

TUESDAY, SEPTEMBER 4, 2018 CITY COUNCIL

TOUR OF MILLER PARK (located at 52451 Miller Road), 6:00 PM

INTERVIEW FOR ECONOMIC DEVELOPMENT COMMITTEE, 6:45 PM Will be held in the Council Chambers located at City Hall, 33568 East Columbia Avenue

CITY COUNCIL MEETING AGENDA

Regular meeting 7:00 p.m. Scappoose Council Chambers 33568 East Columbia Avenue

ITEM AGENDA TOPIC

Action

Call to Order

Pledge of Allegiance

Roll Call

Approval of the Agenda

Public Comments

1. Consent Agenda ~ August 20, 2018 City Council meeting minutes, and appointment to the Planning Commission

New Business

- **2. Request to Establish an Outside Hookup Connection to City Water**Staff: City Manager Michael Sykes

 Approval
- 3. Ordinance No. 873: Updating the City of Scappoose Wastewater Facilities Plan
 Staff: Treatment Plant Supervisor Kevin Turner

 Public Hearing/First Reading

Announcements ~ information only

4. Calendar

City Manager, Police Chief, Councilors, and Mayor

Adjournment

MONDAY, AUGUST 20, 2018 CITY COUNCIL

Scappoose Creek Project Tour, 6:00 p.m. (They met at Veterans Park, in parking lot, 52590 NW Captain Roger Kucera Way)

Present: Mayor Burge, Council President Reed, Councilor Kessi, Councilor Greisen, Councilor Poling, and City Manager Sykes.

Also present: City Planner Laurie Oliver, Public Works Director Dave Sukau.

CITY COUNCIL MEETING AGENDA Regular meeting 7:00 p.m.

Call to Order

Mayor Burge called the meeting to order at 7:04 p.m.

Pledge of Allegiance

Roll Call

City Council Members Present:	Staff Present:
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Scott Burge	Mayor	Michael Sykes	City Manager
Patrick Kessi	Councilor	Norm Miller	Police Chief
Megan Greisen	Councilor	Susan M. Reeves	City Recorder
Josh Poling	Councilor	Dave Sukau Public	Works Director
		Darryl Sykes Treatm	ent Plant Supervisor

Peter Watts Legal Counsel Press: None

Excused: Councilor Natalie Sanders and Councilor Joel Haugen

Also present in the audience: Police Officer Joel Dale, Police Officer Travis Killens, and Consultant Dan Reisinger with Carollo Engineers.

Approval of the Agenda

Councilor Greisen moved, and Council President Reed seconded the motion that Council approve the agenda. (Motion passed 5-0). Mayor Burge, aye; Council President Reed, aye; Councilor Kessi, aye; Councilor Greisen, aye, and Councilor Poling, aye.

Public Comments

Debbie Reed, Scappoose, explained she was asked to come before Council this evening to discuss a cottonwood tree at Creekview Park. She brought a handout with photos of the tree and

the branches that had fallen. She explained she notified Public Works and they came that day with chainsaws to clean up the branches. She stated there is an issue with this tree, and since the photos were taken two additional branches have fallen, not to the degree that need a chainsaw. She stated now there is another one dangling, ready to fall. She stated we need someone to come out and look at this tree and make a determination, but in the interim, she is not walking that way anymore. She asked for a barrier until the tree is looked at and that other branch is down.

Councilor Reed explained a similar thing occurred at his mother's, and he cut it up and the that branch weighed 1,000 pounds. He explained it was a smaller branch, and the ones at Creekview Park are about twice that size. He thinks it would be a good idea to have an arborist come look at the tree.

Public Works Director Dave Sukau replied this is the first he has heard of it, but we will look into it.

Consent Agenda ~ June 25, 2018 Special City Council meeting minutes, July 16, 2018 City Council Work Session, July 16, 2018 City Council meeting minutes, appointment of the Park and Recreation Committee members (staggering terms), appointment of Simon Date, South Columbia County Chamber and Paul Peterson Scappoose School District as Economic Development Committee Liaisons, and appointment to the Economic Development Committee (staggering terms)

Council President Reed moved, and Councilor Poling seconded the motion that Council approve the consent agenda ~ Consent Agenda ~ June 25, 2018 Special City Council meeting minutes, July 16, 2018 City Council Work Session, July 16, 2018 City Council meeting minutes, appointment of the Park and Recreation Committee members (staggering terms), appointment of Simon Date, South Columbia County Chamber and Paul Peterson Scappoose School District as Economic Development Committee Liaisons, and appointment to the Economic Development Committee (staggering terms). (Motion passed 5-0). Mayor Burge, aye; Council President Reed, aye; Councilor Kessi, aye; Councilor Greisen, aye, and Councilor Poling, aye.

Swearing in of Municipal Court Judge ~ Cynthia Phillips

Mayor Burge administered the Oath of Office to Cynthia Philips as Municipal Court Judge.

Cindy Phillips thanked Council for appointing her as municipal court judge.

Council thanked Cindy.

Congratulations Judge Phillips.

Presentation to Scappoose Police Officer

Police Chief Norm Miller explained tonight he wants to recognize Officer Joel Dale and his son. He explained Joel Dale went to a defensive tactic instructors' school and we received a letter from that group, which he read. He explained years ago Joel talked to Norm at a car show and

explained how he wanted to be a patrol office and how he wanted to work for Scappoose. He explained he has been with Scappoose for almost four years now. He explained on June 23, Officer Dale conducted a premises check under the bridge of Scappoose Vernonia Highway and Highway 30, in which he arrested individuals for offensive littering. He explained on June 24, while off duty Joel Dale and his son, took it upon themselves to go to the location again and clean up the garbage left by the offenders. He believes that the actions by Officer Joel Dale truly go above and beyond the expectations of Officers here at the Scappoose Police Department and is deserving of recognition. He explained tonight Officer Dale is receiving a letter of commendation for his actions for going above and beyond the call of duty on June 24, 2018. He also gave his son a letter of commendation for his actions for going above and beyond as a citizen of the City of Scappoose. He thanked them both for what they have done.

Interviews for Scappoose Planning Commission Vacancies

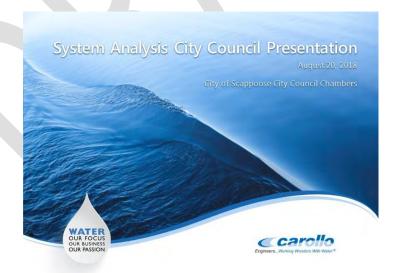
Mayor Burge explained there are openings on the Planning Commission and there are citizens here this evening who have applied for those openings.

Council interviewed Kevin Freimuth first, then Patricia Gibson for the opening positions.

Mayor Burge explained at the next Council meeting he will make recommendations for appointments to the Planning Commission.

WORK SESSION ~ Water Master Plan ~ System Analysis Part 2

Consultant Dan Reisinger with Carollo Engineers and Treatment Plant Supervisor Darryl Sykes went over the System Analysis presentation.



Water System Analyses

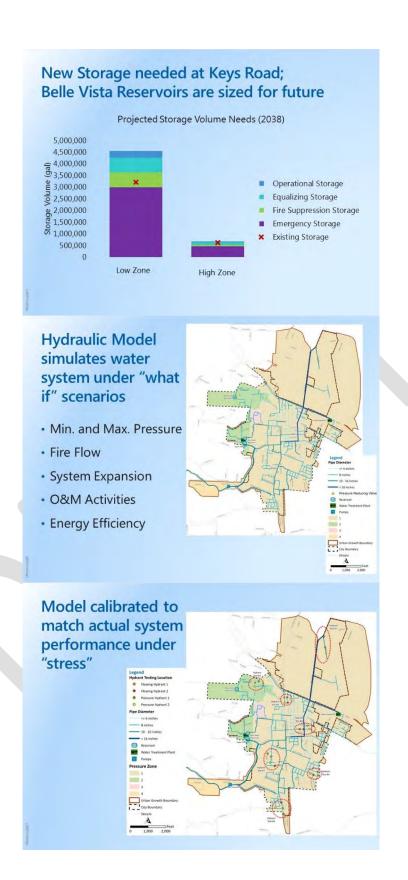
- Capacity Analyses
- Condition Evaluation
- Supply Decisions
- Next Steps

Water System Analyses

- Capacity Analyses
 - Storage
 - Distribution System
- Condition Evaluation
- Supply Decisions
- · Next Steps

Storage analysis considers normal and emergency operations







- · Residential: 1,000 gpm
- · Commercial: 2,000 gpm
- Industrial: 3,000 gpm
- Fire flow requirement can exceed existing building need



System generally provides required fire flows

- Only deficient areas are marked
- Colored pipes have increased risk of maintenance





Water System Analyses

- Capacity Analyses
- Condition Evaluation
 - WTP
 - Pipelines
- Supply Decisions
- Next Steps

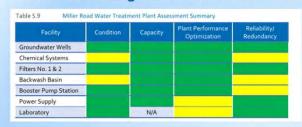
Rehabilitation of existing infrastructure determined through high-level condition assessment

- · Site Tour
 - Surface water intakes
 - Keys Road WTP
 - Miller Road WTP
- GIS Based Analysis
 - Pipeline Remaining Useful Life

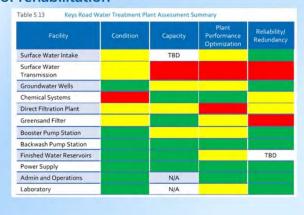
Water treatment plant rated from positive (Green) to negative (Red)



Miller Road WTP provides the system longterm Iron and Manganese treatment



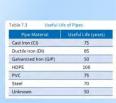
Keys Road Treatment Plant aging and in need of rehabilitation

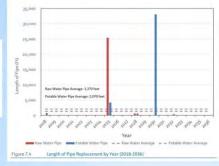


Plan provides recommendations to aid in implementing Keys Road WTP rehabilitation



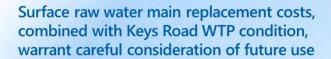
Pipeline replacement planned based on remaining useful life





Replacement activities focused in older City center and surface water raw water main





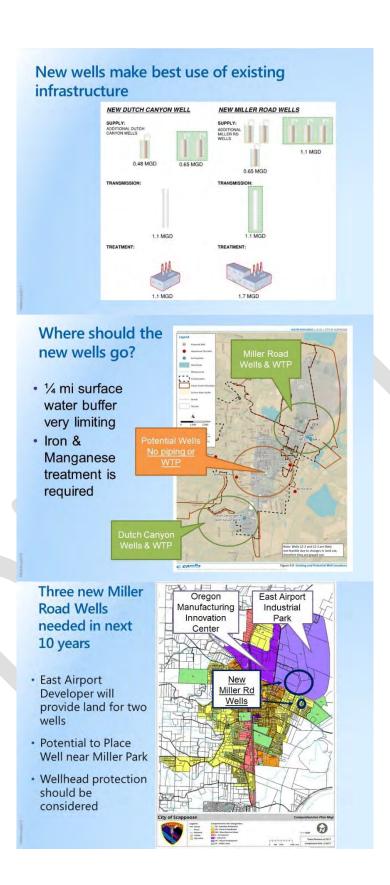


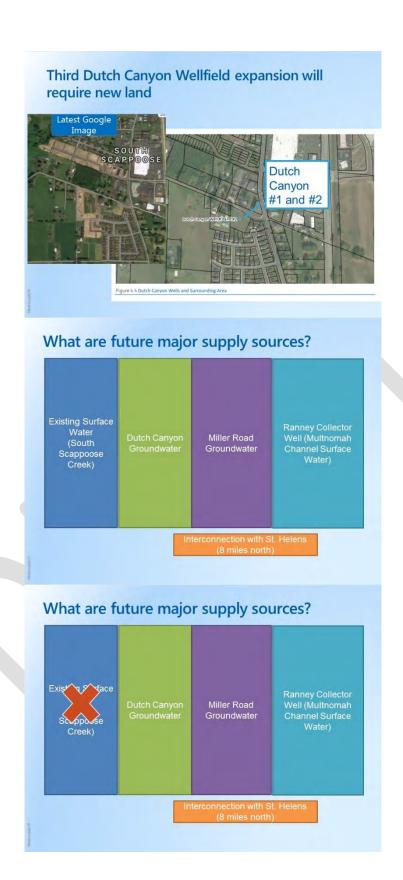
Water System Analyses

- Capacity Analyses
- Condition Evaluation
- Supply Decisions
 - Short-term
 - Long-term
- Next Steps

New water supplies needed in 2031









Existing surface water infrastructure needs rehabilitation



What are future major supply sources?





Collect additional information as City completes new wells

- Highly sensitive to the location and yield of groundwater wells
 - Existing Well Yields = 5 new wells

OR

- 40% greater yield = 3 new wells♥

What are future major supply sources?





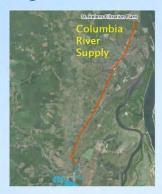
Long transmission length makes St. Helens interconnection less compelling

Benefits

- Proven Supply
- · Shared O&M

Challenges

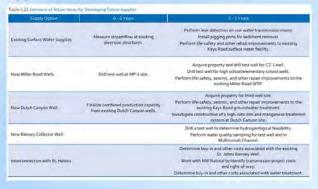
- Single Line
 - Limited redundancy
- Water age
- · Cost of transmission



New groundwater wells or a Ranney well have the least cost per gallon



Action Items defined for City to investigate future supplies



Water System Analyses

- Capacity Analyses
- Condition Evaluation
- Supply Decisions
- Next Steps

Next Steps

- Complete financial analysis of Water System Plan Capital Improvements
- · Present Financial Analysis to the City Council
- City Council to approve sending Agency Draft Plan to Regulator review
- Regulatory review and comment

Council thanked Dan and Darryl.

Announcements ~ information only

Calendar

Mayor Burge went over the calendar.

City Manager, Police Chief, Councilors, and Mayor

City Manager Sykes explained he learned a long time ago that you are only as good as the team that you have and he feels real fortunate in Scappoose that we have a really great staff, that is really jelling as a team and we are all on the same page, and focused on the goals set by the Council, and it really makes his job easy to have talent like that. He talked about how successful the National Night Out was, and stated Norm has done a lot of great work building support with the community by holding events like that. He explained Dave has been with us a year and a half

and it is amazing to see what he is doing with our Public Works Department. He explained Public Works has a new employee Scott Jauron, who was previously with the City of St. Helens, and we were lucky enough to have him apply for an opening we had. He explained tomorrow they break ground on Chief Concomly Park. He talked about improvements happening at Miller Park. He explained staff will be doing an overlay on EM Watts all the way to Dutch Canyon, which is long overdue. He explained Dave has really helped build the partnership with Columbia County, which is really important. He explained last Friday staff met with PCC and they are really excited to move forward. He explained fall of 2020 is when they want to have a facility up and operating. He explained the City will be doing a community survey in the next month or so, if there are any questions you want us to ask the community, let us know. He explained the Council should be congratulated for your contribution to the movie night in Heritage Park.

Police Chief Norm Miller explained National Night Out was a huge success. He explained Red Apple provided a lot of our food and materials. He explained we went through 200 hamburgers in an hour and fifteen minutes. He explained there were approximately 500 people who attended, and it was a blast. He met a lot of people who are new to Scappoose. He explained their back to school event they normally have will be held at a home football game.

Public Works Director Dave Sukau explained the street sweeping schedule for October will hopefully be out in the next couple of weeks. He explained Charles T. Parker Way road should commence any day now.

Councilor Kessi explained they had a Traffic Safety Committee meeting today and there are a couple of areas close to the schools that will hopefully have some additions to help kids get to school safer. He explained they have a meeting setup with an architect, who is an urban planner, for next Thursday.

Councilor Greisen explained she feels proud to be part of a team that works so well together, and everything that Public Works is doing at the parks and being resourceful, she just really appreciates that. She stated you just value where you live, and she notices that a lot about the public. She explained when people can see that we want it to be a better place, she feels proud of that. She stated Wings and Wheels was great, thank you Mayor Burge.

Mayor Burge talked about the County Wide Tourism meeting he attended. He explained Wings and Wheels was a success. He explained they had raffle items, and one of them was for the opening game to the Ducks, which he also got a football to go with it. He explained there was a young man that kept coming back to buy more and more raffle tickets to win this prize. He explained he and Kevin were doing the drawing and they were hoping they might draw his ticket, and they did. He explained he spoke to the young man afterwards and he was so excited and explained his dad went to University of Oregon and he is a huge fan and always watches the game on tv. The young man sent his dad a message and asked him if he was going to watch the home opener on tv, knowing he would say yes, and the kid said, "no you are not, you are going to Eugene with me to watch it".

18

Mayor Burge said it was so cool to see this young man win tickets, so he could take his dad.

Mayor Burge explained they are already working on what they can do next year to make it an even better event.

Adjournment

Mayor Burge adjourned the meeting at 8:49 p.m.	

	Mayor Scott Burge
Attest:	
City Recorder Susan M. Reeves, MMC	

City of Scappoose

Request to establish an outside hookup connection to the City water system by individual lands outside the City limits as defined by SMC 13.04.020

Prior to request, please review the terms and conditions outlined in SMC13.04 located on our website at: http://www.ci.scappoose.or.us/municipalcode which includes the following key provisions;

- 1) No cross connections without approved backflow assembly
- 2) No auxiliary water supply without approved backflow assembly
- 3) Any required backflow assembly shall be maintained and inspected annually, at no cost to the City, with results provided to the City.
- 4) Backflow device may need to be installed at the point of use (house) with access allowed for inspection, as required.
- 5) Any auxiliary water systems which are no longer required, after receiving City services shall be decommissioned in accordance with Oregon Water Resources at; www.wrd.state.or.us, or by calling District 18 Northwest Region Water Master at (503) 846-7780.
- 6) Decrease in water pressure as a result of the required backflow and subsequent need to install a pressure pump will not be the responsibility of the City.
- 7) All costs for compliance with the SMC 13.04 shall be the financial responsibility of the owner, occupant or other person in control of the premises.
- 8) Service shall be terminated for failure to adhere to all of the requirements for testing, cross connections, use of auxiliary services as further outlined in SMC 13.04.

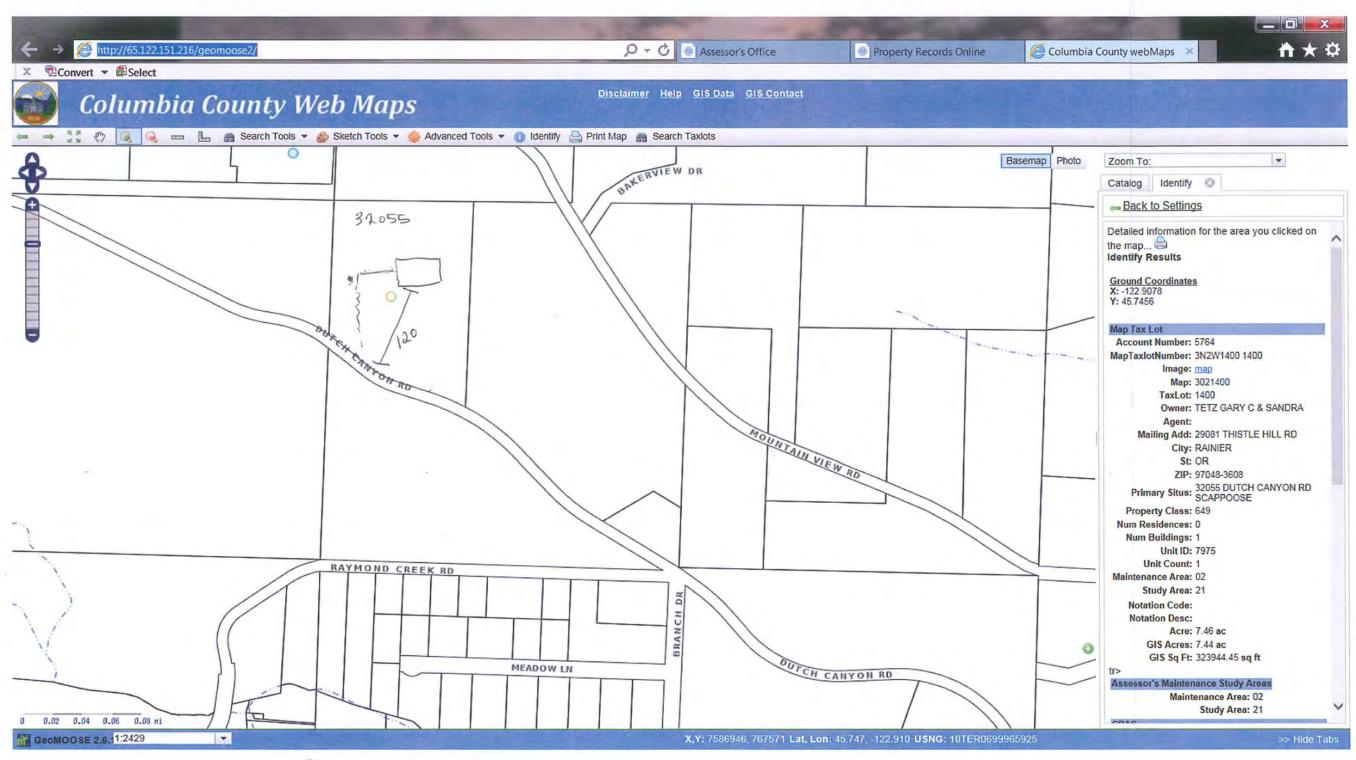
I have read and agree to the requirements, terms and conditions as outlined in SMC 13.04 Water Service System;

Property Owne	r Information;
Date:	Aug 16 2018
Name:	Gary Tetz Sandra Tetz
Signature:	Many tet 1975
Address:	32055 Dutch Cangon Rd
	Scappoose One 97056
Phone Number:	503-556-2004
Email address:	Sandy Tetza a.com

Application information to Community Development Center for review prior to approval;

- 1) Provide a map with the following information for determination of the type of connection required;
 - a) Property location and approximate elevation at the proposed use, i.e., finished floor elevation of house.
 - b) Location and configuration of any auxiliary water sources (non-city water systems).
 - c) Location and configuration of any cross connection systems as outlined in SMC 13.04, i.e., non-drinkable water system or auxiliary systems capable of imparting contamination to the public system as a result of backflow.

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And The very good	wells	in I	Outch	cony	101	are	not
f approved by City Mai The applicant shall con		wing:					
 Attend City Council vote by Council, cor Pay Water SDC Fe 	nnection can b						
Pay Cost of Meter 8 Pay \$1,000 outside	& Time & Mate City user fee						
B) Pay Water Hookup A) Pay Cost of Meter 8 B) Pay \$1,000 outside B) Obtain a Columbia Approved	& Time & Mate City user fee		Den	ied			
Pay Cost of Meter 8 Pay \$1,000 outside Obtain a Columbia	& Time & Mate City user fee County Plumb			ied 3/18			
Pay Cost of Meter 8 Pay \$1,000 outside Obtain a Columbia Approved	& Time & Mate City user fee County Plumb		Den	ied			
Pay Cost of Meter 8 Pay \$1,000 outside Obtain a Columbia Approved City Manager's signature	& Time & Mate City user fee County Plumb		Den	3/18			



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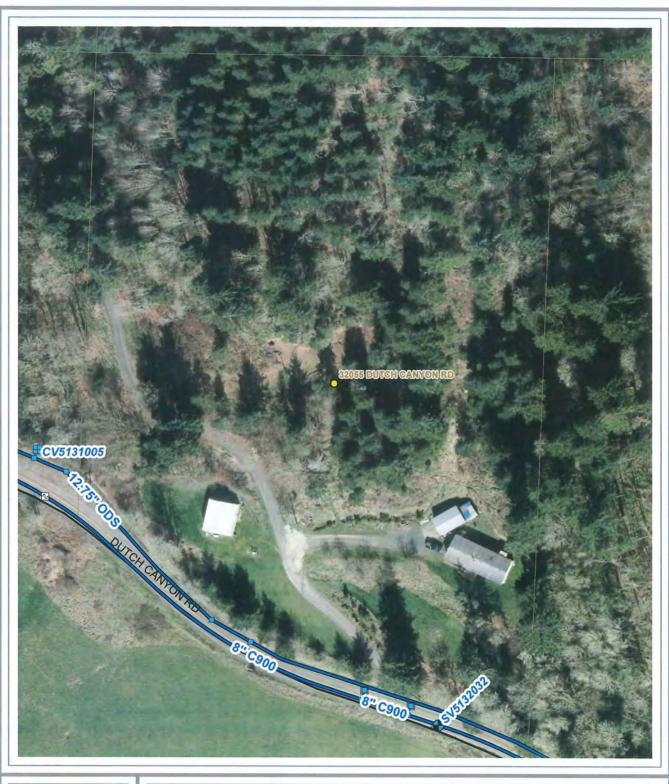
Tetz Property – Outside Water User Request

Appro	ximate Elevation:	140 6
	Meter box:	140-ft
	Home:	180-ft
Systen	n Pressure:	
	Pressure at meter stop valve:	130 psi
	Pressure loss due to change in elev ($\Delta = 40$ -ft x 0.433 psi/ft):	17 psi
	Net pressure at House ¹ Does not account for head loss in piping, meter, reducing valve or backflow device	113 psi
	tions of Approval:	
	Pressure reducing valve required due to static pressure at the meter exce	
	Pressure reducing valve required due to static pressure at the meter exce Backflow device required due to elevation difference between home and	d meter
1. 2. 3.	Pressure reducing valve required due to static pressure at the meter exce Backflow device required due to elevation difference between home and Pipe and backflow device sizing as required by County Building Official	d meter
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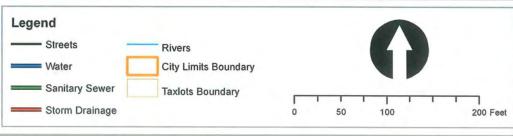
Enc: GIS exhibit of property

Oregon Plumbing Specialty Code section 608

Photos of the meter and well









WATER SUPPLY AND DISTRIBUTION

606.5 Control Valve. A control valve shall be installed immediately ahead of each water-supplied appliance and immediately ahead of each slip joint or appliance supply.

Parallel water distribution systems shall provide a control valve either immediately ahead of each fixture being supplied or installed at the manifold, and shall be identified with the fixture being supplied. Where parallel water distribution system manifolds are located in attics, crawl spaces, or other locations not readily accessible, a separate shutoff valve shall be required immediately ahead of each individual fixture or appliance served.

606.6 Accessible. Required shutoff or control valves shall be accessible.

606.7 Multiple Fixtures. A single control valve shall be installed on a water supply line ahead of an automatic metering valve that supplies a battery of fixtures.

607.0 Potable Water Supply Tunks.

607.1 General. Potable water supply tanks shall be astralled in accordance with the manufacturer's treatellation instructions, and supported in accordance with the haidding make

607.2 Potable Water Tanks. Putable water stipply tanks, interior tank coatings, or task traces introduced to supply drinking water shall be in accordance with NSF 81.

607.3 Venting. Tanks used for putable water dual he tightly covered and vented in accordance with the manufacturer's installation instructions. Such went shall be transmit with a corrosion-resistant material of the base from material 24 mesh.

607.4 Overflow. Tanks shall have first less than a Th square inch (0.01 m²) overflow that is screened with a corosion-resistant material of not less than aurabet 24 mosh

607.5 Valves. Pressurized tanks shall be provided with a listed pressure-relief valve installad in accordance with the manufacturer's installation instructions. This relief valve shall be discharged in accordance with Section 608.5. Where a potable water supply tank is located above the fixtures, appliances, or system components it serves it shall be equipped with a vacuum relief valve that is in accordance with CSA Z21.22.

608.0 Water Pressure, Pressure Regulators, Pressure Relief Valves, and Vacuum Relief Valves.

608.1 Inadequate Water Pressure. Where the water pressure in the main or other source of supply will not provide a residual water pressure of not less than 15 pounds force per square inch (psi) (103 kPa), after allowing for friction and other pressure losses, a tank and a pump or other means that will provide said 15 psi (103 kPa) pressure shall be installed. Where fixtures, fixture fittings, or both are installed that require residual pressure exceeding 15 psi (103 kPa), that minimum residual pressure shall be provided.

608.2 Excessive Water Pressure. Where static water pressure in the water supply piping is exceeding 80 psi (552 kPa), an approved-type pressure regulator preceded by an adequate strainer shall be installed and the static pressure

reduced to 80 psi (552 kPa) or less. Such regulator(s) shall < control the pressure to water outlets in the building unless otherwise approved by the Building Official, Each such regulator and strainer shall be accessibly located aboveground or in a vault equipped with adequate means to provide drainage and shall be protected from freezing, and shall have the strainer readily accessible for cleaning with-out removing the regulator or strainer body or disconnecting the supply piping. Pipe size determinations shall be based on 80 percent of the reduced pressure where using Table 610.4. An approved expansion tank shall be installed in the cold water distribution piping downstream of each such regulator to prevent excessive pressure from develop-ing due to thermal expansion and to maintain the pressure setting of the regulator. Expansion tanks used in potable water systems intended to supply drinking water shall be in accordance with NSF 61. The expansion tank shall be properly sized and installed in accordance with the manufacturer's installation instructions and listing. Systems durigned by registered durigo professionals shall be persented to loss approved pressure relief valves in lieu of reparents canks provided such robef valves have a maxion pressure relationability of 1000 psi (689 kPa) or less.

606.3 Expuredon Tunks, and Combination Temperature and Programs Ballet Valves. A water system provided with a clock valve, backflow preventer, or other namedly absolutely to the provided with an approved, thirted, and adequately wited expansion tank or other approved device having a similar function to control thereproved device having a similar function to control thereproved device in the partition and or other approved device thall be installed on the building side of the check salve, backflow preventer, or after device and shall be streed and installed in accordance with the manufacturer's installation instructions. But shall be installed between the expansion tank and the system.

A native system continuing stringe water heating equipment shall be provided with an approved, listed, adequately sized combination temperature and pressure-relief valve, except for listed nonstorage instantaneous heaters having an inside diameter of not more than 3 inches (80 mm). Each such approved combination temperature and pressure-relief valve shall be installed on the water-heating device in an approved location based on its listing requirements and the manufacturer's installation instructions. Each such combination temperature and pressure-relief valve shall be provided with a drain in accordance with Section 608.5.

608.4 Pressure Relief Valves. Each pressure relief valve shall be an approved automatic type with drain, and each such relief valve shall be set at a pressure of not more than 150 psi (1034 kPa). No shutoff valve shall be installed between the relief valve and the system.

608.5 Draina. Relief valves shall be provided with a full-sized drain; not smaller than the relief valve outlet; of galvanized steel, hard-drawn copper, or CPVC piping and fittings and shall extend from the valve to a discharge location that will avoid the hazard to persons or damage to property. Discharge locations outside of the building shall be not less



Shared driveway looking west away from Dutch Canyon Road (Note meter vault to left of address sign)



Meter vault too small for meter/reducing valve



32055 Dutch Canyon Drive, Scappoose Oregon



Well head looking west



Well head looking south



CITY OF SCAPPOOSE

Council Action & Staff Report

Date Submitted: August 29, 2018

Agenda Date Requested: September 4, 2018

To: Scappoose City Council

Through: Michael Sykes, City Manager

From: Kevin Turner, Wastewater Treatment Supervisor

Subject: Wastewater Facility Plan Update, Ordinance No. 873

TYPE OF ACTION REQUESTED:

]	Resolution	[X] Ordinance
Γ		Formal Action	[] Report Only

ISSUE:

The current Wastewater Master Plan was completed in June of 1998 and addresses flows, loads, and process improvements that are no longer accurate to address current and future demands and operational issues at the treatment facility.

ANALYSIS:

In 2015, the City of Scappoose (City) contracted with Keller Associates, Inc. to complete a comprehensive Wastewater Planning Study for both the City's sanitary sewer collection system and Wastewater Treatment Plant (WWTP).

In 2017, the City contracted with Carollo Engineers (Carollo) to prepare a Facilities Plan Update. The purpose of the Facilities Plan Update is to extend the planning horizon to 2042, peer review the Wastewater Planning Study and provide further, more detailed, evaluation of the WWTP and the Wastewater Planning Study's recommendations for future WWTP improvements.

The scope of the Facilities Plan Update is limited and by itself, is not a fully comprehensive facilities plan. Some elements of a typical facilities plan (e.g. service area description, condition assessment, and system background, etc.), were completed as part of the Wastewater Planning Study and are not reevaluated or redeveloped within this Facilities Plan Update. Therefore, a complete submittal includes both this update and the City's Wastewater Planning Study (see Appendix A.) Where there is overlap or inconsistencies between the two documents, the Facilities Plan Update supersedes the Wastewater Planning Study.

The primary objective for the City's wastewater planning was to fulfill the engineering planning document requirements of the Clean Water State Revolving Fund, Oregon Infrastructure Finance Authority, and USDA-Rural Development. Additional objectives for the Facilities Plan Update are to:

- Provide the City with an amended planning document that identifies and details a logical path forward for the WWTP to sustain the City's ability to provide reliable wastewater treatment for the existing population, address current WWTP operational issues, maintain NPDES discharge permit compliance, and accommodate projected growth within the service area over the 20-year planning period,
- Evaluate potential operational changes and facilities improvements that may be necessary to address future, and potentially more stringent discharge limitations. A number of potential regulatory scenarios were evaluated, including the potential for future nitrification requirements and nutrient limits,
- Develop a comprehensive CIP and for the City's wastewater treatment and collection system, and
- Provide a predesign of the WWTP capital improvements that the City will complete in the near-term (first five years).

Based on the recommended CIP in this Facilities Plan Update, the City developed a Financial Plan to establish rates and system development charges as necessary to allow the City to implement the recommended CIP. The Financial Plan is included in Appendix B.

In July of 2018 the complete blended Facility Plan Update document was submitted and approved by Oregon Department of Environmental Quality.

RECOMMENDATION:

Staff recommends that Council adopt the Ordinance as presented.

SUGGESTED MOTION:

I move that Council adopt Ordinance No. 873 as presented.

ATTACHMENTS:

- Ordinance No. 873
- Exhibit A Facilities Plan Update, Final June 2018
- Exhibit B DEQ Letter Facilities Plan Approval

DRAFT

ORDINANCE NO. 873

AN ORDINANCE UPDATING THE CITY OF SCAPPOOSE WASTEWATER FACILITY PLAN

THE CITY OF SCAPPOOSE ORDAINS AS FOLLOWS:

Section 1. The City of Scappoose Wastewater Facility Plan Update, June 2018 is updated as provided in Exhibit A.

	CITY OF SCAPPOOSE	, OREGON
	Scott Burge, Mayor	
First Reading:		
Second Reading:		
Attest:		
Susan M. Reeves, MMC City Recorder		
	DRAF*	





City of Scappoose Facilities Plan Update

FACILITIES PLAN UPDATE

FINAL | June 2018





City of Scappoose Facilities Plan Update

FACILITIES PLAN UPDATE

FINAL | June 2018

OSTERED PROFESSO OSTERED PROFESSO 76549

PREGON

OREGON

EXPIRES: 12/31/19

Contents

EXECUTIVE SUMMARY

ES.1 Project Objectives	ES-1
ES.2 Basis of Planning	ES-5
ES.3 Collection System Flow Monitoring Data Review	ES-7
ES.4 Existing Wastewater Treatment Facility	ES-8
ES.5 Recommended WWTP Improvements	ES-13
ES.5.1 Capital Improvement Plan (CIP)	ES-14
Chapter 1 - BASIS OF PLANNING	
1.1 Purpose	1-1
1.2 Background	1-1
1.3 Current Discharge Permit Requirements	1-1
1.4 Expected Future Discharge Permit	1-2
1.5 Flow and Load Analysis	1-3
1.5.1 Definitions	1-3
1.5.2 Rainfall Records	1-4
1.5.3 Population	1-4
1.5.4 Industrial Contributions	1-5
1.5.5 Historical Flow Analysis	1-5
1.5.6 Historical Wastewater Loads Analysis	1-11
1.5.7 Flow Projections	1-14
1.5.8 Load Projections	1-14
Chapter 2 - FLOW MONITORING DATA REVIEW AND FLOW PROJECTIONS	
2.1 Introduction	2-1
2.2 Study Area	2-1
2.3 System Description	2-1
2.4 Sewer Collection System Flows	2-2
2.4.1 Dry Weather Flow Components	2-2
2.4.2 Wet Weather Flow	2-8



2.5 Data Overview for Flow Projections	2-11
2.6 Flow Monitoring Data Quality Review	2-11
2.6.1 West side of the Highway	2-13
2.6.2 East side of the Highway	2-15
2.6.3 Flow Monitoring Data Review Summary and Recommendations:	2-17
2.7 Zoning and Land Use Data	2-18
2.7.1 Zoning and Existing Served and Connected Areas	2-19
2.7.2 Future Land Use	2-23
2.7.3 Zoning and Land Use Correspondence	2-23
2.8 Wastewater Flow Coefficients	2-24
2.9 Collection System Flow Projections	2-25
2.9.1 Dry Weather Flows Projections	2-25
2.9.2 Wet Weather Flows Projections	2-29
2.9.3 Existing and Projected Flow Summary	2-29
Chapter 3 - HYDRAULIC AND PROCESS CAPACITY ANALYSIS	
3.1 Introduction	3-1
3.2 Current Plant Operations	3-1
3.3 Plant Hydraulic Capacity Analysis	3-8
3.3.1 Modeling Assumptions	3-8
3.3.2 Results	3-9
3.4 Plant Process Capacity Analysis	3-11
3.4.1 Model Calibration	3-11
3.4.2 Basis of Planning	3-15
3.4.3 Capacity of System for Future Loads	3-19
3.5 Solids Process Capacity Analysis	3-20
Chapter 4 - RECOMMENDED IMPROVEMENTS	
4.1 Objective	4-1
4.2 Cost Estimation	4-1
4.3 Liquid Stream Treatment	4-2
4.3.1 Headworks	4-2



4.3.2 Sec	ondary Clarification	4-3
4.3.3 UV	Disinfection	4-3
4.3.4 Effl	uent Pumping and Outfall	4-5
4.4 Solids Str	eam Treatment	4-5
4.4.1 Dig	estion	4-5
4.4.2 Dev	vatering	4-5
Chapter 5 -	IMPLEMENTATION PLAN	
5.1 Introduct	ion	5-1
5.2 Implemen	ntation Program and Schedule	5-1
5.3 Phase 1 Ir	nprovements	5-3
5.3.1 Spr	ing Lake Lift Station Improvements	5-3
5.3.2 UV	Disinfection Equipment Replacement	5-3
5.3.3 Hyd	Iraulic Improvements	5-4
5.3.4 Sec	ondary Clarifier and RAS/WAS Pumping Upgrades	5-4
5.3.5 Life	Safety and Condition Improvements at the Aerobic Digester	5-5
5.4 Phase 2 Ir	mprovements	5-6
5.4.1 Hea	ndworks and Influent Pumping	5-6
5.4.2 Ope	erational Improvements	5-11
5.5 Phase 3 Ir	nprovements	5-11
5.5.1 UV	Disinfection Channel Addition	5-11
5.5.2 Effl	uent Pumping Station Improvements	5-11
5.5.3 Par	allel Outfall	5-12
5.6 Recommo	ended Site Layout	5-12
Appendi	ces	
Appendix A	Scappoose Wastewater Facilities Draft Planning Study (Keller Associates, December 2016)	
Appendix B	Financial Plan	
Appendix C	Current NPDFS Permit	



Tables Table ES.1 Wastewater Flow and Load Projections ES-6 Table ES.2 ES-7 **Anticipated NPDES Permit Limits** Table ES.3 ES-7 Meter Data Review Results Summary Table ES.4 Hydraulic Capacity of Existing Processes ES-13 Table ES.5 ES-14 Mark up Factors Used in Developing Cost Estimates for Alternatives Table ES.6 Implementation Program and Phasing Schedule ES-15 1-2 Table 1.1 Existing NPDES Permit Limits 1-3 Table 1.2 Expected Future NPDES Permit Limits Table 1.3 Historical Base Wastewater Flows 1-6 Table 1.4 Historical Average Dry and Wet Weather Flows 1-6 Table 1.5 Historical Maximum Month Dry and Wet Weather Flows 1-9 Table 1.6 Current Wastewater Flows 1-11 1-11 Table 1.7 Current BOD Loading 1-13 Table 1.8 Current TSS Loading Table 1.9 Current Ammonia Loading 1-13 Table 1.10 Wastewater Flow Projections 1-14 1-14 Table 1.11 Per Capita Loading 1-15 Table 1.12 Load Projections Table 1.13 Core Design and Reliability/Redundancy Criteria 1-15 Table 2.1 Pipe Inventory 2-2 2-3 Table 2.2 Lift Station Inventory Table 2.3 Meter Data Review Results Summary 2-18 2-19 Table 2.4 Existing Zoning Acreage Table 2.5 Future Land Use Acreage 2-23 2-23 Table 2.6 Zoning vs. Land Use Correspondence Table 2.7 Land Use Flow Factors Development 2-25



2-29

Table 2.8 Largest Customer Projected ADWF

Table 2.9 Pro	ojected Flows Summary	2-30	
Table 3.1 Des	sign Data for the Existing WWTP	3-5	
Table 3.2 Flo	w Capacity of Existing Processes	3-9	
Table 3.3 Mo	del Influent Characteristics	3-11	
Table 3.4 Eff	luent Quality Goals	3-15	
Table 3.5 Ger	neral Secondary Process Design Criteria	3-19	
Table 4.1 Ma	rk up Factors Used in Developing Cost Estimates for Alternatives	4-1	
Table 4.2 Pre	sent Worth Comparison of Headworks Alternatives	4-3	
Table 4.3 Pre	sent Worth Comparison of UV Disinfection Alternatives	4-4	
Table 5.1 Imp	plementation Program and Phasing Schedule	5-2	
Table 5.2 UV	Process Design Criteria	5-4	
Table 5.3 Sub	omersible versus Dry Pit-Type Influent Pumps	5-10	
Table 5.4 Rec	ommended Headworks Design Criteria	5-10	
Figures			
Figure ES.1	WWTP Study Area	ES-3	
Figure ES.2	Projected Population Growth for Scappoose UGB	ES-5	
Figure ES.3	Flow Monitoring Locations	ES-9	
Figure ES.4	Current Plant Process Flow Diagram	ES-11	
Figure ES.5	Recommended Site Layout	ES-16	
Figure 1.1 Pro	ojected Population Growth for Scappoose UGB	1-4	
Figure 1.2 His	storical Base Wastewater Flow	1-5	
Figure 1.3 Av	rerage Dry Weather Flows	1-7	
Figure 1.4 Av	verage Wet Weather Flows	1-7	
Figure 1.5 Ma	aximum Month Flows	1-8	
Figure 1.6 Pe	Figure 1.6 Peak Day Flows 1-10		
Figure 1.7 PIF	Figure 1.7 PIF using DEQ Methodology 1-10		



Figure 1.8 Historical BOD Loading	1-12
Figure 1.9 Historical TSS Loading	1-12
Figure 1.10 Historical Ammonia Loading	1-13
Figure 2.1 Study Area	2-5
Figure 2.2 Typical Wastewater Flow Components	2-7
Figure 2.3 Typical Sources of Inflow and Infiltration	2-9
Figure 2.4 Typical Effects of Inflow and Infiltration	2-10
Figure 2.5 Flow Monitoring Locations	2-12
Figure 2.6 Flow Meter 1 Data Analysis	2-14
Figure 2.7 Flow Meter 2 Data Analysis	2-15
Figure 2.8 Flow Meters 3 and 4 Data Analysis	2-16
Figure 2.9 Flow Meter 5 Data Analysis	2-17
Figure 2.10 Existing Zoning	2-21
Figure 2.11 Future Land Use	2-27
Figure 2.12 ADWF and PWWF Projections	2-30
Figure 3.1 Current Plant Process Flow Diagram	3-3
Figure 3.2 Plant Measured and Model Calculated Influent BOD Concentration	3-12
Figure 3.3 Plant Measured and Model Calculated Influent TSS Concentration	3-13
Figure 3.4 Plant Measured and Model Calculated MLSS Concentration	3-13
Figure 3.5 Plant Measured and Model Calculated WAS Load	3-14
Figure 3.6 Plant Measured and Model Calculated Secondary Effluent Ammonia Concentration	3-14
Figure 3.7 Model Predicted Secondary Effluent Ammonia Concentrations versus SRT	3-16
Figure 3.8 Plant Measured SRT and Secondary Effluent Ammonia Concentrations	3-16
Figure 3.9 Plant Measured SVI	3-17
Figure 3.10 State Point Analysis for 2 Secondary Clarifiers in Service during Peak Hour Flow	3-18
Figure 3.11 State Point Analysis for 1 Secondary Clarifier in Service with 3 mgd Peak Flow	3-18
Figure 3.12 State Point Analysis for 3 Secondary Clarifiers in Service during Peak Hour Flow	3-19
Figure 3.13 Model Predicted Future MLSS Concentration	3-20



Figure 5.1 Headworks Process Flow Schematic	5-6
Figure 5.2 Typical Multi-Rake Bar Screen	5-7
Figure 5.3 Shaftless Screw Conveyor	5-8
Figure 5.4 Screenings Washer/Compactor	5-9
Figure 5.5 Recommended Site Layout	5-13





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

In 2015, the City of Scappoose (City) contracted with Keller Associates, Inc. to complete a comprehensive Wastewater Planning Study for both the City's sanitary sewer collection system and Wastewater Treatment Plant (WWTP). The study prepared by Keller Associates is included in Appendix A.

In 2017, the City contracted with Carollo Engineers (Carollo) to prepare this Facilities Plan Update. The purpose of this Facilities Plan Update is to extend the planning horizon to 2042, peer review the Wastewater Planning Study and provide further, more detailed, evaluation of the WWTP and the Wastewater Planning Study's recommendations for future WWTP improvements.

The scope of this Facilities Plan Update is limited and by itself, is not a fully comprehensive facilities plan. Some elements of a typical facilities plan (e.g. service area description, condition assessment, and system background, etc.), were completed as part of the Wastewater Planning Study and are not reevaluated or redeveloped within this Facilities Plan Update. Therefore, a complete submittal includes both this update and the City's Wastewater Planning Study (see Appendix A.) Where there is overlap or inconsistencies between the two documents, this Facilities Plan Update supersedes the Wastewater Planning Study.

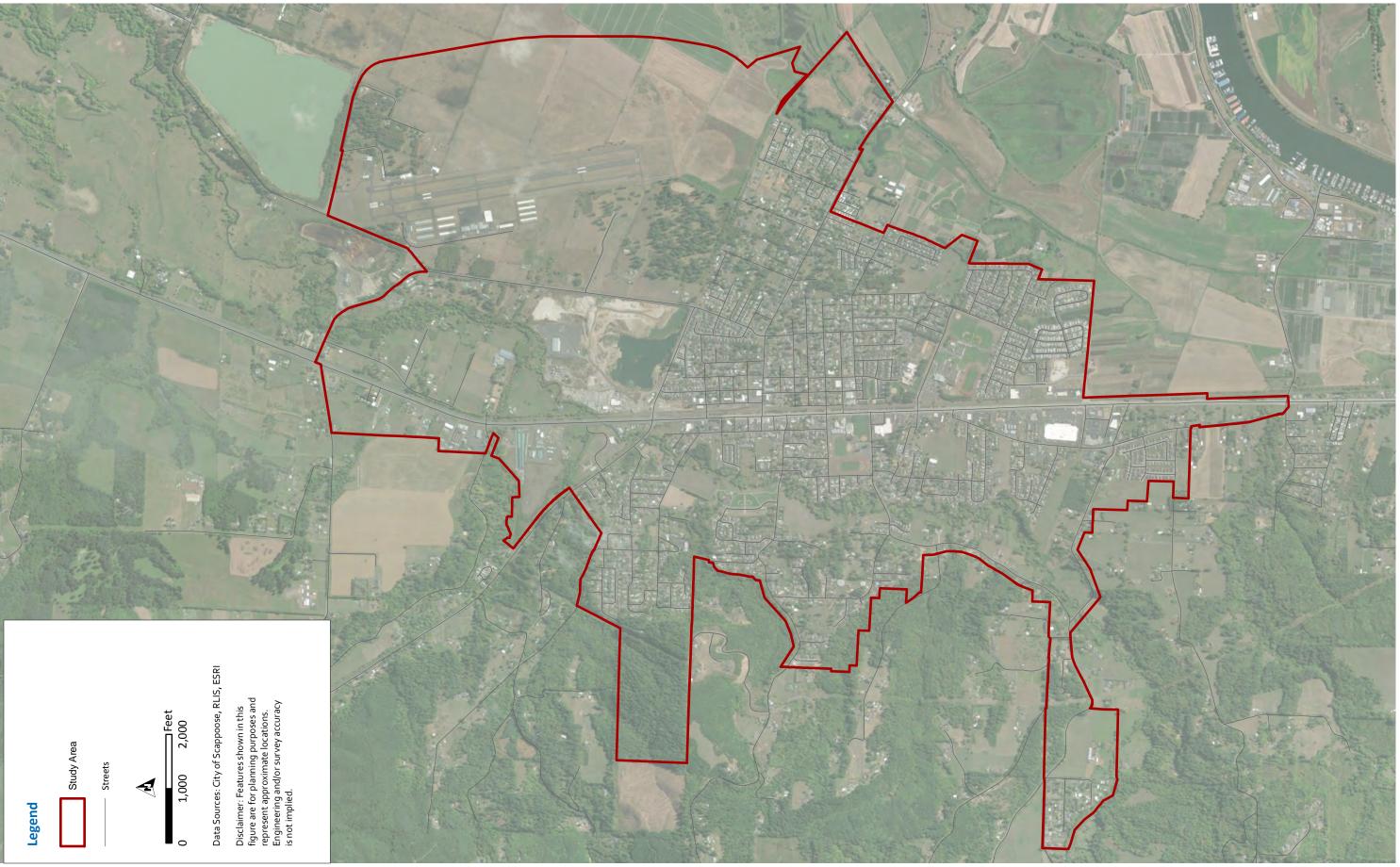
The primary objective for the City's wastewater planning was to fulfill the engineering planning document requirements of the Clean Water State Revolving Fund, Oregon Infrastructure Finance Authority, and USDA-Rural Development. Additional objectives for this Facilities Plan Update are to:

- Provide the City with an amended planning document that identifies and details a
 logical path forward for the WWTP to sustain the City's ability to provide reliable
 wastewater treatment for the existing population, address current WWTP operational
 issues, maintain NPDES discharge permit compliance, and accommodate projected
 growth within the service area over the 20-year planning period,
- Evaluate potential operational changes and facilities improvements that may be
 necessary to address future, and potentially more stringent discharge limitations. A
 number of potential regulatory scenarios were evaluated, including the potential for
 future nitrification requirements and nutrient limits,
- Develop a comprehensive CIP and for the City's wastewater treatment and collection system, and
- Provide a predesign of the WWTP capital improvements that the City will complete in the near-term (first five years).

Based on the recommended CIP in this Facilities Plan Update, the City developed a Financial Plan to establish rates and system development charges as necessary to allow the City to implement the recommended CIP. The Financial Plan is included in Appendix B.









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 Facilities 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.

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 2042 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.

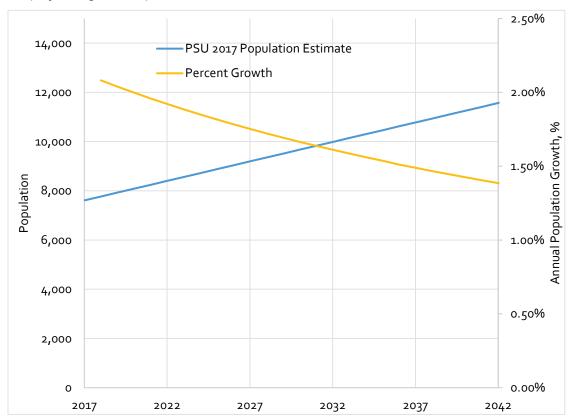


Figure ES.2 Projected Population Growth for Scappoose UGB



Table ES.1 Wastewater Flow and Load Projections

Parameter	Current	2042
Population	7,610	11,568
FLOW SUMMARY		
BWF, mgd	0.607	0.93
ADWF, mgd	0.646	0.99
AWWF, mgd	0.878	1.35
MMDWF, mgd	0.871	1.33
MMWWF, mgd	1.20	1.83
PDF, mgd	2.77	4.23
PIF, mgd	3.96	6.05
LOAD SUMMARY		
BOD ₅		
Average Annual Load, ppd	1,430	2,200
Maximum Month Load, ppd	2,180	3,350
TSS		
Average Annual Load, ppd	1,670	2,600
Maximum Month Load, ppd	2,540	3,900
Ammonia		
Average Annual Load, ppd	264	400
Maximum Month Load, ppd	450	700

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 ⁽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	< 5.0 mg/L (Summer)		

Notes:

- (1) $BOD_5 = 5$ -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 A 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 aeration basin has sufficient capacity but lacks redundancy. However, there is sufficient redundancy in the aeration equipment and limited potential for the basin to break down. Therefore a redundant basin is included beyond the planning period via space planning.

The secondary clarifiers do not have sufficient hydraulic capacity to meet projected 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. Hence a new secondary clarifier is recommended.



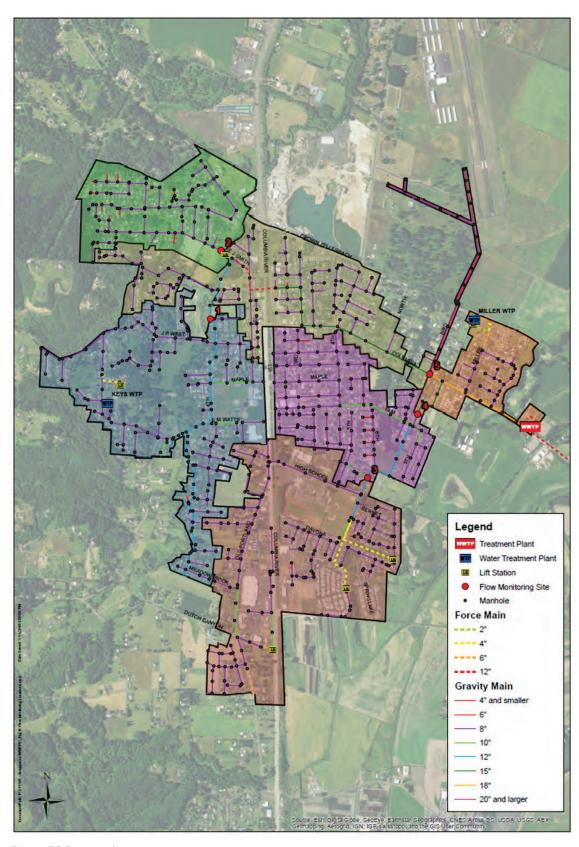


Figure ES.3 Flow Monitoring Locations





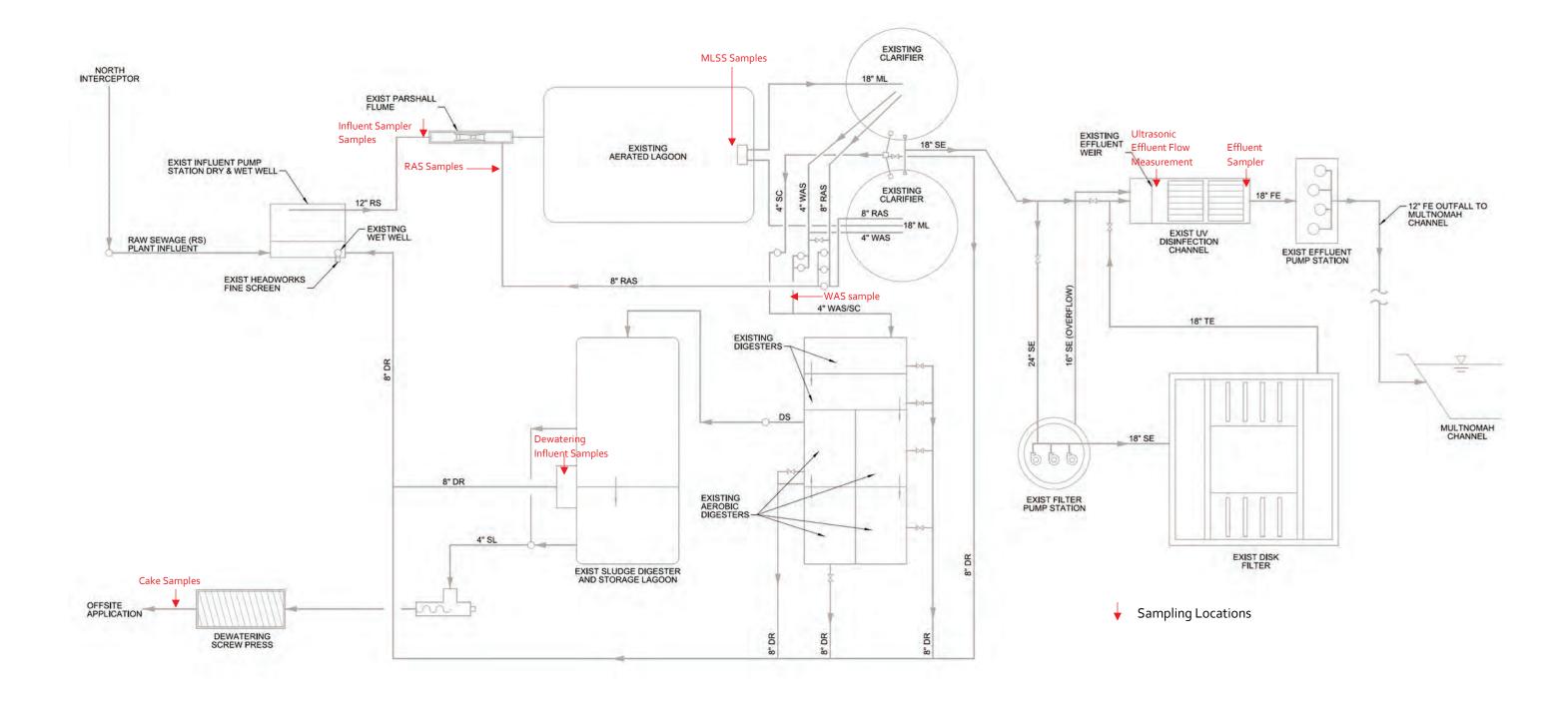


Figure ES.4 Current Plant Process Flow Diagram



Table ES.4 Hydraulic Capacity of Existing Processes

Process Area	Existing Hydraulic Capacity, mgd	Required Hydraulic Capacity, mgd ⁽¹⁾
Influent Screening	4.0 mgd	6.05 mgd
Influent Pumping Station	3.5 mgd	6.05 mgd
Aeration Basin	7.0 mgd ²	6.05 mgd
Secondary Clarifiers	4.7 mgd	6.05 mgd
Tertiary Filtration System		
- Intermediate Pumping Station	4 mgd	3.6 mgd ⁴
- Filtration	2.4 mgd	2.4 mgd⁵
UV Disinfection	5.6 mgd ³	6.05 mgd
Effluent Pump Station	4.4 mgd	6.05 mgd
Outfall	4.3 mgd	6.05 mgd

Notes:

- (1) Through planning period (year 2042)
- (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 2042 Summer Peak Instantaneous Flow
- (5) Year 2042 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 nearing their capacity with existing loading conditions. Additional capacity must be added to provide reliable treatment.

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

ltem	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

			Construci	Construction Year		
ltem Description	Description	Phase 1 2018 – 2021	Phase 2 2021 - 2023	Phase 3 2023 - 2026	Phase 4 2026 – 2028	Total
Phase 1 Improvements						
Spring Lake Lift Station	Replacement of pumps and corroded piping; addition of valve vault to measure flow.	\$271, 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				\$6,430,600
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		
Effluent Pump Station	Replacement of pumps, modification of skylights, addition of flow meter, electrical improvements.			\$536,600		\$4,393,200
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



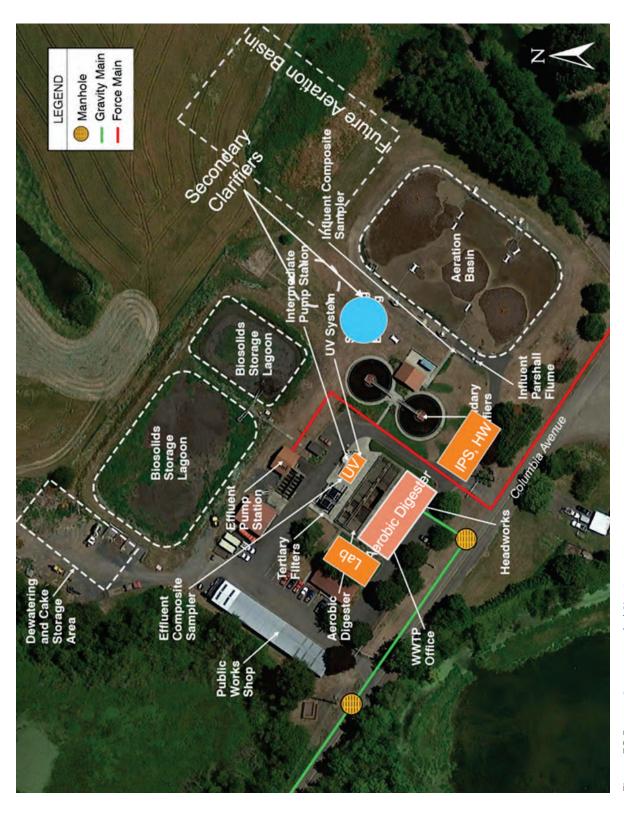


Figure ES.5 Recommended Site Layout

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 a
 logical path forward for the WWTP to sustain the City's ability to provide reliable
 wastewater treatment for the existing population, address current WWTP operational
 issues, maintain NPDES discharge permit compliance, and accommodate projected
 growth within the service area over the 20-year planning period,
- Evaluate potential operational changes and facilities improvements that may be
 necessary to address future, and potentially more stringent discharge limitations. A
 number of potential regulatory scenarios were evaluated, including the potential for
 future nitrification requirements and nutrient limits,
- Develop a comprehensive CIP and for the City's wastewater treatment and collection system, and
- Provide a predesign of the WWTP capital improvements that the City will 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		6.0 and 9.0
E.coli Bacteria	126/100 mL		
Total Phosphorus	No limit	No limit	No limit
Ammonia Notes:	No limit	No limit	No limit

- (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 2042 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.

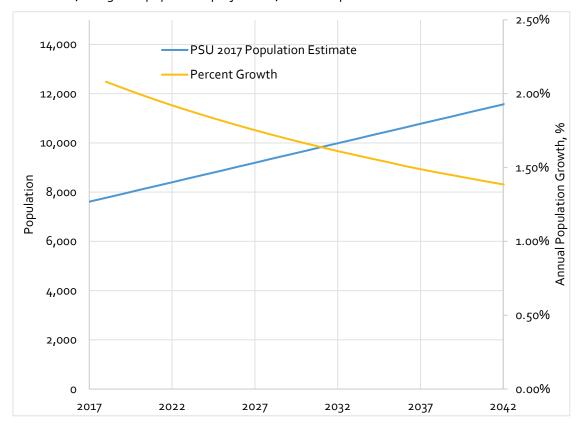


Figure 1.1 Projected Population Growth for Scappoose UGB



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 influent 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.

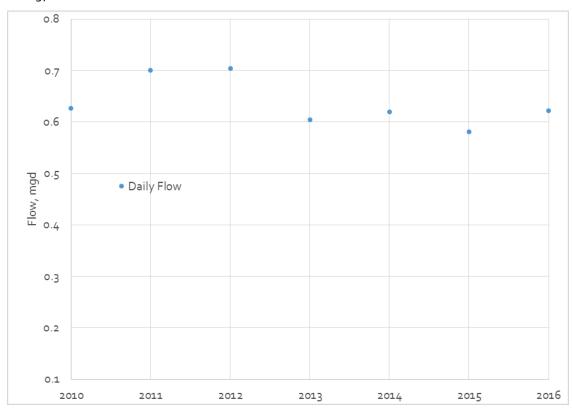


Figure 1.2 Historical Base Wastewater Flow



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 Ave	rage	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
	2010	14.49	0.677
	2011	7.95	0.695
	2012	12.18	0.718
Dry Season Wet Season	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
	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



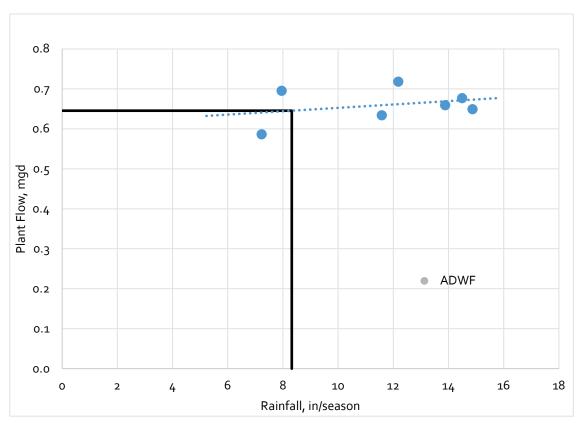


Figure 1.3 Average Dry Weather Flows

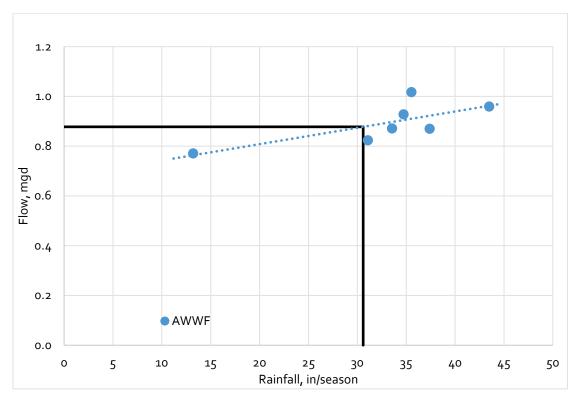


Figure 1.4 Average Wet Weather Flows



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.

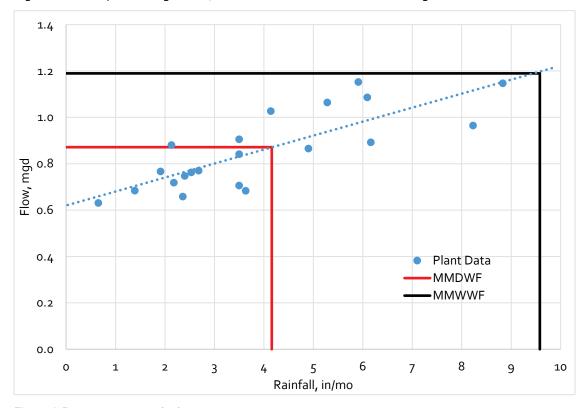


Figure 1.5 Maximum Month Flows

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.



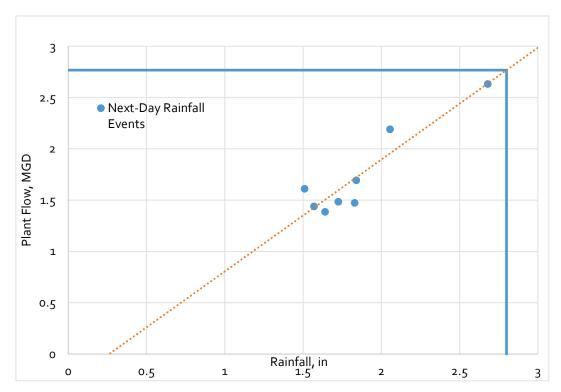


Figure 1.6 Peak Day Flows

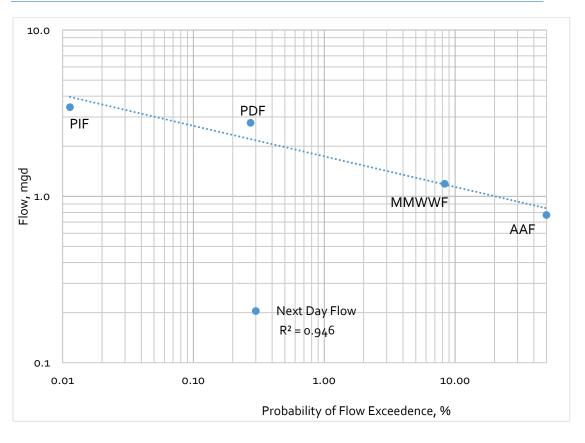


Figure 1.7 PIF using DEQ Methodology



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 influent 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 influent 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.



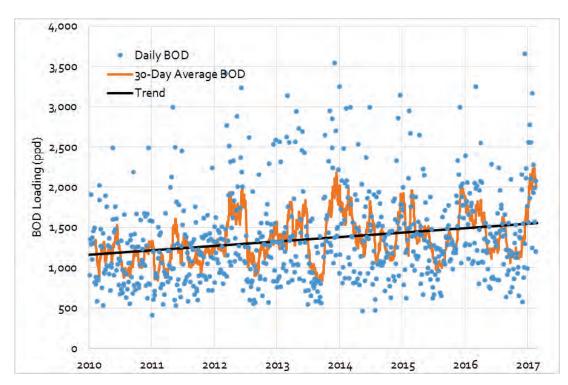


Figure 1.8 Historical BOD Loading

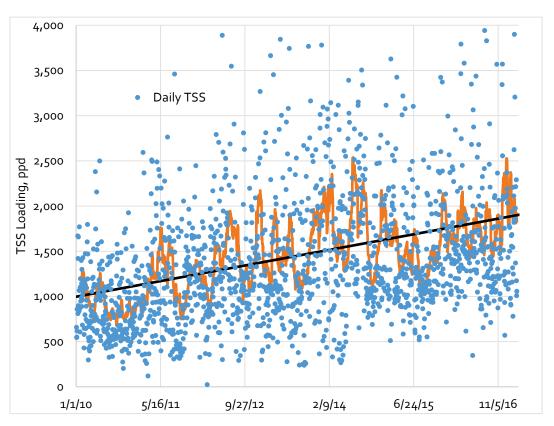


Figure 1.9 Historical TSS Loading



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.

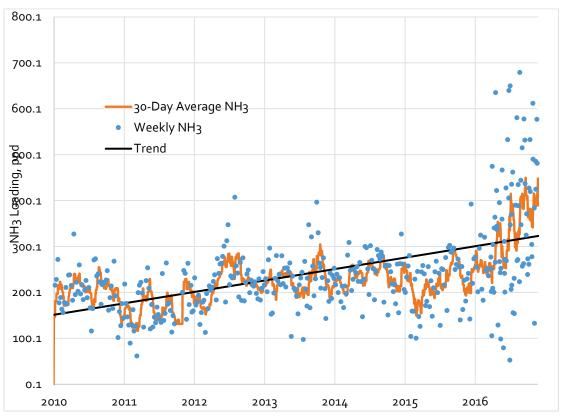


Figure 1.10 Historical Ammonia Loading

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 2042. 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	2042
Population	7,610	11,568
BWF, mgd	0.607	0.93
ADWF, mgd	0.646	0.99
AWWF, mgd	0.878	1.35
MMDWF, mgd	0.871	1.33
MMWWF, mgd	1.20	1.83
PDF, mgd	2.77	4.23
PIF, mgd	3.96	6.05

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 BOD₅ 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	2042
Population	7,610	11,568
BOD₅		
Average Annual Load, ppd	1,430	2,200
Maximum Month Load, ppd	2,180	3,350
TSS		
Average Annual Load, ppd	1,670	2,600
Maximum Month Load, ppd	2,540	3,900
Ammonia		
Average Annual Load, ppd	264	400
Maximum Month Load, ppd	450	700

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 2042 Capacity = 6.05 mgd Pump current low flows without cycling 	 Capacity for PIF with largest unit out of service
Screening	PIF	 Required Year 2042 Capacity = 6.05 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 2042 Capacity = 6.05 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 2042 Capacity = 6.05 mgdSOR = 1500 gal/SF/day	 Largest clarifier out of service for ADW conditions
Tertiary Treatment	MMF (summer)	 Required Year 2042 Capacity = 1.33 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 2042 Capacity = 6.05 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



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 2042, and build condition, estimated for planning year 2067.

This Chapter must be reviewed in conjunction with the Scappoose Wastewater Facilities Draft Planning Study (Keller Associates, December 2016) for the existing system condition assessment.

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 2042).

2.3 System Description

Detailed system description and existing condition assessment is presented in the Scappoose Wastewater Facilities Draft Planning Study (Keller Associates, December 2016) (Appendix A). A summary of the existing system is presented here.

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 WWTP. The pipelines range from 4 to 21 inches in diameter. A summary of pipe type and size summary is presented in Table 2.1 and lift station inventory is presented in Table 2.2.



Table 2.1 Pipe Inventory

Pipe		Pipe N	Naterial Leng	ths (ft)		Total by	% of
Diameter (in)	Concrete	PVC	Cast Iron	Ductile Iron	Unknown	Diameter (ft)	Total
4"		250				250	0%
6"	1,272	1,847		89	303	3,500	2%
8"	62,086	69,104	233	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

2.4 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.

2.4.1 Dry Weather Flow Components

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

2.4.1.1 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.



Table 2.2 Lift Station Inventory

Lift Station	Smith Road	Spring Lake	Kevs Landing	Highway 30	Seven Oaks
LIFT STATION		-		<u> </u>	
Туре	Wet-well, Dry-well, triplex pump station	Wet well, surface mounted, duplex pump station	Wet well, surface mounted, duplex pump station	Wet well, surface mounted, duplex pump station	Wet well, surface mounted, duplex pump station
Pump Type	Vertical, variable speed, non-clog centrifugal	Horizontal, constant speed, self-priming centrifugal	Horizontal, constant speed, self-priming centrifugal	Horizontal, constant speed, self-priming centrifugal	Horizontal, constant speed, self-priming centrifugal
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 (a) approx. 46 ft TDH (with 13 ft suction lift)	Each Pump: 150 gpm (a) approx. 42 ft TDH (with 19 ft suction lift)	Each Pump: 150 gpm (a) approx. 45 ft TDH (with 16 ft suction lift)
Pump HP (each)	15 HP @ 1200 rpm (460V, 60 Hz, 3 ph)	3 HP @ 1150 rpm (230V, 60 Hz, 1 ph)	5 HP @ 1750 rpm (230V, 60 Hz, 3 ph)	7.5 HP @ 1800 rpm (230V, 60 Hz, 3 ph)	7.5 HP @ 1750 rpm (230V, 60 Hz, 3 ph)
Level Control Type	Pressure level transducer in wet well	Four mercury level sensors	Four mercury level sensors	Four mercury level sensors	Four mercury level sensors
Overflow Point	At existing manhole west of pump station	At pump station	At pump station	At pump station	At manhole just north of pump station
Overflow Discharge	To creek west of pump station	To Spring Lake	To residences near pump station	To storm water 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	Auto dialer and red light outside	Auto dialer and red light outside	Auto dialer and red light outside	Auto dialer and red light outside	Auto dialer and red light outside

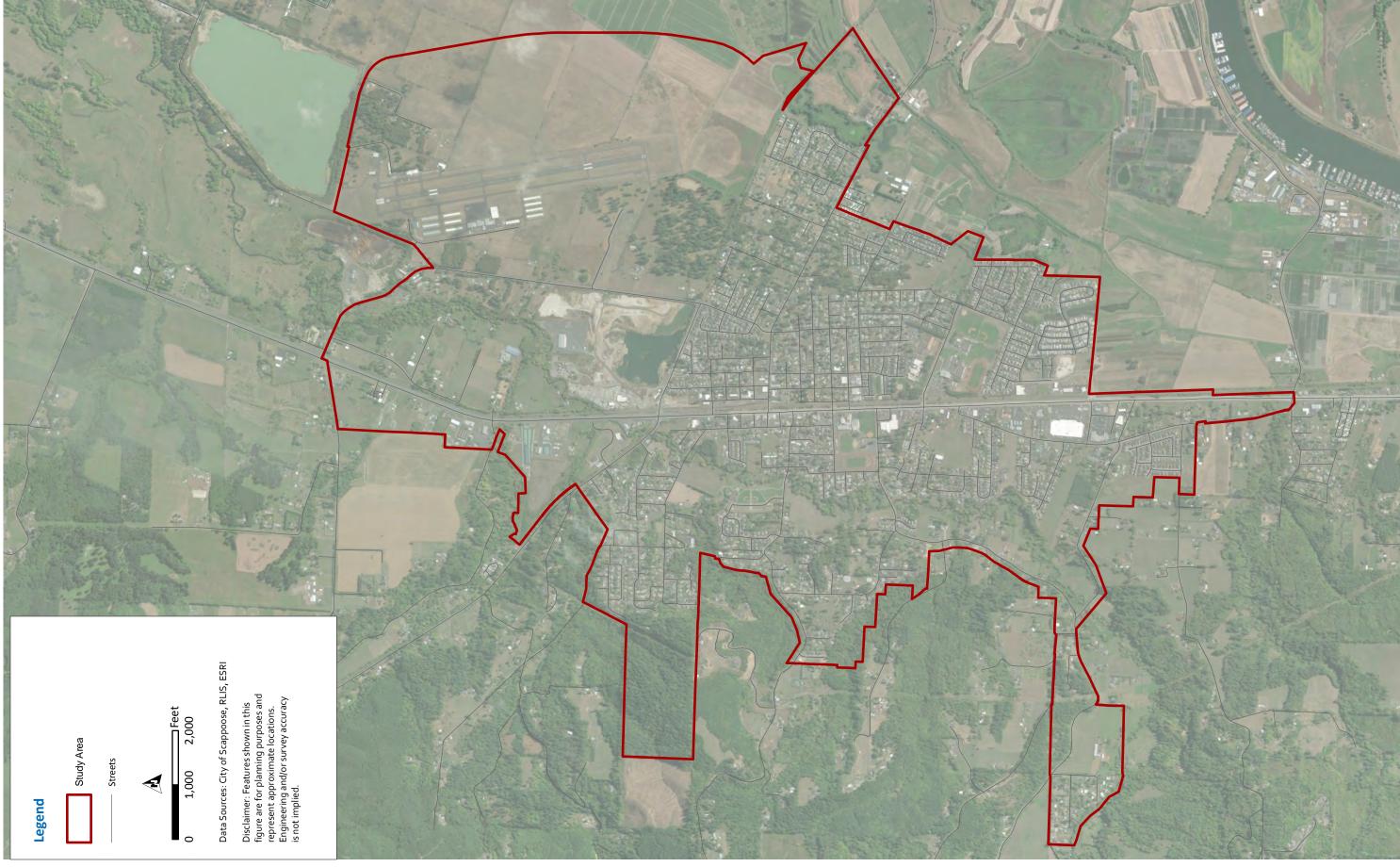


Table 2.2 Lift Station Inventory (Continued)

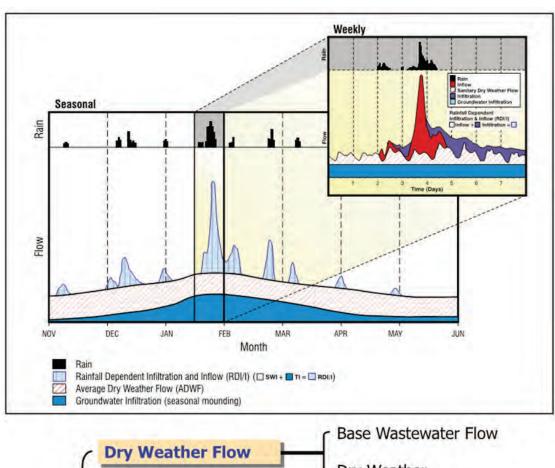
I able 2.2 FILL Station Inventory (Continued)	icoly (collelloed)				
Lift Station	Smith Road	Spring Lake	Keys Landing	Highway 30	Seven Oaks
Originally Constructed	1972	1993	1996	1996	1996
Year(s) Upgraded	2001	2001 and 2003	ΑN	AN	NA
Wet Well Diameter (ft)	9	9	9	8	7
Wet Well Net Storage (gal)	634	528	528	2,237	1,315
FORCE MAIN					
Length, ft	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	Yes
Discharge Location	MH at intersection of Burlington Northern RR and Laurel Street	MH at 6 th Street and Seven Oaks Drive	MH on SW Keys Road and end of SW Keys Landing Way	MH on SW Dutch Canyon Road off Columbia River Hwy	MH at 6 th 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 rated in O&M manual.









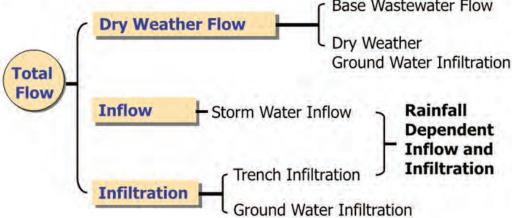


Figure 2.2 Typical Wastewater Flow Components

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

2.4.1.2 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.



2.4.1.3 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.

2.4.2 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.

2.4.2.1 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.

2.4.2.2 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



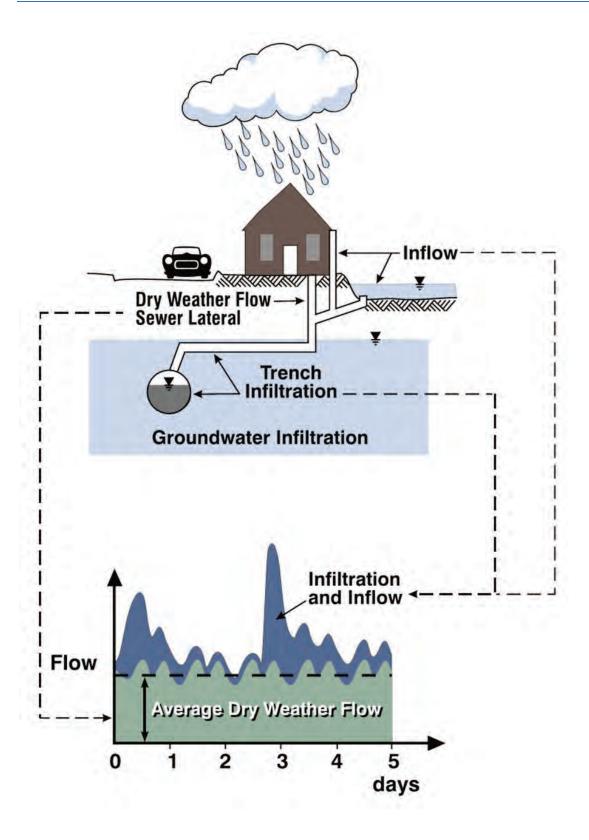


Figure 2.4 Typical Effects of Inflow and Infiltration

2.5 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.

2.6 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.



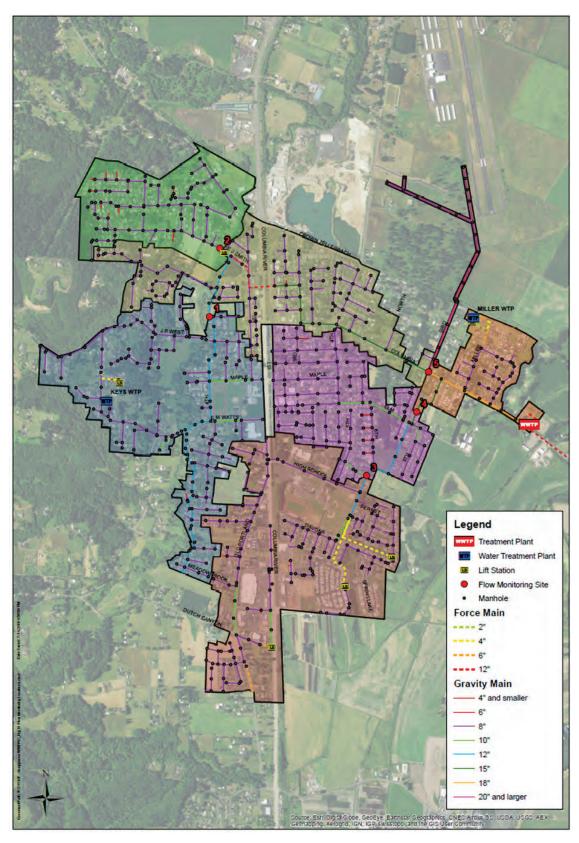


Figure 2.5 Flow Monitoring Locations



2.6.1 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).

2.6.1.1 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.

2.6.1.2 Flow Meter 2

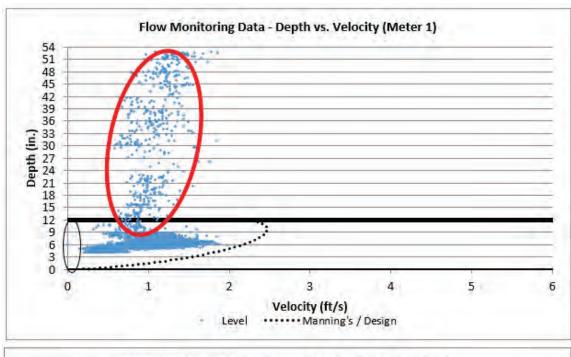
The following observations and conclusion were made from the data analysis for Meter 2. Figure 2.7 shows both scattergraph (bottom) and flow (top) graphs for this meter.

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.





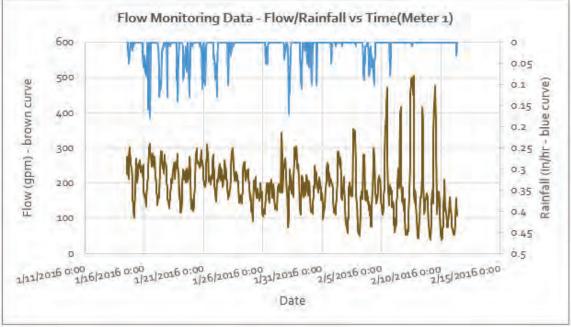
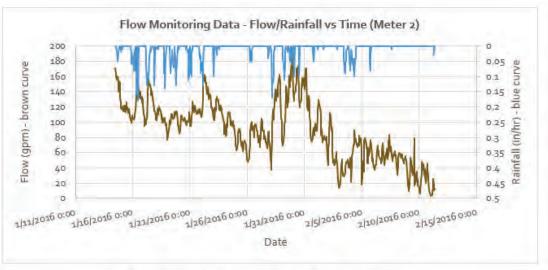


Figure 2.6 Flow Meter 1 Data Analysis



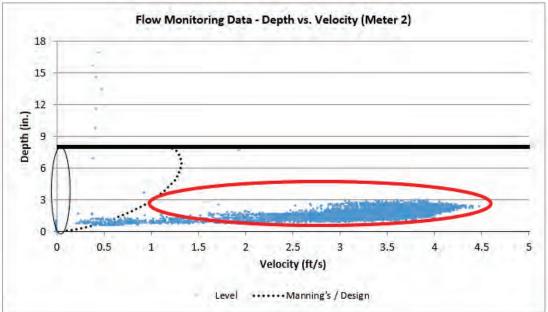


Figure 2.7 Flow Meter 2 Data Analysis

2.6.2 East side of the Highway

2.6.2.1 Flow Meters 3 and 4

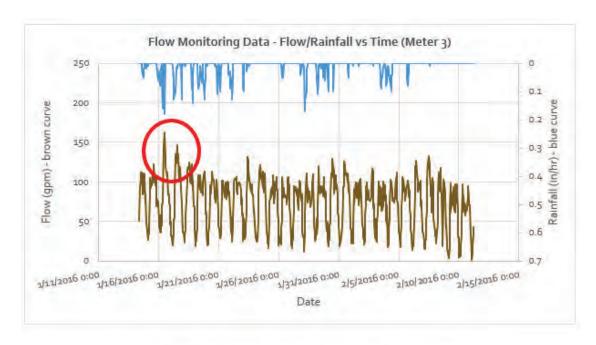
The following observations were made from the data analysis for both Meters 3 and 4. Figure 2.8 shows the flow data graph for Meter 3 at the top and flow data for Meter 4 at the bottom.

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.



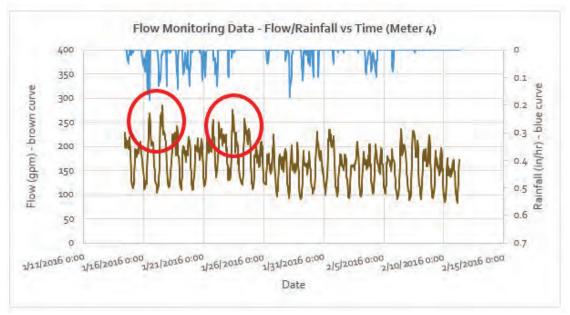


Figure 2.8 Flow Meters 3 and 4 Data Analysis

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.

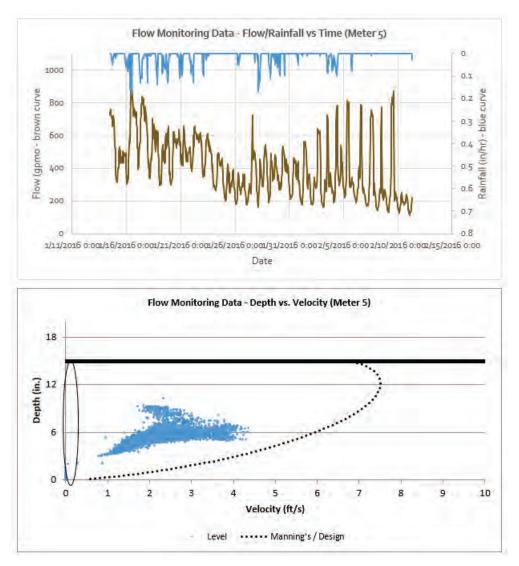


Figure 2.9 Flow Meter 5 Data Analysis

2.6.3 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 2.3 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 2.3 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.

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.

The City plans to perform additional flow monitoring work during 2018-2019 wet weather season. The collection system recommended plan will be completed following the results of flow monitoring.

2.7 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.

2.7.1 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 2.4 provides a summary of the land use zoning categories and acreages for the existing service area.

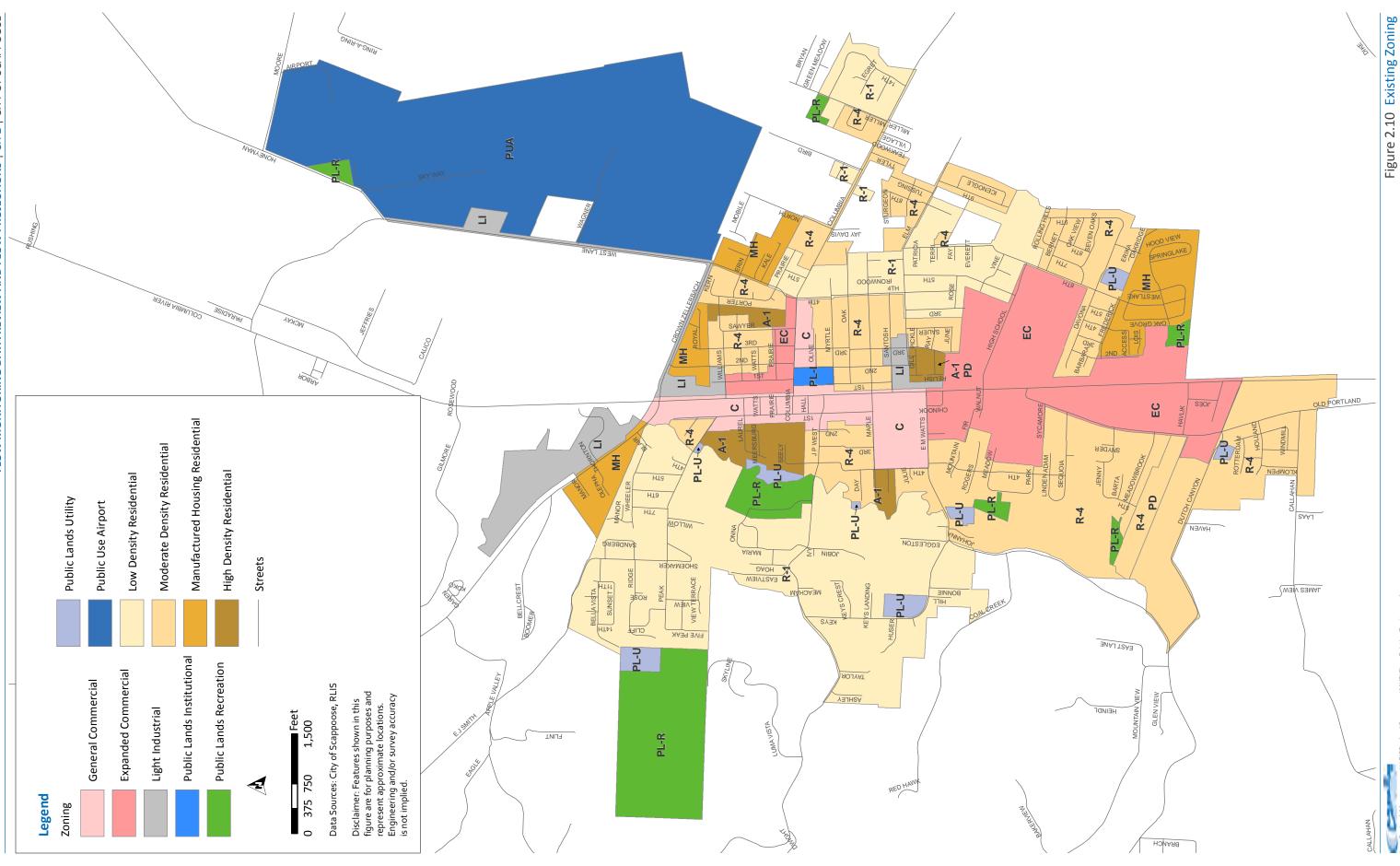
Table 2.4 Existing Zoning Acreage

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.









2.7.2 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 2.5 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 2.5 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

2.7.3 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 2.6 summarizes the correlation between zoning and land use, and will be used to develop wastewater flow coefficients (see Section 7.0).

Table 2.6 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 2.6 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

2.8 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 2.7 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 2.7 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 ADV	0.660	
		Percent Difference	0.26%

2.9 Collection System Flow Projections

2.9.1 Dry Weather Flows Projections

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

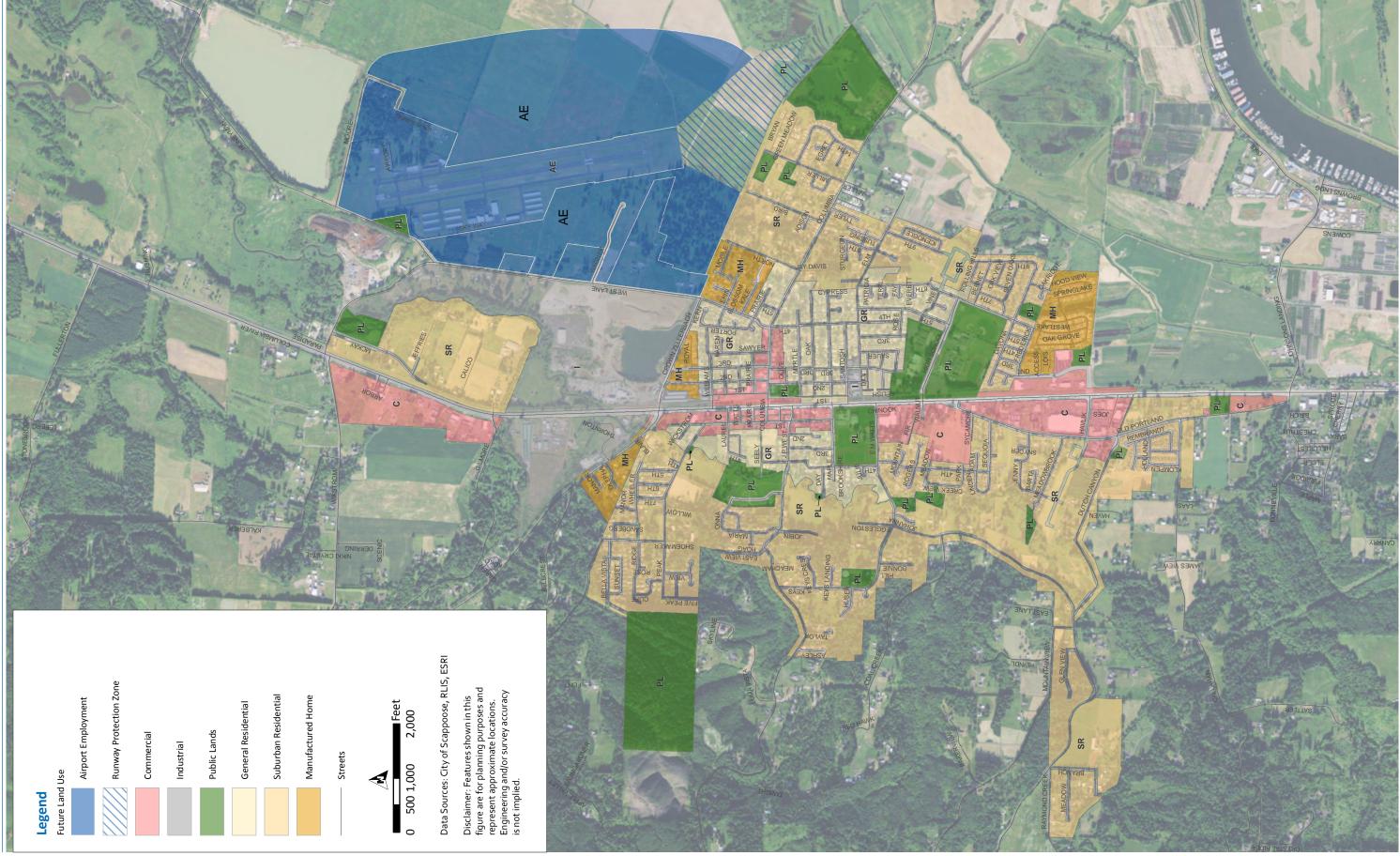
2.9.1.1 Land Use Based Flow Projections

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 2042. 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 and then decreases to 1.2 percent after that.









2.9.1.2 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 2.8.

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 2.8 summarizes the largest customers and corresponding projected ADWFs. Total ADWF from these customers is estimated at 0.14 mgd.

Table 2.8 Largest Customer Projected ADWF

Customer #	Large Customer	# of student/ employee/		Flo	ow Unit	ADWF (gpd)	ADWF (gpm)
1.	Community College	500	Students (next couple years)	15	gpd/per student	7,500	5
		1500	Students (by 2042)	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

2.9.2 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.

2.9.3 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 2.9 summarizes flows under existing



condition, planning year 2042, and build-out condition for the Study Area (i.e. planning year 2067), which are illustrated in Figure 2.12.

Table 2.9 Projected Flows Summary

Flow Condition	Existing Condition	Year 2042	Year 2067 (build-out)
ADWF (w/o largest customer) (mgd)	0.66	0.93	1.14
Largest Customers ADWF (mgd)	0.00	0.14	0.14
Total ADWF (mgd)	0.66	1.07	1.28
Total PWWF (mgd)	3.97	6.03	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 along with current sampling locations. Influent flow measurement is recorded at the parshall flume entering the aeration basin and the effluent flow is measured at the effluent V-notch weir at the UV basin. Table 3.1 summarizes the design flows, loadings, design effluent requirements, and design data for the treatment units and major equipment at 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.



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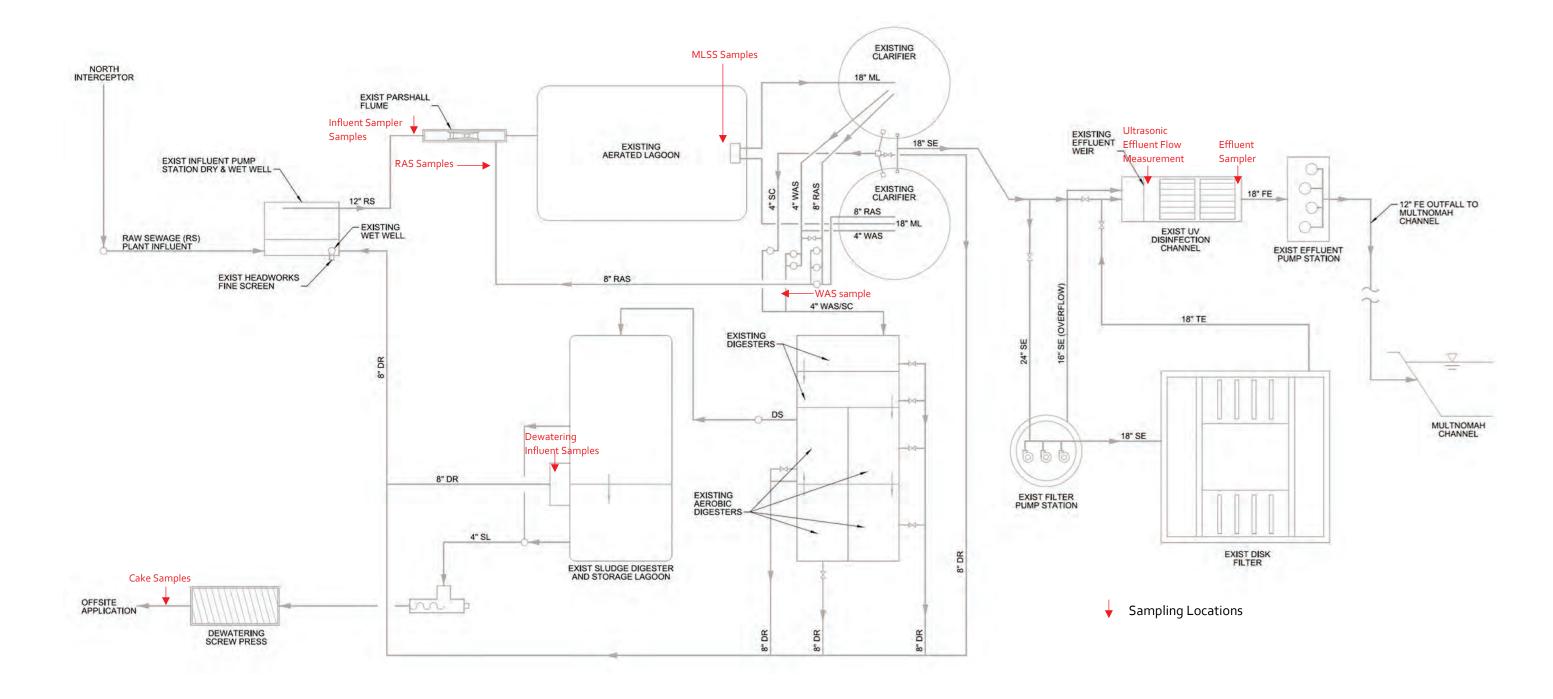


Figure 3.1 Current Plant Process Flow Diagram

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Table 3.1 Design Data for the Existing WWTP

Description	Value
Design Flows	<u>'</u>
ADWF	1056 GPM/1.58 MGD
PIF	2233 GPM/3.28 MGD
Design Loading	
BOD	2947 lbs BOD/day
TSS	2550 lbs TSS/day
Ammonia	375 lbs ammonia/day
NPDES Permit Requirements	
May 1 - October 31	
BOD-Monthly/Weekly	20 MG/L / 24 MG/L
TSS-Monthly/Weekly	32 MG/L / 36 MG/L
November 1 - April 30	
BOD - Monthly/Weekly	32 MG/L / 36 MG/L
TSS-Monthly/Weekly	45 MG/L / 50 MG/L
Raw Sewage Pumping	
Туре	Submersible Dry Pit, Variable Frequency
Number	4
НР	15 HP
TDH, ft	40 ft
Total Capacity	3.5 MGD
Screening	
Туре	Fine screen W/Washing and Compaction
Number	1
Screen Opening Size	1/4 Inch
Maximum Screen Capacity	4 MGD
HP	5
Max Screen Capacity	4 MGD
Aeration Basin	
Volume	1.9 MG
O2 Requirement	
BOD (2LB/HP)	5894 LB/Day
Ammonia (4.6 LB/LB)	1725 LB/Day
HP Requirement	
For Q (2LB/HP)	159 HP



Table 3.1 Design Data for the Existing WWTP (Continued)

Description	Value
High Speed Surface Aerators	5
HP Each/Total	40/200
Hydraulic Retention	40/200
ADWF	29 HRS
PDF (DAY)	14 HRS
Secondary Clarifiers	CALL PT
Number	2
Diameter	50 ft
Surface Area-Each/Total	
	1/963.5 S.F./3,927 S.F.
Side Water Depth	15 FT
Overflow Rate at ADWF/PIF (GAL/SF/DAY)	402/835
Detention time at ADWF/PIF (HRS/DAY)	6.92/3.22
Weir overflow rate at ADWF/PIF (GAL/LF/DAY)	5,029/10,441
Return Activated Sludge Pumps	
Number/Horsepower Per Clarifier/Variable Frequency	3/3 HP
Capacity Each Pump	450 GPM/.648 MGD
Waste Activated Sludge Pumps	
Number/Horsepower Per Clarifier	2/2 HP
Capacity Each Pump	100 GPM/.144 MGD
Filter Pump Station	
Number of Pumps	2
Power Requirements (Per Pump/Total)	16 HP/32 HP
Type of Pump	Submersible with VFD
Capacity Per Pump	0.6-2 MGD
Pump Station Dimensions (Diameter/Depth)	10 Feet/7Feet
Liquid Volume at High Water Level (GAL)	2,191
Disk Filter	
Number of Filters	2
Disks Per Filter (Startup/Future)	4/6
Basin Type	Steel
Capacity Per Filter	2 MGD
Total Redundancy Capacity	2 MGD
Media Type	Polyester Cloth
Media Effective Pore Size	10



Table 3.1 Design Data for the Existing WWTP (Continued)

Description	Value
Dimensions Per Filter (Length/Width/Depth)	12.5 Feet/9 Feet/ 15 Feet
Total Power Requirements	5 HP
Number of Backwash Pumps	2
Aerobic Digestion	
Volume Tankage	192,000 GAL
Volume Lagoon	974,000 GAL
Total Volume Combined	1,166,000 GAL
Hydraulic Loading @ 1% Solids	25,000 GAL/DAY
Tankage Hydraulic Detention Time with No Decanting	7.5 Days
Lagoon Hydraulic Detention Time with No Decanting	39 Days
Total Solids Retention Time	71.5 Days
Air Supply to Converted Tankage	700 SCFM
1 Blower Operating	Diffused Aeration
2 Blower Operating	1270 LBS/DAY
Oxygen Supplied Per Day	
l Blower Operating	740 LBS/Day
2 Blower Operating	1270 LBS/DAY
Mixing and Oxygen Supply for Aerated Lagoon	High Speed Surface Aeration
Number/Horsepower	5/7.5
Oxygen Supplied at 2LB/HP/Hour	1800 LB/Day
Sludge Storage Volume	347.326 GAL/46,500 CU FT
LBS Storage at 3% Solids	86,900 LBS
Days Storage	48 Days
Final Effluent Pumping Station	
Number/Horsepower	4/30 HP 2 Speed Motor
Capacity	760 GPM @ 120'TDH
Pump Type	Vertical Turbine
Field Application Sludge Pump	
Number/Horsepower	1/15
Capacity	100 GPM @ 110 TDH
Field Application Area	40 Acres
Spray Nozzles	1
Application Rate @ 1% Solids	0.023 IN/Day



Table 3.1 Design Data for the Existing WWTP (Continued)

Description	Value
Ultraviolet Disinfection Channel	
Tank Size	2' 9" x 34' - 9"
% Transmission	55%
Number of Channels	1
Number of Banks	2
Number of Modules Per Bank	11
Number of Lights Per Module	8
Total Number of Lights	176
Water Depth	2' - 0"
Velocity at ADWF	0.45 FT/SEC
Velocity at PWWF	0.91 FT/SEC
Lamp Length	58"
Exposure Time Per Bank at ADWF	9.53 SEC
Exposure Time Per Bank at PWWF	4.72 SEC
UV Dose per Bank at ADWF	28,742 Microwatts SEC/SQ CEN
UV Dose Per Bank at PWWF	14,160 Microwatts SEC/SQ CEN

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 3.2 below, and summarized in the subsequent sections.

Table 3.2 Flow Capacity of Existing Processes

Process Area	Existing Hydraulic Capacity, mgd	Required Hydraulic Capacity, mgd ¹
Influent Screening	4.0 mgd	6.05 mgd
Influent Pumping Station	3.5 mgd	6.05 mgd
Aeration Basin	7.0 mgd ²	6.05 mgd
Secondary Clarifiers	4.7 mgd	6.05 mgd
Tertiary Filtration System		
- Intermediate Pumping Station	4 mgd	3.6 mgd ⁴
- Filtration	2.4 mgd	2.4 mgd⁵
UV Disinfection	5.6 mgd ³	6.05 mgd
Effluent Pump Station	4.4 mgd	6.05 mgd
Outfall	4.3 mgd	6.05 mgd

Notes:

- (1) Through planning period (year 2042)
- (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 2042 Summer Peak Instantaneous Flow
- (5) Year 2042 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 6.05 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, 3.5 mgd (total) and 2.80 mgd (firm), is insufficient to accommodate the PIF of 6.05 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 (6.05 mgd) which controls the level in the basin. The aeration basin is therefore restricted to peak flow of 5.7 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 1200 gpd/sf as outlined in Chapter 1, the secondary clarifiers have sufficient capacity (2.35 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 2042 summer peak instantaneous flow of 3.6 mgd.
- Disk filters is to meet year 2042 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 6.05 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 3.6 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 6.05 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 3.3.

Table 3.3 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
$F_{\mbox{\tiny US}}$ (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 Fac	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.

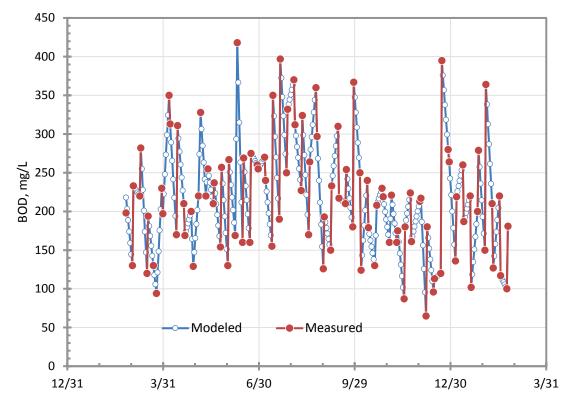


Figure 3.2 Plant Measured and Model Calculated Influent BOD Concentration



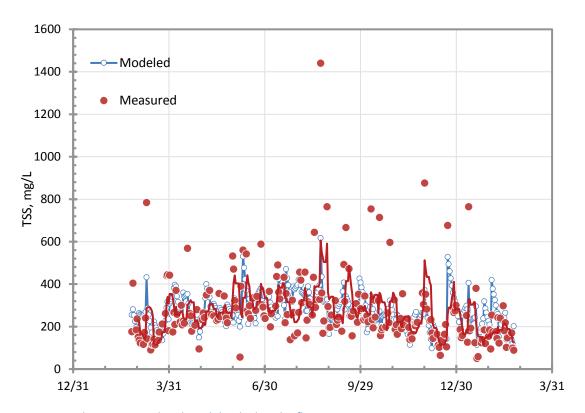


Figure 3.3 Plant Measured and Model Calculated Influent TSS Concentration

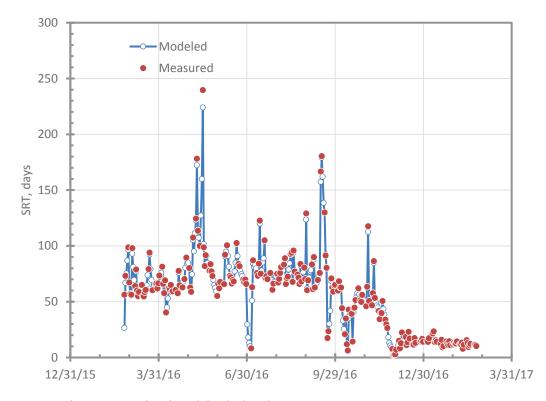


Figure 3.4 Plant Measured and Model Calculated MLSS Concentration



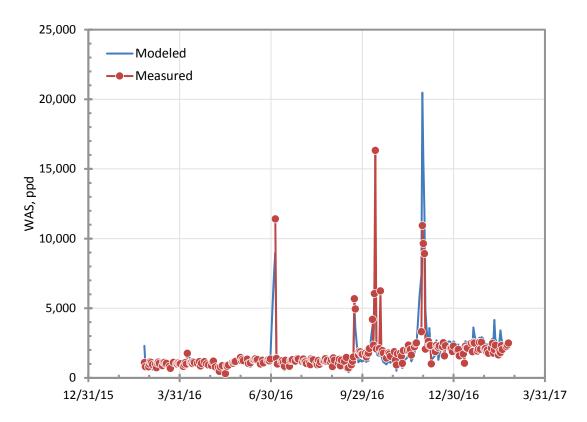


Figure 3.5 Plant Measured and Model Calculated WAS Load

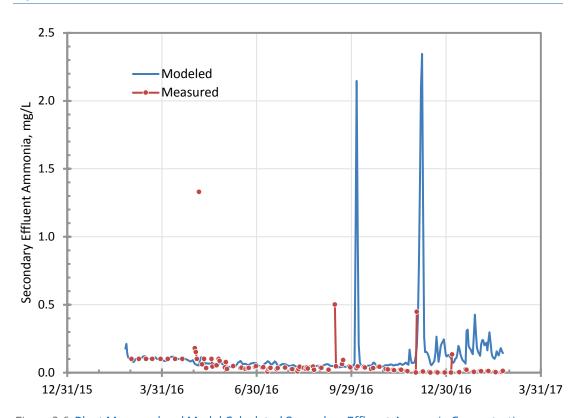


Figure 3.6 Plant Measured and Model Calculated Secondary Effluent Ammonia Concentration

3.4.2 Basis of Planning

Table 3.4 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 3.4 Effluent Quality Goals

Parameter	Average monthly limit Summer	Average monthly limit Winter
BOD₅	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.



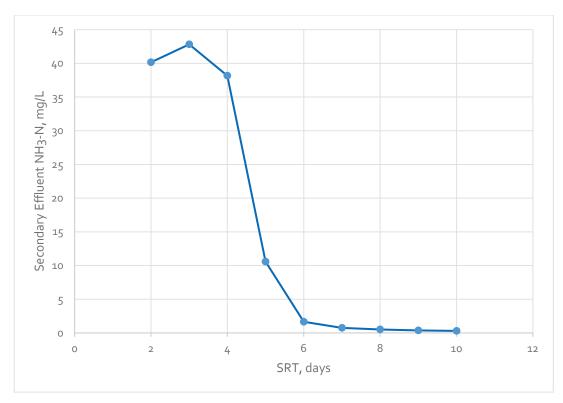


Figure 3.7 Model Predicted Secondary Effluent Ammonia Concentrations versus SRT

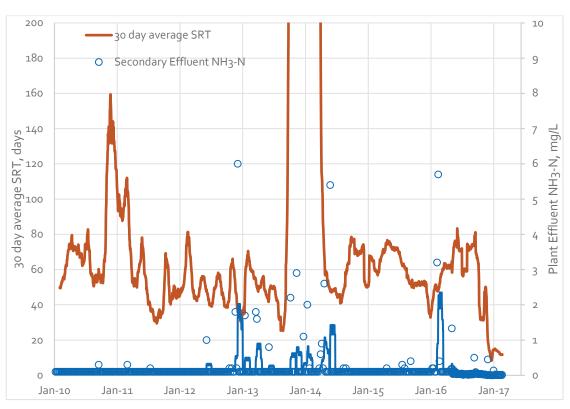


Figure 3.8 Plant Measured SRT and Secondary Effluent Ammonia Concentrations

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 6.05 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 hydraulic capacity and redundancy to take one clarifier out of service during the summer months, one additional secondary clarifier is recommended. 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).

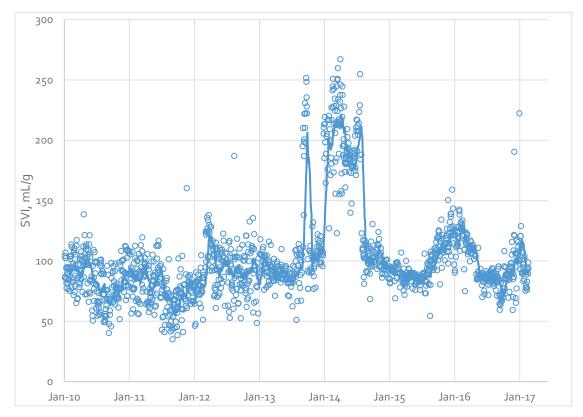


Figure 3.9 Plant Measured SVI



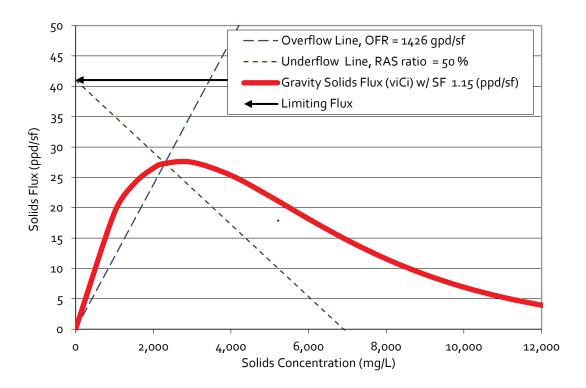


Figure 3.10 State Point Analysis for 2 Secondary Clarifiers in Service during Peak Hour Flow

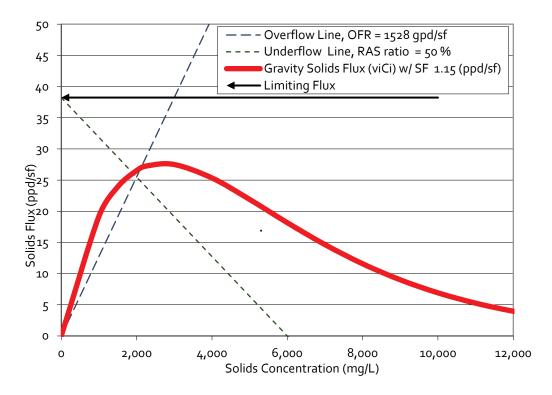


Figure 3.11 State Point Analysis for 1 Secondary Clarifier in Service with 3 mgd Peak Flow



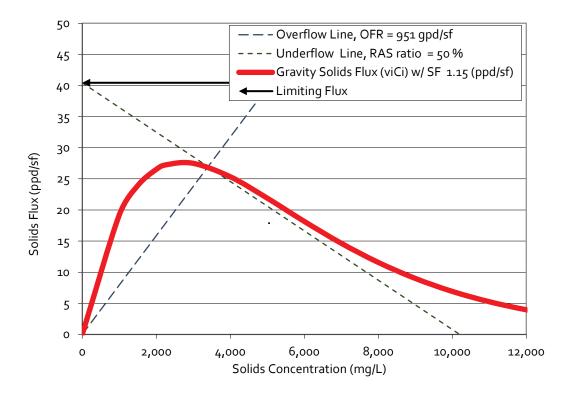


Figure 3.12 State Point Analysis for 3 Secondary Clarifiers in Service during Peak Hour Flow

Table 3.5 summarizes the aerated lagoon and secondary clarifier design and redundancy criteria assumed in the analysis.

Table 3.5 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 2042 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 2042 flows and loads.

Although the current system has sufficient capacity to treat the year 2042 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.

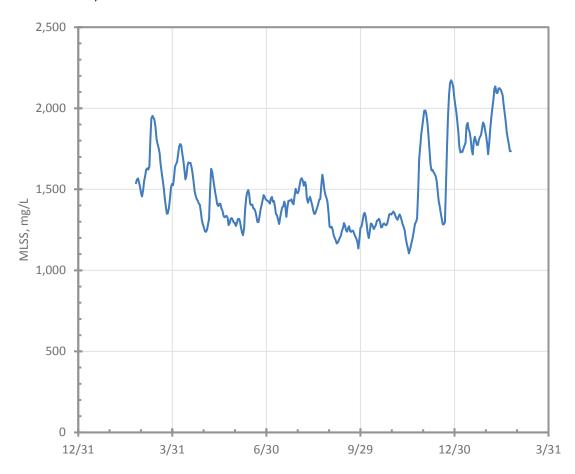


Figure 3.13 Model Predicted Future MLSS Concentration

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 indicates that the existing aerobic digesters are nearing 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.



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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 drypit 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

A summary of 20-year present worth costs for both the alternatives is presented in Table 4.2.

Table 4.2 Present Worth Comparison of Headworks Alternatives

Alternative	Capital Cost	O&M Cost*	Total Present Worth**
Alternative 1: Repair and Expand Existing Headworks	\$5 M	\$0.98 M	\$5.98 M
Alternative 2: Construct New Headworks	\$5.5 M	\$0.88 M	\$6.38 M

Notes:

While Alternative 1 utilizes existing space and is lower in present worth cost, 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.



^{*} Power cost is based on 0.07 \$/KWH and labor cost is based on \$50/hr.

^{**} Present worth calculation is based on interest rate of 6.5% and inflation rate of 2% for power and labor.

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

A summary of present worth cost of the alternatives is presented in Table 4.3.

Table 4.3 Present Worth Comparison of UV Disinfection Alternatives

Alternative	Capital Cost	O&M Cost¹	Total Present Worth²
Alternative 1: Replace Aged Equipment with Updated Technology with Bypass Channel	\$1.5 M	\$0.3 M	\$1.8 M
Alternative 2: Replace Aged Equipment with Updated Technology with Bypass Channel Equipped with UV Banks	\$2.0 M	\$0.3 M	\$2.3 M
Alternative 3: Leave Existing As-Is and Add New channel with Latest Technology	\$1.8 M	\$0.3 M ³	\$2.1 M

Notes

- 2. Present worth calculation is based on interest rate of 6.5% and inflation rate of 2% for power and labor.
- ${\it 3. New channel with latest technology will be duty channel.}\\$

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



^{1.}Power cost is based on 0.07 \$/KWH and labor cost is based on \$50/hr. The O&M costs for UV disinfection is based on power, lamp replacement every 12,000 hours, ballast replacement every 5 years, wiper replacement every 2 years, and sleeve replacement every 5 years.

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.



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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 WWTP. 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.

Additionally, this chapter also presents detailed description of proposed CIP Improvements. The following information regarding near-term projects is provided in this chapter:

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

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 in provided in Appendix D.



Table 5.1 Implementation Program and Phasing Schedule

			Construct	Construction Year		
Item Description	Description	Phase 1 2018 – 2021	Phase 2 2021 - 2023	Phase 3 2023 - 2026	Phase 4 2026 – 2028	Total
Phase 1 Improvements						
Spring Lake Lift Station	Replacement of pumps and corroded piping; addition of valve vault to measure flow.	\$271, 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				\$6,430,600
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		
Effluent Pump Station	Replacement of pumps, modification of skylights, addition of flow meter, electrical improvements.			\$536,600		\$4,393,200
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



5.3 Phase 1 Improvements

Five 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.

5.3.1 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.

5.3.2 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 5.2.

Table 5.2 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 2042 is 6.05 mgd.	6.05 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:

5.3.3 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.

5.3.4 Secondary Clarifier and RAS/WAS Pumping Upgrades

5.3.4.1 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



⁽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

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).

5.3.4.2 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.

5.3.5 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.



5.4 Phase 2 Improvements

Phase 2 includes the following improvements:

- Headworks and influent pumping, and
- Operational improvements.

5.4.1 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 5.1 presents the process flow schematic for the proposed improvement.

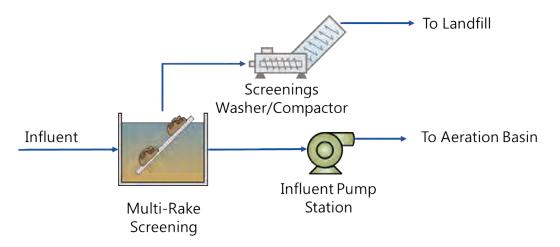


Figure 5.1 Headworks Process Flow Schematic

5.4.1.1 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 5.2 shows a diagram of a typical multi-rake bar screen (Source: Headworks, Inc.).

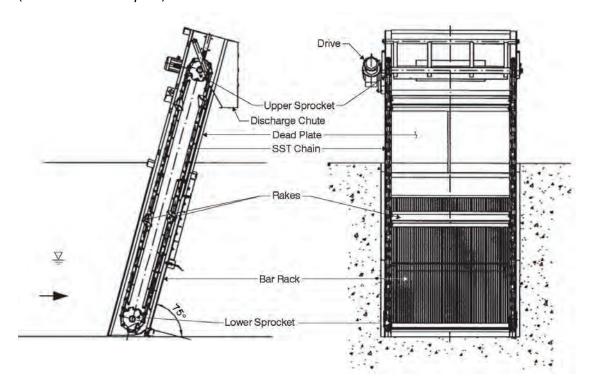


Figure 5.2 Typical Multi-Rake Bar Screen

5.4.1.2 Screenings Handling

Screenings Conveyance

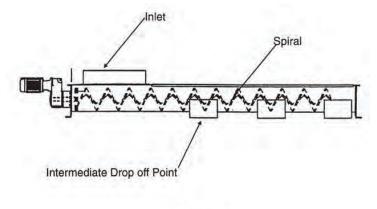
Screenings must be conveyed to a washer/compactor for further washing and dewatering. Conveyance technologies that can be used for screenings conveyance include belt conveyors, sluiceways, and shaftless screw conveyors. Belt conveyors are not recommended for this application because they are odorous 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 5.3 (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.



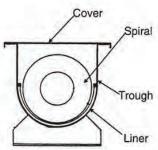


Figure 5.3 Shaftless Screw Conveyor

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 5.4 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.

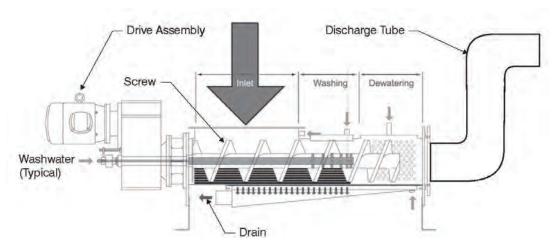


Figure 5.4 Screenings Washer/Compactor

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.

5.4.1.3 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 atgrade 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 5.3 provides a comparison between the two pumps.

Table 5.3 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.

5.4.1.4 Recommended Alternative Design Criteria

The recommended headworks design criteria is presented in Table 5.4.

Table 5.4 Recommended Headworks Design Criteria

Parameter	Value
Screening	
Туре	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

5.4.2 Operational Improvements

5.4.2.1 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.

5.4.2.2 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.

5.5 Phase 3 Improvements

Phase 3 includes the following improvements:

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

5.5.1 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.

5.5.2 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.

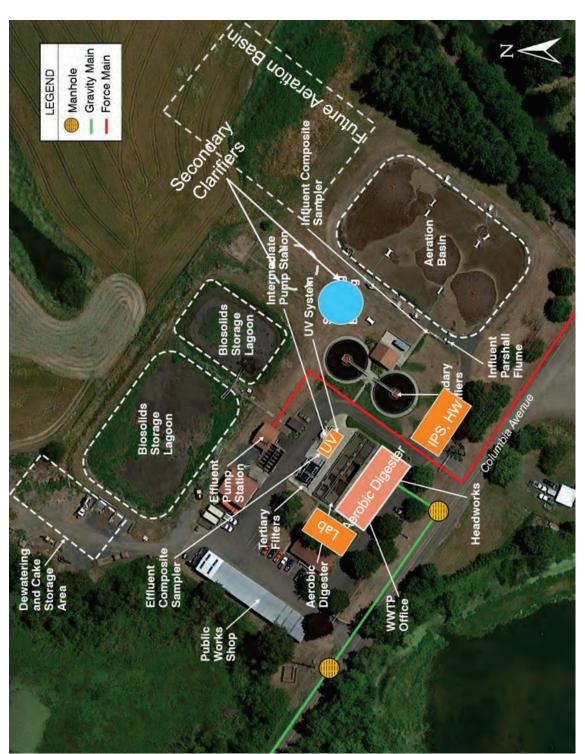
5.5.3 Parallel Outfall

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.

5.6 Recommended Site Layout

The recommended site layout is presented in Figure 5.5.





IMPLEMENTATION PLAN | CH 5 | CITY OF SCAPPOOSE

Figure 5.5 Recommended Site Layout

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Appendix A SCAPPOOSE WASTEWATER FACILITIES DRAFT PLANNING STUDY (KELLER ASSOCIATES, DECEMBER 2016



Scappoose Wastewater Facilities Planning Study Draft

December 2016





City of Scappoose, Oregon

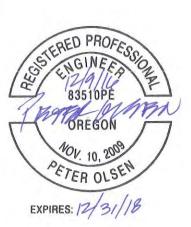
Wastewater Facilities
Planning Study
Draft





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

215107\5\S16-005



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



TABLE OF CONTENTS

ACRONYMS, ABBREVIATIONS, AND SELECTED DEFINITIONS

ES.	EXECU	JTIVE SUMMARY		
	ES.1	PLANNING CRITERIA		ES-1
	ES.2	DESIGN CONDITIONS		ES-1
		ES.2.1 Study Area and Land Use	ES-1	
		ES.2.2 Demographics		
		ES.2.3 Wastewater Flows	ES-4	
	ES.3	COLLECTION SYSTEM EVALUATION		ES-5
		ES.3.1 Lift Station Evaluation	ES-5	
		ES.3.2 Pipeline Condition and Capacity Evaluation	ES-5	
		ES.3.3 Collection System Alternatives	ES-8	
		ES.3.4 Recommended Collection System Improvements.	ES-8	
	ES.4	EFFLUENT DISPOSAL		.ES-10
		ES.4.1 Effluent Disposal Options	ES-10	
		ES.4.2 Effluent Disposal Recommendation	ES-10	
	ES .5	WASTEWATER TREATMENT		.ES-10
		ES.5.1 Existing Facilities	ES-10	
		ES.5.2 Wastewater Composition	ES-12	
		ES.5.3 Treatment Alternatives	ES-13	
		ES.5.4 Recommended Treatment Improvements	ES-13	
	ES.6	CAPITAL IMPROVEMENT PLAN AND FINANCING		.ES-14
		ES.6.1 Summary of Costs	ES-14	
		ES.6.2 Budget and Rate Impacts	ES-16	
		ES.6.3 Other Annual Costs	ES-16	
		ES.6.4 SDCs	ES-16	
		ES.6.5 Financing Options	ES-17	
1.	PROJE	ECT PLANNING		
	1.1	LOCATION		1-1
	1.2	ENVIRONMENTAL RESOURCES PRESENT		
		1.2.1 Land Use / Prime Farmland / Formally Classified L		
		1.2.2 Earthquake Hazards		
		1.2.3 Floodplains		
		1.2.4 Wetlands		
		1.2.5 Cultural Resources		
		1.2.6 Biological Resources	1-3	
		1.2.7 Water Resources		
		1.2.8 Coastal Resources	1-3	
		1.2.9 Socio-Economic Conditions		
		1.2.10 Miscellaneous Issues	1-3	
	1 2	DODIII ATION TDENIDO		1 2



	1.4	FLOWS			1-6
	1.5	NPDES P	ERMIT		.1-12
	1.6	COMMU	JNITY ENGAGEMENT		.1-14
2.	COLLI	ECTION	SYSTEM EXISTING FACILITIES		
	2.1	LOCATION	ON MAP		2-1
	2.2	HISTORY	,		2-1
	2.3	SYSTEM	DESCRIPTION		2-1
	2.4	CONDIT	ION OF EXISTING LIFT STATIONS		2-1
	2.5	CONDIT	ION OF EXISTING COLLECTION SYSTEM PIPELINES		.2-10
		2.5.1	Gravity Mains	2-10	
		2.5.2	Steinfeld Plant's Gravity Line		
	2.6	COLLEC	TION SYSTEM COMPUTER MODEL		.2-11
		2.6.1	Model Selection	2-11	
		2.6.2	Model Creation	2-12	
		2.6.3	Model Calibration	2-12	
		2.6.4	Existing System Capacity Limitations	2-14	
		2.6.5	Pipeline Conditions		
	2.7	FUTURE (COLLECTION SYSTEM PERFORMANCE		.2-15
		2.7.1	Future Flow Rate Projections & Model Scenarios	2-15	
			Future System Capacity Limitations		
	2.8		DINANCES & PRETREATMENT		
	2.9	FINANC	IAL STATUS OF EXISTING FACILITIES		.2-18
	2.10	WATER/	Energy/waste audits		.2-18
3.	WASII	EWATER	TREATMENT PLANT EXISTING FACILITIES		
	3.1		ON MAP		
	3.2		,		
	3.3		DESCRIPTION		
	3.4	CONDIT	ION OF EXISTING FACILITIES		3-2
		3.4.1	Headworks		
		3.4.2	Aeration Basin		
		3.4.3	Secondary Clarifiers		
		3.4.4	Sludge Pumping Building		
		3.4.5	Tertiary Filters		
		3.4.6	UV System		
		3.4.7	Effluent Pump Station		
		3.4.8	Aerobic Digester		
		3.4.9	Biosolids Storage Lagoons		
			SCADA		
			Electricity Plant Water		
			WWTP Office		
			Site Security and Maintenance		
		J.7.17	Cito Cooding and Maintenance	5 15	



		3.4.15	Influent Quality3-15	
		3.4.16	WWTP Operations3-19	
	3.5	CAPAC	ITY LIMITATIONS	3-27
	3.6	FINANC	CIAL STATUS OF EXISTING FACILITIES	3-30
	3.7	WATER/	'ENERGY/WASTE AUDITS	3-30
4.	NEED	FOR PR	OJECT	
	4.1	HEALTH	, SANITATION, AND SECURITY	4-1
	4.2	AGING	INFRASTRUCTURE	4-1
	4.3	SYSTEM	DEFICIENCIES	4-1
	4.4	REASO	NABLE GROWTH	4-4
5.	ALTER	NATIVE	S CONSIDERED	
	5.1	PLANNI	NG CRITERIA	5-1
		5.1.1	Collection System5-1	
		5.1.2	Wastewater Treatment Plant Facilities 5-1	
	5.2	DESCRI	PTION	5-3
		5.2.1	Regionalization 5-3	
		5.2.2	Conveyance System Alternatives 5-3	
		5.2.3	WWTP Alternatives5-7	
	5.3	MAP		5-13
	5.4	ENVIRO	NMENTAL IMPACTS	5-14
		5.4.1	Land Use/Prime Farmland/Formally Classified Lands 5-14	
		5.4.2	Floodplains 5-14	
		5.4.3	Wetlands5-14	
		5.4.4	Cultural Resources 5-14	
		5.4.5	Biological Resources 5-14	
		5.4.6	Water Resources 5-14	
		5.4.7	Socio-Economic Conditions 5-14	
	5.5		EQUIREMENTS	
	5.6		IAL CONSTRUCTION PROBLEMS	
	5.7		NABILITY CONSIDERATIONS	
		5.7.1	Water and Energy Efficiency 5-17	
		5.7.2	Green Infrastructure5-17	
		5.7.3	Other	
	5.8	COSTE	STIMATES	5-17
6.			F AN ALTERNATIVE	
	6.1		ARATIVE ANALYSIS (COSTS & NON-MONETARY FACTORS)	6-1
		6.1.1	Collection System West Side Flow Alternatives 6-1	
		6.1.2	WWTP Disposal Alternatives	
		6.1.3	Headworks Alternatives	
		6.1.4	Primary Treatment Alternatives	
		6.1.5	Secondary Treatment Alternatives 6-4	



		6.1.	.6	Filtration Alternatives	6-8
		6.1.	7	Disinfection Alternatives	6-8
		6.1.	8.	Biosolids Treatment Alternatives	6-11
		6.1.	9	Biosolids Disposal Alternatives	6-13
		6.1.	.10	Dewatering Building Alternatives	6-14
7.	PROP	OSED	PF	OJECT (RECOMMENDED ALTERNATIVES)	
	7.1	PRELI	MII	NARY PROJECT DESIGN	7-1
		7.1.	.1	Collection System	7-1
		7.1.	2	Pipeline Cleaning and CCTV	
		7.1.	.3	Service Lines	
		7.1.	4	Flow Monitoring	
		7.1.	.5	Pipeline Replacement Program	
		7.1.	6	Lift Station Improvements	
		7.1.	7	WWTP Improvements	7-2
		7.1.	.8	Other	7-4
	7.2	PROJ	EC	「SCHEDULE	7-4
	7.3	PERM	IIT E	REQUIREMENTS	7-5
	7.4	SUSTA	AIN	ABILITY CONSIDERATIONS	7-6
		7.4.	.1	Water and Energy Efficiency	7-6
		7.4.	2	Green Infrastructure	
		7.4.	.3	Other	7-6
	7.5	TOTA	LΡ	ROJECT COST ESTIMATE	
		(ENG	INE	ER'S OPINION OF PROBABLE COST)	7-6
	7.6	FINA	NC	AL STATUS OF EXISTING FACILITIES	7-8
	7.7	ANN	UAI	OPERATING BUDGET	7-8
		7.7.	.1	Potential User Rate Impacts	7-8
		7.7.	2	System Development Charges	
		7.7.	.3	Annual O&M Costs	
119	T OF T	ΔRIF	ς.		
LIG				gn and Projected Flows	
				ear Capital Improvement Plan	
				ntial Monthly User Rate Impact to Fund Priority Impro	vements
	Table 1-			lation History and Projections	vements
	Table 1-		-	prehensive Plan Zoning and Corresponding City Zoni	ina
	Table 1-			age Dwellings per Lot and Lot Size	9
	Table 1-			Projected Population Growth	
	Table 1-		•	ear Population Growth Estimates	
	Table 1-		-	vs. Rainfall (MMDWF ₁₀ and MMWWF ₅)	
	Table 1-			al Peak Day Flow / Average Base Flow	
	Table 1-			parison of Projected Flows	
	Table 1-			cted Flows	
			•	ing NPDES Permit Limits	
				tation Inventory	



Table 2-2:	Pipe Type and Size Summary
Table 2-3:	Proposed Lift Station Summary
Table 3-1:	Summary of Influent BOD₅ Data
Table 3-2:	Summary of Influent TSS Data
Table 3-3:	Influent Loading Projections
Table 3-4:	Unit Process Reliability Evaluation
Table 3-5:	Plant Capacity Summary
Table 5-1:	20-Year (2035) Design Criteria for Scappoose WWTP
Table 5-2:	Design Criteria for Component Sizing
Table 5-3:	Requirements for Reuse of Effluent by Category
Table 5-4:	Affected Environment / Environmental Consequences Summary for Alternatives
Table 6-1:	Smith Road Lift Station & Scappoose Creek Line Upsize Cost Estimate
Table 6-2:	New Relief Trunk Line Across Town Cost Estimate
Table 6-3:	New Headworks Cost Estimate
Table 6-4:	Repair and Expand Headworks Cost Estimate
Table 6-5:	New Aeration Basins Cost Estimate
Table 6-6:	Oxidation Ditches Cost Estimate
Table 6-7:	SBR Facility Cost Estimate
Table 6-8:	Summary of Secondary Treatment Advantages and Disadvantages
Table 6-9:	Upgrade and Expand UV Cost Estimate
Table 6-10:	Chlorination and Dechlorination Cost Estimate
Table 6-11:	Peracetic Acid (PAA) Cost Estimate
Table 6-12:	Summary of Disinfection Advantages and Disadvantages
Table 6-13:	New Aerobic Digester Cost Estimate
Table 6-14:	Anaerobic Digesters Cost Estimate
Table 6-15:	Summary of Biosolids Treatment Advantages and Disadvantages
Table 6-16:	Class B – Dewatered Biosolids Cost Estimate
Table 6-17:	Class A – Compost (Covered Aerated Static Pile) Cost Estimate
Table 6-18:	Summary of Biosolids Disposal Advantages and Disadvantages
Table 6-19:	Repurpose Metal Shop Cost Estimate
Table 6-20:	New Dewatering Building Cost Estimate
Table 7-1:	Replacement Budgets
Table 7-2:	Priority 1 Capital Improvement Plan
Table 7-3:	20-Year Capital Improvement Plan
Table 7-4:	User Rate Impact

LIST OF CHARTS:

Chart 1-1:	Above-Ground Cultural Resou	ırces
Chart 1-1:	Above-Ground Cultural Resou	ırces

- Chart 1-2: Flow vs. Rainfall (MMDWF₁₀ and MMWWF₅)
- Chart 1-3: Flow vs. Rainfall (PDAF₅)
- Chart 1-4: 2013 Daily Flow and Precipitation
- Chart 2-1: Sample Dry Calibration Site 1 Modeled vs. Observed Flows (MH0484)
- Chart 2-2: Sample Wet Calibration Site 5 Modeled vs. Observed Flows (MH0537)
- Chart 3-1: WWTP Influent BOD₅ Concentrations
- Chart 3-2: WWTP Influent BOD₅ Loading



Chart 3-3:	WWTP Influent TSS Concentrations
Chart 3-4:	WWTP Influent TSS Loading
Chart 3-5:	WWTP Effluent BOD₅ Concentrations (Monthly)
Chart 3-6:	WWTP Effluent BOD₅ 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 BOD₅ Loading (Average Weekly)
Chart 3-10:	WWTP Effluent BOD₅ Loading (Maximum Daily)
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)
Chart 3-17:	WWTP Effluent E. coli Bacteria (Monthly)
Chart 3-18:	WWTP Effluent E. coli Bacteria (Daily)
Chart 3-19:	WWTP Effluent pH (Maximum Daily)
Chart 3-20:	WWTP Effluent pH (Minimum Daily)
Chart 5-1:	New Aeration Basins – Process Flow Diagram

LIST OF FIGURES INCLUDED IN REPORT:

Chart 5-3: SBR – Process Flow Diagram

Figure ES1: Land Use and Study Area

Figure ES2: Future Growth Areas

Figure ES3: Existing System Pipe Size and Lift Station Basins

Chart 5-2: Oxidation Ditches – Process Flow Diagram

Figure ES4: Smoke Testing

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

Figure ES6: Collection System CIP Figure ES7: Existing WWTP Layout

Figure ES8: Proposed Future WWTP Layout

APPENDIX A: FIGURES

Figure 1: Land Use and Study Area
Figure 2: Topography and Flood Plain

Figure 3: Soils

Figure 4: Relative Earthquake Hazard Figure 5: National Wetlands Inventory

Figure 6: Future Growth Areas

Figure 7: Existing System Size and LS Basins

Figure 8: Existing System Material Figure 9: Smoke Testing Results

Figure 10: Modeled vs Not Modeled Lines Figure 11: Flow Monitoring Locations

Figure 12: Existing System Flooded MH and Pipe Capacities



Figure 13: Existing System Velocities w WTP

Figure 14: Existing WWTP Layout Figure 15: Process Flow Diagram

Figure 16: Alternative A Projects

Figure 17: Alternative B Projects

Figure 18: New Aeration Basins

Figure 19: Oxidation Ditches

Figure 20: Sequencing Batch Reactor (SBR) Figure 21: Alternative B Flow Capacities

Figure 22: Collection System CIP

Figure 23: WWTP CIP

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

gpcd gallons per capita per 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

MMDWF₁₀ maximum monthly average dry-weather flow MMWWF₅ 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.



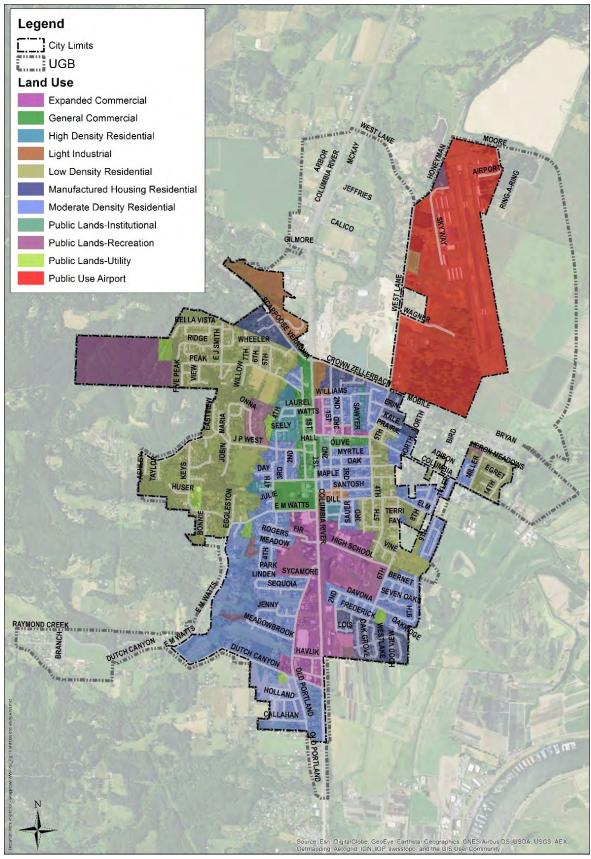


Figure ES1 - Land Use and Study Area



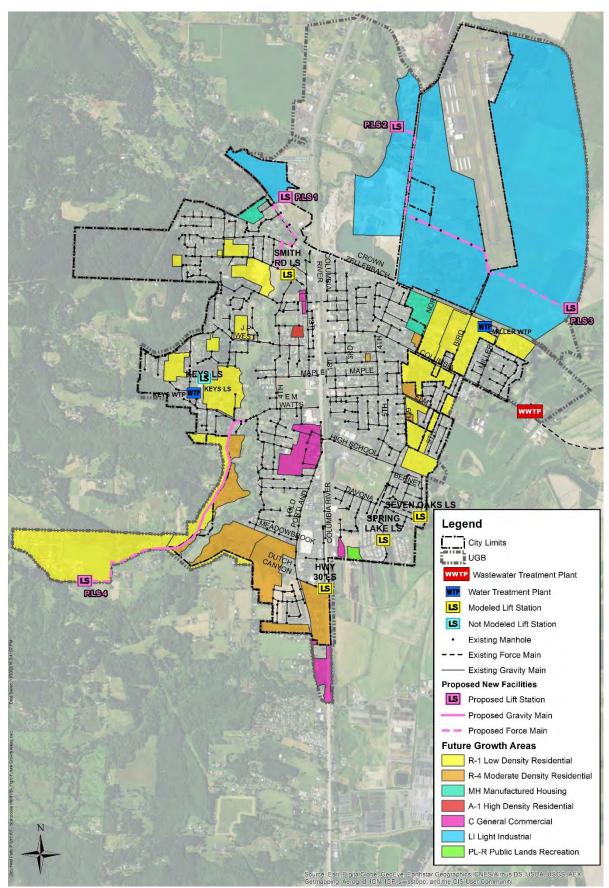


Figure ES2 - Future Growth Areas



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.

	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		

TABLE ES-1: Design and Projected Flows

Notes:

- 1. MGD = millions of gallons per day; gpcd = gallons per capita per day.
- 2. PDAF₅ modified based on actual peak day flow from DMR records. DEQ graphical PDAF₅ is 1.64 MGD.
- 3. PIF₅ modified based on City-provided flowcharts. DEQ graphical PIF₅ 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.

Page ES-4



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.



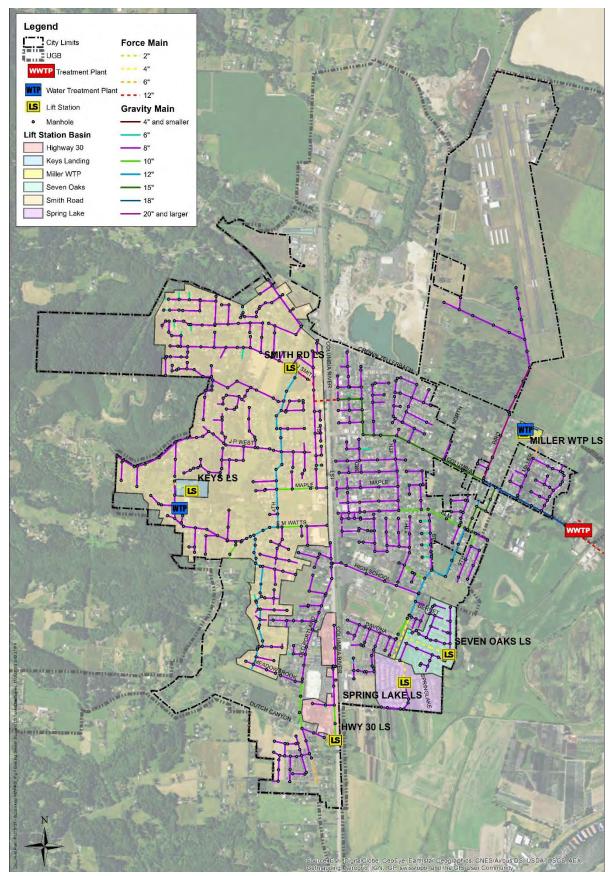


Figure ES3 – Existing System Pipe Size and Lift Station Basins



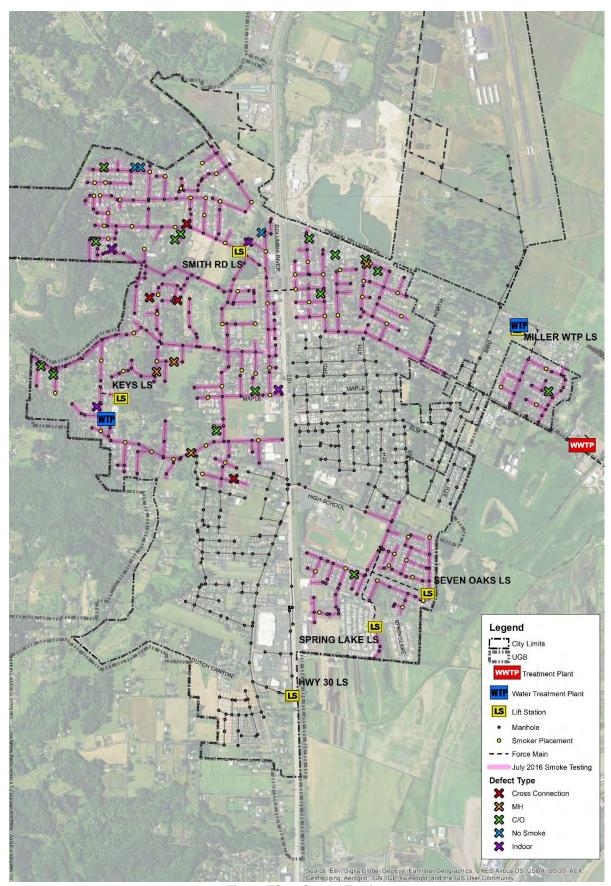


Figure ES4 – Smoke Testing



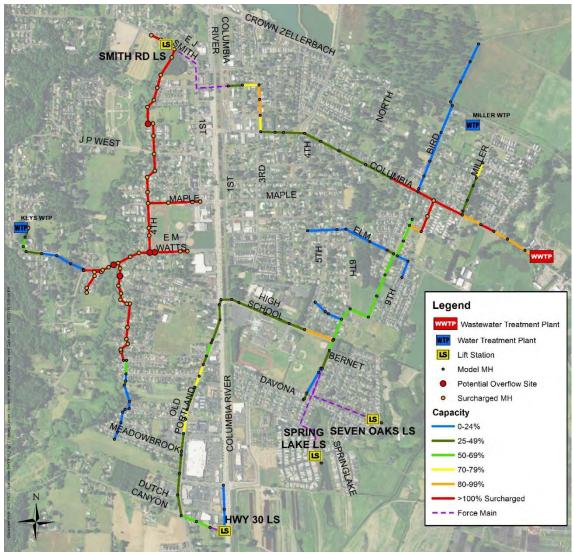


Figure ES5 - Existing System (Design Flow) Potential Overflow Sites and Pipe Capacity

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.



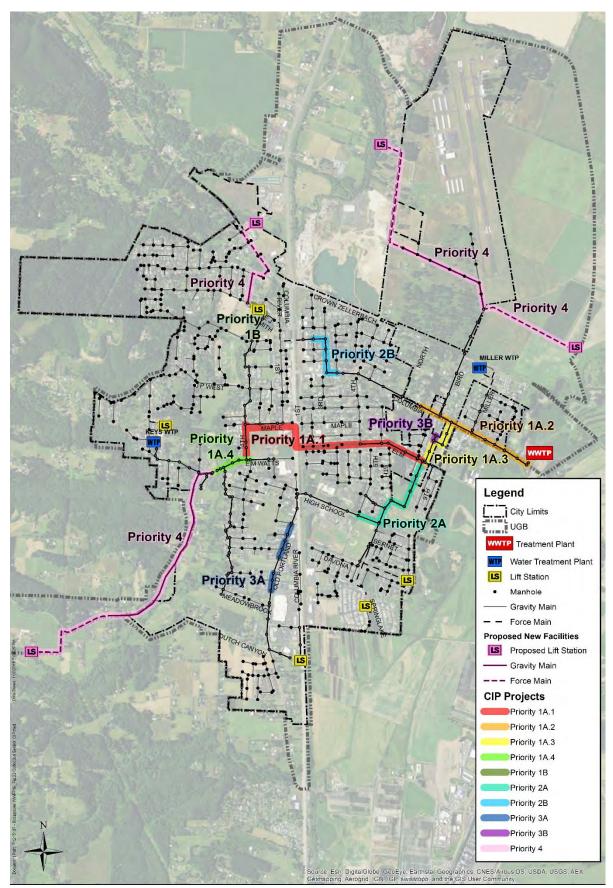


Figure ES6 - Collection System CIP



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.



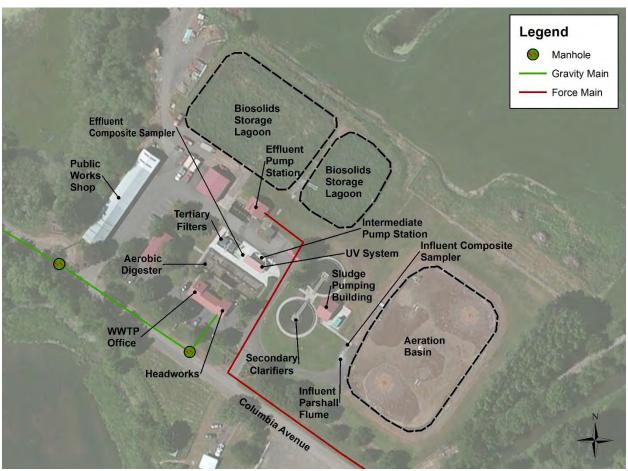


Figure ES7 - Existing WWTP Layout

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 BOD₅ 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 BOD₅ 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 MMWWF5 flow of 450 gpd/EDU and PIF5 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.



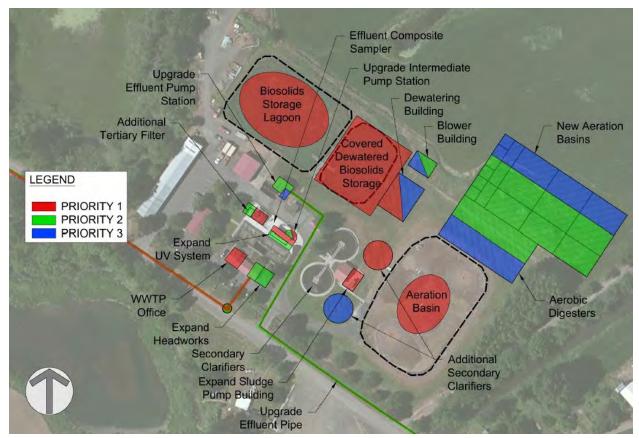


Figure ES8 - Proposed Future WWTP Layout

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

			Additional EDUs to	Tot	al Estimated	SDC Gr	owth Portion	City's Estimate	
ID#	ltem	WWTP Flow Trigger	Meet Flow Trigger		ost (2016)	%	Cost	Portion	
Priority	1 Improvements								
Wastew	ater Collection System								
1A.1	New, Relief Trunk Line			\$	1,720,000	30%	\$ 516,000	\$ 1,204,000	
	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	
	Lift Station Improvements			\$	410,000	0%	\$ -	\$ 410,000	
Wastew	ater 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	
	Add 3rd pump to Inter. Pump Station	Beyond Capacity		\$	35,000	70%	\$ 25,000	\$ 10,000	
	Add disks to existing Tertiary Filters	At Capacity		\$	97,000	100%	\$ 97,000	\$ -	
	SCADA System	Payand Canacity		\$	297,000	63%	\$ 188,000	\$ 109,000 \$ 227,000	
	Aeration for Aeration Basin Sec. Clarifier and Sludge Bldg. Exp.	Beyond Capacity Beyond Capacity		\$	341,000 2,190,000	33% 100%	\$ 114,000 \$ 2,190,000	\$ 227,000	
	iority 1 Improvements (rounded)	beyond capacity		\$	10,340,000	10070	\$ 5,200,000	\$ 5,140,000	
	pact (20 yr, 1.6%)			\$	18.77		, , ,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	\$ 9.3	
	2 Improvements								
_	ater Collection System						•		
2A	SE 6th St Trunk Line			\$	610,000	100%	\$ 610,000	\$ -	
	NE Laurel St and NE 3rd St Trunk Line			\$	370,000	100%	\$ 370,000	\$ -	
	Lift Station Improvements			\$	240,000	26%	\$ 62,000	\$ 178,00	
	ater Treatment			Ÿ	2 10,000	2070	φ 02,000	ψ 170,000	
	New Aeration Basins	1.9 MGD MMWWF ₅	1,530	\$	7,750,000	54%	\$ 4,173,000	\$ 3,577,000	
-	New Aerobic Digester	1.8 MGD MMWWF ₅	1,310	\$	2,020,000	48%	\$ 966,000	\$ 1,054,000	
	Expand Headworks	4.1 MGD PIF ₅	1,500	\$	3,410,000	63%	\$ 2,163,000	\$ 1,247,000	
	Upgrade Influent Pumps	3.5 MGD PIF ₅	880	\$	928,000	63%	\$ 589,000	\$ 339,000	
-	Upgrade Effluent Pumps	3.3 MGD PIF ₅	675	\$	833,000	63%	\$ 528,000	\$ 305,000	
	Increase Effluent Pipe	4.0 MGD PIF ₅	1,400	\$	2,100,000	63%	\$ 1,332,000	\$ 768,000	
	· · · · · · · · · · · · · · · · · · ·	-	·	\$					
2d.1 2d.2	Upgrade Intermediate Pump Station Additional Tertiary Filter Unit	4.0 MGD PIF ₅ 1.8 MGD MMWWF ₅	1,400 1,310	\$	455,000 877,000	100%	\$ 455,000 \$ 877,000	\$ -	
	Upgrade UV System	3.1 MGD PIF ₅	480	\$	1,117,000	63%	\$ 709,000	\$ 408,000	
	iority 2 Improvements (rounded)	3.1 WIGD 1 113	400	_	20,710,000	0370	\$12,830,000	\$ 7,880,000	
	3 Improvements			-			+ ==,==,==	+ 1,222,22	
Wastew	ater Collection System						•		
3A	SW Old Portland Rd Trunk Line			\$	280,000	100%	\$ 280,000	\$ -	
	SE Tussing Wy Trunk Line			\$	50,000	40%	\$ 20,000	\$ 30,000	
	ater Treatment	1			,			. 22,00	
	Additional Aeration Basin	2.6 MGD MMWWF ₅	3,090	\$	3,220,000	100%	\$ 3,220,000	\$ -	
	Additional Secondary Clarifier	3.4 MGD MMWWF ₅	4,870	\$	1,320,000	100%	\$ 1,320,000	\$ -	
	Additional Aerobic Digester	2.3 MGD MMWWF ₅	2,420	\$	1,830,000	100%	\$ 1,830,000	\$ -	
	Additional Screw Presses	2.3 MGD MMWWF ₅	2,420	\$	1,684,000	100%	\$ 1,684,000	\$ -	
	Plant Water System	2.3 MGD MINIW W F5		\$	208,000	63%	\$ 1,084,000	\$ 76,000	
	iority 3 Improvements (rounded)		<u>-</u>	\$	8,590,000	0370	\$ 8,490,000	\$ 110,000	
	4 Improvements			۶	0,050,000		J 0,430,000	110,000	
	•								
	BLS 1 Force Main and Gravity Line			\$	660,000	100%	\$ 660,000	\$ -	
	P.LS 1, Force Main and Gravity Line			_	1 160 000	100%			
	P.LS 2, Force Main and Gravity Line			\$	1,160,000	100%	\$ 1,160,000	\$ -	
	P.LS 3 and Force Main			\$	750,000	100%	\$ 750,000	\$ -	
	P.LS 4, Force Main and Gravity Line			\$	1,210,000	100%	\$ 1,210,000	\$ -	
	iority 4 Improvements (rounded)	الماما		\$	3,780,000		\$ 3,780,000		
	VASTEWATER IMPROVEMENTS COSTS (rour) contractor avai	\$	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%).

Page ES-15



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.

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.

² Assuming SDCs are providing funding portion attributed to growth.



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.



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.

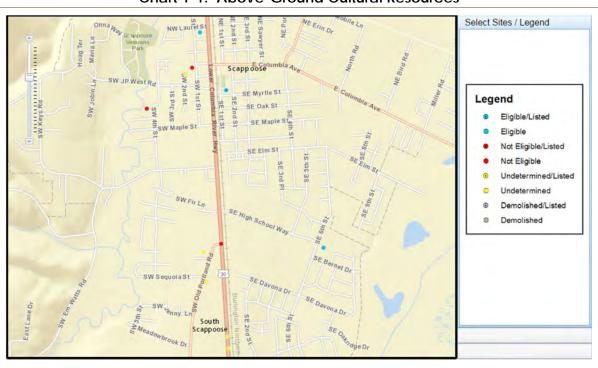


Chart 1-1: Above-Ground Cultural Resources



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	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 (MMDWF₁₀)

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 Normals¹. 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 MMDWF₁₀ using the DEQ guidance methodology. Table 1-2 summarizes the data points illustrated in Chart 1-2.

Maximum Monthly Average Wet-Weather Flow (MMWWF₅)

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.

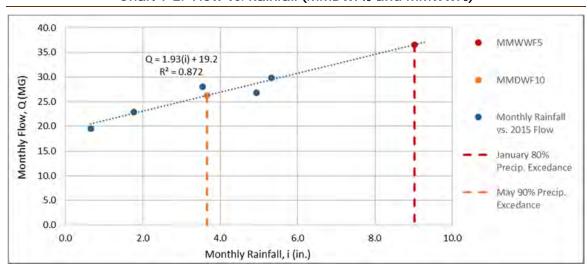


Chart 1-2: Flow vs. Rainfall (MMDWF₁₀ and MMWWF₅)

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¹ Produced by NOAA and the US Department of Commerce



Table 1-6: Flow vs. Rainfall (MMDWF₁₀ and MMWWF₅)

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 (PDAF₅)

As outlined by Oregon DEQ, the PDAF₅ 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 PDAF₅ 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 PDAF₅ 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.



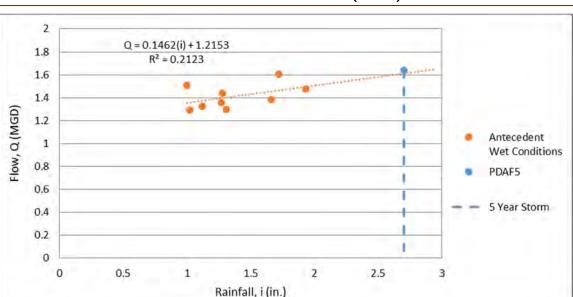


Chart 1-3: Flow vs. Rainfall (PDAF₅)

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 PDAF₅ 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 PDAF₅ was modified to reflect the highest daily DMR flow in the previous five years.

Peak Instantaneous Flow (PIF₅)

In the absence of hourly flow data, DEQ recommends obtaining the peak instantaneous flow (PIF₅) by extrapolation from their own chart titled Graph #3. On Graph #3, PDAF₅, PWkF, MMWWF₅, 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 PIF₅ 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_s 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.

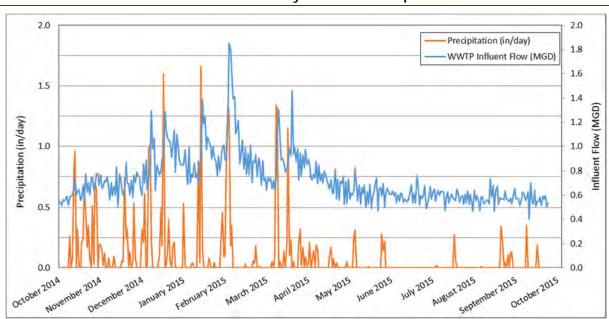


Chart 1-4: 2013 Daily Flow and Precipitation

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.

Projected Unit Design Flow Projected Domestic Flows (MGD) City Projected Flows (MGD) (MGD) Flow (gpcd) (PSU Moderate Growth Rate Pop. Projections) (City Estimated Pop. Projections) 2035 Domestic 2015 2015 2020 2025 2030 2035 2035 Domestic Year 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 98 0.736 0.809 0.889 0.977 1.291 0.660 2.114 AWWF 1.053 1.271 128 0.959 1.157 1.681 0.860 2.628 MMDWF₁₀ 0.954 1.048 1.265 0.855 127 1.151 1.672 2.616 MMWWF₅ 1.485 1.631 1.792 2.370 1.212 180 1.352 3.537 PWkF 1.397 207 1.558 1.711 1.880 2.066 2.732 4.013 PDAF₅ 1.850 274 2.063 2.266 2.490 2.736 3.617 5.181 3.234 2.640 2.943 3.553 3.904 5.162 7.218 PIF_5 391

Table 1-8: Comparison of Projected Flows

Notes:

- 1. PDAF₅ modified based on actual peak day flow from DMR records. DEQ graphical PDAF₅ is 1.64 MGD.
- 2. PIF_5 modified based on City provided flow charts. DEQ graphical PIF_5 is 2.48 MGD.
- 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)	Projecte	d Flows [Dor (Me	nestic and Ir GD)	ndustrial]
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. PDAF₅ modified based on actual peak day flow from DMR records. DEQ graphical PDAF₅ is 1.64 MGD.
- 2. PIF₅ modified based on City provided flow charts. DEQ graphical PIF₅ 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
BOD₅ (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
рН	Daily minimum	and maximum betwe	en 6.0 and 9.0
E. coli Bacteria	126/100 mL		406/100 mL

¹ BOD₅ = 5-Day Biochemical Oxygen Demand

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.

² ppd = pounds per day

³ TSS = total suspended solids



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 BOD₅ load (125 lbs./day) is not expected to increase substantially. However, there is the possibility that the BOD₅ 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					
Туре	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 #1817)	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)	9	9	9	88	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).

The pumps operate in a dutyduty-standby configuration and discharge through a 12-inch force



Smith Road Lift Station

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.

- 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

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



Spring Lake Lift Station

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.

- 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



Highway 30 Lift Station

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.

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.

Deficiencies

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

- 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.

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



Seven Oaks 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.

- 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.

Pipe Pipe Material Lengths (ft) Total by % of Total Diameter (in) Diameter (ft) **PVC** Concrete **Cast Iron Ductile Iron Unknown** 250 250 0% 6" 1,847 3,500 2% 1,272 89 303 8" 62,086 69,104 223 983 137,100 81% 4,658 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 82,300 80,500 400 1,500 170,000 100% 5,000 Material (ft) % of Total 47.4% 0.2% 0.9% 2.9% 32.2 MILES 48.41%

Table 2-2: Pipe Type and Size Summary

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

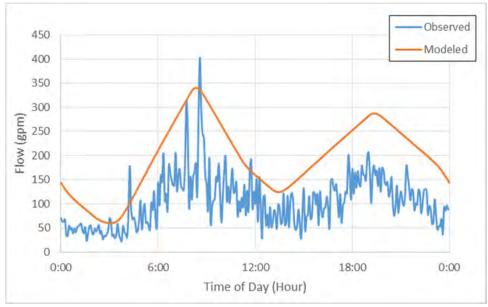


model using GIS to provide a highly accurate distribution of potable water demands. Dry weather wastewater flows were assumed to be 90% of winter potable water demands. Those properties that did not have consumption data were assigned a base flow based on the winter consumption data of similar properties 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

Page 2-13



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.

Observed 1200 Modeled 1000 800 Flow (gpm) 600 400 200 0 18:00 0:00 6:00 12:00 18:00 0:00 Time of Day (Hour)

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 PIF₅ (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.

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



Influent Screen

accept septage. A small amount of ozone from a 3 HP DO₂E 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.



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.

- 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 DO₂E high volume floating aerator/mixers provide oxygen for the activated sludge process. One of the 10 HP DO₂E units recently failed and is being replaced with a 40 HP Aire-O₂ 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 DO₂E 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 DO₂E 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.

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



Secondary Clarifiers

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.

- 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.

- 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 has design capacity unit а 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.

- 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

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



UV System

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.

- The UV seals have been leaking.
- Projected future peak flows will exceed the capacity of the UV system and the Vnotch 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



Effluent Pump Station

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.

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

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



Aerobic Digester

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.

- 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



Biosolids Storage Lagoons

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.

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, BOD₅, total suspended solids (TSS), pH, and temperature. The effluent parameters include BOD₅, 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 BOD5 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.



BOD₅ 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 BOD₅ Concentrations

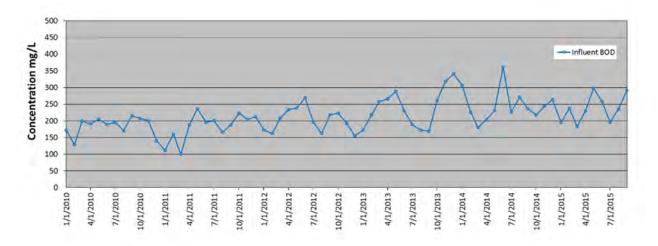
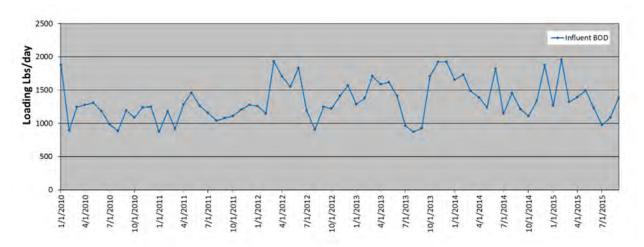


Chart 3-2: WWTP Influent BOD₅ Loading



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 BOD₅ Data

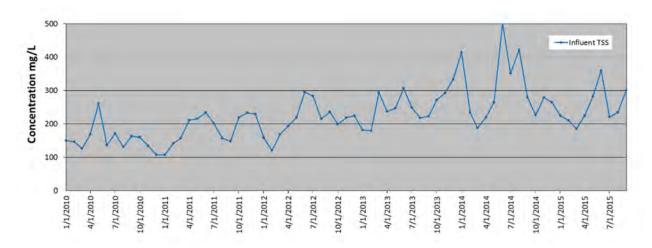
	2010	2011	2012	2013	2014	Jan Sept. 2015	Design
Population	6,592	6,622	6,652	6,683	6,714	6,745	6,745
AADF (PPD)	1,329	1,149	1,394	1,439	1,453	1,344	1,453
ADWF (PPD)	1,114	1,179	1,295	1,249	1,309	1,230	1,309
MMDWF (PPD)	1,309	1,460	1,835	1,708	1,825	1,496	1,835
AWWF (PPD)	1,553	1,119	1,494	1,635	1,596	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 BOD₅ (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







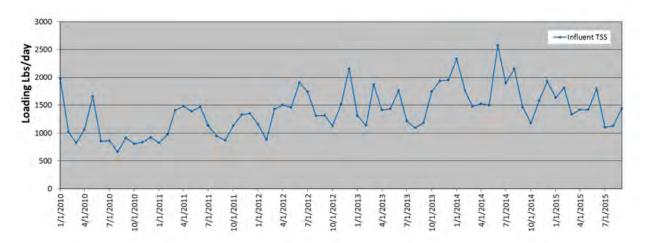


Table 3-2 shows the TSS ppcd summary. The TSS ppcd appears to be fairly constant. The typical range for TSS is shown in the table footnote. The design values for this study are also shown in Table 3-2. Since the loading rates appear to be fairly constant, the maximum year loads from 2010-September 2015 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
BOD₅										
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	ss									
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 BOD₅, TSS, pH, and E. coli bacteria.

BOD₅

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)

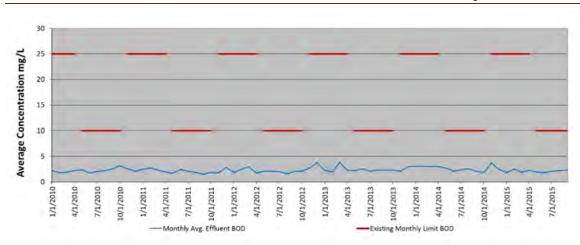




Chart 3-6: WWTP Effluent BOD₅ Concentrations (Weekly)

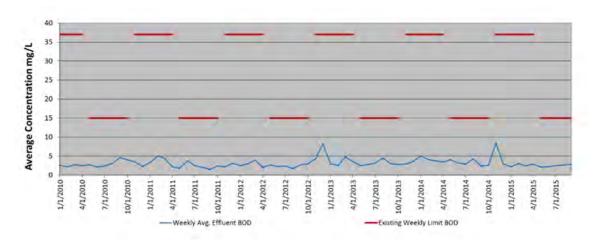


Chart 3-7: WWTP Effluent BOD₅ Percent Removal (Monthly)

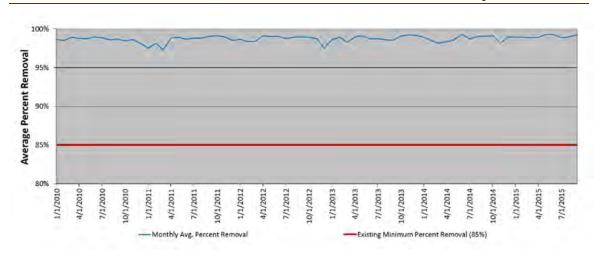


Chart 3-8: WWTP Effluent BOD₅ Loading (Average Monthly)

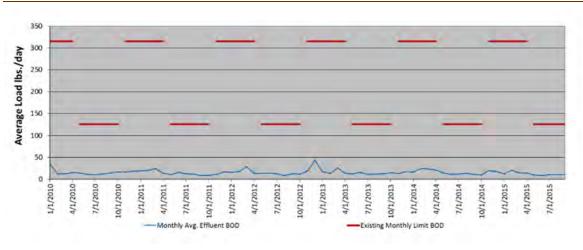




Chart 3-9: WWTP Effluent BOD₅ Loading (Average Weekly)

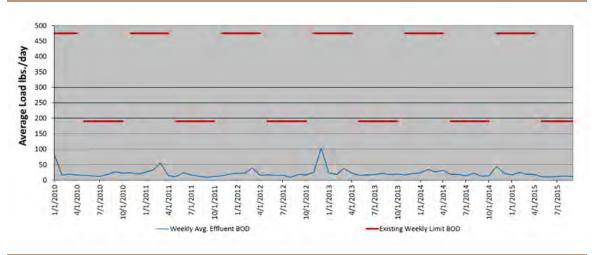
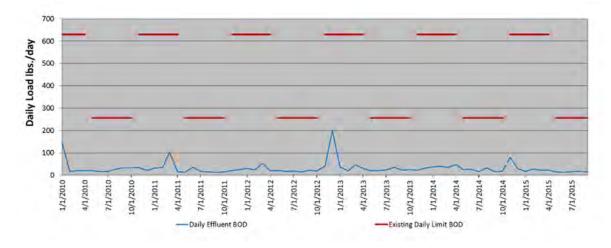


Chart 3-10: WWTP Effluent BOD₅ Loading (Maximum Daily)



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 BOD₅ 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)

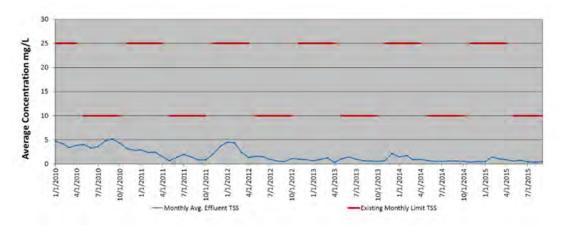


Chart 3-12: WWTP Effluent TSS Concentrations (Weekly)

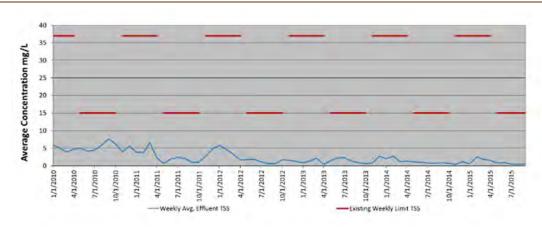


Chart 3-13: WWTP Effluent TSS Percent Removal (Monthly)

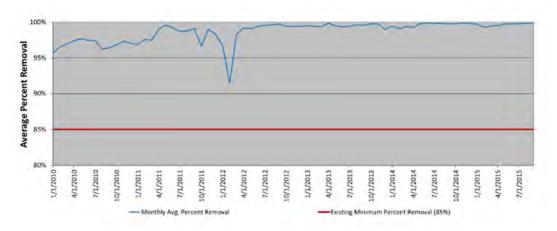




Chart 3-14: WWTP Effluent TSS Loading (Average Monthly)

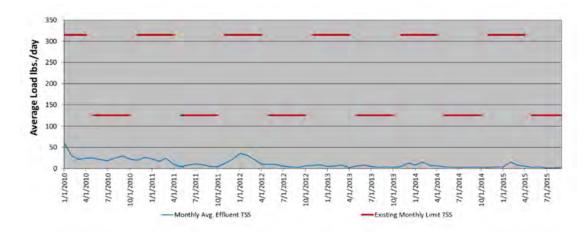


Chart 3-15: WWTP Effluent TSS Loading (Average Weekly)

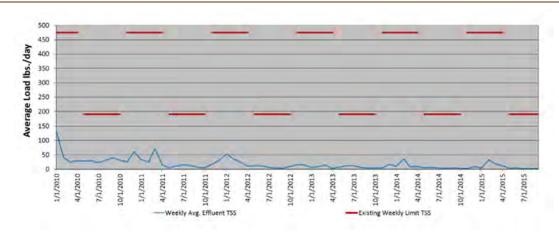
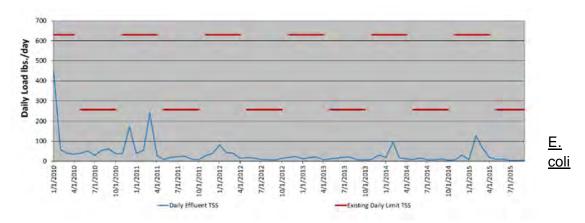


Chart 3-16: WWTP Effluent TSS Loading (Maximum Daily)



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)

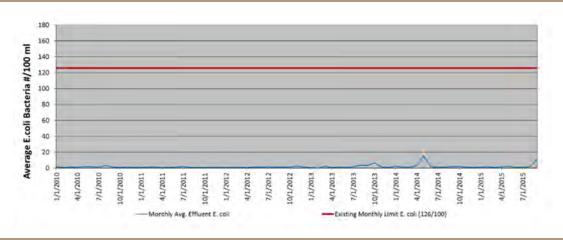
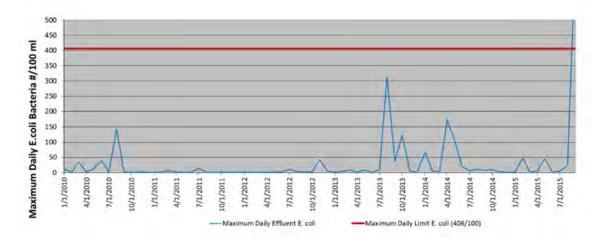


Chart 3-18: WWTP Effluent E. coli Bacteria (Daily)



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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)

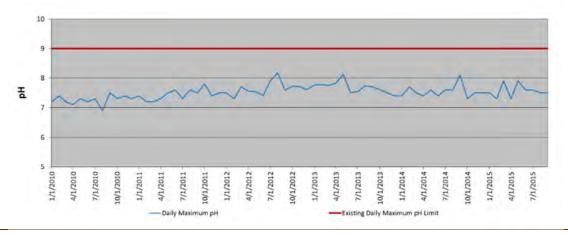
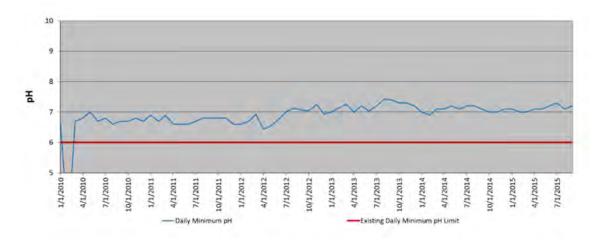


Chart 3-20: WWTP Effluent pH (Minimum Daily)



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

	Backup Equipment Condition					
Equipment		Rating	Criticality Rating	Rating		
Infl	Influent Screen		S/H, EQ, PF, CC	LN		
	uent Pumps	1	S/H, EQ, PF, CC	W/R		
	ration Basin	5	EQ, PF	W/R		
Aeratio	n Basin Aerators	1	S/H, EQ, PF, CC	M		
Secon	dary Clarifiers	5	EQ, PF	W		
R	AS Pumps	1	EQ, PF	M		
W	/AS Pumps	4	PF	M		
Intermed	iate Pump Station	4	EQ, PF	LN		
Ter	tiary Filters	1	EQ, PF	LN		
UV	Disinfection	1	S/H, EQ, PF	W/R		
Effl	uent Pumps	1	S/H, EQ, PF, CC	W/R		
Aero	obic Digester	5	EQ, PF	W/R		
Biosolids	Storage Lagoons	5	PF	W/R		
Backup Ratir	ng					
1	One level of "in kind	' redundancy	(Identical piece of equi	pment 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 Ra	Criticality Rating					
S/H	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)					
DE	Process Functionality	Risk (Loss w	ould affect the function	and/or efficiency of the		
PF	affected processes)					
СС	CC Cost Critical (Loss would have a significant cost impact in short term or long term)					
Equipment Condition Rating						
	New (Equipment is new, or replaced in last 12 months)					
N	New (Equipment is n	cw, or replac				
LN	<u> </u>		·	verhauled to a condition like		
	Like New (Equipmen new) Used But Maintained	t is operated (Equipment	·			
LN	Like New (Equipmen new) Used But Maintained maintained and funct	t is operated (Equipment ions well) ment close to	very little or recently or showing expected wea			



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 (4 th 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 (3 rd 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 (4 th is redundant)
Effluent Pipe	4.0 (PIF ₅)	2.6	7.2	

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 PIF₅ 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
рН		Daily minimum and maximum between 6.0 and 9.0		
E. coli Bacteria		126/100 mL	-	406/100 mL
Ammonia	29.7 mg/L	-	-	-

 $^{^{1}}$ BOD₅ = 5-day biochemical oxygen demand

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

² ppd = pounds per day

³ TSS = total suspended solids



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 predesign 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:

- 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	0
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

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.

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.



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.
- 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 A²O (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.

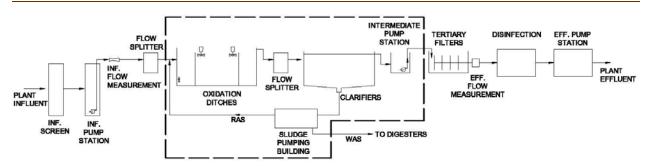
INTERMEDIATE PUMP TERTIARY FILTERS DISINFECTION EFF. PUMP STATION FLOW SPLITTER PLANT EFFLUENT EFF. MEASUREMENT AERATION BASINS PLANT CLARIFIERS PUMP TO DIGESTERS SLUDGE PAW PUMPING BUILDING

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.

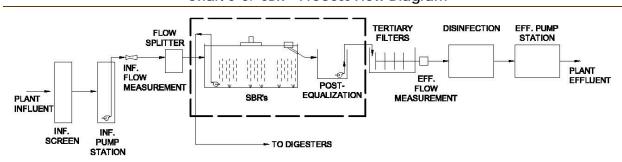


Chart 5-2: Oxidation Ditches - Process Flow Diagram



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



- 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.
- 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:

 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:

- 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:
 - 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

				WWTE	WWTP Alternatives			
Environmental		WWTP Disposal		H 4TWW	WWTP Headworks	S	Secondary Treatment	t
Criteria	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	NoImpact	No Impact	No Impact	No Impact	No Impact	No Impact
Cultural Resources	No Impact	None known	None known	No Impact	NoImpact	No Impact	No Impact	No Impact
Biological Resources	No Impact	No Impact	NoImpact	No Impact	No Impact	No Impact	No Impact	No Impact
Water Quality Issues	No Impact	Less volume (loading)	Less volume (loading)	Improved effluent quality	mproved effluent Improved effluent quality quality	Improved effluent quality	Improved effluent Improved effluent quality 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 (Ioading)	Less volume (Ioading)	No Impact	No Impact	More energy used	More energy used 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

			WWTP Alternatives Cont'd.	atives Cont'd.				Collection
Environmental		WWTP Disinfection		Biosolids 7	Biosolids Treatment	Disposal A	Disposal Alternatives	System
Criteria	۸n	Chlorine	PAA	Aerobic Digester	Anaerobic Digesters	No Action	Class A - Composting	All
Land Use/Important Farmland/Formally Classified Lands	No Impact	NoImpact	No Impact	No Impact	NoImpact	No Impact	NoImpact	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	NoImpact	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	NoImpact	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	A/N	N/A	W/N	N/A	N/A	N/A
Socio-Economic/ Environmental Justice More energy used More	More energy used		chemicals used More energy used	More energy used	More energy used	No Impact	More energy used	NoImpact
Miscellaneous Issues	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	No Impact	NoImpact



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

ltem	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

ltem	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.

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

Table 6-3: New Headworks Cost Estimate

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 A²O 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	 Same technology as used currently at WWTP. 	 Requires construction of additional clarifiers.
(New Aeration Basins)	 Effluent flow equilization is not required. 	Requires more equipment than other alternatives.
(New Actation Busins)		Can be sensitive to peak flows or loads.
	 Resilient to peak flows. 	 Requires construction of additional clarifiers.
	 Effluent flow equilization is not required. 	Effluent suspended solids concentrations can be
Oxidation Ditch		higher than the other two alternatives.
		The aeration system may create more aerosols.
		 Largest footprint of the alternatives.
	 Lowest capital and 20 year life-cycle costs. 	 Most automated technology of the three.
	 Changes can be quickly made based on cycle time. 	Effluent equalization is recommended so
Sequencing Batch Reactor (SBR)	- Changes can be quickly made based on cycle time.	downstream processes can handle the decant flow.
Facility	Smallest footprint.	Peak flows can disrupt process. Influent
	- Smartest rootprint.	equalization may be desired.
	Does not require clarifiers.	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%)	\$ 130,00			
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

Technology	Advantages	Disadvantages				
	Same technology as used currently at WWTP	Low dosage may not effectively inactivate some viruses, spores, and cysts.				
	 Eliminates the need to generate, handle, transport, or store toxic/hazardous or corrosive chemicals. 	 Organisms can sometimes repair and reverse the destructive effects of UV. 				
Upgrade and Expand UV System	No residual effect that can be harmful to humans or aquatic life.	A preventive maintenance program is necessary to control fouling of tubes.				
	Requires shorter contact time compared to other disinfectants (approximately 20 to 30 seconds with low-pressure lamps).	 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. 				
	Requires less space than other methods.	 Not as cost-effective as chlorination, but costs are competitive when chlorination and de-chlorination is used and fire codes are met. 				
	Well-established technology.	 Chlorine residual, even at low concentrations, is toxic to aquatic life and will require a well-controlled de-chlorination system. 				
	 Can be more cost-effective than UV disinfection (dechlorination and fire code requirements can make it cost more than UV disinfection). 	 All forms of chlorine are highly corrosive and toxic, so storage, shipping, and handling pose a risk, requiring increased safety regulations. 				
	 Chlorine residual remaining in the effluent can prolong disinfection even after initial treatment, and can be measured to evaluate effectiveness. 	 Oxidizes some organic matter in wastewater to create more hazardous compounds (disinfection byproducts such as trihalomethanes [THMs] are regulated and would require additional treatment). 				
Implement Chlorination and Dechlorination	Reliable and effective against a wide spectrum of pathogenic organisms.	Level of total dissolved solids is increased in the treated effluent.				
	Effective in oxidizing certain organic and inorganic compounds.	 Chlorine residual is unstable in the presence of high concentrations of chlorine-demanding materials, thus requiring higher doses to effect adequate disinfection. 				
	Beneficial for recycled water to have a chlorine residual for pipeline maintenance.	Some parasitic species have shown resistance to low doses of chlorine.				
	Flexible dosing control.	 Long-term effect of discharging de-chlorinated compounds into the environment is unknown. 				
	 Can eliminate certain noxious odors during disinfection. 					
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.	 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. 				
	Enhances UV effectiveness and reduces cleaning frequency when combined with UV.	Does not maintain a residual in the effluent.				
	mequency meneronismed war ov.	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				
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				
	Lower capital cost and 20 year life-cycle cost.	More energy/power required to aerobically treat sludge than to anaerobically treat sludge.				
	The process is easier to start-up and control.	Aerobically digested solids are more difficult to dewater than anaerobically digested solids.				
Aerobic Digestion	Lower ammonia and BOD in the return stream.	Temperatures are not controlled, so the process fluctuates with changes in temperature.				
	Lower potential for odors. Explosive gases do not					
	have to be dealt with.					
	 Same technology as used currently at WWTP. 					
	The biogas produced by the anaerobic digesters potentially can be used to heat the digesters.	Higher capital cost and 20 year life-cycle cost.				
Angerchie Digestion	 Less energy/power required to treat sludge. 	More difficult to start-up and control.				
Anaerobic Digestion	Sludge is more easily dewatered.	Higher ammonia and BOD in the return stream.				
	More sludge digestion occurs (lower sludge disposal costs).	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	\$ 6,500,00			
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

ltem		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.



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.

Lifespan Cost/Year Item \$ **Pipelines** 75 year 408,000 \$ Manholes 50 year 102,000 \$ Cleanouts 50 year 4,000 \$ Laterals/Cleanouts 50 year 81,000 Ś Total 595,000

Table 7-1: Replacement Budgets

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 A²O 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#		Opinion of Probable Costs					able Costs (2	2016	Dollars)
ID#	ltem		Cost	2017		2018			2019
Waste	water 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
Waste	water 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#	ltem	WWTP Flow Trigger	Additional EDUs to Meet Flow Trigger		tal Estimated Cost (2016)	SDC Growth Portion Cost			Cit	ity's Estimated Portion	
Durin with	44		Weet flow frigger		2031 (2010)	%		Cost		FOILIOII	
	1 Improvements										
	rater Collection System	l		۱ ۵	4 720 000	200/		546.000	٠,	1 201 000	
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	
	rater Treatment			۱,	2 522 222	220/	_	040.000		1.607.000	
1a 1b	Interim Biosolids Plan Rehabilitate UV System	Beyond Capacity		\$	2,530,000 373,000	33% 15%	\$	843,000 55,000	\$	1,687,000 318,000	
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	
	pact (20 yr, 1.6%)	1		\$	18.77				\$	9.33	
	2 Improvements										
Wastew	rater Collection System	1									
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	
Wastew	ater Treatment	T									
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	
	iority 2 Improvements (rounded)	l		\$	20,710,000		\$ 1	12,830,000	\$	7,880,000	
Priority 3 Improvements											
	rater Collection System	1					_		_		
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	
	rater Treatment	0.644004	2.655	۲.	2 222 22- 1	10000	٠,	2 22 2 2 2 2	٠,		
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	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		
	4 Improvements										
Wastew	ater Collection System	1	Г		-						
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		\$3	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%).

Page 7-7



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

	nual Paymet years, 1.6%)	er Rate crease	Use	r Rate Total
Existing User Rate (2016)			\$	43.31
Priority 1 Improvements ^{1,2}	\$ 302,343	\$ 9.33	\$	52.64

¹ Including capital improvements only.

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.

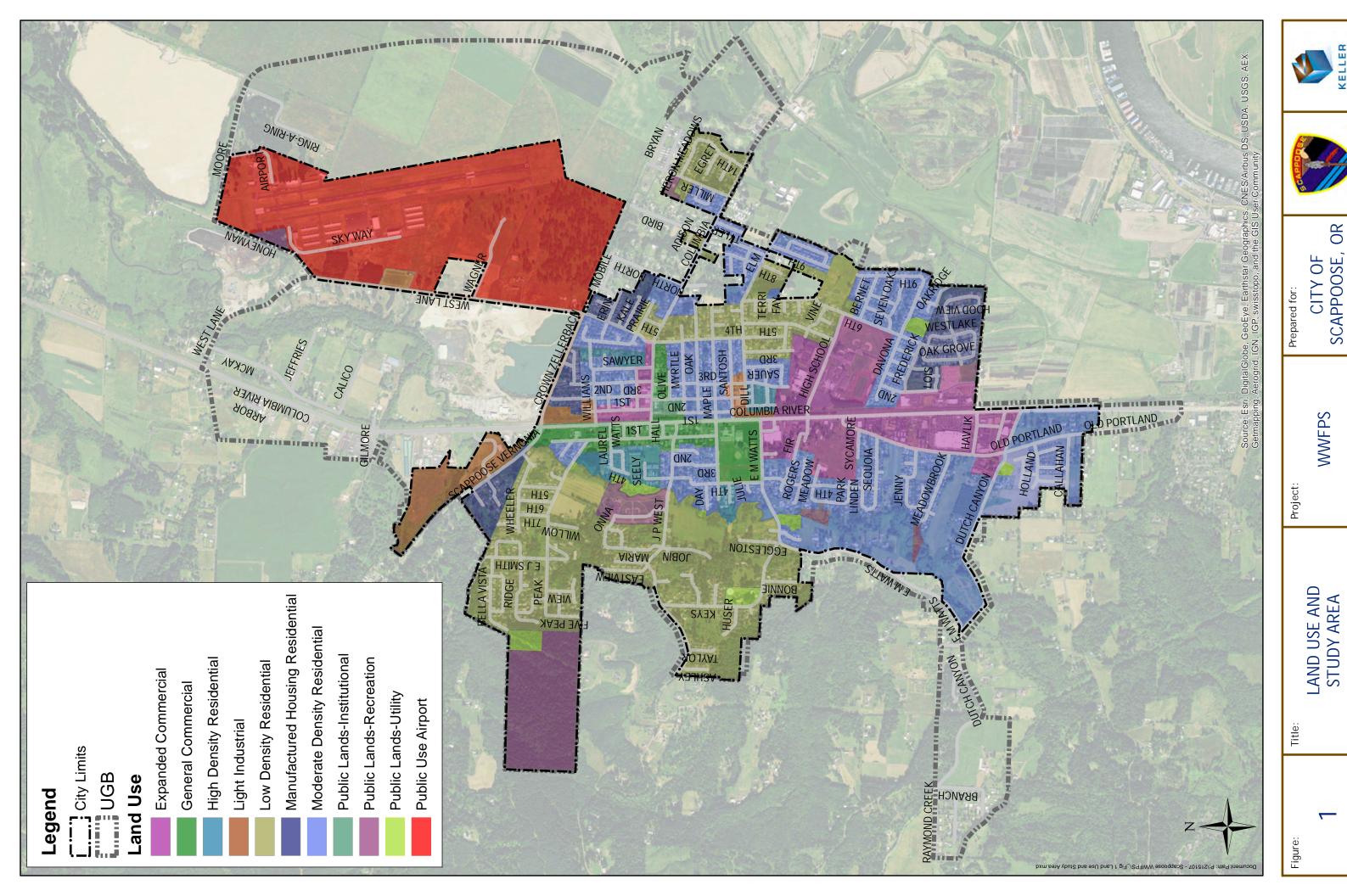
² Assuming SDCs are funding portion attributed to growth.

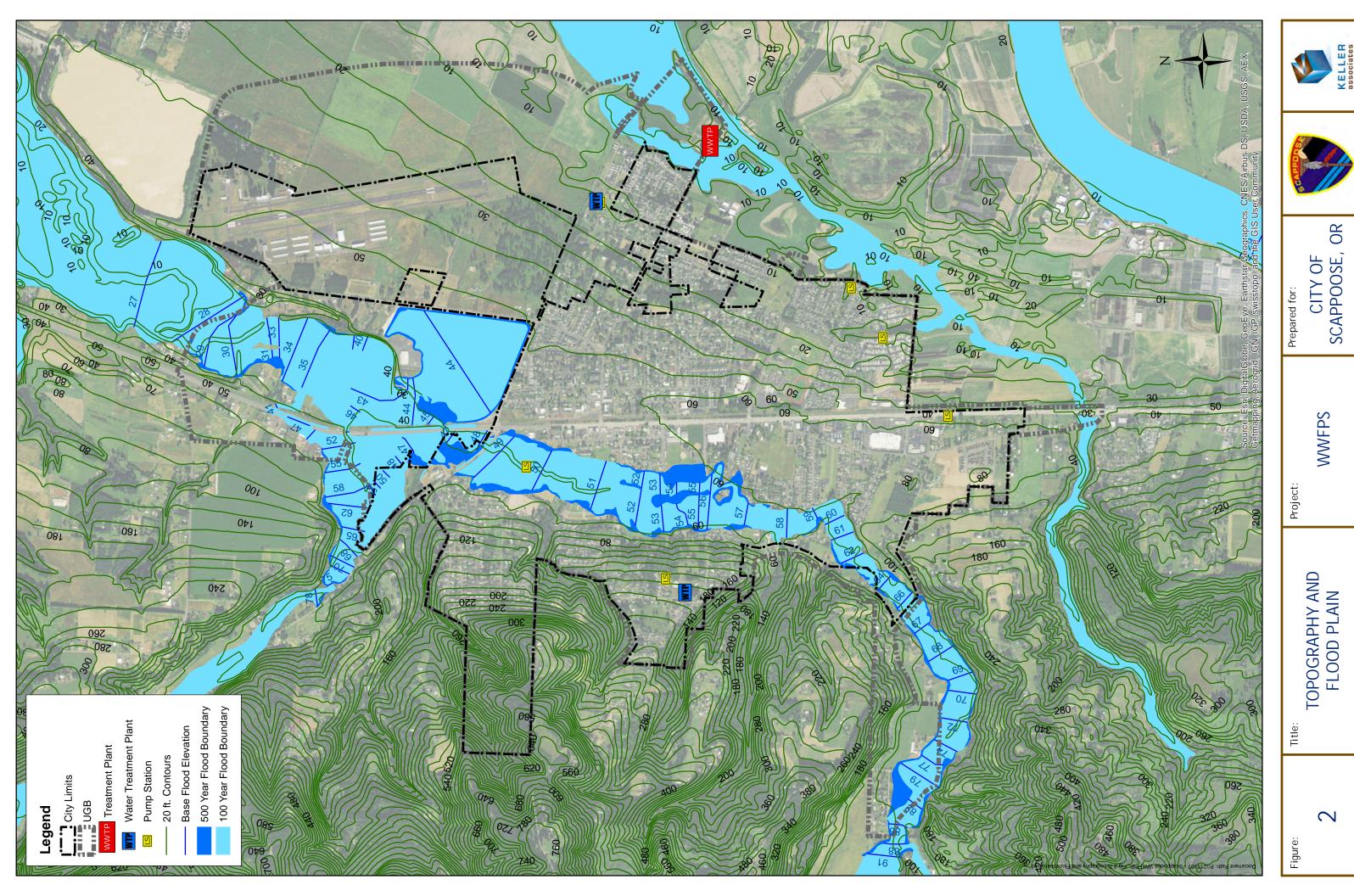


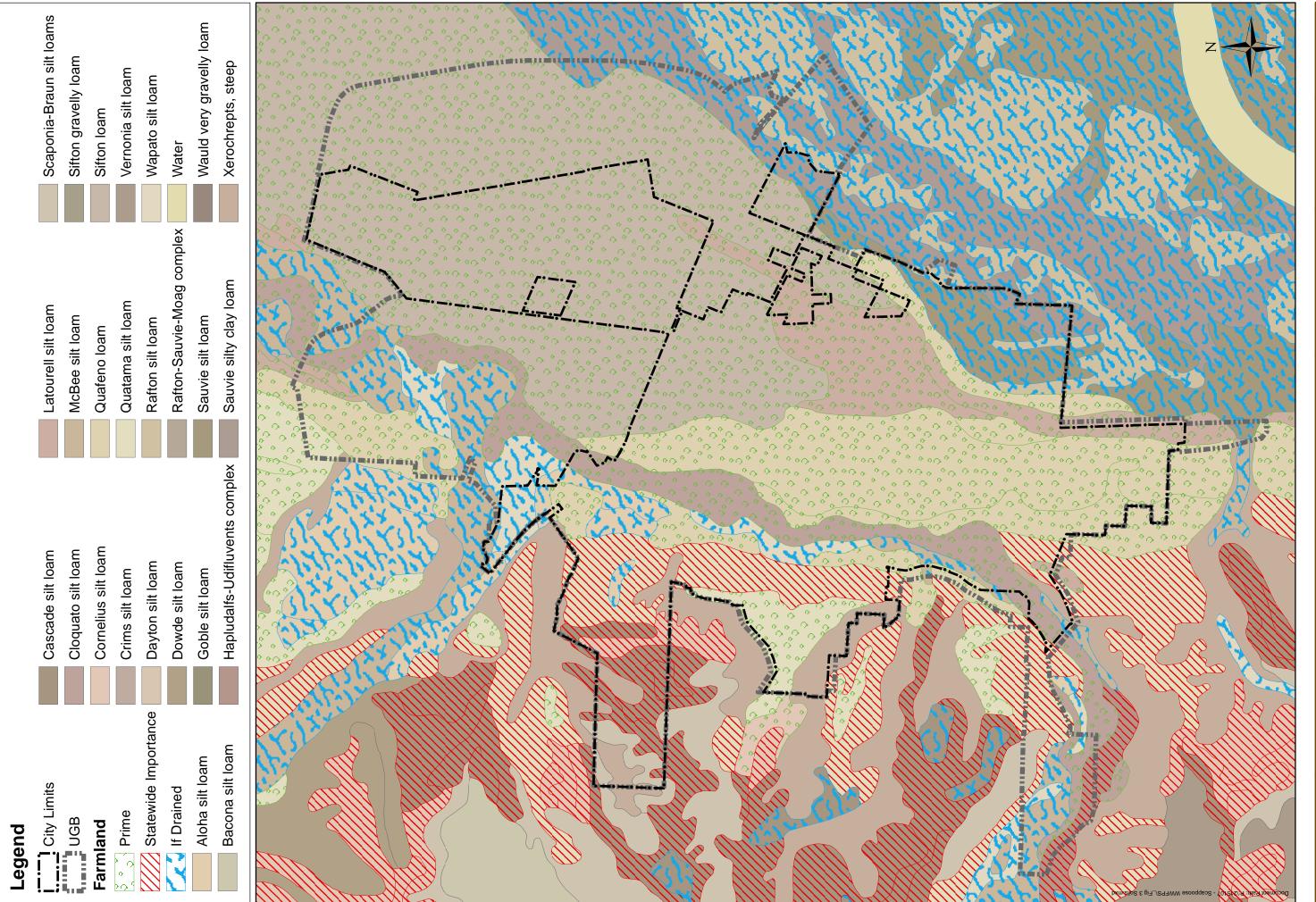
 Other additional annual O&M costs and short-lived assets associated with Priority 1 WWTP CIP (dewatering equipment, additional aeration, filters, pumping, disinfection, etc.) are anticipated to be approximately \$200,000/year. Short-lived assets include pump replacements, motor replacements, etc. A large portion of this is associated with increased power usage.



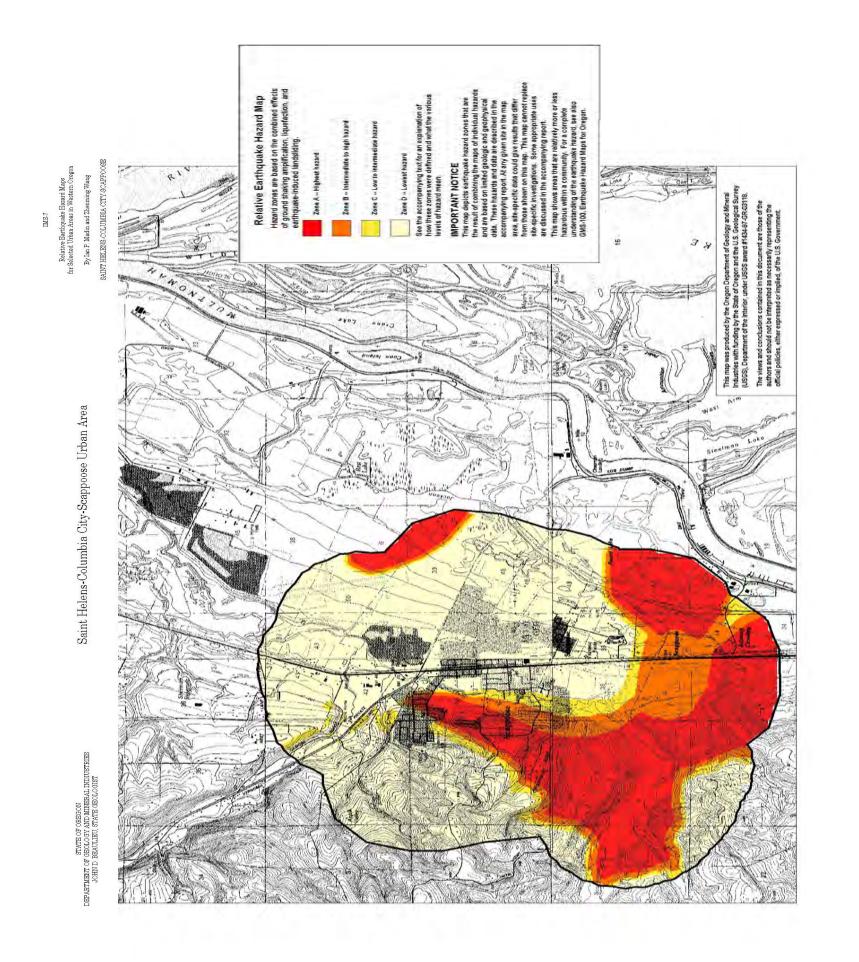
APPENDIX A - Figures











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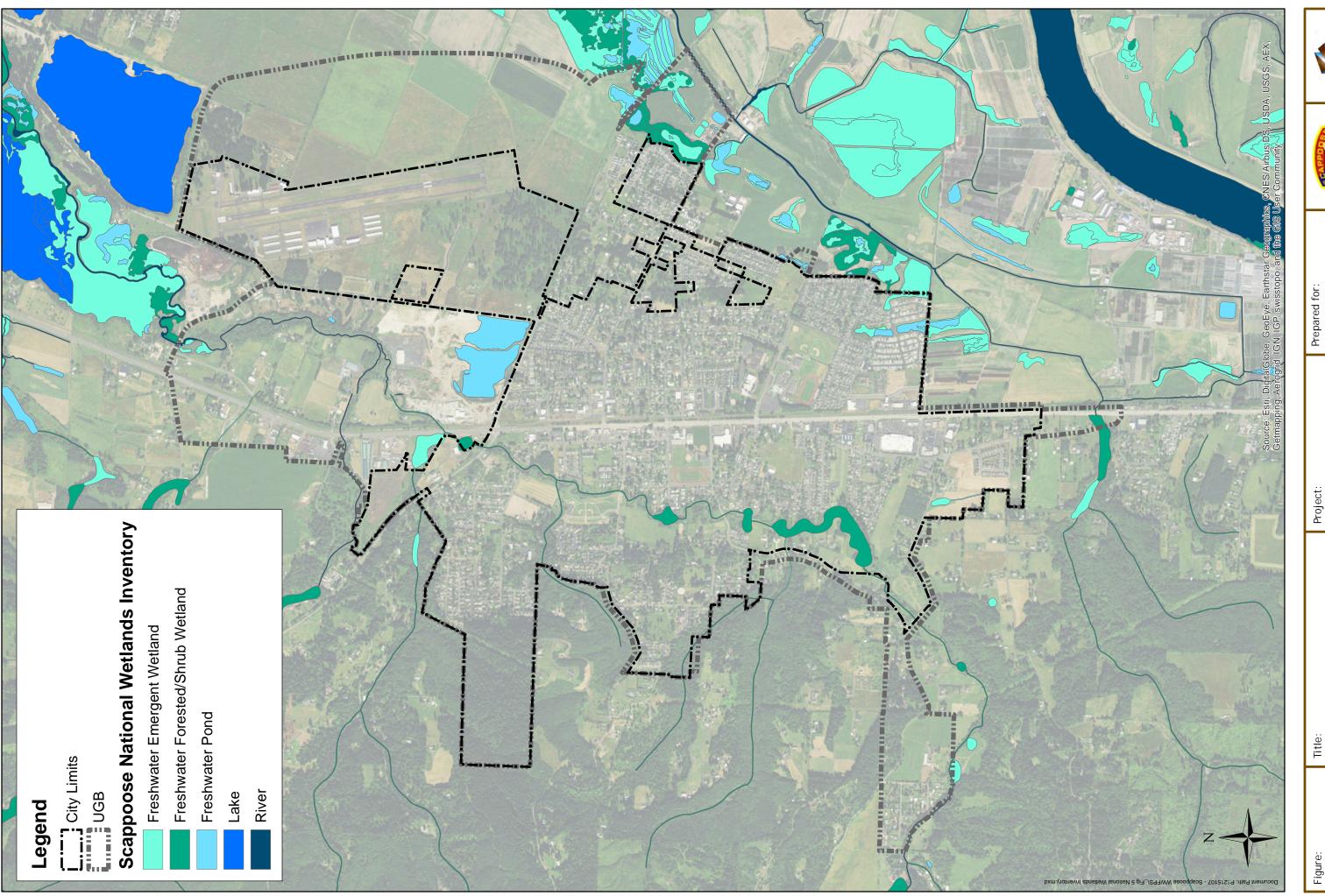
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RELATIVE EARTHQUAKE
HAZARD MAP

CITY OF SCAPPOOSE, OR Prepared for:







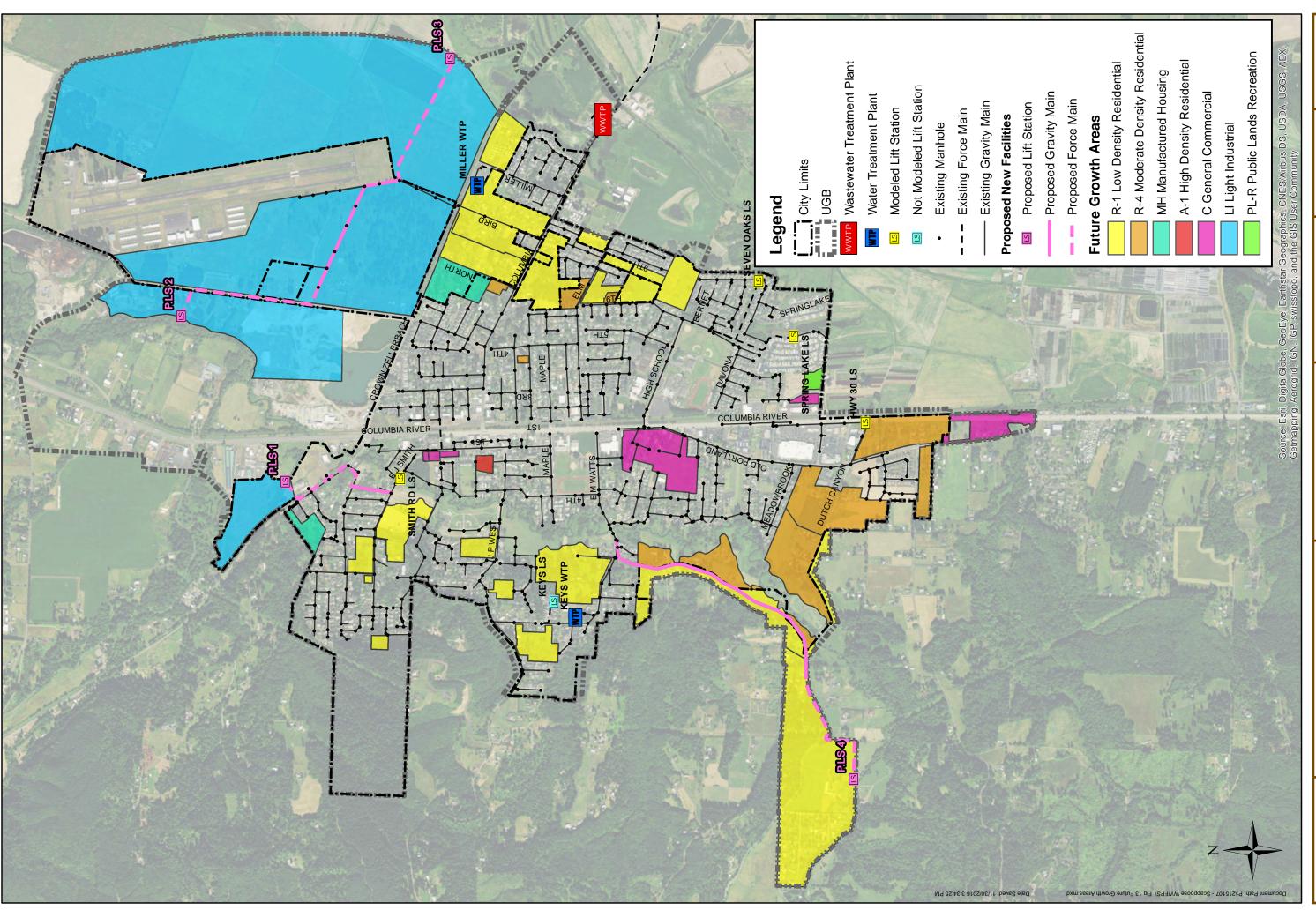


NATIONAL WETLANDS INVENTORY Title:

WWFPS Project:





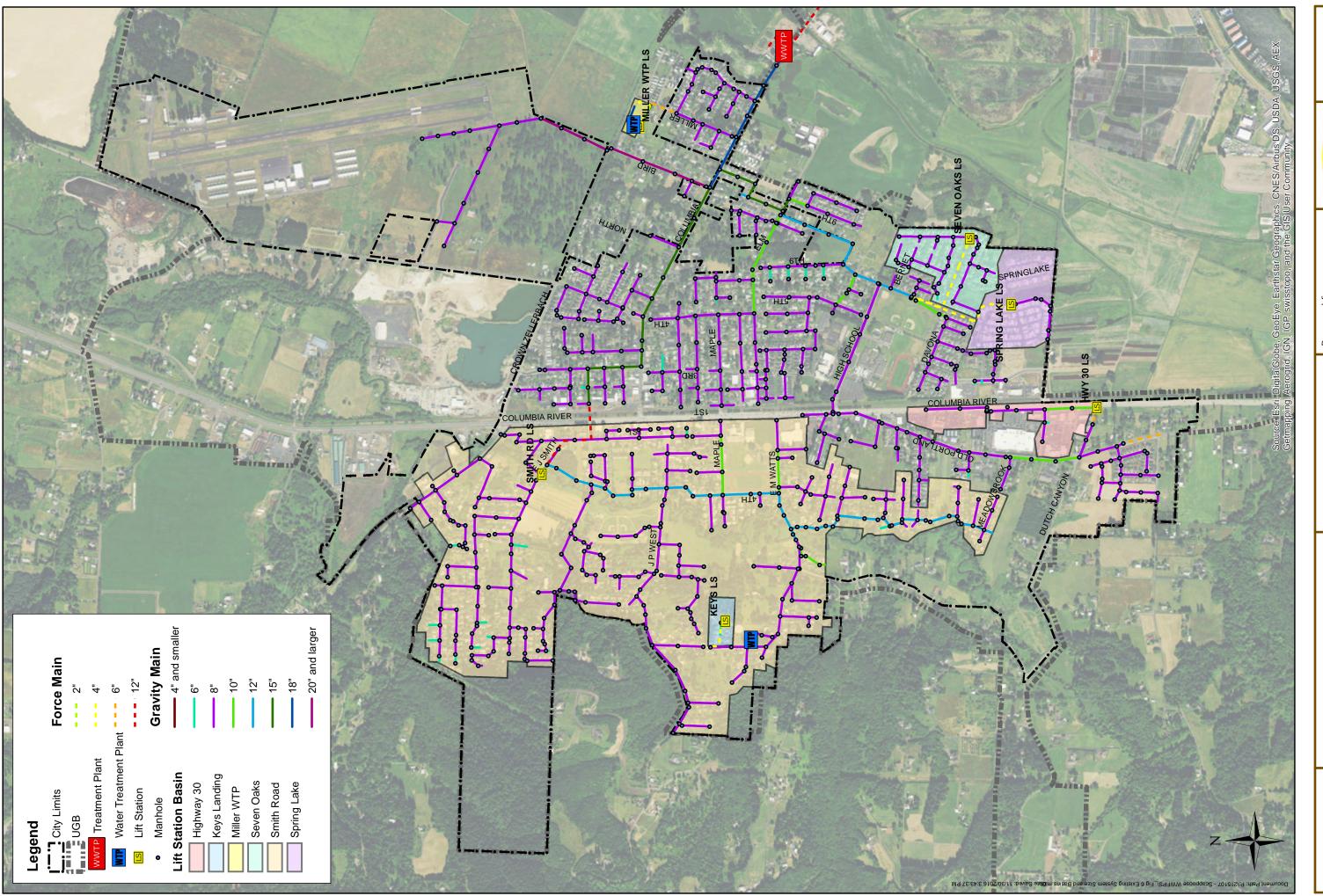


FUTURE GROWTH AREAS

WWFPS Project:







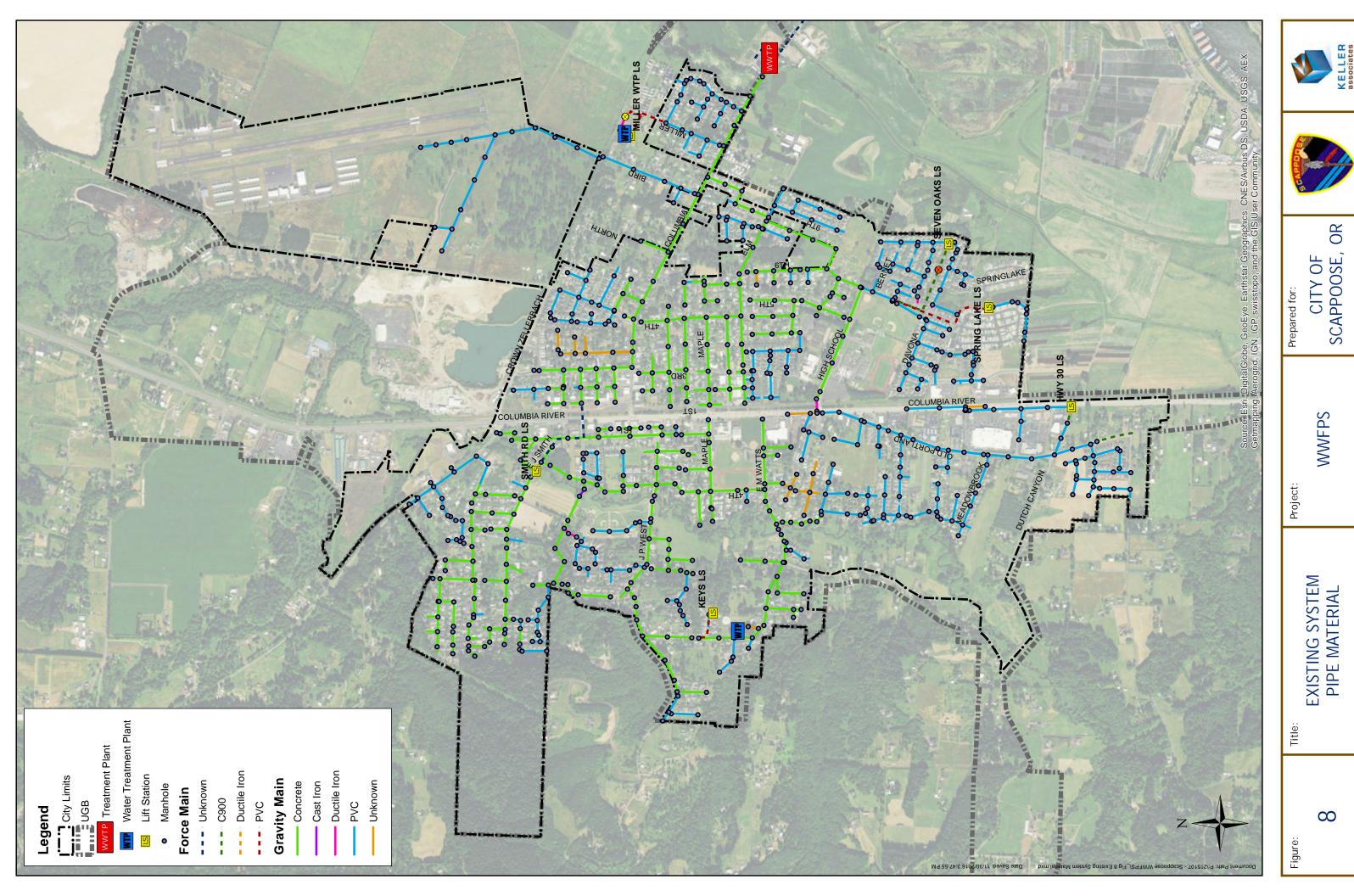
Title: EXISITING SYSTEM
PIPE DIAMETER
& LIFT STATION BASINS

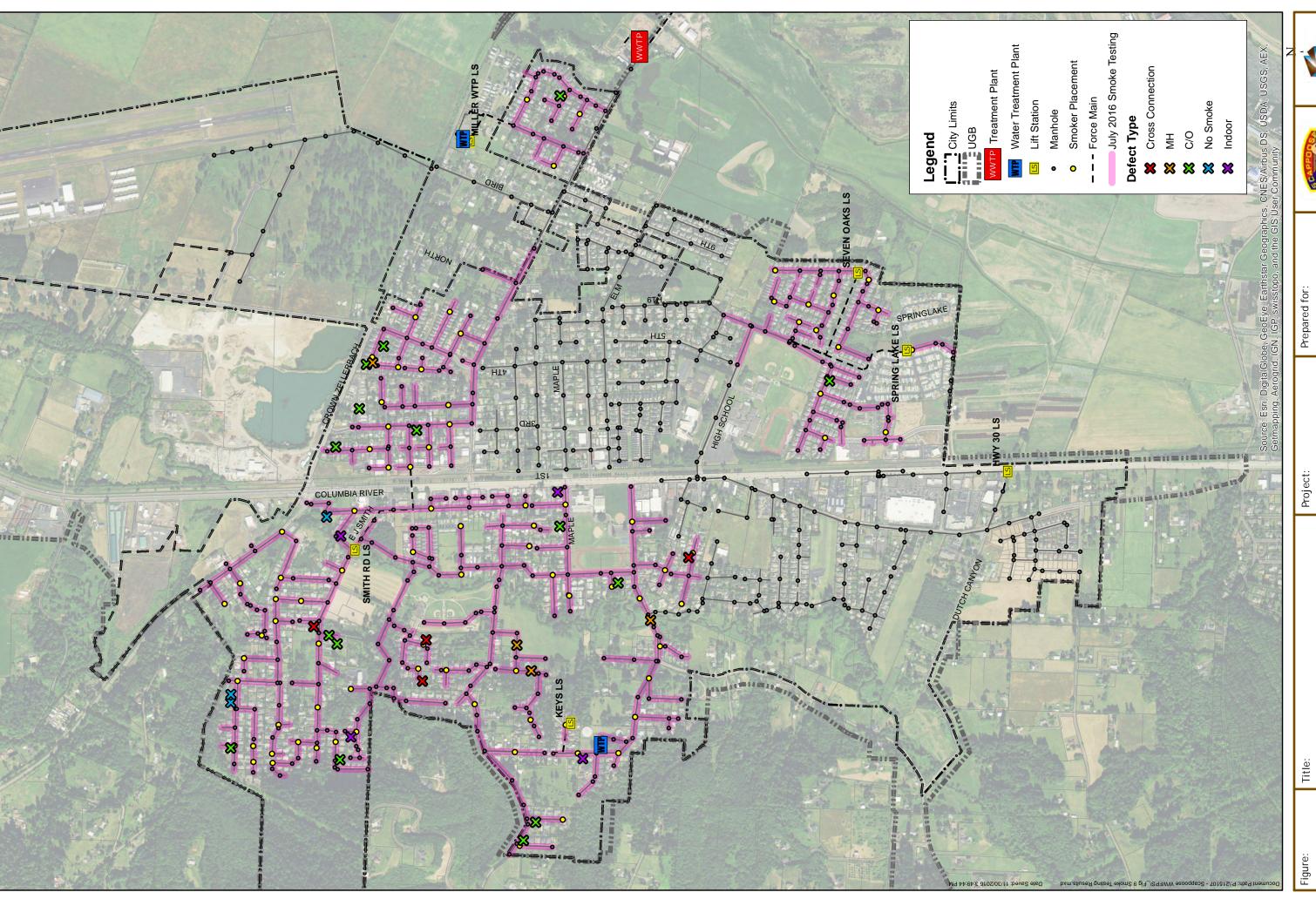
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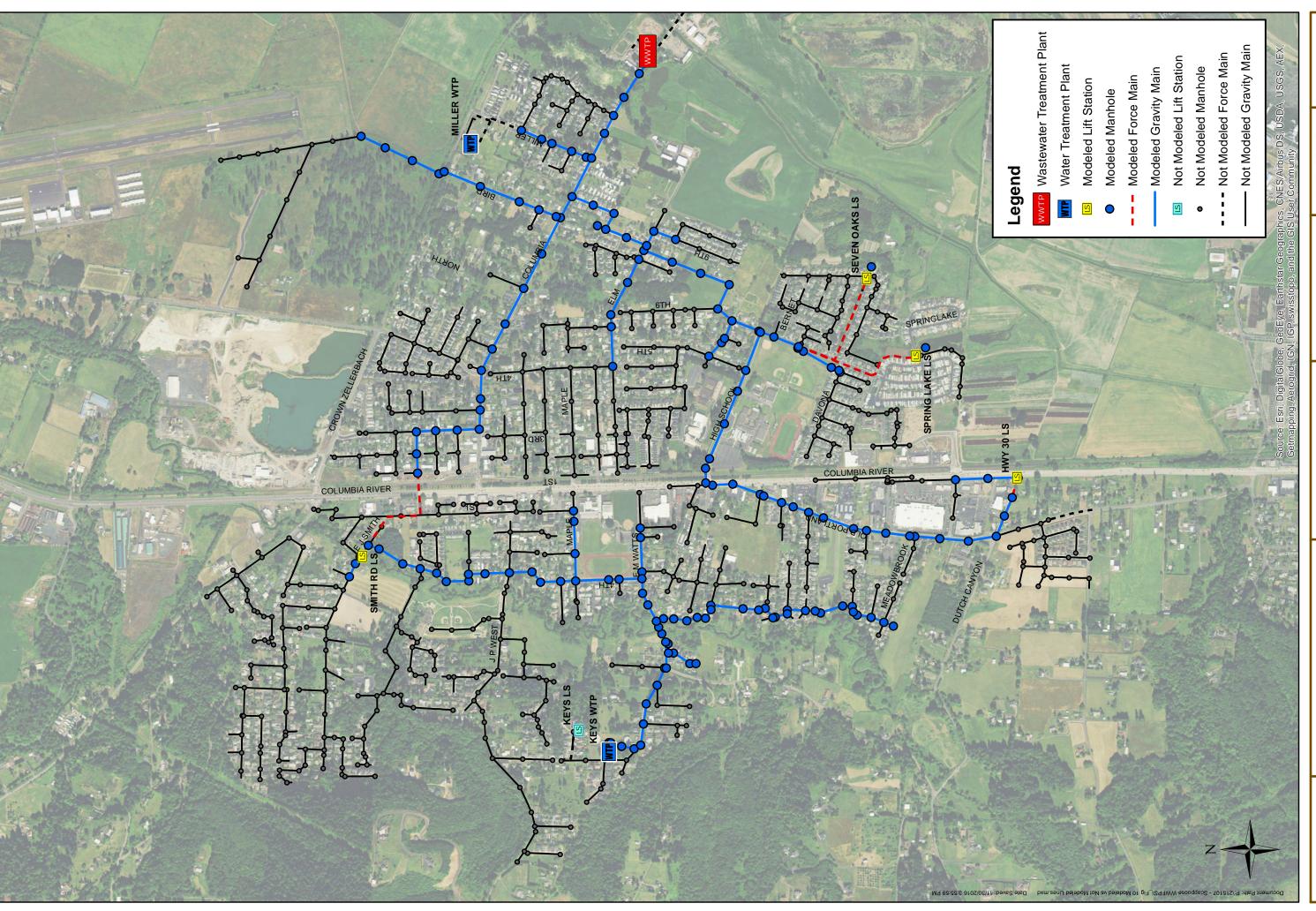












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MODELED VS NOT MODELED MH AND LINES

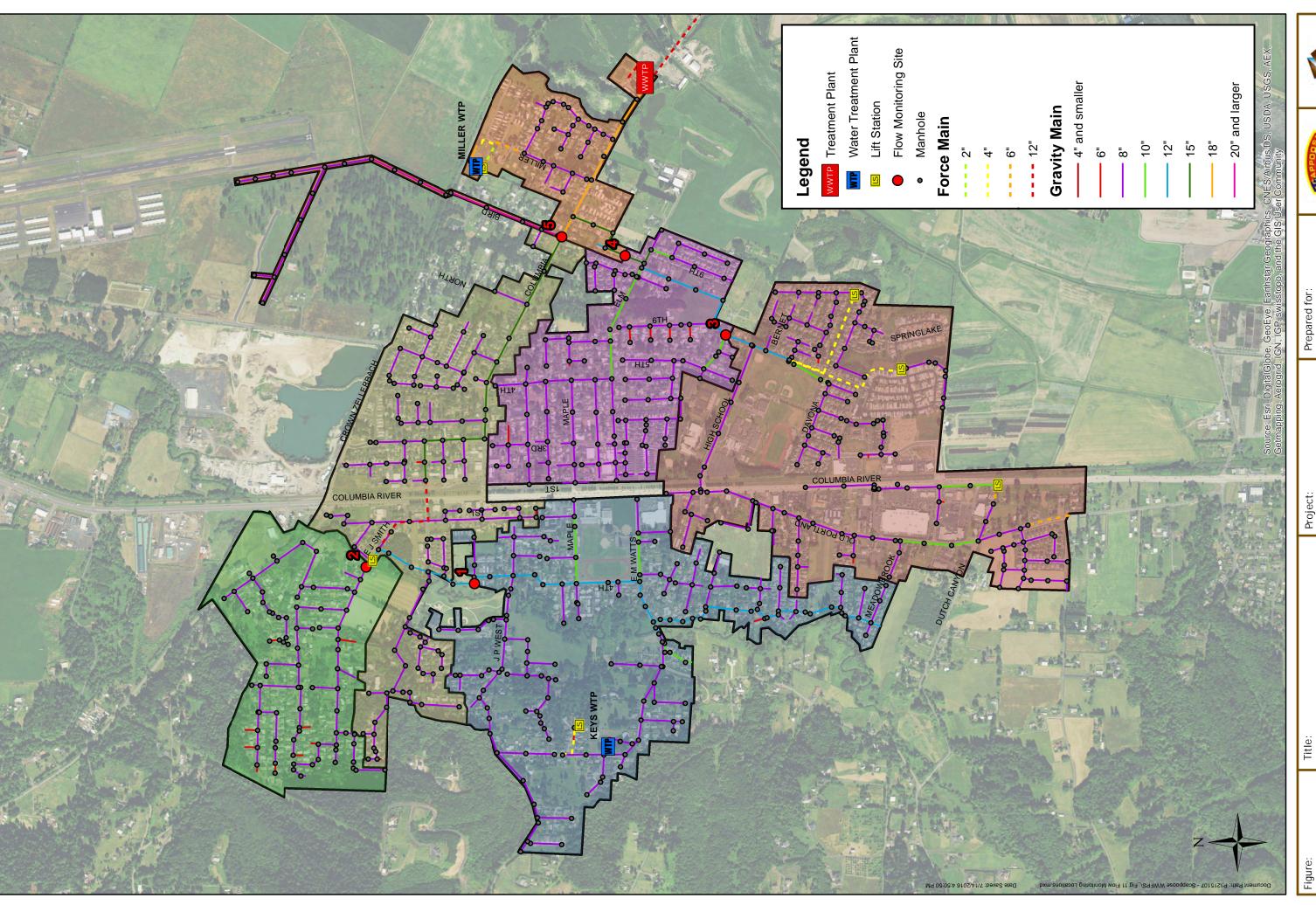
Project:

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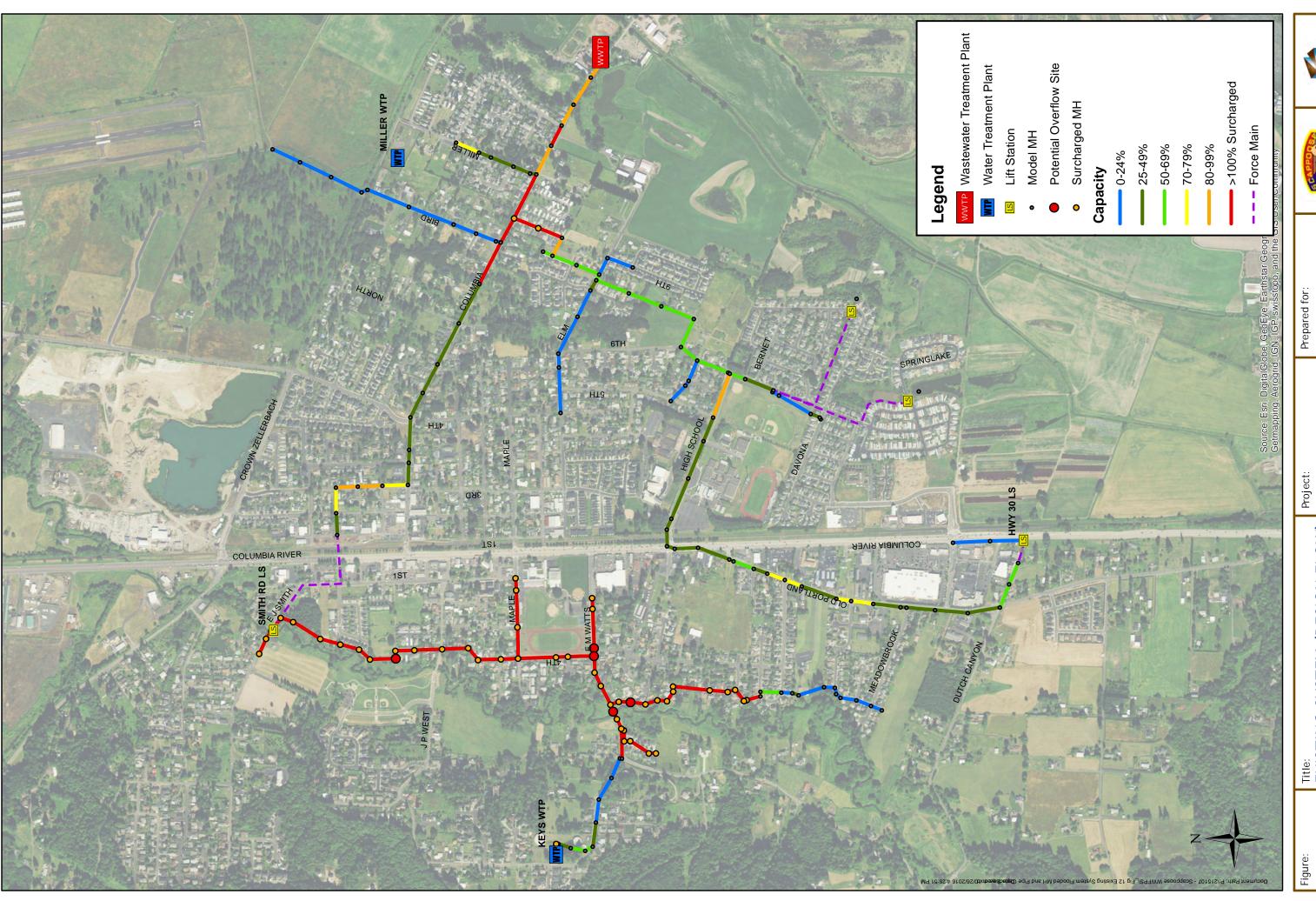
FLOW MONITORING LOCATIONS/BASINS

Project:

WWFPS



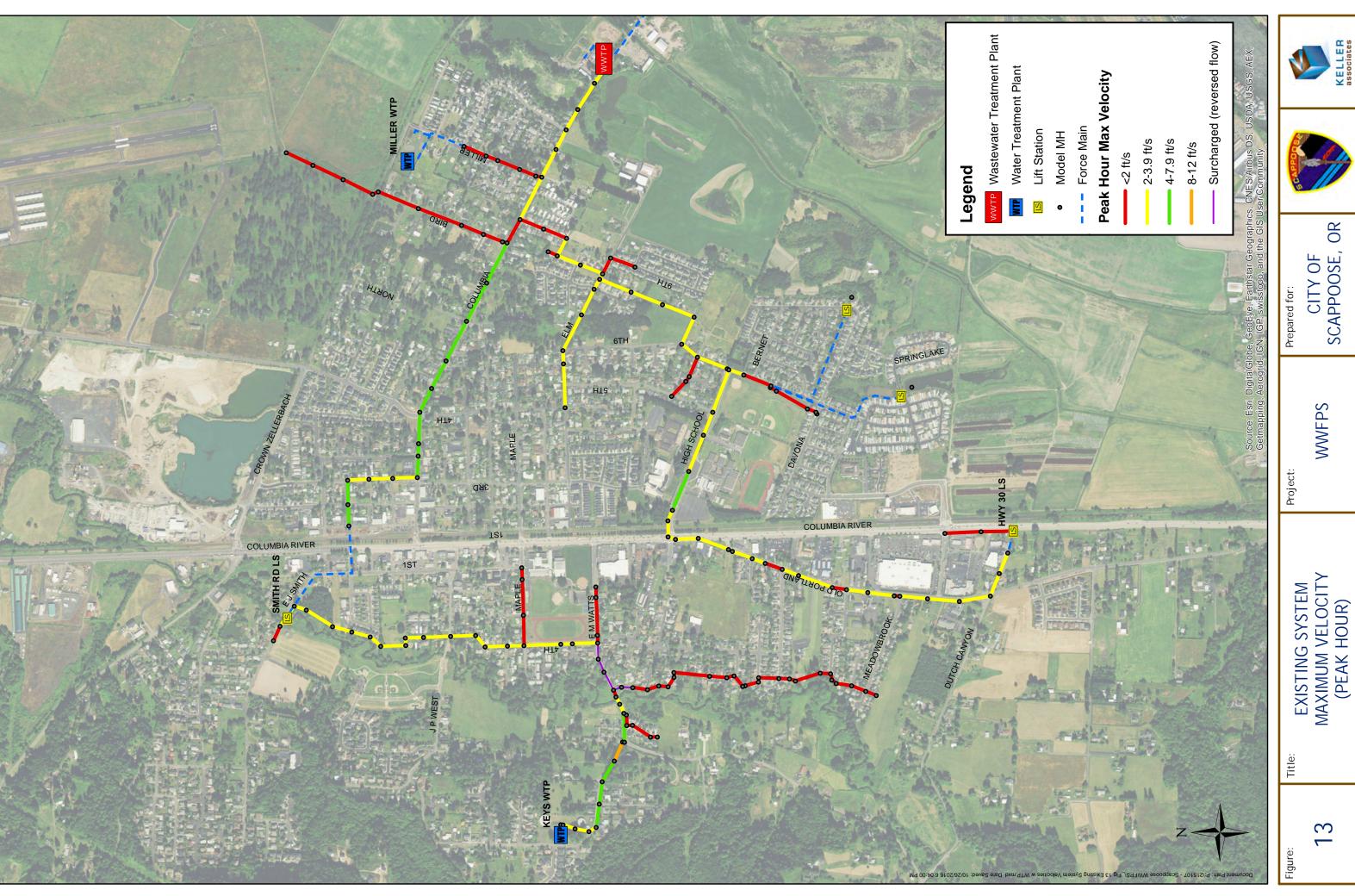




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EXISTING SYSTEM (DESIGN FLOW)
POTENTIAL OVERFLOW SITES
AND PIPE CAPACITY

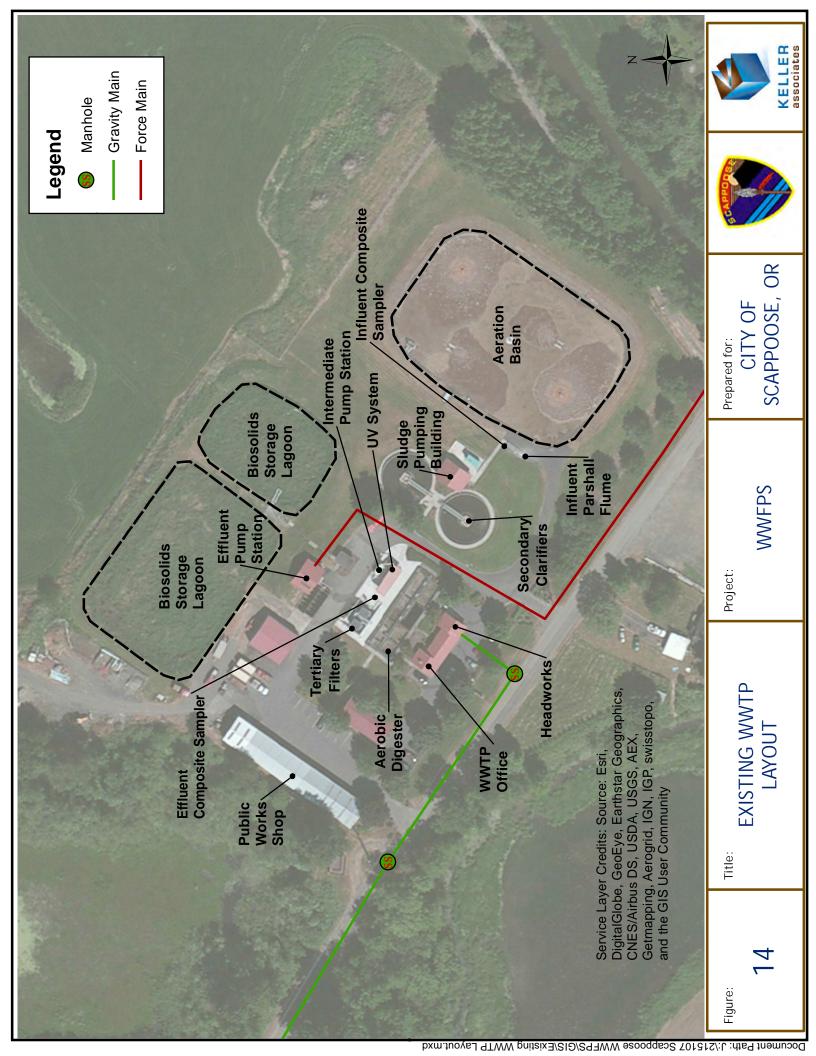


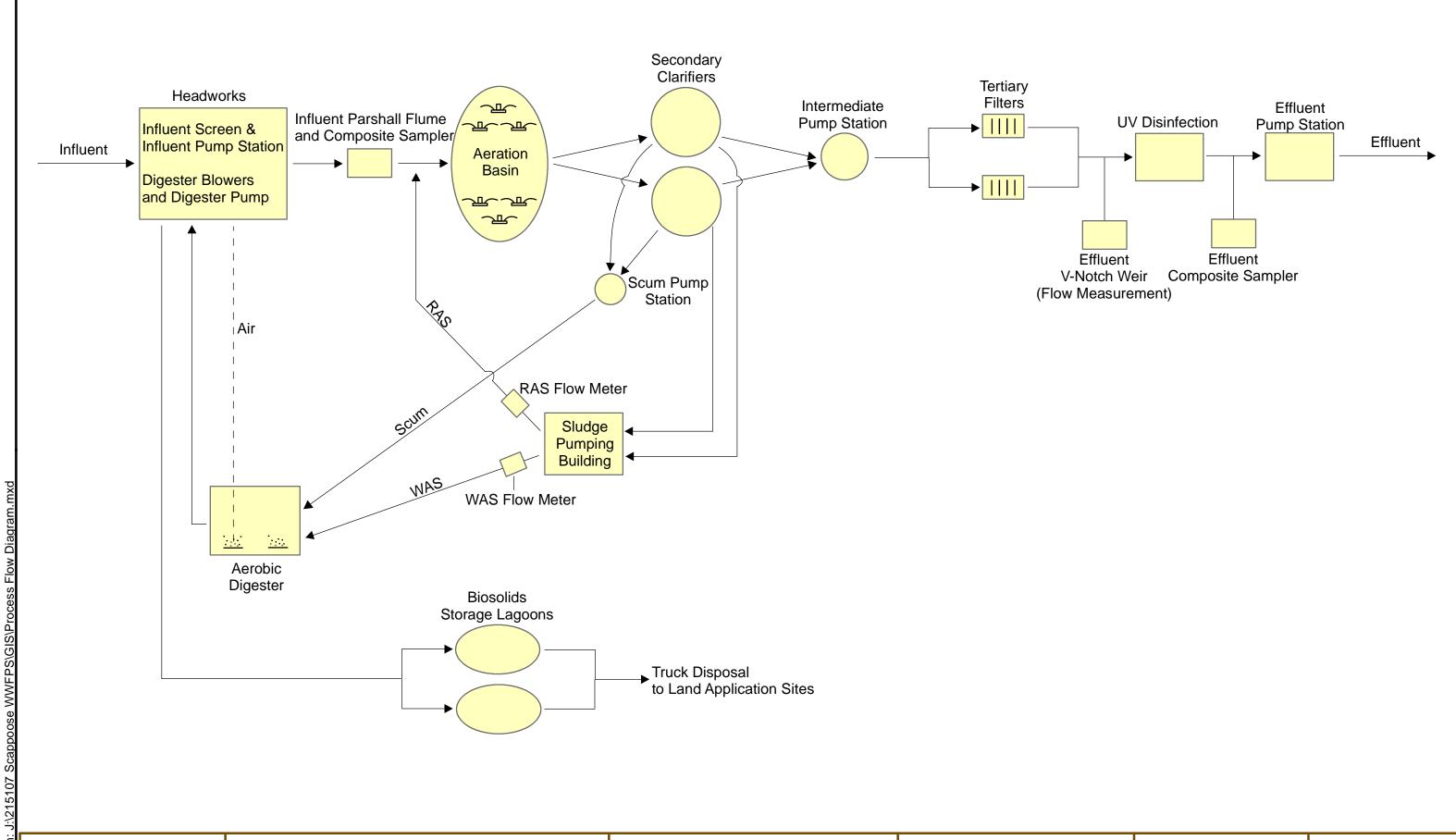












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PROCESS FLOW DIAGRAM

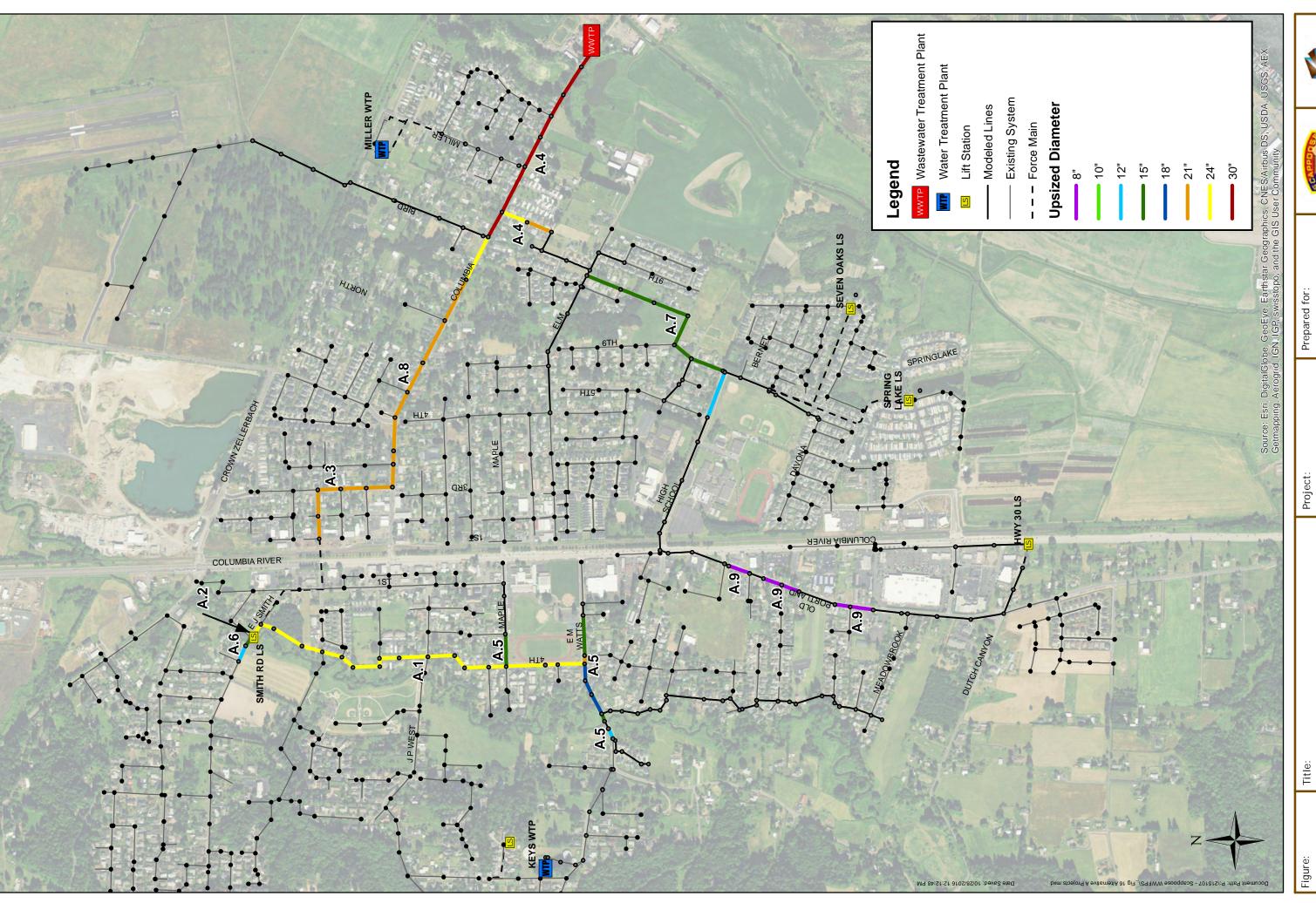
Project:

WWFPS

Prepared for:
CITY OF
SCAPPOOSE, OR

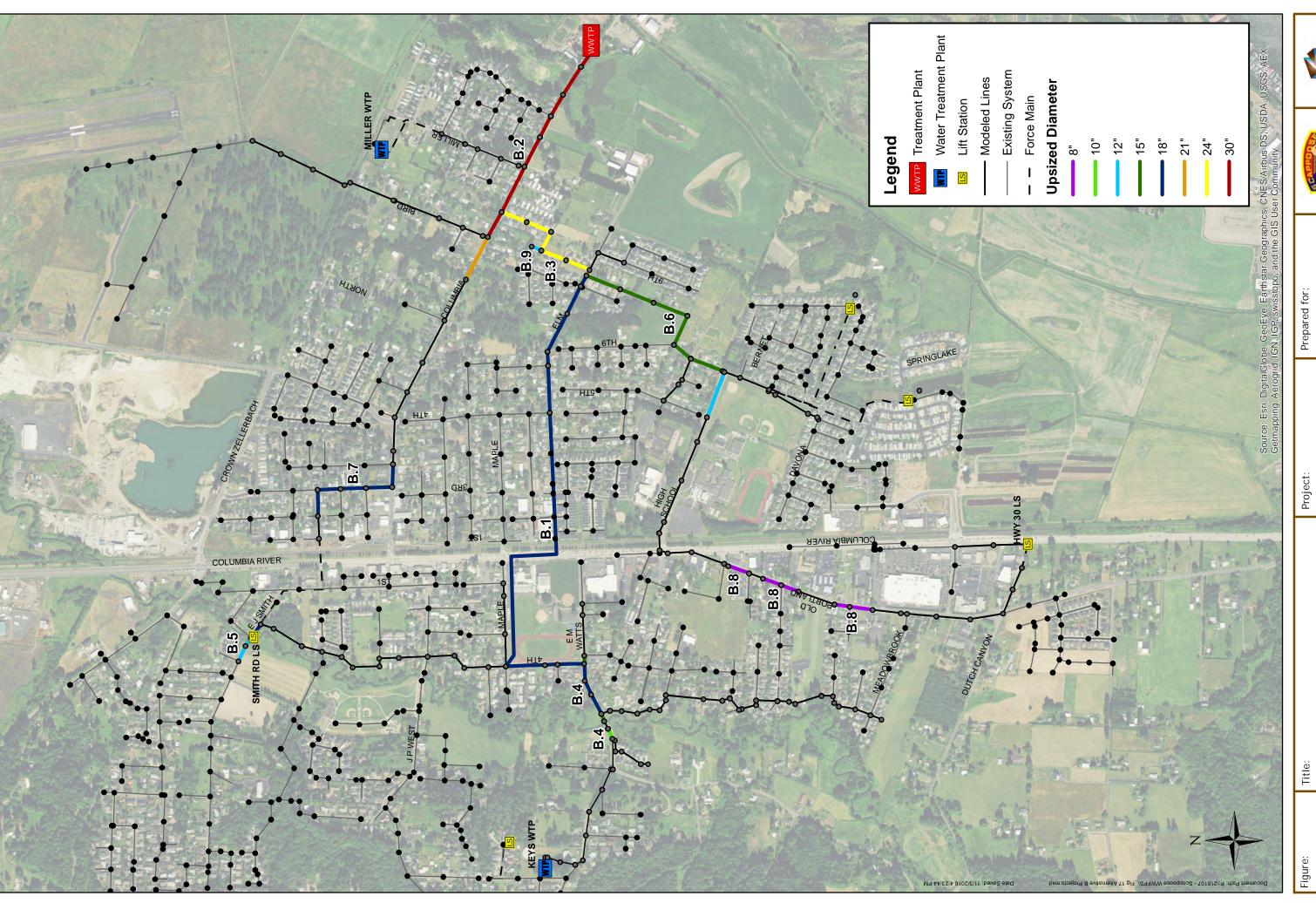












CITY OF SCAPPOOSE, OR



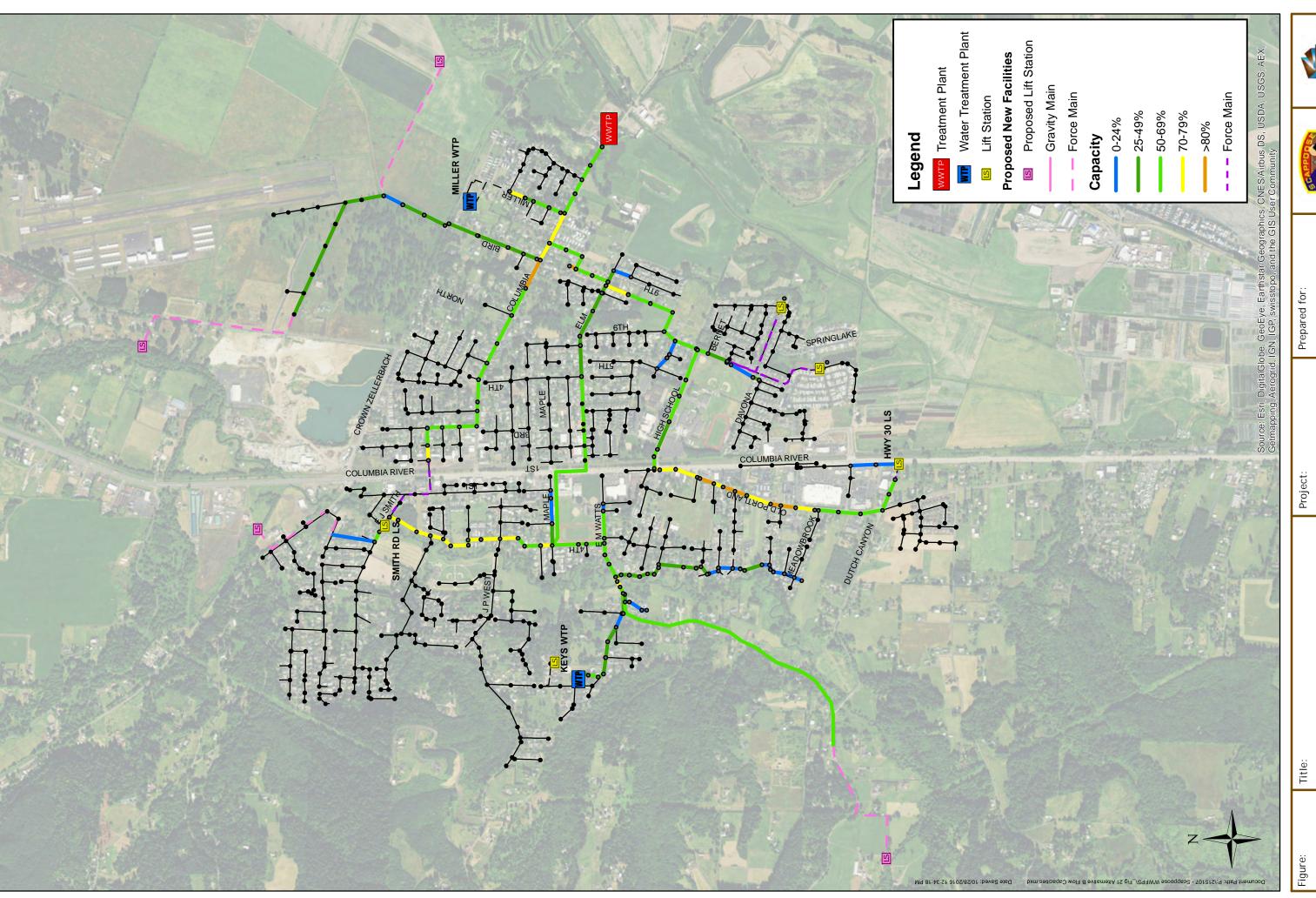


ALTERNATIVE B PROJECTS

SCAPPOOSE, OR **WWFPS**

PROJECT NO: 215107 FIGURE NO.

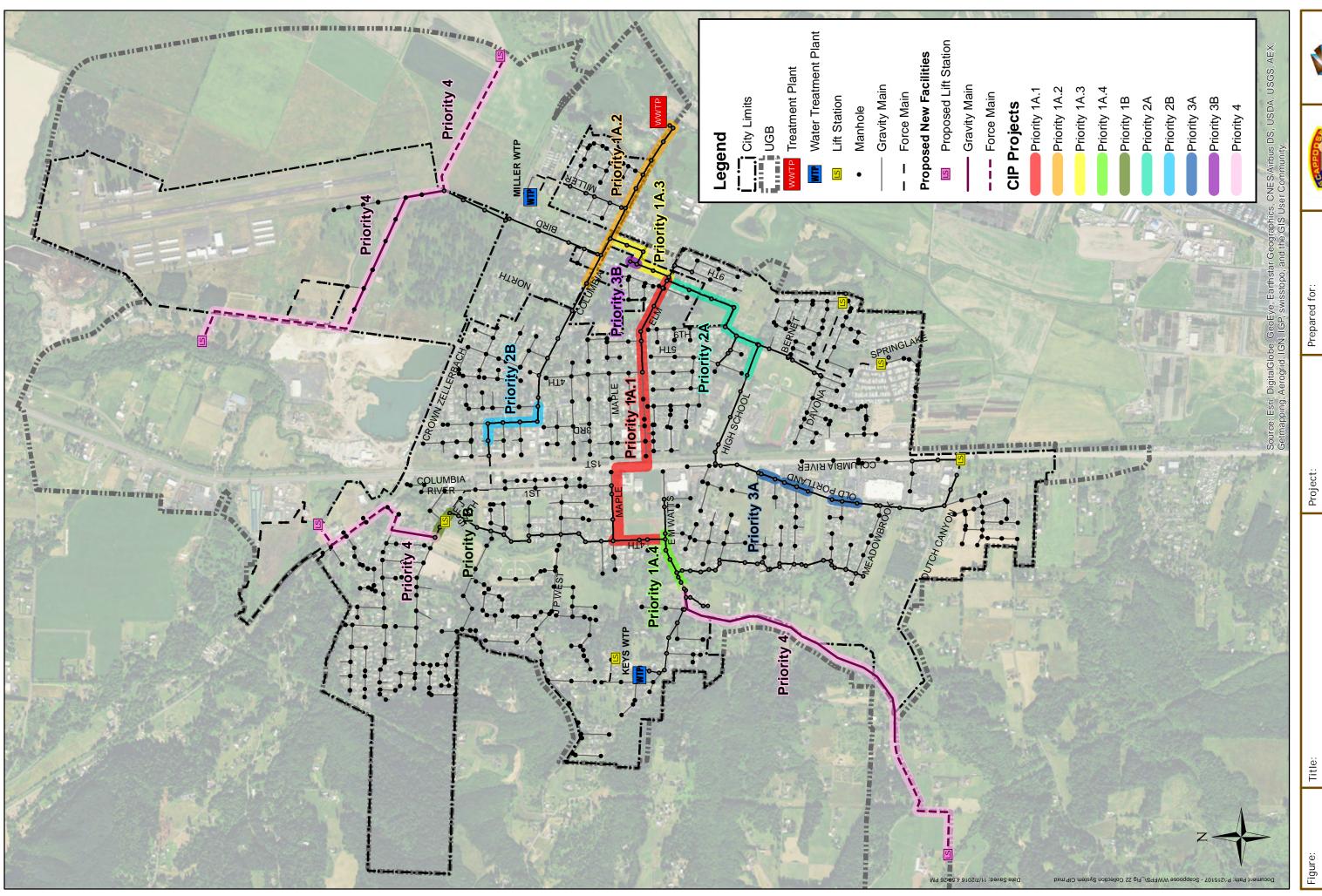
KELLER associates



OR CITY OF SCAPPOOSE,





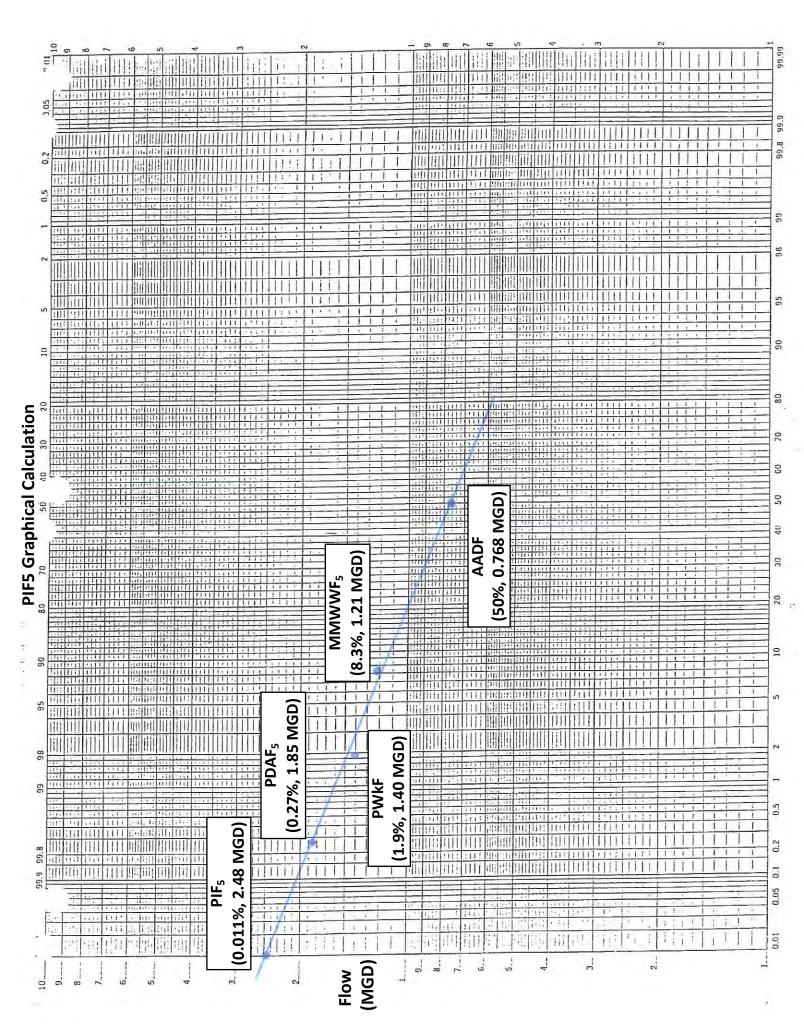








APPENDIX B – DEQ Flow Calculation





APPENDIX C - Clean Water Act Data

C-1: NPDES 100677

Expiration Date: October 31, 2014

Permit Number: 100677 File Number: 78980 Page 1 of 21 Pages

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:	SOURCES COVERED BY THIS PERMIT:				
City of Scappoose 33568 East Columbia Avenue Scappoose, OR 97056	Type of Waste Treated Wastewater	Outfall Number 001	Outfall Location R.M. 10.6		
FACILITY TYPE AND LOCATION:	RECEIVING STREAM INFORM	MATION:			
Activated Sludge/Extended Aeration Scappoose STP	on Basin: Willamette Sub-Basin: Lower Willamette				
34485 East Columbia Avenue Scappoose OR 97056	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 Applicat	ion No. 971614 received June 9, 2009.				
This permit is issued based on the land use fi	indings in the permit record.				
Gregory L. Geist, Manager, Water Quality S Northwest Region	ource Control Section Da	nte			

Page 2 of 21 Pages

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:

	<u>Page</u>
Schedule A - Waste Discharge Limitations not to be Exceeded	3
Schedule B - Minimum Monitoring and Reporting Requirements	5
Schedule C – <i>Not used</i>	8
Schedule D - Special Conditions	9
Schedule E – <i>Not Applicable</i>	
Schedule F - General Conditions	11

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.

Page 3 of 21 Pages

SCHEDULE-A

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

a. <u>Treated Effluent Outfall 001</u>

(1) May 1 - October 31:

May 1 - Octobe	ZI 31.				
	Average Effluent		Monthly*	Weekly*	Daily*
	Concentrations		Average	Average	Maximum
Parameter	Monthly	Weekly	lb/day	lb/day	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:

	Average	Effluent	Monthly*	Weekly*	Daily*
	Concentrations		Average	Average	Maximum
Parameter	Monthly	Weekly	lb/day	lb/day	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 BOD ₅ and TSS Removal Efficiency	Shall be within the range of 6.0 - 9.0 Shall not be less than 85% monthly average for BOD ₅ and 85% monthly for TSS.

(4) <u>Mixing Zone</u>: 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) <u>Chlorine</u>: 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

Page 4 of 21 Pages

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

b. <u>Groundwater</u>: 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
рН	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
pН	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	2/Week	Calculation of Daily Mass
and TSS)		Load
Pounds Discharged (BOD ₅	Weekly	Calculation of Weekly
and TSS)		Average Mass Load
Pounds Discharged (BOD ₅	Monthly	Calculation of Monthly
and TSS)		Average Mass Load
Average Percent Removed	Monthly	Calculation
(BOD ₅ and TSS)		

c. <u>Biosolids Management</u>

Page 6 of 21 Pages

File Number: 78980 Permit Number: 100677

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. <u>Temperature Monitoring (Monitored only required May 1 - October 31)</u>

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

2. <u>Monitoring Report Procedures</u>

File Number: 78980
Permit Number: 100677
Page 7 of 21 Pages

a. <u>Monitoring results</u> 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 <u>15th day</u> 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. <u>Monitoring reports</u> 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. <u>Biosolids Report Submittals</u>

<u>Biosolids Report</u>: For any year in which biosolids are land applied, a report shall be submitted to the Department by <u>February 19</u> 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. <u>UV Intensity Measurement.</u> 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/cm². 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. <u>Biosolids Sampling</u>. Biosolids composite samples from the storage lagoon or pond shall be taken from reference areas in the storage lagoon or pond pursuant to <u>Test Methods for Evaluating Solid Waste, Volume 2</u>; Field Manual, Physical/Chemical Methods, November 1986, Third Edition, Chapter 9.
 - Inorganic pollutant monitoring must be conducted according to <u>Test Methods for Evaluating Solid Waste</u>, <u>Physical/Chemical Methods</u>, Second Edition (1982) with Updates I and II and Third Edition (1986) with Revision I.
- 3. <u>Volatile Solids Reduction</u>. 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. <u>Fecal Coliform Sampling</u>. Analyze and report a fecal coliform result for each sample separately. Calculate and report the geometric mean of all the samples.
- 5. <u>Temperature Monitoring</u>. 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

Page 8 of 21 Pages

not exceed the stream temperature standard of 18.0 $^{\circ}$ C (64.4 $^{\circ}$ F). Temperature monitoring results must be reported on the monthly DMR.

SCHEDULE-C

Permit Compliance Conditions

There are no compliance conditions.

Page 9 of 21 Pages

SCHEDULE-D

Special Conditions

- 1. <u>Sewer Cleaning Report</u>: 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 <u>February 19</u> 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. <u>Biosolids Management</u>: 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. <u>Changes in Biosolids Standards</u>: 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. <u>Operator Certification</u>: 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.
 - <u>Note</u>: 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.

Page 10 of 21 Pages

- 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. <u>Groundwater</u>: 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. <u>Notification</u>: 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)

Page 11 of 21 Pages

SCHEDULE-F NPDES GENERAL CONDITIONS – DOMESTIC FACILITIES

(Schedule-F, last update 9.18.2009)

SECTION-A, STANDARD CONDITIONS

1. <u>Duty to Comply with Permit</u>

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. <u>Duty to Mitigate</u>

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.

File Number: 78980
Permit Number: 100677
Page 12 of 21 Pages

4. <u>Duty to Reapply</u>

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. <u>Property Rights and Other Legal Requirements</u>

Page 13 of 21 Pages

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. <u>Bypass of Treatment Facilities</u>

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:

Page 14 of 21 Pages

- 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. <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

Page 15 of 21 Pages

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.

Page 16 of 21 Pages

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. <u>Flow Measurements</u>

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 \pm 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. <u>Penalties of Tampering</u>

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

Page 17 of 21 Pages

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

Page 18 of 21 Pages

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. 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. <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.

File Number: 78980
Permit Number: 100677
Page 19 of 21 Pages

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

Page 20 of 21 Pages

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.

Page 21 of 21 Pages

- 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

Page 1 of 23
File No: 78980
Permit No: 100677

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

Scappoose FactSheet 20Apr2010.docx

Page 2 of 23
File No: 78980
Permit No: 100677

Table of Contents	Page
Summary of Proposed Permit Changes	3
Documentation on File for Public Review	3
Background	4
Introduction	
Wastewater Treatment Plant History	4
List of Wastewater Treatment and Collection System Facilities	.4
• STP	
Collection System	
STP Flow and Treatment Status Discussion.	5
STP Improvements Proposed for the Upcoming 5-Year Permit Cycle	5
Receiving Stream Water Quality	6
Receiving Stream Water QualityProposed Permit Complies With TMDL	6
Recent Facility Inspections	6
ZID and MZ 7Q10 Dilutions	6
Biosolids Management and Utilization	7
Inflow and Infiltration (I&I)	7
Pretreatment.	
Pollutants Discharged.	
Effluent metals Discussion.	
Whole Effluent Toxicity (WET) Tests	9
pH Reasonable Potential Analysis (RPA)	
• Ammonia	
Outfall	
Receiving Stream Impacts	
Beneficial Uses	
Water Quality Standards	.10
Willamette Basin TMDL Summary	
TMDL and Bacteria	.10
TMDL and Mercury	11
TMDL and Temperature	.12
o TMDL, Small Source Bubble Allocation	14
o Thermal Plume Limitations	15
Groundwater	15
Stormwater	16
Permit History	
Compliance History	16
Permit Discussion	
Face Page.	.16
Schedule-A, Waste Discharge Limitations	16
BOD and TSS concentration and mass limits	16
• BOD ₅ and TSS	
BOD ₅ and TSS Percent Removal Efficiency	18
• pH	
Bacteria.	
• Chlorine Residual	
• Temperature	
Mixing Zone and Zone of Immediate Dilution	$20 \cdot$

Page 3 of 23
File No: 78980
Permit No: 100677
Other Parameters

C WING X GIGINIOUS	
Emergency OverflowsGroundwater	20
Groundwater	20
Schedule-B, Minimum Monitoring and Reporting Requirements	20
• Influent Monitoring	
• Effluent Monitoring	21
Biosolids Sampling and Tracking	
• Temperature Monitoring (May 1 through October 31)	
Discharge Monitoring Reports	
Annual Biosolids Report	
Schedule-C, Compliance Conditions (<i>Not Applicable</i>)	
Schedule-D, Special Conditions	22
Sewer Cleaning Report	22
Biosolids Management	22
Changes in Biosolids Standards	23
Operator Certification	23
• Groundwater	
Notification	
Schedule-E, Pretreatment Program (Not Applicable)	,
Schedule-F, NPDES General Conditions	23

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

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,

Page 4 of 23
File No: 78980
Permit No: 100677

- 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 <u>ADWF = 1.515 mgd</u>. 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, and 100 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,

Page 5 of 23
File No: 78980
Permit No: 100677

- 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.

Collection System

- 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.

STP Flow and Treatment Status Discussion

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.

STP Improvements Proposed for the Upcoming 5-Year Permit Cycle

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

Page 6 of 23
File No: 78980
Permit No: 100677

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 (<u>Attachments-1 & -2</u>). Effluent mercury monitoring (<u>Attachment-3</u>), 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 <u>Attachment-4</u>) determined that dilutions for the zone of immediate dilution (<u>ZID</u>) and the <u>MZ</u> 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).

Page 7 of 23
File No: 78980
Permit No: 100677

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 <u>February 19</u> 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.

Page 8 of 23
File No: 78980
Permit No: 100677

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 (BOD₅), (2) Total Suspended Solids (TSS), (3) *E. coli* bacteria, (4) pH, and (5) Pollutant (BOD₅ & 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 μ g/L and zinc=120 μ g/L). Only one copper test (16.1 μ g/L) showed the effluent copper concentration above the chronic criterion = 12μ g/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 μ g/L)/1.2 = 13.4μ g/L and after MZ dilution = (16.1 μ g/L)/40 = 0.4μ g/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

Page 9 of 23 File No: 78980

Permit No: 100677

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, <u>Attachment-4</u>). 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 (<u>Attachment-4</u>) 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 <u>Attachment-4</u>. Summer season WET tests were completed for the Years 2004 through 2007 (Table-1, <u>Attachment-4</u>). 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 (<u>Attachment-6</u>). 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 \leq 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

Page 10 of 23 File No: 78980

Permit No: 100677

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 <u>Receiving Stream Water Quality</u> 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:

Page 11 of 23 File No: 78980 Permit No: 100677

"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.

Page 12 of 23
File No: 78980
Permit No: 100677

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 = $0.005 \, \mu g/L$ (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 <u>natural thermal potential</u> of all or a portion of a water body exceeds the biologically-based criteria in section (4) of this rule, the <u>natural thermal potential</u> 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-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:

Page 13 of 23 File No: 78980 Permit No: 100677

- 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	Channel Natural Thermal
	Temperature (°C)	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

Page 14 of 23 File No: 78980 Permit No: 100677

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</u>, <u>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*):

Bubble Allocations for Small Point Sources

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 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. 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 using available data where possible, or by assuming an average Willamette Basin TMDL: Temperature September 2006 OREGON DEPARTMENT OF ENVIRONMENTAL QUALITY, 4-71 flow 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. 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. 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.

Page 15 of 23
File No: 78980
Permit No: 100677

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 (<u>Attachment-2</u>, 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.

<u>Thermal Plume Limitations [OAR 340-041-0053 (2) (d) (A-D)]</u>. 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 <u>ambient temperature is 21.0 °C or greater</u>, 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

Page 16 of 23 File No: 78980 Permit No: 100677

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 BOD₅ and TSS must be provided from May

Page 17 of 23
File No: 78980
Permit No: 100677

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 BOD₅ and TSS. Winter limits in the renewal permit are more stringent than secondary, i.e. 25 mg/L for both BOD₅ 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 BOD₅ 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 BOD₅ 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 (BOD₅) and suspended solids (TSS) are based on the design ADWF = 1.515 mgd, and the monthly average BOD₅ 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 BOD₅ 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.

BOD₅ and TSS

The limits are:

(1) May 1 - October 31:

		Effluent trations	Monthly Average	Weekly Average	Daily Maximum
Parameter	Monthly	Weekly	lb/day	lb/day	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:

	Average	Effluent	Monthly	Weekly	Daily
	Concer	trations	Average	Average	Maximum
Parameter	Monthly	Weekly	lb/day	lb/day	Lbs
BOD ₅	25 mg/L	37 mg/L	315	475	630
TSS	25 mg/L	37 mg/L	315	475	630

Calculations:

(1) Summer BOD₅ & TSS

Page 18 of 23
File No: 78980
Permit No: 100677

- (a) $1.515 \text{ mgd x } 8.34 \text{ #/gal x } 10 \text{ mg/L monthly avg.} = \frac{125}{100} \text{ lbs/day}$
- (b) 125 lbs/day monthly avg. $x 1.5 = \underline{190}$ lbs/day weekly avg.
- (c) 125 lbs/day monthly avg. x 2.0 = 255 lbs/day daily max.

(2) Winter BOD₅ 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.

BOD₅ and TSS Percent Removal Efficiency

A minimum level of percent removal for BOD₅ 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.

pН

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.

Page 19 of 23 File No: 78980 Permit No: 100677

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 (<u>Attachment-2</u>) 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.

Page 20 of 23 File No: 78980 Permit No: 100677

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.

Page 21 of 23 File No: 78980 Permit No: 100677

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) BOD₅ and TSS are monitored by composite sampler twice per week, and
- (3) pH is grab-sampled three times per week.

Effluent Monitoring

- (1) BOD₅ 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) BOD₅ and TSS mass loads must be calculated daily (twice per week frequency),
- (7) BOD₅ and TSS average mass loads must be calculated weekly and monthly, and
- (8) BOD₅ 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.

Page 22 of 23 File No: 78980

Permit No: 100677

Discharge Monitoring Reports

Discharge monitoring reports must be submitted to the Department monthly by the <u>15th day</u> 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 <u>February 19</u> 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.

<u>Note 4 to Schedule-B</u>: Fecal coliform sample analysis and reporting must comply with these requirements.

<u>Note 5 to Schedule-B</u>: 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) <u>Sewer Cleaning Report</u>. Permittee is required to annually inspect and clean approximately 20% of its sanitary sewer system. An annual Sewer Cleaning Report is due by <u>February 19</u> of the following year.
- (2) <u>Biosolids Management</u>. 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.

Page 23 of 23 File No: 78980

Permit No: 100677

- (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) <u>Groundwater</u>. 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) <u>Notification</u>. 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

Scappoose - DMR Summary
 Scappoose - Thermal Calculator Runs
 Scappoose - Effluent Mercury Data Summary
 Scappoose - Permit Milestones
 Scappoose - Effluent Metals Data Summary
 Scappoose - STP MZ pH Study
 Scappoose - Freshwater Ammonia Criteria Calculations

Scappoose FactSht Attachments 30April2010.docx

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1. Scappoose - DMR Summary

SCAPPOOSE - DMR DATA SUMMARY	DMR DAT	A SUMMAR	Ž								
Temperature Criterion : 7-day-average maximum temperature of a stream identified as having salmon and trout	Criterion :	7-day-avera	ge maximum	temperatu.	re of a strec	am identifie	d as having	salmon and	trout		
rearing and migration use may not exceed 18.0	nigration u.	se may not e.		C (64.4 °F), I	oer OAR 341	°C (64.4 °F), per OAR 340-041-0028 (4) (c)	(4) (c).				
Effluent 7-day-average maximum temperature	/-average i	naximum tei	mperature >	18.0 C & > (channel ten	18.0 C & > channel temperature (May through October)	May through	h October).			Estimate
					-						Natural
*Ammonia non-detect (N/D) readings were converted to 0.1 mg/L for subsequent calculations.	n-detect (N/D) reading	ss were conv	erted to 0.1	mg/L for su	ubsequent c	alculations			Effluent	Thermal
	:								Multnomah	Exceeds	Potential
	STP	EFFLUENT					ı	*	Channel	Channel	Monthly
	Flow	퐙	7-Day	Temp	ВОБ	TSS	D.O.	Ammonia	Temp	Temp By	Average
Month/Yr	(mgd)	[s:u.]	Average	១	(mg/L)	(mg/L)	(mg/L)	(mg/L)	9	(3)	១
,					-						
Jan-10		J	Meek-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	9.9	Week-3	*	2.7	4.2	4.4	0.1	*	*	
Махітит	1.290	7.2	Week-4	*	2.1	4.6	5.1	0.1	*	*	
		·									
Dec-09		į (Week-1	*	3.6	1.6	5.1	0.1	***	*	
Average	0.726	† I	Week-2	*	1.6	1.8	6.9	0.1	*	*	
Minimum	0.562	9.9	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	ı	Week-2	*	2.1	3.0	4.8	0.1	*	*	:
Minimum	0.495	9. 9.	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	*	*	
		ı									*
Oct-09		ı	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	
3		ı	-		- The state of the						
Sep-09		ŀ	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	

	0.437	٥:/	Week-3	20.6	2.7	3.9	4.2	0.1	20.5	0.1		1-2/10
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	9.9	Week-3	22.4	2.4	5.7	4.4	0.1	22.2	0.2		٠
Maximum	0.765	7.9	Week-4	20.6	2.6	5.1	5.1	0.1	21.6	-1.0		
60-ln()		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	9.9	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		
60-unf			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-a	19.3	1.5	2.4	4.4	0.1	18.4	6.0		
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	6.0	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		
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Apr-09			Week-1	*	2.0	2.4	5.7	0.1	*	*		
Average	0.629	· · ·	Week-2	*	1.8	6.0	4.5	0.1	*	*		
Winimum	0.444	9.9	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	9.9	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	*	*	;	
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	Week-1 *	<u></u>	1.5	1.0	5.6	0.1	*	*	
Week-2 *	!		3.3	3.8	5.4	0.1	*	*	
6.8 Week-3 *			2.3	2.8	5.4	0.1	* .	*	, ,
7.0 Week-4 *			1.9	1.8	4.2	0.1	· · · · · · · · · · · · · · · · · · ·	*	
	- }								
Week-1 *	_:		4.1	7.0	4.8	0.1	*	*	
Week-2 *	_		1.9	2.9	5.1	0.1	*	*	
6.8 Week-3 *		• • • • • • • • • • • • • • • • • • • •	1.5	2.6	5.2	0.1	*	*	
7.4 Week-4 *	' I	-	1.8	1.7	5.8	0.1	*	*	
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Week-1 *	}		2.7	4.2	5.6	0.1	*	*	
Week-2 *	_		8.5	7.1	5.7	0.1	*	*	
6.6 Week-3 *			1.6	3.1	6.4	0.1	*	*	** .
7.3 Week-4 *			4.9	3.5	6.2	0.1	*	*	
	i								
Week-1 *			2.2	3.4	5.8	0.1	**	*	
Week-2 *	.		2.8	3.2	5.4	0.1	*	*	
6.9 Week-3 *	' I		1.9	1.5	5.5	0.1	*	*	
7.1 Week-4 *	.	-	2.4	3.0	5.7	0.1	**	*	
	- 1								
Week-1 18.0	91		3.0	1.8	5.3	0.1	15.7	2.3	13.6
Week-2 16.5	πi		2.7	3.0	6.0	0.2	13.6	2.9	
6.9 Week-3 14.6	ဖ		2.5	1.8	6.8	0.2	12.6	2.0	
7.5 Week-4 14.9	٥		2.7	4.2	6.8	0.2	12.4	2.5	
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Week-1 19.4	4.		2.5	2.2	5.0	0.1	19.8	4.0-	19.4
Week-2 20.4	4.		1.5	2.6	5.5	0.2	19.9	0.5	
6.5 Week-3 20.1	Ħ	.	1.5	2.8	5.0	0.1	19.7	0.4	3
8.2 Week-4 18.5	νį	. [1.4	4.3	5.1	0.1	18.1	0.4	
	- 1				-			. `	2,
_	انتح		1.7	1.7	4.7	0.1	21.0	-0.2	21.8
Week-2	1:1		1.7	2.6	4.8	0.1	21.9	-0.2	
6.5 Week-3 20.9	انہ		1.9	2.7	5.1	0.1	22.2	-1.3	

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-2.0		2.0	0.8	-1.0	-1.3		4.5	4.9	4.2	3.4		3.3	5.2	5.3	6.0		*	*	*	*		*	*	*	*	**************************************	*	*	*	*		*
22.1		18.6	20.7	21.2	21.4		12.4	11.2	14.1	16.1	;	11.7	11.2	12.8	11.2) · · · · · · · · · · · · · · · · · · ·	*:	*.	*	*	, , , , , , , , , , , , , , , , , , ,	*	*	* .	* .		***************************************	*	*	*	? .	**
0.1		*	*	*	*		0.1	0.2	0.1	0.1		0.1	0.2	0.1	0.2		0.1	0.1	0.1	0.1		0.2	1.6	0.1	0.1		0.1	0.1	0.1	0.1		0.1
5.0		3.4	4.7	5.1	4.7		5.6	4.7	4.5	4.5		5.1	4.7	4.3	4.8		6.0	0.9	6.5	5.7		6.2	5.1	6.3	6.1		6.3	8.2	6.6	6.4		6.1
3.0		3.6	3.6	1.7	1.1		2.5	3.3	1.5	3.7		5.6	3.9	2.6	3.5		7.0	9.9	7.6	7.7		5.1	2.7	4.9	4.3		4.2	2.9	2.1	2.7		4.8
2.0		1.7	1.5	3.7	1.9		1.9	1.7	2.4	1.5		4.3	4.0	4.9	3.9		2.0	3.7	3.1	4.9		2.7	10.2	2.0	5.1		1.7	2.4	2.3	2.0		2.8
20.1	a pre in gen feath un a	20.6	21.5	20.2	20.1		16.9	16.1	18.3	19.5		15.0	16.4	18.1	17.2		*	*	*	*		*	*	*	*		*	*	*	*		*
Week-4	ĺ	Week-1	Week-2	Week-3	Week-4		Week-1	Week-2	Meek-3	Week-4	:	Week-1	Week-2	Week-3	Week-4		Week-1	Week-2	Week-3	Week-4	•	Week-1	Week-2	Week-3	Week-4		Week-1	Week-2	Week-3	Week-4		Week-1
7.2				6.9	7.8				6.9	7.8				6.9	7,5				9	7.1			-	6.9	7.7				6.5	. 7.8	r	- /
0.759			0.595	0.403	0.786	Í		0.579	NA	0.722			0.610	0.489	0.793			0.662	NA	0.903			0.628	0.436	0.981			0.700	0.535	1.056		
Maximum		80-Inf	Average	Minimum	Maximum	-	30-unf	Average	inimum	Maximum		May-08	Average	Winimum	Maximum		Apr-08	Average	Minimum	Maximum		Mar-08	verage	Minimum	Maximum		Feb-08	Average	Minimum	Maximum		Jan-08

Week-2 *
K+
Week-4 * 3.9
Week-1 * 2.1
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Week-4 * 2.4
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Week-3 * 1.5
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Week-1 17.2 1.7
Week-2 16.8 2.6
Week-3 16.6 1.5
Week-4 15.5 4.0
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Week-1 20.9 2.2
Week-2 20.4 1.9
Week-3 18.6 2.4
Week-4 18.3 1.6
Week-1 20.6 2.0
Week-2 21.0 2.0
Week-3 21.2 2.1
Week-4 20.6 1.7
Week-2 21.7 2.1
Week-3 21.0 3.5
Week-4 20.8 2.1

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:	0.1	0.1	0.2	0.1	0.2	0.1	0.2	0.1		0.1	0.1	0.1	0.1		0.1	0.1	0.1	0.2		0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.2	0.2
	4.7	4.9	4.6	4.7	4.7	4.8	5.0	4.6		6.9	5.8	4.8	4.6		4.5	4.8	4.5	4.5		4.1	4.7	4.6	4.8	4.5	4.8	5.2	4.9	4.5	4.4
	1.9	2.4	1.7	2.3	9.0	0.7	2.1	1.9		1.3	1.2	1.1	0.5		2.4	2.8	2.2	3.2		0.8	1.3	2.7	1.3	1.9	1.2	1.3	1.1	2.1	3.5
	2.2	2.2	2.7	1.6	 1.9	2.3	1.4	1.7		2.9	1.7	3.2	2.3		2.7	2.5	2.0	1.5		2.8	2.9	2.8	2.9	 2.0	1.8	1.5	1.5	2.0	1.6
	19.0	18.0	19.0	20.0	15.0	17.0	17.0	18.0		*	*	*	*		*	*	*	*		*	*	*	*	*	*	*	*	*	*
	Week-1	Week-2	6.8 Week-3	7.4 Week-4	Week-1	Week-2	Week-3	Week-4		Week-1	Week-2	Week-3	Meek-4	- - - -	Week-1	Week-2	Week-3	Week-4		Week-1	Week-2	Week-3	Week-4	Week-1	Week-2	Week-3	Week-4	Week-1	Week-2
-	,	·	9		 [6.4	7.1	:		Í	6.7	7.0		1		6.7	7.4	ļ		· /.	6.9	7.2	: 1	1	6.8	7.1	·	
	7	0.622	0.487	0.749	7	0.615	0.288	0.833		7	0.602	0.363	0.969	•		0.764	0.506	1.207		7	0.666	0.447	0.927	7	0.838	0.545	1.268		0.914
	Jun-07	Average	Minimum	Maximum	May-07	Average	Minimum	aximum	<u>)</u>	Apr-07	Average	Minimum	Maximum		Mar-07	Average	Minimum	Maximum		Feb-07	Average	linimum	Maximum	Jan-07	Average	Minimum	Maximum	Dec-06	Average

Sheet1

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Data Summary
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	- 1				-	15.5					19.8				22.2				 22.5				 17.7				
*	*	*	*	 *	*	6.0	1.2	2.0	2.1		-0.7	-1.0	-0.3	2.2	 -2.3	-1.2	-1.3	-0.8	-1.7	-0.6	-1.3	0.0	2.6	2.3	1.6	1.2	
*	` *	*	*	*	*	17.2	16.5	14.6	13.5	-	21.5	20.5	19.2	18.0	23.3	22.2	21.3	21.8	21.8	21.5	22.7	24.0	15.9	16.5	17.9	20.6	
0.1	0.1	0.1	0.1	0.1	0.1	0.2	0.2	0.1	0.4		0.1	0.2	0.3	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.2	0.1	0.2	0.2	0.2	0.3	
5.3	4.9	4.6	4.7	4.9	4.8	4.8	5.2	5.1	5.4		4.5	4.4	4.5	4.1	4.5	4.4	5.1	5.1	4.7	4.5	4.4	4.5	4.8	4.8	5.0	4.8	
1.8	2.5	5.0	3.1	5.5	4.8	4.8	7.1	7.2	5.7		6.3	6.2	5.7	4.9	5.0	7.3	4.0	5.8	4.9	4.9	5.7	4.4	3.8	4.5	4.1	5.0	
1.5	2.0	1.7	1.5	3.0	2.0	2.4	2.6	1.9	1.7		2.5	1.9	3.3	2.4	1.9	2.2	2.0	2.1	2.0	1.5	1.7	1.7	2.1	2.8	2.6	2.2	
*	*	*	*	*	*	18.1	17.7	16.6	15.6		20.8	19.5	18.9	20.2	21.0	21.0	20.0	21.0	20.1	20.9	21.4	24.0	18.5	18.8	19.5	21.8	
Week-3	Week-4	Week-1	Week-2	Week-3	Week-4	Week-1	Week-2	Week-3	Week-4		Week-1	Week-2	Week-3	Week-4													
6.8	7.1			6.9	7.1			6.7	7.1				7.0	7.6			7.0	7.4			7.0	7.3			6.8	7.3	
0.614	1.419		0.725	0.317	1.246		0.461	0.328	0.705			0.623	0.420	1.085		0.801	NA	1.115		0.831	0.454	1.240		0.837	0.631	1.033	
Minimum	Махітит	Nov-06	Average	Minimum	Maximum	, Oct-06	Average	Minimum	Maximum		. Sep-06	Average	Minimum	Maximum	Aug-06	Average	Minimum	Maximum	30-lul	Average	Minimum	Maximum	Jun-06	Average	Minimum	Maximum	

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*		1.1	2.3	2.8	2.8		-0.2	-0.2	-0.3	0.1	-1.8	-2.0	-2.6	-1.4		9.0-	-0.9	-0.6	6 0	:	0.5	9.0	8.0	0.5		0.2	1.4	1.5	2.5	*	
*		16.9	15.4	15.2	14.5		21.5	20.5	18.7	18.0	24.2	25.0	23.7	22.8		21.8	21.9	23.3	23.5		18.1	18.1	19.5	20.2	.	16.9	16.1	16.1	16.3	*	
0.78		0.11	0.71	2.50	0.50		0.1	0.11	0.15	0.20	0.11	0.1	0.14	0.12	•	0.10	0.1	0.45	0.14		0.19	0.12	0.11	0.1		0.51	0.38	0.13	0.11	7	0.14
5.9		5.9	5.4	5.9	5.2		5.3	5.6	5.5	5.8	5.7	5.9	5.8	5.8		6.3	6.4	6.3	5.8		5.6	5.7	5.8	6.3		6.5	6.0	5.5	5.4	C	9.9
2.1		4.9	5.1	3.4	6.1		6.2	4.6	4.6	5.5	 4.8	4.8	6.0	4.6		4.5	7.7	7.2	4.7		6.3	2.7	3.6	3.1		3.4	3.5	3.0	2.9	7.1	T.,
2.0		2.1	2.1	2.2	4.4		2.7	1.7	2.2	2.3	3.1	2.7	2.6	1.9		2.4	3.2	4.9	5.9		4.1	2.9	2.5	4.0		2.7	3.0	2.7	4.2	0.5	0.4
*	1	18.0	17.7	18.0	17.3		21.3	20.3	18.4	18.1	22.4	23.0	21.1	21.4		21.0	21.0	22.7	23.8		18.6	18.7	20.3	20.7		17.1	17.5	17.6	18.8	*	
Week-4		Week-1	Week-2	Week-3	Week-4		Week-1	Week-2	Week-3	Week-4	Week-1	Week-2	Week-3	Week-a		Week-1	Week-2	Week-3	Week-4		Week-1	Week-2	Week-3	Week-4		Week-1	Week-Z	Week-3	Week-4	"Moole"	תאבבאה
7.2				6.9	7.2				8.9	7.2			6.7	7.1				9.9	7.1				9.9	7.4				6.9	7.3		
0.990			0.748	0.501	0.951			0.778	0.508	1.020		0.817	0.626	0.992			0.876	0.542	1.198			0.839	0.527	1.080			0.897	0.596	1.110		
Maximum		Oct-05	Average	Minimum	Maximum		Sep-05	Average	Minimum	Maximum	Aug-05	Average	Minimum	Maximum		Jul-05	Average	Minimum	Maximum		Jun-05	Average	Minimum	Maximum		May-05	Average	Minimum	Maximum	70-74C	יט ואר

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*	*	*		*	*	*	*		Effluent	Exceeds	Channel	Temp By	ວ		6.0	-3.3						lan 2010).									
*	*	*		***	*	*	***************************************	-		Multnomah	Channel	Temp	' ପ୍ର	•	25.0	11.2	120	18.4			11/1/14	mum temperature > 18.0 C & > channel temperature (May 2005 through Jan 2010)									
0.20	0.1	0.38		*	*	*	*					Ammonia	(mg/L)		2.5	0.1	228	0.2	0.280	1.432		ture (May 20									
9.9	6.7	6.5		9.9	6.7	6.7	6.9		May throug			D.0.	(mg/L)		8.2	3.4	236	5.3				nel tempera	•••								
4.6	4.4	5.8		15.3	12.4	8.9	6.6		nperature (`	TSS	(mg/L)		15.3	0.5	236	3.8				C & > chan									
4.4	4.3	3.5		10.5	6. 0	7.3	5.3		channel ten			80D	(mg/L)		10.5	1.0	236	2.6				ture > 18.0									
*	*	*		*	*	*	*	A C. O. Spring & String Co.	18.0 C & > 0	,		Temp	១		24.0	14.6	120	19.2			of the above to the second of	m tempera		ıal							
Week-2	Week-3	Week-4		Week-1	Week-2	Week-3	Week-4	**************************************	nperature >			7-Day	Average								Colored to the control of the control	age maximu		COMMENTS							
	.6.8	7.3				8.9	7.2		aximum ter		EFFLUENT	표	(s.u.)		8.2	6.4					The second section of the passion of	t 7-day-aver		SUM	4	17	5	П	5	0	32
986.0	0.650	1.466			0.801	0.522	1.300	and the second	-average m		STP	Flow	(pgш)		1.937	0.263					BBB C OLD MAC A STORY	<u>vith effluen</u>						11	eeks =	<u> </u>	
Average	Minimum	Maximum		Mar-05	Average	Minimum	Maximum		Ruent 7-day-average maximum temperature > 18.0 C & > channel temperature (May through October)	1		-	Month/Yr		MAXIMOM	MINIMUM	SAMPLE NO.	AVERAGE	STD DEVTN	CA		Total weeks with effluent 7-day-average maxil			May Weeks =	June Weeks =	July Weeks =	August Weeks =	September Weeks =	October Weeks =	Total Weeks =

2. Scappoose - Thermal Calculator Runs

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Attachment_20

4/19/2010

Date:

Scappoose STP

Facility Name:

Enter data into white cells below:								
7010=	62.9 cfs	န			,			€
Ambient Temperature or Criterion	18 °C	O.				-		ر ا
Effluent Flow = 1.515 mgd	1.515 m	gq		7			,	3
Effluent Temperature	24 °C	, O	:	;	.		. ,	4
						•		2

15.7 cfs	8 dilution = (Qe+Qr)/Qe
25% of 7Q10 =	25% dilution =

No Reasonable Potential	
18.78 °C	
cross section =	
Temperature at 25% cross section	

- 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) = <u>314.5 cfs</u>. 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 @ Portiand) 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. Duning this week Multnomah Channel's high temperature was also 24.0 °C.
- 5) Notes:
- process; i.e. wet well fills until a preset level is reached and pumps come on. Pumps stop 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 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

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Conflict Momon		

4/22/2010

Date:

Enter data into white cells below:							***************************************
7Q10 ==	62.9 cfs	· · · · · · · · · · · · · · · · · · ·					<u>~</u>
Ambient Temperature or Criterion 19.4 °C	19.4 °C	· · · · · · · · · · · · · · · · · · ·		* ,	,		7
Effluent Flow = 1.515 mgd	1.515 mgd				•,	``	<u> </u>
Effluent Temperature	19.6 °C						4
			,				थ

cfs ·	dilution = (Qe+Qr)/Qe
15.7	œ
25% of 7Q10 =	25% dilution =

No Reasonable Potential	
19.43 °C	
Temperature at 25% cross section =	

- 1) 10% of Willamette R. 7Q10 as measured at river mile 12.0 (Gage 14211720 @ Portland) 0.10(6290 cfs) = <u>629 cfs</u>. 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) = <u>314.5 cfs</u>. 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:
- process; i.e. wet well fills until a preset level is reached and pumps come on. Pumps stop 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 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

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	छ	: ()	Effluent Flow = 1.515 mgd	Ö	
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	52.9	7	715	21.8 °C	
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	7Q10 = 62.9 cfs		1.515		
	11) 	Effluent Temperature	
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Ë		Ambient Temperature or Criterion 21.7 °C			
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Enter data into white cells below:					
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25% of 7Q10 = 15.7 cfs 25% dilution = (Qe+Qr)/Qe

Reasonable Potential	
21.71 °C	
25% cross section =	
Temperature at 25%	

1) 10% of Willamette R. 7Q10 as measured at river mile 12.0 (Gage 14211720 @ Portland) 0.10(6290 cfs) = <u>629 cfs</u>. 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) = <u>314.5 cfs</u>. 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:
- process; i.e. wet well fills until a preset level is reached and pumps come on. Pumps stop 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 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.

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3. Scappoose - Effluent Mercury Data Summary

Effluent Measured Lab Date tration UR/L) Month/Yr (ug/L) (ug/L) 27-Jul-09 ND 0.005 28-Apr-09 ND 0.005 26-Aug-08 ND 0.005 26-Aug-08 ND 0.005 21-Apr-08 ND 0.005 26-Aug-07 ND 0.005 12-Jun-07 ND 0.005 13-Dec-06 ND 0.200 2-Aug-07 ND 0.200 2-Aug-06 ND 0.200 3-Aug-05 ND 0.005 Date tration Lab Date tration NMRL	SCAPPOOSE - EFFLUENT MERCURY DATA SUMMARY
	Lab
	MRL
Apr-09 Apr-09 Apr-09 Apr-09 Apr-08 Apr-06 Ap	(ng/L)
1.11-09 2.10	
Apr-09 Dec-08 Apr-08 Apr-08 Apr-06 Ap	0.005
Apr-08 Apr-08 Apr-08 Apr-08 Apr-06 Ap	0.005
Apr-08 Apr-08 Apr-08 Apr-06 Apr-06 Apr-06 Aug-05 Au	0.005
Nug-07 Nug-07 Nug-07 Nug-07 Nug-05 Nu	0.005
Nug-07 10n-07 10n-07 10n-07 10n-07 10n-06 10n-07 10n	0.005
Nug-07 Unn-07 Unn-07 Apr-06 Apr-07 Apr-07 Apr-06 Apr-06 Apr-07 Apr-07	0.005
Unn-07 Coc-06 Apr-06 Apr-07 A	0.005
20ec-06 24pr-06 24pr-0	0.005
Apr-06 Apr-06 Aug-05 Aug-05 Aug-05 Aug-05 Aug-05 Aug-05 Aug-05 Aug-05 Aug-06 Aug-07 Aug-07 Aug-08 Aug-09 Au	0.005
Aug-05 Aug-05 Aug-05 Aug-05 POOSE Sie Jie Jun-07 Jun-07	0.005
vug-05 vug-05 vug-05 le poose lun-07 un-07	0.200
POOSE	0.200
POOSE Die Die Die Die	0.200
POOSE sle	
POOSE Sile Lih/Yr Lun-07	
Measured le Concentration tration th/Yr (lug/L) lun-07 ND oec-06 ND	- MULTNOMAH CHANNEL MERCURY DATA SUMMARY
Measured	
Sile Concentration th/Yr (ug/L) un-07 ND ND-0ec-06 ND	
tration th/Yr (µg/L) lun-07 ND lec-06 ND	Lab
(<u>ug/L)</u> 7 ND	MRL
QN QN	(ug/L)
DN	0.005
	0.005
5-Apr-06 ND 0.005	0.005

4. Scappoose - Permit Milestones

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 (LC₅₀).
- The concentration of sample causing a 25% reduction in biological measurement, e.g. growth (IC₂₅ 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 & Plmephales 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.
- <u>Chronic testing</u> (Ceriodaphnia dubid & 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.
- <u>Chronic testing</u> (*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 IC₂₅ 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 (Cerlodaphnia dubia) resulted in no statistically significant reduction in survival at any of the effluent concentrations tested when compared to the control.
- <u>Chronic testing</u> (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 IC25 value calculated on Ceriodaphnia dubia reproduction was 25% effluent.

09Jun2009

CH2M-Hill Bioassay Report for whole effluent toxicity (WET) testing was received.

Testing occurred <u>03Aug - 10Aug2004</u>. 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 IC₂₅ 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 IC25 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

<u>Biosolids Management Plan</u> 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	Specific	Station Notes*
•	Outfall (Feet)	Conductance	
,		(µmhos/cm)	
-	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)

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).

⁼ Table of conductance values for effluent/river mixing.

- 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. <u>STP Upgrades Likely To Improve Effluent Quality</u>. 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 BOD₅ concentrations. Increased TSS removal should lower effluent total metals concentrations. Effluent filtration will reduce over-all effluent toxicity and BOD₅ 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)

TABLE 5. Water Quarty sumpring summary (250cp.2000 DEQ INE Study)						
<u>Parameter</u>	<u>Units</u>	<u>Effluent</u>	Channel US	MZ	<u>25 ft</u>	
duction they d	100 1200		of Outfall	<u>Boundary</u>	DS of	
(A) 1 355 (A)			<u>001,</u>	200 ft DS	<u>Outfall</u>	
			taja o serž		<u>@ ZID</u>	
400 to 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1					<u>Edge</u>	
pH 4	s.u.	7.1	7.5	7,5	7.3	
Specific	μmhos/cm	492	103	104	370	
Conductance						
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	NŤU 😭 🗀	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	mg/L	23.0	0.291	0.306	11.2	
as N						
Total	mg/L	2	8	14	41	
suspended				y .		
solids				, , , , , , , , , , , , , , , , , , ,		

TABLE-4: Summary of Priority Pollutant Metals Testing (15Sep2008 DEQ MZ Study)

<u>Parameter</u>	<u>Units</u>	<u>Effluent</u>	Channel US	MŻ	25 ft DS of
8, 2, 8			of Outfall	Boundary	Outfall @
			001	200 ft DS	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.

<u>Proposed Metals & BOD₅ Testing in Renewal Permit</u>. 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.
- O2Mar2009 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 <u>DEQ received City's Temperature & Dissolved Oxygen (DO) Evaluation Study Report.</u>
 - 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 m³/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

300ct2007

DEQ inspection report for STP. Inspection was completed on <u>26Sep2007</u>. 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 <u>Current permit was issued</u>. The current permit requires the following:

- 1. <u>Schedule-B</u>, annual Biosolids Report (for previous year) due each year by February 19 (if land application occurred).
- 2. <u>Schedule-C</u>, 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. <u>Schedule-C</u>, worst-case dilutions are listed as follows:
- ZID ≈ 4∴ and
- MZ ≈ 23.
- 5. <u>Schedule-C</u>, 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. <u>Schedule-C</u>, Install emergency power at treatment facility within 12-months of permit issuance.
- 7. <u>Schedule-D</u>, biosolids site next to treatment facility is 35 acres with 28.1 acres available for application.
- 8. <u>Schedule-D</u>, 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.

Page **10** of **10**

MZ study completed for STP outfall. Year-2004

GLS: Scappoose Permitmilestones 16Mar2010.docx Revised: 20May2010

SCAPPOOSE	SCAPPOOSE - EFFLUENT METALS DATA SUMMARY	MMARY							8/1-5
(All samples	(All samples are grab samples taken at discharge to UV disinfection unit)	harge to UV	disinfection unit)						
Effluent		Measured		Measured		Effluent	Lab	Effluent	•
Sample	A SALATA AND A SAL	Concen-	Lab	Concen-	Lab	Hardness	Hardness	Alkalinity	
Date	THE COLUMN TO TH	tration	MRL	tration	MRL	CaCO3	MRL	CaC03	
Month/Yr	Metal	(T/Sm)	(ng/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	
15-Sep-08 Calcium	Calcium			28.9	1-0	94.7	0.70	r c	
•	Magnesium			5.48	0.1			3	
	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			AND THE PROPERTY OF THE PROPER			
	Copper	9.2	1.5			And the same of th			
	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	*	*			ALANA MITTAN			
	Phenolics	*	*						
	Bis(2-ethylhexyl)phthalate	< 0.500	0.500						
					and the second s				
	Activities and activities of the control of the con								
12-Jun-07 Calcium	Calcium	*	*	29.9	0.100	102	0.662		
	Magnesium	*	*	6.73	0.100				
	Antimony	ND	1.00						
	Arsenic	ON	1.00						
	Beryllium	ND	1.00						

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						1	,																								
													0.662																		
													83.4																	,	
													0.100	0.100							1										
		\$5000 PERSONAL PROPERTY OF THE								0.100			25.3	4.90															0.100		,
1.00	1.00	2.00	1.00	2.00	2.00	1.00	1.00	5.00	2.00	*	9.52		*	*	1.00	1.00	1.00	1.00	1.00	2.00	1.00	2.00	2.00	1.00	1.00	2.00		5.00	*	9.62	
QN	ND	11.4	QN	QN	QN	ND	QN	36.2	ND	ND	ND	:	*	*	ND	ND	QN	ND	. QN	7.7	DN	ND	ND	ND	ND	34.9		QN .	ND	31.9	
Cadmium	Chromium	Copper	Lead	Nickel	Selenium	Silver	Thallium	Zinc	Cyanide	Phenolics	Bis(2-ethylhexyl)phthalate		Calcium	Magnesium	Antimony	Arsenic	Beryllium	Cadmium	Chromíum	Copper	Lead	Nickel	Selenium	Silver	Thallium	Zinc		Cyanide	Phenolics	Bis(2-ethylhexyl)phthalate	
													7-Apr-06 Calcium																		

5-3/8					•																							
										-																		
	1.00																		•									
	82.7																											,
																								ٔ نے	g/L.	ial for		
	0.250	0.500																						Freshwater acute criterion $\approx 18 \mu g/L$	Freshwater chronic criterion = 12 µg/L.	Conclusion: no reasonable potential for	ı toxicity.	
	20.4	4.43														0.100								acute criter	chronic crit	no reason	efffluent Cu toxicity.	
												4									Comments		,	Freshwater	Freshwater	Conclusion:		
	*	*	3.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	10.0	5.00	*	10.0	;	•	MRL	T/Sri			*	1.5	1.5	1.5	
	*	*	ND	ON	QN	ND	QN	16.1	ND	2.66	QN	QN	QN	47.2	ND	ND	12.0		Concentra-	tion	ug/L			4	16.1	7.7	11.1	
	alcium	Magnesium	Antimony	Arsenic	Beryllium	Cadmium	Chromium	Copper	Lead	Nickel	Selenium	Silver	Thallium	Zinc	Cyanide	Phenolics	Bis(2-ethylhexyl)phthalate	EFFLUENT METALS TOXICITY SUMMARY		Sample Location Relative to	Outfall		UV Disinfection Unit Outlet	SAMPLE COUNT =	MAXIMUM =	MINIMUM =	AVERAGE =	
	2-Aug-05 Calcium	2	A	Α	В	၁	J	S	Ţ	2	Ŋ	S	T	Z	C	Ъ	8	EFFLUENT ME			Metal	- 1		COPPER				

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				The state of the s	
	SAMPLE COUNT =	4	*	Freshwater acute criterion = 120 µg/L.	5-4/8
•	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.	

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SCAPPOOSE	SCAPPOOSE - MULTNOMAH CHANNEL METALS DA	TALS DATA SU	IMMARY (DEQ L	TA SUMMARY (DEQ LAB REPORT FOR OUTFALL)	UTFALL)				5-5/8
(All samples	All samples are grab samples taken when effluent is activily being discharged to Multnomah Channel)	ffluent is activ	iily being discha	ged to Multnoma	h Channel)				
		•							
Sample tak	Sample taken upstream (US) of RMZ (Sample 002)	ole 002)							
		Total Recoverable	erable						
		Measured		Channel	Lab	Channel			
Sample		Concen-	Lab	Hardness	Hardness	Alkalinity	Channel	Ammonia	
Date	Metal	tration	MRL	CaCO3	MRL	5002	Hd	as N	
Month/Yr	(total recoverable)	(ng/r)	(ug/L)	(mg/L)	(mg/L)	(mg/L)	s.u.	T/Sm	
15.Cop.O8 Calcium		0323	700	0,00	07.0	Ö	L	0	
200	Nogocium	07.70	100	0.72	0.70	07	ú	0.00	
	IVI STILLS IN IT	7470	TOO	The state of the s					
j	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	- Appendix			37		
	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						
	d.Advisor .								
Sample tak	Sample taken 200 ft DS of Scappoose Outfall (edge of RMZ, Sample 003)	III (edge of RN	/IZ, Sample 003)	-					
15-Sep-08 Calcium	Calcium	6780	100	27.4	0.70	78	7.5	90.0	
	Magnesium	2530	100						
	Antimony	< 2.0	2.0						
	Arsenic	< 2.0	2.0						
								-	

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		-							
	Barıum	9.4	2.0						5-6/8
	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 tak	Sample taken 25 ft DS of Scappoose Outfall (within RMZ, Sample 004)	within RMZ	, Sample 004)	69.5	0.7	41	7.3	0.05	
15-Sep-08 Calcium	Calcium	20200	100						
	Magnesium	4640	100						
	Antimony	< 2.0	2.0						
	Arsenic	< 2.0	2.0						
	Bàrium	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						

Sample tak	en 200 ft DS of Scappoose Outfal	(RMZ edge	, Sample 0	(50)	27.2	0.7	28	7.5	0.05	5-7/8
15-Sep-08 Calcium	15-Sep-08 Calcium 6730 100	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				l l			
	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	INSTREAM METALS TOXICITY SUMMARY						·			
		Concentra-								
	Sample Location Relative to	tion	MRL							
Metal	Outfall	√3 7/8π	∏/gπ	Comments						
		1								
Barium	SU 504 000	8.5	7.0	All in-stream	All in-stream values exceed effluent concentration.	ed effluent	concentrat	ion.		
	25 feet DS	25.5	2.0	Conclusion:	Conclusion: no reasonable potential for	ble potentia	als to be so	ָ הור כני		
	200 feet DS	11.3	2.0		efffluent Ba toxicity.	toxicity.		Stander World		
	UV Disinfection Unit Outlet	7.4	2.0							
			:							
Chromium		3.5	1.0	All in-strean	All in-stream values exceed effluent concentration.	ed effluent	concentrat	ion.		
	200 feet DS	< 1.0	1.0	Suspended	Suspended instream sediment appears to be source.	iment appe	ars to be so	urce.		
	25 feet DS	1.9	1.0	Conclusion:	Conclusion: no reasonable potential for	ble potentic	al for			

E:\EXCEL\Scappoose WQ Data Summary 01Apr2010.xlsx

8/8 -5]														-			and the same of th
effluent Cr toxicity.		And the state of t	Effluent appears to be Cu source.	Freshwater acute criterion = 18 µg/L.	Freshwater chronic criterion = 12 µg/L.	Conclusion: no reasonable potential for	efffluent Cu toxicity.		All in-stream values exceed effluent concentration.	Suspended instream sediment appears to be source.	Conclusion: no reasonable potential for	effluent Pb toxicity.		Effluent appears to be Zn source.	Freshwater acute criterion = 120 µg/L.	Freshwater chronic criterion = 110 µg/L.	Conclusion: no reasonable potential for	effluent Zn toxicity.
1.0	1.0		1.5	1.5	1.5	1.5	1.5	_	0.20	.0.20	0.20	0.20	0.20	3.0	3.0	3.0	3.0	3.0
< 1.0	< 1.0		< 1.5	< 1.5	∞ ∴.	1.6	9.2		4.15	0.23	1.09	0.41	< 0.20	4.3	< 3.0	31.2	9.6	41.3
200 feet <i>DS</i>	UV Disinfection Unit Outlet	•	SU	200 feet DS	25 feet DS	200 feet DS	UV Disinfection Unit Outlet	•	US	200 feet DS	25 feet DS	200 feet DS	UV Disinfection Unit Outlet	US	200 feet DS	25 feet DS	200 feet DS	UV Disinfection Unit Outlet
			Copper						Lead					Zinc				

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 f	or pH	Ī
INPUT	Lower pH	Upper pH	
	Criteria	Criteria	
			1
1. DILUTION FACTOR AT MZ BOUNDARY - (Qe+Qr)/Qe	5	5	DEQ MZ Study 🤏
2. UPSTREAM/BACKGROUND CHARACTERISTICS			:
Temperature (deg C):	25.0		DMR temp max
pH:	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		DMR pH max/min
Alkalinity (mg CaCO3/L):	53.0	53.0	DEQ MZ Study
4. APPLICABLE PH CRITERIA	6.0		Current permit
ÖÜŢŖŲŤ			
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 13 VERY CONSERVATIVE. DEQ MZ STUDY FOUND DILUTION > 40 @ MZ BOUNDARY.

(

7. Freshwater Ammonia Criteria Calcula Scappoose

20-May-10

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© calculated	*	*	no dilution data is available						
≻	1.2	40	ution da		*	#	25	10	-
Dilution Values? (Y/N)	= Dilution @ ZID =	Dilution @ MZ =	Enter data below if no dil	Data to estimate dilution	7Q10 (CFS) =	1Q10 (CFS) =	% dilution at MZ =	% dilution at ZID =	Efficient Flow (mod.) -

	a is							
40	if no dilution data is		*	*	25	10	1	
Dilution @ MZ =	iter data below if no dil	sta to estimate dilution	7Q10 (CFS) =	1Q10 (CFS) =	% dilution at MZ =	% dilution at ZID =	Effluent Flow (mgd) =	

	pH must be between 6.5 and 9.0	Temperature must be between 0 C and 30 C
* Notes:	pH must be betwo	Temperature mus

Water Chemistry	Effluent	Stream	ΩМ. ∴	lixed	学工场中外数
			ZID	ZW	
= ∗.Hd	7.1	7.5	7.1	7.5	(6-2-9)
Temp * =	19.6	19.4	19.6	19.4	ပ
Alkalinity =	53	78			
Salmonids Present? (Y/N)		۸			

ERIA	4 Day	(CCC)	∥/6₩	1.29
WOCRITE	1 Hour	(смс)	mg/l	17.66

1.03 1.41

1.48

2.30

13

20

. У Д О "

4 4 4 4 -

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

7-2/2

Facility Name: Scappoose

Date: 5/20/2010

Scappoose PermitRPAandWQBELWorkbook 27Apr2010.xlsx

Dilution Values? (Y/N)	>	calculated
w Flow Dilution @ ZID (1Q10)	1.2	*
w Flow Dilution @ MZ (7Q10)	40	*
w Flow Dilution @ MZ (30Q5)	40	*
th Flow Dilution @ ZID (1Q10)	1.2	*
gh Flow Dilution @ MZ (7Q10)	40	*
gh Flow Dilution @ MZ (30Q5)	40	*
If no dilution values		4490 2000
Enter flow rates here	Summer	Summer Winter
Effluent Flow (MGD)	*	*
1Q10 (CFS)	*	*
7Q10 (CFS)	*	*
30Q5 (CFS)	*	*
% dilution at MZ	*	*
% dilution at ZID	¥	*
probability basis	99%	
(WLA multipliers)		

Summer data Effluent Stream	Effluent	Stream		Mixed	St. 38/2/19	15 4 Mg 193
Alternational and only the same of the sam			QIZ .	ZM ZW OIZ	MZ	S. S. Salasta
			1010	1010 7010 3005	3005	
= * Hq	7.1	7.5	7.1	7.5	7.5	(6-5-9)
= * dwa]	19.6	19.4	19.6	19.4	19.4	၁့
Alkalinity =	23	28				
Salmonids Present? (Y/N)	e/u	Y	20 4 2 3			
Salmonid Spawning? (Y/N)	n/a	z				
Fresh Water ? (Y/N)	n/a	>				
Salinity	*	*	*	*		
Winter data			9monacian		SHARE HE	Proposition of
= * Hq	7	7	7.0	0.7	7.0	(6-5-9)
= * dwa_	18	18	18.0	18.0	18.0	၁့
Alkalinity =	23	28		(A) 100 (A)		A CASA MARINE
Salmonids Present? (Y/N)	n/a	⊁				
Salmonid Sapwning (Y/N)	n/a	z				
Fresh Water ? (Y/N)	n/a	Ϋ́	\$2.50 Per 20 APRES			
Salinity	*	*	*	¥		

CEXTERIA Allocations Allocations ## Acute 4-day 30-day Min GCCO GCCO ground Acute 4-Day 30 Day Samples LTA LTA LTA Min 0.011 n/3 wg/l mg/l mg/		TAW STATE	TER OUALITY	LTY											Concentration	Tation
4 Day Back- Allocations ## Acute 4 Day 30 Day ## Acute 4 day 30 day Min 959% mg/l		1000	RITERI												5	N.
CCC) CCC) Ground Acute 4 Day 30 Day Samples LTA LTA LTA LTA Monthly	**	388	4 Day	30 Day	Back-	A Secure Assessment	Glocation	Su	discrete	#	2 USS	4 day	30 day	Min		%66
mg/l mg/l mg/l mg/l mg/l cv/ //mo mg/l	S.	線	(၁၁၁)	80	ground	Acute	4 Day	30 Day		Samples	LTA	11.00	LTA	LTA	Monthly	Daily
19 0.011 n/a * n/a * * n/a *	E	SCIAS!	1/6m]/b m		2000	I/bw	mg/L	Ω	/Mo	I) bu-	l/6m	mg/L	Control .	(/Gm	1/6m
19 0.011 n/a * <											L					
7 1.3 n/a n/a n/a 1.432 228 3.17 13.58 n/a 3.17 3.77 3.		0.019	0.011	n/a	*		×	n/a	*	*	*	*	n/a	*	*	*
n/a n/a * <td></td> <td>17.7</td> <td>1.3</td> <td>n/a</td> <td>90.0</td> <td></td> <td>49.31</td> <td></td> <td>1.432</td> <td>228</td> <td>1</td> <td>13.58</td> <td></td> <td>3.17</td> <td>3.7</td> <td>21.2</td>		17.7	1.3	n/a	90.0		49.31		1.432	228	1	13.58		3.17	3.7	21.2
19 0.011 n/a *<	-	n/a	n/a	n/a	*		×	n/a	*	*	*	*	n/a		*	*
19 0.011 n/a * <		21.4	8.2	3.3	×	*		*	*	*	*	*	*	*	*	*
19 0.011 n/a * <		_							,							
.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 n/a n/a * * n/a * * n/a * * * * n/a * * 1.432 228 3.49 15.01 n/a 3.49 4.1 1 11.8 4.7 * * * * * * * * * * * * * * * * * * *	-	0.019	0.011	n/a	*		*	n/a		*	*	*	i	*	*	×
1 11.8 4.7 ** * n/a * * * * n/a * * * * 11.8		19.5	1.4	n/a	90.0	ł		n/a	1.432	228	ı	15.01	n/a	3.49		
11.8 4.7 * * * * * * * * * * * * * * * * * * *	_	n/a	n/a	e/u	*		*	n/a	*	×	*	*	Г	*	*	*
		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.

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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

> > data

Link to LASAR Web

Data

Basin Water Body **Pollutant** Season Criteria Beneficial Status 2012 [Data Assessment Source] Name LLID Uses Action Supporting

Toxic

Substances

River Miles

Subbasin Segment 4th Field Miles

HUC Beach Name

Beach ID

Record ID

Willamette Multnomah Alkalinity Table 20 Aquatic life Cat 3: Year

Round

Channel

Lower 1227863458618

Willamette 0 to 21.7 17090012 21.7

16569

No action Insufficient 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 Multnomah Channel

Willamette 0 to 21.7

17090012 21.7

Ammonia

Round 1227863458618

Table 20 Toxic Substances Aquatic life Cat 3: Insufficient

Year

data

No action

16570

Lower

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

2012 Data:

Willamette Multnomah Channel Lower 1227863458618 Willamette 0 to 21.7 17090012 21.7

25229

Dissolved Oxygen

Dissolved

Oxygen

January 1 Spawning: - May 15 Not less than 11.0 mg/L or 95% of saturation

Cat 5: Added to database Water quality limited, 303(d) list, **TMDL** needed

[DEQ] STATION 36160 at RM 6.4 from 03/14/2006 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 Multnomah Channel 1227863458618 Lower Willamette 0 to 21.7

17090012 21.7

12075

Year Round (Non-

Cold less than spawning) 8.0 mg/l or 90% of saturation

Cold-water Cat 2: water: Not aquatic life Attaining some

Status modification - Attaining criteria/uses criteria/uses

[DEQ] STATION 36160 at RM 6.4 from 06/02/2006 12/12/2011, 0 of 22 (0%) samples < 6.5 mg/L. [DEQ] **STATION** 35110 at RM 10.6 from 09/15/2008 09/15/2008, 0 of 1 (0%) samples <

2012 Data:

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%) < 8mg/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%) < 8mg/l and applicable % saturation. Previous Status: Cat 3: Insufficient data Previous Action: Added to database Previous Assessment Year: 2004

sample >

organisms

per 100 ml

406

11/18/2015 Oregon DEQ: Water Quality - Water Quality Assessment - Oregon's 2012 Integrated Report Database Willamette Multnomah E. Coli Cat 3: Summer 30-day log Water No action Channel mean of contact Insufficient Lower 1227863458618 126 E. coli recreation data Willamette 0 to 21.7 organisms 17090012 21.7 per 100 ml; no 16571 single

Data: [DEQ] LASAR 12938 River Mile 18.6: From 8/5/1997 to 8/5/1997, 0 out of 1 samples (0%) > 406organisms; 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%) > 406organisms; 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%) > 406organisms; 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%) > 406organisms; maximum 30-day log mean of 0 Previous Status: Cat 3: Insufficient data Previous Action: Added to database Previous Assessment Year: 2004

Previous

Willamette Multnomah Table 40 Human Cat 5: Added to Mercury Year

Oregon DEQ: Water Quality - Water Quality Assessment - Oregon's 2012 Integrated Report Database Channel Round Human health Water database Lower 1227863458618 Health quality 2012 Data: Willamette 0 to 21.7 Criteria for limited, [ODEQ] 17090012 21.7 Toxic 303(d) list, **STATION Pollutants** TMDL 10549 at RM 25228 needed 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 Multnomah pН Summer pH 6.5 to Water Cat 3: No action Channel 8.5 contact Insufficient recreation; data Lower 1227863458618 Previous Willamette 0 to 21.7 Resident Data: [DEQ] 17090012 21.7 fish and LASAR 12935 aquatic life River Mile 16572 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 Insufficient data Previous Action: Added to database Previous Assessment Year: 2004

Willamette Multnomah Phosphate Channel Phosphorus
Lower 1227863458618
Willamette 0 to 21.7
17090012 21.7

21504

Summer Total phosphates as phosphorus (P):
Benchmark 50 ug/L in streams to control excessive aquatic growths

Aquatic life Cat 3: No action Insufficient data

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

1/18/2015	Ore	egon DEQ: Water Qu	uality - Water	Quality Assessm	nent - Oregon's	2012 Integrated R	eport Database	
_	hannel 227863458618 to 1.6	Sedimentation	Undefined	formation of appreciable bottom or sludge deposits or the	Salmonid fish	Cat 3: Insufficient data	No action	Previous Status: Insufficient data Previous Action: Added to database Previous Assessment Year: 1998
_	hannel 227863458618 to 21.7	•	Year Round (Non- spawning)	Salmon and trout rearing and migration: 18.0 degrees Celsius 7- day- average maximum		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

Download the spreadsheet (.csv) file: wqaclient955.csv

To select new search criteria click here - DO NOT USE THE BACK ARROW.

Status: Cat 4A: Water quality limited, **TMDL** approved Previous Action: Delisted -**TMDL** approved Previous Assessment Year: 2010



APPENDIX D – Pump Information

D-1: Smith Road Pump Station



TRIANGLE PUMP AND EQUIPMENT INC.

Equipment Approval Data

Project Smith Rd Wastewater Pump St	ation/Scappoose
Customer <u>Emerald Construction</u>	Order No
Engineer KCM	
Equipment Ref 15140 Pumps	
We request your approval to supp	oly 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,

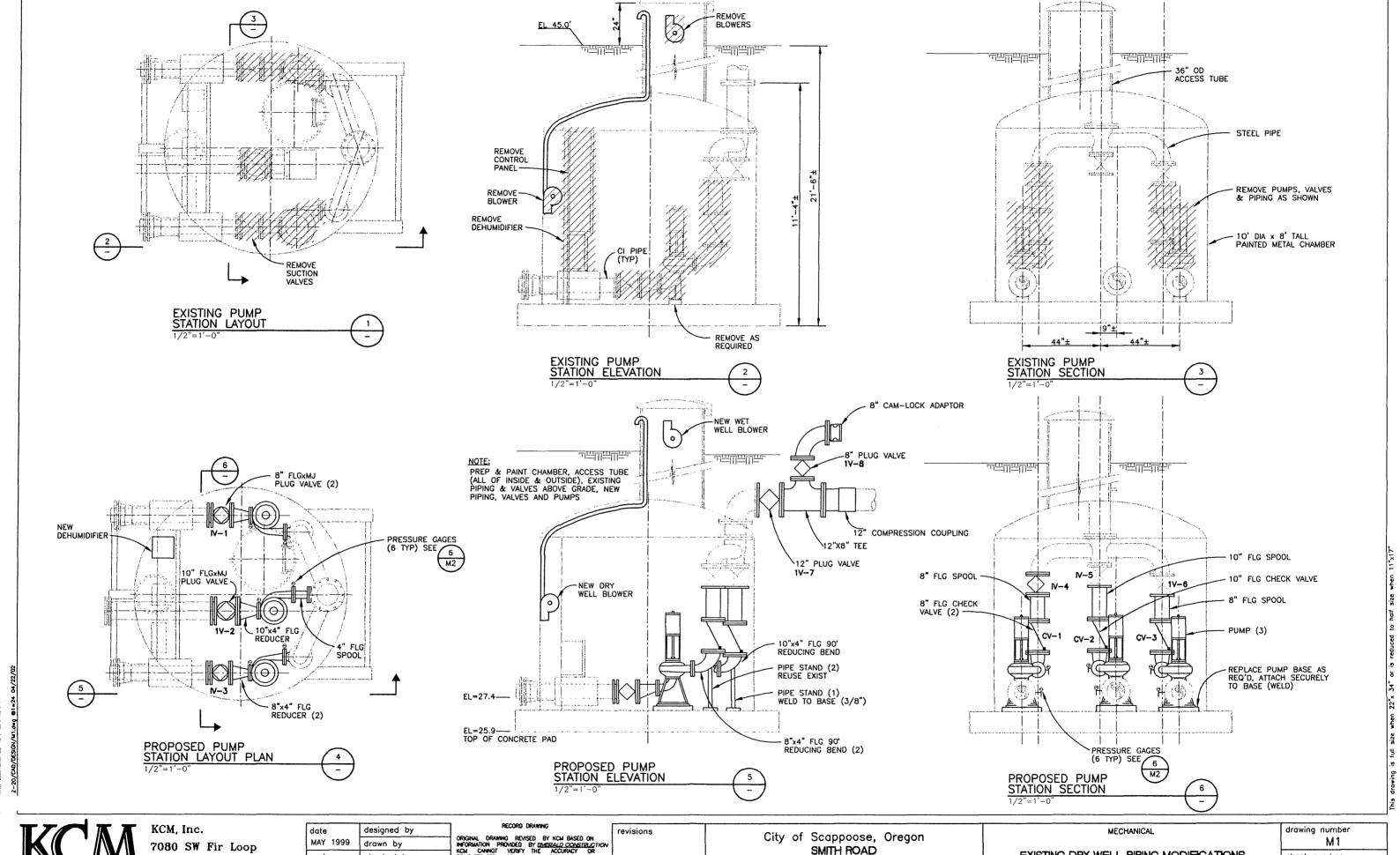
Harold R Clayton

President

attachments

/lc

Contr.: EMEMBLY CONST			۰
120	ST. Tagi	*** :*0*5	CRANE DEMING PUMP DIV.
	C REP.: TRIANGLE PUMP	P.O. 1 19398	Salem, Ohlo, U.S.A.
		196 Size 4x4x12x3	x3 Stgs 1 Impeller 26856
		Design Acting: G.p.m. 900.0	Head 35.0
		Shape of	Curves are Approximate inp. 01a. 11.875
100			ŀ
5920			2) Curve Based on Catalog Information.
06			
80 80			
70			
09	and apality	. / / / / / / / / / / / / / / / / / / /	
05			
40 40		/	
		/	
Э J өмо 49			
L 20 9 20			
0 200 00			-
For Eff	Bro SG= 1.0U		
0 0 0	200 400 600	800 1000	1200 1400 1600
		U. S. Gallons per Minute	
Submitted By:	Date:	Submittal **	Date: 07/02/38



7080 SW Fir Loop Portland, Oregon 97223

	date MAY 1999	designed by	
		drawn by	
3	scale AS NOTED	checked by	
		approved by	

	revisions			
TION				
ì				
ī	КСМ	NO.	1882-020	

SMITH ROAD PUMP STATION

EXISTING DRY WELL PIPING MODIFICATIONS

sheet number 9 of 14

D-2: Spring Lake Pump Station



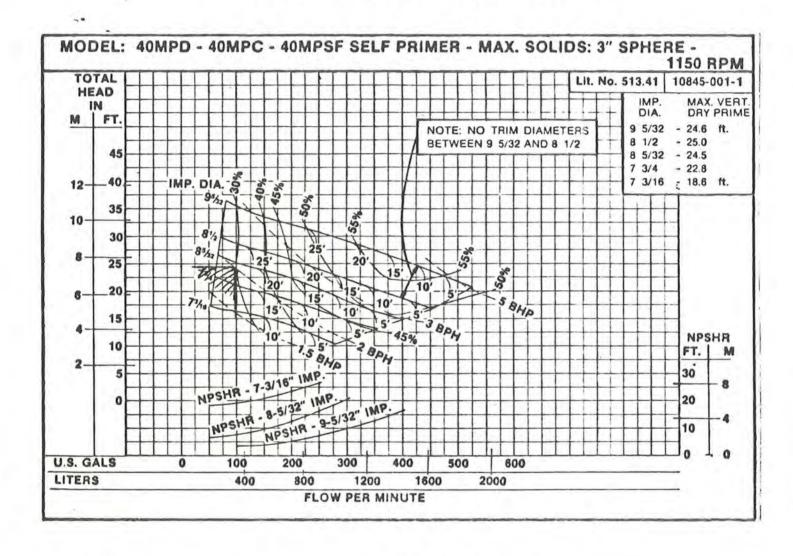
PUMP STATION DATA SHEET

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
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: Imp. Dia Priming: XX Self, 7-7 Vacuum,
(-)Flooded. Stuffing Box Type Mech. Seal , Lubrication H ² O .
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, (INC. Size .

CON	TROL	PANEL
COL	u	EUNDE

Location In pump station	NEMA Type 1
Service Entrance Size AMPS. Pump #1 Pump #2	Pump #3
Circuit BreakerTrip Rating/AMPS	
Magnetic StarterNEMA 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 1 W.W. Floor 8.8 High Water 12.3	2.8
STATION OPTIONS	
(X) 1500 Watt Heater, (*) *** ******************************	* \$ \$ & & & * * * * * * * * * * * * * *
Deminatifen, * * * * * * * * * * * * * * * * * * *	Convenience Outlet,
** * * * * * * * * * * * * * * * * * *	Elapsed Time Meters,
**************************************	Dim +Glow+ A harm, + (+)+Dry
Alarmy Comboners * * (* *) Telementry * Involvided / *Bay* Obligates * * * *) **	aruur Pumps, 1/4 } \$ \$ \$ \$
Emperson, (x) Spare Parts Kit, (x) 0 & M Instru	ction Manuals.
NOTES:	
Application Engineering	
Prepared By LARRY R. CHAPMAN/sf Date:_	5-23-86

HYDROMATIC PUMPS

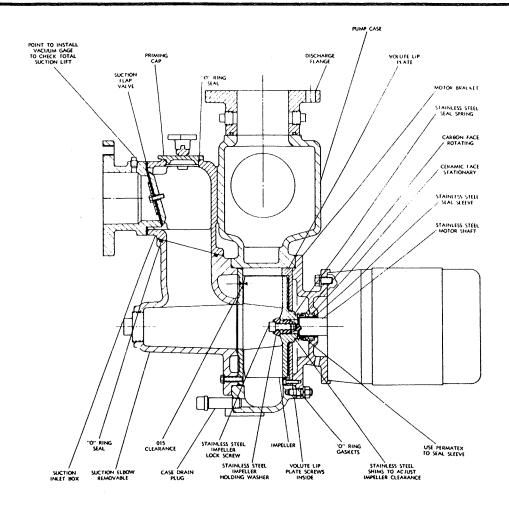


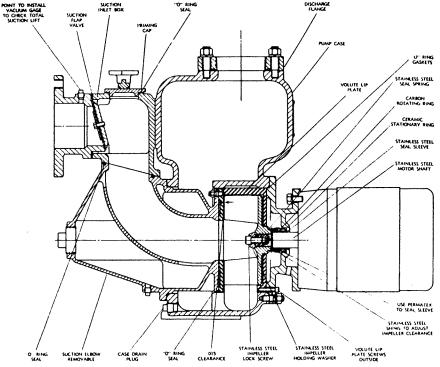
GPM _	100	TDH	241	
A Section of				

EFF. 30 BHP 3

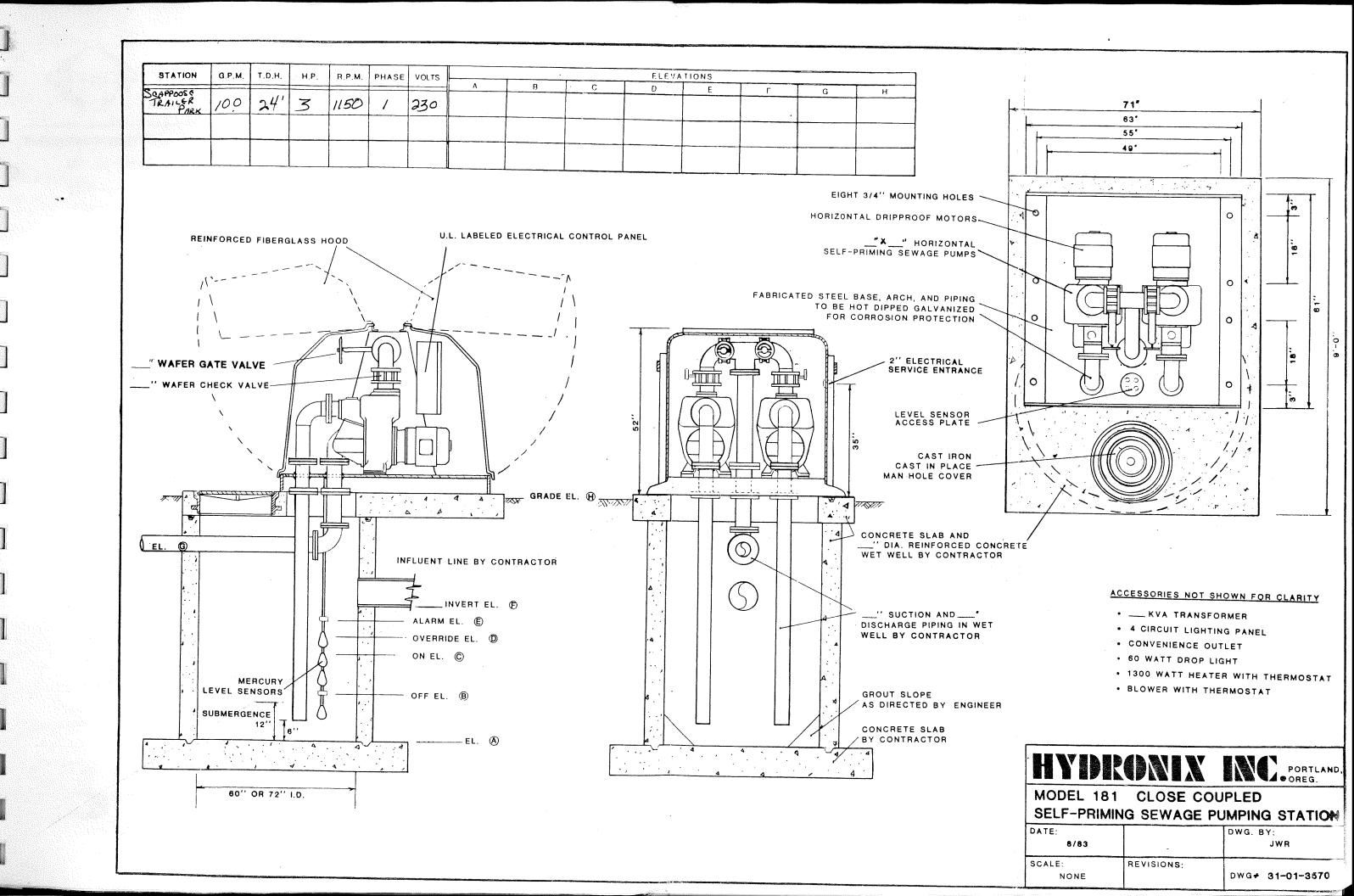
POH & PSIKZII NPSHR 8

SECTIONAL DRAWINGS OF SELF PRIMING, CLOSE COUPLED SEWAGE PUMPS









D-3: Keys Landing Pump Station

PUMP STATION DATA SHEET

LOCATION KEYS CANDING SERIAL NO.
OWNER SCAPOOSE OR ENGINEER DAVE WINSHIP
CONTRACTOR MORRILL STROBET CONSTIDISTRIBUTOR HADRONIK
STATION TYPE 183 Wet Well 6' Dia. X 13' Deep Wet Well Cover
DATE INSTALLED
<u>PUMP</u> <u>STATION</u>
PIPING: Suction 4, Disch. 4; Suction 4, Disch. 4
CONDITIONS OF SERVICE
Design Duty: 120 GPM, 46 T.D.H., 13' Suction Lift; Liquid enul
Solids 3", N.P.S.H. (AVL.), (REQ.)
PUMP DATA
Pump Model: 40MDV Imp. Dia. 97/32 Priming: Self, (1) Vacuum,
()Flooded. Stuffing Box Type wrth. Lubrication 61(
Rotation of Pump #1 <u>cw</u> , #2 <u>cw</u> , #3
MOTOR DATA
Brand CHOICE, Enclosure ODP, Horsepower 5
RPM /750, Phase 3, Cycle 60, Volt 230, Starting Code,
ModificationFrame
ELECTRICAL DATA
3 Phase, 60 Cycle, 230 Volts, 4 Wire, 120 V Control Voltage.
Transformer Required: ()YES, 🎾 NO. Size



TO:

Merrill Strobel Construction 54444 Kalberer Rd. Scappoose, OR 97056

QUOTATION #	91172				
Date 5-28-92	Page # _	1	_of	2	pages
2425 SE Ochoco					
Portland, OR 97222					
503-659-6230 FAX 5	03-659-8	718			
1-800-547-9708	1/6				
1-800-547-9708 Kee	ps Land	ing			
Contact Person: Mer	rill St	robe	1		
Phone: H-543-2581		_ FA	Κ:		
M-936-4303					

QTY.	DESCRIPTION	
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 receptable 4. 60 watt drop light 5. 13/15 M watt heaters 6. Spare parts kit 7. 0 & M manuals 8. T.D. no flow limit switches 9. 156 CFM blower 5. 13/15 M watt heaters Estimated shipping weight is 2,500 pounds Start up included	
	Total Price	

Nove quoted price good for:30_daysOelivery in Approximately6_ wrs• Terms0	• FOB:Portland, Oregon <u>FFA</u>
THIS QUOTATION AND ANY AGREEMENT TO SELL RI HYDRONIX GENERAL TERMS AND CONDITIONS OF S CONDITIONS ON BACK, SIDE.	
co. Name Myrril Strapped Con 3/4 Wefrin	Hydronix, Inc.

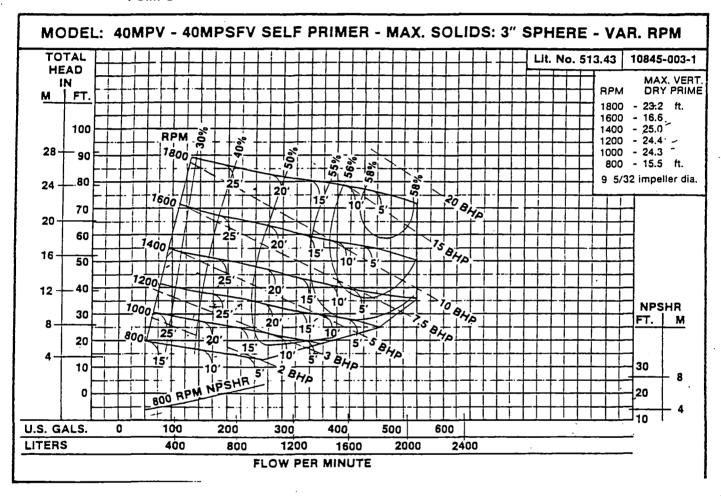
Accepted by: Michael Strobe! Proposed by: Michael atterson. Sa



HYDROMATIC PUMPS

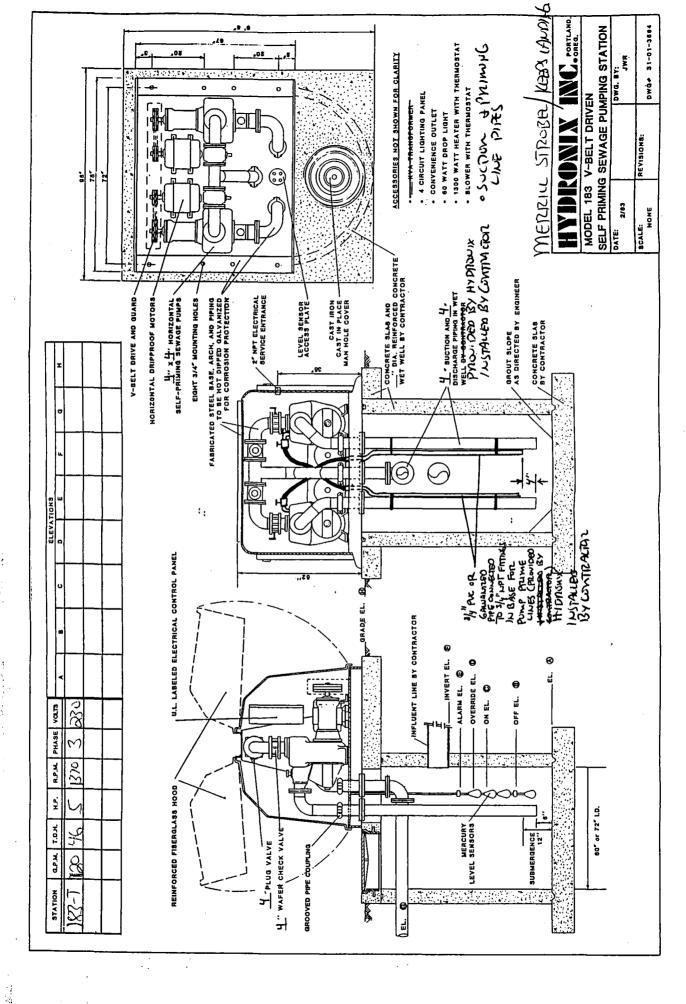
HYDROMATIC PUMPS

SECTION 510 PERFORMANCE DATA



GPM _	120	TDH	46
-------	-----	-----	----

NPSHR /



()

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	<u>183</u> Wet Well <u>8</u> Dia. x <u>19</u> [Deep Wet Well Cover
Date Installe	d June, 1996	
<u>Pump</u>	Station	
Piping: Suct	tion <u>4"</u> ,Disch <u>4"</u> ;Suction <u>4</u>	<u>'_,Disch_ 4"_</u>
Conditions of	of Service	
Design Duty:	300 GPM, 42' TDH, Suction Li	ft; Liquid Sewage
Solids 3",	NPSH (avl), (req)	·
Driven RPM_	1400	
Sheave_4.75>	<u>κ2 Groove</u> , Bushing SH 1-3/8, , Β	elt_3VX475
Pump Data:	Serial No.:	
Pump Model	: 40MP ,lmp.Dia 9-5/32" Priming	:(X)self,()vacuum
()flooded. St	tuffing Box Type, Lubrication	on: <u>Media</u>
Rotation of F	Pump #1_ <u>CW_</u> ,#2_ <u>CW_</u> ,#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 <u>E180-3PB</u>

Control Panel

Location Inside Station NEMA Type 4

Service Entrance SizeAmps	<u>Pump #1</u>	Pump #2	<u>Pump #3</u>
Circuit Breaker-Trip Rating/AMPS Magnetic Starter-NEMA size	30	<u>30</u>	
Overload Heater size	2013B-3	2013B-3	

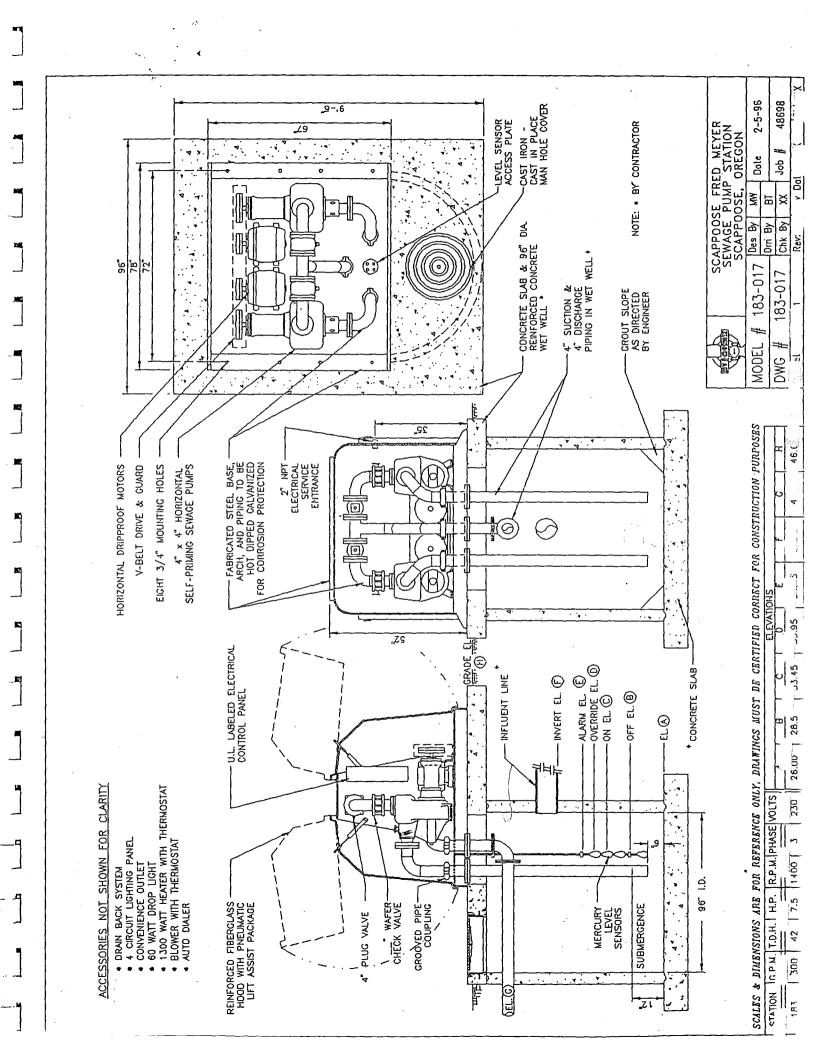
Alternator Type 67C-1COA Mfg Warrick Coil Voltage 110

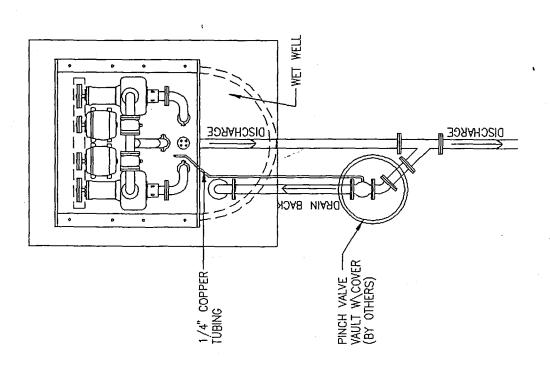
Alarm Functions: Pump 1 & 2 fail, High wet well, Compressor fail

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





PRESSURE RECULATING RELUER VALVE
VALVE SET © 70 PSI SET AT 70 PSI

-UNILOADING VALVE

Ш

1/4 HP AIR COMPRESSOR

SECONDARY PRESSURE GAUGE

3-WAY SOLENOID VALVE

LOW AIR PRESSURE SWITCH

120V MOTOR WIRING -1/4" CALVANIZED STEEL PIPE

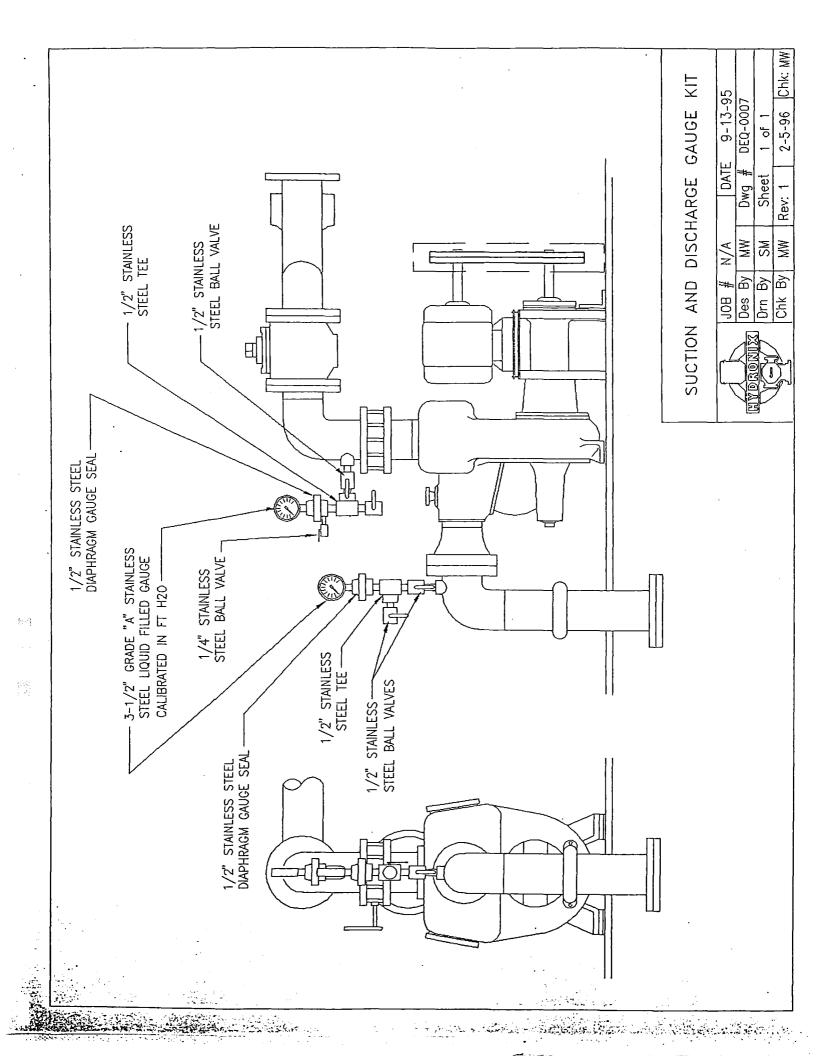
DIFFERENTAL PRESSURE SWITCH

TO PUMP STATION MOTOR CONTROL CENTER

H2S TREATMENT DRAINBACK SYSTEM

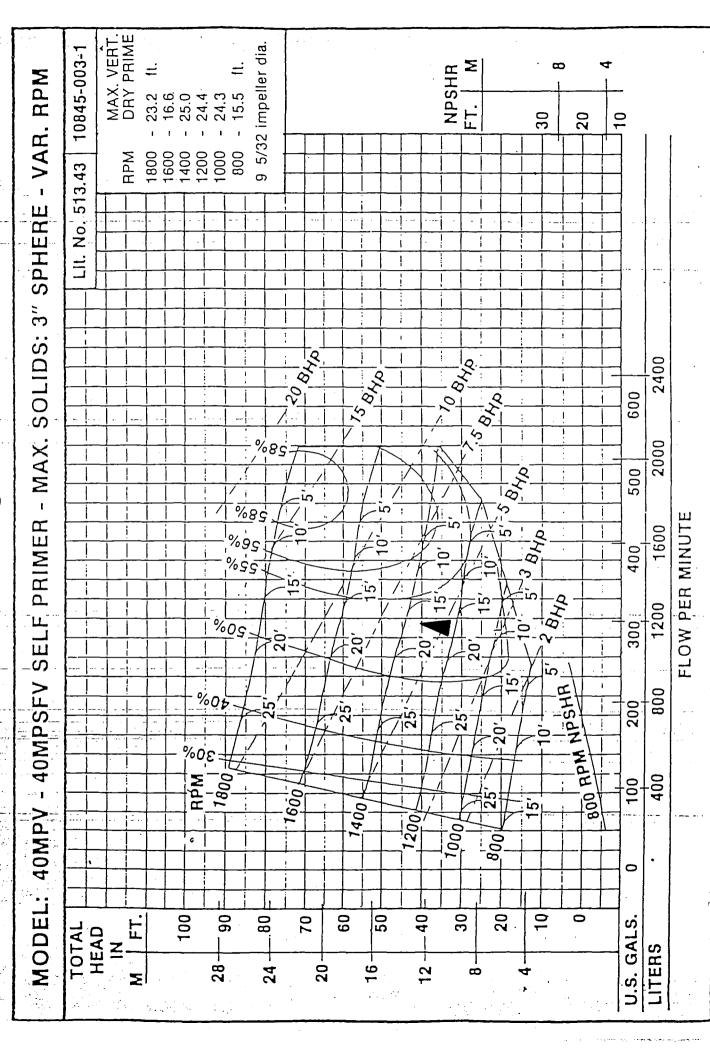
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5			Chk: MW
	DEQ-0002	1 of 1	2-5-96
DATE	∯ 6w0	Sheet	Rev: 1
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JOB	Des	Orn	Chk By
	MY BRANKE		
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DRAINBACK	
42S DRAINBACK CONTROL SCHEMATIC	



T-O-HOTI

300 G.P.M. @ 42' T.D.H.



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 <u>:</u>	Hydronix
Station Type	<u> 183 Wet Well 84 Dia. x 16' Dee</u>	ep Wet Well (Cover
Date Installe	d_4/96_		
Pump	Station		
Piping: Suc	tion_4"_,Disch_4"; Suction_4",Dis	sch. <u>4"</u>	
Conditions of	of Service		
Design Duty	150GPM, 45 TDH, Suction Lift; I	Liquid <u>Sewag</u>	<u>e</u>
Solids <u>3"</u> , N	PSH (avl), (req)		
Driven RPM_	1350		
Sheave_3V5.	3x2 Gr. ,Bushing SH 1-1/2 ,Belt 3	VX400	
Pump Data:	Serial No.:		
Pump Model	40MP_,lmp.Dia. 9 5/32_Priming:(X)self,()vac	uum
()flooded. St	uffing Box Type, Lubrication	on: <u>Media</u>	_ •
Rotation of P	ump #1 <u>CW ,</u> #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 #

<u>Pump #1</u>

Pump #2

Circuit Breaker-Trip Rating/AMPS

Magnetic Starter-NEMA size

Overload Heater size

__50_

AE16FNSOAC H2013B-3 __50_

AE16FNSOAC 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
Station Disch.	8.91
Station Floor_	-3.0
WW Invert	4.56
WW Floor	

Low Water_____Pump Off___-0.5
Lead Pump On___3.07
2 Pump Overload__3.57
High Water____4.07

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_____

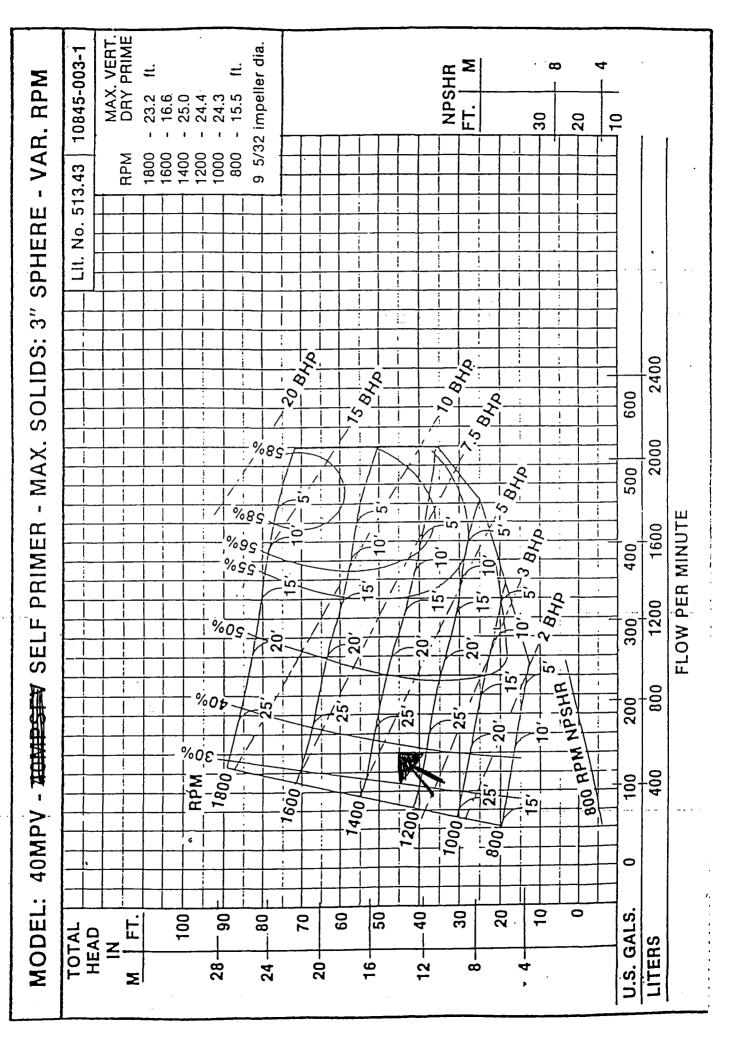
Application Engineering

Prepared By Butch Kline Date 4/22/96

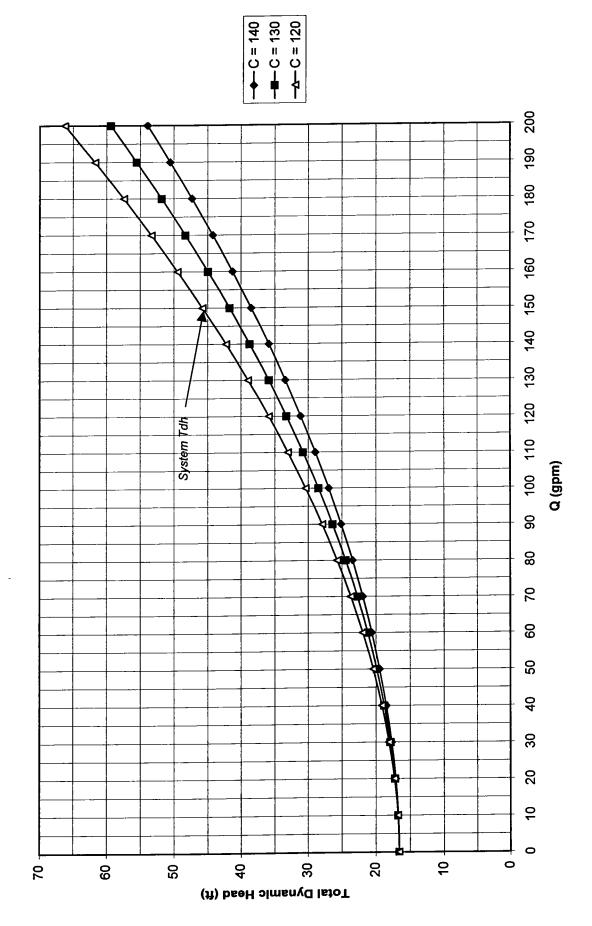
Sawna Sawna

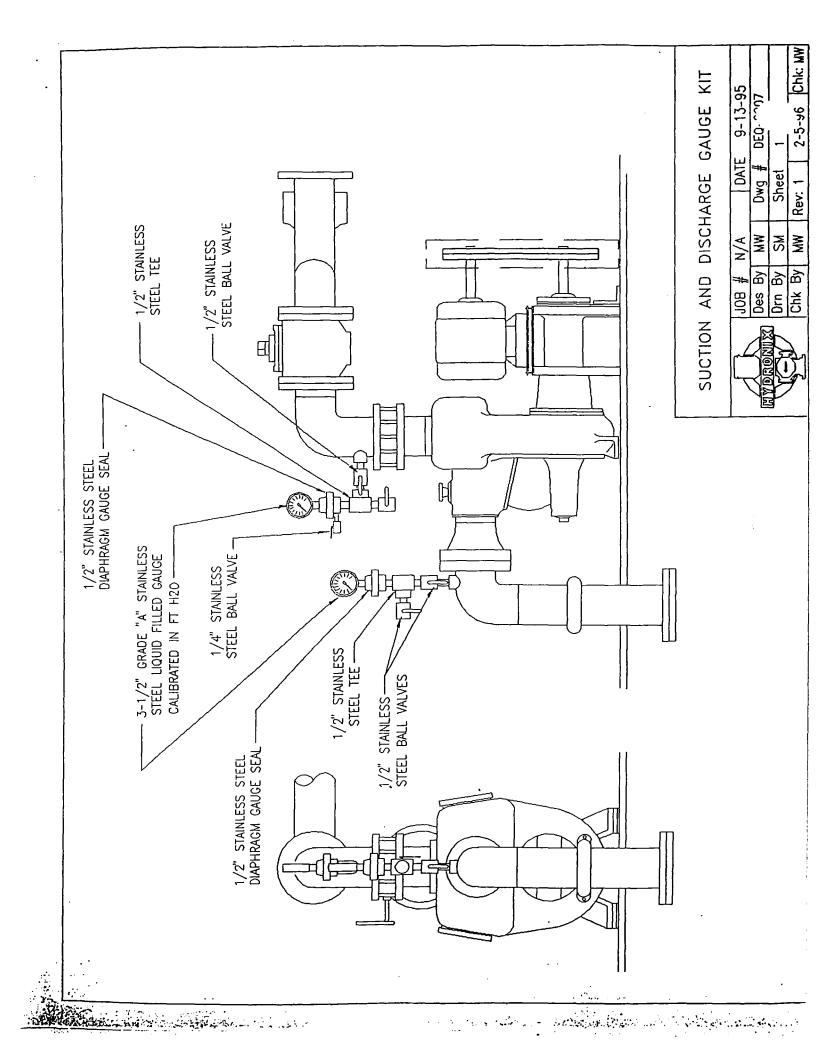
ULS NC. SEC PERFORMAN

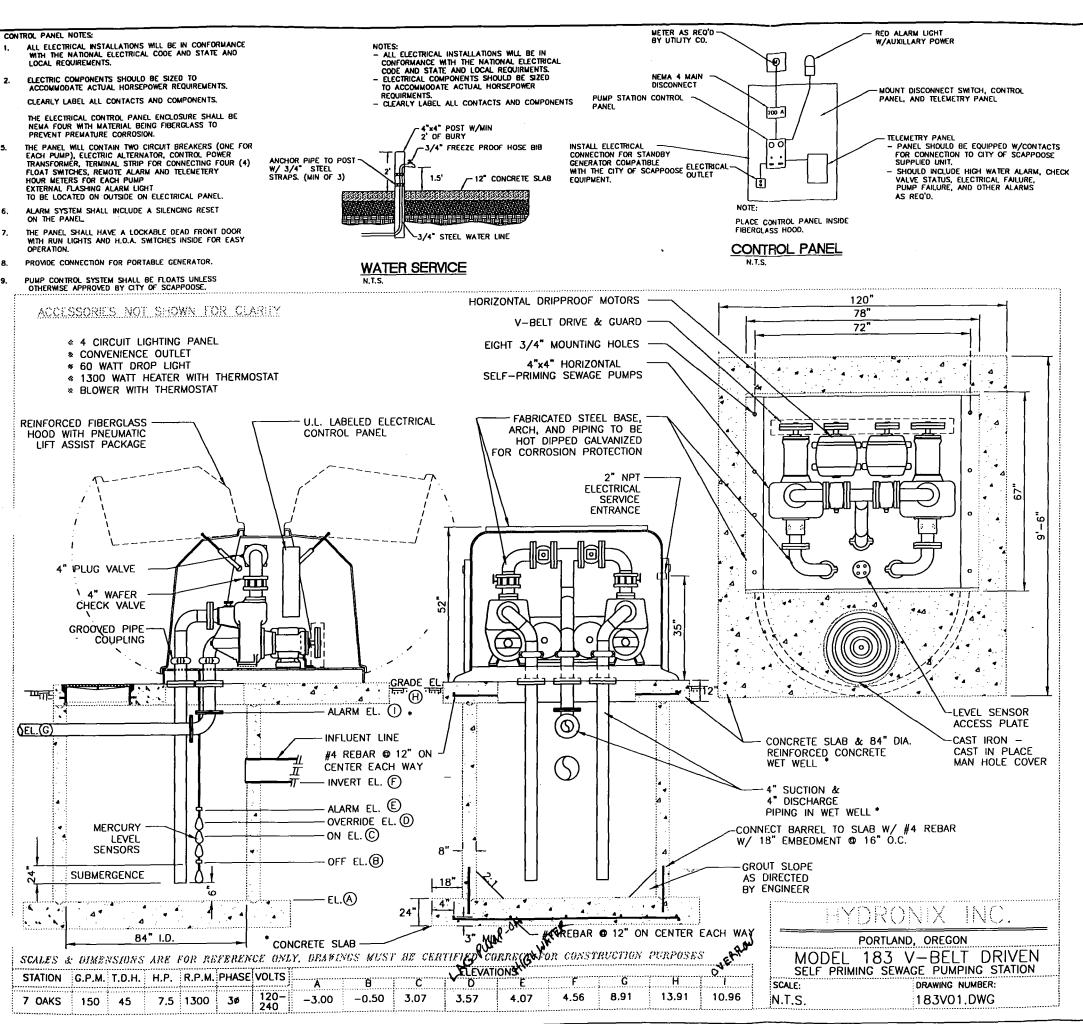
145 G.P.w. @ 45 T.D.H.



System Curves for Varies C Values







PUMP STATION NOTES

- THE CONTRACTOR STALL FURNISH AND INSTALL TWO SUFRACE 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
- PUMP DESIGN: THE PUMPS SHALL BE Y-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.
- I. 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 TOPY AS
- 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.
- 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 CHANCEABLE 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"-O CRUSHED ROCK COMPACTED TO
 95% OF AASHTO T-180. AND THE WETWELL
 WILL BE BACKFILLED WITH 3/4"-O CRUSHED
- 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 44 REBAR AT 12" ON CENTER. TO KEEP THE BASE FROM SEPERATING FOR THE SIDES.



MNWR P

LANDSCAPE ARCHITECTS
233 SW FRONT
PORTLAND, OR
97204

503|225|0822 FAX: 503|273|8353





SOCIATES

SEVEN OAKS SUBDIVISION PUMP STATION DETAILS
SS-BROD-GOODRICH AND ASSOCIA

REV. | DATE | BY

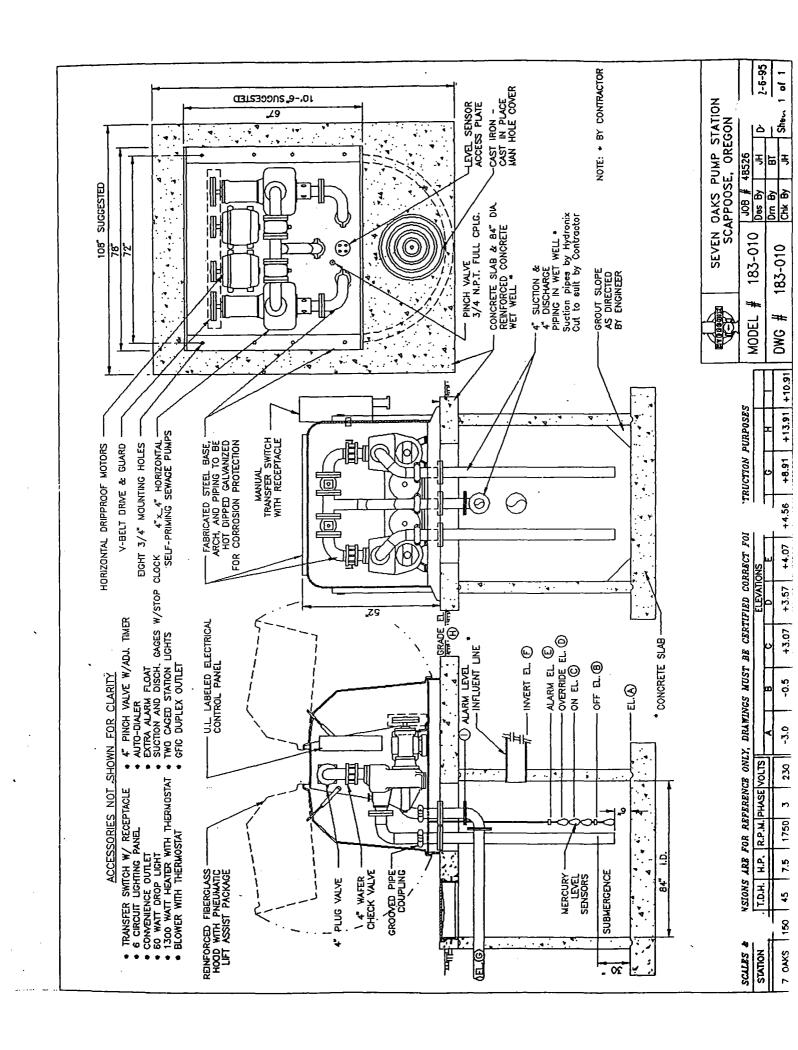
PROJECT NO.

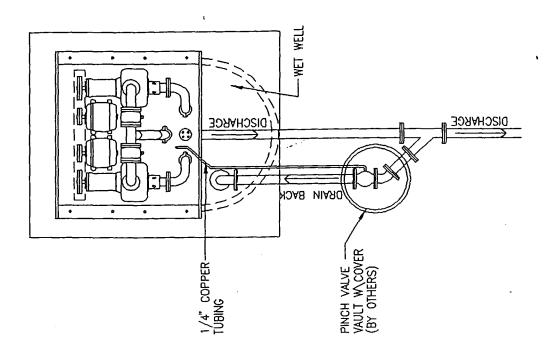
DATE | 10/19/95
DESIGNED | K. ACKERMA
ENGINEER | D. WELBORN
CHECKED | J. REIMANN

SHEET TITLE --

DETAILS

C13A





REUEF VALVE SET AT 70 PSI

PRESSURE RECULATING VALVE SET @ 70 PSI

--- UNLOADING VALYE

1/4 HP AIR COMPRESSOR

() T(

SECONDARY PRESSURE GAUGE

PRESSURE ONLOGE

-1/4" GALVANIZED STEEL PIPE

DIFFERENTIAL PRESSURE SWITCH 1/4" COPPER— TUBING

3-WAY SOLENOID VALVE

LOW AIR PRESSURE SWITCH

120V MOTOR WIRING

TO PUMP STATION MOTOR CONTROL CENTER

H2S DRAINBACK CONTROL SCHEMATIC

H2S TREATMENT DRAINBACK SYSTEM

3-10	DEQ-0	1 of 1	
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#	Ву	Ву	C
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SEWAGE PUMP STATION DESIGN DATA

PUMP STATION
TYPE
PUMP TYPE
CAPACITY
PUMP HP
LEVEL CONTROL
OVERFLOW POINT
OVERFLOW DISCHARGE
AVG. TIME TO OVERFLOW
AUXILIARY POWER TYPE
LOCATION
OUTPUT
FUEL CAPACITY
TRANSFER SWITCH
ALARM TELEMETRY TYPE
EPA RELIABILITY CLASS

LOCATED • SOUTH END OF 9TH STREET SURFACE MOUNTED SELF PRIMING CONSTANT-SPEED NON-CLOG 150 GPM • 45FT TDH 7.5 HP FLOATS

FLOATS
SSMH#9 ELEV=10.96 (INT. OF SEVEN OAKS DR. & 9TH STREET)
STORM SYSTEM ON SITE
15.75 HRS
3.6 CENERATOR
CITY SHOPS
N/A
N/A
MANUAL
AUTODIALER

AUTODIALER

FORCE MAIN
LENGTH, TYPE
PROFILE
DISCHARCE MANHOLE
VACUUM RELEASE VALVE
AVERAGE DETENTION
SULFIDE CONTROL SYSTEM

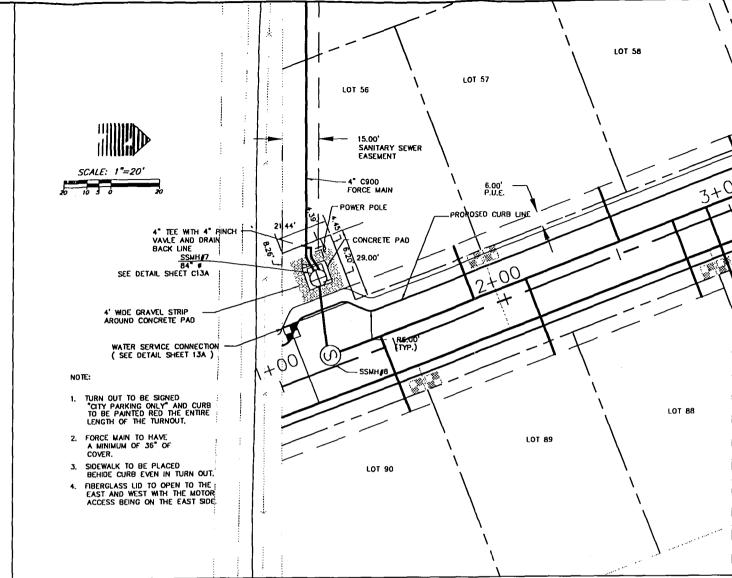
1,565LF OF C900
ASCENDING TO STA. 4+30 THEN CONTINUOUS DECENDING
SIXTH STREET
ONE AT HIGH POINT (STA 4+30)
B5 MIN
BACKORAIN

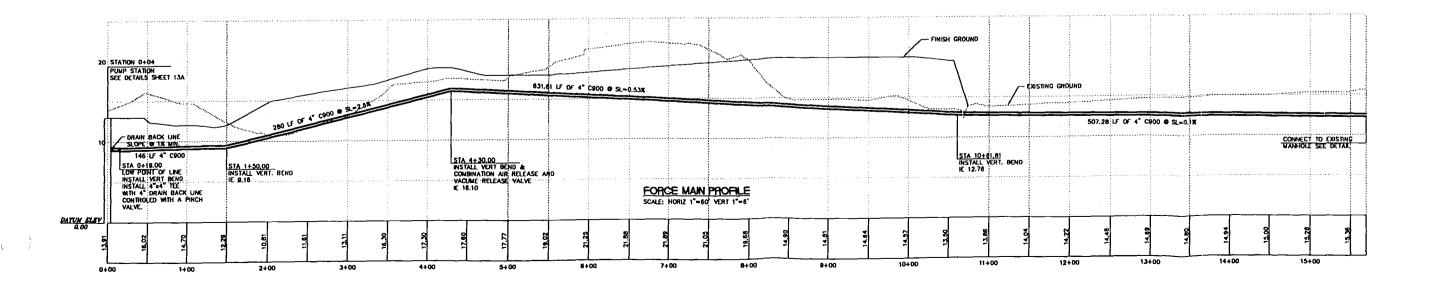
BACKDRAIN SYSTEM CONTROL VALVE TYPE VALVE SIZE

PNUMATIC PINCH

NOTE:

BEFORE INSTALLATION, THE SIZE AND MATERIAL OF THE PINCH VALVE VAULT AND COVER NEEDS TO BE REVIEWED BY THE CITY OF SCAPPOOSE.





MNWRIP PLANNERS ENGINEERS LANDSCAPE ARCHITECTS

233 SW FRONT PORTLAND, OR 97204

503 225 0822 FAX: 503|273|8353

ENGINEER 15,190

ORECON CONTROL OF MARK WELD

EXP 6/30/95 PAN

KOSS-BROD-GOODRICH AND ASSOCIATES 22595 DAY RD. WEST LINN, OPECON 97068

SEVEN OAKS SUBDIVISION FORCE MAIN PROFILE \ PUMP STATION

REV. DATE | BY

PROJECT NO. **KBG001**

DATE 10/26/95 DESIGNED K. ACKERMAN ENGINEER | D. WELBORN CHECKED J. REIMANN

SHEET TITLE -PROFILE

SHEET NUMBER-



APPENDIX E – Smoke Testing & CCTV

E-1: Smoke Testing Results Table

E-2: Smoke Testing Results





4. 52828 NW Five Peak Ter Open C/O (around back of house)









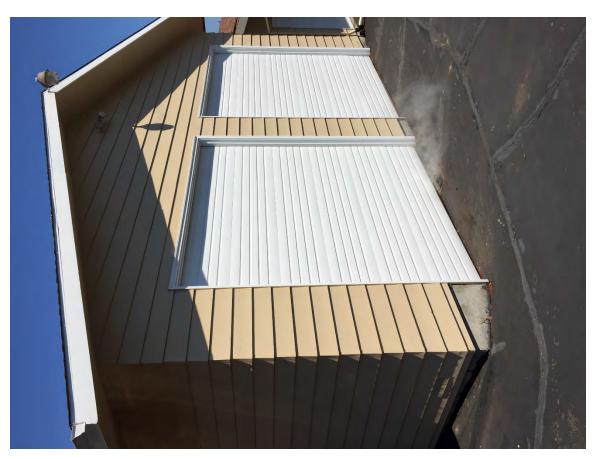


Cross connection; downspout on front porch smoking 8. 52895 NE 7th St



11. 52122 Hoag Ter Cross connection; downspout, driveway drain smoking





12. 33114 Felisha Way Cross connection; driveway drain connected to sewer line



Open C/O in front; under construction, will most likely be sealed later on

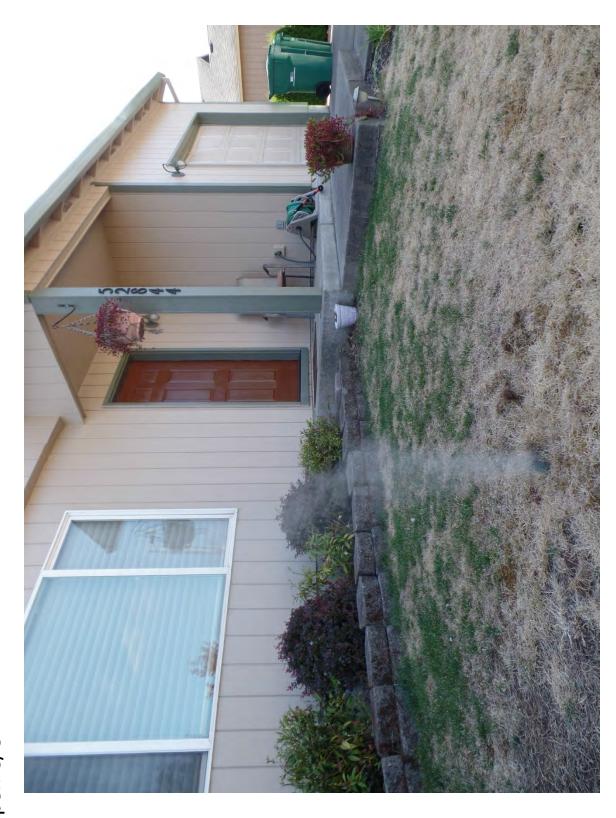
13. Ashley Ct and JP West Rd (SE corner house)

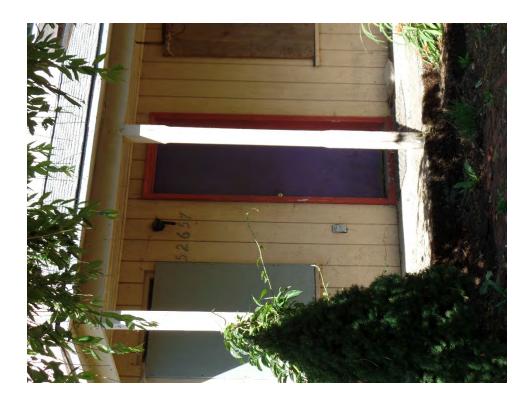




Cross connection; downspout over garage smoking 18. 33343 SW Rogers Rd



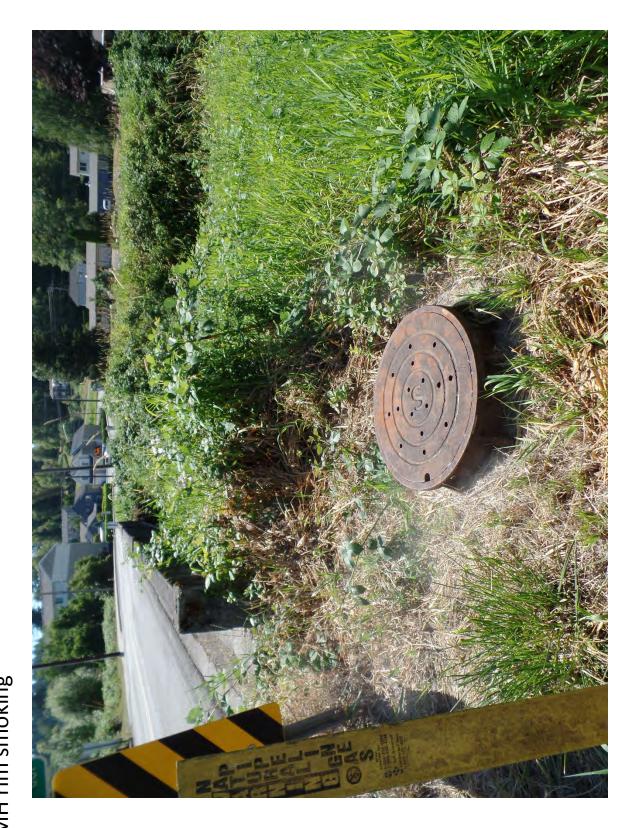














24. 52313 Columbia River Hwy (US Bank) Indoor; light smoke in the bathroom

26. 52753 NE Kern Ct Open C/O (in metal valve can, but uncapped inside can)















APPENDIX F - Model Data & Results

F-1: Existing Link Data

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2015; 5-
Flows (2
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sting S ₁
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LAISTING SYSTEM FIOWS (ZOIS), 3-Year, Z4-11041 STOTIN EVENT,	(2013, 3-yea	1, 24-11-cui cui					[Ī
Link Name	Us Invert (ft)	Us 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	9.9	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	9.9	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	2002	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	2002	PVC	0.011	370.91	1.75	0.47	0	0	0	12.79	0
GM0140	48.94	48.62	2001	PVC	0.011	92	0.833	0.42	95'0	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	DVC	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	7	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	0007	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	IQ	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	9.0	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	96'0	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	Э	0.013	206.3	1	0.19	3.24	1.59	2.642	1.57	6.839
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	6.0	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	80.0	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	U	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	6.0	0.46

362.8 0.667 49.68 0.667 155.71 0.667 203.49 0.667 197.28 0.667 197.28 0.667 40.048 0.667 162.4 0.667 300.01 0.667 242.53 0.667 242.53 0.667 166.26 1 166.26 1 166.26 1 166.27 1.5 153.65 1 165.75 1.5 153.67 1 178				(ft/s)	(#/s)	(CIS)	(cts)	
47.07 46.93 1981 PVC 0.011 49.68 0.667 49.7 48.7 47.83 1981 PVC 0.011 155.71 0.667 49. 48.87 1981 PVC 0.011 137.28 0.667 50.47 49 1981 PVC 0.011 197.28 0.667 52.16 52.27 1981 PVC 0.011 107.39 0.667 52.16 52.27 1981 PVC 0.011 107.48 0.667 52.16 52.27 1981 PVC 0.011 107.48 0.667 52.16 52.27 1981 PVC 0.011 102.48 0.667 52.89 52.36 1981 PVC 0.011 10.24 1 44.2 44.2 1980 C 0.013 126.21 1 44.8 1980 C 0.013 126.21 1 45.8 45.4 1980 C 0.013 <td>362.8</td> <td>0.667</td> <td>0.4</td> <td>2.44</td> <td>0.51</td> <td>0.382</td> <td>0.91</td> <td>0.463</td>	362.8	0.667	0.4	2.44	0.51	0.382	0.91	0.463
48.77 47.83 1981 PVC 0.011 155.71 0.667 50.47 49 1981 PVC 0.011 203.49 0.667 50.47 49 1981 PVC 0.011 20.349 0.667 52.17 50.57 1981 PVC 0.011 400.48 0.667 52.18 52.26 1981 PVC 0.011 20.043 0.667 53.89 52.36 1981 PVC 0.011 30.041 0.667 53.89 52.36 1981 PVC 0.011 20.23 0.667 44.8 44.8 1980 C 0.013 16.26 1 44.8 44.8 1980 C 0.013 16.26 1 45.8 45.4 1980 C 0.013 16.26 1 47.8 45.4 1980 C 0.013 18.62 1 47.8 45.4 1980 C 0.013	49.68	0.667	0.28	2.27	0.45	0.397	0.76	0.508
49 4887 1981 PVC 0.011 203-49 0.667 50,47 49 1981 PVC 0.011 107.28 0.667 52,17 50,47 49 1981 PVC 0.011 107.04 0.667 52,18 52,27 1981 PVC 0.011 102.43 0.667 52,19 53.99 1981 PVC 0.011 20.043 0.667 43,29 53.99 1981 PVC 0.011 20.01 0.667 43,29 1981 PVC 0.011 20.04 0.667 44,8 44,2 1980 C 0.013 166.48 1 44,8 44,2 1980 C 0.013 166.26 1 45,8 45,8 1980 C 0.013 166.26 1 45,8 45,8 1980 C 0.013 156.27 1 47,8 45,8 1980 C 0.013 1	155.71	0.667	9.0	2.93	0.58	0.403	1.11	0.417
50,47 49 1981 PVC 0.011 197.28 0.667 52,17 50,57 1981 PVC 0.011 140.48 0.667 52,19 55,19 53.99 1981 PVC 0.011 160.48 0.667 55,19 53.99 1981 PVC 0.011 242.53 0.667 44,8 44,8 42,9 1980 C 0.013 166.26 1 44,8 44,8 44,8 1980 C 0.013 166.26 1 44,8 44,8 1980 C 0.013 166.26 1 44,8 44,8 1980 C 0.013 165.21 1 44,8 45,6 1980 C 0.013 165.21 1 45,6 45,8 1980 C 0.013 165.21 1 45,8 45,8 1980 C 0.013 18.36 1 47,8 47,3 47,3	203.49	0.667	90.0	1.51	0.25	0.391	0.36	0.728
52.17 50.57 1981 PVC 0.011 400.48 0.667 52.36 52.37 1981 PVC 0.011 162.4 0.667 55.13 52.36 1981 PVC 0.011 106.4 0.667 53.89 52.36 1981 PVC 0.011 242.33 0.667 44.2 43.99 1980 C 0.013 166.48 1 44.48 44.2 1980 C 0.013 166.24 1 45.16 44.85 1980 C 0.013 166.3 1 45.4 45.1 1980 C 0.013 166.3 1 45.4 45.6 1980 C 0.013 166.3 1 45.7 45.8 45.9 1980 C 0.013 1 1 45.7 46.78 45.9 1980 PVC 0.011 156.2 1 47.2 46.78 1980 PVC	197.28	0.667	0.75	2.84	0.5	968'0	1.23	0.728
52.36 52.27 1981 PVC 0011 162.4 0.667 55.19 53.59 1981 PVC 0.011 30.01 0.667 55.19 53.59 1981 PVC 0.011 26.23 0.667 44.2 44.2 1980 C 0.013 166.28 1 44.8 44.2 1980 C 0.013 166.26 1 44.8 44.2 1980 C 0.013 166.26 1 45.4 45.4 1980 C 0.013 166.26 1 45.8 45.8 1980 C 0.013 166.26 1 45.8 45.8 1980 PVC 0.011 40.25 1 45.8 45.8 1980 PVC 0.011 40.85 1 47.22 45.8 1980 PVC 0.011 48.85 1 47.22 46.78 1980 PVC 0.011 42.85	400.48	0.667	0.4	2.43	0.43	0.384	6.0	0.466
55.19 53.99 1981 PVC 0.011 300.01 0.667 53.89 52.36 1981 PVC 0.011 242.53 0.667 44.48 44.2 1980 C 0.013 156.21 1 44.85 44.48 14.2 1980 C 0.013 166.26 1 44.85 44.85 1980 C 0.013 166.26 1 45.68 45.48 1980 C 0.013 166.26 1 45.84 45.68 1980 C 0.013 166.26 1 45.84 45.68 1980 C 0.013 166.26 1 45.88 45.68 1980 C 0.013 1 1 46.78 45.99 1980 C 0.013 1 1 47.73 1980 C 0.013 135.6 1 1 47.84 47.39 1980 C 0.013 18.	162.4	0.667	90.0	1.56	0.21	0.407	0.34	0.73
53.89 52.36 1981 PVC 0.011 24.25 0.667 44.2 43.39 1980 C 0.013 16.48 1 44.8 44.2 1980 C 0.013 16.626 1 44.85 44.85 1980 C 0.013 16.626 1 45.4 45.16 1980 C 0.013 16.626 1 45.4 45.16 1980 C 0.013 16.626 1 45.8 45.6 1980 C 0.013 16.62 1 45.8 45.6 1980 C 0.013 104 1 46.78 45.68 1980 C 0.013 104 1 47.3 47.22 1980 C 0.011 40.26 1 47.3 47.22 1980 PVC 0.011 40.26 1 47.3 47.22 1980 PVC 0.013 18.8 1 <	300.01	0.667	0.4	2.49	0.34	0.422	6.0	0.492
44.2 43.99 1380 C 0.013 56.48 1 44.48 44.2 1380 C 0.013 126.21 1 44.48 44.2 1980 C 0.013 166.26 1 45.16 44.48 1980 C 0.013 165.37 1 45.4 45.16 1980 C 0.013 165.3 1 45.8 45.4 1980 C 0.013 165.3 1 45.8 45.6 1980 C 0.013 165.1 1 47.2 46.78 1980 PVC 0.011 40.26 1 47.22 46.78 1980 PVC 0.011 40.26 1 47.22 46.78 1980 PVC 0.011 48.56 1 47.33 47.22 1980 PVC 0.011 48.85 1 47.34 47.12 1972 DI 0.012 18.8 1	242.53	0.667	0.63	2.78	0.39	0.422	1.14	0.73
44.48 44.2 1980 C 0.013 126.21 1 44.85 44.48 1980 C 0.013 166.26 1 45.6 44.85 1980 C 0.013 166.26 1 45.8 45.4 1980 C 0.013 166.26 1 45.8 45.4 1980 C 0.013 126.21 1 45.8 45.6 1980 C 0.013 126.21 1 45.8 45.6 1980 C 0.013 126.21 1 45.8 45.9 1980 C 0.011 402.6 1 47.2 46.78 47.32 1980 C 0.011 183.6 1 47.8 47.32 1980 C 0.013 183.7 1 47.8 47.73 1980 C 0.013 183.7 1 3.2 47.1 1973 C 0.013 183.8 <	96.48	1	0.22	-2.71	0	-2.21	1.66	7.21
44.85 44.48 1980 C 0.013 166.26 1 45.16 44.85 1980 C 0.013 156.77 1 45.4 45.16 1980 C 0.013 156.75 1 45.8 45.64 1980 C 0.013 156.75 1 45.8 45.84 1980 C 0.013 156.75 1 46.78 45.9 1980 PVC 0.011 402.76 1 46.78 46.78 1980 PVC 0.011 155.6 1 47.30 47.22 1980 PVC 0.011 155.6 1 47.84 47.73 1980 PVC 0.011 155.6 1 47.85 47.73 1980 PVC 0.013 28.85 1 47.87 47.73 1975 DI 0.012 18.85 1 47.87 32.4 1973 C 0.013 14.8 <t< td=""><td>126.21</td><td>1</td><td>0.22</td><td>-2.72</td><td>0</td><td>-2.211</td><td>1.68</td><td>6.655</td></t<>	126.21	1	0.22	-2.72	0	-2.211	1.68	6.655
45.16 44.85 1980 C 0.013 136.37 1 45.4 45.16 1980 C 0.013 104 1 45.8 45.8 45.6 1980 C 0.013 53.65 1 45.8 45.8 45.8 1980 C 0.013 53.65 1 46.78 45.8 1980 PVC 0.011 402.6 1 46.78 45.9 1980 PVC 0.011 402.6 1 47.22 46.78 1980 PVC 0.011 402.6 1 47.73 47.22 1980 PVC 0.011 48.85 1 47.73 47.39 1980 PVC 0.011 48.85 1 47.73 47.39 1980 PVC 0.011 48.85 1 47.73 47.39 1980 PVC 0.011 48.85 1 47.74 47.112 1973 C 0.01	166.26	1	0.22	1.46	0	0.362	1.68	5.92
45.4 45.16 1980 C 0.013 104 1 45.68 45.4 1980 C 0.013 126.21 1 45.68 45.4 1980 C 0.013 53.65 1 45.78 45.68 1980 PVC 0.011 1402.76 1 47.22 46.78 1980 PVC 0.011 1402.76 1 47.23 47.29 1980 PVC 0.011 1402.76 1 47.73 47.39 1980 PVC 0.011 1402.76 1 47.73 47.39 1980 PVC 0.011 184.85 1 47.73 47.39 1980 PVC 0.011 184.85 1 47.8 47.73 1980 PVC 0.011 184.85 1 47.8 47.72 1980 PVC 0.013 28.85 1 41.0 0 1973 C 0.013 34.8	136.37	1	0.23	1.47	0	0.357	1.7	5.577
45.68 45.4 1980 C 0.013 126.21 1 45.8 45.68 1980 C 0.013 53.65 1 46.78 45.68 1980 C 0.011 402.76 1 47.22 46.73 1980 PVC 0.011 84.85 1 47.73 47.39 1980 PVC 0.011 184.85 1 47.73 47.39 1980 PVC 0.011 184.85 1 47.73 47.73 1980 C 0.013 28.87 1 47.73 47.73 1980 C 0.013 28.87 1 47.8 47.73 1980 C 0.013 28.87 1 43.76 41.12 1972 DI 0.012 18.8 1 43.76 41.12 1973 C 0.013 28.7 1 32.47 32.54 34.52 1973 C 0.013 28.	104	1	0.23	1.46	0.01	0.349	1.71	5.289
45.8 45.68 1980 C 0.013 53.65 1 46.78 45.9 1980 PVC 0.011 402.76 1 47.22 46.78 1980 PVC 0.011 195.6 1 47.39 47.22 1980 PVC 0.011 184.85 1 47.8 47.73 1980 C 0.013 29.87 1 0.1 0.1 0.01 1975 DI 0.012 185 0.667 43.76 41.12 1972 DI 0.012 185 0.667 43.76 41.12 1973 CI 0.013 185 0.667 32.47 32.07 1973 C 0.013 143 1 34.8 32.5 1973 C 0.013 143 1 35.5 34.2 1973 C 0.013 188.7 1 35.5 35.5 35.5 1973 C 0.013	126.21	1	0.22	1.44	0	0.35	1.68	5.064
46.78 45.9 1980 PVC 0.011 402.76 1 47.22 46.78 1980 PVC 0.011 195.6 1 47.32 47.22 1980 PVC 0.011 195.6 1 47.33 47.39 1980 PVC 0.011 153.11 1 47.8 47.73 1980 PVC 0.013 29.87 1 47.8 47.73 1972 DI 0.012 185.7 1.5 47.8 47.73 1972 DI 0.012 185.7 1.5 43.76 41.12 1972 DI 0.012 185.7 1.5 32.47 32.07 1973 C 0.013 34.8 1 34.22 1973 C 0.013 184.8 1 35.25 1973 C 0.013 188.5 1 38.26 1973 C 0.013 188.2 1 38.26 3	53.65	1	0.22	1.44	0	0.348	1.68	4.801
47.22 46.78 1980 PVC 0.011 195.6 1 47.39 47.22 1980 PVC 0.011 84.85 1 47.39 47.29 1980 PVC 0.011 153.11 1 47.8 47.73 1980 PC 0.013 29.87 1 47.8 47.73 1980 C 0.013 29.87 1 47.8 47.73 1980 C 0.013 29.87 1 43.6 41.12 1972 DI 0.012 185 0.667 33.4 41.12 1973 C 0.013 178 1 34.8 32.4 1973 C 0.013 143 1 35.5 34.9 1973 C 0.013 143 1 35.5 34.9 1973 C 0.013 143 1 35.5 35.9 35.9 1973 C 0.013 33.45 <	402.76	1	0.22	1.51	0.01	0.317	1.97	4.588
47.39 47.22 1980 PVC 0.011 84.85 1 47.73 47.39 1980 PVC 0.011 153.11 1 47.73 47.73 1980 C 0.013 29.87 1 0.1 0.1 0.1 0.01 185 0.667 43.76 41.12 1972 DI 0.012 185 0.667 33.4 32.4 1973 C 0.013 178 1 34.2 32.4 1973 C 0.013 143 1 34.2 32.57 1973 C 0.013 143 1 34.8 32.57 1973 C 0.013 143 1 35.28 32.57 1973 C 0.013 143 1 35.28 32.57 1973 C 0.013 18.5 1 35.25 34.9 1973 C 0.013 38.7 1 36.26	195.6	1	0.22	1.46	-0.01	0.291	2	3.755
47.73 47.39 1980 PVC 0.011 153.11 1 47.8 47.73 1980 C 0.013 29.87 1 0.1 0.1 0 1975 DI 0.012 65.75 1.5 47.8 47.73 1980 C 0.013 29.87 1 1 43.76 41.12 1973 CI 0.012 185 0.667 15 33.4 32.47 32.07 1973 C 0.013 178 1 34.22 33.73 1973 C 0.013 225 1 1 33.73 32.96 1973 C 0.013 143 1 1 34.22 33.75 1973 C 0.013 158.5 1 1 35.88 32.57 1973 C 0.013 28.201 1 1 36.60 38.74 1973 C 0.013 28.23 1 1	84.85	1	0.2	1.41	0	0.275	1.88	3.332
47.8 47.73 1980 C 0.013 29.87 1 0.1 0 1975 DI 0.012 65.75 1.5 43.76 41.12 1972 DI 0.012 185 0.667 33.4 32.4 1973 CI 0.015 100 0.833 32.47 32.07 1973 C 0.013 178 1 33.42 33.73 1973 C 0.013 178 1 34.22 33.73 1973 C 0.013 225 1 33.73 32.96 1973 C 0.013 348 1 35.8 33.57 1973 C 0.013 158.5 1 36.6 1973 C 0.013 28.20 1 1 38.5 38.6 1973 C 0.013 28.23 1 39.7 48.6 1973 C 0.013 25.66 1	153.11	1	0.22	1.42	0.01	0.254	1.98	3.168
0.1 0 1975 DI 0.012 65.75 1.5 43.76 41.12 1972 DI 0.012 185 0.667 33.4 32.4 1973 CI 0.015 100 0.833 32.47 32.07 1973 C 0.013 178 1 34.22 33.73 1973 C 0.013 348 1 32.88 32.57 1973 C 0.013 348 1 33.89 32.57 1973 C 0.013 143 1 34.8 34.22 1973 C 0.013 143 1 35.25 34.9 1973 C 0.013 18.65 1 35.26 36.07 1973 C 0.013 18.25 1 38.1 37.47 1973 C 0.013 28.39 1 39.2 48.61 1973 C 0.013 28.39 1 <	29.87	1	0.23	1.31	0	0.242	1.72	2.837
43.76 41.12 1972 DI 0.015 185 0.667 33.4 32.4 1973 CI 0.015 100 0.833 32.47 32.07 1973 C 0.013 178 1 34.22 33.73 1973 C 0.013 348 1 32.88 32.57 1973 C 0.013 348 1 32.89 32.57 1973 C 0.013 143 1 34.8 34.22 1973 C 0.013 143 1 35.25 34.9 1973 C 0.013 158.5 1 35.27 1973 C 0.013 158.5 1 36.26 36.07 1973 C 0.013 158.5 1 37.47 36.86 1973 C 0.013 282.01 1 38.56 38.56 1973 C 0.013 251.66 1 48.61 48.74 1973 C 0.013 251.66 1	65.75	1.5	0.15	3.77	1.9	4.448	4.44	0.645
33.4 32.4 1973 CI 0.015 100 0.833 32.47 32.07 1973 C 0.013 178 1 34.22 33.73 1973 C 0.013 225 1 33.73 32.96 1973 C 0.013 348 1 32.88 32.57 1973 C 0.013 143 1 38.8 32.57 1973 C 0.013 143 1 38.1 34.9 1973 C 0.013 158.5 1 38.50 36.26 36.07 1973 C 0.013 183.7 1 38.1 37.47 1973 C 0.013 282.01 1 1 38.5 38.5 1973 C 0.013 282.39 1 1 38.6 1973 C 0.013 28.60 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 <t< td=""><td>185</td><td>0.667</td><td>1.43</td><td>3.72</td><td>0.82</td><td>0.39</td><td>1.57</td><td>0.34</td></t<>	185	0.667	1.43	3.72	0.82	0.39	1.57	0.34
32.47 32.07 1973 C 0.013 178 1 34.22 33.73 1973 C 0.013 225 1 33.73 32.96 1973 C 0.013 348 1 32.88 32.57 1973 C 0.013 143 1 34.8 34.22 1973 C 0.013 158.5 1 35.25 34.9 1973 C 0.013 158.5 1 36.26 36.07 1973 C 0.013 282.01 1 37.47 36.86 1973 C 0.013 282.39 1 38.56 38.5 1973 C 0.013 282.39 1 38.56 38.56 1973 C 0.013 25.166 1 50.55 48.71 1973 C 0.013 25.166 1 48.61 48.61 1973 C 0.013 25.166 1 48.61 48.74 1973 C 0.013 25.39.1 1.25	100	0.833	1	2.04	0	1.114	1.9	4.122
34.22 33.73 1973 C 0.013 225 1 33.73 32.96 1973 C 0.013 348 1 32.88 32.57 1973 C 0.013 143 1 34.8 34.22 1973 C 0.013 158.5 1 35.25 34.9 1973 C 0.013 158.5 1 36.26 36.07 1973 C 0.013 282.01 1 37.47 36.86 1973 C 0.013 282.39 1 38.1 37.47 1973 C 0.013 282.39 1 38.56 1973 C 0.013 251.66 1 38.56 1973 C 0.013 251.66 1 50.55 48.71 1973 C 0.013 251.66 1 48.61 48.61 47.87 1973 C 0.013 251.66 1 48.74 47.87 1973 C 0.013 258.91 1.25	178	1	0.22	3.15	0	2.509	1.69	4.111
33.73 32.96 1973 C 0.013 348 1 32.88 32.57 1973 C 0.013 143 1 34.8 34.22 1973 C 0.013 158.5 1 35.25 34.9 1973 C 0.013 158.5 1 35.26 36.26 1973 C 0.013 158.5 1 36.26 36.07 1973 C 0.013 83.71 1 37.47 36.86 1973 C 0.013 282.39 1 38.56 38.5 1973 C 0.013 251.66 1 39.2 48.71 1973 C 0.013 251.66 1 50.55 48.71 1973 C 0.013 251.66 1 48.61 47.87 1973 C 0.013 251.66 1 48.61 48.61 1973 C 0.013 258.68 1.25 48.74 47.87 1973 C 0.013 268.29 1.25 <td>225</td> <td>1</td> <td>0.22</td> <td>3.11</td> <td>0</td> <td>2.501</td> <td>1.66</td> <td>5.315</td>	225	1	0.22	3.11	0	2.501	1.66	5.315
32.88 32.57 1973 C 0.013 143 1 34.8 34.22 1973 C 0.013 213.42 1 35.25 34.9 1973 C 0.013 158.5 1 35.26 36.07 1973 C 0.013 282.01 1 36.26 36.07 1973 C 0.013 83.71 1 37.47 36.86 1973 C 0.013 282.39 1 38.56 38.5 1973 C 0.013 251.66 1 39.2 48.71 1973 C 0.013 251.66 1 50.55 48.71 1973 C 0.013 251.66 1 48.61 48.24 1973 C 0.013 258.88 1.25 48.61 47.87 1973 C 0.013 28.991 1.25 48.74 47.86 1973 C 0.013 268.29	348	1	0.22	3.12	0	2.503	1.68	4.884
34.8 34.22 1973 C 0.013 213.42 1 35.25 34.9 1973 C 0.013 158.5 1 35.97 35.35 1973 C 0.013 282.01 1 36.26 36.07 1973 C 0.013 83.71 1 37.47 36.86 1973 C 0.013 282.39 1 38.56 38.5 1973 C 0.013 282.39 1 39.2 38.56 1973 C 0.013 251.66 1 50.55 48.71 1973 C 0.013 251.66 1 48.61 48.24 1973 C 0.013 253.69 1 48.61 47.87 1973 C 0.013 268.29 1.25 48.24 47.87 1973 C 0.013 268.29 1.25 48.24 47.87 1973 C 0.013 268.29 1.25 47.87 47.86 1973 C 0.013 268.29	143	1	0.22	3.14	0	2.505	1.66	4.296
35.25 34.9 1973 C 0.013 158.5 1 35.97 35.35 1973 C 0.013 282.01 1 36.26 36.07 1973 C 0.013 83.71 1 37.47 36.86 1973 C 0.013 282.39 1 38.5 38.5 1973 C 0.013 282.39 1 38.5 38.6 1973 C 0.013 251.66 1 50.55 48.71 1973 C 0.013 251.66 1 50.55 48.71 1973 C 0.013 251.66 1 48.61 48.24 1973 C 0.013 238.88 1.25 48.24 47.87 1973 C 0.013 268.29 1.25 48.24 47.86 1973 C 0.013 268.29 1.25 47.87 47.86 1973 C 0.013 268.29	213.42	1	0.27	3.09	0.07	2.495	1.86	5.598
35.97 35.35 1973 C 0.013 282.01 1 36.26 36.07 1973 C 0.013 83.71 1 37.47 36.86 1973 C 0.013 383.75 1 38.1 37.47 1973 C 0.013 282.39 1 38.56 38.2 1973 C 0.013 251.66 1 50.55 48.71 1973 C 0.013 251.66 1 52.19 50.55 1973 C 0.013 276.17 1.25 48.61 48.24 1973 C 0.013 238.88 1.25 48.24 47.87 1973 C 0.013 268.29 1.25 47.87 47.46 1973 C 0.013 268.29 1.25 47.87 47.46 1973 C 0.013 276.3 1.25 47.46 45.3 1973 C 0.013 276.3	158.5	1	0.22	3.08	1.24	2.49	1.67	5.799
36.26 36.07 1973 C 0.013 83.71 1 37.47 36.86 1973 C 0.013 303.75 1 38.1 37.47 1973 C 0.013 282.39 1 38.56 38.2 1973 C 0.013 251.66 1 50.55 48.71 1973 C 0.013 276.17 1.25 48.61 48.74 1973 C 0.013 276.17 1.25 48.61 48.24 1973 C 0.013 238.88 1.25 48.24 47.87 1973 C 0.013 268.29 1.25 47.87 47.46 1973 C 0.013 268.29 1.25 47.46 45.3 1973 C 0.013 276.3 1.25	282.01	1	0.22	3.04	1.45	2.47	1.67	6.2
37.47 36.86 1973 C 0.013 303.75 1 38.1 37.47 1973 C 0.013 282.39 1 38.56 38.2 1973 C 0.013 166.62 1 39.2 38.66 1973 C 0.013 251.66 1 50.55 48.71 1973 C 0.013 276.17 1.25 48.61 48.24 1973 C 0.013 238.88 1.25 48.24 47.87 1973 C 0.013 268.29 1.25 47.87 47.46 1973 C 0.013 268.29 1.25 47.87 47.46 45.3 1973 C 0.013 276.3 1.25	83.71	1	0.23	3.26	1.55	2.643	1.7	6.342
38.1 37.47 1973 C 0.013 282.39 1 38.56 38.2 1973 C 0.013 166.62 1 39.2 38.66 1973 C 0.013 251.66 1 50.55 48.71 1973 C 0.013 276.17 1.25 48.61 48.24 1973 C 0.013 238.88 1.25 48.61 48.24 1973 C 0.013 239.91 1.25 47.87 47.46 1973 C 0.013 268.29 1.25 47.86 45.3 1973 C 0.013 276.3 1.25	303.75	1	0.2	3.21	1.57	2.626	1.6	7.713
38.56 38.2 1973 C 0.013 166.62 1 39.2 38.66 1973 C 0.013 251.66 1 50.55 48.71 1973 C 0.013 276.17 1.25 52.19 50.55 1973 C 0.013 238.88 1.25 48.61 48.64 1973 C 0.013 239.91 1.25 48.24 47.87 1973 C 0.013 268.29 1.25 47.87 47.46 1973 C 0.013 276.3 1.25 47.46 45.3 1973 C 0.013 276.3 1.25	282.39	1	0.22	3.17	1.68	2.606	1.68	8.465
39.2 38.66 1973 C 0.013 25.166 1 50.55 48.71 1973 C 0.013 276.17 1.25 52.19 50.55 1973 C 0.013 238.88 1.25 48.61 48.24 1973 C 0.013 239.91 1.25 48.24 47.87 1973 C 0.013 268.29 1.25 47.87 47.46 1973 C 0.013 276.3 1.25 47.46 45.3 1973 C 0.013 241.55 1.25	166.62	1	0.22	3.15	1.64	2.593	1.66	8.808
50.55 48.71 1973 C 0.013 276.17 1.25 52.19 50.55 1973 C 0.013 238.88 1.25 48.61 48.24 1973 C 0.013 239.91 1.25 48.24 47.87 1973 C 0.013 268.29 1.25 47.87 47.46 1973 C 0.013 276.3 1.25 47.46 45.3 1973 C 0.013 241.55 1.25	251.66	1	0.21	3.11	1.59	2.566	1.65	9.349
52.19 50.55 1973 C 0.013 238.88 1.25 48.61 48.24 1973 C 0.013 239.91 1.25 48.24 47.87 1973 C 0.013 268.29 1.25 47.87 47.46 1973 C 0.013 276.3 1.25 47.46 45.3 1973 C 0.013 241.55 1.25	276.17	1.25	0.67	4.04	1.34	2.571	5.27	0.759
48.61 48.24 1973 C 0.013 239.91 1.25 48.24 47.87 1973 C 0.013 268.29 1.25 47.87 47.46 1973 C 0.013 276.3 1.25 47.46 45.3 1973 C 0.013 241.55 1.25	238.88	1.25	0.69	4.31	1.14	2.564	5.35	0.493
48.24 47.87 1973 C 0.013 268.29 1.25 47.87 47.46 1973 C 0.013 276.3 1.25 47.46 45.3 1973 C 0.013 241.55 1.25	239.91	1.25	0.15	2.43	0.86	2.589	2.54	0.839
47.87 47.46 1973 C 0.013 276.3 1.25 47.46 45.3 1973 C 0.013 241.55 1.25	268.29	1.25	0.14	2.38	0.95	2.593	2.4	0.838
47.46 45.3 1973 C 0.013 241.55 1.25	276.3	1.25	0.15	2.65	1.06	2.595	2.49	0.78
	241.55	1.25	0.89	4.76	2.17	2.608	6.11	0.469
GM0708	506.38	1.25	1.56	5.36	3.06	2.66	8.07	1.14

GM0709 19.11 11.19 1973 C O. GM0710 29.26 19.11 1973 C O. GM0711 40.24 29.26 1973 C O. GM0712 40.12 40.84 1973 C O. GM0714 40.12 40.12 1973 C O. GM0724 45.3 44.12 1973 C O. GM0725 20.91 18.4 1973 C O. GM0726 20.91 18.4 1973 C O. GM0727 10.03 7.3 1973 C O. GM0727 10.93 7.24 5.65 1973 C O. GM0730 4.64 4.22 1973 C O. GM0731 4.64 4.22 1973 C O. GM0732 5.26 2.36 1973 C O. GM0733 3.26 2.83	iviateriai nougiiiless	1112111111	E))	(ft/s)	(ft/s)	(cts)	(cfs)	75 45
29.26 19.11 1973 C 32.47 29.26 1973 C 40.43 32.47 1973 C 44.12 40.8 1973 C 44.12 40.93 1973 C 30.96 20.91 1973 C 20.91 18.4 1003 7.3 1973 C 10.03 7.3 1973 C C 4.64 4.22 1973 C C 4.11 3.78 1973 C C 5.24 5.24 4.22 1973 C 2.65 2.36 1973 C C 5.04 4.64 1973 C C 5.04 4.64 1973 C C 5.04 4.64 1973 C C 6.33 5.49 1973 C C 7.16 6.33 1973 C C 6.34 0.52 1973 C C 7.26<	C 0.013	486.49	1.25	1.63	5.98	2.97	2.659	8.24	0.395
32.47 29.26 1973 C 40.43 32.47 1973 C 40.43 32.47 1973 C 44.12 40.8 1973 C 20.91 18.4 1973 C 20.91 18.4 1973 C 18.4 10.03 1.33 1973 C 10.03 7.3 1973 C 1.04 4.22 1973 C 1.03 7.3 1973 C 1.04 4.22 1973 C 4.64 4.22 1973 C 4.64 4.22 1973 C 2.24 5.24 4.22 1973 C 3.68 3.26 1.973 C 2.62 2.36 1973 C 2.54 4.22 1973 C 2.54 4.42 1973 C 2.62 2.36 1973 C 2.54 1.42 1973 C 3.56 1.57 1973<	C 0.013	528.42	1.25	1.92	6.31	3.11	2.653	8.95	0.391
40.43 32.47 1973 C 44.12 40.8 1973 C 45.3 44.12 1973 C 30.96 20.91 1973 C 20.91 18.4 10.03 1973 C 18.4 10.03 1973 C 10.03 7.3 1973 C 10.03 7.3 1973 C 10.03 7.3 1973 C 10.03 3.26 1973 C 4.64 4.22 1973 C 4.64 4.22 1973 C 2.62 2.36 1973 C 2.62 2.36 1973 C 2.62 2.36 1973 C 2.62 2.36 1973 C 5.04 4.64 1973 C 6.33 5.49 1973 C 5.04 4.64 1973 C 1.34 1973 C C 2.55 0.2 1973 C	C 0.013	349.15	1.25	0.92	4.56	2.24	2.644	6.19	0.49
44.12 40.8 1973 C 45.3 44.12 1973 C 30.96 20.91 1973 C 20.91 18.4 1973 C 20.91 18.4 1003 1973 C 10.03 7.3 1973 C C 10.03 7.3 1973 C C 5.24 5.14 1973 C C 4.64 4.22 1973 C C 4.64 4.22 1973 C C 3.26 2.83 1973 C C 3.26 2.83 1973 C C 5.04 4.64 1973 C C 6.33 5.49 1973 C C 6.34 5.04 4.64 1973 C C 6.35 5.04 4.64 1973 C C 6.35 1.36 1973 C C C 6.25 2.62 1973 C C C<	C 0.013	298.88	1.25	5.66	6.71	3.2	2.63	10.54	0.49
45.3 44.12 1973 C 30.96 20.91 1973 C 20.91 18.4 1973 C 18.4 10.03 1973 C 18.4 10.03 7.3 1973 C 10.03 7.3 1973 C C 10.03 7.3 1973 C C 4.64 4.22 1973 C C 4.64 4.22 1973 C C 3.68 3.26 1973 C C 2.62 2.83 1973 C C 3.26 2.83 1973 C C 5.04 4.64 1973 C C 6.33 5.49 1973 C C 5.04 4.64 1973 C C 6.33 5.49 1973 C C 6.34 5.04 4.64 1973 C 6.35 5.04 4.64 1973 C 6.34 1.37 1973 C 7.16 6.33 1973 C 8.26 1.27 1973 C 4.1.23 40.7 197	C 0.013	353.62	1.25	0.94	4.87	2.34	2.614	6.26	0.451
30.96 20.91 1973 C 20.91 18.4 1973 C 18.4 10.03 7.3 1973 C 10.03 7.3 1973 C 7.24 5.65 1973 C 7.24 5.65 1973 C 7.24 5.65 1973 C 4.64 4.22 1973 C 4.64 4.22 1973 C 4.64 4.22 1973 C 5.24 5.14 1973 C 2.65 2.83 1973 C 5.04 4.64 1973 C 6.33 5.49 1973 C 6.33 5.49 1973 C 6.33 5.40 1973 C 1.34 1.40 1973 C 1.54 42.05 1973 C 42.11 42.05 1973 C 43.25 42.15 1973 C 41.61 43.26 1973 C <td>C 0.013</td> <td>140.75</td> <td>1.25</td> <td>0.84</td> <td>4.64</td> <td>2.13</td> <td>2.611</td> <td>5.91</td> <td>0.469</td>	C 0.013	140.75	1.25	0.84	4.64	2.13	2.611	5.91	0.469
20.91 18.4 1973 C 18.4 10.03 7.3 1973 C 10.03 7.3 1973 C 7.24 5.65 1973 C 7.24 5.65 1973 C 7.24 5.14 1973 C 4.64 4.22 1973 C 4.64 4.22 1973 C 3.68 3.26 1973 C 2.62 2.83 1973 C 2.62 2.84 1973 C 5.04 4.64 1973 C 6.33 5.49 1973 C 7.16 6.33 1973 C 6.33 5.49 1973 C 7.16 6.33 1973 C 6.33 5.49 1973 C 7.16 6.33 1973 C 8.25 1.27 1973 C 42.01 41.61 41.23 1973 C 42.15 42.15 1973	C 0.013	494.46	0.833	2.03	2.83	2.04	0.143	3.12	0.17
18.4 10.03 193 1973 C 10.03 7.3 1973 C 7.24 5.65 1973 C 5.24 5.14 1973 C 4.64 4.22 1973 C 4.64 4.22 1973 C 4.11 3.78 1973 C 3.68 3.26 1.83 1973 C 2.62 2.83 1973 C C 5.04 4.64 1973 C C 6.33 5.49 1973 C C 6.33 5.49 1973 C C 6.33 5.49 1973 C C 7.16 6.33 1.973 C C 6.33 5.49 1973 C C 7.16 6.33 1.973 C C 8.50 1.27 1973 C C 4.1.61 41.61	C 0.013	149.79	0.833	1.68	2.9	2.08	0.179	2.83	0.17
10.03 7.3 1973 C 7.24 5.65 1973 C 5.24 5.14 1973 C 4.64 4.22 1973 C 4.11 3.78 1973 C 4.11 3.78 1973 C 3.68 3.26 1973 C 2.62 2.83 1973 C 2.62 2.36 1973 C 2.95 2.62 1973 C 5.04 4.64 1973 C 6.33 5.49 1973 C 6.33 5.49 1973 C 1.34 0.52 1973 C 6.34 1.40 1973 C 42.05 0.2 1973 C 42.11 42.05 1973 C 43.61 43.25 1973 C 43.61 43.25 1973 C 40.7 39.78	C 0.013	452.82	0.833	1.85	2.93	2.11	0.188	2.98	0.215
7.24 5.65 1973 C 5.24 5.14 1973 C 4.64 4.22 1973 C 4.11 3.78 1973 C 4.11 3.78 1973 C 3.68 3.26 1973 C 3.68 3.26 1973 C 2.95 2.62 1973 C 2.95 2.62 1973 C 5.04 4.64 1973 C 6.33 5.49 1973 C 6.34 6.34 1973 C 7.16 6.33 1973 C 6.34 4.64 1973 C 1.34 0.94 0.52 1973 C 6.35 1.25 1973 C 1 4.205 1.13 1973 C 1 4.1.51 41.23 1973 C 1 4.1.61 41.23 1973 C 1 4.1.61 42.46 1973 C 1	C 0.013	320.07	0.833	0.85	2.38	1.72	0.205	2.02	0.215
5.24 5.14 1973 C 4.64 4.22 1973 C 4.11 3.78 1973 C 4.11 3.78 1973 C 3.68 3.26 1973 C 3.26 2.83 1973 C 2.62 2.36 1973 C 2.95 2.62 1973 C 2.95 2.62 1973 C 6.33 5.04 4.64 1973 C 6.33 5.04 4.64 1973 C 6.34 0.52 1973 C C 1.54 1.43 1973 C C 42.05 1.43 1973 C C 42.05 4.181 1973 C C 42.05 4.181 1973 C C 43.51 42.15 1973 C C 43.52 42.15 1973 C C 40.7 39.79 1973 C C 44.05	C 0.013	130.72	0.833	1.22	2.72	1.97	0.211	2.41	0.27
4.64 4.22 1973 C 4.11 3.78 1973 C 3.68 3.26 1973 C 3.26 2.83 1973 C 2.62 2.36 1973 C 2.95 2.62 1973 C 2.95 2.62 1973 C 7.16 6.33 1973 C 6.33 5.49 1973 C 6.33 5.49 1973 C 7.16 6.33 1973 C 6.33 5.04 1973 C 7.36 1.57 1973 C 7.20 1.81 1973 C 42.05 41.81 1973 C 43.61 42.05 1973 C 43.61 43.25 1973 C 41.61 41.23 40.7 1973 C 40.7 39.79 1973 C 40.7 39.78 1973 C 45.46 43.86 1973 C </td <td>C 0.013</td> <td>66.1</td> <td>1.25</td> <td>0.15</td> <td>2.32</td> <td>1.1</td> <td>1.435</td> <td>2.51</td> <td>0.508</td>	C 0.013	66.1	1.25	0.15	2.32	1.1	1.435	2.51	0.508
4.11 3.78 1973 C 3.68 3.26 1973 C 3.26 2.83 1973 C 2.62 2.36 1973 C 2.95 2.62 1973 C 7.16 6.33 1973 C 6.33 5.49 1973 C 6.33 5.49 1973 C 6.33 5.04 4.64 1973 C 6.34 0.52 1973 C C 7.05 0.24 1973 C C 42.05 1.97 1973 C C 42.05 41.81 1973 C C 43.61 43.25 1973 C C 43.61 43.25 1973 C C 41.63 40.7 1973 C C 40.7 39.79 1973 C C 40.7 39.84 1973 C C 46 45.46 1973 C C	C 0.013	280.43	1.25	0.15	2.14	1.03	1.471	2.5	0.581
3.68 3.26 1973 C 3.26 2.83 1973 C 2.62 2.36 1973 C 2.62 2.36 1973 C 7.16 6.33 1973 C 6.33 5.49 1973 C 6.33 5.04 4.64 1973 C 6.33 6.34 1973 C C 7.04 0.52 1973 C C 1.33 0.94 1973 C C 42.05 1.97 1973 C C 42.05 41.81 1973 C C 43.07 42.05 1973 C C 43.07 42.05 1973 C C 41.03 40.7 1973 C C 40.7 39.79 1973 C C 41.05 39.8 1973 C C 46 45.46 1973 C C 46 45.46 1973 C	C 0.013	222.56	1.25	0.15	1.98	1.05	1.451	2.49	0.809
3.26 2.83 1973 C 2.62 2.36 1973 C 2.95 2.62 1973 C 7.16 6.33 1973 C 6.33 5.49 1973 C 5.04 4.64 1973 C 6.33 5.04 1973 C 7.05 0.52 1973 C 1.34 0.64 1973 C 42.11 42.05 1973 C 42.05 41.81 1973 C 42.05 41.81 1973 C 43.61 42.05 1973 C 43.61 42.05 1973 C 41.61 41.23 1973 C 40.7 39.79 1973 C 46.7 43.86 1973 C 46.46 45.46 1973 C 45.46 43.86 1973 C 45.46 43.86 1973 C 45.46 43.86 1973 C </td <td>C 0.013</td> <td>277.68</td> <td>1.25</td> <td>0.15</td> <td>1.81</td> <td>1.12</td> <td>1.442</td> <td>2.51</td> <td>1.127</td>	C 0.013	277.68	1.25	0.15	1.81	1.12	1.442	2.51	1.127
2.62 2.36 1973 C 2.95 2.62 1973 C 7.16 6.33 1973 C 6.33 5.49 1973 C 5.04 4.64 1973 C 6.33 5.04 4.64 1973 C 1.33 0.94 1973 C C 1.54 1.43 1973 C C 1.54 1.43 1973 C C 42.05 0.2 1973 C C 42.05 41.81 1973 C C 42.05 41.81 1973 C C 43.01 43.25 1973 C C 41.03 40.7 1973 C C 40.7 39.79 1973 C C 46.6 45.46 1973 C C 46.4 43.86 1973 C C 46.4 43.86 1973 C C 45.46 43.86 1973	C 0.013	285.5	1.25	0.15	1.5	0.67	1.443	2.51	1.363
2.95 2.62 1973 C 7.16 6.33 1973 C 6.33 5.49 1973 C 6.34 4.64 1973 C 6.35 6.94 0.52 1973 C 1.33 0.94 1973 C C 1.54 1.43 1973 C C 2.36 1.97 1973 C C 42.05 41.81 1973 C C 42.05 41.81 1973 C C 42.05 41.81 1973 C C 43.25 42.15 1973 C C 41.61 41.23 1973 C C 40.7 39.79 1973 C C 40.7 39.88 1973 C C 46.46 45.46 1973 C C 46.46 43.86 1973 C C 45.46 43.86 1973 C C 45.46 43.86 </td <td>C 0.013</td> <td>536.48</td> <td>1.5</td> <td>0.05</td> <td>2.32</td> <td>1.09</td> <td>4.095</td> <td>2.31</td> <td>1.276</td>	C 0.013	536.48	1.5	0.05	2.32	1.09	4.095	2.31	1.276
7.16 6.33 1973 C 6.33 5.49 1973 C 5.04 4.64 1973 C 0.94 0.52 1973 C 1.33 0.94 1973 C 1.54 1.43 1973 C 2.36 1.97 1973 C 0.52 0.2 1973 C 42.05 41.81 1973 C 42.05 41.81 1973 C 43.51 42.05 1973 C 41.61 41.23 1973 C 40.7 39.79 1973 C 40.7 39.79 1973 C 40.7 39.79 1973 C 40.7 39.8 1973 C 46 45.46 1973 C 45.46 43.86 1973 C 45.46 43.86 1973 C 45.46 43.86 1973 C 45.46 43.86 1973 C		299.72	1.5	0.11	1.77	0.97	2.672	3.49	1.276
6.33 5.49 1973 C 5.04 4.64 1973 C 0.94 0.52 1973 C 1.33 0.94 1973 C 1.54 1.43 1973 C 2.36 1.97 1973 C 0.52 0.2 1973 C 42.01 42.05 1973 C 42.07 43.05 1973 C 43.01 43.25 1973 C 41.61 41.23 1973 C 40.7 39.79 1973 C 40.7 39.78 1973 C 40.7 39.78 1973 C 40.7 39.8 1973 C 40.7 43.86 1973 C 45.46 43.86 1973 C <td>C 0.013</td> <td>380.5</td> <td>1</td> <td>0.22</td> <td>2.31</td> <td>6.0</td> <td>1.262</td> <td>1.66</td> <td>0.682</td>	C 0.013	380.5	1	0.22	2.31	6.0	1.262	1.66	0.682
5.04 4.64 1973 C 0.94 0.52 1973 C 1.33 0.94 1973 C 1.54 1.43 1973 C 2.36 1.97 1973 C 0.52 0.2 1973 C 42.05 0.2 1973 C 42.05 41.81 1973 C 43.04 43.25 1973 C 41.03 40.7 1973 C 41.01 41.23 1973 C 40.7 39.79 1973 C 40.7 39.79 1973 C 40.7 39.8 1973 C 46 45.46 1973 C 45.46 43.86 1973 C 45.47 44.6 1973 C	C 0.013	382.43	1	0.22	2.28	0.86	1.257	1.67	0.682
0.94 0.52 1973 C 1.33 0.94 1973 C 1.54 1.43 1973 C 2.36 1.97 1973 C 0.52 0.2 1973 C 42.11 42.05 1973 C 42.05 41.81 1973 C 43.61 43.25 1973 C 43.61 43.25 1973 C 41.03 40.7 1973 C 40.7 39.79 1973 C 40.7 39.79 1973 C 46 45.46 1973 C 45.46 43.86 1973 C 45.47 43.8 1973 C 45.46 1973 C 45.47 43.8 1973 C 45.	C 0.013	268.42	1.25	0.15	2.11	1.07	1.463	2.49	0.556
1.34 0.94 1973 C 1.54 1.43 1973 C 2.36 1.97 1973 C 0.52 0.2 1973 C 42.01 42.05 1973 C 42.05 41.81 1973 C 43.61 43.25 1973 C 43.51 43.25 1973 C 41.61 41.23 1973 C 40.7 39.79 1973 C 40.7 39.79 1973 C 46 45.46 1973 C 45.46 45.46 1973 C 45.46 43.86 1973 C 44.2 43.8 1973 C 45.46 43.8 1973 C	C 0.013	350	1.5	0.12	2.63	1.5	4.448	3.64	0.924
1.54 1.43 1973 C 2.36 1.97 1973 C 0.52 0.2 1973 C 42.05 41.81 1973 C 42.05 41.81 1973 C 43.61 43.25 1973 C 43.25 42.15 1973 C 41.23 40.7 1973 C 40.7 39.79 1973 C 40.7 39.79 1973 C 46 45.46 1973 C 45.46 45.46 1973 C 45.46 43.86 1973 C 44.2 43.8 1973 C 44.2 43.2 1973 C	C 0.013	261.64	1.5	0.15	2.59	1.6	4.448	4.06	0.936
2.36 1.97 1973 C 0.52 0.2 1973 C 42.01 42.05 1973 C 42.05 41.81 1973 C 43.61 43.25 1973 C 43.25 42.15 1973 C 41.23 40.7 1973 C 41.61 41.23 1973 C 40.7 39.79 1973 C 40.7 39.8 1973 C 46 45.46 1973 C 45.46 43.86 1973 C 45.42 43.8 1973 C 45.42 43.2 1973 C 45.42 43.2 1973 C 45.2 43.2 1973 C 45.2 43.2 1973 C	C 0.013	248.65	1.5	0.04	2.46	1.28	4.442	2.21	1.079
0.52 0.2 1973 C 42.11 42.05 1973 C 42.05 41.81 1973 C 43.61 43.25 1973 C 43.25 42.15 1973 C 41.23 40.7 1973 C 40.7 39.79 1973 C 40.7 39.79 1973 C 40.7 39.8 1973 C 46 45.46 1973 C 45.46 43.86 1973 C 45.46 43.86 1973 C 45.46 43.86 1973 C 45.12 44.6 1973 C 45.12 44.6 1973 C 45.12 43.2 1973 C 45.12 43.2 1973 C 45.12 43.2 1973 C	C 0.013	349.89	1.5	0.11	2.68	1.44	4.438	3.51	0.924
42.11 42.05 1973 C 42.05 41.81 1973 C 43.61 43.25 1973 C 43.25 42.15 1973 C 41.23 40.7 1973 C 40.7 39.79 1973 C 40.7 39.79 1973 C 41.05 39.8 1973 C 46 45.46 1973 C 45.46 43.86 1973 C 45.46 43.86 1973 C 45.46 43.86 1973 C 45.12 44.6 1973 C 45.12 43.2 1973 C 45.12 43.2 1973 C 45.12 43.2 1973 C	C 0.013	268	1.5	0.12	2.97	1.52	4.448	3.63	0.819
42.05 41.81 1973 C 43.61 43.25 1973 C 43.25 42.15 1973 C 41.23 40.7 1973 C 40.7 39.79 1973 C 40.7 39.79 1973 C 41.05 39.8 1973 C 46 45.46 1973 C 45.46 43.86 1973 C 45.46 43.86 1973 C 45.46 43.86 1973 C 45.46 43.86 1973 C 45.12 44.6 1973 C 44.2 43.2 1973 C 44.2 43.2 1973 C 43.2 43.0 1973 C	C 0.013	180.46	1	0.03	-2.55	0	-2.126	0.65	10.98
43.61 43.25 1973 C 43.25 42.15 1973 C 41.23 40.7 1973 C 41.61 41.23 1973 C 40.7 39.79 1973 C 39.79 39.8 1973 C 41.05 39.8 1973 C 46 45.46 1973 C 45.46 43.86 1973 C 45.46 44.6 1973 C 44.2 43.2 1973 C 44.2 43.2 1973 C 44.2 43.2 1973 C 43.2 43.2 1973 C	C 0.013	85.69	0.833	0.28	0.97	0	-0.121	1.16	13.469
43.25 42.15 1973 C 41.23 40.7 1973 C 41.61 41.23 1973 C 40.7 39.79 1973 C 39.79 39.88 1973 C 41.05 39.8 1973 C 46 45.46 1973 C 43.86 40.11 1973 C 45.12 44.6 1973 C 44.2 43.2 1973 C 43.2 43.2 1973 C 43.2 43.2 1973 C		119.48	0.833	0.3	0.8	0	-0.056	1.2	11.746
41.23 40.7 1973 C 41.61 41.23 1973 C 40.7 39.79 1973 C 39.79 39.38 1973 C 41.05 39.8 1973 C 46 45.46 1973 C 45.46 43.86 1973 C 43.86 40.11 1973 C 45.12 44.6 1973 C 44.2 43.2 1973 C 43.2 43.2 1973 C 43.2 43.08 1973 C		426.68	0.833	0.26	0.83	0	-0.122	1.11	13.061
41.61 41.23 1973 C 40.7 39.79 1973 C 39.79 39.38 1973 C 41.05 39.8 1973 C 46 45.46 1973 C 45.46 43.86 1973 C 43.86 40.11 1973 C 45.12 44.6 1973 C 44.2 43.2 1973 C 43.2 43.08 1973 C	C 0.013	129.73	1	0.41	3.03	1.82	2.523	2.28	10.569
40.7 39.79 1973 C 39.79 39.38 1973 C 41.05 39.8 1973 C 46 45.46 1973 C 45.46 43.86 1973 C 43.86 40.11 1973 C 45.12 44.6 1973 C 44.2 43.2 1973 C 43.2 43.08 1973 C	C 0.013	283.85	1	0.13	3.01	1.35	2.521	1.3	11.42
39.79 39.38 1973 C 41.05 39.8 1973 C 46 45.46 1973 C 45.46 43.86 1973 C 43.86 40.11 1973 C 45.12 44.6 1973 C 44.2 43.2 1973 C 43.2 43.08 1973 C		413.76	1	0.22	3.03	1.63	2.525	1.67	10.532
41.05 39.8 1973 C 46 45.46 1973 C 45.46 43.86 1973 C 43.86 40.11 1973 C 45.12 44.6 1973 C 44.2 43.2 1973 C 43.2 43.08 1973 C	C 0.013	186.15	1	0.22	3.09	1.62	2.559	1.67	9.621
46 45.46 1973 C 45.46 43.86 1973 C 43.86 40.11 1973 C 45.12 44.6 1973 C 44.2 43.2 1973 C 43.2 43.08 1973 C	C 0.013	126.8	0.667	0.99	3.12	0.76	0.414	1.2	0.405
45.46 43.86 1973 C 43.86 40.11 1973 C 45.12 44.6 1973 C 44.2 43.2 1973 C 43.2 43.08 1973 C	C 0.013	131.83	0.833	0.41	1.25	0	-0.063	1.4	4.744
43.86 40.11 1973 C 45.12 44.6 1973 C 44.2 43.2 1973 Cl 43.2 43.08 1973 C	C 0.013	402.44	0.833	0.4	1.35	0	-0.119	1.38	6.662
45.12 44.6 1973 C 44.2 43.2 1973 Cl 43.2 43.08 1973 C	C 0.013	341.39	1	1.1	1.88	0	-0.125	3.73	9.301
44.2 43.2 1973 CI 43.2 43.08 1973 C	C 0.013	113.72	0.667	0.46	2.52	0.46	0.593	0.82	9.942
43.2 43.08 1973 C	CI 0.015	91.5	1	1.09	2.89	0.18	0.596	3.23	8
		79.98	1	0.15	1.71	60.0	0.595	1.38	8.12
43.08 42.57 1973 C		229.8	1	0.22	-2.58	0	-2.133	1.68	9.377
GM0809 42.57 42.21 1973 C 0.0	C 0.013	160.35	1	0.22	-2.56	0	-2.131	1.69	10.251

Link Name	(ft)	(#)	Year	Material	Roughness	Length (ft)	(ft)	Slope	(ft/s)	(ft/s)	(cfs)	(cfs)	Max d/D
	26.77	19.58	1973	O	0.013	438.75	0.667	1.64	3.75	0.92	0.415	1.55	0.354
	19.43	16.65	1973	O	0.013	277.63	0.667	1	3.13	0.79	0.412	1.21	0.402
	16.3	10.54	1973	U	0.013	512.2	0.667	1.12	3.02	0.23	0.411	1.28	0.882
	14.01	13.11	1973	J	0.013	232.8	0.833	0.39	1.05	0.75	0.036	1.36	0.129
	13.11	12.85	1973	U	0.013	63.89	0.833	0.41	1.09	0.78	0.044	1.4	0.129
	12.85	10.28	1973	J	0.013	240.49	0.833	1.07	1.62	1.14	0.045	2.26	660'0
	10.49	9.45	1973	C	0.013	359.73	1	0.29	2.52	0.59	1.2	1.92	9.0
	9.45	8.95	1973	U	0.013	230.1	1	0.22	2.35	0.73	1.247	1.66	0.65
	8.85	8.1	1973	U	0.013	335.24	1	0.22	2.34	0.77	1.254	1.69	0.658
	8	7.16	1973	U	0.013	381.13	1	0.22	2.34	0.88	1.261	1.67	0.655
	39.31	26.93	1973	J	0.013	475.22	0.667	2.61	4.44	1	0.416	1.95	0.314
	16.11	15.64	2007	PVC	0.011	69.16	1.75	0.68	0	0	0	15.44	0
	47.73	47.17	1981	PVC	0.011	236.09	0.667	0.24	1.99	0.41	0.395	0.7	0.566
	4.99	4.4	2009	DVC	0.011	111.16	1	0.53	0.46	0.04	0.003	3.07	0.546
	15.29	13.06	2007	DVC	0.011	483.76	1.75	0.46	0	0	0	12.71	0
	12.91	10.88	2007	DVC	0.011	526.75	1.75	0.39	0	0	0	11.62	0
	8.24	7.86	2007	PVC	0.011	53.79	1.75	0.71	0.58	0.32	0.005	15.74	0.011
	10.88	10.68	2007	DVC	0.011	265.82	1.75	0.08	0	0	0	5.14	0
	10.58	8.44	2007	DVC	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
	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
	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
	8	15.5			0.014	1726	0.33	-0.43	1.57	0	0.07	0.11	35.587
	185	181.77			0.014	24.53	0.208	13.17	11.03	3.68	0.501	0.18	59.345
	181.77	179.87			0.014	142.41	0.667	1.33	3.45	1.46	0.501	1.3	0.451
	179.87	178.05			0.014	159.08	0.667	1.14	3.23	1.38	0.501	1.2	0.509
	178.05	176.97			0.014	94.06	0.667	1.15	2.99	1.25	0.501	1.2	0.509
	176.97	154.4			0.014	264.94	0.667	8.52	6.78	2.78	0.501	3.28	0.265
	153.99	124.48			0.014	251.95	0.667	11.71	7.58	3.14	0.501	3.84	0.245
	124.48	89.83			0.014	271.69	0.667	12.75	7.71	3.27	0.501	4.01	0.243
	89.83	26			0.014	233.67	0.667	14.48	8.18	3.34	0.501	4.27	0.232
	54.96	54.33			0.014	21.54	0.667	2.92	4.6	1.96	0.501	1.92	0.431
	54.33	45.97			0.014	323.11	0.667	2.59	4.43	0.58	0.501	1.81	8.26
	6.28	6.21			0.014	268.81	0.667	0.03	0.99	0.01	0.276	0.18	0.78
	6.21	5.63			0.014	146.6	0.667	0.4	1.82	0	9220	0.71	0.457
)	:	;

	US Invert	JS Invert DS Invert	Installation	2	22000	(#) 4±000	Diameter	00013	Max Velocity	Min Velocity	Max Flow	Design Full Flow	0/10/10/10
LIIIK Naille	(ft)	(ft)	Year	Material	Rougilless	Lengin (II)	(ft)	adoic	(ft/s)	(ft/s)	(cfs)	(cfs)	INIAX U/D
nk320	4.49	3.84			0.014	202	0.667	0.32	1.67	0.04	0.276	0.64	0.491
ink321	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	9.9	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	П	Site_1_DWF	0	0	160.27	0	1647.7	5.377
MH0126	55.2	46.77	0.159	П	Site_1_DWF	1	0	26170.13	0	1617.958	4.709
MH0127	56.5	47.25	0.012	П	Site_1_DWF	0	0	1980.5	0	1588.214	4.229
MH0128	55.6	48.42	0.049	П	Site_1_DWF	0	0	8059.39	0	1514.237	3.059
MH0130	18.18	12.35	0	П	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	П	Site_3_DWF	0	0	2030.05	0	0	0.17
MH0136	17.27	13.91	0.093	1	Site 3 DWF	0	0	89.608	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	П	æ	1	0	1212.39	0	0	0.115
MH0184	47.01	37.22	0.1	1	m	1	0	9727.34	0	0	0.11
MH0198	62:09	54.48	0.015	1	Site_1_DWF	0	0	2518.56	0	0	0.086
MH0202	8.09	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	600'0	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	600.0	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

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)
16.5	12.19	0.088	1	Site_3_DWF	1	0	0	0	0	0.275
53.03	41.23	0.005	1	Site_1_DWF	0	0	824.26	0	2169.467	10.569
56.93	45.35	0.002	1	Site_3_DWF	0	0	0	0	0	0.307
58.93	43.76	0.024	1	Site_3_DWF	0	0	2030.05	0	0	0.227
58.93	44.24	0.01	1	Site_3_DWF	0	0	817.66	0	0	0.298
59.81	46.91	0.002	1	Site_3_DWF	0	0	0	0	0	0.309
59.77	47.07	0.004	1	Site_3_DWF	0	0	394.73	0	0	0.339
61.37	48.77	0.025	1	Site_3_DWF	0	0	2424.79	0	0	0.278
61.37	50.47	0.025	1	Site_3_DWF	1	0	2424.79	0	0	0.26
61.77	52.17	0.026	1	Site_3_DWF	1	0	2424.79	0	0	0.311
51.9	44.2	0.003	П	Site_1_DWF	0	0	492.26	0	1752.95	6.655
50.4	44.48	0.01	Т	Site_1_DWF	0	0	1648.51	1264.7	1736.817	5.92
53	44.85	0.003	П	Site_1_DWF	0	0	492.26	0	1716.571	5.577
53	45.16	0.005	1	Site 1 DWF	0	0	824.26	0	1700.398	5.288
51.3	45.4	0.002	1	Site 1 DWF	0	0	331.99	0	1687.75	5.062
57.3	45.68	0.003	1	Site 1 DWF	0	0	492.26	0	1672.75	4.8
57.8	45.8	0.04	1	Site_1_DWF	0	0	6582.6	0	1661.311	4.686
59.2	46.78	0.022	1	Site 1 DWF	0	0	3617.57	0	1607.429	3.751
28	47.22	0.007	1	Site 1 DWF	0	0	1156.25	0	1579.949	3.331
58.2	47.39	0.013	П	Site_1_DWF	0	0	2140.78	0	1568.733	3.168
6.09	47.73	0.003	1	Site_1_DWF	0	0	492.26	0	1547.517	2.837
61	47.8	0	1	Site_1_DWF	0	0	0	0	1472.481	2.768
56.81	41.05	0.046	1	Site_3_DWF	1	0	4454.84	0	0	0.27
53.72	45.3	0.022	1	Site_5_DWF	0	0	817.34	0	0	0.586
44.96	32.07	0.002	1	Site_5_DWF	τ	0	266.11	0	5355.402	3.765
44.63	32.47	0.018	1	Site_5_DWF	τ	0	1634.69	0	5760	4.111
46.2	33.4	0.415	1	Site_2_DWF	1	0	0	0	2577.207	2.446
40.38	34.22	0.031	1	Site_5_DWF	0	0	2737.15	0	3883.897	5.315
41.96	33.73	0.01	1	Site_5_DWF	0	0	817.34	0	5555.849	4.884
39.96	32.88	0.007	1	Site_5_DWF	0	0	551.23	0	2260	4.296
57.33	48.61	0.113	1	Site_5_DWF	0	0	4371.84	0	0	1.048
47.96	37.47	0.042	1	Site_1_DWF	0	0	6914.59	0	2406.075	7.713
46.96	36.26	0.003	1	Site_1_DWF	1	0	492.26	0	2436.069	6.342
42.17	35.97	0.04	1	Site_1_DWF	1	0	6582.6	763.1	2460.046	6.2
44.96	35.25	0.099	1	Site_5_DWF	0	0	8477.57	0	2512.156	5.799
44.96	34.8	0.025	П	Site_5_DWF	0	0	2185.92	0	2565.775	5.598
49.16	39.2	0.025	1	Site_1_DWF	0	0	4109.83	0	2335.11	9.349
48.5	38.56	0.059	1	Site_1_DWF	0	0	97076	0	2364.572	8.808
48.5	38.1	0.028	1	Site_1_DWF	0	0	4613.54	0	2376.806	8.465
57.1	47.46	690.0	1	Site_5_DWF	1	0	2737.15	0	0	0.571
57.17	47.87	0.017	1	Site_5_DWF	0	0	551.23	0	0	0.974
56.83	48.24	0.025	1	Site_5_DWF	0	0	817.34	0	0	1.047
57.97	50.55	0.051	1	Site_5_DWF	1	0	1919.81	0	0	0.616
61.43	52.19	0.046	1	Site_5_DWF	0	0	1634.69	0	0	0.61
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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
МН0639	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	9000	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
ММН0002	10	0	0.001	1	Site_4_DWF	0	0	0	0	0	0.809
ММН0003	63	54.83	0.022	1	Site_1_DWF	0	0	3663.36	0	0	0.071
MMH0004	9:95	48.94	0.004	1	Site_1_DWF	0	0	86:899	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	2760	690.9
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)

4 3.8 2006 4 3.4 33.5 2006 4 34.4 33.5 2007 19.52 18.09 2007 19.52 18.09 2007 17.89 16.16 2001 48.94 48.62 2001 48.94 48.62 2001 48.94 48.62 2001 48.94 48.62 2001 48.94 48.62 2001 48.94 48.62 2001 48.94 48.62 2001 48.94 48.62 2001 48.94 48.62 2001 48.94 48.62 2001 46.17 46.97 2001 46.17 46.97 2001 46.17 46.97 2001 13.91 13.86 2000 13.74 46.5 2000 12.86 1995 21.23 52.93 51.34 1995	PVC	0.011 0.011 0.014 0.011	201.21	1.25	0.4	1.1	0	0.035	1 01	0.371
4 34.4 33.5 2006 4 34.4 33.5 2007 15.22 18.09 2007 2007 17.89 16.16 2007 2001 48.94 48.62 2001 2001 48.94 48.62 2001 2001 48.94 48.62 2001 2001 48.94 48.62 2001 2001 48.94 48.62 2001 2001 46.7 46.97 2001 2001 46.1 45.97 2001 2000 46.7 46.5 2000 2000 13.91 13.86 2000 2000 13.92 12.3 2000 2000 12.6 12.46 2000 2000 12.5 12.76 2000 22.53 1995 52.53 51.8 1995 22.3 1995 52.53 52.63 1995 22.3 1995 54.05	PVC	0.011	292.23		;	1			4.81	1
4 34.4 33.5 22.11 19.77 2007 19.52 18.09 2007 48.94 48.62 2001 48.42 47.45 2001 48.42 47.45 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.9 2000 13.91 13.96 2000 12.66 12.46 2000 12.66 12.46 2000 12.56 12.3 1995 52.53 51.8 1995 52.53 51.8 1995 52.53 53.57 1995 54.05 53.57 1995 54.8	PVC	0.014		0.833	0.44	1.17	0	0.028	1.73	0.09
22.11 19.77 2007 19.52 18.09 2007 48.94 48.62 2001 48.94 48.62 2001 48.42 47.45 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2000 13.91 14.13 14.01 2000 13.6 12.36 2000 2000 12.6 12.46 2000 2000 12.5 13.96 20.00 20.00 12.5 12.3 2000 20.00 12.5 12.3 2000 20.00 12.5 13.95 20.01 20.00 12.5 33.93 1996 20.01 52.	PVC	0.011	180.5	1	0.499	3.14	0	1.574	2.34	0.619
19.52 18.09 2007 48.94 48.62 2001 48.42 47.45 2001 48.42 47.45 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 46.7 46.97 2001 14.13 14.01 2000 13.91 13.86 2000 13.7 46.7 46.5 2000 13.7 12.3 2000 2000 13.7 12.3 2000 2000 13.7 12.3 2000 2000 37.2 33.93 1996 25.53 37.2 35.6 1995 25.93 1995 52.3 51.3 1995 25.93 1995 53.4 53.5 1995 24.05 1995 54.05 53.5 1995 24.05 1995 54.83 54.05 1995 24.05 1995 54.84 54.05 <td>PVC PVC PVC PVC PVC PVC PVC PVC PVC PVC</td> <td>0.011</td> <td>331.89</td> <td>1.75</td> <td>0.71</td> <td>26.85</td> <td>0</td> <td>4.381</td> <td>172.96</td> <td>0.206</td>	PVC	0.011	331.89	1.75	0.71	26.85	0	4.381	172.96	0.206
48.94 48.62 2007 48.94 48.62 2001 48.42 47.45 2001 46.7 46.97 2001 46.7 46.5 2001 46.7 46.5 2001 46.7 46.5 2001 14.13 14.01 2000 13.91 13.86 2000 13.76 12.76 2000 12.66 12.46 2000 12.5 12.6 2000 12.5 12.95 200 12.5 12.6 2000 12.5 12.95 1995 27.7 51.5 1995 52.53 51.8 1995 52.53 53.57 1995 54.05 53.57 1995 54.48 54.05 1995 54.48 54.05 1995 54.48 54.05 1995 54.48 54.05 1995 54.48 54.05 1995 54.83 54.51 1995 54.84 </td <td>PVC PVC PVC PVC PVC PVC PVC PVC PVC PVC</td> <td>7,70</td> <td>375.15</td> <td>1.75</td> <td>0.38</td> <td>4</td> <td>0</td> <td>2.955</td> <td>11.56</td> <td>0.349</td>	PVC	7,70	375.15	1.75	0.38	4	0	2.955	11.56	0.349
48.94 48.62 2001 48.42 47.45 2001 46.72 46.97 2001 46.71 46.97 2001 46.77 46.5 2001 46.77 46.5 2001 14.13 14.01 2000 13.91 13.86 2000 13.76 12.76 2000 12.66 12.46 2000 12.56 12.76 2000 12.56 12.76 2000 12.57 12.33 2000 37.22 33.93 1996 52.53 51.8 1995 52.53 51.8 1995 52.93 52.63 1995 53.21 53.03 1995 54.48 54.05 1995 54.48 54.05 1995 54.48 54.05 1995 54.48 54.05 1995 54.48 54.05 1995 54.48 54.05 1995 60.55 54.51 1995	PVC	0.011	370.91	1.75	0.47	4.21	0	2.694	12.79	0.314
48.42 47.45 2001 47.25 46.97 2001 46.1 45.97 2001 46.77 46.5 2001 14.13 14.01 2000 13.91 13.86 2000 13.76 12.76 2000 12.66 12.46 2000 12.35 12.33 1996 37.22 33.93 1996 37.23 35.62 1996 52.53 51.8 1995 52.93 52.63 1995 52.93 52.63 1995 52.93 52.63 1995 52.93 54.05 1995 54.48 54.05 1995 54.48 54.05 1995 54.48 54.05 1995 54.48 54.05 1995 54.48 54.05 1995 54.48 54.05 1995 60.55 53.57 1998 60.55 53.57 1988 50.55 58.53 1988	PVC	0.011	92	0.833	0.42	0.55	0	0.003	1.68	0.036
47.25 46.97 2001 46.1 45.97 2001 46.77 46.5 2001 46.77 46.5 2001 14.13 14.01 2000 13.91 13.86 2000 13.76 12.76 2000 12.66 12.46 2000 12.85 12.3 2000 12.85 12.3 2000 12.85 12.3 2000 12.85 12.3 1996 37.22 35.62 1995 52.53 51.8 1995 52.93 52.63 1995 54.05 53.31 1995 54.05 53.31 1995 54.48 54.05 1995 54.83 54.51 1995 54.84 54.51 1995 51.84 51.34 1995 51.84 51.34 1995 60.55 59.55 1988 60.55 59.55 1988 58.36 59.55 1988 <t< td=""><td>PVC PVC PVC PVC PVC PVC PVC PVC PVC PVC</td><td>0.011</td><td>242.29</td><td>0.833</td><td>0.4</td><td>1.27</td><td>0</td><td>0.045</td><td>1.64</td><td>0.118</td></t<>	PVC	0.011	242.29	0.833	0.4	1.27	0	0.045	1.64	0.118
46.1 45.97 2001 46.77 46.5 2001 14.13 14.01 2000 13.91 13.86 2000 13.76 12.76 2000 12.66 12.46 2000 12.35 12.3 2000 12.35 12.3 1996 37.22 33.93 1996 37.22 35.62 1995 52.53 51.8 1995 52.93 52.63 1995 53.47 53.31 1995 54.05 53.31 1995 54.48 54.05 1995 54.83 54.51 1995 54.84 54.05 1995 54.83 54.51 1995 54.84 54.51 1995 54.84 54.51 1995 54.84 54.51 1995 60.55 59.55 1988 60.55 59.55 1988 58.36 57.4 1988 56.35 55.25 1988	PVC	0.011	64.95	0.833	0.43	1.41	0	950'0	1.7	0.125
46.77 46.5 2001 14.13 14.01 2000 13.91 13.86 2000 13.76 12.76 2000 12.66 12.46 2000 12.66 12.46 2000 12.65 12.46 2000 12.66 12.46 2000 12.35 12.3 2000 35.42 33.93 1996 51.7 51.52 1995 52.53 52.63 1995 52.93 52.63 1995 54.05 53.31 1995 54.05 53.31 1995 54.05 53.37 1995 54.83 54.51 1995 54.84 54.51 1995 54.85 54.51 1995 54.84 54.51 1995 51.84 51.34 1995 60.55 59.55 1988 59.55 58.53 1988 60.55 59.55 1988 56.35 1988 56.35 55.25 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 55.35 1988	PVC	0.011	33.12	0.833	0.39	2	0	0.193	1.62	0.234
14.13 14.01 2000 13.91 13.86 2000 13.76 12.76 2000 12.66 12.46 2000 12.66 12.46 2000 12.35 12.3 2000 12.35 12.3 2000 12.35 12.3 2000 35.42 33.93 1996 51.7 51.52 1995 52.53 52.63 1995 52.93 52.63 1995 53.47 53.31 1995 54.05 53.57 1995 54.83 54.51 1995 54.84 54.51 1995 54.85 54.51 1995 54.84 54.51 1995 54.85 36.36 1995 51.14 50 1995 60.55 59.55 1988 56.55 58.53 1988 56.35 55.25 1988 56.35 55.25 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 1988 56.35 55.35 1988 56.35 55.35 1988	PVC	0.011	118.56	0.833	0.23	1.6	0	0.192	1.23	0.277
13.91 13.86 2000 13.76 12.76 2000 12.66 12.46 2000 12.65 12.46 2000 12.35 12.3 2000 35.42 33.93 1996 37.22 35.62 1996 51.7 51.52 1995 52.53 52.63 1995 52.93 52.63 1995 53.47 53.31 1995 54.05 53.57 1995 54.83 54.51 1995 54.84 54.51 1995 54.85 54.51 1995 51.14 50 1995 60.55 59.55 1988 59.55 58.53 1988 56.35 55.25 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 55.35 1988 57.4 56.35 1988 56.35 55.35 1988 57.4 1986	PVC PVC PVC PVC PVC PVC PVC PVC PVC	0.011	23.02	0.833	0.52	1.4	0	0.049	1.87	0.14
13.76 12.76 2000 12.66 12.46 2000 12.35 12.3 2000 35.42 33.93 1996 37.22 35.62 1996 37.22 35.62 1996 51.7 51.52 1995 52.53 51.8 1995 52.93 52.63 1995 52.93 52.63 1995 53.21 53.31 1995 54.05 53.57 1995 54.83 54.51 1995 51.84 51.34 1995 51.84 51.34 1995 51.84 51.34 1995 60.55 59.55 1988 60.55 59.55 1988 58.36 57.4 1988 57.4 56.35 1988 56.35 55.25 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 55.35 1988	PVC PVC PVC PVC PVC PVC	0.011	104	0.833	0.05	0.95	0	0.102	0.57	0.26
12.66 12.46 2000 12.35 12.3 2000 35.42 33.93 1996 37.22 35.62 1996 37.22 35.62 1996 51.7 51.52 1995 52.53 51.8 1995 52.93 52.63 1995 52.93 52.63 1995 53.47 53.31 1995 54.48 54.05 1995 54.83 54.51 1995 54.84 54.51 1995 51.84 51.34 1995 51.84 51.34 1995 60.55 53.57 1998 60.55 59.55 1988 58.36 59.55 1988 57.4 56.35 1988 56.35 55.25 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 55.35 1988	PVC PVC PVC PVC PVC PVC	0.011	400	0.833	0.25	1.38	0	0.114	1.29	0.212
12.35 12.3 2000 35.42 33.93 1996 37.22 35.62 1996 51.7 51.52 1995 52.53 51.8 1995 52.93 52.63 1995 52.93 52.63 1995 53.21 53.03 1995 53.47 53.31 1995 54.48 54.05 1995 54.83 54.51 1995 51.84 51.34 1995 51.84 51.34 1995 60.55 59.55 1988 60.55 59.55 1988 57.4 56.35 1988 57.4 56.35 1988 55.25 55.25 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 55.25 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 55.35 1988 56.35 55.35 1988 57.4 1986 57.4 1986 57.5 55.35 1988 57.7 <t< td=""><td>DI PVC PVC</td><td>0.011</td><td>79.17</td><td>0.833</td><td>0.25</td><td>1.46</td><td>0</td><td>0.123</td><td>1.3</td><td>0.215</td></t<>	DI PVC PVC	0.011	79.17	0.833	0.25	1.46	0	0.123	1.3	0.215
35.42 33.93 1996 37.22 35.62 1996 51.7 51.52 1995 52.93 52.63 1995 52.93 52.63 1995 53.21 53.03 1995 53.47 53.31 1995 54.05 53.57 1995 54.83 54.51 1995 51.84 50.51 1995 51.14 50 1995 60.55 59.55 1988 59.55 58.53 1988 57.4 56.35 1988 55.25 55.25 1988 56.35 55.25 1988 12.19 11.54 1986	PVC PVC	0.012	20	0.833	0.25	1.49	0	0.123	1.19	0.213
37.22 35.62 1996 51.7 51.52 1995 52.53 51.8 1995 52.93 52.63 1995 53.21 53.03 1995 53.47 53.31 1995 54.05 53.57 1995 54.48 54.05 1995 54.83 54.51 1995 51.44 50 1995 51.84 51.34 1995 49.4 48.75 1995 49.8 49.6 1995 60.55 59.55 1988 58.36 57.4 1988 55.25 55.25 1988 56.35 55.35 1988 12.19 11.54 1986	PVC	0.011	362.58	0.833	0.41	1.44	0	0.065	1.66	0.138
51.7 51.52 1995 52.53 51.8 1995 52.93 52.63 1995 53.21 53.03 1995 53.47 53.31 1995 53.48 53.31 1995 54.05 53.31 1995 54.05 53.57 1995 54.83 54.05 1995 54.83 54.51 1995 51.44 50 1995 49.4 48.75 1995 49.8 49.6 1995 60.55 59.55 1988 59.55 58.53 1988 57.4 56.35 1988 55.25 55.2 1988 56.35 55.25 1988 56.35 55.25 1988 56.35 55.25 1988 56.35 55.25 1988	PVC	0.011	405.28	0.833	0.39	1.37	0	850'0	1.63	0.132
52.53 51.8 1995 52.93 52.63 1995 53.21 53.03 1995 53.47 53.31 1995 54.05 53.57 1995 54.05 53.57 1995 54.83 54.05 1995 54.83 54.51 1995 36.76 36.36 1995 49.4 48.75 1995 49.8 49.6 1995 49.8 49.6 1995 60.55 59.55 1988 57.4 56.35 1988 57.4 56.35 1988 55.25 55.25 1988 56.35 55.25 1988 56.35 55.25 1988 56.35 55.25 1988 56.35 55.25 1988 56.35 1988 11.54 12.19 11.54 1986		0.011	71.1	1	0.25	0.88	0	0.238	2.12	0.525
52.93 52.63 1995 53.21 53.03 1995 53.47 53.31 1995 54.05 53.57 1995 54.48 54.05 1995 54.48 54.05 1995 54.48 54.05 1995 54.83 54.51 1995 36.76 36.36 1995 49.4 48.75 1995 49.8 49.6 1995 60.55 59.55 1988 57.4 56.55 1988 55.25 55.25 1988 56.35 55.25 1988 56.35 55.25 1988 56.35 55.25 1988 12.19 11.54 1986	PVC	0.011	290.21	1	0.25	1.74	0	0.238	2.11	0.258
53.21 53.03 1995 53.47 53.31 1995 54.05 53.57 1995 54.88 54.05 1995 54.83 54.51 1995 36.76 36.36 1996 51.84 51.34 1995 51.14 50 1995 49.8 49.6 1995 60.55 59.55 1988 58.36 57.4 1988 55.25 55.25 1988 56.35 55.25 1988 56.35 55.25 1988 12.19 11.54 1986	PVC	0.011	118.2	1	0.25	1.72	0	0.231	2.12	0.23
53.47 53.31 1995 54.05 53.57 1995 54.88 54.05 1995 54.83 54.51 1995 36.76 36.36 1996 36.76 36.36 1996 51.84 51.34 1995 49.4 48.75 1995 49.8 49.6 1995 60.55 59.55 1988 59.55 58.33 1988 57.4 56.35 1988 56.35 55.25 1988 56.35 55.25 1988 12.19 11.54 1986	PVC	0.011	71.22	1	0.25	1.7	0	0.213	2.12	0.219
54.05 53.57 1995 54.48 54.05 1995 54.83 54.51 1995 36.76 36.36 1996 51.84 51.34 1995 51.14 50 1995 49.4 48.75 1995 49.8 49.6 1995 60.55 59.55 1988 59.55 58.53 1988 57.4 56.35 1988 55.25 55.25 1988 12.19 11.54 1986	PVC	0.011	61.98	1	0.26	1.72	0	0.213	2.14	0.217
54.48 54.05 1995 54.83 54.51 1995 36.76 36.36 1996 51.84 51.34 1995 51.14 50 1995 49.4 48.75 1995 49.8 49.6 1995 60.55 59.55 1988 59.55 58.53 1988 57.4 56.35 1988 55.25 55.2 1988 56.35 1988 10.154 12.19 11.54 1986	PVC	0.011	175.44	1	0.27	1.7	0	0.209	2.2	0.216
54.83 54.51 1995 36.76 36.36 1996 51.84 51.34 1995 51.14 50 1995 49.4 48.75 1995 49.8 49.6 1995 60.55 59.55 1988 59.55 58.53 1988 57.4 56.35 1988 55.25 55.2 1988 56.35 55.2 1988 12.19 11.54 1986	PVC	0.011	170.77	1	0.25	1.68	0	0.197	2.11	0.216
36.76 36.36 1996 51.84 51.34 1995 51.14 50 1995 49.4 48.75 1995 49.8 49.6 1995 60.55 59.55 1988 59.55 58.53 1988 57.4 56.35 1988 55.25 55.2 1988 56.35 55.35 1988 12.19 11.54 1986	PVC	0.011	126.75	1	0.25	1.6	0	0.184	2.12	0.206
51.84 51.34 1995 51.14 50 1995 49.4 48.75 1995 49.8 49.6 1995 60.55 59.55 1988 59.55 58.53 1988 57.4 56.35 1988 55.25 55.2 1988 56.35 55.3 1988 12.19 11.54 1986	J	0.013	206.3	1	0.19	2.38	0	1.412	1.57	0.723
51.14 50 1995 49.4 48.75 1995 49.8 49.6 1995 60.55 59.55 1988 59.55 58.53 1988 58.36 57.4 1988 57.4 56.35 1988 55.25 55.2 1988 12.19 11.54 1986	PVC	0.011	120.32	1	0.42	2.15	0	0.249	2.71	0.205
49.4 48.75 1995 49.8 49.6 1995 60.55 59.55 1988 59.55 58.53 1988 58.36 57.4 1988 57.4 56.35 1988 55.25 55.2 1988 56.35 55.35 1988 12.19 11.54 1986	PVC	0.011	228.43	1	0.5	2.34	0	0.264	2.97	0.201
49.8 49.6 1995 60.55 59.55 1988 59.55 58.53 1988 58.36 57.4 1988 57.4 56.35 1988 55.25 55.2 1988 56.35 55.2 1988 12.19 11.54 1986	PVC	0.011	155.28	1	0.42	2.26	0	0.294	2.72	0.222
60.55 59.55 1988 59.55 58.53 1988 58.36 57.4 1988 57.4 56.35 1988 55.25 55.2 1988 56.35 55.35 1988 12.19 11.54 1986	PVC	0.011	49.03	1	0.41	2.23	0	0.287	2.69	0.221
59.55 58.53 1988 58.36 57.4 1988 57.4 56.35 1988 55.25 55.2 1988 56.35 55.35 1988 12.19 11.54 1986	PVC	0.011	250	0.667	0.4	2.77	0	0.614	6.0	0.635
58.36 57.4 1988 57.4 56.35 1988 55.25 55.2 1988 56.35 55.35 1988 12.19 11.54 1986	PVC	0.011	272.67	0.667	0.37	2.61	0	0.582	0.87	0.618
57.4 56.35 1988 55.25 55.2 1988 56.35 55.35 1988 12.19 11.54 1986	PVC	0.011	354.18	0.833	0.27	2.58	0	0.802	1.35	0.562
55.25 55.2 1988 56.35 55.35 1988 12.19 11.54 1986	PVC	0.011	358.33	0.833	0.29	2.67	0	0.826	1.4	0.562
56.35 55.35 1988 12.19 11.54 1986	PVC	0.011	63.77	0.833	0.08	2.18	0	0.812	0.72	0.665
12.19 11.54 1986	PVC	0.011	312.25	0.833	0.32	2.76	0	0.822	1.46	0.545
	PVC	0.011	329.67	1	0.2	1.63	0	0.285	1.87	0.28
1986	PVC	0.011	179.05	1	0.52	2.03	0	0.296	3.05	0.624
10.54 10.49 1986	O	0.013	21.37	1.25	0.23	2.73	0	1.667	3.12	0.499
44.24 43.86 1981	PVC	0.011	88.82	0.667	0.43	3.05	0	0.816	0.94	0.725
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 PV	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	292'0	6.0	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	Э	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	С	0.013	166.26	1	0.22	1.72	0	0.375	1.68	0.326
GM0469	45.16	44.85	1980	С	0.013	136.37	1	0.23	1.73	0	0.372	1.7	0.321
GM0470	45.4	45.16	1980	С	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	298'0	1.68	0.318
GM0472	45.8	45.68	1980	J	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	80£.0	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	D	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	Ü	0.015	100	1	1	3.92	0	1.574	3.09	0.554
GM0556	32.47	32.07	1973	С	0.013	178	1.5	0.225	2.01	0	1.491	4.98	0.589
GM0563	34.22	33.73	1973	С	0.013	225	1	0.22	2.38	0	1.465	1.66	0.734
GM0564	33.73	32.96	1973	С	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	Э	0.013	213.42	1	0.27	2.54	0	1.459	1.86	0.731
GM0647	35.25	34.9	1973	С	0.013	158.5	1	0.22	2.52	0	1.454	1.67	0.701
GM0648	35.97	35.35	1973	С	0.013	282.01	1	0.22	2.42	0	1.435	1.67	0.719
GM0649	36.26	36.07	1973	С	0.013	83.71	1	0.23	2.56	0	1.423	1.7	0.671
GM0650	37.47	36.86	1973	С	0.013	303.75	1	0.2	2.33	0	1.401	1.6	0.728
GM0652	38.1	37.47	1973	С	0.013	282.39	1	0.22	2.35	0	1.377	1.68	0.728
GM0653	38.56	38.2	1973	С	0.013	166.62	1	0.22	2.44	0	1.369	1.66	0.681
GM0654	39.2	38.66	1973	С	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	С	0.013	238.88	1.25	0.69	4.71	0	4.023	5.35	0.725
GM0678	49.505	48.613	1973	С	0.013	239.91	1.5	0.372	3.81	0	4.048	6.41	0.587
GM0682	48.513	47.515	1973	2	0.013	268.29	1.5	0.372	3.8	0	4.051	6.41	0.589
GM0686	47.414	46.386	1973	С	0.013	276.3	1.5	0.372	3.79	0	4.038	6.41	0.589
GM0687	46.286	45.3	1973	Э	0.013	241.55	1.5	0.408	3.93	0	4.046	6.71	0.571
GM0708	11.19	3.19	1973	O	0.013	506.38	1.75	1.58	5.54	0	4.196	19.92	0.795
GM0709	19.11	11.19	1973	O	0.013	486.49	1.25	1.63	6.38	0	4.156	8.24	0.534
GM0710	29.26	19.11	1973	U	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	С	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	С	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	U	0.013	452.82	1.5	1.85	6.57	0	3.72	14.28	0.431
GM0727	10.03	7.3	1973	U	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	U	0.013	66.1	2	0.15	2.94	0	5.336	8.8	0.562
GM0730	4.726	4.398	1973	U	0.013	280.43	2	0.117	2.68	0	5.369	7.74	0.61
GM0731	4.398	4.138	1973	J	0.013	222.56	2	0.117	2.7	0	5.372	7.73	909:0
GM0732	4.138	3.813	1973	J	0.013	277.68	2	0.117	2.76	0	5.381	7.74	0.598
GM0733	3.813	3.48	1973	J	0.013	285.5	2	0.117	3.02	0	5.387	7.73	0.57
GM0734	2.62	2.36	1973	U	0.013	536.48	2.5	0.048	2.8	0	10.828	9.03	0.747
GM0735	2.95	2.62	1973	U	0.013	299.72	2.5	0.11	1.85	0	6.184	13.61	0.747
GM0747	7.16	6.33	1973	U	0.013	380.5	1.25	0.22	2.54	0	1.78	3.02	0.572
GM0748	6.33	5.49	1973	U	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	С	0.013	350	2.5	0.12	3.39	0	10.963	14.21	0.631
GM0758	1.33	0.94	1973	С	0.013	261.64	2.5	0.149	3.42	0	10.975	15.84	0.631
GM0759	1.87	1.43	1973	С	0.013	248.65	2.5	0.177	3.72	0	10.981	17.25	0.582
GM0760	2.36	1.97	1973	С	0.013	349.89	2.5	0.11	3.45	0	10.992	13.69	0.628
GM0761	0.52	0.15	1973	U	0.013	268	2.5	0.138	3.78	0	10.954	15.24	0.582
GM0778	42.11	41.9	1973	O	0.013	180.46	1.5	0.116	2.79	0	3.258	3.58	0.645
GM0779	42.05	41.81	1973	С	0.013	85.69	1.25	0.28	0.68	0	0.054	3.42	0.646
GM0782	43.61	43.25	1973	J	0.013	119.48	0.833	0.3	0.82	0	0.022	1.2	0.124
GM0783	43.25	42.15	1973	С	0.013	426.68	0.833	0.26	0.92	0	0.034	1.11	0.562
GM0785	41.23	40.7	1973	J	0.013	129.73	1.5	0.41	3.54	0	3.596	6.71	0.653
GM0786	41.61	41.23	1973	U	0.013	283.85	1.5	0.13	2.65	0	3.275	3.84	0.672
GM0787	40.7	39.79	1973	O	0.013	413.76	1.5	0.22	3.03	0	3.595	4.93	0.653
GM0792	39.88	40.5	1973	U	0.013	186.15	1	-0.333	-0.85	0	-0.021	2.06	0.695
GM0793	41.05	39.8	1973	O	0.013	126.8	0.667	0.99	3.73	0	0.851	1.2	0.622
GM0798	46	45.46	1973	С	0.013	131.83	0.833	0.41	1.24	0	0.059	1.4	0.165
GM0799	45.46	43.86	1973	С	0.013	402.44	0.833	0.4	1.24	0	0.071	1.38	0.165
GM0800	43.86	40.11	1973	С	0.013	341.39	0.833	1.1	1.99	0	0.085	2.29	0.31
GM0805	45.12	44.6	1973	С	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	С	0.013	79.98	1.25	0.15	2.1	0	1.2	2.5	0.702
GM0808	43.06	42.57	1973	С	0.013	229.8	1.5	0.213	2.95	0	3.254	4.85	0.599
GM0809	42.57	42.21	1973	J	0.013	160.35	1.5	0.225	3.02	0	3.254	4.98	0.588
GM0823	26.77	19.58	1973	U	0.013	438.75	0.667	1.64	4.54	0	0.853	1.55	0.53
GM0824	19.43	16.65	1973	U	0.013	277.63	0.667	1	3.75	0	0.849	1.21	0.618
GM0825	16.3	10.54	1973	O	0.013	512.2	1	1.125	3.62	0	0.848	3.78	0.624

(ft)	(ft)	Year	Material	Roughness	Length (ft)	(ft)	Slope	(ft/s)	wiin Velocity (ft/s)	Max Flow (cfs)	Flow (cfs)	Max d/D
14.01	13.11	1973)	0.013	232.8	0.833	0.39	1.05	0	0.036	1.36	0.129
13.11	12.85	1973	ပ	0.013	63.89	0.833	0.41	1.09	0	0.044	1.4	0.129
12.85	10.28	1973	ပ	0.013	240.49	0.833	1.07	1.62	0	0.045	2.26	0.099
10.49	9.45	1973)	0.013	359.73	1.25	0.29	2.75	0	1.667	3.47	0.547
9.45	8.95	1973)	0.013	230.1	1.25	0.22	2.55	0	1.725	3.01	0.547
8.85	8.1	1973	3	0.013	335.24	1.25	0.22	2.54	0	1.732	3.06	0.552
8	7.16	1973	ပ	0.013	381.13	1.25	0.22	2.56	0	1.76	3.03	0.554
39.31	26.93	1973	ပ	0.013	475.22	0.667	2.61	5.4	0	0.853	1.95	0.463
16.11	15.64	2007	ϽΛd	0.011	69.16	1.75	89.0	4.82	0	2.702	15.44	0.285
47.73	47.17	1981	DVC	0.011	236.09	0.667	0.237	2.32	0	0.723	0.7	0.869
4.99	4.64	2009	DΛd	0.011	111.16	1	0.315	0.39	0	-0.011	2.36	0.97
15.29	13.06	2007	PVC	0.011	483.76	1.75	0.44	4.07	0	2.419	12.71	0.297
12.91	10.88	2007	DAG	0.011	526.75	1.75	0.4	3.61	0	2.367	11.62	0.428
8.24	7.86	2007	PVC	0.011	53.79	1.75	0.71	4.57	0	2.137	15.74	0.249
10.88	10.68	2007	PVC	0.011	265.82	1.75	0.075	2.34	0	2.208	5.14	0.428
10.58	8.44	2007	PVC	0.011	502.69	1.75	0.426	3.82	0	2.133	12.22	0.283
32	52.19	0		0.014	1531.04	1	-1.319	4.3	0	4.03	3.8	21.003
								0	0	2.674	0	0
								0	0	2.472	0	0
32.07	32			0.014	28.87	1.5	0.242	3.9	0	4.163	4.8	0.589
								0	0	0.607	0	0
33.93	60.55	0		0.014	256.52	0.5	-10.563	2.28	0	0.589	1.68	54.087
8.91	12.5			0.014	1565	0.33	-0.229	2.1	0	0.128	0.08	46.507
								0	0	1.192	0	0
								0	0	1.014	0	0
8	15.5			0.014	1726	0.33	-0.435	1.56	0	0.07	0.11	35.523
185	181.77			0.014	24.53	0.208	13.168	11.03	0	0.501	0.18	59.344
181.77	179.87			0.014	142.41	0.667	1.334	3.45	0	0.501	1.3	0.451
179.87	178.05			0.014	159.08	0.667	1.144	3.23	0	0.501	1.2	0.508
178.05	176.97			0.014	94.06	0.667	1.148	2.99	0	0.501	1.2	0.508
176.97	154.4			0.014	264.94	0.667	8.519	6.81	0	0.511	3.28	0.267
153.99	124.48			0.014	251.95	0.667	11.713	7.62	0	0.511	3.84	0.251
124.48	89.83			0.014	271.69	0.667	12.754	7.85	0	0.537	4.01	0.251
89.83	99			0.014	233.67	0.667	14.478	8.36	0	0.537	4.27	0.239
54.96	54.33			0.014	21.54	0.667	2.925	4.65	0	0.537	1.92	0.395
54.33	45.97			0.014	323.11	0.667	2.587	4.63	0	0.594	1.81	0.395
6.28	6.21			0.014	268.81	0.667	0.026	66.0	0	0.276	0.18	0.784
6.21	5.63			0.014	146.6	0.667	0.396	1.88	0	0.305	0.71	0.483
5.51	4.59			0.014	264	0.667	0.348	1.76	0	0.305	99.0	0.51
4.49	3.84			0.014	202	0.667	0.322	1.74	0	0.305	0.64	0.513
3.74	3.64			0.014	64.73	0.667	0.154	1.59	0	0.305	0.44	0.556
41.62	37.62			0.014	1600	1	0.25	11.68	0	0.444	23.16	0.101
								0	0	C	O	C
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S Invert	DS Invert	US Invert DS Invert Installation	lcizo+ch/	goddallog	(#) dpag	Diameter	Clone	Max Velocity	Min Velocity	May Flow (cfc)	Design Full	O/PoxeM
(L	(£)	Year	iviaterial	NOUBILIESS	רבוופתוו (ווי)	(£)	adoir	(ft/s)	(ft/s)	IVIAA FIOW (CIS)	Flow (cfs)	ואומא עו ע
								0	0	0	0	0
								0	0	1.109	0	0
								0	0	0	0	0
								0	0	0.439	0	0
99	47.51			0.014	4300	0.667	0.43	2.1	0	0.498	0.74	99.0
118	99			0.014	009	0.667	8.667	34.72	0	0.446	46.31	99.0
								0	0	0	0	0
								0	0	1.175	0	0
48.61	22.16			0.011	2600	1	1.017	26.53	0	1.449	46.72	0.256
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)

Aiternative b ruture riows (2033) 3-year, 24-nour storm event	e Flows (2032, J	-year, 24-110011 St.	אבווו באבוור)			ĺ					
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	9.9	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	89.6068	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	600.0	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	Ţ	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	62.09	54.48	0.015	1	Site_1_DWF	0	0	2518.56	0	0	0.206
MH0202	8.09	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	863.98	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	28	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	8.69	57.4	0.035	1	m'	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	П	Site_3_DWF	0	0	2847.72	0	0	0.412

	Elevation (ft)	E)	Inflow (cfs)	Flow Flag	remporar variation	Factor	Inflow (ft^3)	Inflow (ft^3)	(min)	(min)	(L
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	П	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	П	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	Т	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	660.0	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	969.0
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
0710114	20 01	0,1	1000								

Node Name Elevat	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)
57	57.97	50.55	0.051	1	Site_5_DWF	1	0	1919.81	0	0	0.906
61	61.43	52.19	0.046	1	Site_5_DWF	0	0	1634.69	0	0	0.813
5.	53.3	44.12	0.018	1	Site_5_DWF	0	0	551.23	0	0	0.732
5(50.2	40.43	60:0	1	Site_5_DWF	0	0	3554.5	0	0	0.538
46	46.37	32.47	0.081	1	Site_5_DWF	0	0	3003.26	0	0	0.796
7	40	29.26	0.053	1	Site_5_DWF	1	0	1919.81	0	0	0.591
32	32.37	19.11	0.03	1	Site_5_DWF	1	0	28236.59	0	0	0.668
20	20.49	11.19	0.009	1	Site_5_DWF	1	0	11292.59	0	0	0.545
1.	15.5	2.95	0.004	1	Site_5_DWF	1	0	0	0	0	1.632
41	41.05	30.96	0.22	1	Site_4_DWF	1	0	32932.56	0	0	0.508
22	22.83	14.01	0.056	1	Site_4_DWF	0	0	8175.74	0	0	0.093
30	30.81	20.91	0.056	1	Site_4_DWF	0	0	8175.74	0	0	0.541
3(30.5	18.4	0.015	1	Site_4_DWF	1	0	2138.69	0	0	0.522
19	19.96	10.03	0.026	1	Site_4_DWF	1	0	7803.97	0	0	0.646
16	16.66	5.24	9000	1	Site_4_DWF	0	0	974.59	0	0	1.123
15	15.78	5.04	0.003	1	Site_4_DWF	0	0	392.54	0	0	1.223
16	16.71	6.33	0.015	1	Site_4_DWF	T	0	7046.62	0	0	0.715
16	16.71	7.16	0.02	1	Site_4_DWF	1	0	8177.33	0	0	0.693
15	15.92	4.398	0.02	1	Site_4_DWF	0	0	974.59	0	0	1.212
13	13.48	4.138	0.018	1	Site_4_DWF	1	0	2771.86	0	0	1.195
1,	14.5	2.62	0.02	1	Site_4_DWF	1	0	2720.74	0	0	1.868
11	15.6	4.726	0.016	1	Site_4_DWF	1	0	3113.28	0	0	1.221
11	11.86	3.813	0.028	1	Site_4_DWF	0	0	1367.14	0	0	1.141
14	14.17	2.36	0.052	1	Site_4_DWF	1	0	0	0	0	1.569
12	12.34	0.94	0.009	1	Site_4_DWF	1	0	392.54	0	0	1.577
13	13.82	0.52	0	1	Site_4_DWF	0	0	0	0	0	1.456
12	12.82	0.1	0	1	Site_4_DWF	0	0	0	0	0	1.239
14	14.95	1.87	0.048	1	Site_4_DWF	1	0	2341.73	0	0	1.451
52	52.41	39.31	0.013	1	Site_3_DWF	0	0	1212.39	0	0	0.309
57	57.96	46	690.0	1	Site_1_DWF	0	0	11356.42	0	0	0.117
58	58.02	45.46	0.014	1	Site_1_DWF	0	0	2301.05	0	0	0.137
54	54.12	43.86	0.016	1	Site_1_DWF	0	0	2633.04	0	0	0.11
20	50.94	39.38	0.017	1	Site_1_DWF	0	0	2793.31	0	0	0.988
۱)	52	40.7	0.008	1	Site_1_DWF	0	0	1316.52	0	0	86.0
53	53.03	41.61	0.009	1	Site_1_DWF	0	0	1476.79	0	0	1.008
53	53.03	42.05	0.021	1	Site_1_DWF	0	0	3457.3	0	0	0.568
52	52.71	42.11	0.018	1	Site_1_DWF	0	0	2965.03	0	0	0.968
52	52.83	42.57	0.004	1	Site_1_DWF	0	0	863.98	0	0	0.881
51	51.78	43.06	0	1	Site_1_DWF	0	0	0	0	0	0.898
53	53.73	43.25	0.014	1	Site_1_DWF	0	0	2301.05	0	0	0.104
99	56.03	43.61	0.026	1	Site_1_DWF	0	0	4281.55	0	0	0.079
5.	51.2	43.2	0.001	1	Site_1_DWF	0	0	160.27	0	0	0.782
2.	52.7	44.2	900.0	1	Site_1_DWF	0	0	984.53	0	0	0.424
5.	52.6	45.12	0	1	Site_1_DWF	1	0	0	0	0	0.575
39	39.43	26.77	0.01	1	Site_3_DWF	1	0	817.66	0	0	0.353

ation Constant Inflow (cfs) 0.026		Dry Weather Flow Flag 1	Temporal Variation Site_4_DWF	Peaking Factor 1	Volume of Const. Inflow (ft^3) 0	Volume of DWF Inflow (ft^3) 9981.43	Time Flooded (min)	Time Surcharged (min) 0	Max Depth (ft) 0.684
	1		Site_4_DWF	1	0	2720.74	0	0	0.691
	1		Site_3_DWF	1	0	4454.84	0	0	0.684
0.024	⊣		Site_3_DWF	1	0	394.73	0	0	0.612
12.85 0.003 1	₩ .		Site_4_DWF	0	0	392.54	0	0 0	0.082
0.006			Site 3 DWF	0 0	0	394.73		0 0	0.100
0	1		Site 3 DWF	0	0	0	0	0	0.412
0.017	1		Scappoose_Diurnal	0	0	0	0	0	0.748
12.91 0.019 1	1		Scappoose_Diurnal	1	0	18700.08	0	0	0.536
0.007			Scappoose_Diurnal	1	0	0	0	0	0.435
0.011	1		Scappoose_Diurnal	0	0	1093.28	0	0	0.496
0	1		Miller WTP	1	0	27820.8	0	0	0.523
	1		Site_4_DWF	0	0	582.05	0	0	0.62
0.001	1		Site_4_DWF	0	0	0	0	0	1.108
0.022	1		Site_1_DWF	1	0	32450.64	0	0	0.206
48.94 0.004 1	1		Site_1_DWF	0	0	663.98	0	0	0.03
24 0.021 1	1		Site_3_DWF	τ	0	29604.96	0	0	8.502
24.5 0 1	1		Site_5_DWF	1	0	0	0	0	8.007
32 0 1	1		Site_5_DWF	0	0	0	5607.3	5637.1	37.193
33.93 0 1	1		Scappoose_Diurnal	0	0	0	5428.6	5499	29.048
0 1	0		0	0	0	0	4943.7	5406.071	18.915
0 0 8	0		0	0	0	0	4655.7	5469.083	19.187
	1		Site_3_DWF	1	0	8712.32	0	2760	3.004
0	1		Site_3_DWF	1	0	10911.54	0	2760	6.07
0	1		Site_1_DWF	1	0	23937.77	0	0	0.263
0		0	0	0	0	0	0	0	0.241
0		0	0	0	0	0	0	0	0.16
0		1	Site_1_DWF	1	0	13823.42	0	0	0.168
0		0	0	0	0	0	0	0	0.164
0		1	Site_1_DWF	1	0	5080.11	0	0	0.178
0		0	0	0	0	0	0	0	0.339
		0	0	0	0	0	0	0	0.301
0 2		0	0	0	0	0	0	0	0.288
	0		0	0	0	0	0	0	0.371
)	(0	0	0	0	0	0	0.342
5.51 0 0 0	0		0	0	0	0	0	0	0.34
6.21 0 1	1		Site_4_DWF	1	0	6266.95	0	0	0.322
40 0		1	Industrial	1	0	16174.08	0	5734.3	6.002
41.62 0		0	0	0	0	0	0	0	0.101
0 0		1	Industrial	1	0	58475.52	0	5752.8	6.025
62 0		1	Site 1 DWF	1	0	52217.98	0	5749.8	6.005
		0	0	0	0	0	0	0	0.046
0		1	Site_1_DWF	1	0	33971.06	0	0	0.44
0			Industrial		0	55157.76	0	5752.4	6.019
_	1	ı							

Max Depth	(ft)	0.13
Time Surcharged	(min)	0
Time Flooded	(min)	0
Volume of DWF	Inflow (ft^3)	0
Volume of Const.	Inflow (ft^3)	0
Peaking	Factor	0
20;+c;20/1/202000T	remporar variation	0
Dry Weather	Flow Flag	0
Constant	Inflow (cfs)	0
Invert Elevation	(ft)	48.61
Ground	Elevation (ft)	52
0 000	allie	



APPENDIX G – Project Summaries

Project Identifier:

New Relief Trunk Line 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	U	nit 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%	ı	\$ 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

Project Identifier:

E Columbia Avenue

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



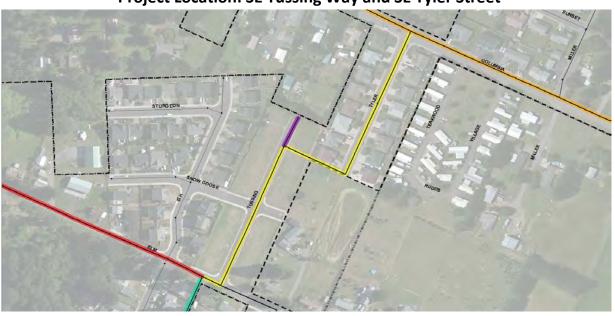
Item	Unit	Uı	nit Price	Quantity	Cost (2016)
21" Pipe - excavation, backfill, less than 10' deep	LF	\$	120	506	\$ 60,768
30" Pipe - excavation, backfill, less than 10' deep	LF	\$	185	2380	\$ 440,319
Reconnect Services	LF	\$	10	2887	\$ 28,865
48" Manhole - 8" thru 18" pipes	EA	\$	4,250	1	\$ 4,250
Connect 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

SE Tussing Way and SE Tyler Street 1A.3

Project Identifier:

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	Uı	nit 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

Project Identifier:

SW Em Watts Road
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	\$	55 114	\$ 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	\$ 4,2	50 7	\$ 29,750
Connect Pipes at Manhole - 8" thru 21"	EA	\$ 1,50	00 7	\$ 10,500
1/2 Lane Pavement Repair	LF	\$	850	\$ 25,497
Bypass Pumping	EA	\$ 5,0	00 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

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	U	Init 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

Lift Station Improvements

Project Identifier:

C

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

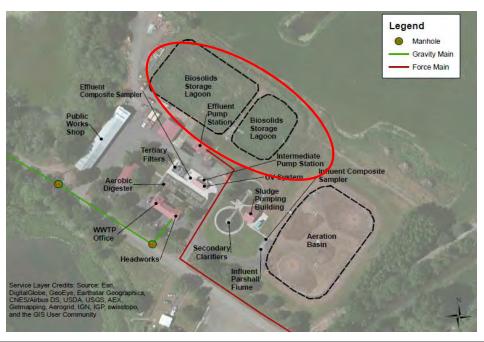
Wastewater Facilities Project: Project Identifier:

Interim Biosolids Plan

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

Wastewater Facilities Project:

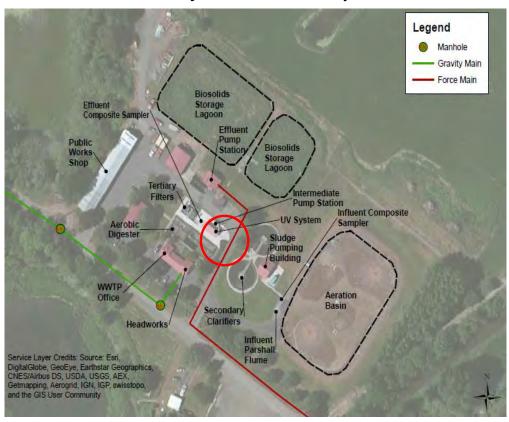
Rehabilitate UV System

1b

Project Identifier:

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

Wastewater Facilities Project: Project Identifier:

Add 3rd pump to Inter. Pump Station 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

Wastewater Facilities Project: Project Identifier:

Add disks to existing Tertiary Filters 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

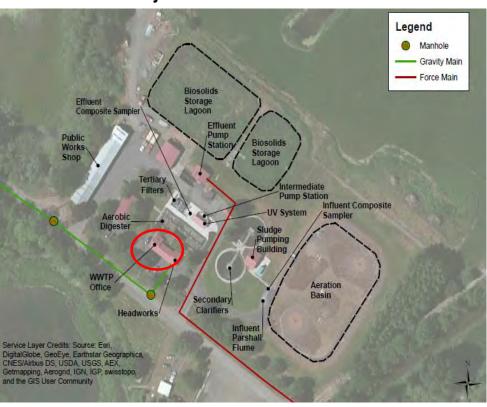
SCADA System

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

Wastewater Facilities Project: Project Identifier:

Aeration for Aeration Basin 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.

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

Project Identifier:

SE 6th Street Trunk Line

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 Prid	e Quanti	ty	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

NE Laurel Street and NE 3rd Street 2B

Project Identifier:

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	Ur	nit 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%	ı	\$ 10,595
Contingency	%		30%	ı	\$ 66,749
Construction Subtotal (rounded)					\$ 290,000
Engineering and CMS	%		25%	ı	\$ 72,500
Legal, Admin, and Permitting	%		2%	-	\$ 5,800
Total Project Cost (rounded)		·			\$ 370,000

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

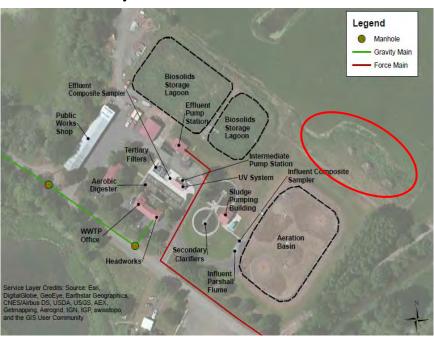
Project Identifier:

New Aeration Basins

2a.1

Objective: Replace the aeration basin with two (2) new aeration basins ($^{\sim}$ 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

Wastewater Facilities Project: Project Identifier:

New Aerobic Digester 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



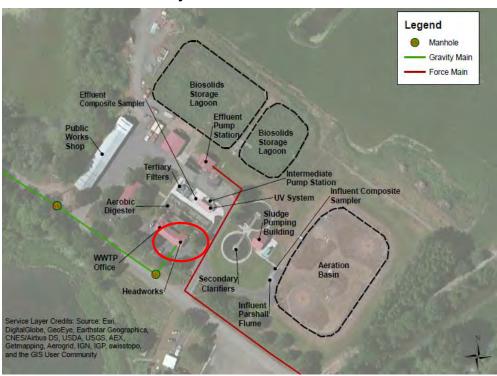
Item	Cost (2016)
Site Work	\$ 90,000
Digester Basin	\$ 400,000
Misc. Metals (guardrail, grating)	\$ 100,000
Piping/Valves and Instrumentation	\$ 60,000
Digester Equipment (including Blowers)	\$ 270,000
Electrical/Controls	\$ 140,000
Mobilization (10%)	\$ 110,000
Overhead and Profit (15%)	\$ 160,000
Contingency (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

Wastewater Facilities Project: Project Identifier:

Expand Headworks 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

Upgrade Influent Pumps 2b.2

Project Identifier:

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

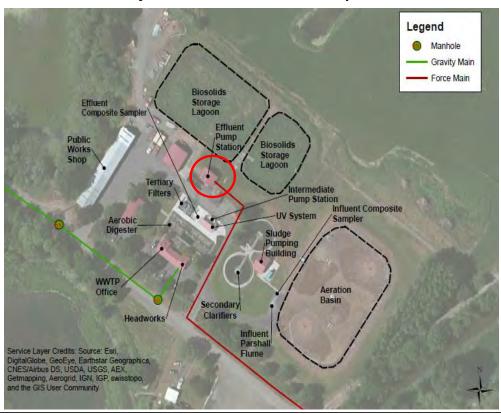
Upgrade Effluent Pumps

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

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

Upgrade Intermediate Pump Station 2d.1

Project Identifier:

Objective: Upgrade the pumps in the Intermediate Pump Station to handle the future 2035 peak instantaneous flow.

Project Location: Intermediate Pump Station



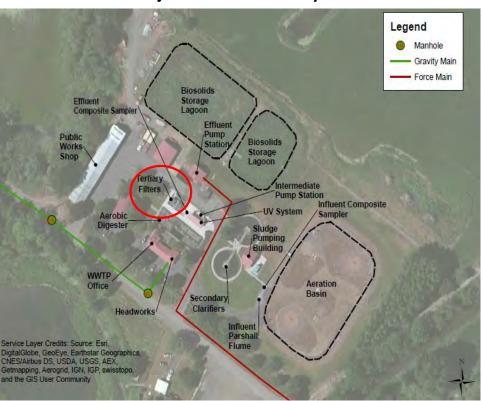
Item	Cost (2016)
Upgrade Intermediate Pump Station	\$ 200,000
Electrical/Controls	\$ 40,000
Mobilization (10%)	\$ 24,000
Overhead and Profit (15%)	\$ 36,000
Contingency (30%)	\$ 72,000
Construction Subtotal	\$ 372,000
Engineering & CMS (20%)	\$ 75,000
Legal, Administrative, and Permitting (2%)	\$ 8,000
Total Project Cost	\$ 455,000

Additional Tertiary Filter Unit 2d.2

Project Identifier:

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



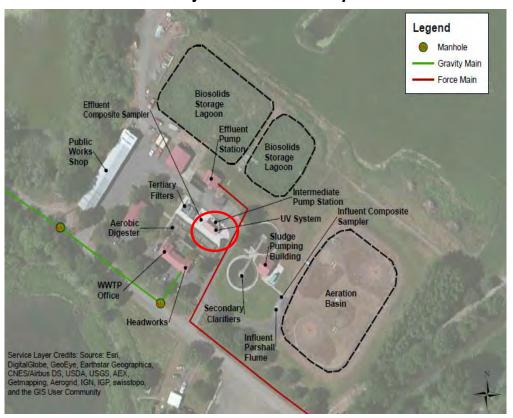
Item	Cost (2016)
Site Work	\$ 50,000
Tertiary Filter	\$ 310,000
Cover	\$ 25,000
Electrical/Controls	\$ 77,000
Mobilization (10%)	\$ 47,000
Overhead and Profit (15%)	\$ 70,000
Contingency (30%)	\$ 139,000
Construction Subtotal	\$ 718,000
Engineering & CMS (20%)	\$ 144,000
Legal, Administrative, and Permitting (2%)	\$ 15,000
Total Project Cost	\$ 877,000

Upgrade UV System
2e

Project Identifier:

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

SW Old Portland Road

Project Identifier:

ЗА

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	Un	it 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

Project Identifier:

SE Tussing Way Segment

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	Uni	it 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

Wastewater Facilities Project: Project Identifier:

Additional Aeration Basin 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



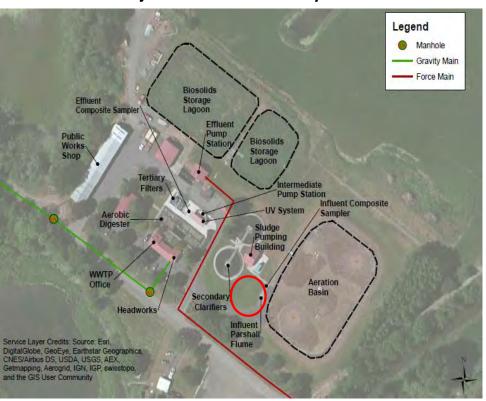
Item	Cost (2016)
Site Work	\$ 80,000
New Aeration Basin	\$ 620,000
Mixers/Pumps	\$ 70,000
Fine Bubble Diffusers	\$ 40,000
Blowers	\$ 300,000
Misc. Metals (guardrail, grating)	\$ 120,000
Piping/Valves and Instrumentation	\$ 200,000
Electrical/Controls	\$ 260,000
Mobilization (10%)	\$ 170,000
Overhead 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

Wastewater Facilities Project: Project Identifier:

Additional Secondary Clarifier 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



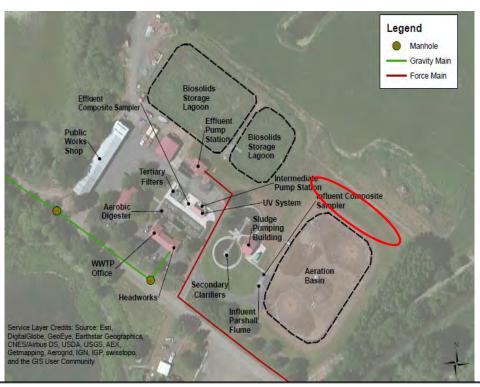
Item	Cost (2016)
Site Work	\$ 60,000
Secondary Clarifier	\$ 420,000
Additional Sludge Pumps	\$ 80,000
Electrical/Controls	\$ 120,000
Mobilization (10%)	\$ 70,000
Overhead and Profit (15%)	\$ 110,000
Contingency (30%)	\$ 210,000
Construction Subtotal	\$ 1,070,000
Engineering & CMS (20%)	\$ 220,000
Legal, Administrative, and Permitting (2%)	\$ 30,000
Total Project Cost	\$ 1,320,000

Additional Aerobic Digester 3b.1

Project Identifier:

Objective: An additional aerobic digester should be added when the maximum month flow is approximately 2.3 MGD.

Project Location: New Aerobic Digester



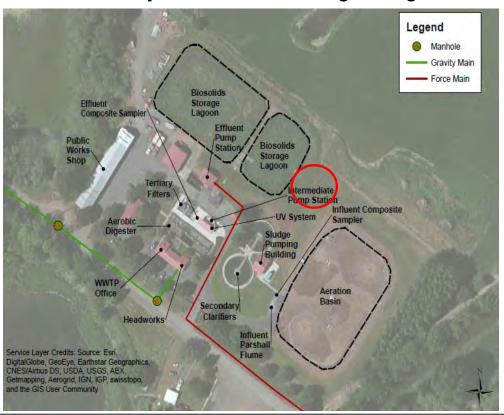
Item	Cost (2016)
Site Work	\$ 90,000
Digester Basin	\$ 360,000
Misc. Metals (guardrail, grating)	\$ 100,000
Piping/Valves and Instrumentation	\$ 60,000
Digester Equipment (including Blowers)	\$ 220,000
Electrical/Controls	\$ 130,000
Mobilization (10%)	\$ 100,000
Overhead and Profit (15%)	\$ 150,000
Contingency (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

Additional Screw Presses 3b.2

Project Identifier:

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

Wastewater Facilities Project: Project Identifier:

Plant Water System

3с

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

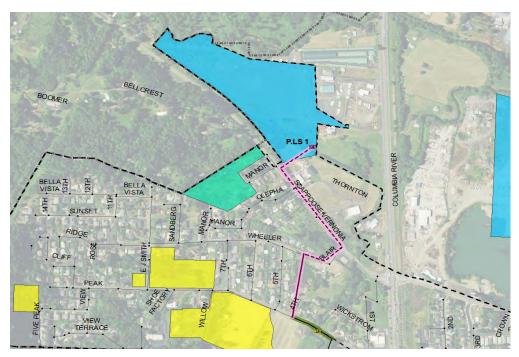


Item	Cost (2016)
Plant Water Pumps	\$ 40,000
Chlorination System	\$ 45,000
Piping and Valves	\$ 8,000
Electrical/Controls	\$ 11,000
Mobilization (10%)	\$ 11,000
Overhead and Profit (15%)	\$ 16,000
Contingency (30%)	\$ 32,000
Construction Subtotal	\$ 163,000
Engineering & CMS (25%)	\$ 41,000
Legal, Administrative, and Permitting (2%)	\$ 4,000
Total Project Cost	\$ 208,000

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

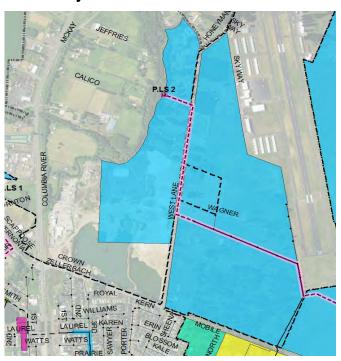
Project Identifier:

Proposed Lift Station 2

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

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 Pr	ice	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

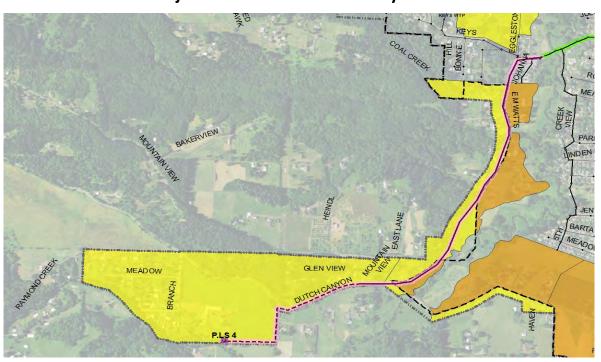
Project Identifier:

Proposed Lift Station 4

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



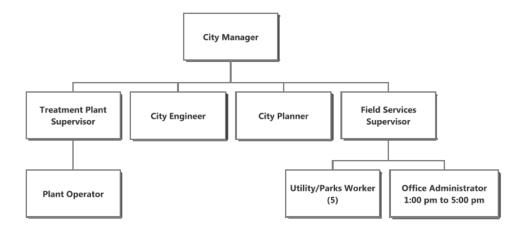
Item	Unit	Unit Price	Quantity	Cost (2016)
Lift Station (peak flow of 185 gpm)	LF	\$ 169,000	1	\$ 169,000
6" Force Main	LF	\$ 70	3000	\$ 210,000
8" Gravity Main	EA	\$ 60	4900	\$ 294,000
Bypass Pumping	EA	\$ 5,000	1	\$ 5,000
Traffic Control	LF	\$ 2	7900	\$ 15,800
Mobilization	%	5%	-	\$ 34,690
Contingency	%	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



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

		·			Minimum	Maximum
	12-13	13-14	14-15	15-16	Salary	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

					Minimum	Maximum
	12-13	13-14	14-15	15-16	Salary	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						Proposed	,	Approved	Adopted
Resources	<u>F</u>	Actual Y 12-13	Actual FY 13-14	Budget FY 14-15	Estimated FY 14-15	Budget FY 15-16		Budget FY 15-16	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 Charges for services Intergovernmental Long term debt proceeds Miscellaneous	\$	3,565 1,048,838 69,822 728	\$ 4,458 1,047,046 700,000 200	\$ 4,000 1,062,250	\$ 4,000 1,056,200 875	\$ 4,000 1,069,500	\$	4,000 1,069,500	\$ 4,000 1,069,500
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
						Proposed	,	Approved	Adopted

Expenditures	<u> </u>	Actual Y 12-13	F	Actual Y 13-14	Budget FY 14-15	Estimated FY 14-15	Proposed Budget FY 15-16	Approved Budget FY 15-16	Adopted Budget Y 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	2013/2014	2014/2015	2014/2015	Account	Utility Wastewater Fund	2015/2016
Actual 3,565	Actual 4,458	Adopted 4,000	Estimated 4,000	41-000-003	Description Interest Earned	Budget 4,000
728	200	4,000	,	41-000-003	Miscellaneous	4,000
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		41-000-220	User Fees	1,063,000
600	3,129	2,250		41-000-240	Hookup Fees	4,500
-	-	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		41-410-110	City Recorder	15,063
	-	22,211		41-410-113	City Engineer	27,646
79,685	83,252	75,794		41-410-114	Treatment Plant Supervisor	80,403
27,291	14,414	17,816		41-410-118	Field Services Supervisor	21,944
124,109	95,078	64,595		41-410-120	Operators	67,924
75,360	50,773	68,211		41-410-121	Utility Workers	69,351
17,998	18,321	18,548		41-410-132	CDC Administrator	12,450
1,431	1,505	1,450		41-410-142	Overtime	4,725
105,256	88,284	129,253		41-410-146	Health Insurance	143,026 78,941
83,130 30,789	70,429 26,621	70,058 28,540		41-410-148 41-410-150	Retirement Benefits Social Security	30,804
11,156	10,779	10,642		41-410-150	Workers Compensation	12,861
640,282	561,814	595,237	535,504	41-410-132	Total Personnel Services	650,678
4,720	2,962	8,000		41-410-200	Building/Facilities Maintenance	9,680
58,805	9,891	7,364		41-410-202	Equipment Maintenance	23,864
1,774	2,158	2,400		41-410-203	Maintenance Agreements	2,200
3,129	2,206	8,460		41-410-204	Vehicle Maintenance	8,190
16,401	17,127	9,050		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		41-410-220	Shop Maintenance Supplies	850
3,586	3,558	7,775		41-410-222	Lab Supplies	8,875
5,466	607	6,400		41-410-224	Chemicals	3,080
78,135	82,966	85,800		41-410-227	Electrical Operations	85,800
10,801	8,835	11,724		41-410-228	Utilities	12,060
6,025	6,549	7,200		41-410-229	Electrical Operations Pumps Contractual/Professional	7,500
171,032	126,016	180,100 100		41-410-230 41-410-234	Miscellaneous	224,800
2,099	1,860	2,000		41-410-235	Property Tax	2,000
	192	-	•	41-410-238	Insurance	-
6,999	2,429	7,475		41-410-240	Travel/Training	9,750
4,162	18,713	4,870		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		41-410-300	Equipment	340,000
324,534	393,121	126,500	85,000	44 440 500	Total Capital Outlay	340,000
899	-	-		41-410-500	Vehicle Lease Principal	-
56	-	100,000		41-410-501 41-410-510	Vehicle Lease Interest	100,000
-	8,453	16,055	•	41-410-510	Principal USNB Interest USNB	13,585
7,125	7,552	7,552		41-410-511	Principal SPWF B92001B	13,303
881	453	453		41-410-541	Interest SPWF B92001B	_
-	-	-		41-410-550	Principal SPWF B95001C	-
17,642	17,642	17,642		41-410-560	Principal CWSRF R80930	17,642
816	772	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		41-410-418	Transfer to Unemployment	-
52,632	56,433	106,429		41-410-419	Transfer to General Fund ISF	63,101
52,632	56,433	111,229	111,229		Total Transfers	63,101
1 440 572	1 /17 525	519,757 1 054 222		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		Actual	Actual	Budget	Estimated	Proposed Budget	Approved Budget		Adopted Budget
Resources	<u>F</u>	Y 12-13	FY 13-14	FY 14-15	FY 14-15	Y 15-16	FY 15-16	F	Y 15-16
Working capital carryover	\$	478,274	\$ 460,587	\$ 525,344	\$ 509,262	\$ 605,930	\$ 605,930	\$	605,930
Current year resources Interest System development charges	\$	2,815 8,332	\$ 2,787 90,992	\$ 3,000 73,648	\$ 2,850 132,500	\$ 2,900 149,160	\$ 2,900 149,160	\$	2,900 149,160
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 Y 12-13	ļ	Actual FY 13-14	Budget FY 14-15	Estimated FY 14-15	Proposed Budget FY 15-16	Approved Budget FY 15-16	Adopted Budget Y 15-16
Capital outlay Transfers Contingency	\$ 28,253 581	\$	43,660 1,443	\$ 450,000 3,682 148,310	\$ 35,000 3,682	\$ 650,000 7,458 100,532	\$ 650,000 7,458 100,532	\$ 650,000 7,458 100,532
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

Exi	sting Budget is Balanced	
	FY 2015-2016 Revenue	\$ 1,073,500

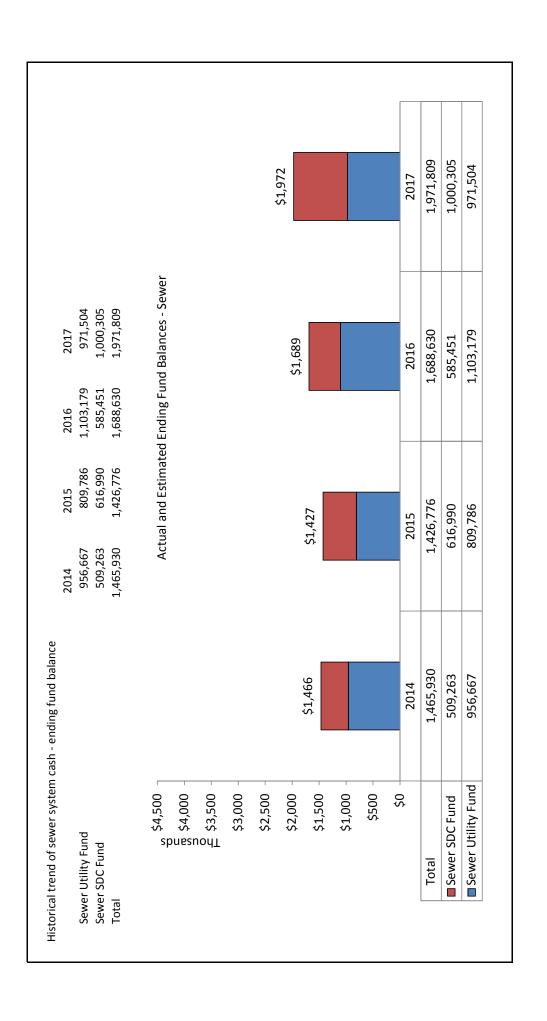
After SDCs, Priority 1 Improvements	
Priority 1 Improvement Costs	\$ 5,140,000
Less Available SDC Funds / Reserves	\$ -
Amount to Finance	\$ 5,140,000
Annual Payment (20yr, 1.6%)	\$ 302,343
Annual O&M Increase	\$ -
Payment % of FY 2015-2016 Revenue	28%
Additional Monthly Cost Per User (2700 EDUs)	\$ 9.33

Appendix B FINANCIAL PLAN

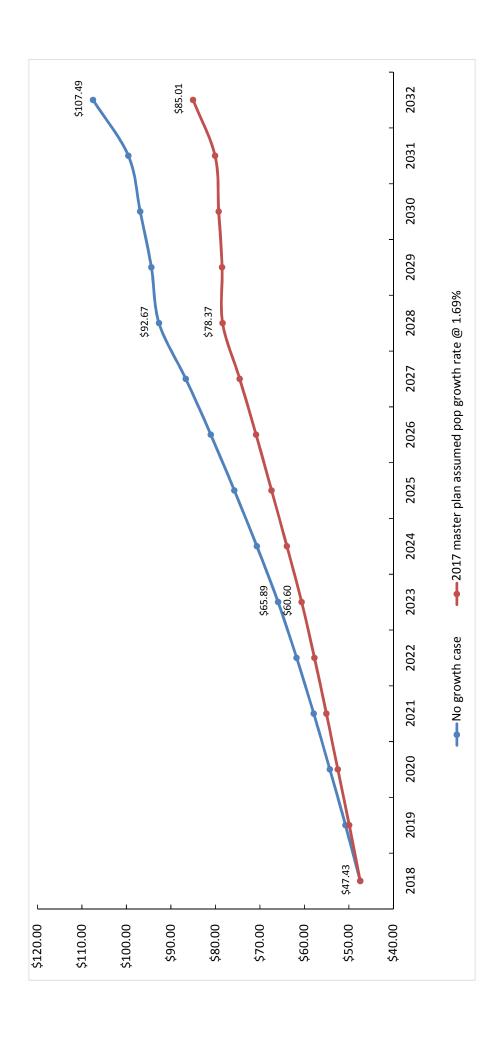


Prototype Scappoose Sewer Financial Forecast Model Summary of Planning Assumptions

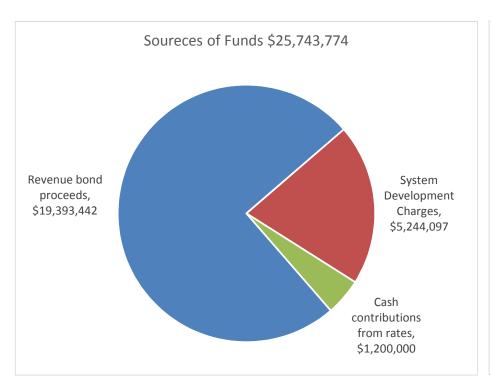
	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036
Percentage Growth in the Customer Base	2.10%	2.10%	2.10%	2.10%	2.10%	2.10%	2.10%	2.10%	2.10%	2.10%	2.10%	2.10%	2.10%	2.10%	2.10%	2.10%	2.10%	2.10%	2.10%
Interest earnings rate Non-rate revenue items inflation rate	0.80% 3.00%																		
	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.007	0.0070	0.0070	0.0070	0.0070	0.0070
Requirements: Personal Services:																			
City manager	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Public works director	3.00% 3.00%																		
City planner Finance/office manager	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Program specialist	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Office administrator I	3.00% 3.00%	3.00%	3.00%	3.00% 3.00%	3.00%	3.00% 3.00%	3.00% 3.00%	3.00%	3.00% 3.00%										
City recorder City engineer	3.00%	3.00% 3.00%	3.00% 3.00%	3.00%	3.00% 3.00%	3.00%	3.00%	3.00% 3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Treatment plant supervisor	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Field services supervisor	3.00% 3.00%	3.00% 3.00%	3.00% 3.00%	3.00% 3.00%	3.00%	3.00% 3.00%													
Operators Utility workers	3.00%	3.00%	3.00%	3.00%	3.00% 3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Engineering assistant PW	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
CDC administrator	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Overtime Health insurance	3.00% 7.00%																		
Retirement benefits	8.00%	8.00%	8.00%	8.00%	8.00%	8.00%	8.00%	8.00%	8.00%	8.00%	8.00%	8.00%	8.00%	8.00%	8.00%	8.00%	8.00%	8.00%	8.00%
Social security	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Workers compensation Total personal services	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Materials and Services:																			
Building/facilities maintenance	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Equipment maintenance	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Maintenance agreements Vehicle maintenance	3.00% 3.00%																		
Small equipment	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Fuel/oil/lube	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Office supplies Operational supplies	3.00% 3.00%																		
Shop maintenance supplies	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Lab supplies	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Chemicals Electrical operations	3.00% 3.00%																		
Utilities	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Electrical operations pumps	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Contractual/professional Miscellaneous	3.00% 3.00%																		
Property tax	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Insurance	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Travel/training Dues/fees/subscriptions	3.00% 3.00%																		
Publications/notices/advertise	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Uniforms/safety	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Sludge disposal Equipment rental	3.00% 3.00%																		
Total materials and services	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Capital Outlay:																			
Equipment & plant upgrades	3.00%	3.00%	3.00%	3.00%	3.00%	3.00% 3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Master plan projects Other	3.00% 3.00%	3.00% 3.00%	3.00% 3.00%	3.00% 3.00%	3.00% 3.00%	3.00%	3.00% 3.00%												
Other	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Other	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Other Capital reserve	3.00% 3.00%																		
Total capital outlay	3.00 /6	3.00 /6	3.00 /0	3.00 /6	3.00 /0	3.00 /6	3.00 /0	3.00 /0	3.00 /6	3.00 /0	3.00 /0	3.00 /0	3.00 /0	3.00 /6	3.00 /0	3.00 /0	3.00 /6	3.00 /0	3.00 /0
Transfers:																			
Transfer to unemployment	3.00% 3.00%	3.00%	3.00% 3.00%	3.00% 3.00%	3.00% 3.00%	3.00% 3.00%	3.00% 3.00%	3.00%	3.00% 3.00%	3.00% 3.00%	3.00% 3.00%	3.00% 3.00%	3.00%	3.00% 3.00%	3.00% 3.00%	3.00% 3.00%	3.00% 3.00%	3.00% 3.00%	3.00% 3.00%
Transfer to general fund ISF Transfer to Building	3.00%	3.00% 3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00% 3.00%	3.00%	3.00%	3.00%	3.00%	3.00% 3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Transfer to Planning	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Transfer to Sewer Construction Fund	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Total transfers																			
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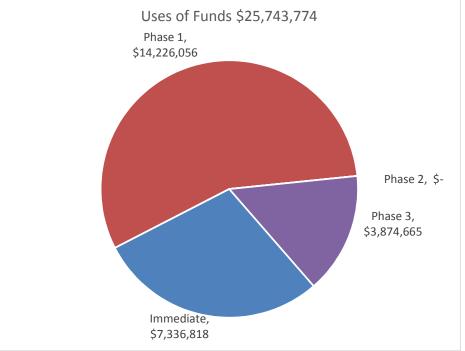


		2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032
Single family residential monthly rate:																
Base	\$	47.36														
Use:																
Assumed winter ave. water consumption - gallons		4,115														
Amount of water allowed in base charge - gallons		4,000														
Use charge - \$/100 gallons over allowance	\$	0.06														
Single family residential monthly rate:	\$	47.43														
Francisco de la companya de la francisco de la companya de la comp	. .	725.000 6	4.045.545	4.072.4026	2.405.424 6	2 245 224 6	2 206 506 6	2.570.204	2754567 6	2046 564 6	2.454.200 6	2 270 205 6	2.422.5626	2.524.262 6	2 620 056 6	2 000 252
Forecasted revenues required from rates	\$ 1,	/25,000 \$	1,845,545 \$	1,9/3,403 \$	2,105,131 \$	2,245,221 \$	2,396,596 \$	2,570,281 \$	2,/54,56/ \$	2,946,561 \$	3,151,300 \$	3,370,385 \$	3,432,562 \$	3,524,363 \$.	3,620,856 \$	3,909,353
Forecasted EDUs:																
Start year		3,031														
Forecast:		,														
No growth case		3,031	3,031	3,031	3,031	3,031	3,031	3,031	3,031	3,031	3,031	3,031	3,031	3,031	3,031	3,031
2017 master plan assumed pop growth rate @ 1.69%		3,031	3,082	3,134	3,187	3,241	3,296	3,351	3,408	3,466	3,524	3,584	3,644	3,706	3,769	3,832
2015 rate study assumed pop growth rate @ 2.098%		3,031	3,121	3,214	3,310	3,409	3,510	3,615	3,723	3,833	3,948	4,065	4,186	4,311	4,440	4,572
, , , , , ,																
Forecasted monthly rates - \$/EDU																
No growth case	\$	47.43 \$	50.74 \$	54.26 \$	57.88 \$	61.73 \$	65.89 \$	70.67 \$	75.74 \$	81.02 \$	86.64 \$	92.67 \$	94.38 \$	96.90 \$	99.56 \$	107.49
2017 master plan assumed pop growth rate @ 1.69%	\$	47.43 \$	49.90 \$	52.47 \$	55.04 \$	57.73 \$	60.60 \$	63.91 \$	67.35 \$	70.85 \$	74.51 \$	78.37 \$	78.49 \$	79.25 \$	80.07 \$	85.01
2015 rate study assumed pop growth rate @ 2.098%	\$	47.43 \$	49.27 \$	51.16 \$	53.00 \$	54.89 \$	56.90 \$	59.25 \$	61.66 \$	64.05 \$	66.52 \$	69.09 \$	68.33 \$	68.12 \$	67.96 \$	71.26
			5%	5%	5%	5%	5%	5%	5%	5%	5%	5%	0%	1%	1%	6%



	2016	2035
2017 WW master plan population forecast	7,610	10,461
Compounded annualaized population growth rate		1.69%
Master Plan CIP Funding Plan:		
Sources of funds:		
Revenue bond proceeds	\$ 19,393,442	
System Development Charges	\$ 5,244,097	
Cash contributions from rates	\$ 1,200,000	
	\$ 25,837,539	
Uses of funds:		
Water projects cost	\$ 25,437,539	





Appendix C CURRENT NPDES PERMIT



Expiration Date: October 31, 2014

Permit Number: 100677 File Number: 78980 Page 1 of 21 Pages

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:	SOURCES COVERED B	Y THIS PERM	MIT:			
City of Scappoose 33568 East Columbia Avenue Scappoose, OR 97056	Type of Waste Treated Wastewater	Outfall Number 001	Outfall Location R.M. 10.6			
FACILITY TYPE AND LOCATION:	RECEIVING STREAM INFORM	MATION:				
Activated Sludge/Extended Aeration Scappoose STP	Basin: Willamette Sub-Basin: Lower Willam	nette				
34485 East Columbia Avenue Scappoose OR 97056	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 Applicat	ion No. 971614 received June 9, 2009.					
This permit is issued based on the land use fi	indings in the permit record.					
Gregory L. Geist, Manager, Water Quality S Northwest Region	ource Control Section Da	nte				

Page 2 of 21 Pages

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:

	<u>Page</u>
Schedule A - Waste Discharge Limitations not to be Exceeded	3
Schedule B - Minimum Monitoring and Reporting Requirements	5
Schedule C – <i>Not used</i>	8
Schedule D - Special Conditions	9
Schedule E – \hat{Not} <i>Applicable</i>	
Schedule F - General Conditions	11

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.

Page 3 of 21 Pages

SCHEDULE-A

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

a. <u>Treated Effluent Outfall 001</u>

(1) May 1 - October 31:

Triay 1 Coloce 51.						
	Average Effluent		Monthly*	Weekly*	Daily*	
	Concentrations		Average	Average	Maximum	
Parameter	Monthly	Weekly	lb/day	lb/day	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:

Trovenioer i ripin so.					
	Average Effluent		Monthly*	Weekly*	Daily*
	Concentrations		Average	Average	Maximum
Parameter	Monthly	Weekly	lb/day	lb/day	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 BOD ₅ and TSS Removal Efficiency	Shall be within the range of 6.0 - 9.0 Shall not be less than 85% monthly average for BOD ₅ and 85% monthly for TSS.	

(4) <u>Mixing Zone</u>: 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) <u>Chlorine</u>: 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

Page 4 of 21 Pages

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

b. <u>Groundwater</u>: 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	
рН	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
pН	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	2/Week	Calculation of Daily Mass
and TSS)		Load
Pounds Discharged (BOD ₅	Weekly	Calculation of Weekly
and TSS)		Average Mass Load
Pounds Discharged (BOD ₅	Monthly	Calculation of Monthly
and TSS)		Average Mass Load
Average Percent Removed	Monthly	Calculation
(BOD ₅ and TSS)		

c. <u>Biosolids Management</u>

Page 6 of 21 Pages

File Number: 78980 Permit Number: 100677

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. <u>Temperature Monitoring (Monitored only required May 1 - October 31)</u>

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

2. <u>Monitoring Report Procedures</u>

File Number: 78980
Permit Number: 100677
Page 7 of 21 Pages

a. <u>Monitoring results</u> 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 <u>15th day</u> 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. <u>Monitoring reports</u> 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. <u>Biosolids Report Submittals</u>

<u>Biosolids Report</u>: For any year in which biosolids are land applied, a report shall be submitted to the Department by <u>February 19</u> 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. <u>UV Intensity Measurement</u>. 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/cm². 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. <u>Biosolids Sampling</u>. Biosolids composite samples from the storage lagoon or pond shall be taken from reference areas in the storage lagoon or pond pursuant to <u>Test Methods for Evaluating Solid Waste, Volume 2</u>; Field Manual, Physical/Chemical Methods, November 1986, Third Edition, Chapter 9.
 - Inorganic pollutant monitoring must be conducted according to <u>Test Methods for Evaluating Solid Waste</u>, <u>Physical/Chemical Methods</u>, Second Edition (1982) with Updates I and II and Third Edition (1986) with Revision I.
- 3. <u>Volatile Solids Reduction</u>. 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. <u>Fecal Coliform Sampling</u>. Analyze and report a fecal coliform result for each sample separately. Calculate and report the geometric mean of all the samples.
- 5. <u>Temperature Monitoring</u>. 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

Page 8 of 21 Pages

not exceed the stream temperature standard of 18.0 $^{\circ}$ C (64.4 $^{\circ}$ F). Temperature monitoring results must be reported on the monthly DMR.

SCHEDULE-C

Permit Compliance Conditions

There are no compliance conditions.

Page 9 of 21 Pages

SCHEDULE-D

Special Conditions

- 1. <u>Sewer Cleaning Report</u>: 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 <u>February 19</u> 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. <u>Biosolids Management</u>: 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. <u>Changes in Biosolids Standards</u>: 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. <u>Operator Certification</u>: 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.
 - <u>Note</u>: 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.

Page 10 of 21 Pages

- 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. <u>Groundwater</u>: 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. <u>Notification</u>: 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)

Page 11 of 21 Pages

SCHEDULE-F NPDES GENERAL CONDITIONS – DOMESTIC FACILITIES

(Schedule-F, last update 9.18.2009)

SECTION-A, STANDARD CONDITIONS

1. <u>Duty to Comply with Permit</u>

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.

File Number: 78980
Permit Number: 100677
Page 12 of 21 Pages

4. <u>Duty to Reapply</u>

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

Page 13 of 21 Pages

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. <u>Bypass of Treatment Facilities</u>

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:

Page 14 of 21 Pages

- 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. <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

Page 15 of 21 Pages

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.

Page 16 of 21 Pages

SECTION-C, MONITORING AND RECORDS

1. <u>Representative Sampling</u>

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 \pm 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. <u>Penalties of Tampering</u>

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

Page 17 of 21 Pages

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

Page 18 of 21 Pages

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. 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. <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.

File Number: 78980
Permit Number: 100677
Page 19 of 21 Pages

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

Page 20 of 21 Pages

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.

Page 21 of 21 Pages

- 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



Department of Environmental Quality Northwest Region Portland Office/Water Quality

700 NE Multnomah Street, Suite 600 Portland, OR 97232 (503) 229-5263 FAX (503) 229-6957 TTY 711

July 27, 2018

Kevin Turner Wastewater Supervisor City of Scappoose 34485 East Columbia Ave. Scappoose, Oregon 97056

RE: Facilities Plan Approval

WQ – COLUMBIA COUNTY CITY OF SCAPPOOSE FACILITY PLAN File #78980 /Permit # 100677 /EPA ID # OR0022420

Dear Mr. Turner,

I have reviewed and approved the Final City of Scappoose (City) Facility Plan (Plan) dated June 2018 and prepared by Corollo Engineering, Inc. (stamped by Bhargavi Maremanda PE), with supporting documents by Keller and Associates (stamped by Peter Olson PE). Together these documents fulfill the Clean Water State Revolving Fund (SRF) planning document requirements.

The existing wastewater treatment plant (WWTP) works is reaching capacity quickly. The treatment system will need improvement to multiple treatment units in the near future to address higher incoming wastewater flows.

The Plan is comprehensive with the two different documents combining to complete the planning document requirements. Planning period and cost estimating issues were resolved since the draft was submitted.

Consider the Facility plan complete. I look forward to seeing construction documents for first projects coming out of the planning in the near future.

If you have any questions about the contents of this letter, I can be reached at 503-229-5310 or by email at pinney.mike@deq.state.or.us.

Regards,

Michael L. Pinney PE

Senior Environmental Engineer

CC: Bhargavi Maremanda, Carollo Engineering, 720 SW Washington Street, Suite 550

Portland, Oregon 97205

Ecc: Tiffany Yelton-Bram, DEQ-NWR; Jennifer Purcell

Evan Haas, CWSRF-NWR

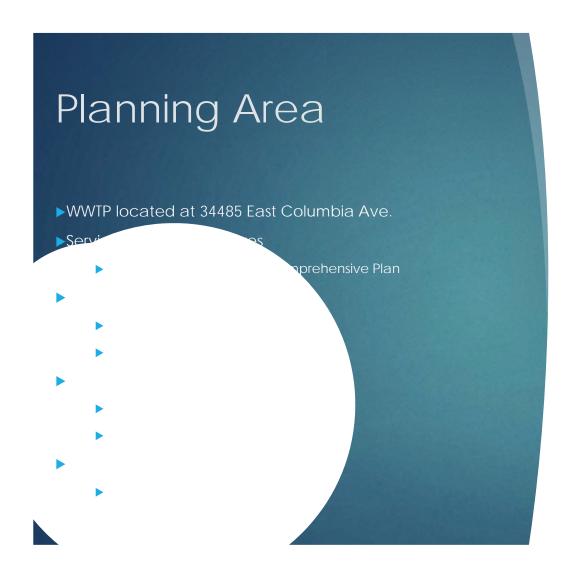


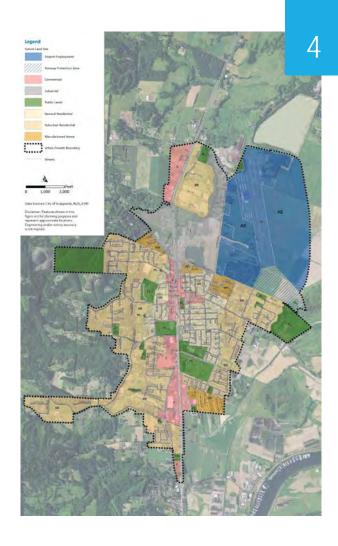
Summary

- 2015 Keller Associate Inc.
 - Comprehensive Wastewater Planning Study
 - ▶ Collections System
 - ▶ Treatment Plant
 - Appendix A
- ▶ 2017 Carollo Engineers
 - ► Facility Plan Update
 - ▶ Planning horizon to 2042
 - Peer Review
 - ▶ Evaluation of WWTP and Wastewater Planning Study recommendations

Objectives

- Planning document that identifies and details a logical path forward for
 - Reliable treatment for existing and future populations
 - Address operational issues
 - Maintain NPDES permit compliance
 - Accommodate projected growth in the planning period
- Evaluate operational changes and facility improvements
 - Potential discharge limitations nutrient removal
- Develop a Capital Improvement Plan
- Provide a predesign for WWTP capital improvements





Wastewater Flow Summary

Parameter	2017	2042
Population	7610	11,568
Flow Summary, (mgd, million gallons per day)		
BWF, Base Wastewater Flow	.607	.930
ADWF, Average Dry Weather Flow	.646	.990
AWWF, Average Wet Weather Flow	.878	1.35
MMDWF, Max Monthly Dry Weather Flow	.871	1.33
MMWWF, Max Monthly Wet Weather Flow	1.20	1.83
PDF, Peak Daily Flow	2.77	4.23
PIF, Peak Instantaneous Flow	3.96	6.05

Wastewater Load Summary

Load Summary	2017	2042
BOD ₅ , Biochemical Oxygen Demand		
Average Annual Load, ppd	1,430	2,200
Maximum Month Load, ppd	2,180	3,350
TSS, Total Suspended Solids		
Average Annual Load, ppd	1,670	2,600
Maximum Month Load, ppd	2,540	3,900
Ammonia		
Average Annual Load, ppd	264	400
Maximum Month Load, ppd	450	700

Anticipated NPDES Permit Limits

Parameter	Average Monthly	Average Weekly	Maximum Daily				
Summer (May 1-October 31)							
BOD₅	10 mg/L 125 ppd 85% removal	15 mg/L 190 ppd	225 ppd				
TSS	10 mg/L 125 ppd 85% removal	15 mg/L 190 ppd	225 ppd				
Winter (November 1-April 30)							
BOD₅	25 mg/L 37 mg BOD₅ 315 ppd 475 p 85% removal		630 ppd				
TSS	25 mg/L 315 ppd 85% removal	37 mg/L 475 ppd	630 ppd				

Anticipated NPDES Permit Limits

No change

рΗ

Daily minimum 6.0

Daily Maximum 9.0

Daily Limit

E. coli Bacteria 126/100 ml MPN Monthly Limit

Potential Limit

Ammonia

< 5.0 mg/L
Monthly Average
Summer Months

No Limit

Phosphorus

Hydraulic Capacity

Process Area	Existing Capacity, mgd	Required Capacity, mgd
Influent Screen	4.0 mgd	6.05 mgd
Influent Pump Station	3.5 mgd	6.05 mgd
Aeration Basin	7.0 mgd	6.05 mgd
Secondary Clarifiers	4.7 mgd	6.05 mgd
Tertiary Filters Pumps	4.0 mgd	3.6 mgd
Tertiary Filter Disk	2.4 mgd	2.4 mgd
UV Disinfection	5.6 mgd	6.05 mgd
Effluent Pump Station	4.4 mgd	6.05 mgd
Outfall	4.3 mgd	6.05 mgd

	Construction Year					
Item Description	Phase 1 2018-2021	Phase 2 2021-2023	Phase 3 2023-2026	Phase 4 2026-2028	Total	
Phase 1 Improvements						
Springlake Lift Station	\$271,600					
UV Disinfection	\$616,600					
Hydraulic Improvements	\$519,700				\$6,460,600	
Secondary Clarifier	\$4,590,100					
Aerobic Digester	\$432,600					
Phase 2 Improvements						
Headworks and Influent Pump Station		\$5,504,400			\$7,204,400	
Operational Improvements		\$1,700,000			\$7,204,400	
Phase 3 Improvements						
UV Disinfection II			\$1,685,900			
Effluent Pump Station			\$536,600		\$4,393,200	
Outfall			\$2,170,700			
Phase 4 Improvements						
Aerobic Digester				\$2,486,900	\$2,486,900	
Total WWTP CIP					\$20,515,000	

Phase 1

Springlake Lift Station

Replacement of pumps and corroded piping

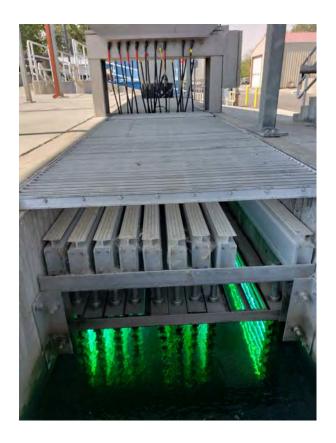
Addition of valve vault to measure flow



Phase 1

UV Disinfection

Replacement of existing UV banks with newer technology UV system.

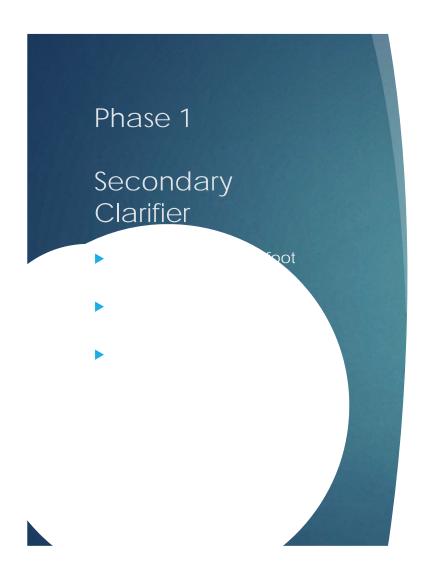


Phase 1

Hydraulic Improvements

Rebuild secondary splitte structure







PHASE 1

Aerobic Digester Improvements

Replace

Damaged coarse bubble diffusers,

Blowers

Sludge pump

Sludge flow meter

Repair hand rails





Moving Forward

- Clean Water State Revolving Fund Loan Application
 - ▶ Submitted to DEQ
 - Waiting for approvals, November 2018
- Collections System Flow Testing
 - ▶ Bid flow testing project November 2018
 - ▶ Test complete and Update Plan Spring 2019
- Rates
 - Rate update to fund CIP projects

CITY OF SCAPPOOSE

September 2018

ocpicition 2010						
Sunday	Monday	Tuesday	Wednesday	Thursday	Friday	Saturday
						1 Scappoose Farmers Market 9 am - 2pm
2	3 City Offices closed ~Labor Day	4 Work Session 6:00 pm ~ Tour of Miller Park (52451 Miller Road) City Council 7pm	5	6	7	8 Scappoose Farmers Market 9 am - 2pm
9	10	11	12	13 Planning Commission 7pm	14	15 Scappoose Farmers Market 9 am - 2pm
16	17 Work Session 6:15 pm City Council 7pm	18	19	20 EDC ~ noon Park & Rec Comm. meeting ~ 6pm Friends of Park meeting 7pm	21	22 Farmers Market 9 am - 2pm
23/30	24	25	26	27 Planning Commission 7pm	28	29 Farmers Market 9 am - 2pm