0.547
GM0833	9.45	8.95	1973	C	0.013	230.1	1.25	0.22	2.55	0	1.725	3.01	0.547
GM0836	8.85	8.1	1973	C	0.013	335.24	1.25	0.22	2.54	0	1.732	3.06	0.552
GM0837	8	7.16	1973	C	0.013	381.13	1.25	0.22	2.56	0	1.76	3.03	0.554
GM0838	39.31	26.93	1973	C	0.013	475.22	0.667	2.61	5.4	0	0.853	1.95	0.463
GM0848	16.11	15.64	2007	PVC	0.011	69.16	1.75	0.68	4.82	0	2.702	15.44	0.285
GM0849	47.73	47.17	1981	PVC	0.011	236.09	0.667	0.237	2.32	0	0.723	0.7	0.869
GM0878	4.99	4.64	2009	PVC	0.011	111.16	1	0.315	0.39	0	-0.011	2.36	0.97
GM0880	15.29	13.06	2007	PVC	0.011	483.76	1.75	0.44	4.07	0	2.419	12.71	0.297
GM0881	12.91	10.88	2007	PVC	0.011	526.75	1.75	0.4	3.61	0	2.367	11.62	0.428
GM0882	8.24	7.86	2007	PVC	0.011	53.79	1.75	0.71	4.57	0	2.137	15.74	0.249
GM0883	10.88	10.68	2007	PVC	0.011	265.82	1.75	0.075	2.34	0	2.208	5.14	0.428
GM0893	10.58	8.44	2007	PVC	0.011	502.69	1.75	0.426	3.82	0	2.133	12.22	0.283
Link295_SmithLS	32	52.19	0		0.014	1531.04	1	-1.319	4.3	0	4.03	3.8	21.003
Link297 (FM)									0	0	2.674	0	0
Link297 (FM)									0	0	2.472	0	0
Link298	32.07	32			0.014	28.87	1.5	0.242	3.9	0	4.163	4.8	0.589
Link299 (FM)									0	0	0.607	0	0
Link300_HWY30_F	33.93	60.55	0		0.014	256.52	0.5	-10.563	2.28	0	0.589	1.68	54.087
Link303	8.91	12.5			0.014	1565	0.33	-0.229	2.1	0	0.128	0.08	46.507
Link304_SevenOa									0	0	1.192	0	0
Link305_SpringL									0	0	1.014	0	0
Link306	8	15.5			0.014	1726	0.33	-0.435	1.56	0	0.07	0.11	35.523
Link307	185	181.77			0.014	24.53	0.208	13.168	11.03	0	0.501	0.18	59.344
Link308	181.77	179.87			0.014	142.41	0.667	1.334	3.45	0	0.501	1.3	0.451
Link309	179.87	178.05			0.014	159.08	0.667	1.144	3.23	0	0.501	1.2	0.508
Link310	178.05	176.97			0.014	94.06	0.667	1.148	2.99	0	0.501	1.2	0.508
Link311	176.97	154.4			0.014	264.94	0.667	8.519	6.81	0	0.511	3.28	0.267
Link312	153.99	124.48			0.014	251.95	0.667	11.713	7.62	0	0.511	3.84	0.251
Link313	124.48	89.83			0.014	271.69	0.667	12.754	7.85	0	0.537	4.01	0.251
Link314	89.83	56			0.014	233.67	0.667	14.478	8.36	0	0.537	4.27	0.239
Link315	54.96	54.33			0.014	21.54	0.667	2.925	4.65	0	0.537	1.92	0.395
Link316	54.33	45.97			0.014	323.11	0.667	2.587	4.63	0	0.594	1.81	0.395
Link317	6.28	6.21			0.014	268.81	0.667	0.026	0.99	0	0.276	0.18	0.784
Link318	6.21	5.63			0.014	146.6	0.667	0.396	1.88	0	0.305	0.71	0.483
Link319	5.51	4.59			0.014	264	0.667	0.348	1.76	0	0.305	0.66	0.51
Link320	4.49	3.84			0.014	202	0.667	0.322	1.74	0	0.305	0.64	0.513
Link321	3.74	3.64			0.014	64.73	0.667	0.154	1.59	0	0.305	0.44	0.556
Link324	41.62	37.62			0.014	1600	1	0.25	11.68	0	0.444	23.16	0.101
Link325 (LS)									0	0	0	0	0
Link325 (FM)									0	0	0.39	0	0

Link Name	US Invert (ft)	DS Invert (ft)	Installation Year	Material	Roughness	Length (ft)	Diameter (ft)	Slope	Max Velocity (ft/s)	Min Velocity (ft/s)	Max Flow (cfs)	Design Full Flow (cfs)	Max d/D
Link326 (LS)									0	0	0	0	0
Link326 (FM)									0	0	1.109	0	0
Link328 (LS)									0	0	0	0	0
Link328 (FM)									0	0	0.439	0	0
Link329	66	47.51			0.014	4300	0.667	0.43	2.1	0	0.498	0.74	0.66
Link330	118	66			0.014	600	0.667	8.667	34.72	0	0.446	46.31	0.66
Link331 (LS)									0	0	0	0	0
Link331 (FM)									0	0	1.175	0	0
Link333	48.61	22.16			0.011	2600	1	1.017	26.53	0	1.449	46.72	0.256
Link334	39.38	31.06			0.013	3200	1.5	0.26	3.03	0	3.617	5.36	0.659

F-4:
Future Manhole Data

Alternative B Future Flows (2035; 5-year, 24-hour storm event)

Node Name	Ground Elevation (ft)	Invert Elevation (ft)	Constant Inflow (cfs)	Dry Weather Flow Flag	Temporal Variation	Peaking Factor	Volume of Const. Inflow (ft^3)	Volume of DWF Inflow (ft^3)	Time Flooded (min)	Time Surcharged (min)	Max Depth (ft)
Keys_WTP	202	185	0	1	Keys WTP	1	0	116581	0	5144.798	12.361
MH0019	22.5	16.11	0	1	Scappoose_Diurnal	0	0	0	0	0	0.497
MH0020	23.7	17.89	0	1	Scappoose_Diurnal	0	0	0	0	0	0.55
MH0021	25	19.52	0	1	Scappoose_Diurnal	0	0	0	0	0	0.61
MH0022	26.2	22.11	0	1	Site_5_DWF	1	0	125003.52	0	0	0.297
MH0048	62.73	55.13	0.02	1	Site_3_DWF	0	0	2030.05	0	0	0.674
MH0049	67.27	56.35	0.002	1	Site_3_DWF	1	0	0	0	0	0.449
MH0059	15.34	6.6	0.012	1	Site_4_DWF	1	0	1746.14	0	0	0.075
MH0061	15.78	7.9	0.042	1	Site_4_DWF	0	0	6226.56	0	0	0.073
MH0098	62.41	55.11	0.012	1	Site_3_DWF	0	0	1212.39	0	0	0.571
MH0099	62.35	53.89	0.015	1	Site_3_DWF	0	0	1212.39	0	0	0.417
MH0125	52.5	46.1	0.001	1	Site_1_DWF	0	0	160.27	0	0	0.195
MH0126	55.2	46.77	0.159	1	Site_1_DWF	1	0	26170.13	0	0	0.231
MH0127	56.5	47.25	0.012	1	Site_1_DWF	0	0	1980.5	0	0	0.105
MH0128	55.6	48.42	0.049	1	Site_1_DWF	0	0	8059.39	0	0	0.098
MH0130	18.18	12.35	0	1	Site_3_DWF	0	0	0	0	0	0.178
MH0131	20.62	12.66	0.016	1	Site_3_DWF	0	0	1635.32	0	0	0.179
MH0132	21.5	13.76	0.02	1	Site_3_DWF	0	0	2030.05	0	0	0.177
MH0133	59.93	47.73	0.011	1	Site_3_DWF	0	0	817.66	0	0	0.579
MH0136	17.27	13.91	0.093	1	Site_3_DWF	0	0	8909.68	0	0	0.217
MH0137	17.76	14.13	0.158	1	Site_3_DWF	1	0	8120.22	0	0	0.093
MH0154	17.7	7.24	0.009	1	Site_4_DWF	0	0	1367.14	0	0	0.587
MH0173	12.7	1.33	0.044	1	Site_4_DWF	0	0	2138.69	0	0	1.554
MH0182	46.9	35.42	0.013	1	Site_3_DWF	1	0	1212.39	0	0	0.115
MH0184	47.01	37.22	0.1	1	Site_3_DWF	1	0	9727.34	0	0	0.11
MH0198	65.09	54.48	0.015	1	Site_1_DWF	0	0	2518.56	0	0	0.206
MH0202	60.8	54.05	0.015	1	Site_1_DWF	0	0	2472.77	0	0	0.216
MH0203	59.5	53.47	0.004	1	Site_1_DWF	0	0	663.98	0	0	0.217
MH0204	58.5	52.93	0.021	1	Site_1_DWF	0	0	3457.3	0	0	0.23
MH0205	59.4	53.21	0	1	Site_1_DWF	0	0	0	0	0	0.219
MH0206	60.07	52.53	0.009	1	Site_1_DWF	0	0	1476.79	0	0	0.227
MH0207	59.44	51.7	0	1	Site_1_DWF	0	0	0	0	0	0.358
MH0212	22.2	15.29	0.009	1	Scappoose_Diurnal	0	0	0	0	0	0.519
MH0214	46.96	36.76	0.035	1	Site_1_DWF	1	0	5758.34	0	0	0.723
MH0215	16.84	11.48	0.045	1	Site_3_DWF	1	0	2058.25	0	0	0.21
MH0251	61.66	52.36	0.032	1	Site_3_DWF	0	0	2847.72	0	337.744	0.864
MH0254	57.4	49.8	0.027	1	Site_1_DWF	0	0	4441.82	0	0	0.221
MH0255	58	49.4	0.009	1	Site_1_DWF	0	0	1476.79	0	0	0.222
MH0256	62	51.14	0.018	1	Site_1_DWF	0	0	2965.03	0	0	0.201
MH0257	61.4	51.52	0.013	1	Site_1_DWF	0	0	2175.12	0	0	0.525
MH0262	61.6	49	0.032	1	Site_3_DWF	0	0	2847.72	0	479.896	0.915
MH0284	69.8	57.4	0.035	1	Site_3_DWF	0	0	3242.45	0	0	0.468
MH0285	68.49	58.36	0.03	1	Site_3_DWF	1	0	35356.78	0	0	0.465
MH0286	64.6	59.55	0.033	1	Site_3_DWF	0	0	2847.72	0	0	0.412

Node Name	Ground Elevation (ft)	Invert Elevation (ft)	Constant Inflow (cfs)	Dry Weather Flow Flag	Temporal Variation	Peaking Factor	Volume of Const. Inflow (ft^3)	Volume of DWF Inflow (ft^3)	Time Flooded (min)	Time Surcharged (min)	Max Depth (ft)
MH0287	63	60.55	0.024	1	Site_3_DWF	0	0	2030.05	0	0	0.423
MH0290	17.3	10.54	0	1	Site_3_DWF	0	0	0	0	0	0.624
MH0291	16.5	12.19	0.088	1	Site_3_DWF	1	0	0	0	0	0.28
MH0293	53.03	41.23	0.005	1	Site_1_DWF	0	0	824.26	0	0	0.822
MH0304	56.93	45.35	0.002	1	Site_3_DWF	0	0	0	0	0	0.515
MH0305	58.93	43.76	0.024	1	Site_3_DWF	0	0	2030.05	0	0	0.345
MH0306	58.93	44.24	0.01	1	Site_3_DWF	0	0	817.66	0	0	0.483
MH0307	59.81	46.91	0.002	1	Site_3_DWF	1	0	20441.52	0	0	0.521
MH0308	59.77	47.07	0.004	1	Site_3_DWF	0	0	394.73	0	0	0.507
MH0309	61.37	48.77	0.025	1	Site_3_DWF	0	0	2424.79	0	0	0.401
MH0310	61.37	50.47	0.025	1	Site_3_DWF	1	0	2424.79	0	0	0.392
MH0311	61.77	52.17	0.026	1	Site_3_DWF	1	0	2424.79	0	0	0.474
MH0318	51.9	44.2	0.003	1	Site_1_DWF	0	0	492.26	0	0	0.333
MH0319	50.4	44.48	0.01	1	Site_1_DWF	0	0	1648.51	0	0	0.326
MH0320	53	44.85	0.003	1	Site_1_DWF	0	0	492.26	0	0	0.321
MH0321	53	45.16	0.005	1	Site_1_DWF	0	0	824.26	0	0	0.318
MH0322	51.3	45.4	0.002	1	Site_1_DWF	0	0	331.99	0	0	0.316
MH0323	57.3	45.68	0.003	1	Site_1_DWF	0	0	492.26	0	0	0.318
MH0324	57.8	45.8	0.04	1	Site_1_DWF	0	0	6582.6	0	0	0.316
MH0325	59.2	46.78	0.022	1	Site_1_DWF	0	0	3617.57	0	0	0.293
MH0326	58	47.22	0.007	1	Site_1_DWF	0	0	1156.25	0	0	0.268
MH0327	58.2	47.39	0.013	1	Site_1_DWF	0	0	2140.78	0	0	0.274
MH0328	60.9	47.73	0.003	1	Site_1_DWF	0	0	492.26	0	0	0.262
MH0329	61	47.8	0	1	Site_1_DWF	0	0	0	0	0	0.275
MH0373	56.81	41.05	0.046	1	Site_3_DWF	1	0	4454.84	0	0	0.415
MH0376	53.72	45.3	0.022	1	Site_5_DWF	0	0	817.34	0	0	0.764
MH0393	44.96	32.07	0.002	1	Site_5_DWF	1	0	2305.49	0	0	0.883
MH0395	44.63	32.47	0.018	1	Site_5_DWF	1	0	1634.69	0	0	0.649
MH0401	48.2	34.4	0	1	Site_2_DWF	1	0	51418.8	0	0	0.619
MH0402MTR2	46.2	33.4	0.415	1	Site_2_DWF	1	0	0	0	0	0.506
MH0421	40.38	34.22	0.031	1	Site_5_DWF	0	0	2737.15	0	0	0.731
MH0422	41.96	33.73	0.01	1	Site_5_DWF	0	0	817.34	0	0	0.734
MH0423	39.96	32.88	0.007	1	Site_5_DWF	0	0	551.23	0	0	0.701
MH0431	57.33	49.505	0.113	1	Site_5_DWF	0	0	4371.84	0	0	0.88
MH0473	47.96	37.47	0.042	1	Site_1_DWF	1	0	11752.79	0	0	0.728
MH0483	46.96	36.26	0.003	1	Site_1_DWF	1	0	4639.29	0	0	0.671
MH0484MTR1	42.17	35.97	0.04	1	Site_1_DWF	1	0	6582.6	0	0	0.719
MH0485	44.96	35.25	0.099	1	Site_5_DWF	0	0	8477.57	0	0	0.701
MH0486	44.96	34.8	0.025	1	Site_5_DWF	0	0	2185.92	0	0	0.68
MH0487	49.16	40.5	0.025	1	Site_1_DWF	1	0	4109.83	0	0	0.075
MH0488	48.5	38.56	0.059	1	Site_1_DWF	0	0	9707.9	0	0	0.681
MH0489	48.5	38.1	0.028	1	Site_1_DWF	0	0	4613.54	0	0	0.696
MH0506	57.1	46.286	0.069	1	Site_5_DWF	1	0	2737.15	0	0	0.856
MH0507	57.17	47.414	0.017	1	Site_5_DWF	0	0	551.23	0	0	0.884
MH0510	56.83	48.513	0.025	1	Site_5_DWF	0	0	817.34	0	0	0.884

Node Name	Ground Elevation (ft)	Invert Elevation (ft)	Constant Inflow (cfs)	Dry Weather Flow Flag	Temporal Variation	Peaking Factor	Volume of Const. Inflow (ft^3)	Volume of DWF Inflow (ft^3)	Time Flooded (min)	Time Surcharged (min)	Max Depth (ft)
MH0513	57.97	50.55	0.051	1	Site_5_DWF	1	0	1919.81	0	0	0.906
MH0514	61.43	52.19	0.046	1	Site_5_DWF	0	0	1634.69	0	0	0.813
MH0525	53.3	44.12	0.018	1	Site_5_DWF	0	0	551.23	0	0	0.732
MH0526	50.2	40.43	0.09	1	Site_5_DWF	0	0	3554.5	0	0	0.538
MH0530	46.37	32.47	0.081	1	Site_5_DWF	0	0	3003.26	0	0	0.796
MH0533	40	29.26	0.053	1	Site_5_DWF	1	0	1919.81	0	0	0.591
MH0534	32.37	19.11	0.03	1	Site_5_DWF	1	0	28236.59	0	0	0.668
MH0536	20.49	11.19	0.009	1	Site_5_DWF	1	0	11292.59	0	0	0.545
MH0537MTR5	15.5	2.95	0.004	1	Site_5_DWF	1	0	0	0	0	1.632
MH0549	41.05	30.96	0.22	1	Site_4_DWF	1	0	32932.56	0	0	0.508
MH0561	22.83	14.01	0.056	1	Site_4_DWF	0	0	8175.74	0	0	0.093
MH0565	30.81	20.91	0.056	1	Site_4_DWF	0	0	8175.74	0	0	0.541
MH0566	30.5	18.4	0.015	1	Site_4_DWF	1	0	2138.69	0	0	0.522
MH0567	19.96	10.03	0.026	1	Site_4_DWF	1	0	7803.97	0	0	0.646
MH0568	16.66	5.24	0.006	1	Site_4_DWF	0	0	974.59	0	0	1.123
MH0569	15.78	5.04	0.003	1	Site_4_DWF	0	0	392.54	0	0	1.223
MH0570	16.71	6.33	0.015	1	Site_4_DWF	1	0	7046.62	0	0	0.715
MH0571	16.71	7.16	0.02	1	Site_4_DWF	1	0	8177.33	0	0	0.693
MH0572	15.92	4.398	0.02	1	Site_4_DWF	0	0	974.59	0	0	1.212
MH0573	13.48	4.138	0.018	1	Site_4_DWF	1	0	2771.86	0	0	1.195
MH0574	14.5	2.62	0.02	1	Site_4_DWF	1	0	2720.74	0	0	1.868
MH0575MTR4	15.6	4.726	0.016	1	Site_4_DWF	1	0	3113.28	0	0	1.221
MH0576	11.86	3.813	0.028	1	Site_4_DWF	0	0	1367.14	0	0	1.141
MH0577	14.17	2.36	0.052	1	Site_4_DWF	1	0	0	0	0	1.569
MH0578	12.34	0.94	0.009	1	Site_4_DWF	1	0	392.54	0	0	1.577
MH0579	13.82	0.52	0	1	Site_4_DWF	0	0	0	0	0	1.456
MH0580	12.82	0.1	0	1	Site_4_DWF	0	0	0	0	0	1.239
MH0581	14.95	1.87	0.048	1	Site_4_DWF	1	0	2341.73	0	0	1.451
MH0591	52.41	39.31	0.013	1	Site_3_DWF	0	0	1212.39	0	0	0.309
MH0594	57.96	46	0.069	1	Site_1_DWF	0	0	11356.42	0	0	0.117
MH0595	58.02	45.46	0.014	1	Site_1_DWF	0	0	2301.05	0	0	0.137
MH0597	54.12	43.86	0.016	1	Site_1_DWF	0	0	2633.04	0	0	0.11
MH0598	50.94	39.38	0.017	1	Site_1_DWF	0	0	2793.31	0	0	0.988
MH0603	52	40.7	0.008	1	Site_1_DWF	0	0	1316.52	0	0	0.98
MH0604	53.03	41.61	0.009	1	Site_1_DWF	0	0	1476.79	0	0	1.008
MH0605	53.03	42.05	0.021	1	Site_1_DWF	0	0	3457.3	0	0	0.568
MH0606	52.71	42.11	0.018	1	Site_1_DWF	0	0	2965.03	0	0	0.968
MH0607	52.83	42.57	0.004	1	Site_1_DWF	0	0	663.98	0	0	0.881
MH0608	51.78	43.06	0	1	Site_1_DWF	0	0	0	0	0	0.898
MH0609	53.73	43.25	0.014	1	Site_1_DWF	0	0	2301.05	0	0	0.104
MH0610	56.03	43.61	0.026	1	Site_1_DWF	0	0	4281.55	0	0	0.079
MH0614	51.2	43.2	0.001	1	Site_1_DWF	0	0	160.27	0	0	0.782
MH0615	52.7	44.2	0.006	1	Site_1_DWF	0	0	984.53	0	0	0.424
MH0616	52.6	45.12	0	1	Site_1_DWF	1	0	0	0	0	0.575
MH0634	39.43	26.77	0.01	1	Site_3_DWF	1	0	817.66	0	0	0.353

Node Name	Ground Elevation (ft)	Invert Elevation (ft)	Constant Inflow (cfs)	Dry Weather Flow Flag	Temporal Variation	Peaking Factor	Volume of Const. Inflow (ft^3)	Volume of DWF Inflow (ft^3)	Time Flooded (min)	Time Surcharged (min)	Max Depth (ft)
MH0635	15.71	8	0.026	1	Site_4_DWF	1	0	9981.43	0	0	0.684
MH0636	20.71	8.85	0.019	1	Site_4_DWF	1	0	2720.74	0	0	0.691
MH0637MTR3	20.49	9.45	0.011	1	Site_3_DWF	1	0	4454.84	0	0	0.684
MH0638	17.78	10.49	0.024	1	Site_3_DWF	1	0	394.73	0	0	0.612
MH0639	21.91	12.85	0.003	1	Site_4_DWF	0	0	392.54	0	0	0.082
MH0640	21.91	13.11	0.012	1	Site_4_DWF	0	0	1746.14	0	0	0.108
MH0642	22.79	16.3	0.006	1	Site_3_DWF	0	0	394.73	0	0	0.322
MH0643	30.44	19.43	0	1	Site_3_DWF	0	0	0	0	0	0.412
MH0663	18.5	10.88	0.017	1	Scappoose_Diurnal	0	0	0	0	0	0.748
MH0664	20.2	12.91	0.019	1	Scappoose_Diurnal	1	0	18700.08	0	0	0.536
MH0665	19	8.24	0.007	1	Scappoose_Diurnal	1	0	0	0	0	0.435
MH0674	18.5	10.58	0.011	1	Scappoose_Diurnal	0	0	1093.28	0	0	0.496
Miller_WTP_Infl	10.4	6.28	0	1	Miller WTP	1	0	27820.8	0	0	0.523
MMH0001	15	4.99	0.01	1	Site_4_DWF	0	0	582.05	0	0	0.62
MMH0002	10	0	0.001	1	Site_4_DWF	0	0	0	0	0	1.108
MMH0003	63	54.83	0.022	1	Site_1_DWF	1	0	32450.64	0	0	0.206
MMH0004	56.6	48.94	0.004	1	Site_1_DWF	0	0	663.98	0	0	0.03
Node449	46.6	24	0.021	1	Site_3_DWF	1	0	29604.96	0	0	8.502
Node450	45	24.5	0	1	Site_5_DWF	1	0	0	0	0	8.007
Node451	45	32	0	1	Site_5_DWF	0	0	0	5607.3	5637.1	37.193
Node452	52.93	33.93	0	1	Scappoose_Diurnal	0	0	0	5428.6	5499	29.048
Node455	13.91	8.91	0	0	0	0	0	0	4943.7	5406.071	18.915
Node456	18.4	8	0	0	0	0	0	0	4655.7	5469.083	19.187
Node457	18.4	8.8	0	1	Site_3_DWF	1	0	8712.32	0	5760	3.004
Node458	10.96	-3	0	1	Site_3_DWF	1	0	10911.54	0	5760	6.07
Node461	67.5	54.33	0	1	Site_1_DWF	1	0	23937.77	0	0	0.263
Node462	69.5	54.96	0	0	0	0	0	0	0	0	0.241
Node463	105.5	89.83	0	0	0	0	0	0	0	0	0.16
Node464	153.5	124.48	0	1	Site_1_DWF	1	0	13823.42	0	0	0.168
Node465	177.5	153.99	0	0	0	0	0	0	0	0	0.164
Node466	184	176.97	0	1	Site_1_DWF	1	0	5080.11	0	0	0.178
Node467	192	178.05	0	0	0	0	0	0	0	0	0.339
Node468	194	179.87	0	0	0	0	0	0	0	0	0.301
Node469	192	181.77	0	0	0	0	0	0	0	0	0.288
Node471	12.63	3.74	0	0	0	0	0	0	0	0	0.371
Node472	13.51	4.49	0	0	0	0	0	0	0	0	0.342
Node473	13.01	5.51	0	0	0	0	0	0	0	0	0.34
Node474	13.26	6.21	0	1	Site_4_DWF	1	0	6566.95	0	0	0.322
Node476_P.LS1	50	40	0	1	Industrial	1	0	16174.08	0	5734.3	6.002
Node478	50	41.62	0	0	0	0	0	0	0	0	0.101
Node479_P.LS3	10	0	0	1	Industrial	1	0	58475.52	0	5752.8	6.025
Node482_P.LS4	72	62	0	1	Site_1_DWF	1	0	52217.98	0	5749.8	6.005
Node483	121	118	0	0	0	0	0	0	0	0	0.046
Node484	70	66	0	1	Site_1_DWF	1	0	33971.06	0	0	0.44
Node485_P.LS2	24	14	0	1	Industrial	1	0	55157.76	0	5752.4	6.019

Node Name	Ground Elevation (ft)	Invert Elevation (ft)	Constant Inflow (cfs)	Dry Weather Flow Flag	Temporal Variation	Peaking Factor	Volume of Const. Inflow (ft^3)	Volume of DWF Inflow (ft^3)	Time Flooded (min)	Time Surcharged (min)	Max Depth (ft)
Node488	52	48.61	0	0	0	0	0	0	0	0	0.13

APPENDIX G – Project Summaries

Collection System Project: New Relief Trunk Line

Project Identifier: 1A.1

Objective: The new, 18", relief trunk line will alleviate current surcharging and flooding on the western side of town. This line will eliminate the need to upszie Smith Road Lift Station during the 20-year planning period. The pipeline along Maple Street will need to be re-graded to flow the opposite direction of the existing line.

Key Issues: The pipeline will need to be bored under the highway and railroad.

Project Location: SW Maple Street & SE Elm Street

Item	Unit	Unit Price	Quantity	Cost (2016)
18" Pipe - excavation, backfill, less than 10' deep	LF	$ 95	5575	$ 529,644
Reconnect Services	LF	$ 10	5575	$ 55,752
48" Manhole - 8" thru 18" pipes	EA	$ 4,250	23	$ 97,750
Connect Pipes at Manhole - 8" thru 21"	EA	$ 1,500	4	$ 6,000
1/2 Lane Pavement Repair	LF	$ 30	5575	$ 167,256
Bore and Jacking	LF	$ 650	180	$ 117,000
Bypass Pumping	EA	$ 5,000	1	$ 5,000
Traffic Control	LF	$ 2	5575	$ 11,150
Mobilization	%	5%	-	$ 49,478
Contingency	%	30%	-	$ 311,709
Construction Subtotal (rounded)				**$ 1,351,000**
Engineering and CMS	%	25%	-	$ 337,750
Legal, Admin, and Permitting	%	2%	-	$ 27,020
Total Project Cost (rounded)				**$ 1,720,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Collection System Project: E Columbia Avenue

Project Identifier: 1A.2

Objective: The existing 15" and 18" line along E Columbia Avenue are undersized for current peak flows. The replacement for this section is a 30" pipe. This new, 30" line will eliminate current surcharging and flooding and provide capacity for future growth.

Project Location: E Columbia Avenue

Item	Unit	Unit Price	Quantity	Cost (2016)
21" Pipe - excavation, backfill, less than 10' deep	LF	$ 120	506	$ 60,768
30" Pipe - excavation, backfill, less than 10' deep	LF	$ 185	2380	$ 440,319
Reconnect Services	LF	$ 10	2887	$ 28,865
48" Manhole - 8" thru 18" pipes	EA	$ 4,250	1	$ 4,250
Connect Pipes at Manhole - 8" thru 21"	EA	$ 1,500	1	$ 1,500
72" Manhole - >18" pipes	EA	$ 5,250	9	$ 47,250
Connect Pipes at Manhole - 24" thru 36"	EA	$ 3,000	18	$ 54,000
1/2 Lane Pavement Repair	LF	$ 30	2887	$ 86,595
Bypass Pumping	EA	$ 10,000	1	$ 10,000
Traffic Control	LF	$ 2	2887	$ 5,773
Mobilization	%	5%	-	$ 36,966
Contingency	%	30%	-	$ 232,886
Construction Subtotal (rounded)				**$ 1,010,000**
Engineering and CMS	%	25%	-	$ 252,500
Legal, Admin, and Permitting	%	2%	-	$ 20,200
Total Project Cost (rounded)				**$ 1,290,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Collection System Project: SE Tussing Way and SE Tyler Street

Project Identifier: 1A.3

Objective: The existing 15" line along SE Tussing Way and SE Tyler Street is undersized for current peak flows. The replacement for this section is a 24" pipe eliminate current surcharging and flooding.

Project Location: SE Tussing Way and SE Tyler Street

Item	Unit	Unit Price	Quantity	Cost (2016)
24" Pipe - excavation, backfill, less than 10' deep	LF	$ 150	1401	$ 210,105
Reconnect Services	LF	$ 10	1401	$ 14,007
72" Manhole - >18" pipes	EA	$ 5,250	6	$ 31,500
Connect Pipes at Manhole - 24" thru 36"	EA	$ 3,000	5	$ 15,000
Full Lane Pavement Repair	LF	$ 60	1401	$ 84,042
Bypass Pumping	EA	$ 5,000	1	$ 5,000
Traffic Control	LF	$ 2	1401	$ 2,801
Mobilization	%	5%	-	$ 18,123
Contingency	%	30%	-	$ 114,173
Construction Subtotal (rounded)				**$ 495,000**
Engineering and CMS	%	25%	-	$ 123,750
Legal, Admin, and Permitting	%	2%	-	$ 9,900
Total Project Cost (rounded)				**$ 630,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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

Collection System Project: SW Em Watts Road

Project Identifier: **1A.4**

Objective: The existing line on SW Em Watts Road from SW Johanna Drive to just east of SW 4th Street is undersized for current flows. This line should be replaced with 10", 15", and 18" sections as labeled below.

Project Location: SW Em Watts Road

Item	Unit	Unit Price		Quantity	Cost (2016)	
10" Pipe - excavation, backfill, less than 10' deep	LF	$	65	114	$	7,391
15" Pipe - excavation, backfill, less than 10' deep	LF	$	75	166	$	12,420
18" Pipe - excavation, backfill, less than 10' deep	LF	$	95	571	$	54,207
Reconnect Services	LF	$	10	850	$	8,499
48" Manhole - 8" thru 18" pipes	EA	$	4,250	7	$	29,750
Connect Pipes at Manhole - 8" thru 21"	EA	$	1,500	7	$	10,500
1/2 Lane Pavement Repair	LF	$	30	850	$	25,497
Bypass Pumping	EA	$	5,000	1	$	5,000
Traffic Control	LF	$	2	850	$	1,700
Mobilization	%	5%		-	$	7,748
Contingency	%	30%		-	$	48,813
Construction Subtotal (rounded)					**$**	**212,000**
Engineering and CMS	%	25%		-	$	53,000
Legal, Admin, and Permitting	%	2%		-	$	4,240
Total Project Cost (rounded)					**$**	**270,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Collection System Project: NW Smith Road

Project Identifier: 1B

Objective: The existing lines on NW Smith Road discharging into the Smith Road LS wet well are undersized for current flows.

Key Issues: 8" and smaller lines upstream of the Smith Road LS to the NW should be evaluated during predesign to determine if the lines require upsizing. The scope for this study did not include analysis of lines smaller than 10" in the system.

Project Location: NW Smith Road

Item	Unit	Unit Price	Quantity	Cost (2016)
12" Pipe - excavation, backfill, less than 10' deep	LF	$ 70	281	$ 19,635
18" Pipe - excavation, backfill, less than 10' deep	LF	$ 95	178	$ 16,910
Reconnect Services	LF	$ 10	459	$ 4,585
48" Manhole - 8" thru 18" pipes	EA	$ 4,250	4	$ 17,000
Connect Pipes at Manhole - 8" thru 21"	EA	$ 1,500	6	$ 9,000
1/2 Lane Pavement Repair	LF	$ 30	459	$ 13,755
Bypass Pumping	EA	$ 5,000	1	$ 5,000
Traffic Control	LF	$ 2	459	$ 917
Mobilization	%	5%	-	$ 4,340
Contingency	%	30%	-	$ 27,343
Construction Subtotal (rounded)				**$ 119,000**
Engineering and CMS	%	25%	-	$ 29,750
Legal, Admin, and Permitting	%	2%	-	$ 2,380
Total Project Cost (rounded)				**$ 160,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Collection System Project: Lift Station Improvements

Project Identifier: 1C

Objective: Address lift station deficiencies at Spring Lake lift station and Highway 30 lift station.

Key Issues: Spring Lake lift station needs testing for a potential air lock in the force main. If there is an air lock, an air release valve should be installed. Spring Lake lift station should then operate well and replacement of the pumps/lift station would be unneccessary. This would reduce the cost of improvements by approximately $140,000.

Project Location: Spring Lake LS and Highway 30 LS

Item	Cost (2016)	
Spring Lake Lift Station		
Test for air lock in force main	$	7,500
Replace both pumps/whole lift station	$	150,000
Provide VFDs to allow 3-phase pumps	$	20,000
Clean and coat piping in wet well and valves	$	8,500
Upgrade SCADA to include level readout and HMU at LS	$	9,500
Perform engineering review of electrical system	$	3,500
HWY 30 Lift Station		
Install bollards to protect station from traffic	$	3,500
Clean and coat piping in wet well and valves	$	8,500
Perform engineering review of electrical system	$	3,500
OHP (15%)	$	32,175
Contingency (30%)	$	74,003
Construction Subtotal (rounded)	**$**	**321,000**
Engineering and CMS (25%)	$	80,250
Legal, Administration & Permitting (2%)	$	6,420
Total Project Cost (rounded)	**$**	**410,000**

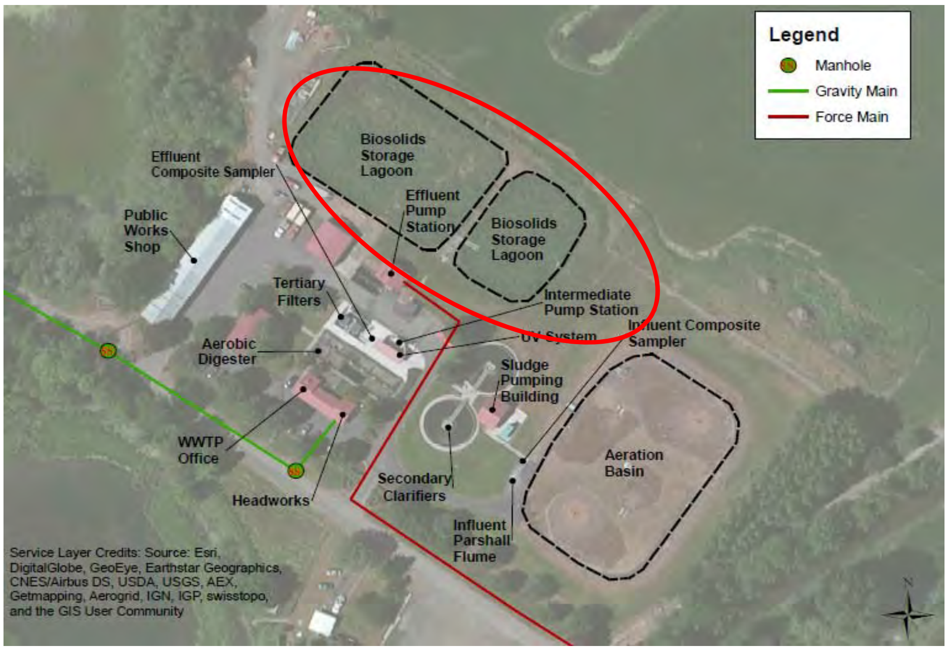
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: Interim Biosolids Plan

Project Identifier: 1a

Objective: Meet Class B requirements for current sludge flows, maximize existing resources, and position the City to implement future improvements. Install four (4) 7.5-horsepower and two (2) 15-horsepower surface aerators in the western Biosolids Storage Lagoon. Construct a new dewatering building. Convert the eastern Biosolids Storage Lagoon into a dewatered sludge storage area with a cover.

Project Location: Biosolids Storage Lagoons



Item	Cost (2016)	
Site Work	$	50,000
Dewatering Building	$	400,000
Equipment (screw press, polymer, pump, conveyor)	$	440,000
Piping/Valves and Instrumentation	$	50,000
Aerobic Digester Aerators	$	70,000
Concrete for Sludge Storage Area	$	50,000
Cover for Sludge Storage Area	$	100,000
Electrical/Controls	$	160,000
Mobilization (10%)	$	140,000
Overhead and Profit (15%)	$	200,000
Contingency (30%)	$	400,000
Construction Subtotal	**$**	**2,060,000**
Engineering & CMS (20%)	$	420,000
Legal, Administrative, and Permitting (2%)	$	50,000
Total Project Cost	**$**	**2,530,000**

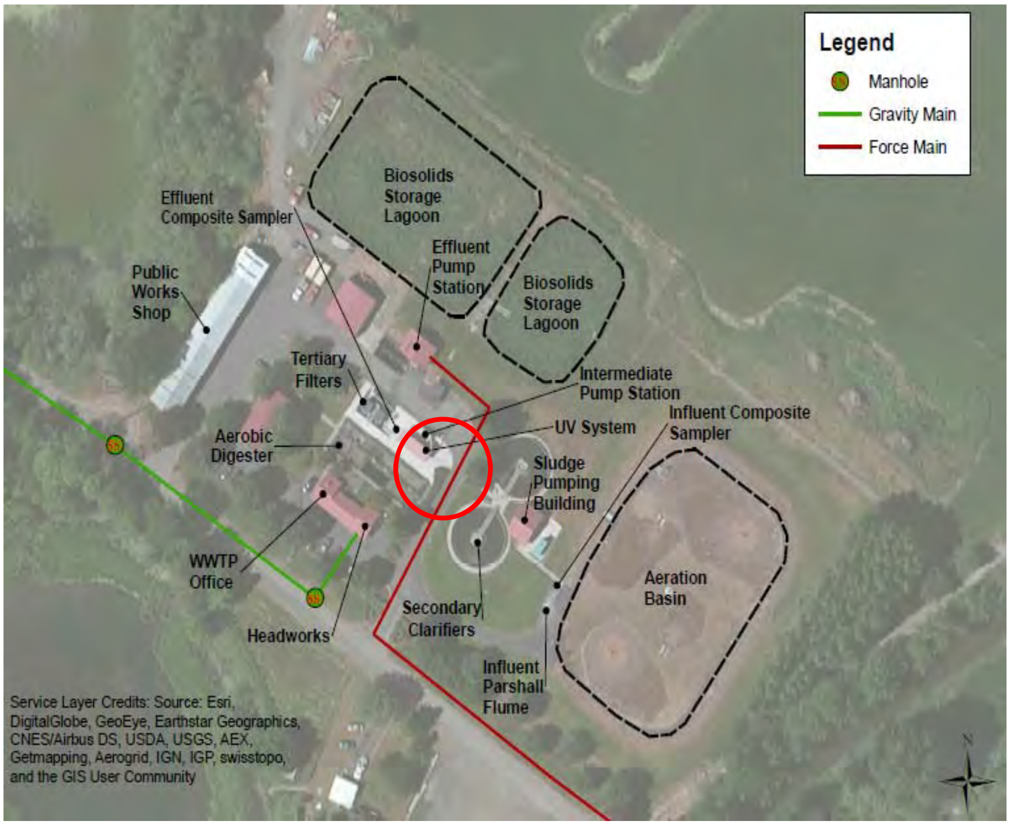
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: Rehabilitate UV System

Project Identifier: 1b

Objective: Rehabilitate aging UV System.

Project Location: UV System



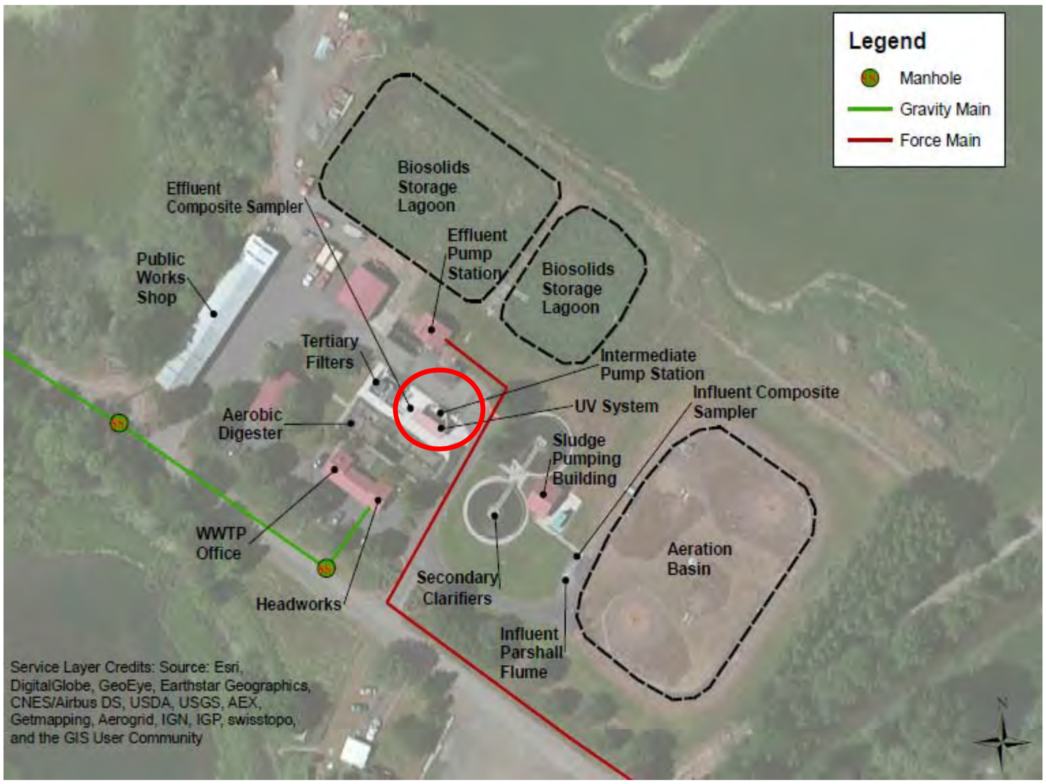
Item	Cost (2016)	
UV Equipment	$	170,000
Electrical/Controls	$	26,000
Mobilization (10%)	$	20,000
Overhead and Profit (15%)	$	30,000
Contingency (30%)	$	59,000
Construction Subtotal	**$**	**305,000**
Engineering & CMS (20%)	$	61,000
Legal, Administrative, and Permitting (2%)	$	7,000
Total Project Cost	**$**	**373,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: **Add 3rd pump to Inter. Pump Station**
Project Identifier: **1c.1**

Objective: Provide redundancy for the current peak hour flows by adding a third pump to the existing Intermediate Pump Station.

Project Location: Intermediate Pump Station



Item	Cost (2016)	
Add Third Pump	$	13,000
Electrical/Controls	$	3,000
Mobilization (10%)	$	2,000
Overhead and Profit (15%)	$	3,000
Contingency (30%)	$	5,000
Construction Subtotal	**$**	**26,000**
Engineering & CMS (30%)	$	8,000
Legal, Administrative, and Permitting (2%)	$	1,000
Total Project Cost	**$**	**35,000**

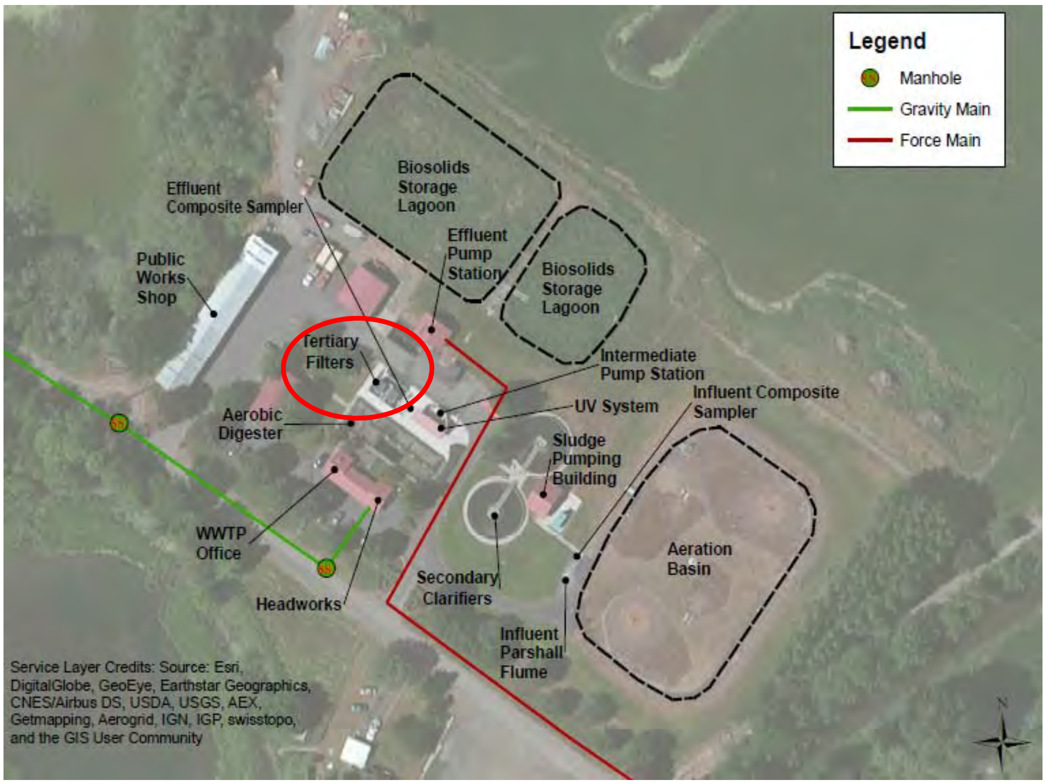
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: Add disks to existing Tertiary Filters

Project Identifier: 1c.2

Objective: Provide redundancy for the current maximum month flows by adding a disks to the existing Tertiary Filters.

Project Location: Tertiary Filters



Item		Cost (2016)
Add Disks to Tertiary Filters	$	40,000
Electrical/Controls	$	8,000
Mobilization (10%)	$	5,000
Overhead and Profit (15%)	$	8,000
Contingency (30%)	$	15,000
Construction Subtotal	**$**	**76,000**
Engineering & CMS (25%)	$	19,000
Legal, Administrative, and Permitting (2%)	$	2,000
Total Project Cost	**$**	**97,000**

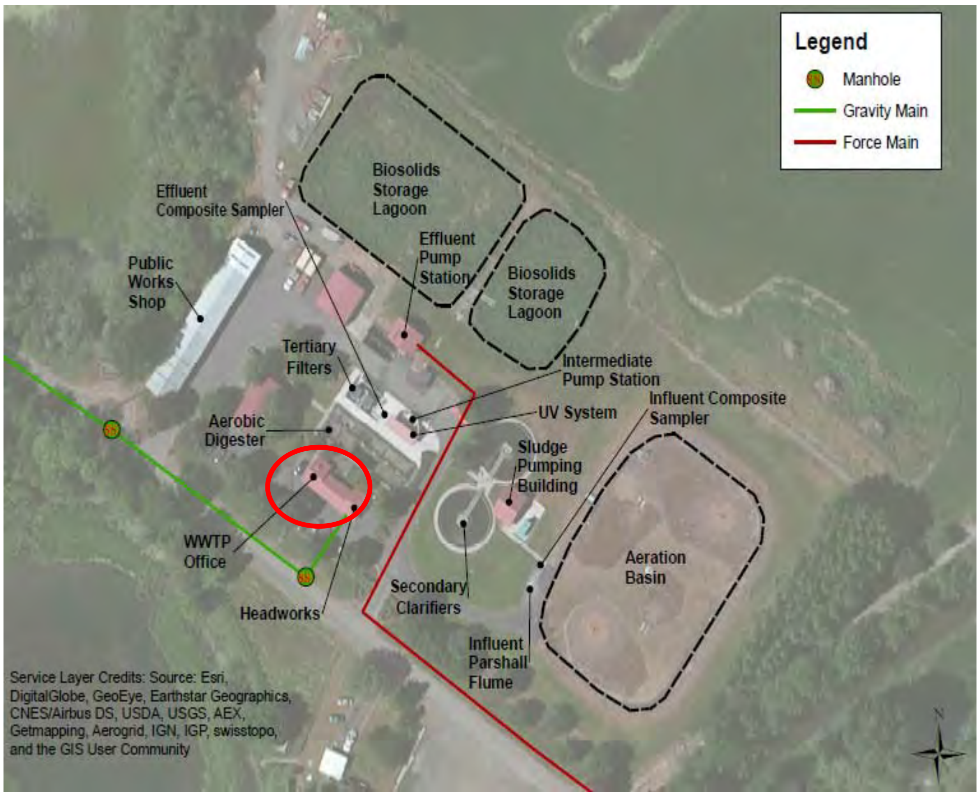
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: SCADA System

Project Identifier: 1d

Objective: A formal SCADA system should be added to provide process trending as well as alarms to the City staff. Additionally the existing control panels and new SCADA system should not be located in the lab area.

Project Location: WWTP Office



Item	Cost (2016)
SCADA System	$ 150,000
Mobilization (10%)	$ 15,000
Overhead and Profit (15%)	$ 23,000
Contingency (30%)	$ 45,000
Construction Subtotal	**$ 233,000**
Engineering & CMS (25%)	$ 59,000
Legal, Administrative, and Permitting (2%)	$ 5,000
Total Project Cost	**$ 297,000**

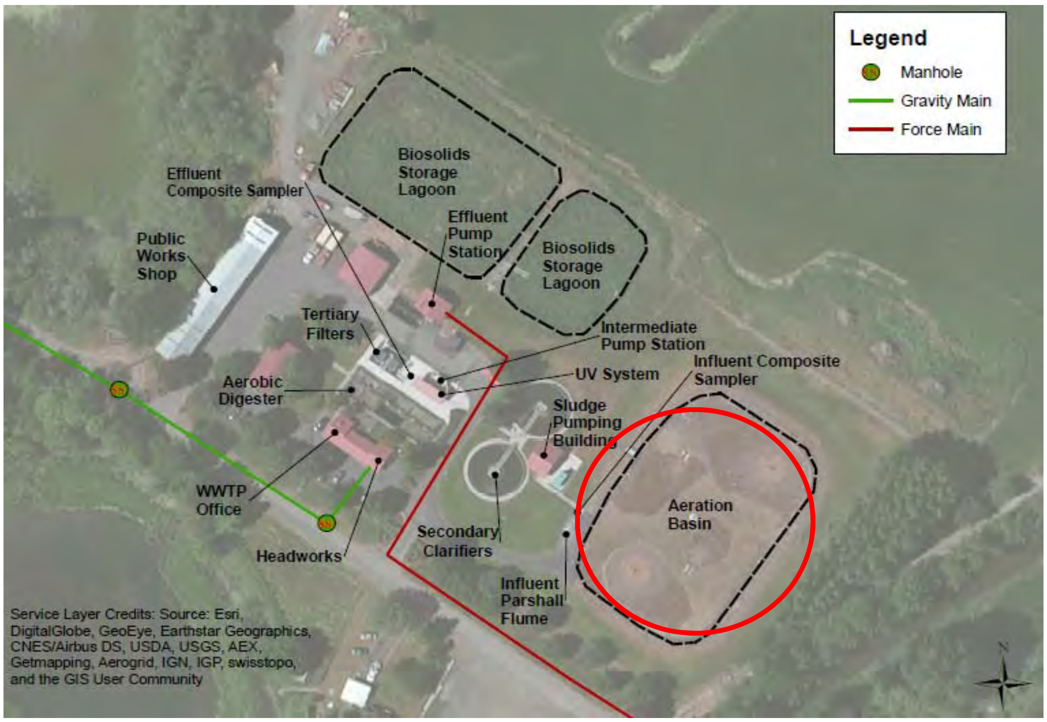
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: Aeration for Aeration Basin

Project Identifier: 1e.1

Objective: The aeration basin does not have adequate aeration capacity. Four (4) 40 HP surface aerators would be added to extend the capacity of the existing aeration basin.

Project Location: Aeration Basin



Item	Cost (2016)	
Aeration Basin Aerators	$	150,000
Electrical/Controls*	$	30,000
Mobilization (10%)	$	18,000
Overhead and Profit (15%)	$	27,000
Contingency (30%)	$	54,000
Construction Subtotal	**$**	**279,000**
Engineering & CMS (20%)	$	56,000
Legal, Administrative, and Permitting (2%)	$	6,000
Total Project Cost	**$**	**341,000**

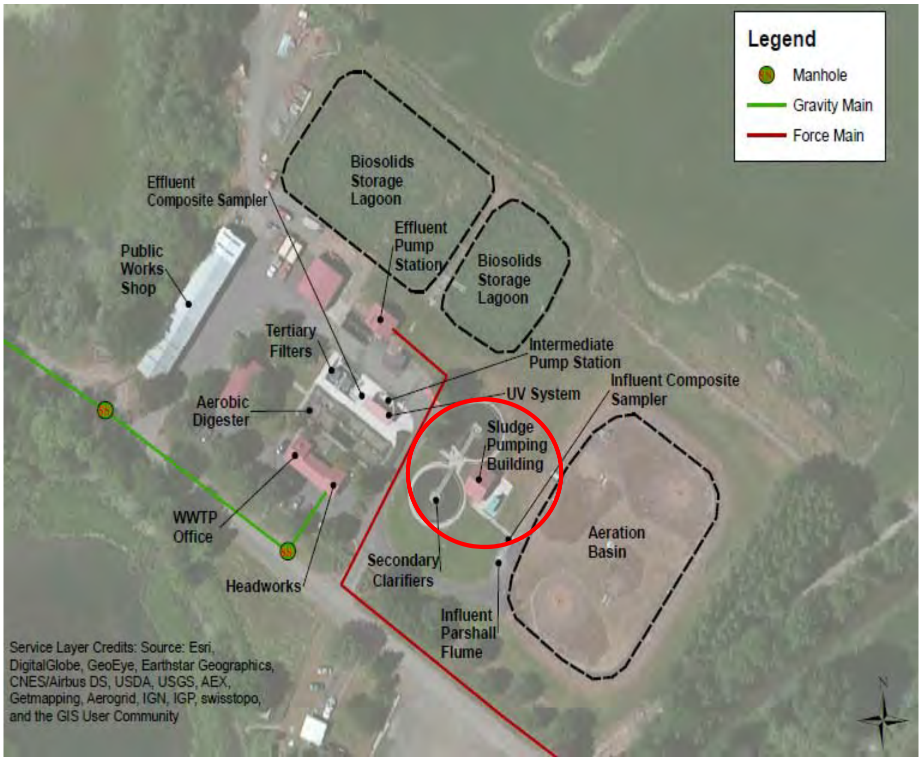
* Assumed to take place with Sludge Pumping Building Expansion to save on electrical/controls costs.

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 2016 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.

Wastewater Facilities Project: **Sec. Clarifier and Sludge Bldg. Exp.**
Project Identifier: **1e.2**

Objective: Add a secondary clarifier (50 ft. diameter). Also add a secondary effluent flow splitter box, upgrade the existing clarifier wiring, and expand the sludge pump building for the new RAS and WAS pumps.

Project Location: Secondary Clarifiers and Sludge Pumping Building



Item	Cost (2016)
Demolition and Site Work	$ 80,000
Effluent Splitter Box	$ 70,000
Secondary Clarifier	$ 420,000
Existing Clarifier Wiring	$ 20,000
Upgrade Sludge Pump Building (including pumps)	$ 360,000
Electrical/Controls	$ 190,000
Mobilization (10%)	$ 120,000
Overhead and Profit (15%)	$ 180,000
Contingency (30%)	$ 350,000
Construction Subtotal	**$ 1,790,000**
Engineering & CMS (20%)	$ 360,000
Legal, Administrative, and Permitting (2%)	$ 40,000
Total Project Cost	**$ 2,190,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Collection System Project: SE 6th Street Trunk Line

Project Identifier: **2A**

Objective: The existing 10" and 12" line from SE High School Way to SE Elm Street & SE 8th Street will be undersized for 2035 flows. The segment on SE High School Way needs to be upsized to 12" and the rest of the line needs to be upsized to 15" to handle future flows.

Project Location: SE 6th Street Trunk Line

Item	Unit	Unit Price	Quantity	Cost (2016)
12" Pipe - excavation, backfill, less than 10' deep	LF	$ 70	512	$ 35,854
15" Pipe - excavation, backfill, less than 10' deep	LF	$ 75	2091	$ 156,788
Reconnect Services	LF	$ 10	2603	$ 26,027
48" Manhole - 8" thru 18" pipes	EA	$ 4,250	8	$ 34,000
Connect Pipes at Manhole - 8" thru 21"	EA	$ 1,500	5	$ 7,500
1/2 Lane Pavement Repair	LF	$ 30	2603	$ 78,081
Bypass Pumping	EA	$ 5,000	1	$ 5,000
Traffic Control	LF	$ 2	2603	$ 5,205
Mobilization	%	5%	-	$ 17,423
Contingency	%	30%	-	$ 109,763
Construction Subtotal (rounded)				**$ 476,000**
Engineering and CMS	%	25%	-	$ 119,000
Legal, Admin, and Permitting	%	2%	-	$ 9,520
Total Project Cost (rounded)				**$ 610,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Collection System Project: NE Laurel Street and NE 3rd Street

Project Identifier: **2B**

Objective: The existing 15" line on NE Laurel Street and NE 3rd Street will be undersized to meet 2035 flows. The line should be re-graded with a uniform slope. The replacement pipe should be 18".

Project Location: NE Laurel Street and NE 3rd Street

Item	Unit	Unit Price	Quantity	Cost (2016)
18" Pipe - excavation, backfill, less than 10' deep	LF	$ 95	1302	$ 123,709
Reconnect Services	LF	$ 10	1302	$ 13,022
48" Manhole - 8" thru 18" pipes	EA	$ 4,250	6	$ 25,500
Connect Pipes at Manhole - 8" thru 21"	EA	$ 1,500	2	$ 3,000
1/2 Lane Pavement Repair	LF	$ 30	1302	$ 39,066
Bypass Pumping	EA	$ 5,000	1	$ 5,000
Traffic Control	LF	$ 2	1302	$ 2,604
Mobilization	%	5%	-	$ 10,595
Contingency	%	30%	-	$ 66,749
Construction Subtotal (rounded)				**$ 290,000**
Engineering and CMS	%	25%	-	$ 72,500
Legal, Admin, and Permitting	%	2%	-	$ 5,800
Total Project Cost (rounded)				**$ 370,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Collection System Project: Lift Station Improvements

Project Identifier: 2C

Objective: Address lift station deficiencies at Smith Road lift station, Keys Landing lift station, and Seven Oaks lift station.

Project Location: Smith Road, Keys Landing and Seven Oaks Lift Stations

Item	Cost (2016)	
Smith Road LS		
Clean and coat access hatch for corrosion control	$	4,500
Install screen ahead of pumps	$	39,000
Replace drywell blower	$	7,500
Replace bypass cap	$	2,500
Perform engineering review of electrical system	$	3,500
Keys Landing LS		
Address odor control so overflow can be opened	$	7,200
Clean and coat piping in wet well and valves	$	8,500
Install standby power from the WTP to the LS	$	24,000
Repair clamshell door so it remains open without support	$	3,200
Perform engineering review of electrical system	$	3,500
Seven Oaks LS		
Repair bottom clam shell insulation	$	6,800
Clean and coat piping in wet well and valves	$	8,500
Perform engineering review of electrical system	$	3,500
OHP (15%)	$	18,330
Contingency (30%)	$	42,159
Construction Subtotal (rounded)	**$**	**183,000**
Engineering and CMS (25%)	$	45,750
Legal, Administration & Permitting (2%)	$	3,660
Total Project Cost (rounded)	**$**	**240,000**

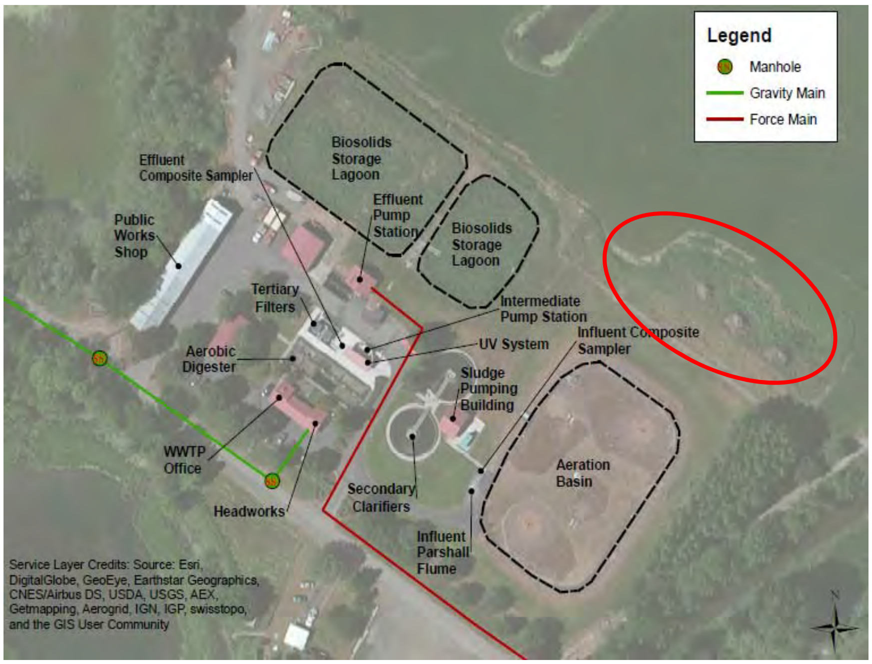
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: **New Aeration Basins**

Project Identifier: **2a.1**

Objective: Replace the aeration basin with two (2) new aeration basins (~240 ft. x 40 ft.). The new aeration basins should include diffused aeration, divider walls, submersible mixers, and a mixed liquor recycle pump to create an A2O or MLE process. This project would also include a blower building and influent splitter box.

Project Location: New Aeration Basins



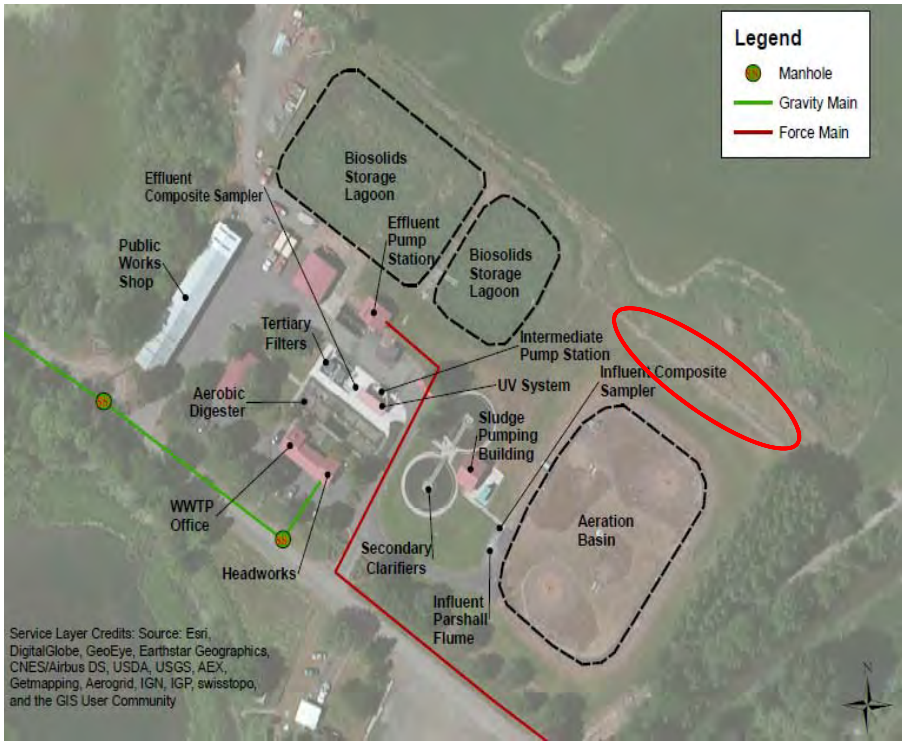
Item	Cost (2016)	
Site Work	$	160,000
Influent Splitter Box	$	70,000
New Aeration Basins	$	1,300,000
Mixers/Pumps	$	140,000
Fine Bubble Diffusers	$	70,000
Blowers	$	600,000
Misc. Metals (guardrail, grating)	$	240,000
Piping/Valves and Instrumentation	$	400,000
Blower Building	$	380,000
Electrical/Controls	$	730,000
Mobilization (10%)	$	410,000
Overhead and Profit (15%)	$	620,000
Contingency (30%)	$	1,230,000
Construction Subtotal	**$**	**6,350,000**
Engineering & CMS (20%)	$	1,270,000
Legal, Administrative, and Permitting (2%)	$	130,000
Total Project Cost	**$**	**7,750,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: New Aerobic Digester
Project Identifier: 2a.2

Objective: New aerobic digesters should be added to achieve Class B biosolids (60 day SRT in the winter). The digester blowers should be located in the blower building. Sludge pumps and a sludge flow meter should be located in a dewatering building and are a part of that project.

Project Location: New Aerobic Digester



Item	Cost (2016)	
Site Work	$	90,000
Digester Basin	$	400,000
Misc. Metals (guardrail, grating)	$	100,000
Piping/Valves and Instrumentation	$	60,000
Digester Equipment (including Blowers)	$	270,000
Electrical/Controls	$	140,000
Mobilization (10%)	$	110,000
Overhead and Profit (15%)	$	160,000
Contingency (30%)	$	320,000
Construction Subtotal	**$**	**1,650,000**
Engineering & CMS (20%)	$	330,000
Legal, Administrative, and Permitting (2%)	$	40,000
Total Project Cost	**$**	**2,020,000**

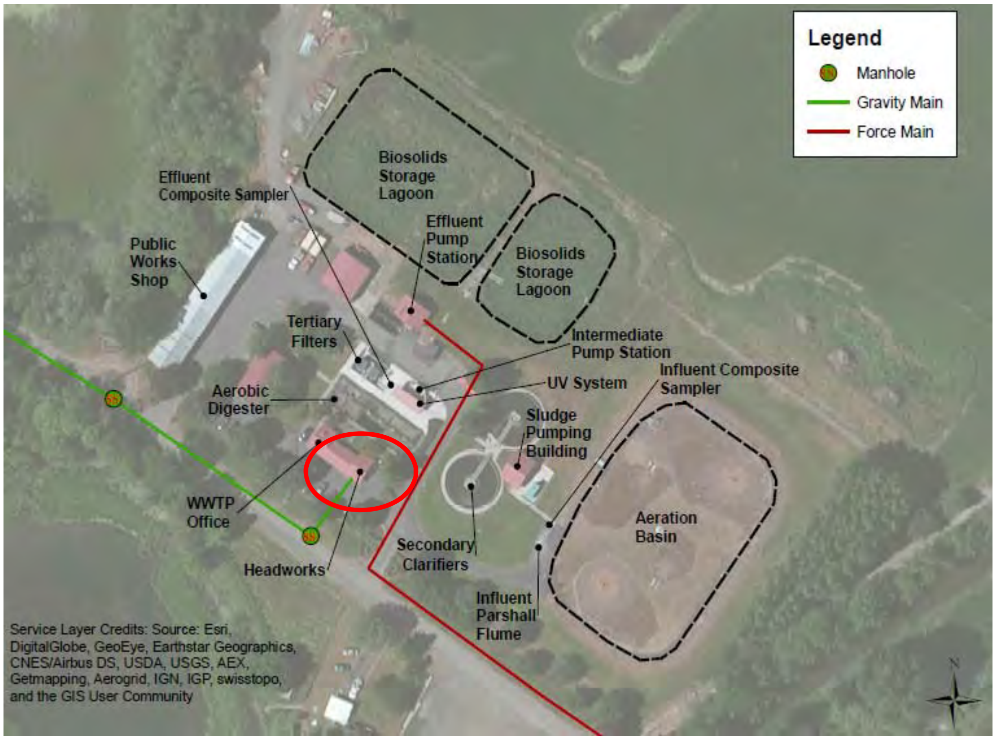
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: **Expand Headworks**

Project Identifier: **2b.1**

Objective: Upgrade the headworks to include influent screens with sufficient capacity for the 2035 peak instantaneous flow. Also add grit removal, freeze protection, and expand the cover over the screens.

Project Location: Headworks



Item	Cost (2016)	
Demolition and Heat Tape	$	70,000
Expand Cover and Channels	$	60,000
New Influent Screens and Channels	$	470,000
Grit Chambers and Classifiers	$	900,000
Electrical/Controls	$	300,000
Mobilization (10%)	$	180,000
Overhead and Profit (15%)	$	270,000
Contingency (30%)	$	540,000
Construction Subtotal	**$**	**2,790,000**
Engineering & CMS (20%)	$	560,000
Legal, Administrative, and Permitting (2%)	$	60,000
Total Project Cost	**$**	**3,410,000**

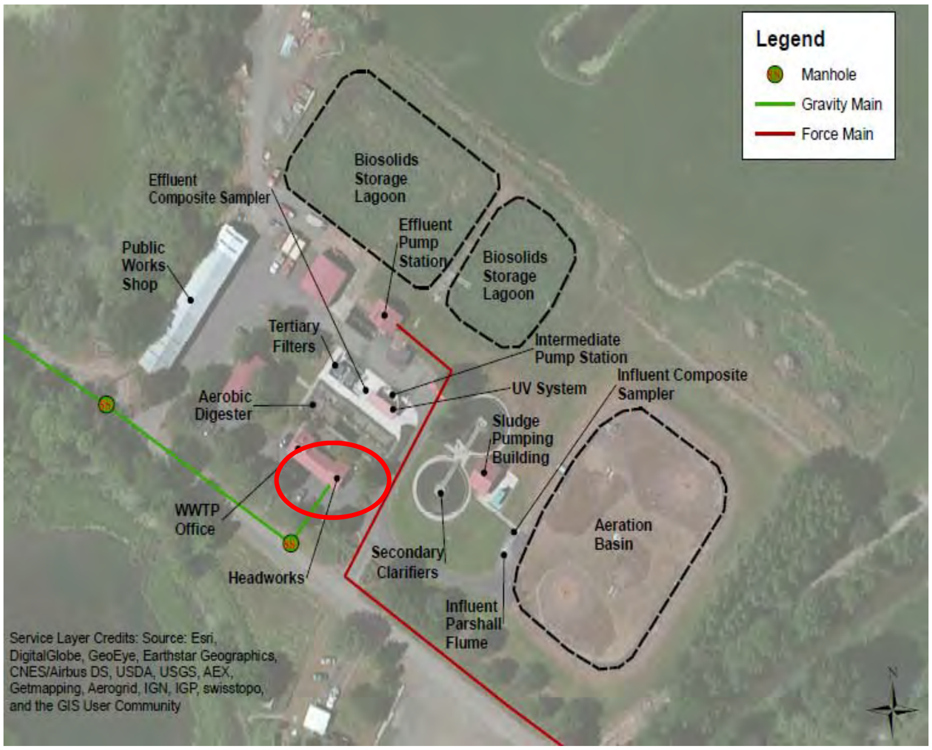
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: Upgrade Influent Pumps

Project Identifier: 2b.2

Objective: Upgrade the headworks to include influent flow measurement and influent pumps with sufficient capacity for the 2035 peak instantaneous flow. Also add accurate level measurement, a redundant sump pump, upgraded HVAC, and enhanced influent pump controls.

Project Location: Headworks



Item	Cost (2016)	
New Influent Pumps	$	210,000
Piping/Valves and Instrumentation	$	30,000
HVAC	$	90,000
Electrical/Controls	$	160,000
Mobilization (10%)	$	49,000
Overhead and Profit (15%)	$	74,000
Contingency (30%)	$	147,000
Construction Subtotal	**$**	**760,000**
Engineering & CMS (20%)	$	152,000
Legal, Administrative, and Permitting (2%)	$	16,000
Total Project Cost	**$**	**928,000**

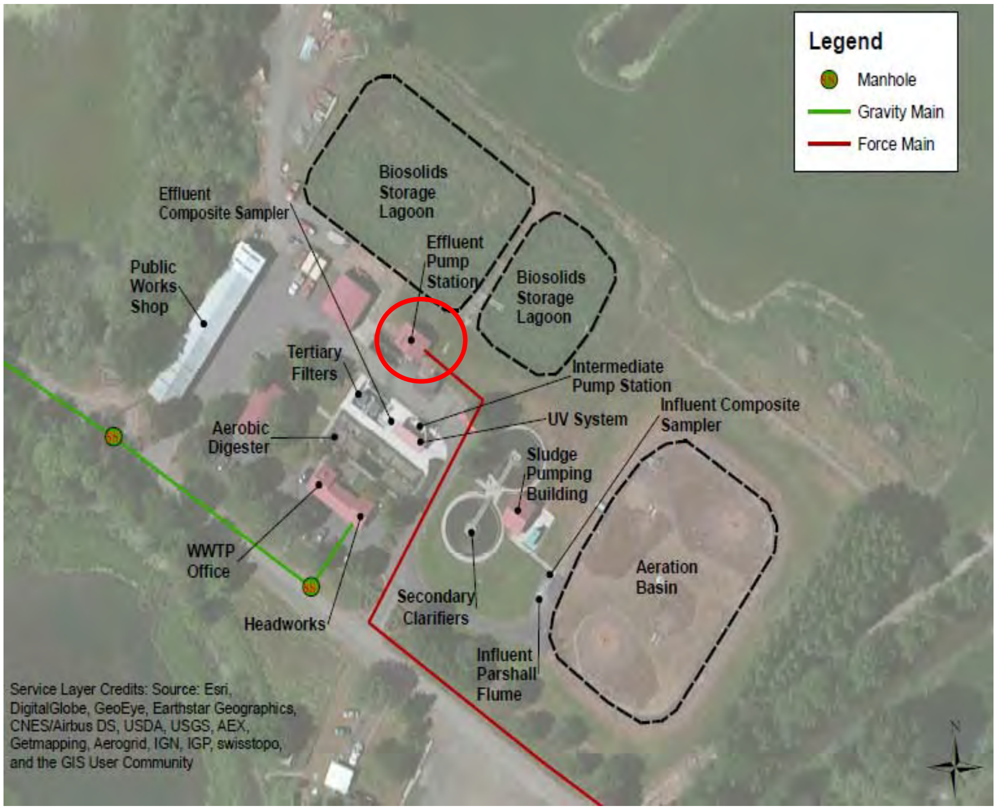
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: Upgrade Effluent Pumps

Project Identifier: 2c.1

Objective: Upgrade the Effluent Pump Station to handle the future 2035 peak instantaneous flows.

Project Location: Effluent Pump Station



Item		Cost (2016)
New Effluent Pumps	$	330,000
Piping/Valves and Instrumentation	$	30,000
Electrical/Controls	$	80,000
Mobilization (10%)	$	44,000
Overhead and Profit (15%)	$	66,000
Contingency (30%)	$	132,000
Construction Subtotal	**$**	**682,000**
Engineering & CMS (20%)	$	137,000
Legal, Administrative, and Permitting (2%)	$	14,000
Total Project Cost	**$**	**833,000**

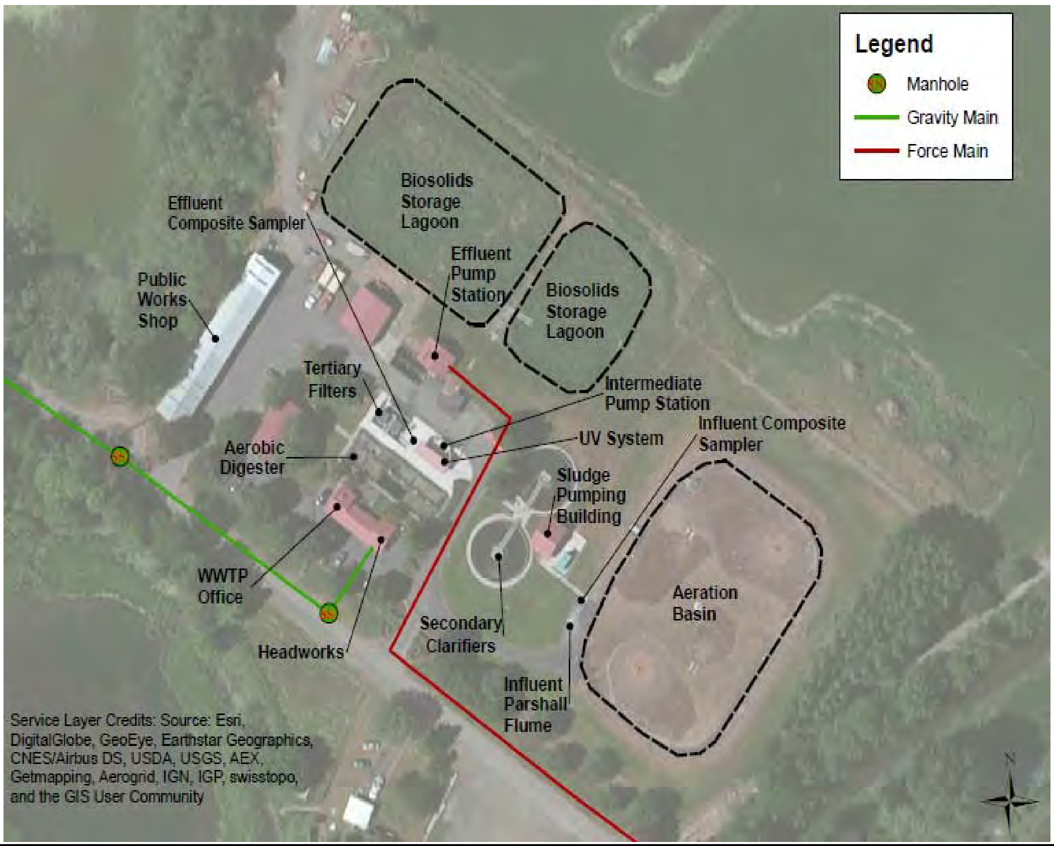
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: Increase Effluent Pipe

Project Identifier: 2c.2

Objective: Upgrade the Effluent Pipeline to handle the future 2035 peak instantaneous flows.

Project Location: Effluent Pipeline



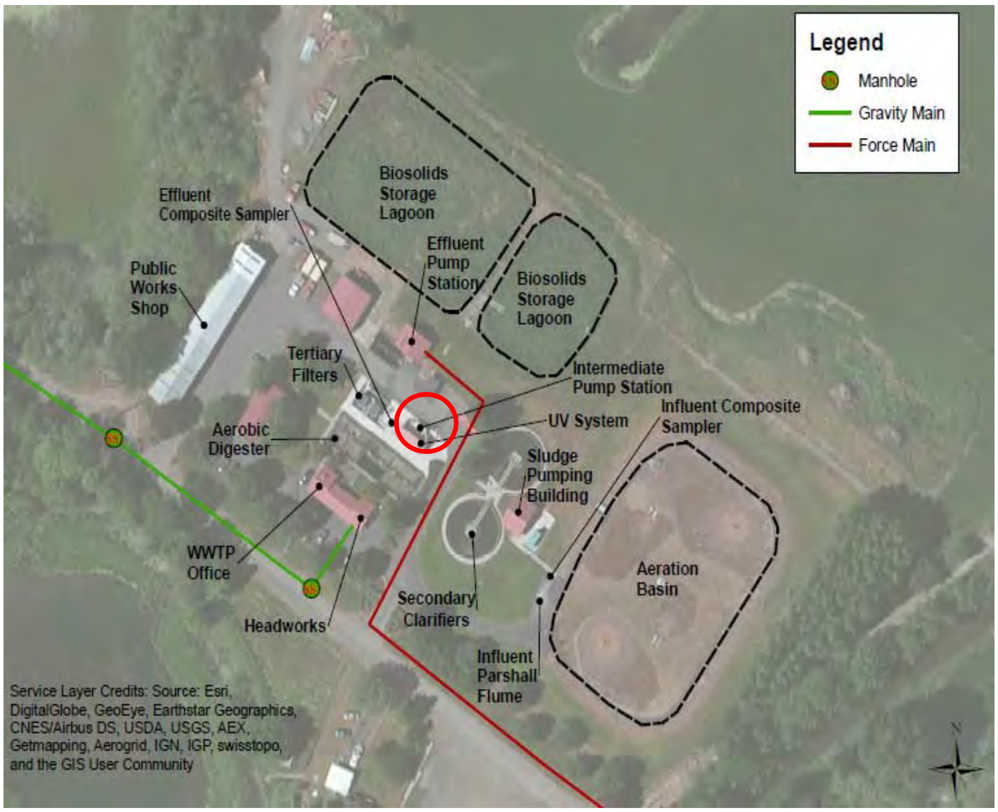
Item	Cost (2016)	
Replace Effluent Pipeline	$	1,100,000
Mobilization (10%)	$	110,000
Overhead and Profit (15%)	$	170,000
Contingency (30%)	$	330,000
Construction Subtotal	**$**	**1,710,000**
Engineering & CMS (20%)	$	350,000
Legal, Administrative, and Permitting (2%)	$	40,000
Total Project Cost	**$**	**2,100,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: Upgrade Intermediate Pump Station
Project Identifier: 2d.1

Objective: Upgrade the pumps in the Intermediate Pump Station to handle the future 2035 peak instantaneous flow.

Project Location: Intermediate Pump Station



Item	Cost (2016)	
Upgrade Intermediate Pump Station	$	200,000
Electrical/Controls	$	40,000
Mobilization (10%)	$	24,000
Overhead and Profit (15%)	$	36,000
Contingency (30%)	$	72,000
Construction Subtotal	**$**	**372,000**
Engineering & CMS (20%)	$	75,000
Legal, Administrative, and Permitting (2%)	$	8,000
Total Project Cost	**$**	**455,000**

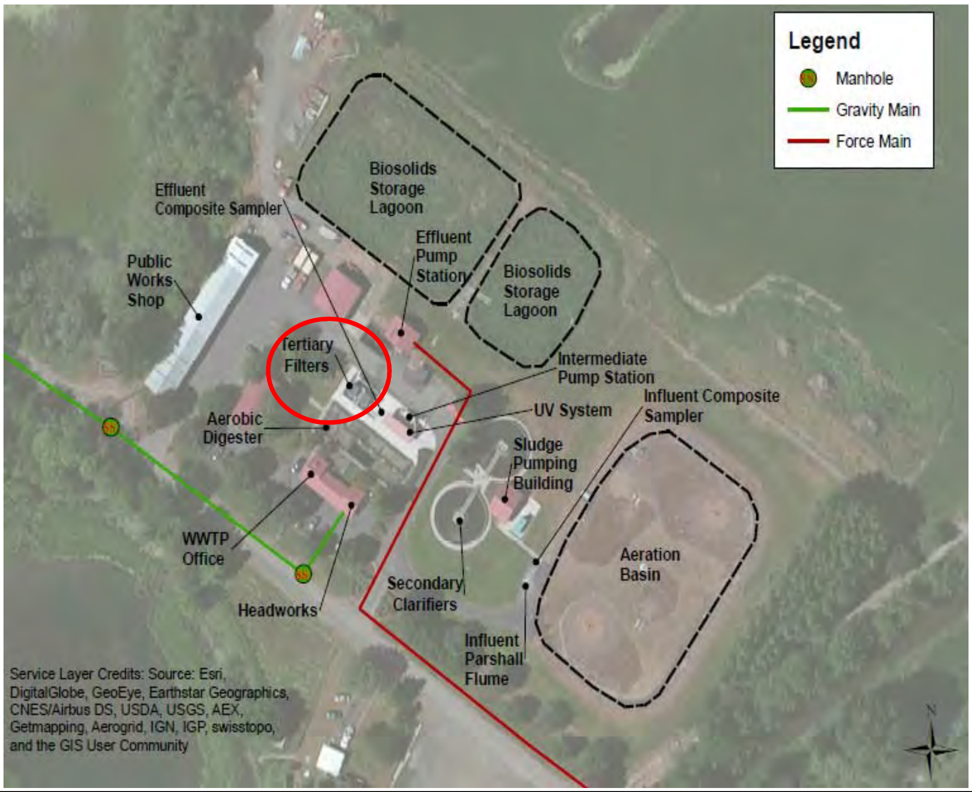
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: Additional Tertiary Filter Unit

Project Identifier: **2d.2**

Objective: Add a third filter unit to allow the Tertiary Filters to handle the future 2035 maximum month flow with redundancy.

Project Location: Tertiary Filters



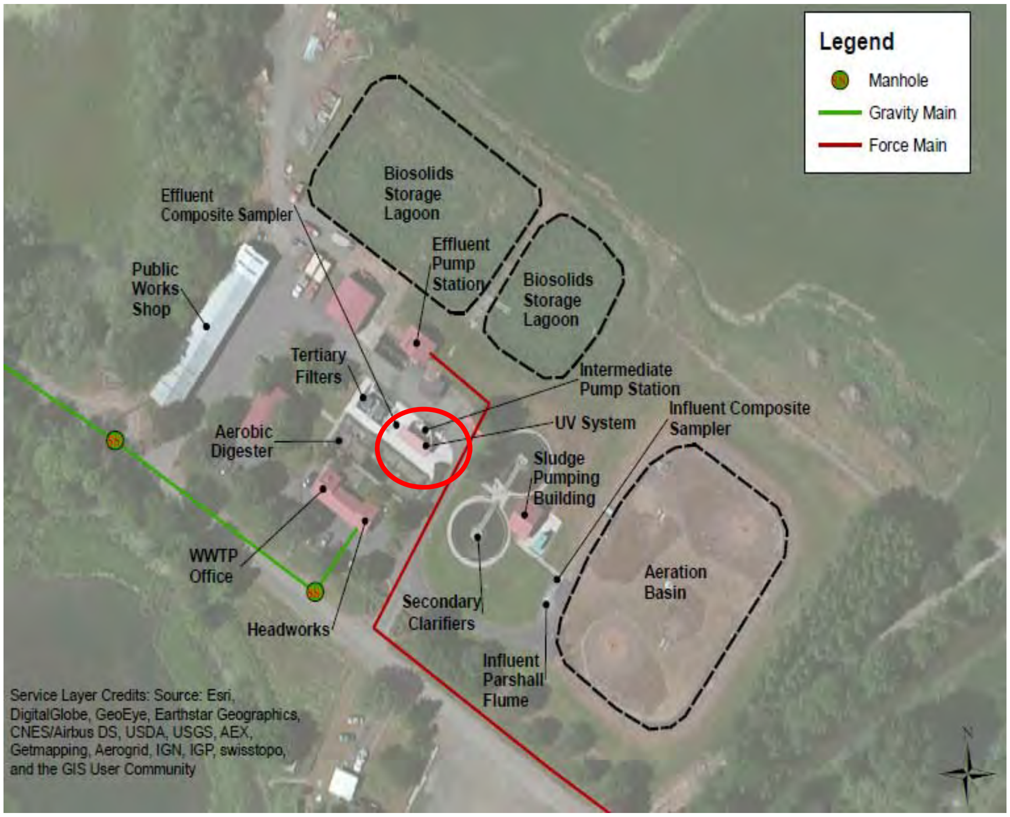
Item	Cost (2016)	
Site Work	$	50,000
Tertiary Filter	$	310,000
Cover	$	25,000
Electrical/Controls	$	77,000
Mobilization (10%)	$	47,000
Overhead and Profit (15%)	$	70,000
Contingency (30%)	$	139,000
Construction Subtotal	**$**	**718,000**
Engineering & CMS (20%)	$	144,000
Legal, Administrative, and Permitting (2%)	$	15,000
Total Project Cost	**$**	**877,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: Upgrade UV System
Project Identifier: 2e

Objective: Upgrade and expand the UV System to handle the future 2035 peak instantaneous flows.

Project Location: UV System



Item	Cost (2016)	
Expand Channels	$	50,000
UV Equipment	$	460,000
Electrical/Controls	$	80,000
Mobilization (10%)	$	59,000
Overhead and Profit (15%)	$	89,000
Contingency (30%)	$	177,000
Construction Subtotal	**$**	**915,000**
Engineering & CMS (20%)	$	183,000
Legal, Administrative, and Permitting (2%)	$	19,000
Total Project Cost	**$**	**1,117,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Collection System Project: SW Old Portland Road

Project Identifier: 3A

Objective: A few sections along SW Old Portland Road will just pass capacity at 2035 flows. CCTV inspections show the line along SW Old Portland Road has numerous sags and a variety of other condition concerns. This pipeline should be replaced with a 10" line for future flows and condition issues. The highlighted segments below reach capacity, but the entire line along SW Old Portland Road should replaced if necessary based on a condition assessment.

Project Location: SW Old Portland Road

Item	Unit	Unit Price	Quantity	Cost (2016)
10" Pipe - excavation, backfill, less than 10' deep	LF	$ 65	1042	$ 67,717
Reconnect Services	LF	$ 10	1042	$ 10,418
48" Manhole - 8" thru 18" pipes	EA	$ 4,250	8	$ 34,000
Connect Pipes at Manhole - 8" thru 21"	EA	$ 1,500	6	$ 9,000
1/2 Lane Pavement Repair	LF	$ 30	1042	$ 31,254
Bypass Pumping	EA	$ 5,000	1	$ 5,000
Traffic Control	LF	$ 2	1042	$ 2,084
Mobilization	%	5%	-	$ 7,974
Contingency	%	30%	-	$ 50,234
Construction Subtotal (rounded)				**$ 218,000**
Engineering and CMS	%	25%	-	$ 54,500
Legal, Admin, and Permitting	%	2%	-	$ 4,360
Total Project Cost (rounded)				**$ 280,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Collection System Project: SE Tussing Way Segment

Project Identifier: **3B**

Objective: The existing 12" line on SE Tussing Way reaches capacity at 2035 peak flows. The line could be upsized to 15" to alleviate future surcharging. The line should be monitored for surcharging issues before making the capital investment.

Project Location: SE Tussing Way Segment

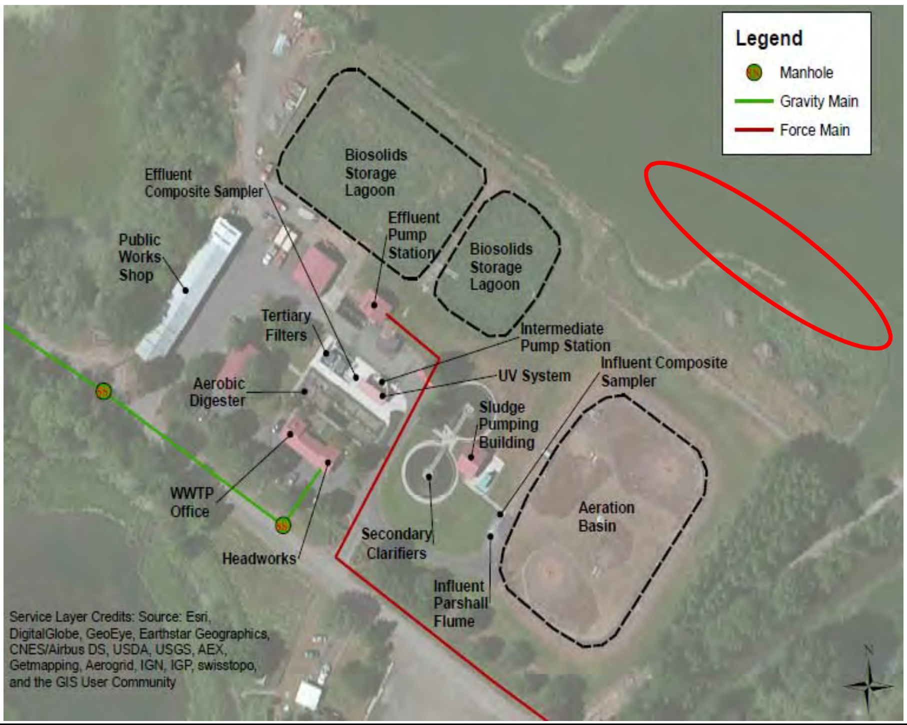
Item	Unit	Unit Price	Quantity	Cost (2016)
15" Pipe - excavation, backfill, less than 10' deep	LF	$ 75	111	$ 8,340
Reconnect Services	LF	$ 10	111	$ 1,112
48" Manhole - 8" thru 18" pipes	EA	$ 4,250	1	$ 4,250
Connect Pipes at Manhole - 8" thru 21"	EA	$ 1,500	2	$ 3,000
1/2 Lane Pavement Repair	LF	$ 30	111	$ 3,336
Bypass Pumping	EA	$ 5,000	1	$ 5,000
Traffic Control	LF	$ 2	111	$ 222
Mobilization	%	5%	-	$ 1,263
Contingency	%	30%	-	$ 7,957
Construction Subtotal (rounded)				**$ 35,000**
Engineering and CMS	%	25%	-	$ 8,750
Legal, Admin, and Permitting	%	2%	-	$ 700
Total Project Cost (rounded)				**$ 50,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: **Additional Aeration Basin**
Project Identifier: **3a.1**

Objective: An additional aeration basin should be added when the maximum month flow is approximately 2.6 MGD. Space for the new blowers should be provided in the blower building.

Project Location: New Aeration Basin



Item	Cost (2016)	
Site Work	$	80,000
New Aeration Basin	$	620,000
Mixers/Pumps	$	70,000
Fine Bubble Diffusers	$	40,000
Blowers	$	300,000
Misc. Metals (guardrail, grating)	$	120,000
Piping/Valves and Instrumentation	$	200,000
Electrical/Controls	$	260,000
Mobilization (10%)	$	170,000
Overhead and Profit (15%)	$	260,000
Contingency (30%)	$	510,000
Construction Subtotal	**$**	**2,630,000**
Engineering & CMS (20%)	$	530,000
Legal, Administrative, and Permitting (2%)	$	60,000
Total Project Cost	**$**	**3,220,000**

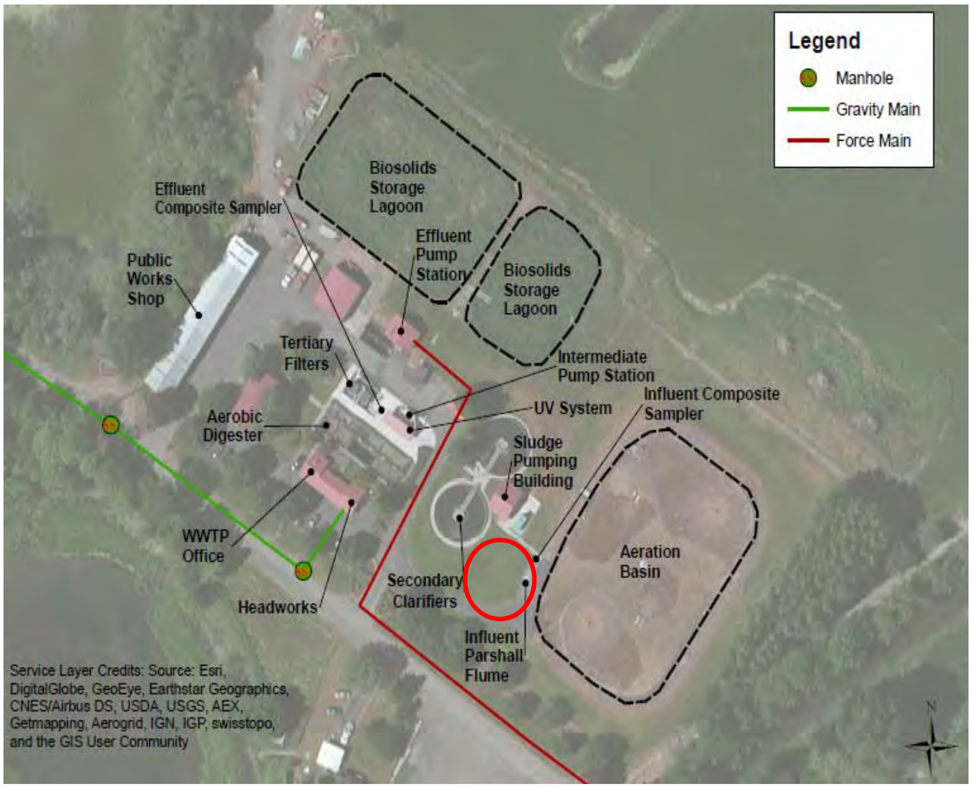
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: Additional Secondary Clarifier

Project Identifier: 3a.2

Objective: An additional 50 ft. diameter clarifier should be added when the maximum month flow is approximately 3.4 MGD.

Project Location: Secondary Clarifiers



Item	Cost (2016)
Site Work	$ 60,000
Secondary Clarifier	$ 420,000
Additional Sludge Pumps	$ 80,000
Electrical/Controls	$ 120,000
Mobilization (10%)	$ 70,000
Overhead and Profit (15%)	$ 110,000
Contingency (30%)	$ 210,000
Construction Subtotal	**$ 1,070,000**
Engineering & CMS (20%)	$ 220,000
Legal, Administrative, and Permitting (2%)	$ 30,000
Total Project Cost	**$ 1,320,000**

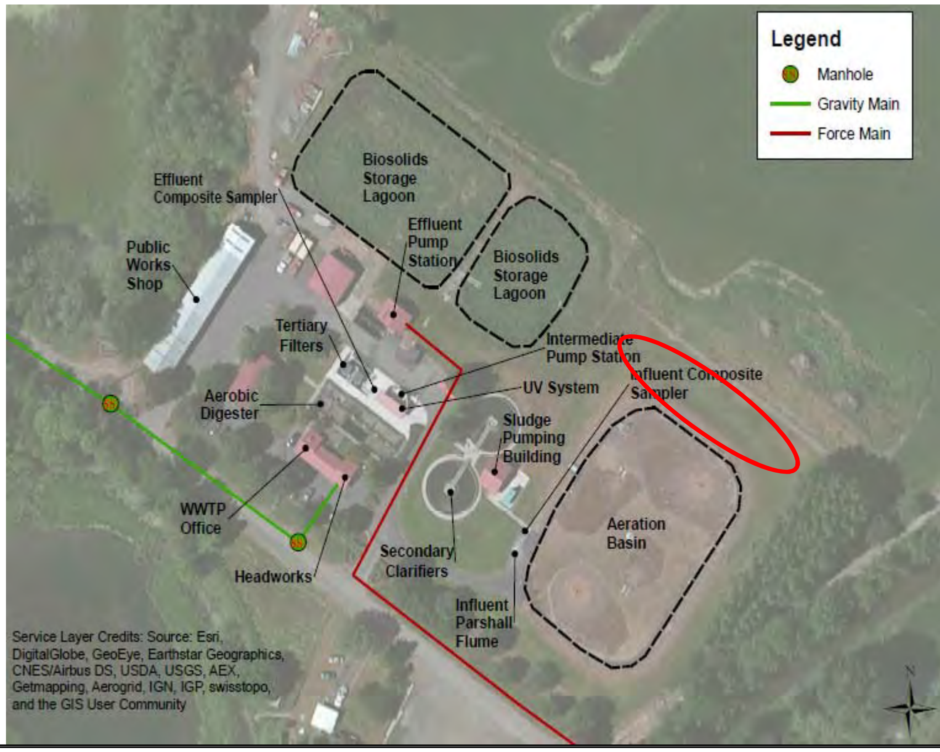
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: Additional Aerobic Digester

Project Identifier: 3b.1

Objective: An additional aerobic digester should be added when the maximum month flow is approximately 2.3 MGD.

Project Location: New Aerobic Digester



Item	Cost (2016)	
Site Work	$	90,000
Digester Basin	$	360,000
Misc. Metals (guardrail, grating)	$	100,000
Piping/Valves and Instrumentation	$	60,000
Digester Equipment (including Blowers)	$	220,000
Electrical/Controls	$	130,000
Mobilization (10%)	$	100,000
Overhead and Profit (15%)	$	150,000
Contingency (30%)	$	290,000
Construction Subtotal	**$**	**1,500,000**
Engineering & CMS (20%)	$	300,000
Legal, Administrative, and Permitting (2%)	$	30,000
Total Project Cost	**$**	**1,830,000**

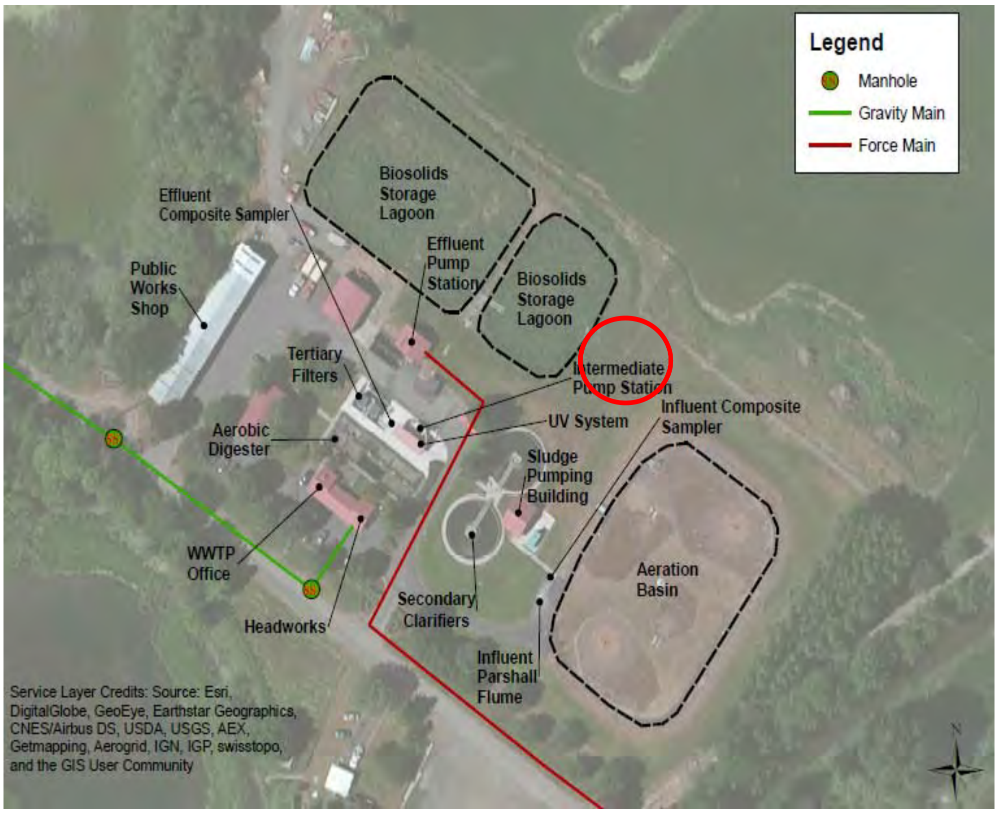
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: Additional Screw Presses

Project Identifier: 3b.2

Objective: Additional screw presses (2) should be added when the maximum month flow is approximately 2.3 MGD to provide capacity and redundancy.

Project Location: Dewatering Building



Item		Cost (2016)
Equipment (screw presses, polymer, pump, conveyor)	$	770,000
Electrical/Controls	$	120,000
Mobilization (10%)	$	89,000
Overhead and Profit (15%)	$	134,000
Contingency (30%)	$	267,000
Construction Subtotal	**$**	**1,380,000**
Engineering & CMS (20%)	$	276,000
Legal, Administrative, and Permitting (2%)	$	28,000
Total Project Cost	**$**	**1,684,000**

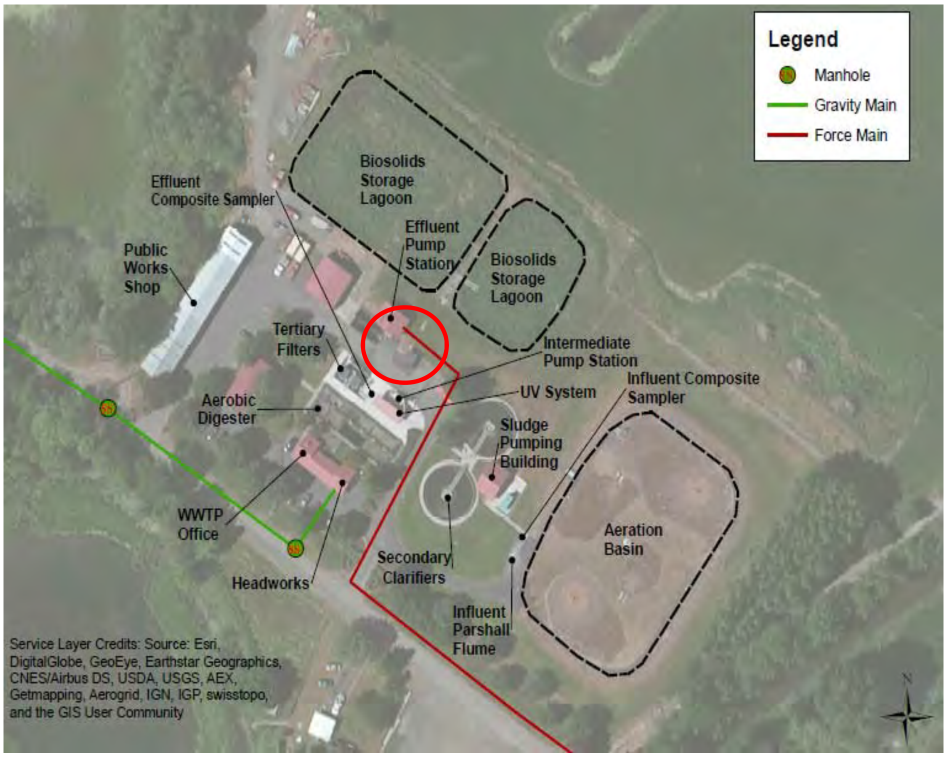
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Wastewater Facilities Project: Plant Water System

Project Identifier: 3c

Objective: Currently the plant uses groundwater for seal water and washdown activities. By installing a chlorination system and pumps, the City could use treated, non-potable, wastewater from the chlorine contact chamber for plant water.

Project Location: Effluent Pump Station



Item	Cost (2016)	
Plant Water Pumps	$	40,000
Chlorination System	$	45,000
Piping and Valves	$	8,000
Electrical/Controls	$	11,000
Mobilization (10%)	$	11,000
Overhead and Profit (15%)	$	16,000
Contingency (30%)	$	32,000
Construction Subtotal	**$**	**163,000**
Engineering & CMS (25%)	$	41,000
Legal, Administrative, and Permitting (2%)	$	4,000
Total Project Cost	**$**	**208,000**

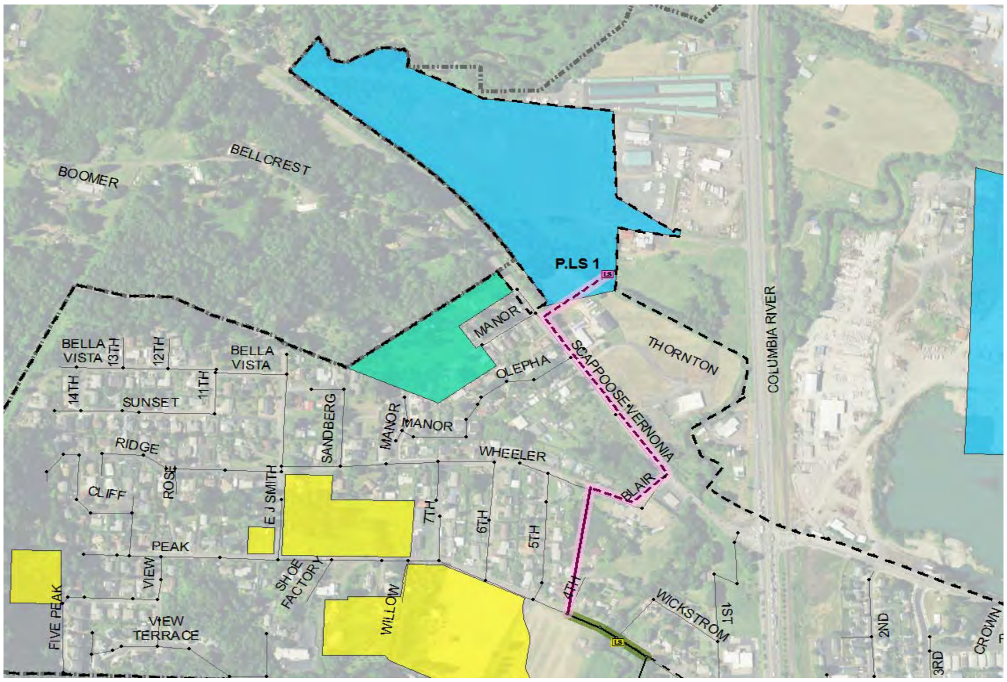
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Collection System Project: Proposed Lift Station 1

Project Identifier: 4A

Objective: New lift station, force main and gravity main to service City projected future industrial development north of Smith Road lift station.

Project Location: Scappoose-Vernonia Highway



Item	Unit	Unit Price	Quantity	Cost (2016)
Lift Station (peak flow of 90 gpm)	LF	$ 150,000	1	$ 150,000
6" Force Main	LF	$ 70	1500	$ 105,000
12" Gravity Main	EA	$ 70	1600	$ 112,000
Bypass Pumping	EA	$ 5,000	1	$ 5,000
Traffic Control	LF	$ 2	3100	$ 6,200
Mobilization	%	5%	-	$ 18,910
Contingency	%	30%	-	$ 119,133
Construction Subtotal (rounded)				**$ 517,000**
Engineering and CMS	%	25%	-	$ 129,250
Legal, Admin, and Permitting	%	2%	-	$ 10,340
Total Project Cost (rounded)				**$ 660,000**

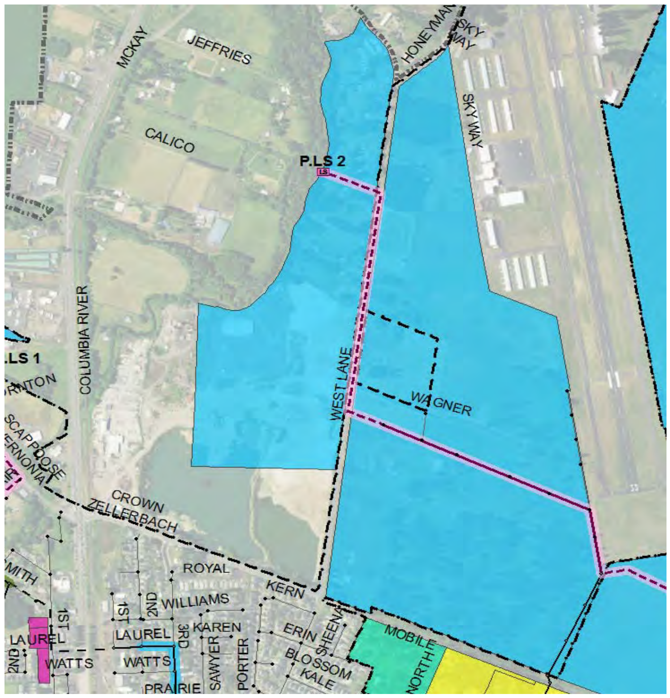
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Collection System Project: Proposed Lift Station 2

Project Identifier: **4B**

Objective: New lift station, force main and gravity main to service City projected future industrial development off West Lane Road.

Project Location: West Lane Road



Item	Unit	Unit Price	Quantity	Cost (2016)
Lift Station (peak flow of 600 gpm)	LF	$ 263,000	1	$ 263,000
8" Force Main	LF	$ 75	2700	$ 202,500
12" Gravity Main	EA	$ 70	2600	$ 182,000
Bypass Pumping	EA	$ 5,000	1	$ 5,000
Traffic Control	LF	$ 2	5300	$ 10,600
Mobilization	%	5%	-	$ 33,155
Contingency	%	30%	-	$ 208,877
Construction Subtotal (rounded)				**$ 906,000**
Engineering and CMS	%	25%	-	$ 226,500
Legal, Admin, and Permitting	%	2%	-	$ 18,120
Total Project Cost (rounded)				**$ 1,160,000**

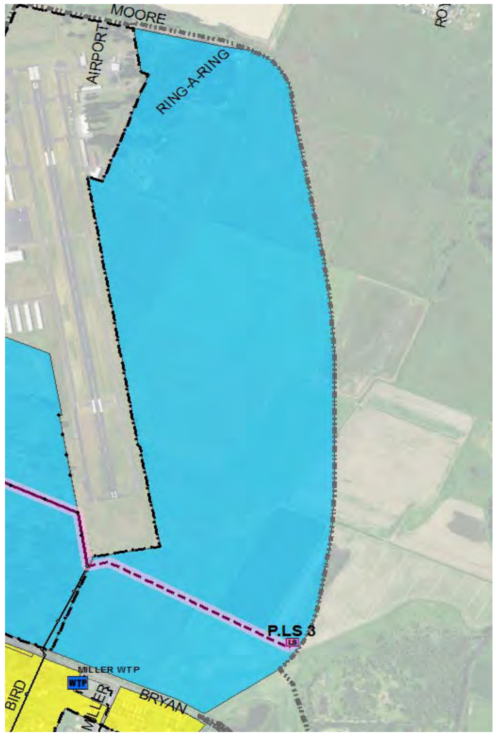
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Collection System Project: Proposed Lift Station 3

Project Identifier: 4C

Objective: New lift station and force main to service City projected future industrial development in the airport industrial area.

Project Location: Airport Industrial Area



Item	Unit	Unit Price	Quantity	Cost (2016)
Lift Station (peak flow of 630 gpm)	LF	$ 263,000	1	$ 263,000
8" Force Main	LF	$ 75	2100	$ 157,500
Bypass Pumping	EA	$ 5,000	1	$ 5,000
Traffic Control	LF	$ 2	2100	$ 4,200
Mobilization	%	5%	-	$ 21,485
Contingency	%	30%	-	$ 135,356
Construction Subtotal (rounded)				**$ 587,000**
Engineering and CMS	%	25%	-	$ 146,750
Legal, Admin, and Permitting	%	2%	-	$ 11,740
Total Project Cost (rounded)				**$ 750,000**

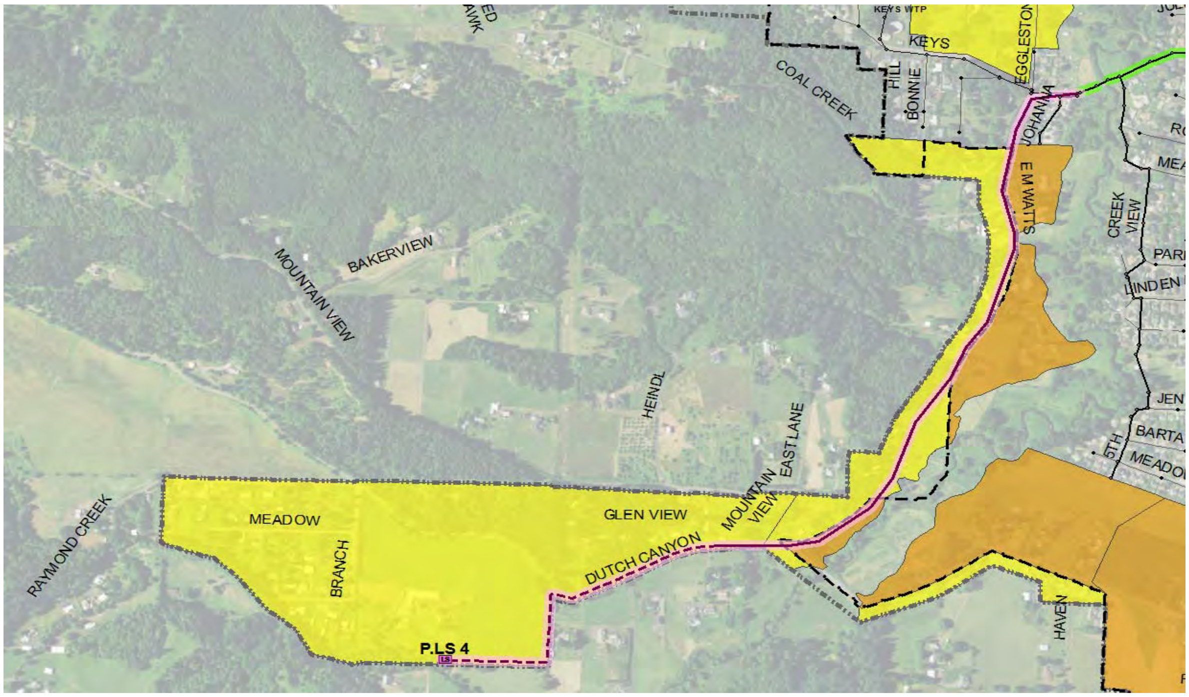
The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

Collection System Project: Proposed Lift Station 4

Project Identifier: 4D

Objective: New lift station, force main and gravity main to service City projected future industrial development out SW Dutch Canyon Road.

Project Location: SW Dutch Canyon Road



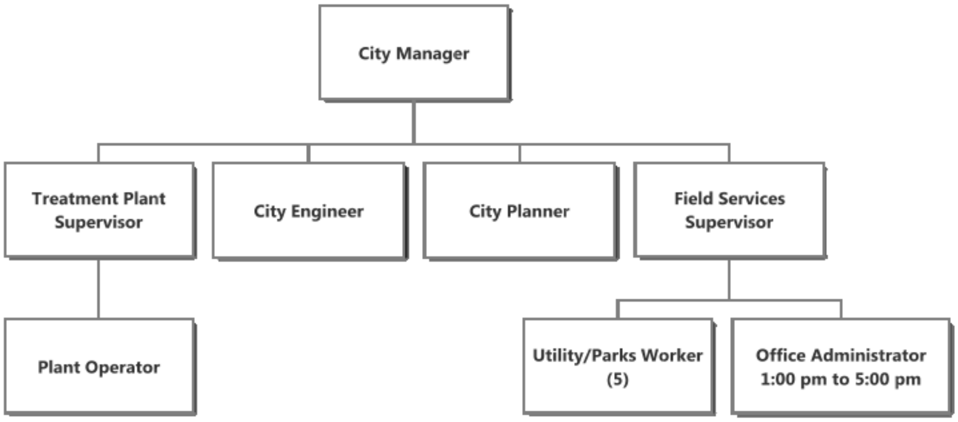
Item	Unit	Unit Price	Quantity	Cost (2016)
Lift Station (peak flow of 185 gpm)	LF	$ 169,000	1	$ 169,000
6" Force Main	LF	$ 70	3000	$ 210,000
8" Gravity Main	EA	$ 60	4900	$ 294,000
Bypass Pumping	EA	$ 5,000	1	$ 5,000
Traffic Control	LF	$ 2	7900	$ 15,800
Mobilization	%	5%	-	$ 34,690
Contingency	%	30%	-	$ 218,547
Construction Subtotal (rounded)				**$ 948,000**
Engineering and CMS	%	25%	-	$ 237,000
Legal, Admin, and Permitting	%	2%	-	$ 18,960
Total Project Cost (rounded)				**$ 1,210,000**

The cost estimate herein is concept level information only based on our perception of current conditions at the project location and its accuracy is subject to significant variation depending upon project definition and other factors. This estimate reflects our opinion of probable costs at this time and is subject to change as the project design matures. This cost opinion is in 2016 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.

APPENDIX H – Financial

H-1:
Excerpt of Adopted Budget 2015-2016

UTILITY WASTEWATER FUND 41



PURPOSE:
The Utility Wastewater Fund is a dedicated "Enterprise" fund. The City of Scappoose operates and maintains a 1.58 M.G.D. activated sludge wastewater treatment plant with tertiary treatment, 36 miles of sewer lines and five pumping stations. The purpose of these facilities is to protect public health and the beneficial uses of the Columbia River and Multnomah Channel by providing secondary effluent treatment. Treatment plant staff is responsible for the operation of the Wastewater facility to ensure proper treatment of all raw wastewater at all times. The Operator is ultimately responsible to ensure all state and federal regulations are met. The City was issued a new permit in March 2009 and will be required to monitor influent, effluent and process control. This monitoring will require the Treatment Plant Operator to sample and test many functions within the treatment plant process, and to sample the receiving stream.

VISION FOR THE YEAR:
The Wastewater fund will focus on biosolids handling issues, as well as continue to make improvements to the collection system. One of two biosolids storage lagoons will be cleaned out and applied to newly certified land. Staff will continue with cleaning and inspection of at least 20% of the collection system.

COMPLIANCE WITH COUNCIL GOALS:
Address aging infrastructure
Update Comprehensive and Master Plans

BUDGET NOTES:
The Wastewater fund will begin the year with a $765,016 beginning cash position. The fund anticipates revenue of $1,073,500. Total fund resources are projected to be $1,838,516. Expenditures within the department include $650,678 for personnel services. The fund budgets $628,324 for materials and services and $340,000 for capital outlay. These items include the cleaning of the biosolids storage lagoon and collection system maintenance and cleaning. The fund budgets $132,594 for principle and interest payments for loan R06809 which matures in 2031 and US Bank loan which matures in 2021. The fund budgets $63,101 for transfers and contingency of $23,819.

PERCENTAGE OF TIME ALLOCATION:

Full Time Equivalent Positions

	12-13	13-14	14-15	15-16	Minimum Salary	Maximum Salary
Treatment Plant Supervisor	100%	100%	100%	100%	24.89	40.37
Operator II	100%	100%	100%	100%	22.89	29.21
Operator III	100%	100%	0%	0%	25.25	32.22
Field Services Supervisor	20%	30%	35%	35%	24.89	40.37

	12-13	13-14	14-15	15-16	Minimum Salary	Maximum Salary
Utility/Parks Worker I	0%	0%	40%	40%	17.43	22.25
Utility/Parks Worker II	100%	145%	55%	55%	20.77	26.51
Utility/Parks Worker III	0%	0%	35%	35%	22.89	29.21
Office Administrator III	38%	38%	38%	25%	18.78	23.97
Associate City Planner	0%	0%	17%	17%	18.78	23.97
City Planner	20%	20%	0%	0%	26.12	33.34
City Engineer	0%	0%	30%	30%	28.47	45.02
City Manager	20%	20%	22%	22%	46.80	59.13
City Recorder	20%	20%	20%	20%	23.37	36.24
Finance Administrator/Office Manager	22%	22%	23%	23%	27.58	43.63
Program Specialist	0%	0%	0%	22%	20.19	28.85
Office Administrator I	44%	44%	44%	44%	15.44	19.71
TOTAL FTE'S	5.84	6.39	5.59	5.68		

Wastewater Fund 41

Resources	Actual FY 12-13	Actual FY 13-14	Budget FY 14-15	Estimated FY 14-15	Proposed Budget FY 15-16	Approved Budget FY 15-16	Adopted Budget FY 15-16
Working capital carryover	$ 940,106	$ 622,487	$ 889,973	$ 956,666	$ 765,016	$ 765,016	$ 765,016
Current year resources							
Interest	$ 3,565	$ 4,458	$ 4,000	$ 4,000	$ 4,000	$ 4,000	$ 4,000
Charges for services	1,048,838	1,047,046	1,062,250	1,056,200	1,069,500	1,069,500	1,069,500
Intergovernmental	69,822						
Long term debt proceeds		700,000		875			
Miscellaneous	728	200					
Total current year resources	$ 1,122,953	$ 1,751,704	$ 1,066,250	$ 1,061,075	$ 1,073,500	$ 1,073,500	$ 1,073,500
Total resources	$ 2,063,059	$ 2,374,191	$ 1,956,223	$ 2,017,741	$ 1,838,516	$ 1,838,516	$ 1,838,516

Expenditures	Actual FY 12-13	Actual FY 13-14	Budget FY 14-15	Estimated FY 14-15	Proposed Budget FY 15-16	Approved Budget FY 15-16	Adopted Budget FY 15-16
Personnel services	$ 640,282	$ 561,814	$ 595,237	$ 535,504	$ 650,678	$ 650,678	$ 650,678
Materials & services	395,705	371,284	460,343	385,840	628,324	628,324	628,324
Capital outlay	324,534	393,121	126,500	85,000	340,000	340,000	340,000
Debt service							
1994 Principal SPWF B92001B 12/01	7,125	7,552	7,552				
1994 Interest SPWF B92001B 12/01	881	453	453				
1995 Principal SPWF B92001C 12/01							
2009 Principal CWSRF R06809 3/01 & 9/01	17,642	17,642	17,642	17,642	17,642	17,642	17,642
2009 Interest CWSRF R06809 9/01	816	772	1,455	1,455	1,367	1,367	1,367
2013 Principal USNB 12/01			100,000	100,000	100,000	100,000	100,000
2013 Interest USNB 6/01 & 12/01		8,453	16,055	16,055	13,585	13,585	13,585
2008 Principal Ford Motor Lease 12/01	899						
2008 Interest Ford Motor Lease 12/01	56						
Transfers	52,632	56,433	111,229	111,229	63,101	63,102	63,102
Contingency			519,757		23,819	23,818	23,818
Total expenditures	$ 1,440,572	$ 1,417,525	$ 1,956,223	$ 1,252,725	$ 1,838,516	$ 1,838,516	$ 1,838,516
Ending working capital	**$ 622,487**	**$ 956,666**	**$ -**	**$ 765,016**	**$ -**	**$ -**	**$ -**

2012/2013 Actual	2013/2014 Actual	2014/2015 Adopted	2014/2015 Estimated	Account	Utility Wastewater Fund Description	2015/2016 Budget
3,565	4,458	4,000	4,000	41-000-003	Interest Earned	4,000
728	200	-	875	41-000-100	Miscellaneous	-
69,822	-	-	-	41-000-150	Intergovernmental Revenue	-
-	700,000	-	-	41-000-151	Long Term Debt Proceeds	-
1,048,238	1,043,917	1,058,000	1,050,000	41-000-220	User Fees	1,063,000
600	3,129	2,250	4,200	41-000-240	Hookup Fees	4,500
-	-	2,000	2,000	41-000-260	Infrastructure Inspection Fees	2,000
1,122,953	**1,751,704**	**1,066,250**	**1,061,075**		**Total Revenue**	**1,073,500**
20,604	38,257	22,562	12,000	41-410-100	City Manager	28,408
13,214	12,704	13,581	6,000	41-410-104	Associate City Planner	7,232
18,795	19,308	19,555	19,555	41-410-106	Finance/Office Manager	20,852
-	-	-	-	41-410-107	Program Specialist	11,043
17,163	17,504	17,655	17,655	41-410-108	Office Administrator I	18,005
14,301	14,585	14,766	14,766	41-410-110	City Recorder	15,063
-	-	22,211	19,500	41-410-113	City Engineer	27,646
79,685	83,252	75,794	75,794	41-410-114	Treatment Plant Supervisor	80,403
27,291	14,414	17,816	17,816	41-410-118	Field Services Supervisor	21,944
124,109	95,078	64,595	64,595	41-410-120	Operators	67,924
75,360	50,773	68,211	66,500	41-410-121	Utility Workers	69,351
17,998	18,321	18,548	18,548	41-410-132	CDC Administrator	12,450
1,431	1,505	1,450	3,500	41-410-142	Overtime	4,725
105,256	88,284	129,253	104,000	41-410-146	Health Insurance	143,026
83,130	70,429	70,058	60,000	41-410-148	Retirement Benefits	78,941
30,789	26,621	28,540	25,000	41-410-150	Social Security	30,804
11,156	10,779	10,642	10,275	41-410-152	Workers Compensation	12,861
640,282	**561,814**	**595,237**	**535,504**		**Total Personnel Services**	**650,678**
4,720	2,962	8,000	8,000	41-410-200	Building/Facilities Maintenance	9,680
58,805	9,891	7,364	16,000	41-410-202	Equipment Maintenance	23,864
1,774	2,158	2,400	2,400	41-410-203	Maintenance Agreements	2,200
3,129	2,206	8,460	3,000	41-410-204	Vehicle Maintenance	8,190
16,401	17,127	9,050	7,000	41-410-205	Small Equipment	10,500
3,934	4,021	3,760	1,900	41-410-206	Fuel/Oil/Lube	4,200
5,081	5,545	7,220	4,000	41-410-216	Office Supplies	9,320
8,166	4,917	7,770	7,770	41-410-218	Operational Supplies	7,770
-	3	1,650	500	41-410-220	Shop Maintenance Supplies	850
3,586	3,558	7,775	3,500	41-410-222	Lab Supplies	8,875
5,466	607	6,400	2,000	41-410-224	Chemicals	3,080
78,135	82,966	85,800	85,000	41-410-227	Electrical Operations	85,800
10,801	8,835	11,724	12,000	41-410-228	Utilities	12,060
6,025	6,549	7,200	6,800	41-410-229	Electrical Operations Pumps	7,500
171,032	126,016	180,100	170,100	41-410-230	Contractual/Professional	224,800
-	-	100	-	41-410-234	Miscellaneous	-
2,099	1,860	2,000	1,870	41-410-235	Property Tax	2,000
-	192	-	-	41-410-238	Insurance	-
6,999	2,429	7,475	5,000	41-410-240	Travel/Training	9,750
4,162	18,713	4,870	4,500	41-410-242	Dues/Fees/Subscriptions	5,610
61	538	500	500	41-410-244	Publications/Notices/Advertise	2,000
3,939	1,126	2,725	2,000	41-410-252	Uniforms/Safety	2,275
-	65,010	80,000	40,000	41-410-253	Sludge Disposal	172,000
1,390	4,058	8,000	2,000	41-410-254	Equipment Rental	16,000
395,706	**371,284**	**460,343**	**385,840**		**Total Materials & Services**	**628,324**
324,534	393,121	126,500	85,000	41-410-300	Equipment	340,000
324,534	**393,121**	**126,500**	**85,000**		**Total Capital Outlay**	**340,000**
899	-	-	-	41-410-500	Vehicle Lease Principal	-
56	-	-	-	41-410-501	Vehicle Lease Interest	-
-	-	100,000	100,000	41-410-510	Principal USNB	100,000
-	8,453	16,055	16,055	41-410-511	Interest USNB	13,585
7,125	7,552	7,552	-	41-410-540	Principal SPWF B92001B	-
881	453	453	-	41-410-541	Interest SPWF B92001B	-
-	-	-	-	41-410-550	Principal SPWF B95001C	-
17,642	17,642	17,642	17,642	41-410-560	Principal CWSRF R80930	17,642
816	772	1,455	1,455	41-410-561	Interest CWSRF R80930	1,367
27,418	**34,872**	**143,157**	**135,152**		**Total Debt Services**	**132,594**
-	-	4,800	4,800	41-410-418	Transfer to Unemployment	-
52,632	56,433	106,429	106,429	41-410-419	Transfer to General Fund ISF	63,101
52,632	**56,433**	**111,229**	**111,229**		**Total Transfers**	**63,101**
-	-	519,757	-	41-410-600	Contingency	23,819
1,440,572	**1,417,525**	**1,956,223**	**1,252,725**		**Total Expenditures**	**1,838,516**

WASTEWATER SDC FUND 55

PURPOSE:

The Wastewater System Development Charge fund is a dedicated fund and is the mechanism by which the City of Scappoose collects funds from developers to pay for previous excess capacity improvements. It also allows funds to be available to pay for future improvement needs generated by development. Wastewater SDC's are calculated based on the size of the water meter needed for the development. This account includes both the revenue and the capital outlay for those projects.

VISION FOR THE YEAR:

Continue capital project improvements along with updating the Wastewater Master Plan.

COMPLIANCE WITH COUNCIL GOALS:

Address aging infrastructure
Update Comprehensive and Master Plans

BUDGET NOTES:

The Wastewater SDC fund has a beginning cash position of $605,930. The City anticipates receiving $2,900 in interest income and $149,160 in SDC fees. Total resources for the fund are $757,990. Projects proposed for FY 15-16 include $425,000 to continue working on biosolids handling improvements and $225,000 to update the Wastewater Master Plan. The fund contains a transfer to the General Fund of $7,458 for administrative costs and a contingency of $100,532.

Wastewater SDC 55

Resources	Actual FY 12-13	Actual FY 13-14	Budget FY 14-15	Estimated FY 14-15	Proposed Budget FY 15-16	Approved Budget FY 15-16	Adopted Budget FY 15-16
Working capital carryover	$ 478,274	$ 460,587	$ 525,344	$ 509,262	$ 605,930	$ 605,930	$ 605,930
Current year resources							
Interest	$ 2,815	$ 2,787	$ 3,000	$ 2,850	$ 2,900	$ 2,900	$ 2,900
System development charges	8,332	90,992	73,648	132,500	149,160	149,160	149,160
Total current year resources	$ 11,147	$ 93,779	$ 76,648	$ 135,350	$ 152,060	$ 152,060	$ 152,060
Total resources	$ 489,421	$ 554,366	$ 601,992	$ 644,612	$ 757,990	$ 757,990	$ 757,990

Expenditures	Actual FY 12-13	Actual FY 13-14	Budget FY 14-15	Estimated FY 14-15	Proposed Budget FY 15-16	Approved Budget FY 15-16	Adopted Budget FY 15-16
Capital outlay	$ 28,253	$ 43,660	$ 450,000	$ 35,000	$ 650,000	$ 650,000	$ 650,000
Transfers	581	1,443	3,682	3,682	7,458	7,458	7,458
Contingency			148,310		100,532	100,532	100,532
Total expenditures	$ 28,834	$ 45,103	$ 601,992	$ 38,682	$ 757,990	$ 757,990	$ 757,990
Ending working capital	**$ 460,587**	**$ 509,262**	**$ -**	**$ 605,930**	**$ -**	**$ -**	**$ -**

2012/2013 Actual	2013/2014 Actual	2014/2015 Adopted	2014/2015 Estimated	Account	Wastewater SDC Fund Description	2015/2016 Budget
2,815	2,787	3,000	2,850	55-000-003	Interest Earned	2,900
8,332	90,992	73,648	132,500	55-000-993	Sewer SDC Ext. Cap. Improvements	149,160
11,147	**93,779**	**76,648**	**135,350**		**Total Revenue**	**152,060**
28,253	43,660	400,000	25,000	55-550-314	Sewer Extra Capacity Improvements	425,000
-	-	50,000	10,000	55-550-326	Council Approved Projects	225,000
28,253	**43,660**	**450,000**	**35,000**		**Total Capital Outlay**	**650,000**
581	1,443	3,682	3,682	55-550-409	Transfer to GF SDC Admin.	7,458
581	**1,443**	**3,682**	**3,682**		**Total Transfers**	**7,458**
-	-	148,310	-	55-550-600	Contingency	100,532
28,834	**45,103**	**601,992**	**38,682**		**Total Expenditures**	**757,990**

H-2:
User Rate Calculations

City of Scappoose
Wastewater Facilities Planning Study
User Rate Impacts

Existing Budget is Balanced		
	FY 2015-2016 Revenue	$ 1,073,500

After SDCs, Priority 1 Improvements		
	Priority 1 Improvement Costs	$ 5,140,000
	Less Available SDC Funds / Reserves	$ -
	Amount to Finance	$ 5,140,000
	Annual Payment (20yr, 1.6%)	$ 302,343
	Annual O&M Increase	$ -
	Payment % of FY 2015-2016 Revenue	28%
	Additional Monthly Cost Per User (2700 EDUs)	**$ 9.33**