City of Sutherlin
Douglas County, Oregon

WASTEWATER FACILITIES PLAN AMENDMENT

NOVEMBER 2013



The Dyer Partnership Engineers & Planners, Inc.

1330 Teakwood Avenue
Coos Bay, Oregon 97420
(541) 269-0732 ◘ Fax (541) 269-2044
www.dyerpart.com

Project No. 146.02A
759 West Central Avenue
Sutherlin, Oregon 97479
(541) 459-4619◘ Fax (541) 459-4679

City of Sutherlin

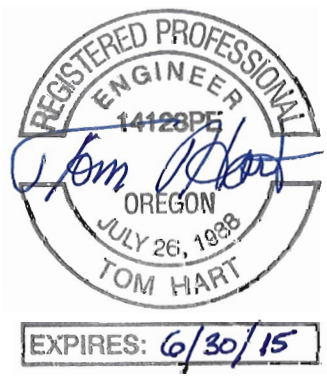
Douglas County, Oregon

Wastewater

Facilities Plan

November 2013

Project No. 146.02A



The Dyer Partnership
Engineers & Planners, Inc.

759 W. Central Avenue
Sutherlin, Oregon 97479
(541) 459-4619 Fax (541) 459-4679
www.dyerpart.com

Table of Contents

SECTION ES – EXECUTIVE SUMMARY

ES.1 Background and Purpose ES-1
ES.2 Population and Flow Projections ES-2
Population ES-2
Flows and Loads ES-2
ES.3 Collection System ES-3
ES.4 Treatment System ES-3
WWTP Condition ES-3
WWTP Improvements ES-4
ES.5 Recommendations and Costs ES-5
ES.6 Finance ES-5
User Rates ES-6

SECTION 1 – INTRODUCTION

1.1 Background 1-1
1.2 Objectives 1-1
1.3 Scope of Study 1-2
1.4 Previous Studies and Information 1-2
1.5 Authorization 1-3
1.6 Acknowledgments 1-3

SECTION 2 – STUDY AREA CHARACTERISTICS

2.1 Study Area 2-1
2.2 Physical Environment 2-3
Climate 2-3
Geologic Hazards 2-3
Public Health Hazards 2-4
Water Resources 2-4
Flora and Fauna 2-5
Environmentally Sensitive Areas 2-5
Air Quality and Noise 2-5
Energy Production and Consumption 2-6
Endangered Species Act 2-6
Wild and Scenic River System 2-6
2.3 Socioeconomic Environment 2-6
Population 2-6
Land Use 2-8

SECTION 3 – EXISTING WASTEWATER FACILITIES

3.1 System History 3-1
3.2 Wastewater Conveyance System 3-2
Collection System Description 3-2
Pump Stations 3-11
Plant Influent Pump Station 3-15

Everett Avenue Pump Station 3-16
Airport Pump Station 3-17
Quail Run Pump Station 3-18
Church Road Pump Station 3-19
Page Street Pump Station 3-20
Existing I/I Problems 3-21
Collection System Deficiencies 3-22
3.3 Wastewater Treatment Facility 3-22
History 3-22
Existing Treatment Process Description 3-24
WWTP Condition 3-27
Mechanical Bar Screen and Flow Measurement 3-27
Influent Pump Station 3-28
Headworks 3-28
Aeration Basins 3-29
Clarifier and Return Activated Sludge Pumps 3-29
Digesters 3-30
Disinfection 3-32
Contact Basin Volume 3-32
Laboratory, Chlorine and Filter Building 3-32
Pump, Blower and Generator Building 3-33
3.4 Effluent Disposal 3-34
Calapooya Creek 3-34
Recycled Water (Golf Course Irrigation) 3-34
3.5 Sludge and Biosolids Management 3-35
3.6 WWTP Sludge Quality 3-36
3.7 Emergency Standby Power Systems 3-37

SECTION 4 – WASTEWATER CHARACTERISTICS

4.1 Wastewater Volume 4-1
DEQ Guidelines for Flow Projections 4-1
Precipitation Rates for Calculations 4-1
4.1.1 Dry Weather Flow 4-2
4.1.2 Wet Weather Flow 4-3
4.1.3 Infilitration and Inflow 4-7
4.2 Wastewater Composition 4-9
4.3 Projected Wastewater Characteristics 4-10

SECTION 5 – BASIS OF PLANNING

5.1 Regulatory Environment 5-1
5.1.1 Beneficial Uses 5-1
5.1.2 CWA Water Quality Assessment, Integrated Report and 303(d) List 5-2
5.1.3 Umpqua Basin Total Maximum Daily Load (TMDL) 5-3
5.1.4 Water Quality Limited Pollutants not addressed by TMDL 5-5
5.1.5 Mixing Zone Review 5-5
5.1.6 Antidegradation Review 5-5
5.1.7 Potential Future Regulatory Issues 5-6
5.2 NPDES Permit 5-6
5.2.1 Current NPDES Permit Requirements 5-6
5.2.2 Compliance with current NPDES Permit Requirements 5-7
5.2.3 Anticipated NPDES Permit Limits 5-8

5.3 Design Criteria and Considerations 5-9
Design Period 5-9
Collection System 5-9
Wastewater Treatment Facility 5-10
Plant Reliability Criteria 5-11
5.4 Basis for Cost Estimate 5-13
Construction Costs 5-13
Contingencies 5-14
Engineering 5-14
Legal and Administrative 5-15

SECTION 6 – DEVELOPMENT AND EVALUATION OF ALTERNATIVES

6.1 Conveyance System Alternatives 6-1
I/I Reduction 6-1
Collection System Expansion 6-2
6.2 Existing Pump Station Improvements 6-5
Plant Influent Pump Station 6-5
Everett Street Pump Station 6-6
6.3 Treatment Facility Improvements 6-7
General 6-7
Influent Screen 6-8
Headworks 6-9
Biological Treatment 6-9
6.4 Disinfection Alternatives 6-29
6.5 Recycled Water 6-34
6.6 Biosolids Treatment, Storage and Disposal Alternatives 6-35
Biosolids Stabilization 6-35
Biosolids Storage 6-36
Disposal of Biosolids 6-38
6.7 Summary of Complete Wastewater Treatment Alternative 6-41

SECTION 7 – RECOMMENDED PLAN

7.1 Existing Piping System Improvements 7-1
7.2 Collection System Expansion 7-1
7.3 Pump Station Recommendations 7-1
Plant Influent Pump Station 7-1
Everett Avenue Pump Station 7-1
7.4 Treatment Facility Expansion 7-2
7.5 Biosolids Disposal 7-2
7.6 Project Cost Summary
for Wastewater Treatment Plant 7-3
7.7 Project Schedule 7-4
Project Schedule 7-4

SECTION 8 – FINANCING

8.1 Grant and Loan Programs 8-1
Economic Development Administration (EDA)
Public Works Grant Program 8-1
Water and Waste Disposal Loans and Grants (RDA) 8-1
Technical Assistance and Training Grants (TAT) 8-4
(Oregon) Community Development Block Grant (CDBG)
Program 8-4

Oregon Special Public Works Fund 8-5
Water/Wastewater Financing Program 8-6
Department of Environmental Quality, State Revolving Fund (CWSRF) 8-8
Oregon Department of Energy, Small Scale Energy Loan Program (SELP) 8-9
8.2 Local Funding Sources 8-9
General Obligation Bonds 8-9
Revenue Bonds 8-10
Capital Construction (Sinking) Fund 8-11
System Development Charges 8-11
Ad Valorem Taxes 8-12
User Fees 8-12
Assessments 8-12
8.3 Financing Strategy 8-13
Project Expenses 8-13
Funding Sources 8-13
User Rates 8-15

LIST OF TABLES

ES.2.1 Wastewater Flows and Loads ES-2
ES.2.2 Wastewater Treatment Design Values ES-3
ES.5.1 Capital Costs of Recommended Projects, Project Phase 1 ..... ES-5
ES.6.1 Potential WWTP Project Funding Sources ES-6
ES.6.2 WWTP Expansion Potential User Rates ES-7

2.2.1 Known Overflow & Bypass Locations 2-4
2.3.1 Historical Population Growth 2-6

3.2.1 Wastewater Conveyance System Inventory 3-2
3.2.2 Pump Station Design Data 3-12
3.2.3 Pump Station Design Data 3-13
3.2.4 Observed I/I Flow Amounts (GPM) in Each Basin 3-21
3.3.1 Sutherlin WWTP Component Design Specifications 3-24
3.3.2 Existing Plant Design Flows and Loading 3-27
3.3.3 Existing Plant Design Flows and Loading 3-31
3.6.1 Summary of Heavy Metals Concentration in WWTP Sewage Sludge 3-36

4.1.1 City of Sutherlin Storm Events 4-2
4.1.2 Sutherlin WWTP (2012) Existing Inflow Rates 4-6
4.1.3.1 I/I Analysis Summary 4-8
4.2.1 WWTP Influent Concentrations and Loads 4-9
4.2.2 Wastewater Design Composition Values 4-10
4.3.1 Wastewater Flows and Loads 4-11
4.3.2 Wastewater Treatment Design Values 4-12

5.1.2.1 Calapooya Creek 303(d) List 5-2
5.1.3.1 Sutherlin STP WLAS in the Umpqua Basin TMDL 5-4
5.1.3.2 October Discharge Limitations 5-5
5.2.1.1 NPDES Permit Waste Discharge Limitations 5-7

5.4.1 ENR Index – 2002 to 2012 5-14

6.3.1 Treatment Alternatives Construction Cost Summary 6-27
6.3.2 Present Worth Costs for Liquid Stream Treatment Alternatives 6-29
6.3.3 Matrix Evaluation of Liquid Stream Treatment Alternatives 6-30
6.4.1 Present Worth Costs for Disinfection Alternatives 6-33
6.4.2 Matrix Evaluation of Disinfection Alternatives 6-35
6.6.1 Present Worth Costs for Biosolids Storage Alternatives 6-41
6.6.2 Matrix Evaluation Sludge Disposal Alternatives 6-41

7.4.1 Sutherlin WWTP Component Design Specifications 7-2
7.6.1 WWTP & WW System Improvements Cost Estimate 7-3
7.6.2 WWTP Estimated Yearly O&M 7-4

8.1.1 Maximum Rural Development Grant Funds based on Median Household Income 8-3
8.1.2 CWSRF Interest Rates 8-8
8.3.1 Sutherlin Wastewater Rate Structure 8-14
8.3.2 Potential WWTP Project Funding Sources 8-15
8.3.3 WWTP Expansion Funding Source User Rates 8-15

LIST OF FIGURES

2.1.1 City Location 2-1
2.1.2 City of Sutherlin Study Area 2-2
2.3.1 Historic and Projected Population Growth 2-7
2.3.2 City of Sutherlin Zoning Map 2-9

3.2.1 Collection System 3-3
3.2.2 Wastewater System Schematic 3-10
3.2.3 Wastewater Treatment Plant Influent Pump Station 3-15
3.2.4 Inside Everett Avenue Pump Station 3-16
3.2.5 Airport Main Pump Station 3-17
3.2.6 Quail Run Pump Station 3-18
3.2.7 Church Road Pump Station 3-19
3.2.8 Page Street Pump Station 3-20
3.3.1 Wastewater Treatment Plant 3-23
3.3.2 Existing Treatment Unit 3-25
3.3.3 Process Flow Schematic 3-26
3.3.4 Mechanical Bar Screen Before Headworks 3-28
3.3.5 Pump, Blower and Generator Building & Headworks (on roof) 3-28
3.3.6 Laboratory, Chlorine and Filter Building 3-33
3.3.7 Flooded Pipe and Valve Space under Pump Room 3-33
3.4.8 Reclaimed Water Storage Pond 3-35

4.1.1.1 MMWWF & MMDWF 4-3
4.1.2.1 $PDAF_5$ Graph 4-5
4.1.2.2 Sutherlin WWTP (2012) Existing Inflow Rates 4-6
4.1.3.1 WWTP Plant Flow Trends 2008-2012 4-7
4.3.1 Future peak Flows and Existing Peak Flows 4-11

6.1.1 Future Collection System 6-4
6.3.1A Alternative No. 2– Conventional with Filter 6-12
6.3.1B Alternate No. 2 – Conventional with Filter 6-13
6.3.1C Alternative No. 2 - Conventional with MBR 6-14
6.3.1D Alternate No. 2 – Conventional with MBR 6-15
6.3.2A Alternative No. 3 - SBR with Filter 6-18
6.3.2B Alternate No. 3 - SBR with Filter 6-19
6.3.2C Alternative No. 3 - SBR with MBR 6-20
6.3.2D Alternate No. 3 - SBR with MBR 6-21
6.3.3A Alternative No. 4 – Oxidation Ditch with Filter 6-23
6.3.3B Alternate No. 4 - Oxidation with Filter 6-24
6.3.3C Alternative No. 4 - Oxidation Ditch with MBR 6-25
6.3.3D Alternate No. 4 - Oxidation with MBR 6-26

GLOSSARY

REFERENCES

APPENDICES

Appendix A Permits
Appendix B Miscellaneous Maps
Appendix C Cost Estimates
Appendix D Miscellaneous Reports
Appendix E Everett Avenue Pump Station and Force Main Evaluation

Executive Summary

Section

ES

Section

ES

Executive Summary

ES.1 Background and Purpose

The city of Sutherlin's wastewater system was originally constructed in 1956 for a design population of 3,500 people. Prior to this time, treatment consisted of individual septic tank and drain field systems. Percolation rates are poor in the Sutherlin area of the Umpqua Valley due to the preponderance of heavy clays, shale and rock. Due to population growth, it was necessary to construct a new wastewater treatment facility in 1977, which is located near the intersection of Highway 138 and Stearns Lane. It consists of an activated sludge process operated in contact stabilization mode. The wastewater treatment plant provides secondary level treatment. During the wet weather season plant effluent is discharged to Calapooya Creek. During the dry season plant effluent is recycled and land applied. Many basic components of the wastewater system have reached the end of their design life. The condition of the plant is such that it is not possible to meet current National Pollutant discharge Elimination System (NPDES) limits, particularly with regard to total maximum daily load limitations (TMDL). In addition, during dry periods effluent is irrigated (recycled) on the Umpqua Golf Resort. At times the land application exceeds the capacity of the golf course greens and surface runoff occurs.

Although the design population for the plant was theoretically sized for 8,000 users, inflow and infiltration (I/I), more stringent regulatory requirements and age of the facility have led to treatment facility deficiencies. These deficiencies exist for each major element of the treatment process, including the headworks, secondary treatment units, biosolids treatment and disposal, and effluent reuse. The city entered into a Mutual Agreement and Order (MAO) with Oregon Department of Environmental Quality (DEQ) on June 13, 2004 to set a schedule and interim compliance standards while the city works to resolve the compliance issues. Specified compliance issues include failure of wastewater effluent to meet Class A reuse regulations and the discharge of recycled water from the golf course to Cook Creek.

Since the beginning of work on the Wastewater Facilities Plan (WWFP), the city has completed a series of significant collection system improvements and has reduced the infiltration and inflow into the collection system. Mid-way through the plan, which began in 2004, DEQ delayed the plan until it completed a study of the effluent outfall into Calapooya Creek. This study led to further restrictions for effluent discharge from May through October, which will require significant storage volume of treated effluent until stream flows reach minimum flow levels.

The city of Sutherlin has experienced relatively steady growth during the period of time that it has provided a wastewater utility to the community. Sutherlin's location in the I-5 corridor, available industrial and commercial lands lends itself a steady population growth over the next 25 years.

The State or Oregon's 303(d) list for 2006 for water quality limited waters shows Calapooya Creek as Water Quality Limited (WQL). The Umpqua Basin TMDL was issued on October 31, 2006 and approved by the EPA on April 12, 2007. There is general consensus among stakeholders that TMDL requirements for Sutherlin's level of effluent phosphorus concentrations cannot be met with any practical method or technology other than storage or additional reuse areas. The effluent phosphorus level as required by the TMDL is zero between May through October, with conditional discharge in October

based upon the minimum stream flow requirement.

This plan addresses the inability of the existing wastewater system to effectively treat wastewater in accordance with current NPDES and TMDL requirements, and the improvements which are necessary to meet the specified requirements. Wastewater planning is for a 20-year period from the expected project completion (start-up) date. The WWTP is beyond its design life and the treatment capacity for biosolids is inadequate. This is partly due to rules that have changed during the 1990s and to recent changes in the regulatory environment. Continuing I/I rehabilitation projects are addressed in the study as well.

Improvements will be completed in two phases. Phase 1 improvements will bring the Wastewater Treatment Plant in compliance with the NPDES permit and corrects deficiencies at the Everett Avenue Pump Station. Phase 2 will be collection improvements such as I/I repair projects and increasing conveyance capacity.

ES.2 Population and Flow Projections

Population

Sutherlin has a full-time resident population of approximately 7,905. Census data indicates that there is an average of 1.85 people per household, although our calculations show there are approximately 2 people per wastewater equivalent dwelling unit (EDU). Projected population for the year 2037 is 11,594 people. Based on historical averages in the study area, a 1.5% per year growth rate was selected for the residential population for use in this Facilities Plan over the next 25 years. Douglas County and the Department of Land Conservation and Development (DLCD) have agreed that this rate coordinates with their efforts. Population and EDU growth are discussed in more detail in Section 2 of this Plan.

Flows and Loads

Recent WWTP Daily Monitoring Report (DMR) records were analyzed to provide the existing wastewater flows and loads. DEQ guidelines were used to extrapolate the future projected flows and loads and are discussed in Section 4. Existing users are estimated to have an average higher per capita flow than newer users due to higher infiltration. Current flows exceed the WWTP design hydraulic capacity. A disciplined I/I rehabilitation program has been implemented and applied to the system, with special focus on areas identified in the February 2004 I/I study. These efforts have reduced peak daily average flows by about 30%. Projected flows and loads for 2037 exceed the WWTP hydraulic and biosolids treatment capacity. Tables ES.2.1 and ES.2.2 below summarize the existing and projected flows and loads for the city's wastewater system.

TABLE ES.2.1
WASTEWATER FLOWS AND LOADS

Parameter	2012	2037	
Population	7,905	11,594	
Base Sewage	0.617 MGD	80 gpcd	0.93 MGD
Base Infiltration	0.11 MGD	14 gpcd	0.16 MGD
AAF	1.07 MGD	135 gpcd	1.57 MGD
ADWF	0.72 MGD	92 gpcd	1.06 MGD
AWWF	1.37 MGD	173 gpcd	2.01 MGD
$MMDWF_{10}$	1.23 MGD	156 gpcd	1.80 MGD
$MMWWF_5$	1.70 MGD	233 gpcd	2.70 MGD
Peak Average Week	2.94 MGD	351 gpcd	4.07 MGD
$PDAF_5$	5.57 MGD	604 gpcd	7.00 MGD
PIF	7.30 MGD	779 gpcd	8.80 MGD

TABLE ES.2.2
WASTEWATER TREATMENT DESIGN VALUES

	Flow Rate	BOD_5		TSS	
AAF	1.57 MGD	165 mg/L	2,250 ppd	179 mg/L	2,450 ppd
$MMDWF_{10}$	1.88 MGD	135 mg/L	2,120 ppd	150 mg/L	2,350 ppd
$MMWWF_5$	2.70 MGD	110 mg/L	2,460 ppd	117 mg/L	2,640 ppd
PDAF	7.00 MGD	n/a	n/a	n/a	n/a
PIF	8.8 MGD	n/a	n/a	n/a	n/a

ES.3 Collection System

The Sutherlin wastewater conveyance system currently consists of approximately 141,000 feet (27 miles) of gravity sewer pipe (6 to 27-inch), an estimated 700 manholes, and 15,000 feet of pressure piping (2-inch through 10-inch). The system also has five collection system pump stations (Everett Avenue, Church Road, Airport, Page Street and Quail Run) in addition to the Plant Influent Pump Station.

The city has executed annual I/I rehabilitation projects from 2004 through 2011, and has completed improvements to Airport Pump Station and Everett Avenue Pump Station. I/I repair projects included approximately 9,000 lineal feet (lf) of inversion lining, lateral repairs and 30 sanitary sewer manhole repairs. Smoking testing was performed in 2012 to indentify potential inflow areas and areas to concentrate on for ongoing I/I repair projects. See Appendix D.

ES.4 Treatment System

Raw wastewater is conveyed to the treatment facility via a 27-inch asbestos cement gravity sewer, flows to a Rotamat mechanical screen and through the influent flow meter. Then influent flows to the Influent Pump Station wet well. From the Influent Pump Station, the liquid stream is lifted to the headworks

which includes a grit separator, comminutor and Parshall flume. Flow is split from the headworks and continues by gravity to the north and south treatment units commonly referred to as "donut units". The units each have a contact zone, stabilization zone, decant zone, Return Activated Sludge (RAS) and center clarifier. The north treatment unit includes the irrigation holding reservoir and the south unit includes the chorine contact tank and the filter sump.

WWTP Condition

The facility is reaching the end of its design life. Much of the equipment is sound and operable, and the buildings and tanks are structurally sound. There is some surface corrosion of steel components due to age and environment. The biological process provides treatment and experiences minimal upsets. The hydraulic flow for the plant regularly exceeds the flow capacity of the mechanical screen chamber in the winter, and the plant is operating at capacity for mass loads. Each component of the treatment plant was examined for condition, capacity and operability. Details for each component are discussed briefly in the paragraphs below.

The existing mechanical screen is undersized for current wet-weather peak flow and commonly is bypassed into the treatment process due to excessive flows and mechanical clogging from solids. The influent pump station does not meet redundancy or peak flow requirements and solids that bypass the mechanical screen settle into the wet well. The pumps' performance has decreased from design capability into the treatment process; regardless, they must be upsized to meet existing and future demand requirements. The existing degritting system is no longer functioning, and allows solids and grit to settle into the treatment processes. Flow is measured at the parshall flume, but not after the flow is split, which may affect operational flexibility between the basins. Due to sludge wasting limitations, WWTP operators often hold mixed liquor at a concentration of 8,800 mg/L MLSS. Inadequate mixing results in operational difficulty in maintaining necessary levels of dissolved oxygen in the aeration processes. Existing clarification capacity is inadequate to treat current WWTP flows and loads. The tertiary filter has not been operable for the last twenty years. The existing digester only allowed for approximately 20 days detention based upon a hydraulic detention time calculation, yet 60 days is the requirement at 15° C. Oxygenation and mixing limit the operational capacity of digestion. Deliverable oxygen is calculated to be only 55% of what is required for optimal efficiency and the mixing system is below the fluid level, which results in a lack of surface agitation and thereby negates the maximum benefit of mixing for aeration in the digesters. During the wet season, the city periodically pays a local hauler to remove excess biosolids. The existing disinfection system consists of a chlorine gas system. This type of system has safety and supply chain issues.

Discharge from the Sutherlin WWTP is regulated under a NPDES permit. In order to protect aquatic life, the permit prohibits the discharge of effluent that violates water quality standards. In addition, Calapooya Creek is located in the Umpqua Basin, which is water quality limited for several parameters, including temperature (summers), pH (summers), fecal coliform (year-round), and phosphorus. Due to these restrictions, discharge to Calapooya Creek is not allowed between June 1st and October 31st unless approved by DEQ due to extraordinary circumstances. May and November discharges may be allowed if stream flow conditions listed in the NPDES permit are met. Effluent is pumped to the Umpqua Golf Resort course from June through September for irrigation (recycled water). The city is considering replacement of the existing chlorine gas system with a new mixed-oxidant on-site generation system. This will decrease costs and will mitigate safety issues. The system will be used to provide chlorine residuals during summertime reuse and a new UV system will be used for disinfection prior to discharge into Calapooya Creek in the winter months. In October the effluent discharge is allowed, depending upon the minimum stream flow requirements in Calapooya Creek.

The City applies the treated sludge from the digesters on agricultural land for soil enhancement utilizing a 3,200-gallon truck which spreads the biosolids directly on four sites: the Reddekopp site (80 acres); the Rust site (80 acres); the Williams site (80 acres); and the Crouch site (35 acres). In addition, sludge is hauled to a private facility (Heard Farm). Biosolids are land applied at agronomic rates as allowed under the biosolids management plan. See Appendix D for application rates.

WWTP Improvements

In short, the WWTP is deficient in preliminary treatment, secondary treatment, biosolids treatment and effluent disposal. Each area was reviewed with a minimum of three alternatives. The alternatives were examined based on initial capital costs, operation and maintenance (O&M) expenses, and salvage valve. The alternative with the lowest life cycle cost was chosen for each item. A summary of the Phase 1 improvements is described as follows:

- Provide two new mechanical bar screens.
- Construct new influent pump station.
- Add new grit removal system.
- Update influent flow metering.
- Replace existing secondary treatment system with new sequencing batch reactors (SBR).
- Replace existing tertiary filter with new tertiary filter system.
- Secure an effluent storage pond by either acquiring an existing facility or by constructing a new pond.
- Convert existing secondary treatment units to biosolids digesters and improve mixers.
- Add biosolids process facility for dewatering and storage of dried sludge.

ES.5 Recommendations and Costs

The recommended projects for improving the city's existing collection system and WWTP are summarized in Table ES.5.1. Capital improvements cost for the Everett Avenue Pump Station is $925,000 and, for the WWTP plant, it is $20,317,000, for a total of $21,242,000.

TABLE ES.5.1
WWTP & WW SYSTEM IMPROVEMENTS COST ESTIMATE

Item	Cost
Influent Pump Station	$ 816,000
Headworks with Screen	$ 1,247,000
SBR	$ 4,487,000
Tertiary Filtration	$ 1,004,000
UV Disinfection	$ 529,000
Recycled Water Disinfection System	$ 250,000
Convert Existing Tanks to Digestions	$ 775,000
Biosolids Processing Facility	$ 1,160,000
Site Structures	$ 555,000
Recycled Water Storage Pond	$ 742,000
Miscellaneous Equipment	$ 380,000
Site Improvements and Yard Piping	$ 723,000
Basic WWTP Construction Total	**$ 12,668,000**
Engineering – Design – Bidding Services	$ 1,267,000
Engineering – Construction Services	$ 1,267,000
Value Engineering	$ 85,000
Contingency 15%	$ 1,900,000
Environmental Report	$ 45,000
Fords Pond Land Acquisition	$ 3,000,000
Review Fees	$ 10,000
Admin / Legal	$ 75,000
WWTP Project Total	**$ 20,317,000**
Everett Avenue Pump Station Improvements	$ 925,000
Project Total	**$ 21,242,000**

ES.6 Finance

With any of the proposed funding sources contained in the financial strategy, the city is advised to confirm specific funding amounts with the appropriate funding agencies prior to making local financing arrangements. It is recommended that the city undertake efforts to secure funding in the form of grants and low interest loans. To achieve this goal, a "One-Stop" meeting with funding agencies is recommended when the WWFP is approved and the city has a time-line commitment for the proposed project.

Most federal grant programs require that the project address a DEQ issued violation or order and meet or exceed a predetermined user fee before the project is eligible for grant funding. Rural Development looks closely at sewer user rates and expects local rates to be at or above that of similar communities before the project becomes eligible for grants.

It is recommended that the City undertake efforts to secure funding in the form of grants and loans. Sewer collection fees for the 2013 budget year were $33.50 per single family residential unit and 3,210 EDUs. Table ES.6.1 presents various potential funding scenarios that were discussed at a one-stop meeting in Salem Oregon Janusry14, 2014. These scenarios show the various terms associated with several potential funding sources. Table ES.6.2 provides the anticipated rate for each existing rate type for each funding scenario. Changes to the City's current rate structure may be required and a sewer rate study may be necessary in order to clarify the rate structure and to ensure that system costs are fairly allocated between each rate type.

TABLE ES.6.1
POTENTIAL WWTP PROJECT FUNDING SOURCES

Project Description	Cost		Funding Source	
Everett Avenue Pump Station	$925,000			
WWTP Expansion	$20,317,000			
Total	**$21,242,000**		See Options Below	
	Loan Amount	**Grant**	**Years**	**Interest Rate**
Scenario #1: IFA Loan & RD				
RD	$12,000,000		40	2.625%
IFA - WW	$9,492,000	$300,000	25	3.180%
Scenario #2: DEQ & IFA				
DEQ - CWSRF	$18,242,000		30	2.390%
IFA – WW	$3,000,000	$300,000	25	3.180%
Scenario #3: DEQ & IFA				
IFA – WW	$3,000,000		25	3.180%
IFA – WW	$7,700,000	$300,000	25	3.180%
DEQ - CWSRF	$10,242,000		30	2.390%

User Rates

This discussion will consider the least costly potential alternative. Once the City has determined what funding is available, the current rate structure should be revised to determine the actual impact to ratepayers. All grants, loans, existing debts and reserves, and surpluses should be taken into account when calculating the final impact to ratepayers.

TABLE ES.6.2
WWTP EXPANSION POTENTIAL USER RATES

Source of Funding	Loan Cost/EDU per Month	O & M Cost/EDU per Month	Required Min. Rate per Month
Scenario #1: IFA Loan & RD	$ 25.33	$ 33.64	$ 58.96
Scenario #2: DEQ & IFA	$ 25.45	$ 33.64	$ 59.09
Scenario #3: DEQ & IFA	$ 27.75	$ 33.64	$ 61.39

With any of the proposed funding sources within the financial strategy, the City is advised to confirm specific funding amounts with the appropriate funding agencies prior to making local financing arrangements. Sutherlin may qualify for significant grant funding. A one-stop meeting with funding agencies is recommended as soon as the City has made a firm commitment as to schedule and the extent of capital improvements. The total anticipated sewer cost per EDU is dependent on the amount of grant funding available. User rates may vary from $59 per EDU per month to $61.39 per EDU per month.

Introduction

Section

1

Section 1

Introduction

1.1 Background

The city of Sutherlin has operated a public wastewater system since 1977 when the existing wastewater system was constructed. Numerous extensions, additions, upgrades, and improvements have brought the wastewater system to its current configuration. At this time, the wastewater system includes pipe of various sizes and materials in the collection system, five raw sewage pump stations, and a wastewater treatment plant providing primary and secondary level treatment. During winter months, treated effluent is discharged to Calapooya Creek. During the summer months effluent is land applied at the nearby Umpqua Golf Resort.

At the outset of this study, the wastewater system's major problems were identified. These include: 1. high inflow and infiltration flowing into the collection system; 2. the existing treatment plant operating at full capacity; and 3. summertime disposal capacity exceeding limits. During winter months, high infiltration and inflow continue to tax the collection system. As the population increases the city does not have adequate treatment plant capacity for the 20-year planning period. The city does not have enough land with sufficient storage capacity to irrigate the amounts of recycled water. Consequently, during summer months, reclaimed water has overflowed from the golf course to Cook Creek.

To allow for growth and to provide an adequate wastewater infrastructure capable of complying with the current and anticipated pollution discharge limitations established for Calapooya Creek, the City has undertaken this Wastewater Facilities Plan, which will lead to the development of a Wastewater Treatment Facility that will meet the needs of the community and regulatory requirements.

1.2 Objectives

The overall objectives of the Plan are to:

- Evaluate the condition of the existing collection system and assess its capacity while identifying current system deficiencies;
- Estimate current and projected wastewater flows from within the existing Urban Growth Boundary (UGB) as well as current and projected wastewater flows required by future expansion of the UGB;
- Develop potential wastewater collection system improvements to correct existing problems and to serve existing and future development within the City limits consistent with DEQ regulations and requirements;
- Fulfill the engineering and planning document requirements of the DEQ Clean Water State Revolving Fund, Oregon Business Development's Infrastructure Finance Authority, and the U.S.D.A Rural Development;

- Provide cost estimates and phasing recommendations for the recommended improvements;

- Provide recommendations for improvements to the wastewater treatment facility that will, once in operation, meet the conditions of the effluent discharge permit, comply with total maximum daily load (TMDL) restrictions and provide treatment capacity for future growth.

1.3 Scope of Study

The scope of the Sutherlin Wastewater Facilities Plan (WWFP) is intended to address problems and deficiencies at the WWTP, and to comply with the applicable requirements of the State of Oregon's Department of Environmental Quality (DEQ) and its State Revolving Fund (SRF) program.

Study area characteristics were identified; they include both physical and socioeconomic conditions. City population and land use are addressed and projected into the future.

E**xisting wastewater facilities** were investigated in detail. Data was collected on the existing wastewater collection and treatment systems from such sources as operating records, conversations with City staff, on-site investigation, maps, as-built records and other pertinent documentation. Existing facilities were evaluated in terms of location, sizing, capacity, condition, limitations, and performance. Consideration was given to the manner in which existing facilities could be utilized in the future. The infiltration and inflow (I/I) contribution to the wastewater flow was evaluated based on past and recent I/I investigations and historic plant operating data.

Wastewater characteristics were identified in terms of loads, flows, and strength during various times of the year. Future characteristics were projected to establish capacity requirements. Flows were addressed for both dry period and wet period conditions, and unit design values were established. Future wastewater characteristics were projected.

Basis for planning was established. Applicable regulatory requirements were identified and addressed, including the City's Mutual Agreement and Order (MAO) with Oregon DEQ, management plans, current and future treatment criteria, and discharge standards. The present design capacity of the City's conveyance system and treatment plant was estimated to assess the present and future operation of wastewater facilities.

Alternatives were identified for conveyance and treatment. Unfeasible options were screened out, and a limited number of selected alternatives were established and evaluated in detail.

A recommends plan was formulated which will enable the City to meet the present and future demands and requirements of their wastewater facilities. This plan includes preliminary design data, capital improvement and operational costs, improvements to be completed in two phases, a project schedule, and a financing strategy.

1.4 Previous Studies and Information

The following studies, reports and other sources of information have been used in preparation of this Facilities Plan:

- *Comprehensive Plan for the City of Sutherlin*, September 1990 - 1991, The City of Sutherlin and The Umpqua Regional Council of Governments.

- *Sutherlin Wastewater Treatment Facility, Reclaimed Water Reuse Plan,* Brown and Caldwell, September 30, 2003.

- *Sutherlin Sewage Treatment Plant Construction Record Drawings,* HGE Engineers, February 1977.

- USGS 7.5 Minute (Topographic) Quadrangle Maps - Sutherlin, 1984 Provisional Edition.

- *Water, Wastewater, and Stormwater Engineering Study, Part III Wastewater,* City of Sutherlin, HGE, Inc., 1997.
- Infrastructure and City Mapping Files.

- Discharge Monitoring Reports for City of Sutherlin Wastewater Treatment Plant (2007-09).

- *Sludge Monitoring Report for City of Sutherlin*, Umpqua Research Company, 2002 to 2004.

Proposed improvements discussed in this Wastewater Facilities Plan are based on the limited amount of topographic information that was available at the time of this study. Locations of pump stations and the extent of gravity flow sewers may change when more accurate topographical information is made available. The information in this Plan is for preliminary planning and budgeting purposes. Detailed surveys and elevation information must precede design, and some changes from this Plan are anticipated.

1.5 Authorization

The city of Sutherlin authorized The Dyer Partnership Engineers & Planners, Inc. to proceed with this Wastewater Facilities Plan in March 2013. Services are provided in accordance with a Professional Services Agreement between the City and The Dyer Partnership.

1.6 Acknowledgments

This plan is the result of contributions made by a number of individuals and agencies. We wish to acknowledge the efforts of Denny Cameron, Mayor; the City Council of Sutherlin; Jerry Gillham, City Manager; Community Development Director Vickie Luther; and Brian Elliott, Wastewater Treatment Plant Supervisor. The assistance of the city of Sutherlin office staff was invaluable in compiling information on City services and the community.

Section

2

Study Area Characteristics

Section

2

Study Area Characteristics

2.1 Study Area

The city of Sutherlin is north of Roseburg in Douglas County, near Interstate 5. Figure 2.1.1 illustrates the location of the City.

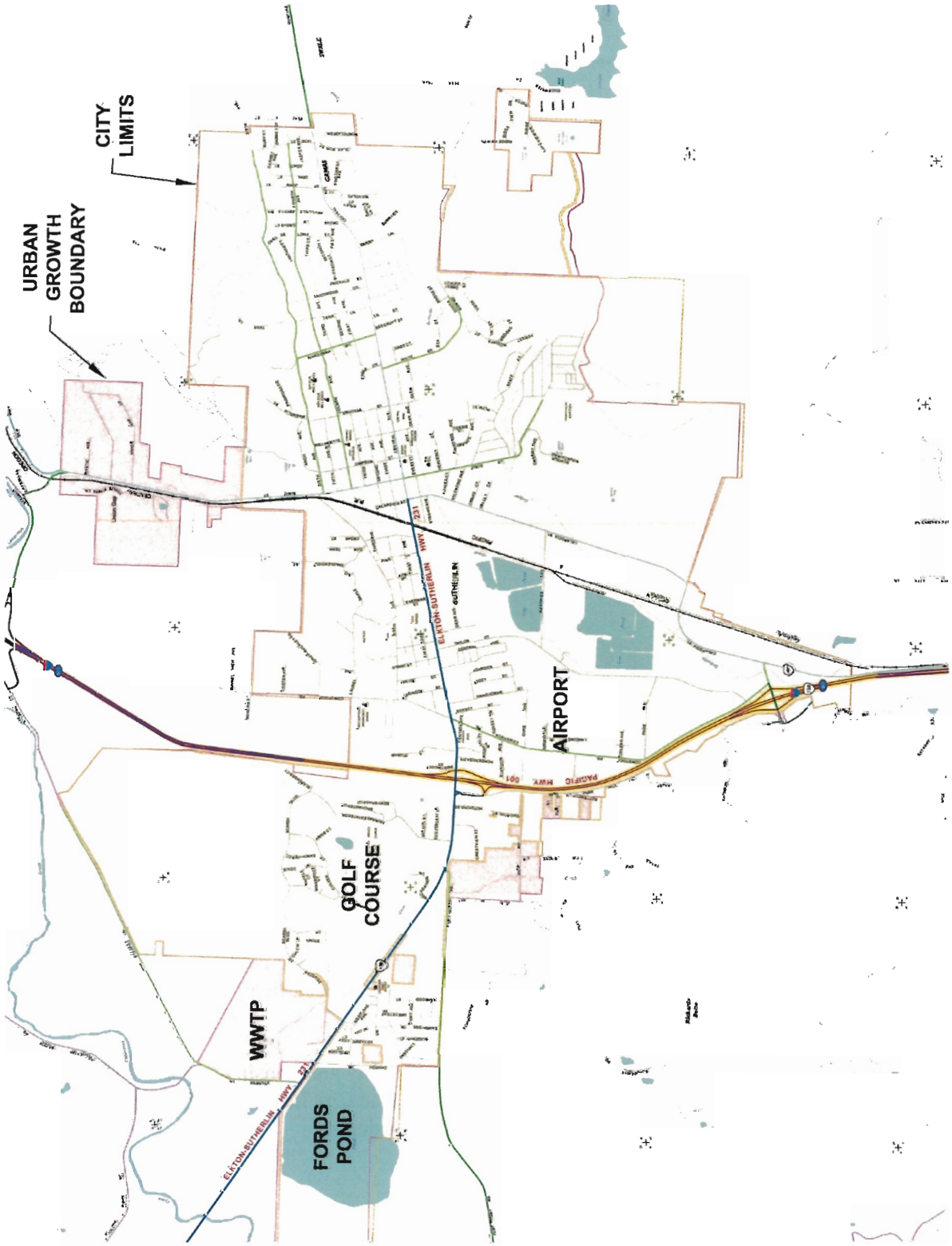
FIGURE 2.1.1
CITY LOCATION



Most of the community lies east of Interstate 5, with newer residential areas and the wastewater treatment plant located west of the Interstate. Highway 138 intersects Interstate 5 and continues into the City as Central Avenue. The Umpqua River, Cooper Creek and Calapooya Creek lie in close proximity to the City. The study area is illustrated in Figure 2.1.2

Sutherlin continues to attract retirement residents with its low utility and housing costs, and close proximity to shopping and outdoor recreational opportunities. The City has made efforts to recruit new industry to the area, with incentives similar to those given to residents.

Figure 2.1.2
CITY OF SUTHERLIN STUDY AREA



The area within the city limits is approximately 3,259 acres or over 5 square miles. The study area for this Facilities Plan is within the city limits and the Urban Growth Boundary (UGB). Wastewater service in Sutherlin is limited to the Sutherlin UGB, unlike the water system, which includes services outside of the UGB. Union Gap Sewer District constructed a system in the late 1980s. This District is outside the city limits but within the UGB, and it maintains a septic tank effluent pumped (STEP) system, which discharges to the City's collection system. With the exception of Union Gap Sanitary District, there are no other wastewater customers outside the city limits.

2.2 Physical Environment

The following summarizes information about the physical environment in and around the city of Sutherlin.

Climate

The climate of Sutherlin is moist in the wet season and dry in the summer season. Temperatures average 41° F in January and 68° F in August. The yearly mean temperature is approximately 54° F. The average low temperature is 34° F, while the average high temperature is 84° F. Extreme temperatures range from 5 to 106° F. Sutherlin experiences prevailing winds year-round, which average approximately 7 miles per hour.

Sutherlin receives an average of about 40 inches of precipitation per year. Nearly all precipitation occurs as rainfall, with the majority (approximately 77%) falling between the months of November and April. The wettest month is December with a historic average of approximately 6.5 inches of rainfall. The driest month is July with an historic average of approximately 0.57 inches of rainfall. Records show that the maximum 24-hour rainfall is 2.5 inches.

Geologic Hazards

There are several areas within Sutherlin that are susceptible to geologic hazards. These hazards include river flooding, earthquakes, high groundwater and erosion. A discussion of each hazard and its expected locations are discussed below. Specific hazard maps are included in Appendix B.

- **River Flooding.** The Federal Emergency Management Agency (FEMA) has declared the City of Sutherlin a "No Special Flood Hazard Area". All areas within the UGB have been designated Zone C, areas of minimal flood hazard (FEMA, 2005), (Moran, 2005). The Wastewater Treatment Plant (WWTP) is located near Calapooya Creek. Upstream of the State Highway 138 bridge, Calapooya Creek has a 100-year flood plain elevation of 390.9 NGVD (393.9 NAV 88) and 500-year flood plane of 392.6 NGVD (395.6 NAV 88). The WWTP site's finish grade averages 396.0. Refer to Appendix B for Calapooya Creek Basin flood profiles and FIRM Map Number 41019C0990F.

- **Earthquakes.** Earthquakes are the products of deep-seated geologic faulting and the subsequent release of large amounts of energy. Earthquake hazard maps (Oregon Department of Geology and Mineral Industries, 2000) located in Appendix B summarize the relative earthquake hazard. The relative earthquake hazard includes factors such as earthquake-induced landslides, liquefaction and shaking amplification. As summarized in the maps, areas of relatively steep slope that reside directly north, south and southeast of town present the greatest relative hazard.

- **High Groundwater.** High groundwater is apparent in specific areas within the Sutherlin UGB. This water may be due to land contours, springs, hillside seepage, or saturated soil conditions following periods of wet weather.

Public Health Hazards

Most of the development within the UGB is connected to the existing sanitary sewer system; a small percentage of the current population is served by septic tank systems. These locations are in the furthest southern reaches of the City limits. There are no known ongoing problems with on-site systems within the UGB. Since soils within Sutherlin consist mainly of heavy clay, shale and hard rock, drainage is poor. These soils are not well-suited to on-site septic systems.

Sutherlin has a history of frequent minor sewage spills from specific locations, usually caused by storm events. Table 2.2.1. summarizes these locations and their characteristics. Everett Avenue Pump Station is a frequent problem during the rainy season in that it cannot maintain low wet well levels.

Inflow and infiltration (I/I) is a major contributor to sewage spills. Current operation and maintenance practices have worked to reduce the frequency of these spills. See Section 4 – Infiltration and Inflow for a more detailed discussion on reduction efforts.

TABLE 2.2.1
KNOWN OVERFLOW & BYPASS LOCATIONS

Location	Description	Frequency
Camas Court (Will be corrected after improvement of Everett St. Pump Station within next 3 years)	Specific manholes surcharge and overflow when Everett St. Pump Station lags behind demand. Overflows enter storm system, which discharges into Sutherlin Creek.	Each year at least once since construction. These occur at Manholes O7, M7, M17 and M18.
Sunset Ave. & Comstock Rd. (This was repaired by pipe burst replacement recently.)	Obstruction causing manhole surcharge and overflow. Overflows enter storm system, which discharges into Sutherlin Creek.	Every few years (summer and winter).
Wastewater Treatment Plant	Catch basins near plant. Short duration flows cause puddling when influent pump lags behind demand.	Several times annually.
Wastewater Treatment Plant	Catch basins near plant. Longer duration flows cause overflow and discharge into Cook Creek when influent pump lags behind demand.	Once annually.

Water Resources

The city of Sutherlin has access to water resources from Cooper Creek, Calapooya Creek and the Umpqua River. Water rights exist for each of these resources. The City's municipal water supply comes from upper Calapooya Creek at Nonpareil and from impounded water from Cooper Creek Reservoir. The city of Sutherlin utilizes a surface water outfall for the wastewater treatment plant. The City holds an NPDES permit for the discharge of treated wastewater to Calapooya Creek during the winter months.

During the summer months, the City pumps treated wastewater effluent to the golf course holding pond, which is used for irrigation.

Flora and Fauna

Vegetation, fish and wildlife within the Sutherlin area are typical of inland river valley regions of Oregon. The river, wetlands, terraces and mountains contain a wide variety of plant and animal species.

Forestlands support mainly native Douglas fir, big leaf maple, western red cedar, vine maple, spruce, and many smaller plants. Riparian vegetation consists of blackberries and several species of ferns.

The common species of fish in the river are anadromous fish species, including summer and winter steelhead, fall and spring chinook salmon, coho salmon, and sea-run cutthroat trout. Wetlands and floodplains provide habitat for many waterfowl, including mallard, pintail, widgeon, coot, ruddy duck, canvasback, green-winged teal, gadwall, redhead, ring-necked duck, scaup, and merganser. Other animals found in the study area include beaver, muskrat, river otter, raccoon, mink, skunk, squirrel, deer, elk and bear.

The riparian communities act as important buffers for water users and urban development. They are important to wildlife for shelter, food, and ecosystem diversity. The clearing of vegetation causes considerable effects on the diversity and stability of the ecosystem of an area. Removal can also bring about the loss of a significant ecotone (transition between water-related environments and upland areas). A large amount of information is available on the distribution of fish in the Umpqua River watershed, although most fish data pertains to the salmonoids. Much of this information is available on-line through the Oregon Department of Fish and Wildlife (ODFW), the U.S. Forest Service (USFS) and the Bureau of Land Management (BLM).

Environmentally Sensitive Areas

Sutherlin not only lies near sensitive environmental areas, but also affects those downstream. The combination of forests, rangeland, pasture and other wetlands provides a unique surrounding for the City of Sutherlin that should be considered and protected in facilities planning. A discussion of environmentally sensitive areas and environmental topics pertinent to public facilities planning is presented below:

Wetlands. Wetland areas, as determined by the U.S. Department of Fish and Wildlife, are shown in Appendix B.

Riparian Zones. At the writing of this WWFP, a map of specific riparian vegetation corridors in the City was not available. Although riparian vegetation varies in width depending upon the location, the average width of riparian vegetation in Douglas County is estimated to be 50 feet (Douglas County, 1997). For the purposes of this study, all riparian vegetation located within 50 feet of the stream bank adjacent to identified perennial and intermittent streams shall be considered important.

Air Quality and Noise

With Sutherlin's favorable prevailing winds, low population with corresponding low auto emissions, and absence of heavy industrial development result in few air quality problems, there are no problems with air pollution. Sutherlin is not in an Air Quality Management Area. Noise levels within the area are quite

low, except near Interstate 5. Automobile and truck traffic along Interstate 5 would likely be the source of any future air quality or noise problems in the City.

Energy Production and Consumption

No major energy resources have been identified in the study area. Energy consumption is expected to increase within the study area due to population growth during the planning period. Pacific Power serves the study area with electrical energy.

Endangered Species Act

No known threatened or endangered species reside within the study area. A number of listed threatened and endangered species are known to occur outside the study area in Douglas County. The projects proposed in this study will not knowingly harm or threaten any species protected under the Endangered Species Act.

Wild and Scenic River System

There are no designated Wild and Scenic Rivers within the study area.

2.3 Socioeconomic Environment

Population

Since 1990 Sutherlin has experienced a growth rate higher than most other communities in Oregon. Economic conditions were difficult in the early 1980s due to the decline of the forest products industry, and some uncertainty remains over the long-term sustainability of the timber industry. However, Sutherlin's livability characteristics, nearness to the I-5 corridor and outdoor recreational opportunities make Sutherlin attractive, and have resulted in a growing long-term population.

The updated coordinated population projection of 1.5% per year has been recommended by Douglas County and the State of Oregon Department of Land Conservation and Development (DLCD) for the next 25 years (to the year 2037). See Appendix D for the population projections and agency approvals. Historically, the city of Sutherlin has experienced considerable variation in growth at a rate than exceeds the Douglas County growth rate. The City's annual growth rate in each of the past four decades has averaged from 0.92 percent to 2.47 percent. A comparison of Douglas County's historical population growth to the City's historical growth rate is presented in Table 2.3.1.

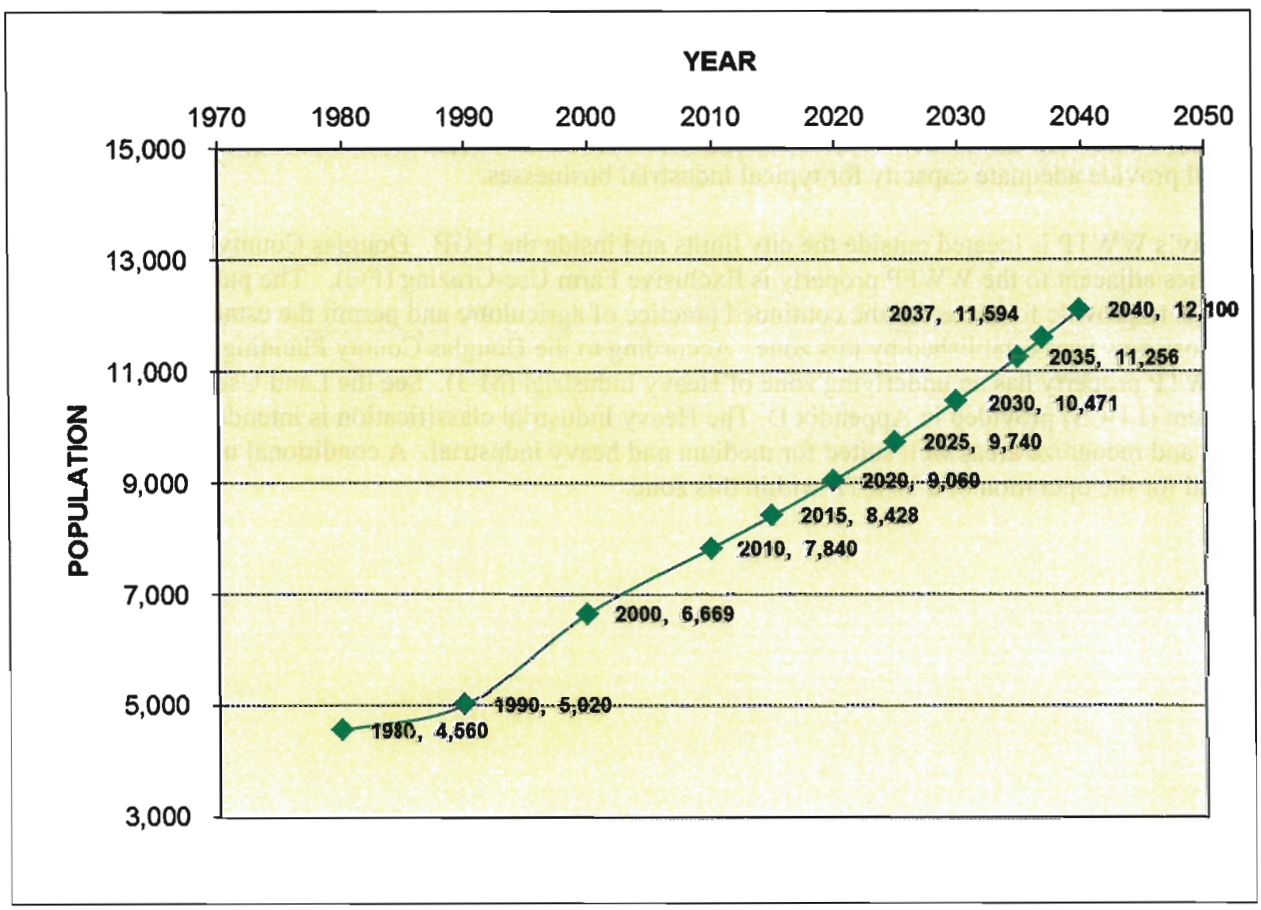
TABLE 2.3.1
HISTORICAL POPULATION GROWTH

Time frame	Douglas County Annual Growth	Sutherlin Annual Growth
1980-90	0.14%	0.92%
1990-2000	0.55%	2.47%
2000-2010	0.70%	1.50%

Projected population growth is an extremely important consideration, as it is the basis for determining future water demand and the associated capacity and cost of the City's wastewater system. Predicting the City's future population is difficult due to the many factors and uncertainties involved. If the population within the study period is underestimated, the additional growth and associated demand may require the City to install additional capital improvements sooner than expected or desired. Likewise, if the population is overestimated, then some facilities may have more capacity than is required for satisfactory operation, and, in extreme cases, may sit idle. Many projections are based upon the extrapolation of past trends from an area or region. While history may not repeat itself, past trends serve as an initial basis for population projections.

The Oregon Department of Environmental Quality (DEQ) recommends that design flows and loads be based on 20 years from the expected initiation date of the proposed project operations. The anticipated time needed to complete collection system improvements, reassess influent flows, secure financing, and complete design and construction is five years from the base year (2012). The recommended planning period is therefore 25 years. Rounded to the nearest 5-year period, the recommended planning period is shown to the year 2037. Figure 2.3.1 represents the historic and projected population growth for the city of Sutherlin.

FIGURE 2.3.1
HISTORIC AND PROJECTED POPULATION GROWTH



Sutherlin's 2012 population estimate is 7,905 as determined by Portland State University for Population Research.

Land Use

Land use within Sutherlin city limits is categorized into four general uses: residential, commercial, industrial and public facilities. There is an estimated 3,259 acres within the current city limits. The Sutherlin zoning map is shown as Figure 2.3.2. Four land use classifications are briefly discussed below:

- **Residential Lands.** Sutherlin residential lands are distributed throughout the community and on either side of Interstate 5. Residential lands also occupy the elevated surrounding hills on the north side of the UGB. New subdivisions are being constructed in the areas surrounding the city. Residential land uses include single-family dwellings, multi-family dwellings, bed and breakfast and motel land uses.

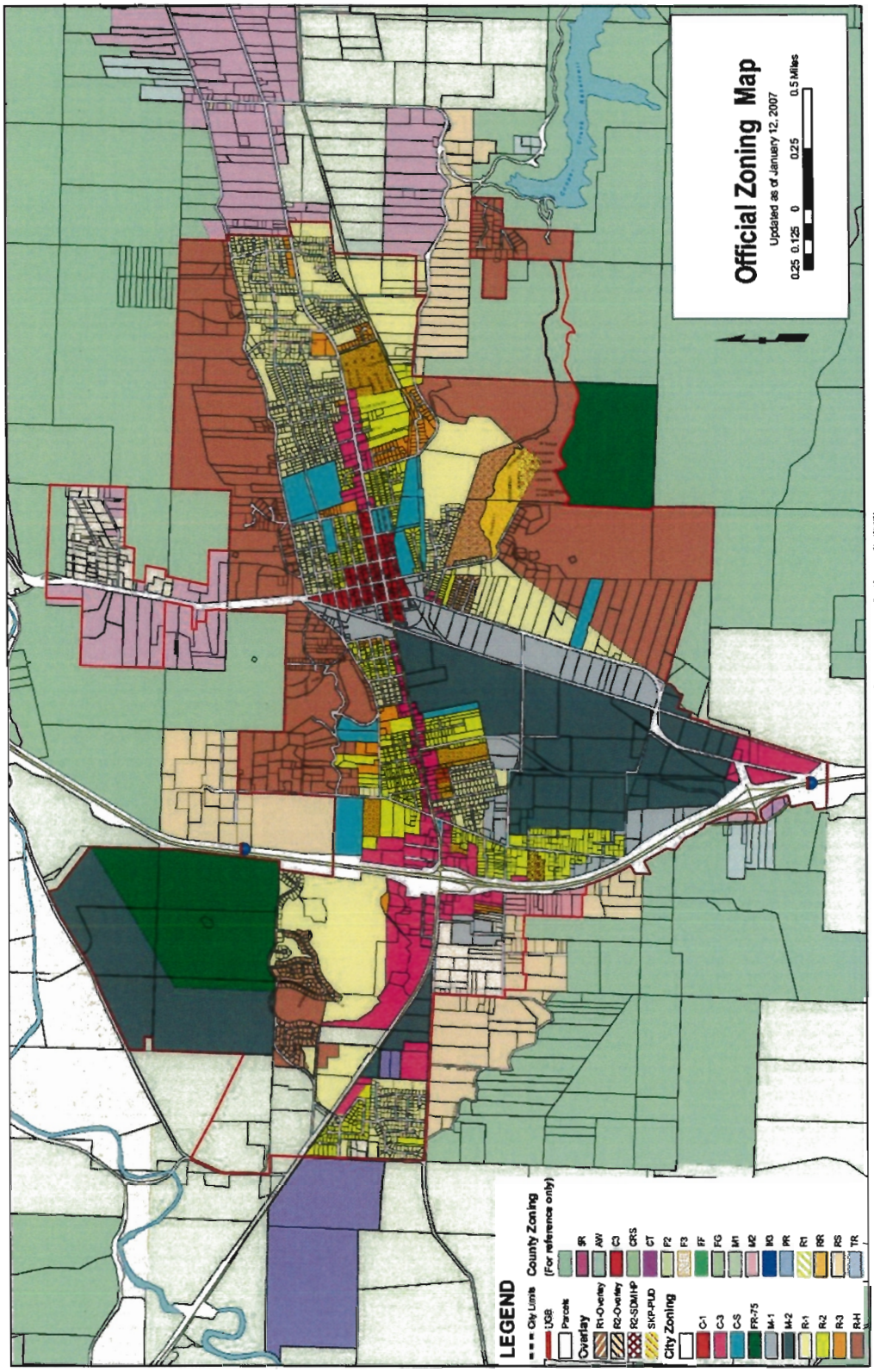
- **Commercial Lands.** The majority of commercial land use areas are adjacent to Highway 138 and Central Avenue. Commercial activities generally include retail and tourist-related services. Small shops and restaurants catering to the tourist market make up the majority of the commercial properties in the City.

- **Public Facilities Lands.** Public lands consist of those required for government offices, schools, hospital, transportation facilities, parks, and recreation areas. The wastewater treatment plant and city shops are included within the public facilities lands.

- **Industrial Park.** Industrial lands exist in multiple locations throughout the City, but are primarily located west and south of the City. This WWTP includes recommendations for improvements that will provide adequate capacity for typical industrial businesses.

The City's WWTP is located outside the city limits and inside the UGB. Douglas County's zoning for properties adjacent to the WWFP property is Exclusive Farm Use-Grazing (FG). The purpose of the FG zoning is to provide for areas for the continued practice of agriculture and permit the establishment of only those new uses established by this zone. According to the Douglas County Planning Department, the WWTP property has an underlying zone of Heavy Industrial (M-3). See the Land Use Compatibility Statement (LUCS) provided in Appendix D. The Heavy Industrial classification is intended to provide, protect and recognize areas well suited for medium and heavy industrial. A conditional use permit is not required for the operation of a WWTP within this zone.

Figure 2.3.2
CITY OF SUTHERLIN ZONING MAP



Section

3

Existing Wastewater Facilities

Existing Wastewater Facilities

Section 3

3.1 System History

The city of Sutherlin conventional activated sludge wastewater plant was originally constructed in 1956 for a design population of 3,500 people. Prior to this time, treatment was by individual septic tank and drain field systems. Percolation rates are poor in the Sutherlin area of the Umpqua Valley due to the preponderance of heavy clays, shale and rock. Because of population growth, it was necessary to construct a new treatment facility in 1977 for a design population of 8,000 people. The existing wastewater facility cannot meet the treatment requirements even for the existing population, especially since the new regulatory requirements have been implemented by the EPA since its construction. The current wastewater treatment plant, constructed in 1977, is located near the intersection of Highway 138 and Stearns Lane. The plant consists of an activated sludge process operated in contact stabilization mode. Arrangements were made to pump treated effluent during the summer to a private golf course holding pond. The treated effluent is used to irrigate the golf course.

Since the plant's construction the collections system has been expanded. In the late 1980s Union Gap Sanitary District, which is located outside of the city limits but within the UGB, constructed a Septic Tank Effluent Pump (STEP) system which discharges to Sutherlin's gravity collection system. A section of trunk line along Everett Avenue adjacent to Sutherlin Creek was found to be contributing about 30% of the infiltration measured during flow mapping in March 1995. A 15-inch section of this line between Calapooia Street and Umpqua Street was replaced in 1997. More recent collection system improvements include: Page Avenue Pump Station; a 15-inch gravity line in Taylor Road; a 4-inch diameter low pressure force main collection system (STEP) in Hastings Avenue and Calapooia Street; replacement of the Airport Pump Station; and upgrades to the Everett Avenue Pump Station.

Ford's Pond is a nearby, privately owned water body that has the potential for recycled water (effluent) storage. Summer effluent could be stored in the pond and then discharged into the permitted receiving stream per the conditions of the NPDES permit. Adjacent land owners have expressed an interest in the effluent's irrigation potential. The Army Corp of Engineers (ACE) indicated the pond would not be regulated, provided no changes were made to the existing pond outfall.

The wastewater system is operated under Waste Discharge Permit No. 101993 from the National Pollutant Discharge Elimination System, a copy of which is included in Appendix A. The system has experienced raw sewage overflows and has received Notice of Noncompliance letters from DEQ for exceeding the allowed effluent concentration levels. Sanitary Sewer Overflows (SSO) in Manhole Numbers O7, M7, M17 and M18 commonly occur during periods of high ground water and heavy rain. The City has entered into an MAO (See Appendix A.) with DEQ for non-permitted summertime discharge. The MAO established a schedule and interim compliance level for operating the WWTP while working to correct the system deficiencies.

The following sections include more detailed descriptions and analyses of each component of the wastewater system, including capacity, performance, and operation and maintenance issues.

3.2 Wastewater Conveyance System

Collection System Description

The Sutherlin wastewater conveyance system currently consists of approximately 141,000 feet (27 miles) of gravity sewer pipe (6 to 27-inch), an estimated 700 manholes, and 8,770 feet of pressure piping (4 and 10-inch). The system also has five lift stations (Everett Avenue, Church Road, Airport, Page Street and Quail Run). The piping system inventory is presented in Table 3.2.1.

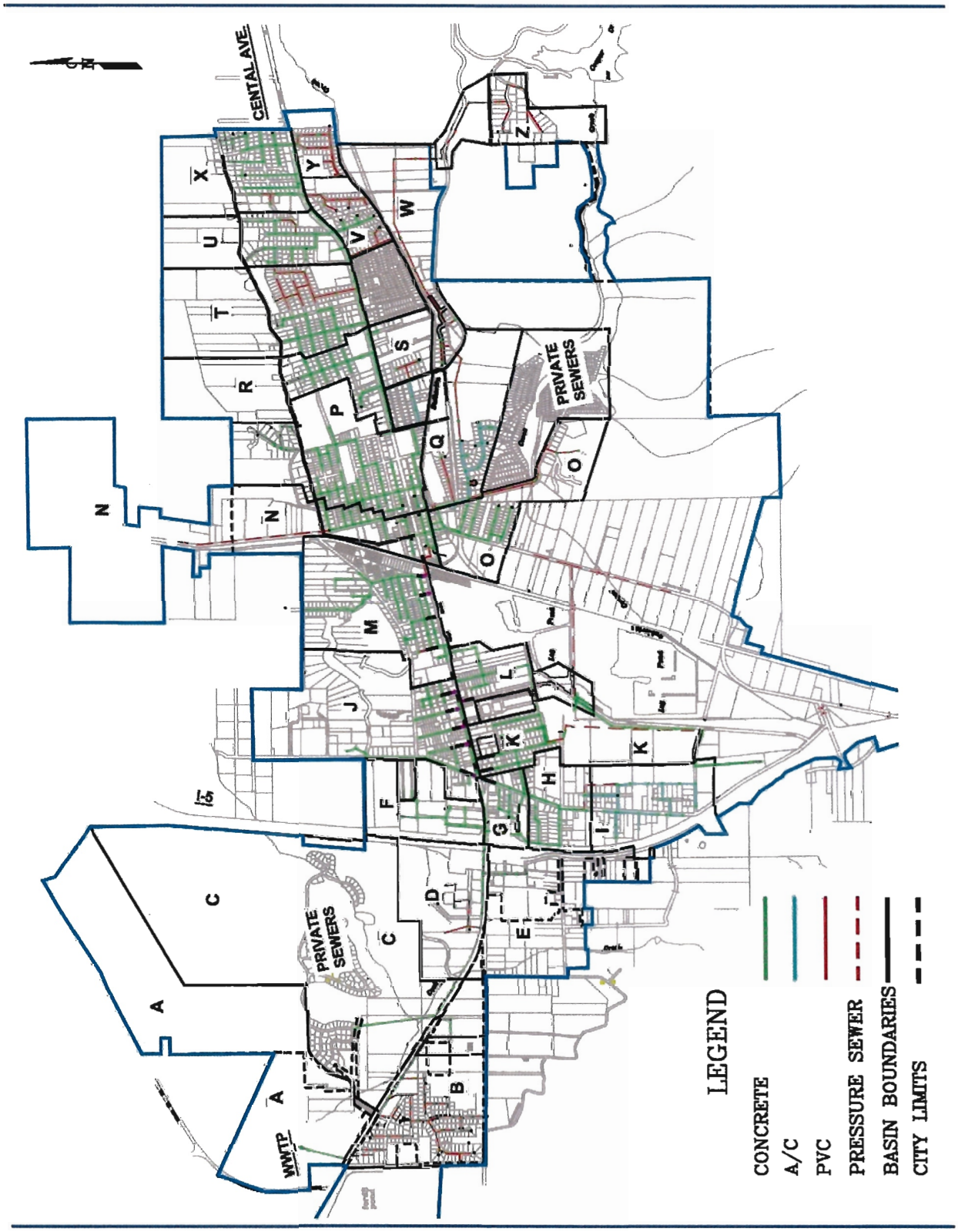
TABLE 3.2.1
WASTEWATER CONVEYANCE SYSTEM INVENTORY

Pipe Diameter	Orig. System Const. 1956	Total as of 1975	Total as of 2012	% of Total	Material Type
Gravity Lines					
6	0	0	1,190	1%	CP, AC, PVC, OB
8	36,192	51,915	100,255	71%	CP, AC, PVC
10	1,350	2,700	2,900	2%	CP, AC, PVC
12	7,043	9,880	17,210	12%	CP, AC, PVC
15	4,962	4,962	5,950	4%	CP, AC, PVC
18	4,474	4,474	5,414	4%	CP, AC
21	0	0	3,225	2%	CP, AC
24	0	0	3,750	3%	CP
27	0	0	1,100	1%	CP
Total	54,021	73,931	140,994	100%	
Force Mains					
2	0	0	1,014	7%	PVC
4	0	0	1,530	10%	PVC
4 (STEP)	0	0	5,290	35%	PVC
6	0	0	3,175	21%	PVC
10	890	890	4,065	27%	PVC, CI
Total	890	890	15,074	100%	

The collection system was largely constructed in 1956 with a new treatment plant designed for 3,500 people. Prior to this time, wastewater was treated by individual septic tanks and drain fields. Original construction consisted of Orangeburg pipe (OB) possibly without joint seals, concrete pipe (CP) with mortar joints and cast iron (CI) pipe. Construction from the1960s to the 1970s consisted mostly of concrete and asbestos-cement (AC) pipe with rubber gaskets. Since then, sewer construction has consisted primarily of PVC pipe. Most of the piping in the collection system is 8-inch. Approximately half of this is either concrete or asbestos-cement piping. Some of the collection system utilizes STEP systems, which is a low head pressure collection system. A 4-inch diameter STEP force main has been constructed in South Calapooia Street. Multiple-house private laterals utilize STEP systems as well. STEP system force mains and laterals discharge to city gravity mains. As part of the Facilities Plan, the collection system was separated into 26 distinct subbasins based upon areas of gravity drainage. A plan of the collection system and subbasins is shown in Figure 3.2.1. A complete set of collections systems maps is located in Appendix B.

The overall hydraulic capacity of the collection system is greater than the peak flow rates, with the exception of the line immediately downstream of the Everett Avenue Pump Station. In peak flow conditions, the force main discharge can cause the receiving line to surcharge. See the Everett Avenue Pump Station and Force Main Evaluation in Appendix E. The main trunk line leading to the WWTP has a flow capacity greater than 10 MGD.

FIGURE 3.2.1
COLLECTION SYSTEM



A brief discussion of each subbasin is as follows:

Subbasin A

Subbasin A is located in the northwestern portion of the City. At this time, the wastewater treatment plant is the only user in this subbasin.

All other subbasins flow into this subbasin via a 27-inch concrete gravity line which flows into the treatment plant. The subbasin has an area of about 127 acres.

Subbasin B

Subbasin B is located on the west side of town, south of Highway 138 and west of Highway Haven (which has a private system), north of Kurt McKay Road and east of Church Road. The western portion of the subbasin contains the Church Road Pump Station that receives flow from Manhole (MH) B-30 and discharges via two 2-inch lines to MH B-33 to the north.

The system generally consists of 8-inch PVC line. This subbasin discharges into the main 24-inch trunk line along Highway 138 in five locations. No other subbasins discharge through this subbasin. The subbasin has an area of about 124.7 acres.

Flow mapping investigations conducted in March 2005 indicate a leaking manhole at MH B-9 and pipe leakage between Manholes B-22 and B-23 ranging from 11 to 15 gallons per minute (gpm). Smoke testing completed in October 2012 identified main lines located in the ditch flow line which may be receiving inflow during high runoff events.

Subbasin C

Subbasin C is located on the north side of Highway 138. The subbasin serves a residential STEP system. Discharge into the 24-foot trunk line along Highway 138 occurs only at MH C-1. The subbasin has an area of about 230 acres.

Subbasin D

Subbasin D is located on the north side of Highway 138 and west of Interstate 5. The northern portion of the subbasin includes the Oak Hills Golf Club. Eight-inch PVC collection piping carries flow to the 21-inch trunk line along Highway 138 at MH D-4. The subbasin has an area of about 89.6 acres.

Subbasin E

Subbasin E is located on the south side of Highway 138 and the west side of Interstate 5. There is currently only a single concrete line draining this subbasin on Hutchins Way. This line drains into the 21-inch main trunk line on Highway 138 at MH D-14. The subbasin has an area of about 123.0 acres.

Subbasin F

Subbasin F is located on the north side of Highway 138 and east of Interstate 5. It is bounded on the east by N.W. Comstock Road. The northern limit boundary is the city limit, just north of West School. A private collection system located just south of West School drains into the upper reaches of the system at

MH F-8. The system consists of 8-inch diameter concrete pipe, which drains into a branch of the main trunk line on East Central Avenue at MH G-17. The subbasin has an area of about 68.6 acres.

Subbasin G

Subbasin G is located on the south side of East Central Avenue and east of Interstate 5. The southern boundary lies between Hospitality Way and Gleason Avenue. The eastern boundary lies west of Sunset Avenue. The main 24-inch diameter trunk line runs through this subbasin, bringing flow from Subbasin K. All other lines consist of 8-inch and 10-inch sewers. All sewer lines are concrete. Subbasin G receives flow from Subbasin H via a 10-inch gravity line at MH G-3 and from an 8-inch gravity line at MH G-13. The subbasin has an area of about 28.6 acres.

Subbasin H

Subbasin H is bound on the west by Interstate 5 and on the south by the city limit. It is bounded on the east by the airport and Subbasin K. It drains into Subbasin G at MH G-3 and MH G-13. The subbasin receives flow from all of Subbasin I at MH H-12 via a 4-inch force main originating from the Airport Pump Station and from a private sewer system discharging into Subbasin I. The collection system consists primarily of concrete pipe. The subbasin has an area of about 49.7 acres.

Flow mapping investigations conducted in March 2005 indicated a leaking manhole at MH H-5 and pipe leakage ranging from 6 to 10 gpm between Manholes H-5, H-1 and G-4.

Subbasin I

Subbasin I is bound on the west by Interstate 5; it extends to just south of Page Avenue. It is bound on the east by the airport. It consists of 8-inch asphalt concrete (AC) sewer and receives flow at its southeast corner from a private collection system via a 2-inch force main discharging into MH I-25. The subbasin has an area of about 87.0 acres.

Subbasin J

Subbasin J is located on the north side of West Central Avenue, which forms its boundary with Subbasins K and L and extends to the north city limit. It is located west of Subbasin F, being divided by North Comstock Road, and is east of Subbasin M. Sewer pipe consists of 12-diameter concrete pipe along West Central Avenue, into which 8-inch diameter concrete pipe sewer pipe draining this subbasin flows at Manholes J-1, J-14, J-18, J-22 and J-26. The 12-inch main drains into Subbasin K at MH J-1. The subbasin has an area of about 178.2 acres.

Subbasin K

Subbasin K is located on the south side of West Central Avenue and extends south to Page Street. A private collection system is contained within the boundaries of this subbasin, and discharges into it at MH K-5. A main trunk line consisting of 24-inch and 18-inch diameter concrete pipe brings flow from Subbasin L at MH K-16 and leaves this subbasin at MH K-1 into Subbasin G. Subbasin J discharges into this subbasin via a 12-inch diameter concrete sewer. The remainder of the sewer pipe in the subbasin is 8-inch concrete sewer pipe.

Subbasin K includes the South Calapooia Sewer Project and the Page Street pump station. The Page Street Pump Station project extended sewer along Taylor Street. A STEP collection system was

constructed in Hastings Avenue and extended south along South Calapooia Street. The South Calapooia Street STEP collection system can be extended south to the intersection of South Calapooia Street and the Wilbur-Umpqua Road intersection.

Subbasin L

Subbasin L is located on the south side of West Central Avenue and extends south to Fir Avenue. The boundary on the east side is an undeveloped area north of the Log Ponds. A private collection system to the south along Taylor Street discharges into it at MH L-2. A main trunk line consisting of 15-inch and 18-inch diameter concrete pipe brings flow from Subbasin M at MH M-1 into this subbasin at the northeast corner and leaves this subbasin at MH L-1 into Subbasin K at the southwest corner. The remainder of the sewer pipe in this subbasin is 8-inch concrete pipe. The subbasin has an area of about 36.8 acres.

Subbasin M

Subbasin M is located on the north side of West Central Avenue, which forms the southern boundary and extends north to the city limit. It includes the Advent School. It receives flow from the Everett Avenue Pump Station in Subbasin N via a 10-inch force main, which runs under the railroad and discharges at MH M-18. A 15-inch diameter concrete sewer main carries this flow and other flow generated from Subbasin M westward and into Subbasin L at MH M-1. The remainder of the sewer pipe in the subbasin is 8-inch concrete sewer pipe except for an 8-inch diameter segment of PVC sewer main between Manholes M-10 and 11. The subbasin has an area of about 130.5 acres.

Subbasin N

Subbasin N is located on the north side of Everett Avenue, which forms the southern boundary, and extends north to the city limit. It receives flow from the Union Gap private sewer system located to the north via a 2-inch force main at MH N-16, which is located near East Sixth and North State Streets. A 15-inch diameter concrete sewer main enters at MH P-1 from the east, draining Subbasin P and continues through this subbasin collecting flow. Another 15-inch diameter concrete sewer main enters at MH N-4 from the south, draining Subbasin O. All flow is discharged from Subbasin N by means of a 15-inch diameter concrete sewer flowing to Everett Avenue Pump Station. The remainder of the sewer pipe in the subbasin is 8-inch concrete sewer pipe. The subbasin has an area of about 244.8 acres, including Union Gap.

Subbasin O

Subbasin O is located on the south side of Everett Avenue, which forms the northern boundary, and extends to the south of Valley Court and Schoon Mountain Road. It receives flow from Subbasin Q at MH Q-1, located near the northeast corner of this subbasin. A 15-inch diameter concrete sewer main also enters at MH Q-7 from the east, draining Subbasin Q, and continues through this subbasin collecting flow. A 10-inch diameter PVC sewer main carries flow from this subbasin to Subbasin Q and re-enters, combined with the flow discussed above at MH Q-7. The large private system to the east contributes flow by gravity sewer at MH O-20 and O-24. The remainder of the sewer pipe in the subbasin is 8-inch concrete sewer pipe. Sutherlin Creek flows through the north and west portion of this subbasin. The subbasin has an area of about 91.7 acres.

Subbasin P

Subbasin P is located on the north side of Sutherlin Creek Street, which forms the southern boundary and extends to the north city limit. It contains the East Extension School, the middle school and the high school. It receives flow from Subbasin S via an 8-inch AC sewer at MH P-60, located near the southeast corner of this subbasin. A 12-inch diameter concrete sewer main also enters at MH P-59 from the east, draining Subbasin R, and continues through this subbasin, collecting all flow from this subbasin. A 12-inch diameter concrete sewer main carries flow from this subbasin to Subbasin N at MH P-1. The remainder of the sewer pipe in the subbasin is 8-inch concrete sewer pipe. Sutherlin Creek flows through the north and west portion of this subbasin. The subbasin has an area of about 143.8 acres.

Flow mapping investigations conducted in March 2005 indicated pipe leakage between MH P-38 and P-36, which requires additional investigation, and in pipe between Manholes P-2, 3, 5, 8, 9, 11, 12, 13 and P-2, 53 to 57, inclusive, of more than 25 gpm.

Subbasin Q

Subbasin Q is located on the south side of Sutherlin Creek Street, which forms the northern boundary and is north of a large private system. It receives flow from Subbasin W via a 12-inch PVC sewer line at MH Q-30. Flows from the southern private system and a portion of Subbasin O also enter this subbasin at the southwest corner briefly via a 10-inch PVC main at MH Q-8, and then return into Subbasin O at MH Q-7 carrying a portion of Subbasin Q with it. Flows also leave this subbasin near the northwest corner into Subbasin O at MH Q-1. The remainder of the sewer pipe in the subbasin is 8-inch and 12-inch PVC and AC sewer pipe. Sutherlin Creek flows through the north portion of this subbasin. The subbasin has an area of about 100.7 acres.

Subbasin R

Subbasin R is located on the north side of West Central Avenue and extends to the north city limit. Sewer pipe consists of 12-inch diameter concrete pipe entering from Subbasin T along West Central Avenue, into which 8-inch diameter concrete pipe sewer pipe draining this subbasin flows at MH R-2A and R-7. The 12-inch sewer main leaves this subbasin at MH P-59 and flows into Subbasin P. The portion of the subbasin north of East Sixth Street is not developed. The subbasin has an area of about 110.2 acres.

Subbasin S

Subbasin S is located on the north side of Sutherlin Creek Street, which forms the southern boundary. It receives flow from Subbasin W via a 12-inch PVC sewer at MH Q-30. Flow from the southern private system and a portion of Subbasin O also enters this subbasin at the southwest corner briefly via a 10-inch PVC main at MH Q-8, and then returns into Subbasin O at MH Q-7 carrying a portion of Subbasin Q with it. Flow also leaves this subbasin near the northwest corner into Subbasin O at MH Q-1. The remainder of the sewer pipe in the subbasin is 8-inch and 12-inch PVC and AC sewer pipe. Sutherlin Creek flows through the north portion of this subbasin. The subbasin has an area of about 47.9 acres.

Subbasin T

Subbasin T is located on the north side of West Central Avenue and extends to the north city limit. To the south is a large private collection system. Sewer pipe consists of 12-inch diameter concrete pipe entering from Subbasin U at MH U-1 along West Central Avenue, into which 8-inch diameter concrete pipe sewer pipe draining this subbasin intersects at MH T-1, and 8-inch diameter PVC pipe intersects at MH T-23. The 12-inch sewer main leaves this subbasin at MH T-1 into Subbasin R. The portion of the

subbasin north of East Sixth Street is not developed. A private system to the south drains into this system at a manhole located at Wildwood and West Central Avenue. The subbasin has an area of about 147.8 acres.

Subbasin U

Subbasin U is located on the north side of West Central Avenue and extends to the north city limit. Sewer pipe consisting of 12-inch diameter concrete pipe enters from Subbasin X at MH U-31 along West Central Avenue, into which an 8-inch diameter concrete sewer pipe draining this intersects at MH U-8 and U-3. Subbasin V drains into this subbasin via 8-inch sewers at MH U-2 and U-7. The 12-inch sewer main leaves this subbasin at MH U-1 into Subbasin T. The portion of the subbasin north of East Sixth Street is not developed. The remainder of the sewer pipe in the subbasin is 8-inch concrete sewer pipe with a small portion of 8-inch PVC sewer pipe. The subbasin has an area of about 78.7 acres.

Flow mapping investigations conducted in March 2005 indicated a leaking manhole at Manholes U-2 and U10. Pipe leakage ranging from 21 to 25 gpm was noted between MH U-1, U-2, U-3, U-6 and U-7.

Subbasin V

Subbasin V is located on the south side of West Central Avenue. No other subbasins drain into this one. Subbasin V drains into Subbasin U at MH U-2 and U-7, and into Subbasin X at MH X-2 along West Central Avenue. The remainder of the sewer pipe in the subbasin is 8-inch PVC and concrete sewer pipe. The subbasin is about 34.8 acres in area. The subbasin has an area of about 34.6 acres.

Subbasin W

Subbasin W is located in the southeast corner of Sutherlin. The southern boundary is generally Southside Road. Flow enters the southeast corner via an 8-inch PVC sewer from Subbasin Z and exits at the southwest corner into Subbasin Q at MH W-1. The remainder of the sewer pipe in the subbasin is 12-inch and 8-inch PVC. The subbasin is about 115.1 acres in area.

Flow mapping investigations conducted in March 2005 indicated a leaking manhole at MH W-5.

Subbasin X

Subbasin X is located on the north side of West Central Avenue and extends to the north city limit. Sewer pipe consisting of 12-inch diameter concrete pipe enters from Subbasin X at MH U-31 along West Central Avenue, into which an 8-inch diameter concrete pipe sewer pipe draining this intersects at Manholes U-8 and U-3. Subbasin V drains into this subbasin via 8-inch sewers at Manholes U-2 and U-7. The 12-inch sewer main leaves this subbasin at MH U-1 into Subbasin T. The portion of the subbasin north of East Sixth Street is not developed. The remainder of the sewer pipe in the subbasin is 8-inch concrete sewer pipe with a small portion of 8-inch PVC sewer pipe. The subbasin has an area of about 78.7 acres.

Subbasin Y

Subbasin Y is located on the south side of West Central Avenue and extends south to Sutherlin Creek. Quail Run Pump Station collects flow in the southwest corner of this subbasin and pumps it via a 4-inch PVC force main to MH Y-3. Flow exits this subbasin northward by gravity into Subbasin X at MH X-21. Sewer pipe consists of 8-inch diameter PVC pipe. The subbasin has an area of about 19.0 acres.

Subbasin Z

Subbasin Z extends off the southeast corner of Subbasin W. It includes the area of the Cooper Creek Water Treatment Plant. Cooper Creek runs through the southern portion of this subbasin. Flow exits this subbasin northward by gravity into Subbasin W at MH Z-1. Sewer pipe consists of 8-inch diameter PVC pipe. The subbasin has an area of about 45.0 acres. Only about one-half of this subbasin is developed.

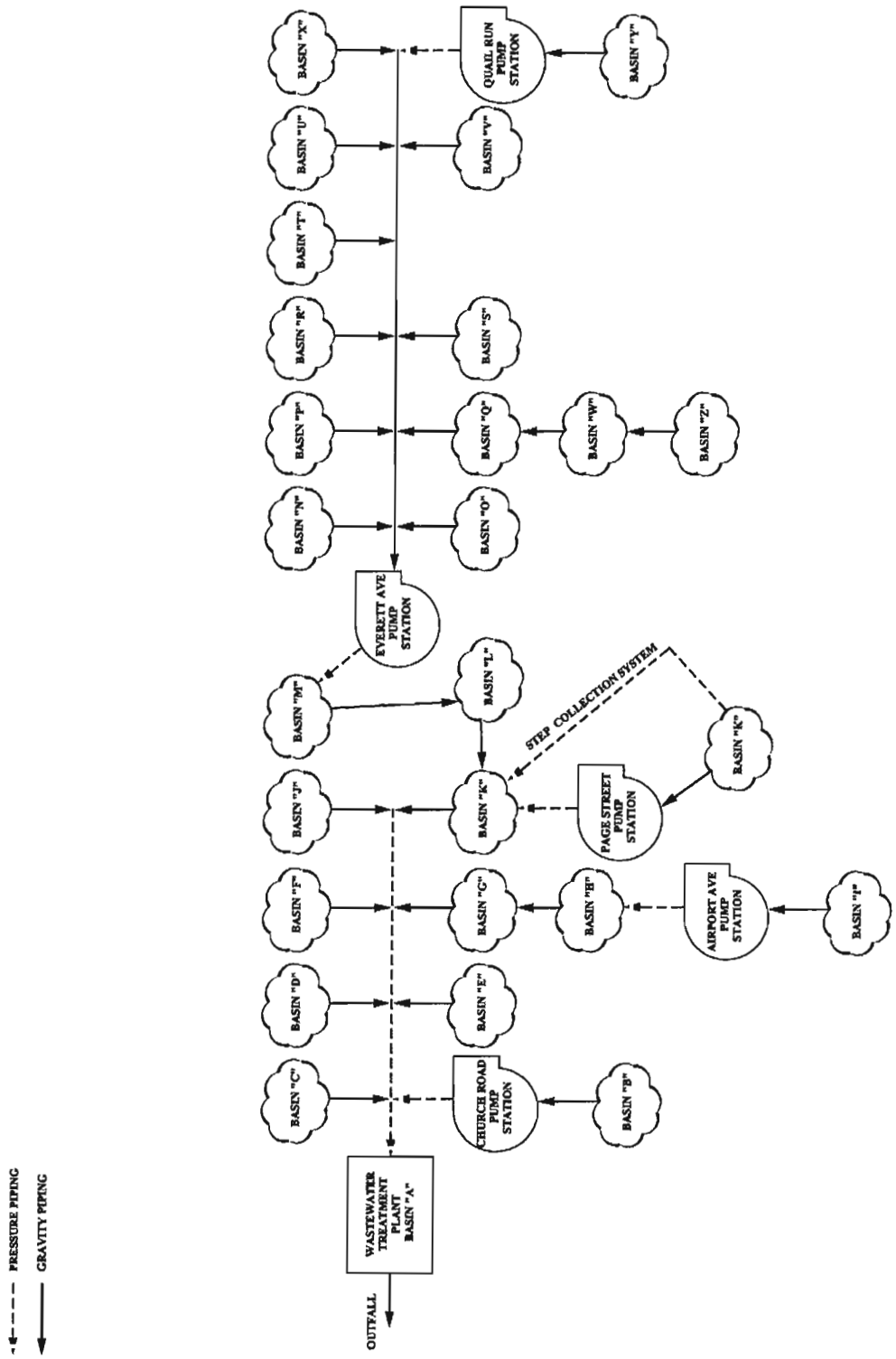


FIGURE 3.2.2
WASTEWATER SYSTEM SCHEMATIC

Pump Stations

The Sutherlin wastewater conveyance system includes five raw sewage pump stations and an influent pump station at the wastewater treatment plant. The location of each pump station is shown on Figure 3.2.1A. A visual inspection and drawdown test was performed on each pump station. The following sections provide a brief summary of each pump station. Figure 3.2.2 provides a schematic detailing the relationship of each pump station, and the treatment plant to the rest of the wastewater system.

Key design data for the pump stations is summarized in Table 3.2.2 and Table 3.2.3.

TABLE 3.2.2
PUMP STATION DESIGN DATA

Pump Station	**Everett Avenue**	**Airport**	**Plant Influent**
Location	Front St. & Everett St.	East of Cue Ave. & Landing St.	Sutherlin WWTP
Date Built	1978[1]	1979	1977
Last Upgrade	2011 (added grinder)	2011	1977
Type	Vacuum primed, centrifugal pumps	Duplex Packaged	Quadraplex Vacuum Primed Centrifugal
EPA Classification	I	II	I
Brand	Cornell 4NHTVM5-8 (1ea.) & 6NHGVM-8 (2ea.)	ITT Flygt (2ea.)	
Level Control	Sonic	Transducer	Transducer
Pump Type	Vacuum Primed	Self-Priming	Vacuum Primed, Non Clog
Pump Installation Year	1978	2011	1977
Motor Size	One 5 HP & Two 15 HP	Two 4 HP	Two 7.5 HP & Two 15 HP
VFD	No	No	No
Rated Flow GPM @ (TDH ft.)	400 Pump 1 @ (24) 1080 Pump 2 & 3 @ (50) 1500 Pump 2 & 3 @ (36)	140 @ (50)	450 Pump 1 & 2 ea @ (37) 1250 Pump #3 & 4 ea @ (35) Pump Sequence # 1 - 450 gpm (0.65 MGD) # 1 &2 - 900 gpm (1.3 MGD) # 1,2 &3 - 2,150 gpm (3 MGD) # 1,2 &4 – 2,950 gpm (4.2 MGD) All on – 3,400 gpm (4.9 MGD)
Flow Monitoring	Yes (Mag Meter)	None	**
Overflow Point	Nearest Manhole	Nearest Manhole	Flow Meter Manhole
Overflow Elevation	**	**	**
Overflow Discharge Stream	Sutherlin Creek	Sutherlin Creek	Cook Creek
Time to Overflow[2]	3 hr 20 min.	50 min.	**
Alarm Type	Auto loop dedicated phone line	Auto loop dedicated phone line	Plant alarm system
AUXILIARY POWER			
Type	Kohler	Kohler	Stationary Plant Generator – Onan Diesel
Aux. Power Output	125 kW	25 kW	350 kW
Diesel Fuel Tank Capacity	500 gallon Diesel	N/A uses Nat. Gas	500 gallon Diesel
Run Hours per Tank	48	N/A	Greater than 48
ELEVATIONS			
Ground	**	**	**
Wet well Floor	18' deep	20' deep	377.00

FORCE MAIN	Everett Avenue	Airport	Plant Influent
Force Main Length feet	890	1,290	**
Force main Diameter Inches	10	4	**
Detention Time minutes	21	16	Less than 2
Material	CI	AC	DI
Profile	Ascending	Ascending	Ascending
Discharge Manhole	Central Ave. & Oak St., MH18	Airway Ave. & Landing St., MH12	N/A
Condition Discharge MH	Good	Good	Good
Air/Vacuum Release Valve	None	None	None
Sulfide Control	None	None	None

(1) Re-build – original construction 1957
(2) Estimated

TABLE 3.2.3
PUMP STATION DESIGN DATA

Pump Station	Quail Run	Church Road	Page Street
STATION			
Location	Quail Run Subdivision	Church Road	Page Street and Taylor
Date Built	2001	2000	2005
Last Upgrade	2002		
Type	Duplex Submersible Wet well	Duplex Submersible Wet well	Triplex Submersible Wet well
EPA Classification	I	I	I
Brand	Flygt C-3085 (2ea.)	Flygt M.P. 3068 (2ea.)	Flygt NP3127 (3ea.)
Level Control	Mercury Switch (float)	Mercury Switch (float)	Ultra Sonic
Pump Type	Submersible	Submersible	Submersible
Pump Installation Year	2001	2000	2005
Motor Size	Two 2.4 HP	Two 2.3 HP	Three 10 HP
VFD	No	No	No
Rated Flow GPM @ (TDH ft.)	150 @ (21)	48 @ 47'	340 @ (60)
Flow Monitoring	None	None	None
Overflow Point	Nearest Manhole	Nearest Manhole	Nearest Manhole
Overflow Elevation	534.00	441.05	499
Overflow Discharge Stream	Sutherlin Creek	Cook Creek	Sutherlin Creek
Time to Overflow[1]			
Alarm Type	Protection alarm control panel (phone)	Protection alarm control panel (phone)	Auto loop dedicated phone line

AUXILIARY POWER	**Quail Run**	**Church Road**	**Page Street**
Type	Stationary Outside Bldg. – gas powered	Stationary Outside Bldg. – gas powered	Stationary Inside Bldg. – gas powered
Aux. Power Output	16kW	15 kW	100kW
Station Requirement	16 kW		
Diesel Fuel Tank Capacity	N/A uses Nat. Gas	N/A uses Nat. Gas	N/A uses Nat. Gas
Run Hours per Tank	N/A uses Nat. Gas	N/A uses Nat. Gas	N/A uses Nat. Gas
ELEVATIONS			
Ground	534.00	441	499
Wet well Floor	519.50		479
FORCE MAIN			
Force Main Length feet	700	507	3175
Force main Dia. Inches	4	2-each 2-inch dia.	1-each 6-inch dia. 1-each 10-inch dia.
Detention Time minutes	40	30	180 minutes
Material	PVC	PVC	PVC
Profile	Ascending		Ascending
Discharge Manhole	Quail Run MHY3	Church Road, MH B31	Page Street MHK8
Condition Discharge MH	Excellent	Excellent	Excellent
Air/Vacuum Release Valve	None	None	None
Sulfide Control	None	None	Air Injection

(1) Estimated.

Plant Influent Pump Station

All of the flows in the Sutherlin conveyance system arrive by gravity via a 27-inch gravity line to the Influent Pump Station. All flows are screened prior to entering the wet well. Currently, this screening device is experiencing problems with blockage on a regular basis. Flow discharges into the Plant Influent Pump Station wet well.

The station is located within the wastewater treatment plant and lifts influent to the headworks. The station was originally constructed in 1977 with the upgraded wastewater treatment system. The station is site-built with a deep wet well. The station has 4 non-clog vacuum primed centrifugal pumps. Two of them are 7.5 HP and have a 450-gpm capacity each. The other two are 15 HP and have a 1250-gpm capacity each. The Influent Pump Station discharges to the headworks.

Firm capacity of the pump station is approximately 3 MGD. Current Peak Wet Weather flows exceed the capacity of the pumps and risk system overflow during a major winter storm. Instantaneous flow is approximately 7.3 MGD. The pump station is undercapacity.

While the Plant Influent Pump Station appears to be in good physical condition, the pumps are so worn that they are only able to pump 61% of their original rated flow capacity. The WWTP currently has a diesel backup power generator, which is adequate to operate the pumps.

FIGURE 3.2.3
WASTEWATER TREATMENT PLANT INFLUENT PUMP STATION



Everett Avenue Pump Station

This station is located in the southwest corner of Subbasin N near the intersection of Front Street and Everett Avenue. The pump station collects flows from Subbasin N through Subbasin Z and transmits flows to Subbasin M through a 10-inch diameter, 890-foot long CI pressure main. The pump station was reconstructed in 1978. The pump station is supported by a 125 kW Kohler generator with a 150 amp breaker for current use and a 400 amp breaker for future use; it is located outside the building.

In the past, during high flow events, the station was not able to pump all of the influent sewage and which caused overflows. In addition, the receiving gravity conveyance lines surcharge during pump operation. Manholes O7, M7, M17 and M18 commonly overflow during periods of high ground water and heavy rain.

In 2010 an Everett Avenue Pump Station and Force Main Evaluation study was prepared by The Dyer Partnership Engineers & Planners, Inc. The report recommended replacement of the Everett Avenue Pump Station. The existing pumps are relatively old and the City is no longer able to find parts to repair the pumps. Prior to 2011, the pumps presented significant operational and maintenance difficulties, including seizing, or ragging, and had to be taken apart weekly for service. The study also evaluated two alternatives to increase the force main capacity and prevent sanitary sewer overflows.

Between 2009 and 2011 the city completed 5,800 lineal feet (lf) of sewer lining, and repaired and sealed 30 manholes (See Section 4). Also in 2009, Pump No. 3 was rebuilt. In early 2012, the Everett Avenue Pump Station Improvements Phase 1 project was completed. The project included a new influent grinder station and installation of a new flow meter. Between 2011 and 2012, surcharging of the collection system was reduced and the pump station reliability has increased. The reported flow capacity of the pump station is 1,500 gpm. Sanitary sewer overflows still continue during winter storms and high groundwater conditions.

The Everett Avenue Pump Station should continue to be evaluated over the next few years under various flow conditions to determine the effectiveness of the Phase 1 repairs. Recommended Phase 2 repairs and equipment upgrades include: new wet well, pumps, level control, alarm system, and other equipment.

FIGURE 3.2.4
INSIDE EVERETT AVENUE PUMP STATION



Airport Pump Station

This station is located in Subbasin I east of South Comstock Road and Airport Way, inside the Orenco Systems, Inc. facility complex. The pump station collects flows from Subbasin I and transmits flows to Subbasin H through a 4-inch diameter, 830-foot long ductal iron (DI) pressure main. The pump station was constructed in 1979. The original station consisted of a duplex Hydromatic self-priming packaged unit mounted on top of the wet well, with two pumps rated at 175 gpm each at 50 feet TDH each.

The City adopted a study (Dyer, 2008) that provided recommendations for improvements to the pump station. These recommendations led to a project that rehabilitated the pump station in the summer of 2011. These improvements included the addition of a backup power generator and transfer switch, wet well rehabilitation, new duplex 4 HP submersible pumps, piping and valves, valve vault, portable hoist, new control panel and other miscellaneous improvements, as called for in the study.

FIGURE 3.2.5
AIRPORT MAIN PUMP STATION



Quail Run Pump Station

This station is located in Subbasin Y located south of East Central Avenue, and west of the intersection of Quail Run Street and Aspen Run Avenue. The pump station collects flows from Subbasin Y and transmits flows through a 4-inch diameter 700-foot long pressure main to a manhole which gravity feeds into the East Central Avenue trunk line. The pump station was constructed in 2001.

During draw down testing, it was determined that the pump station is capable of pumping approximately 120 gpm with Pump No. 1 and 130 gpm with Pump No. 2. This rate is about 85% of the 150-gpm design capacity and is within the acceptable range for pump performance. Velocity in the force main is 3 feet per second. This difference is not anticipated to present any significant operational or maintenance problems. The discharge manhole condition was checked by probing with a screwdriver and is in good condition. The pump station is supported by a backup power generator.

FIGURE 3.2.6
QUAIL RUN PUMP STATION



Church Road Pump Station

This station is in Subbasin B located west of Interstate 5 near the intersection of Church Road and Golf View Avenue. The pump station collects flows from a low-lying area and transmits flows through a 2-inch diameter 430-foot long pressure main to a manhole in Golfview Avenue. The pump station was built in 2000. A natural gas backup power generator is operated at the site.

During drawdown testing, it was determined that the pump station is capable of pumping approximately 34 gpm with Pump No. 1 and 28 gpm with Pump No. 2. Velocity in the force main is 3 feet/second (ft/sec), which is the minimum desired velocity. The discharge manhole condition was checked by probing with a screwdriver and is in good condition. An upgrade will be completed in 2013 which will include a 10-foot by 12-foot building, eye wash station, utility basin, automatic transfer switch, and alarm system.

FIGURE 3.2.7
CHURCH ROAD PUMP STATION

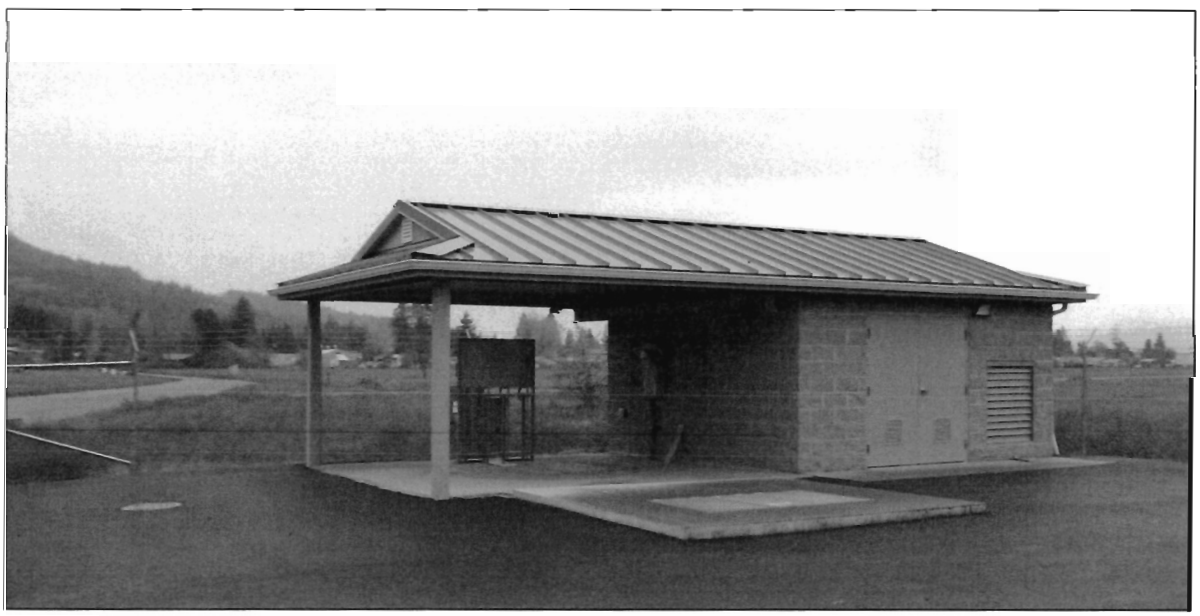


Page Street Pump Station

This station is located in Subbasin K located at the intersection of Taylor Street and Page Street. The pump station collects flows from the east side of an industrial area and pumps north to a manhole in Hawthorn Street. The pump station is designed to be upgraded by replacing the existing pumps, which transmit flows through a 6-inch diameter 3,175 foot long pressure main, with three 25 HP pumps conveying through an existing 10-inch diameter force main. The pump station was built in 2005. A backup power generator is operated at the site.

This pump station is newer and should be operating at the design rating of 340 gpm. The pump station is upgradeable to a project flow of 1,460 gpm.

FIGURE 3.2.8
PAGE STREET PUMP STATION



Existing Inflow/Infiltration Problems

The core problem within the existing collection system is the presence of large amounts of I/I. Flows at the wastewater treatment plant (WWTP) are very responsive to the previous 48 hours of rainfall. The immediate flow response at the WWTP indicates that there is a significant amount of inflow and rain-induced infiltration in the system. As part of the 1991 Comprehensive Wastewater Plan, flow mapping and smoke testing were conducted. The conclusion was that most of the winter flow in the system at that time was due to rain-induced infiltration. Rain-induced infiltration is difficult to locate and eliminate from the system.

Wet weather peak flows increased 52% between 1991 and 2001, while dry weather flows (an indicator of population) have increased only 31%. Domestic sewage does not account for the wet weather increase. The higher wet weather flows suggest an increase in prohibited downspout and area drain connections to the sanitary sewer, or a deterioration of the collection system. Smoke testing, the most efficient method of detecting inflow, was performed in October 2004 and numerous uncapped cleanouts and broken laterals were discovered. The 2005 I/I mapping is included in Appendix D.

Flow mapping (Appendix D) in March 2005 located several areas of infiltration. These figures were developed by The Dyer Partnership as part of an infiltration and inflow study conducted in 2004. These identified flows totaled 629 gpm, but flows at the plant indicated higher I/I flows of a magnitude of 1,209 gpm. Infiltration and inflow (I/I) repair work on the problems already identified was completed in the spring and early summer of 2005.

Listed below in Table 3.2.4 are the summary amounts of infiltration observed in each basin from the March 2005 flow mapping activities. Smoke testing was completed in 2012. Eighty-seven percent of the deficiencies were corrected or repaired. The report findings and maps are included in Appendix D.

TABLE 3.2.4
OBSERVED I/I FLOW AMOUNTS (GPM) IN EACH BASIN

A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z
0	26	0	1	0	13	32	0	0	90	0	0	12	12	0	185	20	138	0	40	50	10	0	0	0	0

I/I Repair History

An Infiltration/Inflow Study prepared in 2004 identified a number of infiltration and inflow problems in the existing collection system. Mapping which summarizes the 2004 study is included in Appendix D. Based on the 2004 study, the City implemented I/I improvement projects and continues its effort to cost-effectively mitigate peak wastewater flows. I/I related improvements projects include:

- **2009** The City lined 3,100 lineal feet of sewer lines.
- **2010** The City lined 5,800 lineal feet of sewer lines.
- **2011** The City repaired and sealed 30 sanitary manholes.
- **2011** The Everett Avenue Pump Station added a new 84-inch sanitary sewer manhole with grinder and 40 lineal feet of 18-inch sewer pipe.
- **2011** The Airport Pump Station was replaced.

- **2012** Additional collection system smoke testing was performed to identify potential inflow areas. See Appendix D.

The I/I projects have demonstrated some success. In 2005 the MMWWF was determined to be 1.85 MGD versus a 2012 MMWWF of 1.70 MGD. In 2005 the PIF was determined to be 8.20 MGD versus a 2012 PIF of 7.30 MGD.

Collection System Deficiencies

The Everett Avenue Pump Station's deficiencies limit capacity within the existing system. Gravity lines immediately downstream of the receiving manhole surcharge during pump operation. In addition, pumping capacity of pump stations and especially that of the Influent Pump Station can be overtaxed due to infiltration and inflow; continuing I/I reduction is essential. The Everett Avenue Pump Station reconstruction would significantly improve the collection system performance. A continuing program of manhole rehabilitation is in place. Each year 10 to 20 are repaired or replaced.

The gravity sewer system which receives flow from Union Gap was damaged by hydrogen sulfide. Previously, the receiving manhole was repaired and 300 feet of line were replaced. Union Gap was required by Sutherlin to install a bioxide system, and Union Gap appears to have addressed the hydrogen sulfide issue.

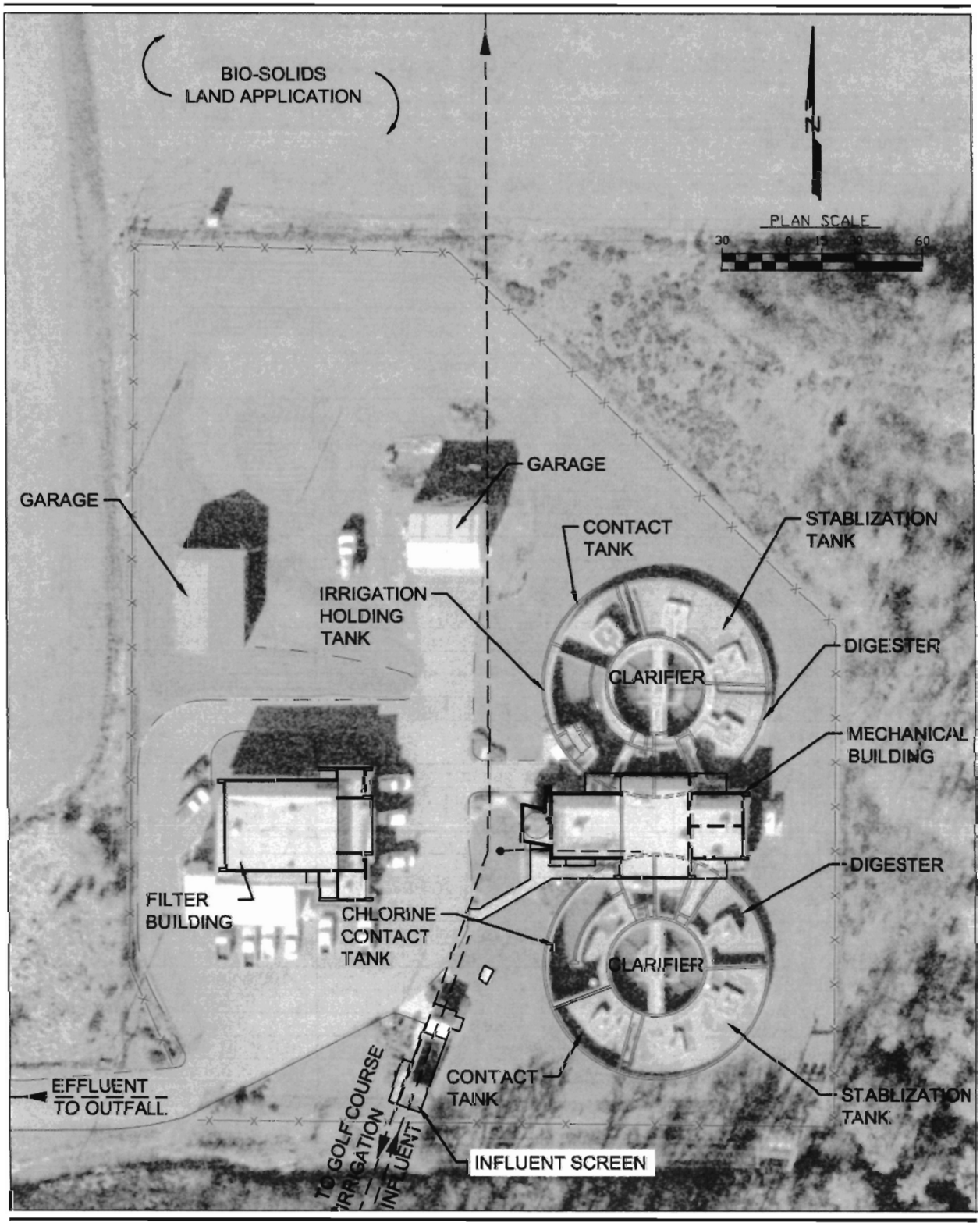
The original sewer system was constructed of concrete pipe with mortar joints. These joints allow significant infiltration. After 1975, concrete and asbestos cement pipe with rubber gaskets were installed. PVC has most recently been installed.

3.3 Wastewater Treatment Facility

History

The Sutherlin wastewater treatment plant was constructed in 1977 as a complete mix, activated sludge, secondary treatment facility. The style of plant is commonly referred to as a donut-type packaged plant, comprised of a central clarifier with aeration and digester chambers around the perimeter of the donut. Constructed between the two treatment units is a combination control building, with laboratory and housing for equipment and supplies related to the wastewater treatment plant. In 1995, a rotary drum screen and ISCO flow meter were installed. Other improvements include the replacement of RAS and sludge pumps. See Figure 3.3.1 for the existing Wastewater Treatment Plant site plan.

FIGURE 3.3.1
WASTEWATER TREATMENT PLANT



The facility has a 350 kW generator and transfer switch with 500-gallon diesel fuel storage to provide emergency power to essential treatment components. A summary of component specifications is included in Table 3.3.1.

TABLE 3.3.1
SUTHERLIN WWTP COMPONENT DESIGN SPECIFICATIONS

Component	Type	Capacity
Influent Pump Station	Non-clog Vacuum Primed 2 –450 gpm, 7.5 HP & 2 –1250 gpm, 15 HP	3.1 MGD firm 4.9 MGD all pumps on
Influent Flow Meter	Are Vel. Flow Meter – ISCO 4250	+10 MGD
Influent Screen	Lakeside Rotomat 47 S - .25	5.5 MGD
Grit Removal **(non-working)**	Centrifugal Vortex – Peabody Welles 12' w/screw conveyor	6.0 MGD
Grit Washer **(non-working)**	None	N/A
Comminutor **(rebuilt in 2012)**	Worthington, Type C Size 20	4.6 MGD
Influent Parshall Flume	12" throat	10.4 MGD
Aeration Contact Zone	Contact Stabilization – 2 units	170,000 gal. total
Aeration Capacity		85 cfm
Mixing Capacity		15 HP
Re-aeration Zone	Contact Stabilization – 2 units	398,000 gal. total
Aeration Capacity		170 cfm
Mixing Capacity		30 HP
Treatment Blowers	3 units	440 cfm, 20 HP each
Secondary Clarifier Zone	2 Units 2,200 ft^2 total surf. Area	228,000 gal. total
Digester	Aerobic Digester – 2 units	199,999 gal. total
Aeration Capacity		185 cfm
Mixing capacity		15 HP
Chlorine Contact Chamber	Single unit	106,000 gal
Chlorinator (2 units)	W&T V-800 Gas w/Vac. Reg.	500 lbs/day each
Chlorine Cylinders	Gas	1-1,200 lb cylinder, 2-150 cylinders
Filter (Not Operational)	Single 530 ft^2 - 5 gpm/ft^2	3.8 MGD
Filter Pumps	Vert. turbine 1 – 450 gpm, 7.5 HP; 2 – 1200 gpm, 20 HP	2.4 MGD firm, 4.1 MGD with all pumps on
Filter Backwash Blower	One - 25 HP	500 cfm
Irrigation Reservoir/Wet well	Holding tank	150,000 gal.
Irrigation Pumps	Vert. turbine 1-450 gpm, 20 HP; 2-1200 gpm, 50 HP	2.4 MGD firm, 4.1 MGD with all pumps on
RAS Pumps	Two pos. displacement, self-priming	520 gpm each
Scum Pumps	Homa Submersible Grinder Pumps	unknown
Decant Return Pumps	Two – Airlift	100 gpm w/20 scfm air
Effluent Flow Meter, Gravity	One - Rectangular Weir, 5 ft	6 MGD @ 1 ft.
Effluent Flow Meter, Irrigation	Two - Propeller Meter	6 MGD each
Outfall – 3200 ft. long	27" AC Pipe W/River Outfall	6.0 MGD @ High Water
Sludge Truck	Tank	3,200 gallons
Generator	Diesel	350 kW

Existing Treatment Process Description

Raw wastewater arrives at the treatment facility from the outlying subbasins via a 27-inch AC gravity sewer to the Rotamat screen. It then flows to the Influent Pump Station wet well. From the Influent Pump Station, flows are lifted to the headworks which include a grit separator, comminutor and Parshall

flume. Flow is split from the headworks and continues by gravity to the north and south treatment units commonly referred to as "donut units". Each unit has identical, circular tanks with interior dividing walls that form tanks (or basins) for each process. Each unit has a tank for a contact zone, a stabilization zone, a decant zone, a digester and a center clarifier. The clarifier has a 12 horizontal to 1 vertical sloping bottom leading to a sump, and external launder; the tank is 42 feet in diameter and has a side water depth of 11 feet. The north treatment unit includes the irrigation holding reservoir and the south unit includes the chorine contact tank and the filter sump. Refer to Figure 3.3.3 for the process flow schematic

FIGURE 3.3.2
EXISTING TREATMENT UNIT

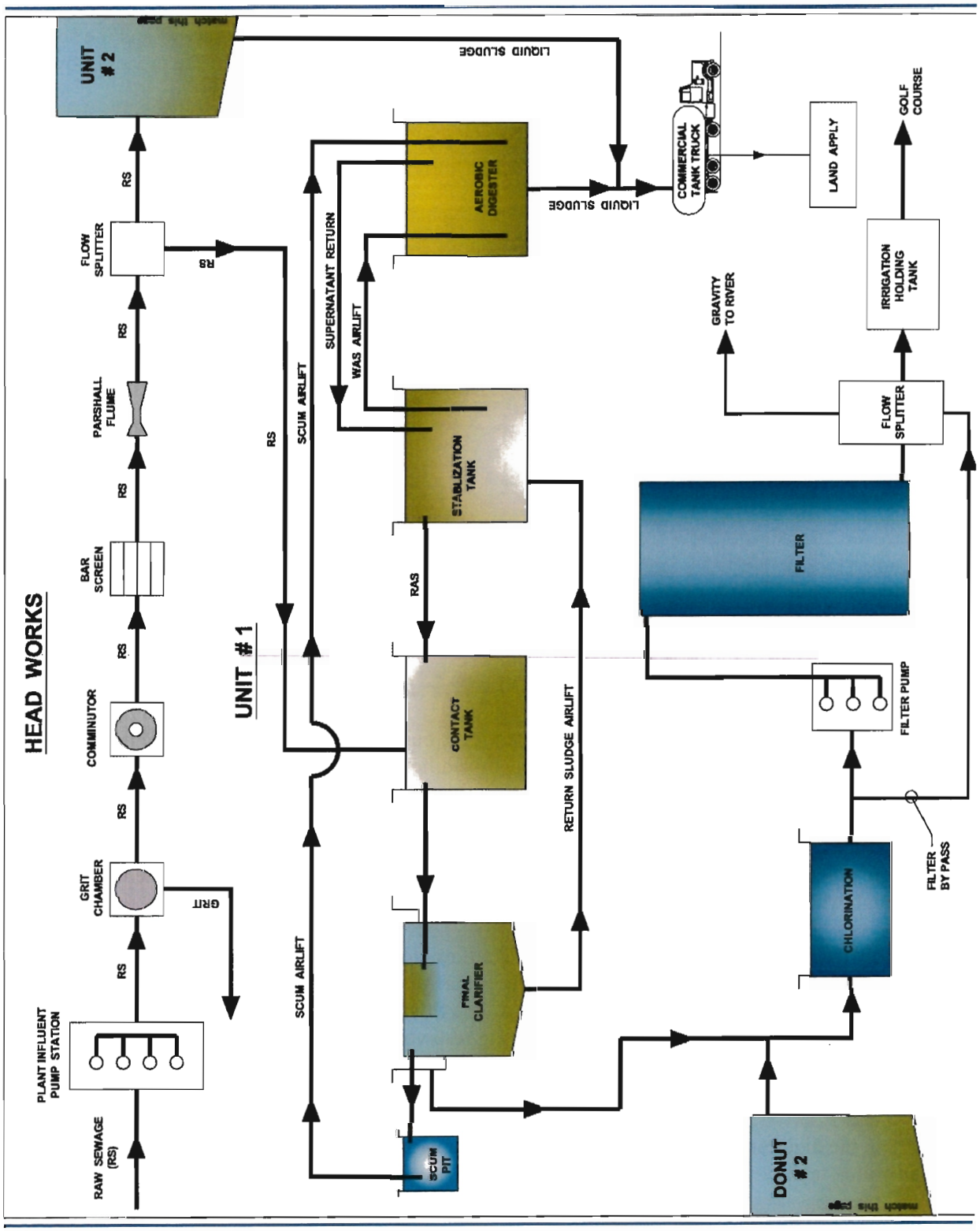


At each treatment unit, flow enters the contact tank via a trough into which return flow from the stabilization tank mixes with the raw sewage. The mixed liquor is aerated and then proceeds to the clarifier. Settled sludge underflow enters the return activated sludge (RAS) pump from which pumping to the stabilization tank occurs. A portion of the RAS is pumped to the digester as waste activated sludge (WAS) in order to maintain operational mixed liquor suspended solids concentrations. The RAS in the stabilization tank (also known as the re-aeration tank) is aerated and leaves the tank via the influent mixing trough where it stabilizes the incoming raw sewage.

The overflow from both the north and south clarifiers flows to the chorine contact channel located in the south treatment unit. After detention in the chorine contact basin, effluent is pumped to the irrigation holding reservoir. It is also possible to direct flow to the filters, but these have not been used for some time. The treated wastewater in the irrigation holding reservoir then overflows into a discharge box connected to the 27-inch outfall line or is pumped again, via the effluent irrigation pumps and an 18-inch force main, to the golf course holding pond for use as reclaimed irrigation water.

The WAS in the digester is aerated and decanted, then supernatant flow is returned by airlift pumps to the stabilization tank.

FIGURE 3.3.3
PROCESS FLOW SCHEMATIC



The plant was originally designed to provide an effluent discharge quality of 95% dry-weather removal efficiency for BOD_5 and TSS, and 85% removal efficiency for wet weather flow. Plant flows and processes are diagrammed in Figure 3.3.3. Design flows for the existing WWTP are listed in Table 3.3.2.

TABLE 3.3.2
EXISTING PLANT DESIGN FLOWS AND LOADING

Original Design Flows & Loading	
Design Flow	1.3 MGD
Peak Hourly Flow	4.9 MGD
Plant Loading BOD_5 & TSS mg/l	220
Plant Loading BOD_5 & TSS lbs/day	2,400
Dry Weather Efficiency BOD_5 &TSS reduction	95%
Wet Weather Efficiency BOD_5 & TSS reduction	85%
Current Actual Flows (Based on 2012 flows)	
Max. Month Wet Weather	1.70 MGD
Average Month Wet Weather	1.37 MGD
Average Month Dry Weather	0.72 MGD
Peak Daily Average Flow	5.57 MGD
Peak Hourly	7.30 MGD
Current Actual Loading (Based on 2007-2009 flows)	
Average BOD_5	1,170 ppd
Peak BOD_5	5,605 ppd
Average TSS	1,661 ppd
Peak TSS	3,368 ppd

WWTP Condition

The facility is reaching the end of its design life. Much of the equipment is sound and operable, and the buildings and tanks are structurally sound. There is some surface corrosion of steel components due to age and environment. The biological process provides treatment and experiences minimal upsets.

The hydraulic flow for the plant regularly exceeds the flow capacity of the mechanical screen chamber in the winter, and the plant is operating at capacity for mass loads. Each component of the treatment plant was examined for condition, capacity and operability. Details for each component are discussed below.

Mechanical Bar Screen and Flow Measurement

The existing screen is reported to be designed for a flow of up to 5.5 MGD but experiences flows which exceed its capacity due to blockage in the screening equipment, flow limits of the screening equipment itself, and the limited capacity of the Plant Influent Pump Station. As a result of screen flow capacity, potential screen clogging and exit channel carrying capacity, influent flows bypass the screen and carry debris and trash to the Influent Pump Station. The screen is inefficient for removal of solids, is undersized for wet weather flows and needs replacement.

FIGURE 3.3.4
MECHANICAL BAR SCREEN BEFORE HEADWORKS



Influent Pump Station

The pump station wet well acts as a settling basin for grit and solids. While the influent pumps are rated to handle the flows up to 4.9 MGD, their firm rated capacity (largest pump out of service) is only 3.1 MGD. The vacuum prime systems for the pumps have been upgraded with dual head vacuum pumps.

FIGURE 3.3.5
PUMP, BLOWER AND GENERATOR BUILDING & HEADWORKS (on roof)



Headworks

The headworks include a Pista Grit grit removal system which does not perform its function due to an inoperative grit auger. The grit chamber therefore acts as a debris catcher so the Pista paddle motor must be left running to keep grit in suspension, even though the unit does not remove grit from the process. The comminutor is designed to act as a backup to the screen. The manual bypass bar screen at this location has bar spacing too far apart for effective removal of rags and other debris. (This unit was completely rebuilt in 2012.)

Influent flow is measured at the Parshall Flume just past the comminutor. The flow measurement is then performed using an ultrasonic level sensor that is calibrated annually. The raw sewage then enters a flow splitter. However, there is inadequate flow control to either of the treatment units. Because of this, it is difficult to maintain a matching biological community in each treatment train.

The following describes the process basins within the donut units.

Aeration Basins

There are two aeration basins providing a total of 170,000 gallons (22,727 cubic feet) of contact zone and a total of 398,000 gallons (52,208 c.f.) of re-aeration (or stabilization) zone. Preliminary treated sewage from the headworks enters the mixing trough where it is combined with overflow from the stabilization zone. The combined flow is aerated and mixed in the contact tank. The plant operators try to maintain an MLSS of 3500 mg/l in the contact tank. Flow then proceeds to the clarifier. The return activated sludge (RAS) flow is then pumped to the stabilization or re-aeration zone. A small quantity of RAS is diverted to the aerobic digester in order to maintain an optimum MLSS between 6,500-7,500 mg/l. Occasionally the MLSS increases to 8,800 mg/l.

Each contact zone was originally provided with 85 cubic feet per minute (cfm) aeration capacity and each re-aeration zone with 170 cfm capacity. The air was supplied by one 20 HP, 440 cfm blower for each treatment unit including air for the digesters. The 25 HP blower was previously used for air supply to the filter operation, which is no longer functional. The blowers are currently used for air supply to existing aeration operations. This provides an additional 83 cfm to each contact zone and an additional 167 cfm to each re-aeration zone. The aeration basins have mechanical mixers with air supplied through the center of the mixer shaft. Operations personnel believe that the mixer propellers are too high to provide ideal mixing. The contact tank portion has one platform mixer and the stabilization portion of the tank has two platform mixers. A standard recommendation used (at the time of design) is 1,500 cubic feet of air applied per pound of BOD_5 aeration tank load for contact stabilization. The equipment should also be capable of transferring at least 1.0 pound of oxygen into the mixed liquor per pound (lb) of BOD_5 aeration tank loading. As the initial design standard capacity was for 2,400 pounds of BOD_5 per day, assumed to be produced by 10,000 persons, then 3,600,000 cf/day of air would be required (2,500 cfm). All three blowers provide a total of only 1,010 cfm or 1,454,440 cf/day of air to the aerations units. The current BOD_5 loading actually averages 1,170 lbs/day during wet weather and 1,114 lbs/day during dry weather. Based on the above standards, 1,010 cfm would only be enough air for a BOD_5 load of 970 lbs/day. This may be an explanation of why the plant experiences difficultly in providing adequate DO at times. The operators find that they must divert digester air to aeration use at times to keep dissolved oxygen levels up in the aeration process.

Published recommendations for contact stabilization volume requirements are based on 30 to 50 pounds of BOD_5 per 1,000 cubic feet of aeration tank volume. The current load of 2,400 lbs/day BOD_5 requires a range of 48,000 to 80,000 cf of aeration volume; 75,000 cf are provided. Therefore, the volume appears just adequate for the current load. It may need to be increased for the anticipated year 2037 load requirements.

Clarifier and Return Activated Sludge Pumps

Each of the two treatment units contains a clarifier designed to separate the activated sludge from the liquid. Contained within each clarifier is a rotating sweep arm with six sludge collection tubes. As the arm rotates, activated sludge is continuously drawn off the floor of the tank by the tubes. These tubes are spaced so that collection is uniform over the entire tank bottom. The tubes are individually connected to one collection box at the center of each clarifier and the six tubes within each clarifier. The activated

sludge flows from these boxes into a center well of the clarifier where it continues to the RAS sump. Each clarifier has a hopper, a sludge removal system (hydraulic), an internal launder, and is 42 feet in diameter with a depth of 11 feet.

The existing clarifiers do not remove sludge evenly. In addition, the WAS pumps appear to remove sludge at too high a rate, causing vortexing (or piping) in the center of the sludge blanket. Neither the WAS nor the RAS pumps have flow meters and the pumps sit too high in relation to liquid level so that loss of prime is a concern during operation.

Published standards at the time of design recommended 2.5 hours detention time and a maximum overflow rate of 700 gpd/sf for a final clarifier used in the contact stabilization process with an average design flow of 1.5 MGD or greater. The existing units have a total of 228,000 gallons. This provides a 2.5-hour detention time for up to 2.2 MGD. The surface area totals 2,200 square feet (sf). At an overflow rate of 700 gpd/sf, this should provide adequate treatment for 1.54 MGD. With the average month wet weather flow now at 1.3 MGD and maximum month wet weather flow at 1.85 MGD, the clarifier capacity based on these criteria is near capacity. EPA regulations addressing redundancy for a Class II WWTP require clarification capacity of 75% of design flow with one unit down. DEQ requirements call for either the secondary clarifiers to be sized for either the peak day with all clarifiers operational or the MMDWF with the largest clarifier offline, whichever results in the greater treatment capacity. Overflow rates for the separate seasons should be used, for example, 1,200 gpd/sf for winter and 800 gpd/sf for summer. Based on 1,200 gpd/sf, the winter-time overflow rate is 2.6 MGD per unit.

The WWTP needs additional clarification capacity. Modern clarifiers are available that can provide satisfactory service at flow rates as high as 1,500 gpd/sf with adequate depth, proper baffling and inlet flow distribution. DEQ current standards are 800 gpd/sf for summer and 1,200 gpd/sf for winter. Calculations indicate that the requirement to meet peak flow ($PDAF_5$) with all clarifiers on line and a 1,200 gpd/sf flow rate will control the design. This will require a total clarifier surface area of 7,883 square feet. With 2,200 square feet already provided, the facility would need 5,583 square feet of additional clarifier capacity if the existing contact stabilization process were simply expanded. If a membrane filter plant with a peak flow capacity of 1.65 MGD is provided, as is one alternative, then only 2,833 square feet of additional clarifier capacity associated with the contact stabilization process would be required. This issue will be further developed in Section 6.

The two RAS pumps were originally sized at 520 gpm each. The original pumps have been replaced with pumps with a current capacity of 700 gpm each. Not withstanding the poor placement of these pumps, the return rate is currently adequate for the AWWF of 1.54 MGD.

Digesters

Volume

Biosolids treatment at the WWTP was reviewed in terms of actual and required tankage to comply with 40 CFR Part 503 regulations on control of pathogens and vector attraction. Control of pathogens for WWTP biosolids was evaluated using *Class B Alternative 2: Use of Processes to Significantly Reduce Pathogens*, PSRP (EPA 1995). For aerobic digestion, the mean cell residence time and temperature shall be between 40 days at 20°C and 60 days at 15°C. As constructed, the existing aerobic digester has a volume of 200,000 gallons, and has an aeration tower consisting of mixing turbine blades with a 3-inch diameter air supply bubbler. Based on the hydraulic retention time (HRT) calculation provided in Appendix D, and on a 2009 average flow of 0.99 MGD, there are only 20 days of detention time. Additional storage is required in order to meet the 60-day aeration at 15°C requirement.
The City currently disposes of 1,030,000 gallons of biosolids during the dry season by land application. Land application disposal periods are restricted during wet weather by site conditions, the crop growing

and cattle grazing season. Because biosolids land application is not possible year-round, excess sludge is sometimes held in the digester and in the aeration tank. Due to the limitations of land disposal and storage, approximately 670,000 gallons of excess sludge is hauled to a year-round disposal facility located near Roseburg (Heard Farm).

Oxygenation Capacity

The digestion of sludge produced by contact stabilization has an advantage with respect to the oxygenation required by other activated sludge processes, except for extended aeration as shown in Table 3.3.3.

TABLE 3.3.3
EXISTING PLANT DESIGN FLOWS AND LOADING

Plant Type	Lbs Sludge per Capita per Day	Lbs O_2 Capacity per Lbs BOD_5
Extended Aeration	0.10	0.5
Contact Stabilization	0.12	0.8
Activated Sludge (no primary treatment)	0.15	1.0
Activated Sludge (separate anaerobic primary digestion)	0.10	0.8
Activated Sludge (aerobic digestion primary and WAS)	0.20	1.5

From Clow Engineering Data Form 13,358-A

The commonly used equation for oxygen required for this process is shown below:

$$[(\text{Lbs O.C. /Lbs Raw } BOD_5) \times (\text{population} \times \text{Lbs Raw } BOD_5)] = [\text{O.C. Req'd Lbs / Day}]$$

Using 0.17 days as the pounds of raw BOD_5 per capita and a current population of 7,905 persons, the following calculation results:

$$(0.8 \times 7{,}905 \times 0.17) = 1{,}075 \text{ Lbs } O_2 / \text{ Day} = 44.8 \text{ Lbs } O_2 / \text{ Hour}$$

The projected requirement for aerobic digestion oxygenation capacity in the year 2037 with a population of 11,594 is 65.7 pounds O_2/hour. The actual oxygen field transfer rate of diffusers in sludge is affected by temperature, oxygen concentration already in the sludge and the MLSS concentration of the sludge. Oxygen transfer systems typically are listed by clean water transfer capacity under ideal conditions. The actual oxygen transfer rate ranges from 30% to 60% of this value. The type of system provided at the Sutherlin WWTP uses diffused air with mechanical mixer introduction. This type of arrangement typically has a clean water transfer rate of 2.5 to 3.0 pounds. O_2/hr/HP. After deduction for the air volumes provided for both contact and re-aeration zones, 42% of the air volume remains for use in digestion (185 cfm). If power and air volumes correspond, the two 20 HP blowers can provide a total of 16.8 HP for the digestion process. Assuming a clean water transfer rate of 2.75 pounds O_2/hr/HP and a 50% field transfer factor, the existing equipment is estimated to provide only 23.1 pounds O_2/hr. This is only 55% of the ideal value currently required. The mechanical mixers have blades located below the surface and in theory provide no aeration capacity. However, it is likely that some aeration assistance is provided by atmospheric oxygen transfer due to the mixer surface agitation.

Mixing

Values for mixing requirements range from 0.5 to 4.0 HP per 1,000 cubic feet, depending on the geometry of the basin. The existing facilities have a 15 HP mixer in each of two 100,000-gallon basins. This provides 0.15 HP/1,000 cf. This is not likely to provide ideal digester mixing and suggests that solids are not likely kept in suspension. This can create removal problems and a reduction in digester capacity.

Disinfection

Chlorine Gas System

The chlorinator is a Wallace & Tiernan, V-800 unit able to provide 500 pounds of chlorine per 24 hours and is equipped with a vacuum regulator. The system uses one 1,220-pound short fill and two 150-pound cylinders on standby. The 22 pound/day chlorine demand at 1.3 MGD for a 2 mg/l dosage is easily met with the existing equipment. However, the use of chlorine gas in general has fallen into disfavor in recent years, due to concerns regarding accidental gas leakage and the requirement in most jurisdictions for a gas scrubber system. Other concerns regarding the use and storage of liquid chlorine ton cylinders involve the potential risk of terrorist theft at the facility or hijacking of shipments to the facility and subsequent gas release. In addition, the facility's NPDES permit requires that mixing zone studies and toxicity testing for chlorine be conducted.

The chlorinator uses an injector to draw gas under vacuum from the system and mix it with water, creating a strong chorine solution. This chorine solution is introduced to the treated effluent at the inlet to the contact basin. Chlorine gas is becoming increasingly costly, is a potential hazard when workers are exposed to it, and is toxic to aquatic life.

Contact Basin Volume

During summer months, effluent will be recycled as irrigation water and/or stored in an open reservoir. Excess Class A recycled water will be stored in an open reservoir. Open storage reservoirs are subject to poor water quality created by algae growth (clogs sprinklers), breeding of vectors, slimes, aesthetically displeasing deposits and odors. Chlorine improves a storage reservoir's water quality. Since a storage reservoir is required, chlorine-based disinfection is recommended to treat summertime recycled effluent.

Based on DEQ guidelines, minimum chlorine contact times are 60 minutes at the average daily flow and 20 minutes at peak daily flow. Based on an existing ADWF of 0.9 MGD, a 106,200 gallon tank with some baffles extends the existing contact time to 85 minutes. For an existing PDAF of 4.54 MGD, the contact time is 17 minutes. The chlorine contact chamber is adequate for dry weather flows and is currently undersized for peak day flows.

Laboratory, Chlorine and Filter Building

This building is in good condition and houses the chlorine gas system, offices, meeting rooms, the laboratory and the tertiary filter room. The filters are no longer functional. The existing laboratory lacks adequate space but is functional. The plant lab has basic equipment necessary to analyze the wastewater stream. With filters removed from the large space they currently occupy, this building presents a number of expansion possibilities.

FIGURE 3.3.6
LABORATORY, CHLORINE AND FILTER BUILDING



Pump, Blower and Generator Building

While the buildings appear to be in good condition, there is a very serious problem with respect to the pump building space below the pump room floor. A large number of valves and the pump suction and discharge pipes are located in this crawl space below the floor. This space is excavated below the grade of the surrounding grounds. Access to this space is through a 3-foot by 3-foot side access door. It is a simply a pit without walls or a floor and it is filled with groundwater. This makes access to valves difficult or impossible as some of the valves are underwater. The ground water in this region is within 3 to 4 feet of the surface. Valves in this space cannot be maintained. Plant crews have rigged board catwalks to get to some of the valves but electrical or heavy equipment cannot be brought into the space. When attempts have been made to pump out this space, groundwater continues to enter. This space is not accessible, safe or suitable for rehabilitation. This area presents one of the greatest challenges to the continued use of pumps in their present location and configuration.

FIGURE 3.3.7
FLOODED PIPE AND VALVE SPACE UNDER PUMP ROOM





3.4 Effluent Disposal

Discharge from the Sutherlin WWTP is regulated under an NPDES permit. In order to protect aquatic life, the permit prohibits the discharge of waste that violates water quality standards. In addition, Calapooya Creek is located in the Umpqua Basin, which is water quality limited for several parameters, including temperature (summers), pH (summers), fecal coliform (year-round), and phosphorus. No discharge to waters of the state is allowed between June 1st and October 31st. Calapooya Creek effluent discharge is allowed from November 1st through May 31st. Discharge in May is allowed only when Calapooya Creek flow is over 82 cfs.

Calapooya Creek

The effluent is conveyed to Calapooya Creek in an 18-inch diameter AC pipe. The outfall pipe is approximately 3,200 feet long and flows from the irrigation holding reservoir in Treatment Unit No. 2 (at surface elevation 396.5 feet) to the pipe outfall with a pipe invert elevation of approximately 371 feet. Creek water elevation may be as high as 392.10 feet. During high creek conditions, the capacity of the outfall is approximately 6.7 MGD.

Recycled Water (Golf Course Irrigation)

During Calapooya Creek low flow periods, effluent is irrigated on approximately 95 acres of the golf course. At the WWTP, effluent is disinfected with chlorine and then is held in an irrigation holding reservoir in Treatment Unit 2. The filter is non-functional and has been bypassed. Treated effluent flows directly from the chlorine contact chamber to the irrigation holding chamber and then is pumped through an 18-inch diameter force main to a holding pond at the golf course. The storage capacity of the golf course pond is not adequate in size for the amount of recycled water available. Each year the June through October (summer) effluent is discharged to Calapooya Creek and land applied at the golf course. In the summer of 2012, 99 million gallons (MG) of effluent was produced, of which 60 MG was discharged to Calapooia Creek and 39 MG was land applied (irrigated) at the golf course.

There are three irrigation pumps. The smallest pump has a rated capacity of 450 gpm; the largest pumps have a capacity of 1,200 gpm. With the largest pump out of service, the irrigation pumping system has a capacity of 1,650 gpm (2.4 MGD).

The NPDES permit requires the effluent to be treated to a minimum Level II (Class B) prior to land application. Rules require that when Level II effluent is spray irrigated, there must be a minimum of 10 feet from the site to the property line. However, the golf course (irrigation site) has a bordering residential community and, therefore, effluent must be treated to Level IV (Class A) prior to spray irrigation.

FIGURE 3.4.8
RECYCLED WATER STORAGE POND



3.5 Sludge and Biosolids Management

Sludge from the WWTP is treated aerobically to achieve Class B pathogen standards. Meeting Class B standards allows the sludge to be used for beneficial soil enhancements on agricultural and forestlands. 40 CFR Part 53 rules restrict the sites to non-public access sites with nitrogen-depleted soils. Strict limits are enforced on the amount of nitrogen and trace elements in the municipal sludge applied.

Disposal of the WWTP biosolids was evaluated with respect to regulatory requirements for pollutant limits (i.e. 40 CFR Part 503, Subpart B) and to agronomic rates for the on-site vegetation (i.e. nitrogen). The Part 503 rule requires that biosolids be land applied at a rate that is equal to or less than the agronomic rate for nitrogen at the application site. Additional Part 503 requirements include the following (EPA 1995):

- Biosolids cannot be land applied unless trace element concentrations in the sludge are below ceiling concentrations specified in Part 503.

- Biosolids must meet either (1) the pollutant concentration limits specified in Table 3 of Part 503 or (2) the Part 503 cumulative pollutant loading rate (CPLR) limits for bulk biosolids.

The amount of plant available nitrogen (PAN) currently applied to the City's biosolids reuse site was first calculated and then compared with the nitrogen requirements of the site. The PAN provided at the reuse site from 1999 to 2002 was calculated using procedures outlined in EPA's *Process Design Manual - Land Application of Sewage Sludge and Domestic Septage* (1995). For the PAN calculation, average nitrogen concentrations (e.g. TKN, nitrate, etc.) from 1999 to 2002 biosolids analysis were utilized. It was assumed in this analysis that the biosolids contained 2.0% solids and were applied annually to 16 acres. Volatilization of applied ammonia was assumed to be negligible. A summary of the PAN calculations is presented in Appendix C. The reported nitrogen uptake for biosolids application to rye grass is 100 pounds per acre per year (EPA 1995).

The City applies the treated sludge from the digesters for soil enhancement on agricultural land. The City utilizes a 3,200-gallon truck which spreads the biosolids directly on four sites: the Reddekopp site (80 acres), the Rust site (80 acres), the Williams site (80 acres) and the Crouch site (35 acres). In addition sludge is hauled to a private facility (Heard Farm). See Appendix D for application rates.

3.6 WWTP Sludge Quality

The WWTP is operated to produce a sludge in compliance with the Class B pathogen reduction requirements of the 40 CFR Part 503 regulations. Reduction of vector attraction to the digested sludge is achieved at the WWTP by reducing the mass of volatile solids in the sludge by 38 percent where possible. The staff has not tracked actual volatile solids reductions in the past

Based on the results from the annual metals sampling and analysis of sludge from 1999 to 2002, the sludge hauled for land application was in compliance with the regulatory concentration limits. A summary of sludge analysis results on the WWTP sludge is given in Table 3.6.1.

TABLE 3.6.1
SUMMARY OF HEAVY METALS CONCENTRATION IN WWTP SEWAGE SLUDGE

	Concentration Limits (mg/kg) [1]		Concentration (mg/kg) in Sludge			
Metal	Cumulative	Ceiling	01/06/01	03/19/02	08/13/03	03/10/04
Arsenic	41	75	ND	ND	ND	ND
Cadmium	39	85	2.18	2.0	ND	ND
Chromium	1,200	-	15.3	23.7	15.5	1.76
Copper	1,500	4,300	344	381	270	258
Lead	300	840	31.7	31.7	18.3	17.6
Mercury	17	57	ND	23.2	1.28	7.09
Molybdenum	18	75	12.7	ND	22.5	ND
Nickel	420	420	8.19	22.7	ND	ND
Selenium	100	100	ND	ND	5.62	ND
Zinc	2,800	7,500	600	59	155	ND

[1] 40 CFR Part 503, Subpart B, ND – non-detected

For all of the trace elements covered under the rules, the trace element concentrations in the WWTP biosolids were below the cumulative and ceiling concentration limits. Since the WWTP biosolids are in compliance with these limits, metal loadings are not required to be tracked (EPA 1995). Based on the cumulative loadings allowed by EPA, copper is the limiting pollutant in the applied biosolids. For the current average rate of biosolids application at each active site and current metal levels, the estimated site life based on copper loadings is 1,556 years. The life of the site exceeds the study period.

3.7 Emergency Standby Power Systems

Each pump station is equipped with a standby power generator. The wastewater treatment plant has a 350 kW diesel backup generator and an automatic transfer switch. The generator is connected to the treatment systems at the WWTP. Equipment that does not run on standby power includes:

1. Aerator
    a. Stabilizer No. 1
    b. Unit No. 2
2. Aerator
    a. Stabilizer No. 1
    b. Unit No. 1
3. RAS Pump
    a. RAS Pump No. 1
    b. Unit No. 1
4. RAS Pump
    a. RAS Pump No. 2
    b. Unit No. 2
5. Filter Blower
6. Hydraulic Blower
7. Irrigation Pump No. 2
8. Irrigation Pump No. 3

Section

4

Wastewater Characteristics

Section 4

Wastewater Characteristics

4.1 Wastewater Volume

Information regarding dry weather and wet weather flows as well as infiltration and inflow (I/I) are important in the design of wastewater collection, treatment and disposal facilities. The Maximum Monthly Dry Weather Flow (MMDWF) usually determines the maximum organic loading of the major treatment process units. The Maximum Monthly Wet Weather Flow (MMWWF) determines the size and hydraulic capacity of the major process units necessary to provide the desired degree of treatment. The Peak Instantaneous Flow (PIF) determines the hydraulic capacity of pipelines, pumps, channels, and inlet structures, and the reserve capacity of units such as clarifiers and disinfection facilities.

The data used for this report is taken from the 2008 through 2012 Discharge Monitoring Reports (DMRs) for the Wastewater Treatment Plant (WWTP). The City's wastewater system staff records the readings on the influent meter and checks the rain gauge at the plant daily. Wastewater is sampled and BOD_5 and TSS tests are run twice weekly. Rain data for this analysis is from the daily readings of the rain gauge at the wastewater facility.

The City has completed significant I/I improvements each year since 2005. A summary of recent I/I improvements is provided in this study. City staff has indicated that peak flows have been reduced significantly as a result of the I/I improvements, although the magnitude of reduction has not been determined. The most recent flow records are used for this study in order to predict futures flow and to ensure proper sizing of infrastructure.

DEQ Guidelines for Flow Projections

Unless otherwise noted, all calculations were made using the DEQ *Guidelines for Making Wet Weather and Peak Flow Projections for Sewage Treatment in Western Oregon*, 1996 revision. A summary of the calculations is included at the end of this chapter.

Precipitation Rates for Calculations

In Western Oregon, there is a relationship between peak storm events, ground water elevations and seasonal sewage flows. To reduce the probability of plant failure, Maximum Month Dry Weather Flows ($MMDWF_{10}$), Maximum Month Wet Weather Flows ($MMWWF_5$), Peak Day Average Flow (PDAF) and Peak Instantaneous Flow (PIF) are utilized in the design process. $MMDWF_{10}$ is the flow during a ten-year event (10% monthly probability of occurrence in May). $MMWWF_5$ is the flow during a five-year event (20% monthly probability of occurrence in January). Monthly probability rainfall values were taken from the National Oceanic and Atmospheric Administration's (NOAA) *Climatography of the United States No. 20, 1971-2000* for the city of Roseburg (COOP ID: 357331), Oregon (See Appendix E.). Roseburg is

located 10 miles south of Sutherlin. Peak design flows are based on a 24-hour (daily) rainfall event and are taken from NOAA Isopluvial charts. Rainfall events are summarized in Table 4.1.1.

TABLE 4.1.1
CITY OF SUTHERLIN STORM EVENTS

Source	20% Probability	10% Probability
NOAA – Isopluvial	3.7" per day	4.1" per day
NOAA – Monthly Climatography	7.14" (Jan.)	3.52" (May)

4.1.1 Dry Weather Flow

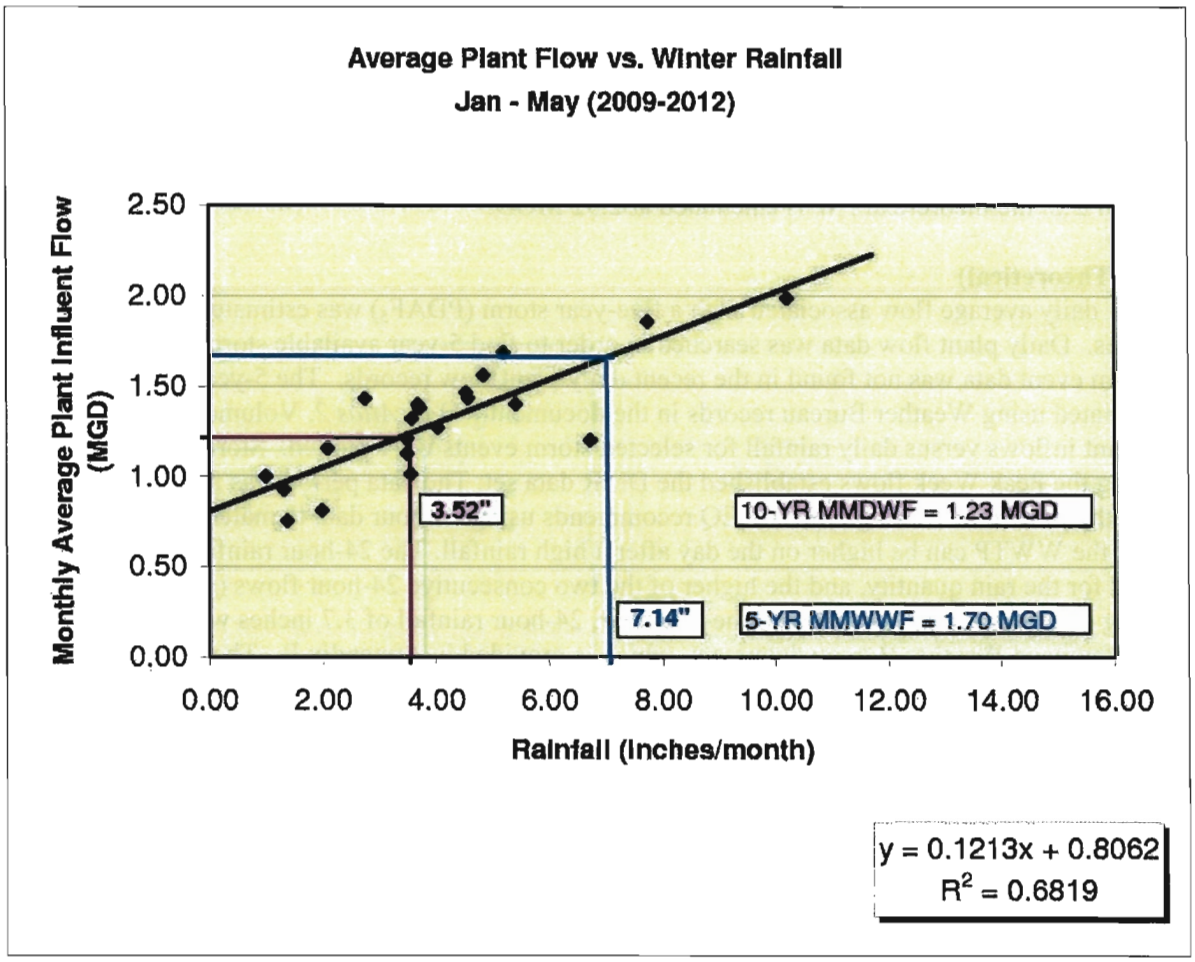
AAF (Record)
Average Annual Flow (AAF) is the average of all flows over a year. The AAF was determined from analysis of treatment plant flow records for the years 2010 through 2012. Commercial and industrial contributions have been included in the total flow analysis, and AAF determined to be 1.07 MGD or 135 gpcd.

ADWF (Record)
The average dry weather flow (ADWF) was determined to be 0.72 Million Gallons per Day (MGD) from analysis of treatment facility flow records for the months June through October. For the years 2008 through 2012, the ADWF was determined to be 0.724 MGD or 92 gpcd.

The ADWF can be divided into two components: base sewage flows and base infiltration. The base sewage flow is the portion of the treatment plant flow attributed to sanitary sewage. Base Sewage flows were determined to be the ADWF for the months of July, August and September when ground water is not present. The Base Sewage flow to the treatment plant is calculated to be 0.617 MGD. In determining projected flows, allowance must be made for unavoidable infiltration that is dependent upon such factors as the quality of materials and workmanship in the sewers and building connections, the character of maintenance, and elevation of the surrounding groundwater in relation to that of the sewers. The base infiltration is found by calculating the difference of the ADWF and the Base Sewage flow. Base Infiltration is calculated (0.724-0.617) to be approximately 0.107 MGD or 14 gpcd.

$MMDWF_{10}$ (Theoretical)
The Maximum Monthly Dry Weather Flow ($MMDWF_{10}$) is defined as the flow recorded at the plant when total rainfall quantities are at 10% probability of occurrence in any one year (usually in May). The $MMDWF_{10}$ was determined, following DEQ guidelines, by plotting December through May average plant flow versus monthly rainfall. This is illustrated in Figure 4.1.1 MMDWF (Monthly Dry Weather Flow vs. Winter Rainfall). Linear regression was used to fit a line to the data. Ninety percent probability May rainfall values were taken from the National Ocean and Atmospheric Administration's *Climatography of the United States No. 81, 1971-2000* for the city of Roseburg, Oregon. The Sutherlin May 90% approximate rainfall total was 3.52 inches. This calculated $MMDWF_{10}$ based on a 3.52 inch event is 1.23 MGD or 156 gpcd.

FIGURE 4.1.1.1
MMWWF & MMDWF



4.1.2 Wet Weather Flow

AWWF (Record)
The Average Wet Weather Flow (AWWF) is the average flow at the plant during the wet weather season (November through April). The AWWF was determined from an analysis of the treatment plant flow records for the years 2008 through 2012 and was determined to be 1.37 MGD (173 gpcd).

$MMWWF_5$ (Theoretical)
The maximum monthly wet weather flow (MMWWF) was determined in a manner similar to that employed for determination of the MMDWF, following DEQ guidelines. The results can be seen in Figure 4.1.1. The analysis period was from December 2003 through May 2004. With linear regression analysis of average monthly plant inflow versus monthly rainfall, a MMWWF of 1.70 MGD was calculated. DEQ guidelines suggest that the MMWWF represents the highest monthly average flow attained during the winter high groundwater period, and has a 20% chance of occurring in any one year.

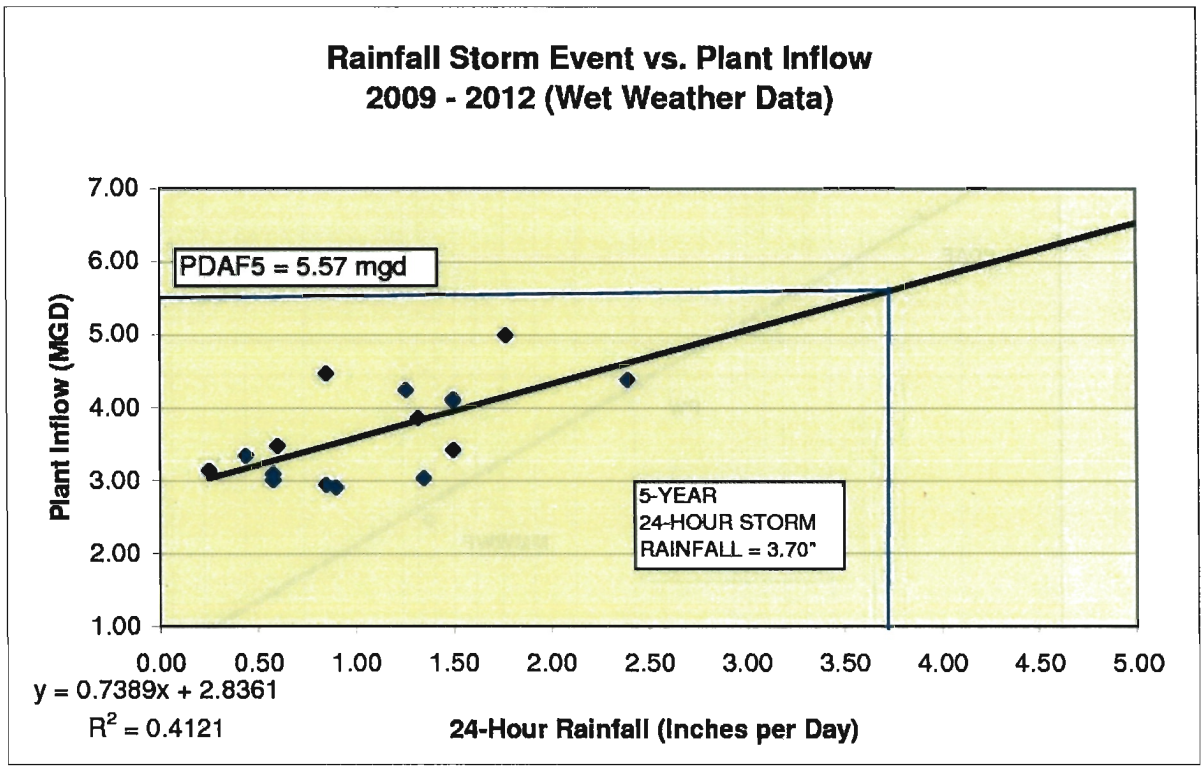
Peak Average Week (Record)
For this study, the Peak Average Week (PW) flow was taken as an average of the 2009 through 2012 flows and is the highest average daily flow rate during a seven-day wet weather data set. The peak average week is 2.94 MGD (or 371 gpcd) which occurred in January 2012.

Peak Average Week (Theoretical)
Following DEQ guidelines, the theoretical Peak Average Week flow was estimated by assuming the PW occurs 1/52 of the time (1.9% probability). Based on extrapolation from the probability graph shown in Figure 4.1.2.2, the theoretical PW is calculated at 2.92 MGD.

$PDAF_5$ (Theoretical)
The peak daily average flow associated with a five-year storm ($PDAF_5$) was estimated by following DEQ guidelines. Daily plant flow data was searched in order to find 5-year available storm event data. Five-year storm event data was not found in the recent daily plant flow records. The 5-year storm rainfall was approximated using Weather Bureau records in the document *NOAA Atlas 2*, Volume X Isopluvial Map. Daily plant inflows versus daily rainfall for selected storm events were plotted. Storms with rain exceeding the Peak Week flows established the DMR data set. The data period was from January through May for the years 2009 through 2012. DEQ recommends using 24-hour data to match plant flow data. Flows at the WWTP can be higher on the day after a high rainfall. The 24-hour rainfall for the first day was used for the rain quantity, and the higher of the two consecutive 24-hour flows (that day or the following day) was used for the flow. The five-year, 24-hour rainfall of 3.7 inches was taken from the above-referenced Weather Bureau document, which is provided in Appendix B. The resulting $PDAF_5$ is 5.57 MGD. Refer to Figure 4.1.2.1.

Peak Day (Record)
A Peak Day flow from the DMR records (2009-2012) was used to check the calculated theoretical Peak Daily Average Flow ($PDAF_5$). The Peak Day (highest 24-hour flow) in the 2009-2012 time period was 4.98 MGD. The total rainfall causing the high flow event was a three-day event totaling 3.42 inches (vs. a 3.7 inch 5-year event). Based on a Peak Day flow of 4.98 MGD, a PDAF5 of 5.57 MGD is reasonable.

FIGURE 4.1.2.1
PDAF$_5$ GRAPH



PIF$_5$ (Theoretical)
The peak instantaneous flow associated with a 5-year storm event (PIF$_5$) was estimated by using the annual average flow (AAF), maximum monthly wet weather flow (MMWWF$_5$) and peak daily average flow (PDAF$_5$). These values were plotted on logarithmic probability paper, as outlined by DEQ (1996). Such a projection is based on the principle that an average monthly flow is likely to occur 6/12 of the time or 50%, and a peak monthly flow occurs 1/12 of the time or 8.3%. Likewise, peak weekly flow will take place 1/52 of the time or 1.9%; peak daily flow occurs once in 365 days or 0.27%, a peak hour flow happens once in 8,760 hours or .011%. Plotting these numbers against probability gives a current PIF of 7.30 MGD.

A summary of existing flow rates as developed from flow data from 2009 to 2012 is provided below in Figure 4.1.2.2 and Table 4.1.2.

FIGURE 4.1.2.2
SUTHERLIN WWTP (2012) EXISTING INFLOW RATES

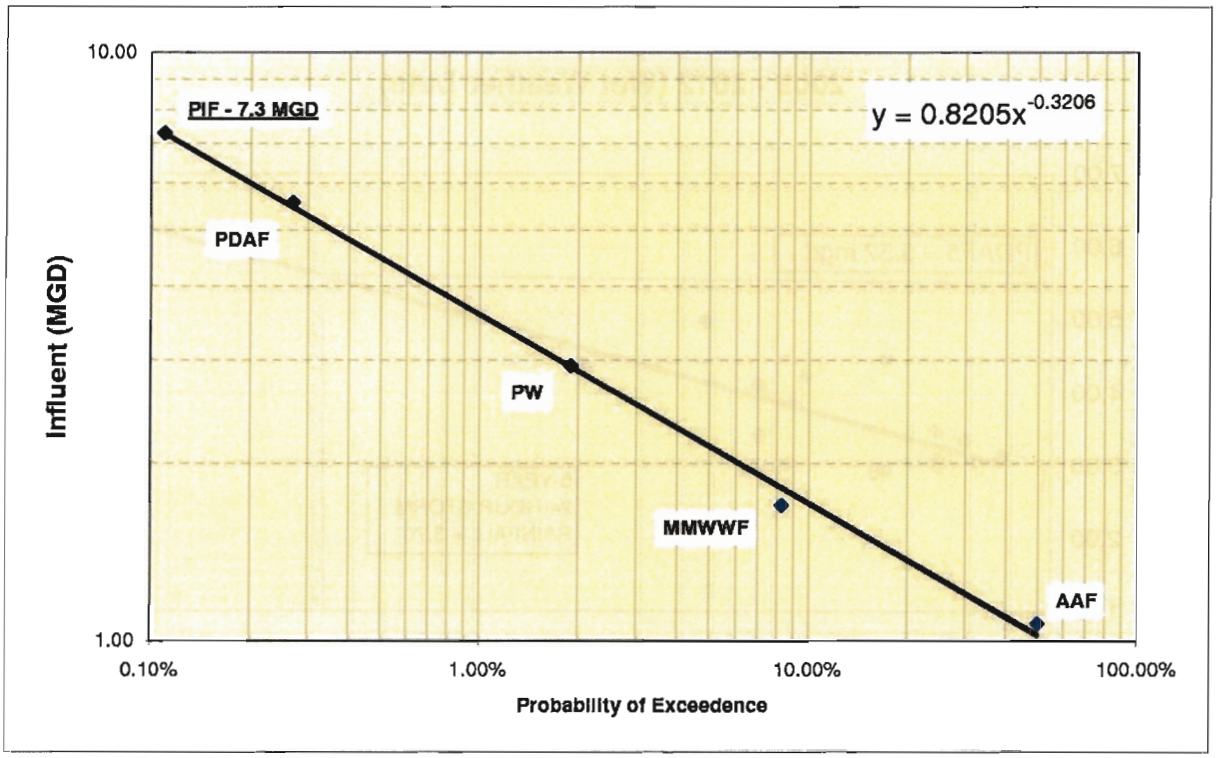


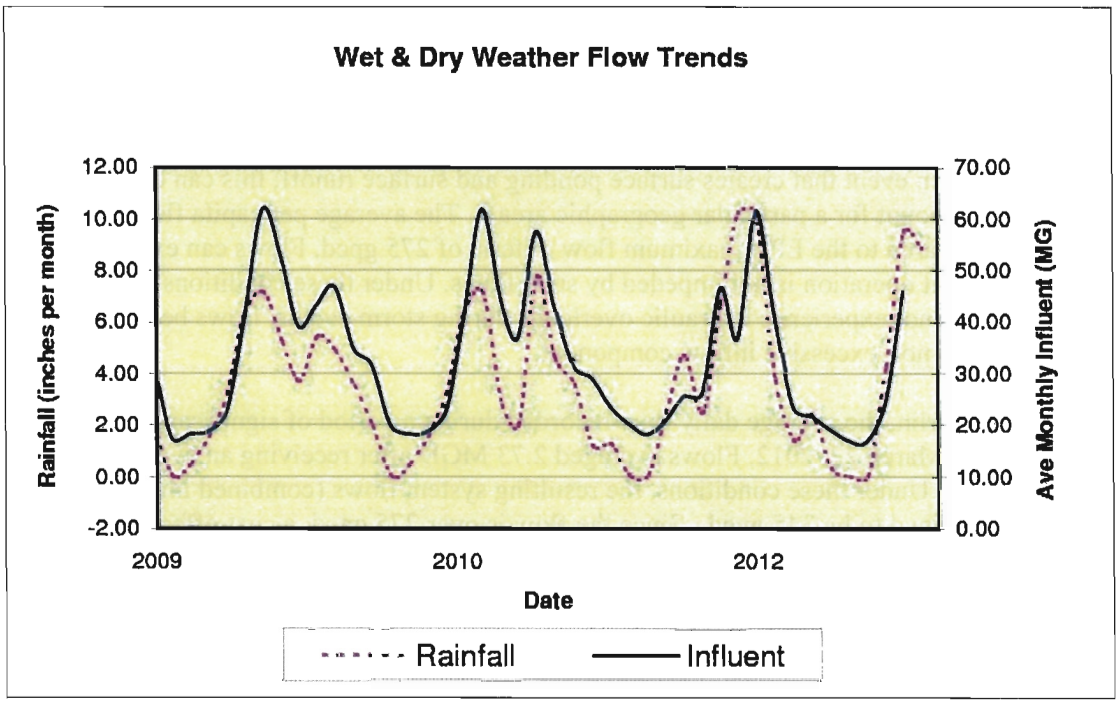
TABLE 4.1.2
SUTHERLIN WWTP (2012) EXISTING INFLOW RATES

Parameter	Flow Values		Unit Design Factors
Population	7,905		
Base Sewage	0.617 MGD	78 gpcd	
Base Infiltration	0.107 MGD	14 gpcd	
AAF	1.07 MGD	135 gpcd	1.5
ADWF	0.72 MGD	92 gpcd	1.0
AWWF	1.37 MGD	172 gpcd	1.9
$MMDWF_{10}$	1.23 MGD	156 gpcd	1.7
$MMWWF_5$	1.70 MGD	215 gpcd	2.3
PW	2.94 MGD	371 gpcd	4.1
$PDAF_5$	5.57 MGD	705 gpcd	7.7
PIF	7.30 MGD	923 gpcd	10.1

4.1.3 Infiltration and Inflow

Flow Trends
Sutherlin receives very little rainfall from June through September. Monthly average flows at the WWTP for June are lower than in May, reflecting the reduction in rain and subsequent I/I. Wet weather flows are heavily influenced by rain and the condition of the system, with the highest flows occurring between December and March. The average wet weather monthly plant inflow volume has a strong correlation to total monthly wet weather rainfall for the years of the study period.

FIGURE 4.1.3.1
WWTP PLANT FLOW TRENDS 2008-2012



The WWTP is operating at less than the original design capacity during dry weather flows and exceeds the plant capacity during wet weather flows. The close correlation between plant flows and rainfall indicate that I/I is still a primary factor in the hydraulic overload.

EPA Non-excessive Infiltration
The EPA guidelines establish procedures on how to determine whether excessive I/I exists, and how to certify that excessive I/I has been sufficiently reduced through sewer rehabilitation. Infiltration occurs when groundwater enters a sewer system through broken pipes, defective pipe joints or illegal connections of foundation drains. System flows are analyzed under various conditions and compared to benchmarks that have been established for acceptable sanitary sewage flow rates.

Non-excessive infiltration is analyzed by investigating plant flows during periods of seasonal high groundwater with little sustained rainfall. Seven to fourteen-day periods during winter months of high groundwater (December through May) were identified where little or no rainfall is measured. The

average per capita flow for the system is calculated and compared to the EPA maximum flow criteria of 120 gpcd. Under these conditions, all flows below 120 gpcd are considered to have a non-excessive infiltration component.

The 7-day period with little or no rainfall and the highest flow day occurred during the week ending January 26, 2011. It is assumed that groundwater levels were high during this period. The highest flow day of that week was 1.443 MGD. Based on a population of 7,905, the resulting flow rate is calculated at 183 gpcd. Since the flow is over 120 gpcd, a cost-effective analysis is needed to determine if the infiltration is excessive.

EPA Non-excessive Inflow

Non-excessive inflow is analyzed by investigating plant flows during periods of intense winter rainfall. Major rainfall events and the resulting system flows during winter months are analyzed. Inflow is surface runoff that enters a sewer system through manhole covers, cleanout covers, cross connections between storm sewers and sanitary sewers, and illegal connections of roof drains, yard drains, or catch basins. EPA's non-excessive inflow criteria is based on "the average daily flow during periods of significant rainfall (i.e. and storm event that creates surface ponding and surface runoff; this can be related to a minimum rainfall amount for a particular geographic area)". The average per capita flow for the system is calculated and compared to the EPA maximum flow criteria of 275 gpcd. Flows can exceed EPA guidelines if the plant operation is not impeded by such flows. Under these conditions, provided the treatment plant does not experience hydraulic overloads during storm events, flows below 275 gpcd are considered to have a non-excessive inflow component.

For the city of Sutherlin, the average daily flow recorded during a period of significant rainfall occurred the week ending on March 22, 2012. Flows averaged 2.73 MGD after receiving an average rainfall of 0.58 inches per day. Under these conditions, the resulting system flows (combined infiltration and inflow) were determined to be 345 gpcd. Since the flow is over 275 gpcd, a cost-effective analysis is needed to determine if the inflow is excessive.

The EPA I/I analysis is summarized in Table 4.1.3.1

TABLE 4.1.3.1
I/I ANALYSIS SUMMARY

Description of Flow Condition	Flow Rate	EPA Criteria (Maximum Flow)
Base Sewage	78 gpcd	NA
Infiltration (High Ground Water)	183 gpcd	120 gpcd
Inflow (High Rainfall Levels)	345 gpcd	275 gpcd

Since the I/I flow in the system is greater than the EPA guidelines, and the City has completed a significant I/I repair projects over the past few years, additional I/I investigation is required to determine the source of the I/I. Further collection system Sanitary Sewer Evaluation (SSE) is required. A complete SSE program consist of smoke testing, TV of lines, flow evaluation, mapping, development of capital improvement projects, preparation of a cost-effective analysis, determination of funding sources and verification of the effectiveness of the I/I improvement project. An SSE program takes at least five years to complete. Relying solely on an I/I rehabilitation program to bring Sutherlin back into compliance with the NPDES permit and, concurrently, meeting the MAO schedule, is not feasible.

4.2 Wastewater Composition

Population growth and development will increase the organics and silts collected and conveyed to the treatment plant. All wastewater generated in Sutherlin is domestic in quality. No wastewater is collected from industrial processes. Wastewater is generated by residential, commercial and industrial sources. The wastewater composition and load from these separate sources cannot be ascertained, since they are not separately monitored for flows and composition. Monitoring results of influent wastewater represent wastewater combined from these sources. Treatment plant DMRs were reviewed for the years from 2008 to 2012 to determine the BOD_5 and TSS levels within the existing wastewater influent stream.

Analysis of Plant Records
The BOD_5 and TSS influent concentration and loads are summarized in Table 4.2.1 and are based upon DMRs and a sewered population of 7,905 persons. The accuracy of the findings depends wholly upon the accuracy of the DMR reports themselves.

Untreated domestic wastewater typically consists of 110 to 400 mg/L BOD_5 (0.17 to 0.20 ppcd) and 100 to 350 mg/L TSS (0.20 to 0.22 ppcd). In comparison, both the average BOD_5 and TSS concentrations in the city of Sutherlin's influent wastewater are within the typical range during wet weather and dry weather periods. The average BOD_5 and TSS unit loading at the WWTP are within the normal ranges for similar communities.

TABLE 4.2.1
WWTP INFLUENT CONCENTRATIONS AND LOADS

2008-2012 Data	WET WEATHER			DRY WEATHER		
PARAMETER	Average	Range		Average	Range	
BOD_5						
mg/L	136	30 -	416	229	40 -	432
Ppd	1,553	582 -	5,605	1,394	516 -	2,608
Ppcd	0.20	0.07 -	0.71	0.18	0.07 -	0.33
TSS						
mg/L	146	44 -	774	225	70 -	637
Ppd	1,661	530 -	4,481	1,537	321 -	3,368
Ppcd	0.21	0.07 -	0.61	0.19	.04 -	0.43

Listed below in Table 4.2.2 are average wastewater composition values based on the 2008-2012 study period data from plant records with outlying values removed.

TABLE 4.2.2
WASTEWATER DESIGN COMPOSITION VALUES

Unit Wastewater Values	Concentration	Load
BOD_5 Avg. Day	183 mg/L	1,468 ppd
BOD_5 Max. Ave. Month	337 mg/L	2,197 ppd
BOD_5 Max. Day	416 mg/L	5,605 ppd
TSS Avg. Day	200 mg/L	1,599 ppd
TSS Max. Ave. Month	392 mg/L	2,168 ppd
TSS Max. Day	530 mg/L	4,841 ppd

4.3 Projected Wastewater Characteristics

Future sanitary sewer flows generated within the city come from a wide variety of collection system users. The average wastewater flows from these users are expected to grow at approximately the same rate as the overall population. Therefore, future sanitary flows are projected by applying the anticipated population growth rate to the current sanitary flows. Projections for ADWF, $MMDWF_{10}$ and AAF are made using a unit design value method based on gpcd extrapolated from the DMRs.

Projections of future wet weather flows require additional consideration due to the variability of I/I rates among existing and future developments. When utilizing a straight unit design factor based on existing peaking factors, future peak flows are overestimated. Therefore, 2040 peak flows are estimated using current wet weather I/I rates among for existing portions of the collection systems while using lower rates in areas with new sewers.

The current PIF of 7.30 MGD is greatly influenced by the presence of collection system deficiencies in the older parts of town. Older construction can be subject to poor or no joint gaskets, ridge pipe materials that develop cracking, and poor construction practices. Since improved construction materials and techniques in the new portions of the collection system will greatly reduce I/I sources, the projections of future peak wet weather flow must account for lower wet weather I/I rates in new developments. For the purposes of wet weather flow projections, new developments are assigned an EPA non-excessive I/I wet weather unit design factor of 395 gpcd (120 gpcd infiltration + 275 gpcd inflow). Future dry weather flow unit design factors are based on the existing AAF of 135 gpcd.

To maintain consistency with the growth of the PIF relative to the ADWF, the wet weather flows are estimated by interpolating a linear relationship between the PIF and the AAF on a logarithmic flow probability chart. $MMWWF_5$ is calculated at 8.3% probability, peak week based at 1.9% probability and $PDAF_5$ based at 0.27% probability. A comparison of the 2012 and 2037 flow rates is shown in Figure 4.3.1 and flow projections are summarized in Table 4.3.2.

FIGURE 4.3.1
FUTURE PEAK FLOWS AND EXISTING PEAK FLOWS

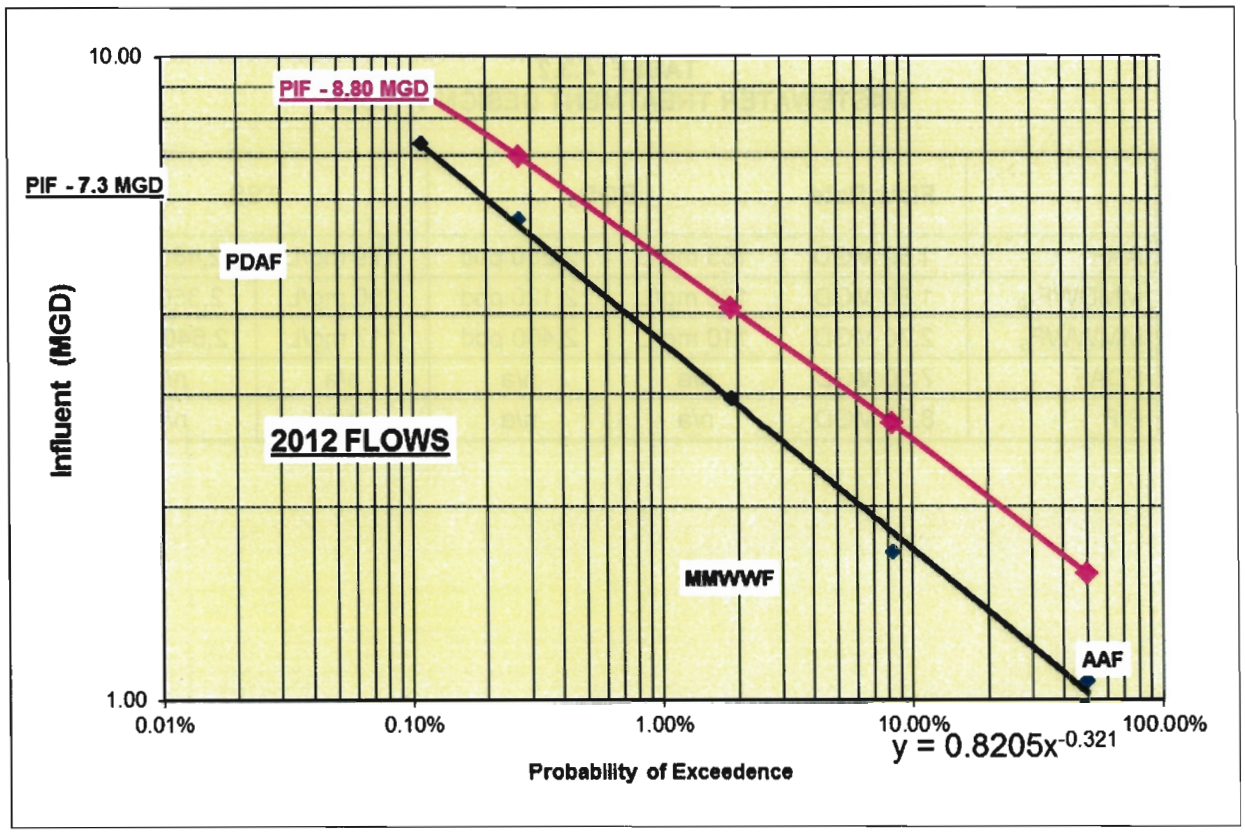


TABLE 4.3.1
WASTEWATER FLOWS AND LOADS

Parameter	2012	2037	
Population	7,905	12,100	
Base Sewage	0.617 MGD	80 gpcd	0.93 MGD
Base Infiltration	0.11 MGD	14 gpcd	0.16 MGD
AAF	1.07 MGD	135 gpcd	1.57 MGD
ADWF	0.72 MGD	92 gpcd	1.06 MGD
AWWF	1.37 MGD	173 gpcd	2.01 MGD
$MMDWF_{10}$	1.23 MGD	156 gpcd	1.80 MGD
$MMWWF_5$	1.70 MGD	233 gpcd	2.70 MGD
Peak Average Week	2.94 MGD	351 gpcd	4.07 MGD
$PDAF_5$	5.57 MGD	604 gpcd	7.00 MGD
PIF	7.30 MGD	759 gpcd	8.80 MGD

Treatment unit design values are extrapolated from Tables 4.2.2 and 4.3.1. Table 4.3.2 summarizes the recommended design values for unit sizing. These values should be verified in the pre-design report based on completion of I/I projects and updated influent flow data.

TABLE 4.3.2
WASTEWATER TREATMENT DESIGN VALUES

	Flow Rate	**BOD_5**		**TSS**	
AAF	1.62 MGD	165 mg/L	2,250 ppd	179 mg/L	2,450 ppd
$MMDWF_{10}$	1.80 MGD	135 mg/L	2,120 ppd	150 mg/L	2,350 ppd
$MMWWF_5$	2.70 MGD	110 mg/L	2,460 ppd	117 mg/L	2,640 ppd
PDAF	7.00 MGD	n/a	n/a	n/a	n/a
PIF	8.80 MGD	n/a	n/a	n/a	n/a

Basis of Planning

Section

5

Section 5

Basis of Planning

5.1 Regulatory Environment

The Clean Water Act (CWA) prohibits discharges of wastewater to waters of the state without a National Pollutant Discharge Elimination System (NPDES) permit. NPDES permits contain effluent limits that are developed to protect the beneficial uses. NPDES permits are generally renewed every 5 years, at which time any changes to the rules will be included in the renewed permit. The U.S. Environmental Protection Agency (EPA) has delegated NPDES permitting authority in Oregon to the Oregon Department of Environmental Quality (DEQ).

The Clean Water Act (CWA) requires states to develop water quality standards. Oregon Administrative Rules (OAR) Chapter 340 Division 41 contains Oregon's water quality standards. These standards are benchmarks established to assess whether the quality of Oregon's rivers and lakes is adequate for beneficial uses. Oregon DEQ also collects water quality data through its ambient monitoring network. This data is compared against the standard to develop a water quality assessment report that includes a list of impaired waters. This list is known as the "303d list". When 303d-listed, water quality standards are violated and beneficial users are impacted. DEQ then develops a plan to restore the impaired waters. This plan is known as the "October 2006 Umpqua Basin Total Maximum Daily Load (TMDL) and Water Quality Management Plan (WQMP)". The WQMP contains Waste Load Allocations (WLAs) for point sources, such as wastewater treatment plants.

The OARs also contain both statewide and basin-specific minimum design criteria and rules regarding sanitary sewage overflows. A discussion of these regulations, known future changes, and potential future changes is included in this section.

5.1.1 Beneficial Uses

The city of Sutherlin's wastewater treatment plant discharges to Calapooya Creek at River Mile 9.8, which is in the Umpqua Basin. Per OAR 340-041-0314, the beneficial uses for Calapooya Creek are contained in Table 320A, and Figures 320A and 320B. The applicable beneficial uses from Table 320A are summarized below:

- Public and Private Domestic Water Supply [1]
- Industrial Water Supply
- Irrigation
- Livestock Watering
- Fish and Aquatic Life
- Wildlife and Hunting
- Fishing
- Boating
- Water Contact Recreation
- Aesthetic Quality, and
- Hydro Power.

[1] With adequate pretreatment (filtration & disinfection) and natural quality to meet drinking water standards.

The applicable fish use designations are salmon and trout rearing and migration (Figure 320A) and no spawning use (Figure 320B).

5.1.2 CWA Water Quality Assessment, Integrated Report, and 303(d) List

Section 305(b) of the Clean Water Act (CWA) requires DEQ to assess water quality in Oregon and publish a report on the overall condition of waters (Integrated Report). DEQ assigns an assessment status category to each water body where data are available to evaluate. Water bodies that do not meet water quality standards are Water Quality Limited and are assigned Category 4 or Category 5. Water bodies in Category 5 need pollutant Total Maximum Daily Loads (TMDLs) developed, and comprise the Section 303(d) list.

The following is DEQ's assessment of the water quality in Calapooya Creek in the vicinity of the city of Sutherlin's wastewater discharge from the 2010 Integrated Report:

TABLE 5.1.2.1
CALAPOOYA CREEK 303d LIST

River Miles	Parameter	Season	Status	Assessment Action
0 to 12.7	Alkalinity	Year Around	Cat 3B: Potential concern	No 2010 action
0 to 36.2	Ammonia	Year Around	Cat 2: Attaining some criteria/uses	No 2010 action
0 to 36.2	Antimony	Year Around	Cat 3: Insufficient data	No 2010 action
0 to 36.2	Arsenic (tri)	Year Around	Cat 3B: Potential concern	No 2010 action
0 to 36.2	Barium	Year Around	Cat 3: Insufficient data	No 2010 action
0 to 36.2	Beryllium	Year Around	Cat 3B: Potential concern	No 2010 action
0 to 36.2	Cadmium	Year Around	Cat 3: Insufficient data	No 2010 action
0 to 36.2	Chloride	Year Around	Cat 3: Insufficient data	No 2010 action
0 to 18.7	Chlorophyll a	Summer	Cat 2: Attaining some criteria/uses	No 2010 action
0 to 18.7	Chlorophyll a	Fall/Winter/Spring	Cat 3: Insufficient data	No 2010 action
0 to 36.2	Chromium (hex)	Year Around	Cat 3: Insufficient data	No 2010 action
0 to 36.2	Copper	Year Around	Cat 3: Insufficient data	No 2010 action
0 to 36.1	Dissolved Oxygen	Year Around (Non-spawning)	Cat 4A: Water quality limited, TMDL approved	Delisted - TMDL approved
0 to 24.8	Dissolved Oxygen	October 15 - May 15	Cat 5: Water quality limited, 303(d) list, TMDL needed	No 2010 action
0 to 18.7	E. Coli	Summer	Cat 2: Attaining some criteria/uses	No 2010 action
0 to 18.7	E. Coli	Fall/Winter/Spring	Cat 4A: Water quality limited, TMDL approved	Delisted - TMDL approved

River Miles	Parameter	Season	Status	Assessment Action
0 to 36.2	Iron	Year Around	Cat 5: Water quality limited, 303(d) list, TMDL needed	No 2010 action
0 to 36.2	Lead	Year Around	Cat 3: Insufficient data	No 2010 action
0 to 36.2	Manganese	Year Around	Cat 3B: Potential concern	No 2010 action
0 to 36.2	Mercury	Year Around	Cat 3: Insufficient data	No 2010 action
0 to 36.2	Nickel	Year Around	Cat 3: Insufficient data	No 2010 action
0 to 18.7	pH	Fall/Winter/Spring	Cat 2: Attaining some criteria/uses	No 2010 action
0 to 18.7	pH	Summer	Cat 4A: Water quality limited, TMDL approved	Delisted - TMDL approved
0 to 36.1	Phosphate Phosphorus	Summer	Cat 3B: Potential concern	No 2010 action
0 to 36.2	Selenium	Year Around	Cat 3: Insufficient data	No 2010 action
0 to 36.2	Silver	Year Around	Cat 3: Insufficient data	No 2010 action
0 to 36.1	Temperature	Year Around (Non-spawning)	Cat 4A: Water quality limited, TMDL approved	Delisted - TMDL approved
0 to 36.2	Thallium	Year Around	Cat 3: Insufficient data	No 2010 action
0 to 36.2	Zinc	Year Around	Cat 3: Insufficient data	No 2010 action

NOTES:
Category 1: All standards are met. (This category is not used.)
Category 2: Attaining - Some of the pollutant standards are met.
Category 3: Insufficient data to determine whether a standard is met.
- 3B: Potential concern - Some data indicate non-attainment of a criterion, but data are insufficient to assign another category.

Category 4: Water is water quality limited but a TMDL is not needed. This includes:
- 4A: TMDL approved - TMDLs needed to attain applicable water quality standards have been approved.
- 4B: Other pollution control requirements are expected to address all pollutants and will attain water quality standards.
- 4C: Impairment is not caused by a pollutant (e.g., flow or lack of flow is not considered a pollutant.)

Category 5: Water is water quality limited and a TMDL is needed, Section 303(d) list.

5.1.3 Umpqua Basin Total Maximum Daily Load (TMDL)

The Umpqua Basin TMDL was issued on October 31, 2006 and was approved by EPA on April 13, 2007. Calapooya Creek is within the Umpqua Basin. This TMDL addresses bacteria, temperature, nutrients, and bio-criteria, and has a separate chapter regarding Diamond Lake. Waste Load Allocations (WLAs) for the city of Sutherlin's wastewater treatment plant are summarized in Table 5.1.3.1:

TABLE 5.1.3.1
SUTHERLIN STP WLAS IN THE UMPQUA BASIN TMDL

Parameter	Applicable Time Period	Waste Load Allocation
E. Coli Bacteria	Year Round	Not to exceed a monthly log mean of 126 organisms per 100ml. No single sample exceeding 406 per 100 ml.
Temperature	May 1 – October 31	$H_{wla} = (0.1)(Q_e + Q_r)C_f$ Where, H_{wla} = heat load WLA, million kcal/day Q_e = effluent flow rate, ft^3/sec Q_r = river flow rate, upstream, ft^3/sec C_f = conversion factor, 2.446 million kcal-s / °C·ft^3·day
Total Phosphorus	May 1 – October 31	Zero. Point sources may, at their option, collect additional data, perform the necessary evaluations and request a portion of the reserve capacity.

E. Coli Bacteria
The TMDL wasteload allocation is the same as the current permit requirement. Therefore, this plan will include provisions to continue to meet this requirement.

Temperature
The City's NPDES permit prohibits discharge from June 1 through October 31. During this period, all wastewater is required to be used for irrigation at the Umpqua Golf Resort. Assuming this prohibition remains in future permits, compliance with the temperature waste load allocation would only be an issue in May. However, DEQ's analysis is that there is no reasonable potential to exceed the 18° C non-spawning criterion during May (See attached DEQ memo.).

While the TMDL did not address the spawning period (October 15 through May 15), DEQ also evaluated the potential to exceed the spawning criterion (13° C) during this period. DEQ found that not only there is a potential to exceed the spawning criterion, but also that thermal reductions and/or thermal credit trading is needed to meet the excess thermal load limit that will likely be in the city's next NPDES permit renewal.

Additionally, DEQ found that the discharge meets DEQ's thermal plume requirements for acute impairment or instantaneous lethality, thermal shock, and migration blockage. However, additional information is needed to determine if the thermal plume causes an impairment to an active salmonid spawning area where spawning redds are located or are likely to be located.

Total Phosphorus
Again, the City's NPDES permit prohibits discharge from June 1 through October 31. During most of TMDL development, DEQ understood that there was adequate capacity and established the zero

phosphorus WLA. However, late in TMDL development, DEQ discovered that the Umpqua Golf Resort does not have adequate capacity discharge during October. Therefore, at the City's request, DEQ collected additional data to support a WLA during October. DEQ completed a Calapooya Creek Analysis Report (January 8, 2009) that determined acceptable phosphorus WLAs as follows:

TABLE 5.1.3.2
OCTOBER DISCHARGE LIMITATIONS

Parameter	Weekly Average Creek Flow (Qr) at USGS Gage Calapooya Creek near Oakland 143100000	Weekly Average Limit lb/day
Total Phosphorus	< 10	0
	> 10 and < 250	= Qr*0.25
	> 250	44

DEQ also found that thermal reductions and/or thermal trading would also be needed to allow an October discharge.

5.1.4 Water Quality Limited Pollutants not addressed by the TMDL

As mentioned above, Category 5 parameters are important in wastewater planning because, while these parameters must be addressed, DEQ has not yet completed a TMDL to determine waste load allocations. Therefore, the facilities plan must determine future requirements on a "best guess" basis. Dissolved oxygen (DO) during the spawning period (October 15 through May 15) is Category 5. Additionally, the TMDL did not address the spawning period temperature criterion (13° C).

In 1997 DEQ conducted studies to determine the impact of the discharge on dissolved oxygen in Calapooya Creek. These studies determined that the wastewater was likely a contributing factor during November and during low flow periods in May. The study also determined conditions under which the discharge would cause a DO depletion of 0.1 mg/L or less. The results of this study were used to establish the additional BOD_5, TSS and flow restrictions in the current NPDES permit. DEQ considers a 0.1 mg/L or less impact not measureable. Therefore, the additional BOD_5 and TSS loading restrictions in November and flow restrictions during low flow in May likely address the DO issue with regard to Sutherlin's discharge. See discussion below regarding compliance with the existing permit conditions.

5.1.5 Mixing Zone Review

On October 9, 2008, DEQ staff conducted a mixing zone study. This study determined that the minimum dilution at the edge of the zone of initial dilution (ZID) is about 2 and the average dilution at the edge of the mixing zone is about 8. The stream flow at the time of the study was about 19 cfs.

5.1.6 Antidegradation Review

OAR 340-41-026 (2) requires that, unless otherwise approved by the Environmental Quality Commission, growth and development shall be accommodated within the existing permitted loads by the application of increased treatment and control efficiency. The city of Sutherlin's BOD_5 and TSS load limits are flow based. For evaluation purposes, the projected 20-year (2035) average monthly wet weather flow condition is used during the lowest stream flow as a critical case. Under this condition, the effluent BOD_5 and TSS concentration required to meet the mass loads in the current permit is 31 mg/L, which is well within the

capabilities of a modern wastewater treatment plant. Therefore, this requirement will not be a design consideration.

5.1.7 Potential Future Regulatory Issues

Future regulatory requirements cannot be thoroughly analyzed, as they have not yet been implemented. This section discusses some of the potential future requirements that could become important to municipalities based on current regulatory trends.

Toxics and persistent pollutants are currently a focus of study for DEQ. In 2010, DEQ required Oregon's 52 largest municipal wastewater treatment plants to analyze the treated wastewater for persistent pollutants. In 2011, DEQ established new water quality criteria to protect human health and aquatic species. DEQ is now requiring all municipal wastewater treatment plants with a design flow greater than 1 MGD to conduct testing for these toxics. This includes the city of Sutherlin. Based on the results of these tests, municipal treatment systems may be required to implement source reduction programs and/or construction of additional treatment capabilities.

Sutherlin conducted Whole Effluent Toxicity (WET) tests in September 2007, October 2008, October 2009, and November 2010. The tests showed no acute toxicity at 100 percent effluent. The IC_{25} (Inhibition Concentration) for all tests was above 100 percent effluent except the November 2010 test, which was 76.9 percent effluent. The IC_{25} is the concentration of effluent that would cause a 25 percent reduction of a non-quantal biological measurement (i.e. growth, reproduction, etc). These results indicate that the effluent does not contain toxic substances.

Not all potential contaminants are addressed in the TMDL, so it is possible that requirements could arise in the future. These could include mixing zones, metals, results of reasonable potential analyses, ammonia, contaminants of emerging concern, wet weather policy (including blending), and aquatic life criteria. Other microconstituents could arise in regulatory practice as well.

Every two years, DEQ is required to report a new assessment of Oregon's waters. These assessments must show that water quality standards are being met for the water body, or a new TMDL must be established. This could lead to additional future TMDLs for Calapooya Creek.

5.2 NPDES Permit

5.2.1 Current NPDES Permit Requirements

The city of Sutherlin operates its wastewater system under National Pollutant Discharge Elimination System (NPDES) Waste Discharge Permit No. 101993, issued October 24, 2006 by the Department of Environmental Quality (DEQ). This permit expired September 30, 2011, but has been administratively extended. A copy of the City's NPDES permit is included in Appendix A. A summary of regulatory requirements within the NPDES permit is provided below.

From June 1 through October 31, no discharge is allowed to waters of the State (Outfall 001, Calapooya Creek). During this time, all effluent is required to be disposed of through Recycled Water Reuse (Outfall 002). Prior to land application, recycled wastewater must receive at least level II treatment. Level II treatment is defined as no more than 240 total coliform organisms per 100 mL in two consecutive samples, and a 7-day median of 23 total coliform organisms per 100 mL.

From November 1 through May 31, discharge must meet the effluent limitations in Table 5.2.1.1 Mass load effluent limits for BOD_5 and TSS are based on weekly average stream flow during November.

TABLE 5.2.1.1
NPDES PERMIT WASTE DISCHARGE LIMITATIONS

Parameter	Ave Effluent Conc mg/l		Monthly Ave lb/day	Weekly Ave lb/day	Daily Max lb/day
	Monthly	Weekly			
Nov. 1 to Nov. 30					
BOD_5	30	45	440		
TSS	30	45	440		
Steam Flow					
<45 CFS				170	260
45 - 60 CFS				320	480
>60 - 96 CFS				440	660
>96 CFS				660	880
Dec. 1 - May 31					
BOD_5	30	45	560	840	1100
TSS	30	45	560	840	1100
Discharge in May allowed only while daily flow >82 CFS					
Year Round					
E Coli	126 org./100 ml No single sample > 406 org./100 ml.				
pH	6.0 - 9.0				
BOD5 & TSS eff.	85%				
Ammonia-N	21 mg/L daily maximum and 7.8 mg/L monthly average				
Total Chlorine Residual	0.04 mg/L daily maximum and 0.01 mg/L monthly average				

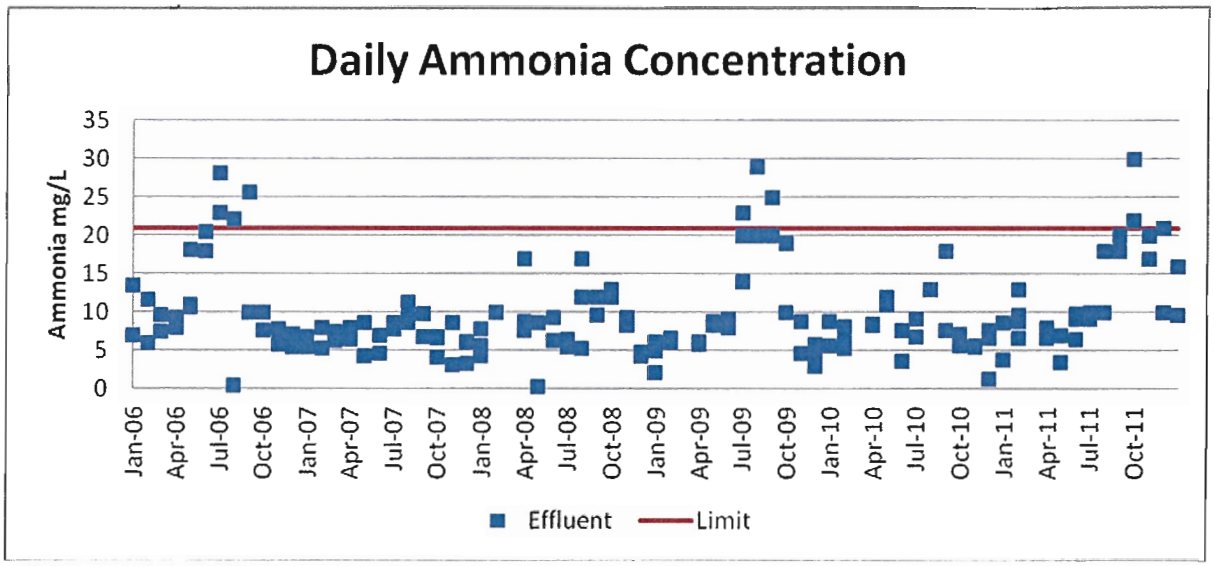
5.2.2 Compliance with Current NPDES Permit Requirements

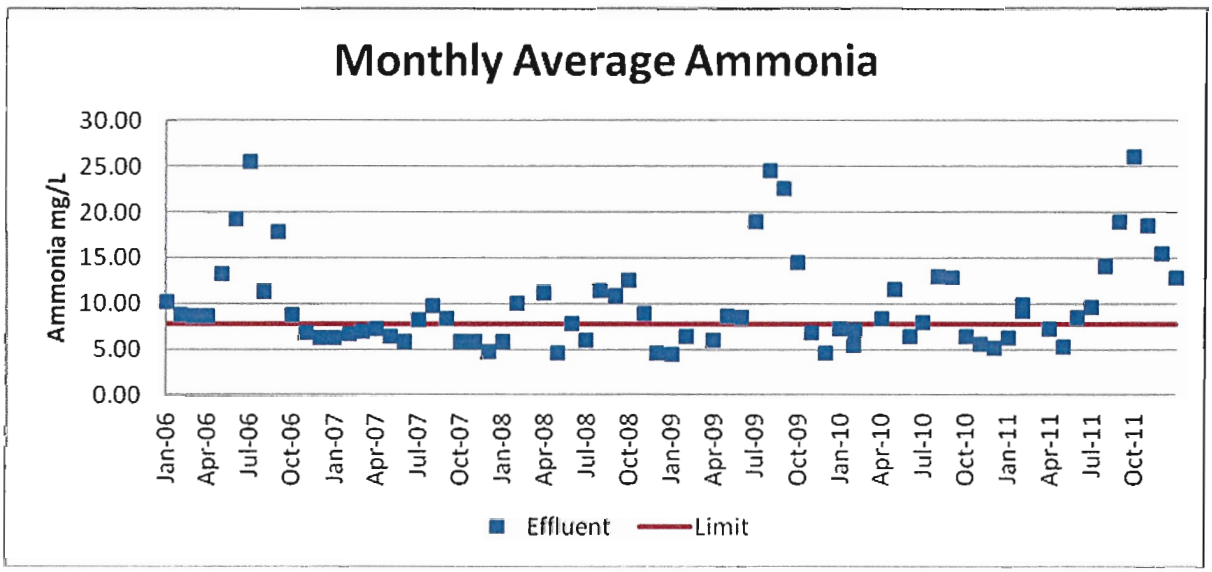
The existing facility has not been meeting the limits for the discharge restrictions of total residual chlorine and ammonia. Sutherlin and DEQ have entered into a MAO that allows higher interim limits and requires Sutherlin to address these issues. The MAO allows the summer discharge and requires Sutherlin to address this issue.

The city cannot comply with the summertime discharge prohibition (June 1 and October 31) because the land application system is too small to land apply all recycled water during this time period. The NPDES permit also prohibits discharge when the flow in Calapooya Creek is less than 82 cfs. The city cannot comply with this requirement for the same reasons.

To comply with the chlorine limit, the city has installed a temporary dechlorination system. However, this system does not meet DEQ's requirement for automation, alarms and redundancy. Therefore, a more permanent solution is needed.

The following charts compare the effluent ammonia concentrations to the limits:





These charts show that additional ammonia removal is needed.

5.2.3 Anticipated NPDES Permit Limits

As discussed above, additional phosphorus limits and temperature limits are expected.

The recycled water use rules have changed since issuance of the permit. When renewed, the permit will include requirements for Class C recycled water, which are very similar to the current requirements for Level II recycled water. One notable difference is that there is no specific restriction prohibiting the use of Class C recycled water on golf courses with contiguous homes. Instead, there are minimum setbacks that must be maintained. There must be a minimum of 70 feet from the edge of the site used for irrigation and the site property line. There must also be a minimum of 100 feet from the edge of an irrigation site to a water supply source used for human consumption.

5.3 Design Criteria and Considerations

Design criteria for future conveyance system expansions are based on topography and the estimated future flows discussed in Section 4. Treatment planning must take into account existing and projected loadings and flows, and regulatory requirements. General design considerations incorporated in the development and evaluation of alternatives in Section 6 are discussed below.

Design Period

The design period must be long enough to ensure the new facilities will be adequate for future needs, but short enough to ensure effective use within their economic life. The improvement plan for serving the properties within the Urban Growth Boundary (UGB) will be based on a design period of 20 years for pump stations. Gravity collection line sizing will be based on ultimate build-out. Treatment facility recommendations will be based on a 20-year planning period from the date the WWFP is commissioned.

Collection System

Gravity Sewers
Collection systems must be designed to consider natural ground slope, subsurface conditions, capacity requirements, minimum slope considerations, minimum flow velocities required to maintain solids suspension, and potential sulfide and odor generation. Collection sewers should be designed for ultimate development of areas.

Force Mains

The City currently operates numerous force mains from its sanitary sewer pump stations and STEP systems. DEQ guidelines for force mains are important to help ensure proper performance and longevity.

Pump Stations
Design of pump (lift) stations is a critical element of sanitary sewer collection systems. New pump stations must be designed to meet the *Oregon Standards for Design and Construction of Wastewater Pump Stations*, issued in May 2001 by DEQ. The pump station must be able to handle the peak instantaneous flows in the system without overflowing. The pump stations should be designed so as not to increase the total sulfide generation potential of the collection system. Contemporary design practice requires some wet well storage of wastewater plus retention in the force main, both of which tend to increase the potential sulfide generation when supplemental aeration is not provided.

To minimize sulfide generation, wet wells should be as small as possible while still allowing for future growth. Wet well detention times of 30 minutes or less are recommended to avoid sulfide generation. When detention times in the pump station force main exceed 25 to 30 minutes, a system to control hydrogen sulfide generation and the accompanying odor and corrosion problems is recommended.

Pump stations should have redundant pump equipment and provisions for emergency generator operation. Power outage frequency and duration must be considered in pump station design to ensure that overflows do not occur due to power loss. In some cases, a portable generator connected to the pump station with a manual transfer switch will suffice. In larger pump stations, a permanent standby generator may be required. Level controls should include a redundant high wet well level sensor.

DEQ Design Standards
DEQ issued the current version of *Oregon Standards for Design and Construction of Wastewater Pump Stations* in 2001. The EPA Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability were used to determine the minimum number and sizing of components using the guidelines for Reliability Level One facilities.

Other
Wastewater treatment facilities, including pump stations, are also regulated under National Fire Protection Association (NFPA) 820, *Fire Protection in Wastewater Treatment and Collection Facilities*. OSHA Permit Required Confined Spaces Standard 29 CFR 1910.146 limits individual access to spaces that might trap a person or contain noxious atmospheres. The influent wet well qualifies as a Permit Required Confined Space and requires special equipment and multiple personnel present for entry. The existing influent screen structure was reclassified by SAIF in 2013 and is no longer listed as a confined space.

Wastewater Treatment Facility

Primary consideration will be given to the degree of treatment required to meet the discharge requirements of the receiving water, and to sufficient sizing of the facility to handle future projected peak hydraulic and organic loads.

Flexibility
Conveyance and treatment design should allow for flexibility in operation and maintenance. The treatment plant operator must have the ability to alter plant flows around the major process units without significantly degrading effluent quality. This goal can be achieved by providing redundant units and multiple interconnections between units when appropriate. Conveyance and treatment equipment design should also be such that maintenance, both routine and emergency, can be performed without excessively loading other components. Flexibility is also needed to ensure that discharge requirements can be met during changing influent conditions and to allow construction and connection of new process units as needed.

Plant Reliability Criteria

Reliability of treatment processes depends on proper application of unit loading factors and conservative selection of equipment to ensure long life and minimum maintenance costs. Each unit process should be selected based on its capabilities to effectively treat the waste characteristics for the specific application. Capabilities of the treatment plant operator and the community should also be considered. Processes that require a high degree of manual labor and specialized instrumentation should be avoided in most cases. Redundancy is also a key factor in reliability. This proposed facility will be designed to meet EPA Reliability I standards because the facility discharges into a "public water supply, shellfish, or primary contact recreation waters, or as a result of its volume and/or character, could permanently or unacceptably damage or affect the receiving waters or public health if normal operations were interrupted." EPA reliability criteria are interpreted differently in Western Oregon by the DEQ, since PDF and ADWF can be atypically high as compared to other regions. This is discussed in more detail in Section 6 under unit processes.

For components included in the design of Reliability Class I, the following backup requirements apply:

- **Mechanically Cleaned Bar Screens.** A backup bar screen, designed for mechanical or manual cleaning, shall be provided. Facilities with only two bar screens shall have at least one bar screen designed to permit manual cleaning.

- **Pumps.** A backup pump shall be provided for each set of pumps performing the same function. The capacity of the pumps shall be such that, with any one pump out of service, the remaining pumps will have the capacity to handle the peak flow.

- **Comminution Facility.** If comminution of the total wastewater flow is provided, an overflow bypass with a manually-installed or mechanically-cleaned bar screen shall be provided. The hydraulic capacity of the comminutor overflow bypass should be sufficient to pass the peak flow with all comminution units out of service.

- **Primary Sedimentation (Clarifier) Basins.** The units should be sufficient in number and size so that, with the largest-flow-capacity unit out of service, the remaining units should have a design flow capacity of at least 50 percent of the total design flow.

- **Final Sedimentation (Clarifier) Basins and Trickling Filters.** The units shall be sufficient in number and size so that, with the largest-flow-capacity unit out of service, the remaining units shall have a design flow capacity of at least 75 percent of the total design Peak Day flow. These units are sized for peak day winter flows and large enough to treat MMDWF with one out of service.

- **Aeration Basin.** A backup basin will not be required; however, at least two equal-volume basins are recommended. A single basin is permissible.

- **Aeration Blowers, Rotors or Mechanical Aerators.** There shall be a sufficient number of blowers or mechanical aerators to enable the design oxygen transfer to be maintained with the largest-capacity unit out of service. It is permissible for the backup unit to be an uninstalled unit, provided that the installed units can be easily removed and replaced. However, at least two units shall be installed.

- **Air Diffusers.** The air diffusion system for each aeration basin shall be designed so that the largest section of diffusers can be isolated without measurably impairing the oxygen transfer capability of the system.

- **Disinfectant Contact Basins or Units.** The units shall be sufficient in number and size so that, with the largest-flow-capacity unit out of service, the remaining units shall have a design flow capacity of at least 50 percent of the total design flow.

- **Electrical Power Sources.** Two separate and independent sources of electric power shall be provided to the plant either from two separate utility substations or from a single substation and a works-based generator located at the plant. As a minimum, the capacity of the backup power source for the treatment plant shall be sufficient to operate all vital components and critical lighting and ventilation during peak wastewater flow conditions.

Operability

Operation of a wastewater system entails considerable responsibility and cost while providing public health benefits. For these reasons, personnel assigned to operate and maintain a treatment facility must be trained appropriately. The more sophisticated the process or equipment, the greater the level of expertise needed. Qualified individuals are usually available in metropolitan areas, as is financial support for their employment. However, small communities often have a problem in finding the personnel and the money with which to pay them. Consequently, the selection of a treatment process and equipment should reflect the regional and local level of training of operations and maintenance.

Durability
Conveyance and treatment systems should consist of materials and equipment that are capable of satisfactory performance over the entire design life of the wastewater system components. The selection of durable wastewater system components is a matter of judgment based on a number of factors including type and intensity of use, type and quality of materials used in construction, quality of workmanship during the initial installation, and expected maintenance to be performed during life of the component.

Capacity
Individual treatment components must be capable of handling the hydraulic flow through the plant during peak wet weather periods and be capable of being sized to treat the mass loads projected for the facility. The following guidelines will be used in this plan:

- In general, all processes after the headworks are designed to operate for peak daily flow.
- Influent pump stations are designed to operate for peak instantaneous flow.
- The headworks should be sized for peak instantaneous flows.
- Primary clarifiers, when present, should be sized for peak daily flows.
- Aeration basins should be sized using modeling to generate desired treatment in the final effluent.
- Per DEQ, the secondary clarifiers should be sized for either the peak day with all clarifiers operational or the MMDWF with the largest clarifier off-line, whichever results in the greater treatment capacity. Overflow rates for the separate seasons should be used, e.g. 1200 for winter and 800 for summer.
- Per EPA, the secondary clarifiers should be able to handle 75% of peak daily flow with the largest unit out of service.
- The disinfection system should be sized for peak daily flow. The contact chamber should be sized for at least 15 minutes of contact time at the peak hour flow, 20 minutes at peak day, or 60 minutes at ADWF, whichever results in the largest basin.
- Sizing of the digester is based on the estimated volume of suspended solids of the incoming mixed liquor, in addition to the holding time required for the digesters. The assumption is made that sludge is held for a minimum 60 days and that biosolids are removed at 2% solids.

Expandability
Expandability is a difficult factor for consideration in the design of wastewater treatment facilities. Designs are created that meet the current regulatory environment. Future regulatory requirements could have dramatic effects on the compliance of even the newest designs. Therefore, expandability is considered from a current regulatory compliance viewpoint. The treatment alternatives considered are expandable as long as specific design capacities of the system are not exceeded.

Miscellaneous
Consideration of site location, daily operational tasks, public perception, health and safety concerns, noise, access to equipment, human factors, and hazardous areas all have to be analyzed when assessing the conveyance and treatment alternatives.

5.4 Basis For Cost Estimate

The cost estimates presented in this Plan will include four components, each of which is discussed in this section. The estimates presented herein are preliminary and are based on the level and detail of planning presented in this study. As projects proceed and as site specific information becomes available, the estimates may require updating.

Construction Costs

The estimated construction costs in this Plan are based on actual construction bidding results from similar work, published cost guides, and other construction cost experience. Reference was made to the drawings of the existing facilities to determine construction quantities, elevations of the major components, and treatment of wastewater during construction. Estimates will be based on preliminary layouts of the proposed improvements.

Future changes in the costs of labor, equipment, and materials may justify comparable changes in the cost estimates presented herein. For this reason, common engineering practices usually tie the cost estimates to a particular index, which varies in proportion to long-term changes in the national economy. The Engineering News Record (ENR) construction cost index is most commonly used. This index is based on the value of 100 for the year 1913. Average yearly values for the past ten years are summarized in Table 5.4.1.

Estimates in this Plan are based on year 2012 costs. Future yearly ENR indices can be used to calculate the cost of projects for their construction year based on the annual growth in the ENR index. Without using the future ENR Index, costs for construction performed in later years should be projected on an increase of three percent per year.

TABLE 5.4.1
ENR INDEX – 2002 TO 2012

Year	Index	% Change
2002	6538	3.07%
2003	6694	2.39%
2004	7126	6.45%
2005	7421	4.14%
2006	7751	4.45%
2007	7966	2.77%
2008	8310	4.32%
2009	8570	3.13%
2010	8799	2.67%
2011	9070	3.08%
2012	9308	2.62%
	Avg. Annual %	3.55%

Contingencies

A contingency factor equal to 15% of the estimated construction cost has been added to account for uncertainties present with respect to growth scenarios and topography. In recognition that the cost estimates presented are based on conceptual design, allowances must be made for variations in final

quantities, bidding market conditions, adverse construction conditions, unanticipated specialized investigations and studies, and other difficulties which cannot be foreseen at this time, but may tend to increase final costs.

Engineering

The cost of engineering services for major projects typically includes special investigations, a predesign report, surveying, foundation exploration, preparation of contract drawings and specifications, bidding services, construction management, inspection, construction staking, startup services, and the preparation of operation and maintenance manuals. Depending on the size and type of project, engineering costs may range from 15 to 25% of the contract cost when all of the above services are provided. The lower percentage applies to large projects without complicated mechanical systems. The higher percentage applies to small, complicated projects. The engineering costs for design and construction of this project will average between 18% and 20% of the construction cost.

Legal and Administrative

An allowance of three percent of construction cost has been added for legal and administrative services. This allowance is intended to include internal project planning and budgeting, grant administration, liaison, interest on interim loan financing, legal services, review fees, legal advertising, and other related expenses associated with the project.

Development and Evaluation of Alternatives

Section

6

Development and Evaluation of Alternatives

Section 6

An evaluation in terms of condition and capacity has been completed for the existing wastewater conveyance system and treatment plant processes. The City faces issues with aging equipment and structures, wintertime flow capacity and increasingly more restrictive permit requirements triggered by Total Maximum Daily Loads for the Umpqua River and its tributaries. Upgrades to the treatment plant will be required to address plant operational longevity, environmental issues and increasingly more stringent regulatory requirements. The primary objective of this section is to evaluate alternatives and recommend preferred alternatives that address the system conditions, performance and capacity issues. Detailed construction cost estimates and present worth operation and maintenance cost estimates are provided in Appendix C.

6.1 Conveyance System Alternatives

A number of successful I/I projects have been completed through 2011. See Section 3 for a summary of the I/I reduction projects that have been completed. The PDAF dropped from 5.88 MGD in 2009 to 5.57 MGD in 2012. Even with the I/I reduction projects, EPA standards are still exceeded.

I/I Reduction

Continuing I/I reduction projects are anticipated to be conducted annually in the future. Reducing I/I should enable the system to handle current and future flows during the study period without surcharging in the manholes. Records of collection system overflows document that peak flows exceed the collection system's hydraulic and plant treatment capacities periodically during the wet weather season. Mitigating alternatives include lining or replacing the pipe to repair separated joints and holes. In addition, manhole repair is necessary in several cases to correct leaking conditions. Due to the large number of I/I projects completed, new flow measuring and mapping is recommended to determine project locations. The City has a budget line item for I/I repair and improvements. This amount varies from year to year based on available funding and Council approval.

Basic Alternatives

1. **No action.** The existing sewer collection system is mostly capable of handling current peak flows without surcharging of manholes. Additional sewer connections would increase flows, which would increase surcharging. DEQ recommends against allowing manholes to surcharge, due to the buildup of solids on manhole walls which cause unsanitary conditions. This alternative continues to allow extraneous water into the treatment system, thereby overwhelming the capacity of the treatment plant.

2. **Continue I/I investigation and reduction projects as an annual program:** Each year the City continues to repair and improve the wastewater collection system. Significant improvements to peak flows have been realized, per discussions with wastewater system

employees and evaluation of the DMRs. The City is committed to continue I/I reduction projects:

a. The City instituted a TV program for the entire collection system over a five year period (20% per year) and continues to do so on a five-year rotation.

b. Serious maintenance and repair issues will be corrected as funding sources become available. Excess funding from the WWTP improvements projects may be applied toward I/I repair projects.

c. A complete SSE should be completed at least every 10-years.

Selection

Alternative 2 is recommended without further analysis because it complies with the requirements of the City's MAO and NPDES permit. It is recommended that I/I investigation and repair work be continued on an annual basis.

Collection System Expansion

Several areas of sewer extension are anticipated in the southern areas of the collection system to provide service for existing and future development within the urban growth boundary during this study period. However, these expansions are not scheduled for construction in the immediate future. Development to ultimate build-out levels is not expected during the next 20-year period, but sewers extended into these areas should be based on the ultimate build-out requirements. Sewer extensions are anticipated to occur in the southern region of the City as shown in Figure 6.1.1.

Basic Alternatives

1. **No action, maintain existing service area.** This action does not provide service to the areas identified in the Comprehensive Development Plan as eventually requiring sanitary sewer service.

2. **Provide Service as Required for Future Development.** The areas of proposed service expansion are discussed below and correspond to the recommendations of the City's Comprehensive Development Plan. An anticipated future sewer line extension is shown on Figure 6.1.1. This improvement is not immediately necessary and depends upon future growth.

   Cliffs Corner, South Sutherlin. This area is inside the city limits, and is currently unsewered commercial and industrial area. The collection for this area would consist of approximately 4,500 lineal feet of force main, and would discharge to the Page Street Pump Station. A collection system extension into this area is anticipated to be privately funded. Depending on the funding source, an agency may require this line extension. The estimated cost for the Cliffs Corner collection system extension is $300,000.

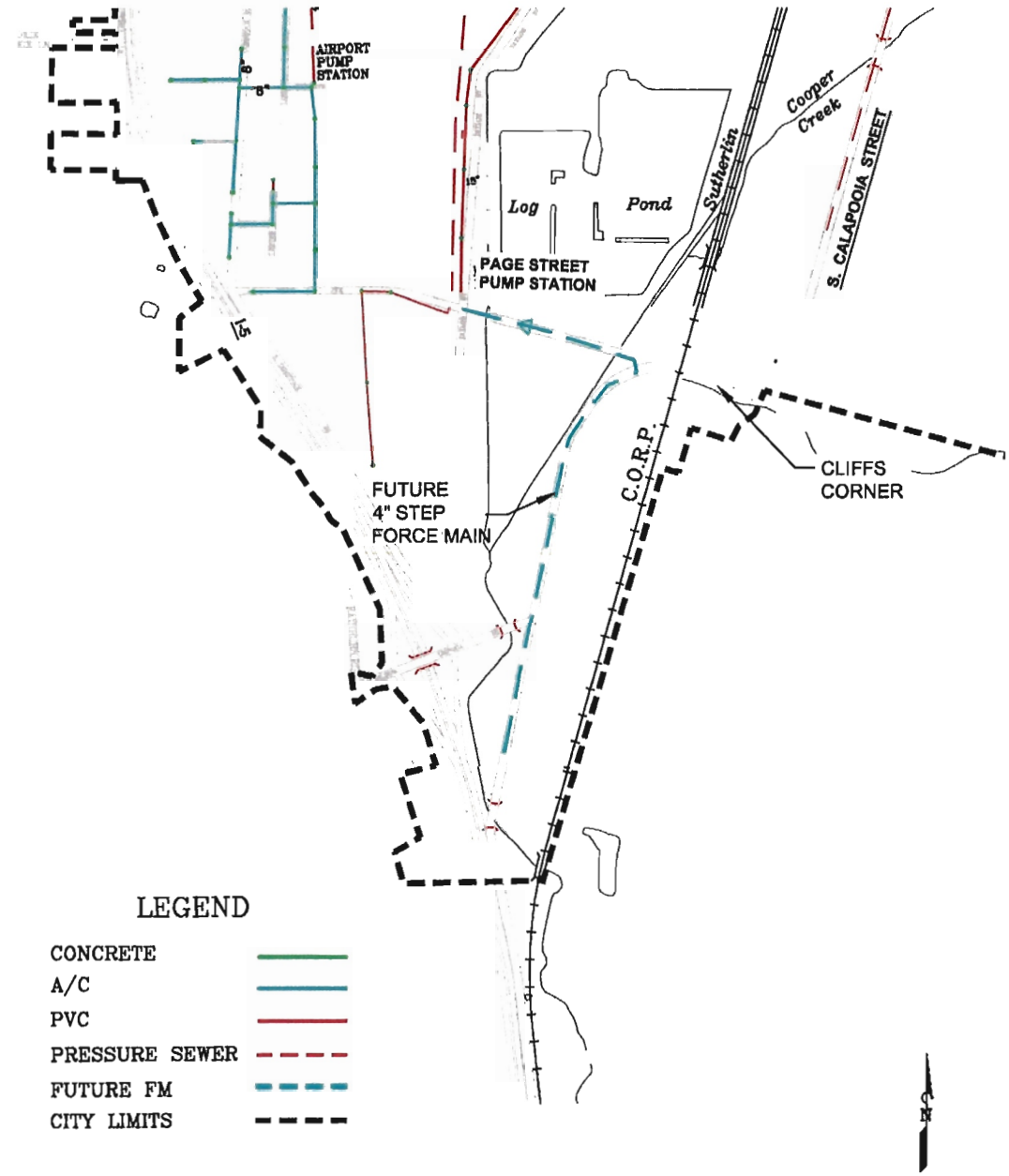
   North of Oak Hills Golf Course. This is an area located north of the Oak Hills Golf Course between Stearns Lane and the Interstate 5 corridor. Property fronting Stearns Lane is zoned light industrial and the area adjacent to the Interstate 5 corridor is zoned residential. Collection system extensions into these areas are anticipated to be privately funded.

Everett Avenue Pump Station Force Main. The force main discharging from this station services a large area of the city that is east of the railroad tracks. During peak flow operation, the downstream gravity line has become surcharged and multiple manholes overflow during high wet weather conditions. When the new Everett Avenue Pump Station is put on line, variable frequency drives, new wet well sizing and new flow controls may reduce the number of sanitary sewer overflows. Further evaluation of the flows is required to determine when a force main reroute is required. Reference the Everett Avenue Pump Station and Force Main Evaluation Report. The estimated cost for the new force main is $1,150,000. See the Everett Avenue Pump Station and Force Main Evaluation in Appendix E.

West of Church Road (Fords Pond area). This is a mixed zone of residential and farm reserve located west of Church Road. A collection system extension into this area is anticipated to be privately funded.

Calapooia Street, North of Cliffs Corner. This area is inside the city limits, and is currently a mix of single family residential and industrial zones. Service to this area is by a low head force main that was constructed in 2012.

FIGURE 6.1.1
FUTURE COLLECTION SYSTEM



Selection

Alternative 2 is recommended without further analysis because it is consistent with the City's Comprehensive Development Plan and with the operational requirements of the wastewater system. Future collection system expansion elements following plant expansion will be constructed as funding becomes available through formation of Local Improvement Districts (LIDs), grants, System Development Charges (SDCs) or loans.

6.2 Existing Pump Station Improvements

The Plant Influent Pump Station is undersized for current and future flows. Options for correcting the problems are discussed below.

Plant Influent Pump Station

The rated capacity of Pumps 1 and 2 is 450 gpm at 37 feet total dynamic head, and Pumps 3 and 4 are rated at 1250 gpm at 35 feet total dynamic head. With all pumps running at once, the existing station has capacity of 4.9 MGD (3,400 gpm). The current peak hour flow (PIF) is 7.30 MGD. DEQ guidelines require redundancy in pumping capacity, with the pump system capable of handling the entire peak hour with the largest pump out of service. The Influent Pump Station is not capable of providing pumping capacity for current peak flows with all pumps running, and does not meet the redundancy requirement for a peak hour flow. The station has a below-grade wet well with limited access that qualifies as an OSHA confined space. If the station is entered, personnel harnesses and hazardous gas detection gear are required. Because of a lack of adequate headworks grit removal, the wet well contains a large amount of grit.

Options:

1. **No action, operate the pump station in existing condition**. The pump station is currently undersized to meet peak hourly flows. If left in its existing state, overflow could occur.

2. **Build new submersible pump station.** This option involves abandoning the existing pump station, and constructing a new wet well with variable speed pumps. The pump configuration would consist of triplex high flow pumps and duplex pumps running to meet peak instantaneous flows. The advantages are that maintenance will be reduced, the confined space access problem for this station will be eliminated, and the City will have capacity to meet the needs for the next 20 years. The disadvantages include the large capital cost and the space limitation adjacent to the existing pump station. Estimated construction cost is approximately $816,000.

3. **Build new screw pump influent pump station**. This option also replaces the existing pump station, but would provide three 48-inch diameter enclosed screws to lift the influent to the headworks. The advantages are that maintenance will be reduced, the confined space access problem for this station will be eliminated, and the City will have capacity projected to meet the needs for the next 20 years. Control of flow is also greatly simplified without instrumentation because the screws pump any flow of water they receive up to the limits of their capacity without speed change. The disadvantages include the high capital cost and the large land area required to site the station. Estimated construction cost is approximately $840,000.

4. **Refurbish pump station and replace pumps**. To meet future capacity needs, new and larger pumps would be required to be placed in the pump room. The location of the pumps above the water level of the wet well would require the installation of a vacuum priming system, which has been very problematic with the existing pumps. The replacement of pumps will not solve the problem of piping and valves located in an inaccessible position below the pump room floor. Modifications would include upgrades to intrinsically safe relays, structural improvements and access improvements. In addition, bypass pumping would be required during construction. This alternative is not considered a viable alternative due to poor vacuum prime reliability and confined space issues.

Selection

Alternative 2 is selected since it requires the least initial capital cost, can be constructed without disruption to the operation of the treatment facility, and is best suited for future expansion (if required).

Everett Avenue Pump Station

In 2010 an Everett Avenue Pump Station and Force Main Evaluation study was prepared by The Dyer Partnership Engineers & Planners, Inc. The report recommended replacement of the Everett Avenue Pump Station. The existing pumps are relatively old and the City has a difficult time finding parts to repair the pumps. Prior to 2011, the pumps present significant operational and maintenance difficulties, including seizing, or ragging, and are taken apart weekly for service. Since the 2011 I/I improvement projects and construction of the Everett Avenue Pump Station Phase 1 Improvements, pump reliability has significantly improved. Phase 2 improvements will be required due to the aging equipment.

1. **No action, operate the pump station in existing condition**. The pump station is conveying current flow conditions. Further study and evaluation of the effectiveness of the I/I projects and the Phase 1 improvements (grinder station) is required. Monitoring flow and station performance for two more years is recommended.

2. **New Pump Station.** This option involves decommissioning the existing station after constructing a new triplex submersible pump station and a new wet well. The existing building and diesel fuel tank could potentially be utilized. The existing building could be utilized to house electrical and control equipment. The existing wet well would be abandoned. A new building will not be required. The new wet well will include a traffic-rated lid, hatch and hoist for pump maintenance. The existing pump station would be left in service during construction of the new pump station. Continuing to utilize the existing pump station during construction eliminates the need for bypass pumping. The advantages are that this option would provide a reliable station with adequate pump capacity for existing and growth demands. The estimated construction cost for a new pump station is $975,000. See Appendix C for the construction cost estimate.

3. **Repair existing pumps**. Replacement parts for the existing pumps are difficult to refurbish, the existing wet well is too small, the control system must be completely replaced and extensive modifications would be required to the building, piping and structural elements. The interior of the pump station is a classified area which would require all controls and electrical gear to be intrinsically safe and all motors to be explosion proof. Newer models of suction lift pumps could be installed but would require significant piping changes and structural modifications. This alternative could be

constructed with lower capital cost as compared to a new pump station, but it would be incapable of providing service for future growth.

Selection

Alternative 2 is chosen as the preferred option because it can most readily solve a number of pump station problems. The use of three submersible variable speed pumps along with an increased wet well volume equalizes flows, thus reducing peak flow force main conveyance. With submersible pumps, any requirement to enter a wet well is greatly reduced. The confined space entry problem will be eliminated.

6.3 Treatment Facility Improvements

Based on the projected wastewater flows and loads presented in this study, a major improvement/expansion project will be required at the wastewater treatment plant (WWTP). The collection system experiences high Inflow and Infiltration (I/I), which has caused the WWTP to exceed its design hydraulic capacity. The facility is also not meeting the requirements for Class A effluent, and the volume of summertime effluent exceeds the capacity of the golf course irrigation. The headworks equipment is not functioning as required, nor is the mechanical screen. The existing Plant Influent Pump Station is not adequately sized for current peak flows. The mass load treatment capacity of the WWTP has been reached, and is inadequate for future needs. The clarifier scraper arms do not adequately remove RAS nor adequately move heavy sludge to the sludge sump for removal. Clarifier surface area is inadequate for future treatment requirements. The Return Activated Sludge (RAS) and WAS pumps are located higher than the sludge they pump and require vacuum priming to start pumping. The pumps cannot be stopped and then restarted without re-priming.

The WWTP is difficult to operate, despite the experience and skills of the trained staff. There is not adequate sludge storage capacity and solids are retained in the treatment process longer than is desirable. The two digester tanks do not have adequate aeration capacity and operate for periods of time in an anaerobic mode.

General

The projected flows for the study period exceed both the hydraulic and biological treatment capacity of the existing WWTP. For the year 2040 the projected MMDWF of 1.88 MGD, and MMWWF of 2.7 MGD. The PDAF is 7.0 MGD and the PIF is 9.0 MGD.

Existing BOD_5 and TSS mass loads on the plant are at current WWTP treatment capacity, and future loads are expected to exceed the capacity of the plant to produce an effluent within permit limits. The current plant is also unable to meet the chlorine and ammonia effluent limits. Projections are based on current data and flow rates, without considering future potential reductions that could result from continued I/I repairs. BOD_5 and TSS are more dependent on population than flow; therefore, these estimates are conservative. The treatment capacity of the existing plant will not handle the projected loading.

The NPDES permit requires that no effluent be discharged to Calapooya Creek from June 1st through October 31st (unless otherwise approved by the department). The Umpqua Basin TMDL addresses bacteria, temperature, nutrients, bio-criteria and related Waste Load Allocations. DEQ completed a Calapooya Creek Analysis Report (January, 2009) and determined discharge in October has limitations dependent on total Phosphorus (related to creek flow) and temperature (See Section 5). Currently summertime effluent is being irrigated on a residential golf course that requires a Class A effluent.

Effluent is currently pumped to an irrigation holding pond located on the golf course site. The golf course operations staff irrigate the golf course during the night. An average of 25 MG of recycled water (effluent) is land applied each summer. An average of 68 MG of effluent is released to Calapooya Creek each summer. Additional effluent storage and/or land application area is required to meet the NPDES permit.

Class A treated effluent must be produced by the new WWTP. This requirement is not possible with existing operational processes alone. Class A summertime-treated effluent volumes will be approximately 1.11 MGD based on the ADWF projected for the year 2040. The peak flow $MMDWF_{10}$ is projected to be 1.88 MGD during the summer of year 2040. A summer treatment process must be able to handle a MMDWF. Options considered for the facility assume that the permit effluent discharge concentration limits are maintained for that portion of the year when discharge to Calapooya Creek during the winter is allowed.

Influent Screen

The existing screen is designed for a flow of up to 5.5 MGD but receives peak flows which exceed this rate. The current peak instantaneous flow (PIF) is 7.3 MGD and is expected to reach 9.0 MGD by 2040. Large rainfall events, combined with a mechanical screen that clogs easily, create water surface elevations which are above the channel walls in the screen chamber. Under these conditions, flows bypass the screen and carry debris and trash to the Influent Pump Station. The inside of the influent screen vault is below-grade and has restricted access for maintenance. The screen is undersized for current peak day flows, clogs easily, is inefficient at removal of solids and needs to be replaced. Options include no change, upgrade of the existing screen unit, or installation of new equipment that meets the 2040 flow requirements.

Basic Alternatives

1. **No Action.** The screen is not functional in all flow conditions and is undersized for existing peak day flows. Partial clogging of the drum causes the channel to overflow. Staff is unable to perform routine maintenance due to the vaults' confined space limitations. Even if confined space issues are resolved, the unit clogs easily and is undersized, resulting in the channel overtopping.

2. **Add a Rotary Bar Screen**. This alternative would involve adding a second rotary unit in the vault bypass channel and adding stairs to resolve the access issue created by the confined space. The existing vault is only 11 feet wide. A rotary unit requires direct drum access for cleaning, which in this location is below-grade. The existing vault would need to be reconstructed to provide enough width for maintenance staff to maneuver around two drums and access stairs would be added. The drums will continue to clog and cause the channels to overtop. Therefore this alternative is not recommended.

3. **Perforated Plate/Climbing Screens.** Install a perforated climbing screen in the new headworks. This would resolve access issues and greatly increase performance and accessibility. The perforated plate screens have a higher hydraulic capacity and contain a washer and compactor. Maintenance staff can access the screen at floor level, providing more room and easier access for screen cleaning. This alternative would significantly improve the ability to clean the unit and provide increased capacity for the 2040 flows. Estimated construction cost is approximately $526,000.

Selection

Alternative 3 is the only viable alternative. Climbing screen technology allows for higher flow capacity, provides the ability to retrofit the existing channels for 2040 flows, and provides for accessible maintenance.

Headworks

Most of the existing headworks components are not operational; consequently, the existing headworks have not been used for many years. The grit removal equipment and flow splitting box either do not work or do not perform as required. Simply replacing the existing components would not be a satisfactory solution because they would be undersized for future flows. Due to the existing high ground water elevations, all new treatment units will be set at a high elevation, making the existing headworks hydraulically obsolete.

Basic Alternatives

1. **No action.** The existing headworks is undersized both in hydraulic capacity and in capability, and is currently being bypassed. The existing hydraulic capacity is 6 MGD. The comminutor, when operating, is a constant maintenance problem. The existing grit removal equipment is undersized for peak flows, and grit washes into the system during major storms.

2. **Upgrade existing Headworks.** Upgrades to the headworks must include upgraded grit removal equipment and significant raising of the structure in order to provide adequate elevation to serve the future treatment units. Due to the high groundwater, all new treatment units will be set at a higher elevation. In addition, the influent screen removes sewage solids, which eliminates the need for a comminutor. Independent flow measurement may be required, depending upon the treatment alternative selected.

3. **Construct new Headworks.** Construction of a new headworks structure, sized to handle current and projected future flows, provides newer, more efficient technology for grit removal, and can be set at an elevation which can supply treatment units without additional pumping. Cost estimates for this alternative are included in Appendix D. The option is necessary as the existing headworks has inadequate hydraulic capacity and treatment capabilities (grit removal and independent flow channel measurement). Construction cost is estimated at $721,000.

Selection

Alternative 3 is the only viable alternative that can provide necessary preliminary treatment of wastewater influent volume and quality. The following biological treatment alternatives will be analyzed, assuming Alternative 3 is included in the overall treatment process.

Biological Treatment

The existing liquid stream treatment of the WWTP consists of two donut-style treatment units; each includes a contact chamber, stabilization tank, clarifier chamber, and a digester. In addition, one unit contains a chlorine contact chamber and the other an irrigation-holding reservoir. This type of treatment is classified as a conventional activated sludge process.

The original loading capacity for the aeration basins is 970 lbs/day of BOD_5 in terms of supplied air for recommended operation in the contact stabilization mode. Per Table 4.2.1, the plant currently receives 1,553 lbs/day of BOD_5 during wet weather and 1,394 lbs/day of BOD_5 during dry weather. Air needs to be diverted from digestion to maintain treatment at certain times. Therefore, the plant is currently operating at aeration capacity below recommended values, and experiences difficulty at times in maintaining Dissolved Oxygen (DO) levels in the aeration basin. The aeration basin volumes are adequate for the current load; however they will need to be increased for the anticipated year 2040 load requirements. Per Table 4.3.4 the recommended 2040 MMWWF design loading is 2,288 lbs/day of BOD_5.

The existing clarifiers do not meet DEQ redundancy requirements and are undersized for the current peak daily flow. Based on a clarifier diameter of 42 feet, each clarifier has a surface area of 1,385 square feet. At 1,200 gpd/sf overflow rate, each clarifier has a capacity of 1.66 MGD. Each clarifier should be sized for a MMDWF of 1.88 MGD (2040) or with both clarifiers operational, a PDAF of 7.0 MGD (2040). The existing sludge rake arms do not function properly and need replacement. The plant upgrade will require that RAS and WAS pumps be installed that do not require vacuum priming to operate; they must be sized to provide flow rates required for future loads and flows.

Due to NPDES permit and Umpqua Basin TMDL limitations, discharge to Calapooya Creek will not be allowed during the summer months (June through September), with conditional discharge in October depending on a minimum stream flow requirement. This WWFP recommends eliminating discharge of effluent in October. Summer effluent is to be managed as recycled water by either irrigation or pond storage. Setbacks need to be met with each class of recycled water. The existing spray irrigation system has adequate setbacks for Class A effluent only. Class B and C effluent could be utilized if the existing irrigation system is reconstructed. For Class B, the property line and right-of-way line setback distance is 10 feet when utilizing direct soil application, and 50 feet for spray irrigation. For Class C, the property line and right-of-way line setback distance is 10 feet when utilizing direct soil application and 70 feet for spray irrigation. Also, Class B and C effluent may be used only if irrigation occurs during off-public contact hours.

Various alternatives are evaluated, based on technologies that meet wintertime effluent limitations and summertime recycled effluent standards. Biological treatment technologies alone cannot meet Class A recycled water treatment standards, with the exception of membrane biological reactor (MBR) technology, which provides both biological treatment and filtration. Class A recycled water standards require oxidation, filtration and disinfection. Three biological treatment technologies are evaluated so that summertime filtered effluent meets Class A standards.

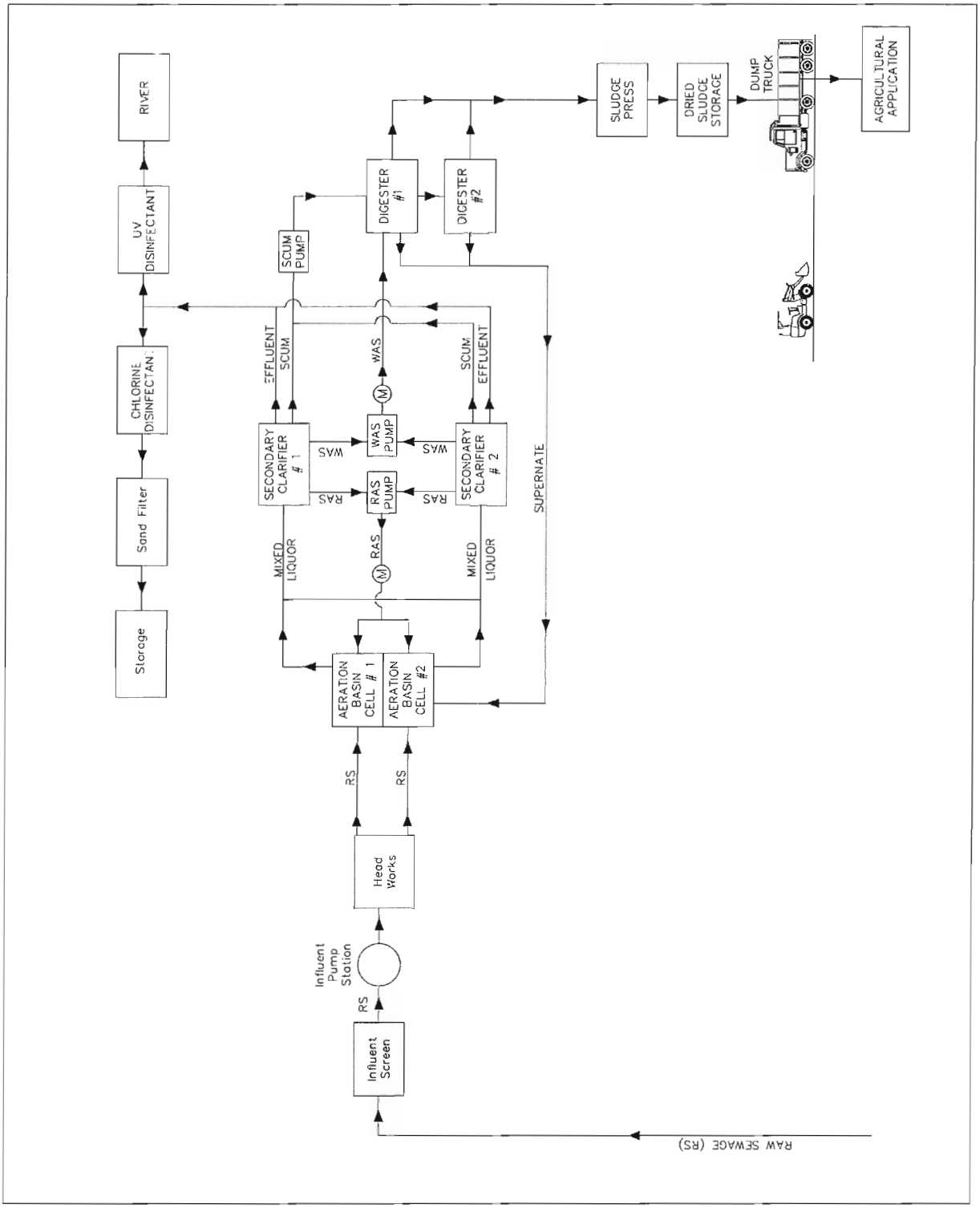
A Membrane Bioreactor (MBR) option is evaluated, working in parallel with alternate treatment units. During winter months, the liquid stream would be split. The MBR would continue to treat the liquid stream at summertime flow rates and a biological unit would treat the remainder of the wintertime flow rates. Detailed cost estimates are provided in Appendix C.

Secondary Treatment Alternatives

1. **No Action**. Continue to operate the existing wastewater treatment plant with one active treatment unit during the dry weather period and both treatment units during the wet-weather period. The system is currently operating at or near the design hydraulic capacity and at the mass load capacity during peak wet-weather conditions, and will soon be unable to meet wintertime treatment standards. Summertime flows are irrigated to a residential golf course which requires Class A recycled water. The City's existing treatment facility does not meet the Class A recycled water standard. The current configuration does not satisfy reliability standards due to a lack of redundancy during wet

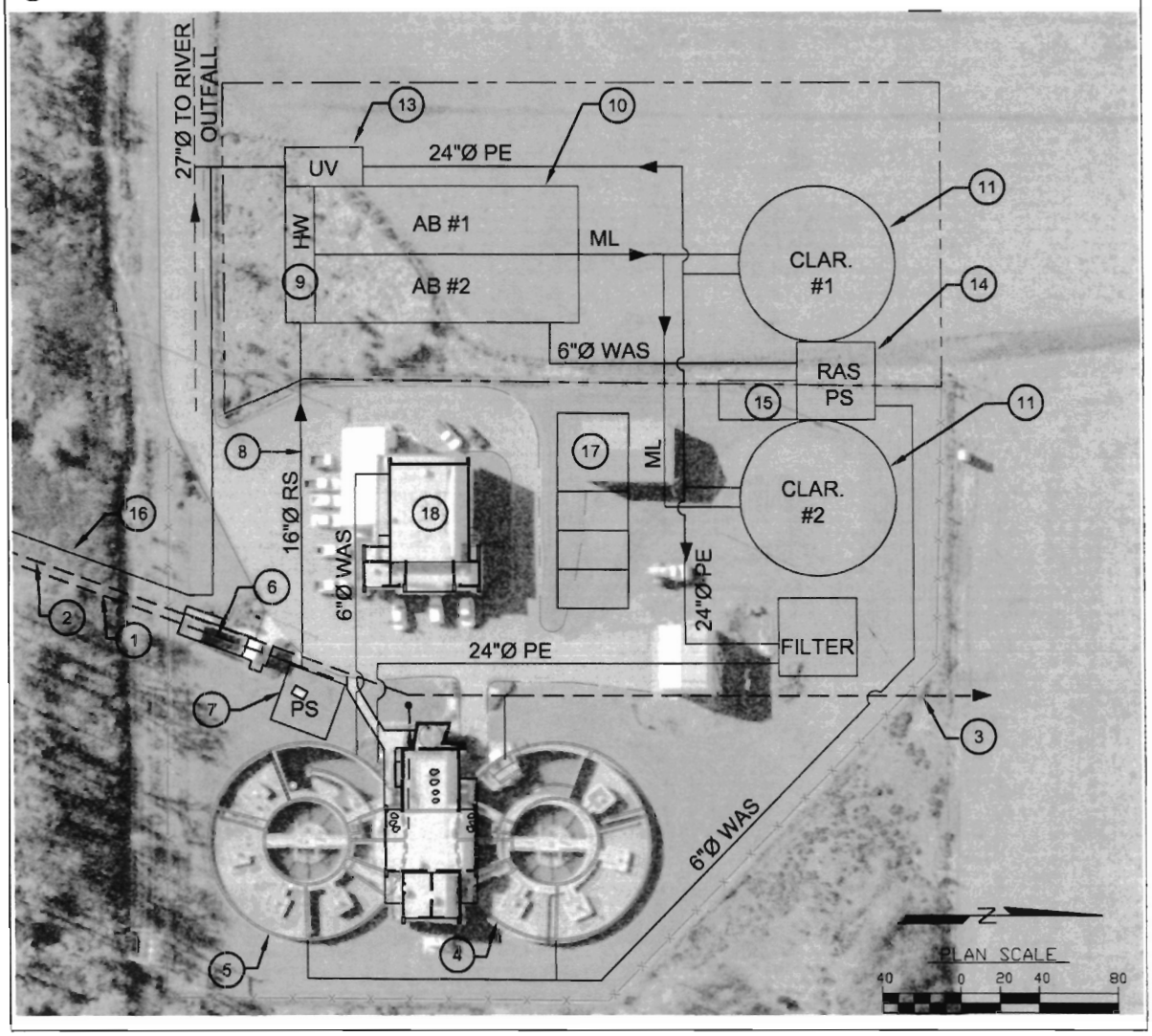
weather flows, making the maintenance required impossible without bypassing untreated flow. Because the existing system cannot meet current treatment standards and requirements, Alternative 1 is rejected without further analysis.

2. **Conventional Activated Sludge Treatment Processes**. A schematic of this process may be seen by reference to Figures 6.3.1A and 6.3.1C. The difference between the two is that one utilizes a Tertiary Filter and the other utilizes an MBR to achieve Class A effluent. Component layouts as related to the two processes are shown in Figures 6.3.1B and 6.3.1D, respectively. The activated sludge conventional process is typically accomplished using two process units: an aeration basin and a clarifier or settling basin. Screened wastewater is introduced into the aeration basin where it is mixed by the use of diffused or mechanical aeration to maintain a completely aerobic state. The solution of wastewater and microorganisms, or mixed liquor, flows from the aeration basin to the clarifier where the solids settle. A portion of the settled solids or sludge is recycled back to the aeration basin and a portion is wasted, typically to a digester. The clarified effluent is usually removed from the top of the settling basin for further treatment (e.g. disinfection) and disposal.

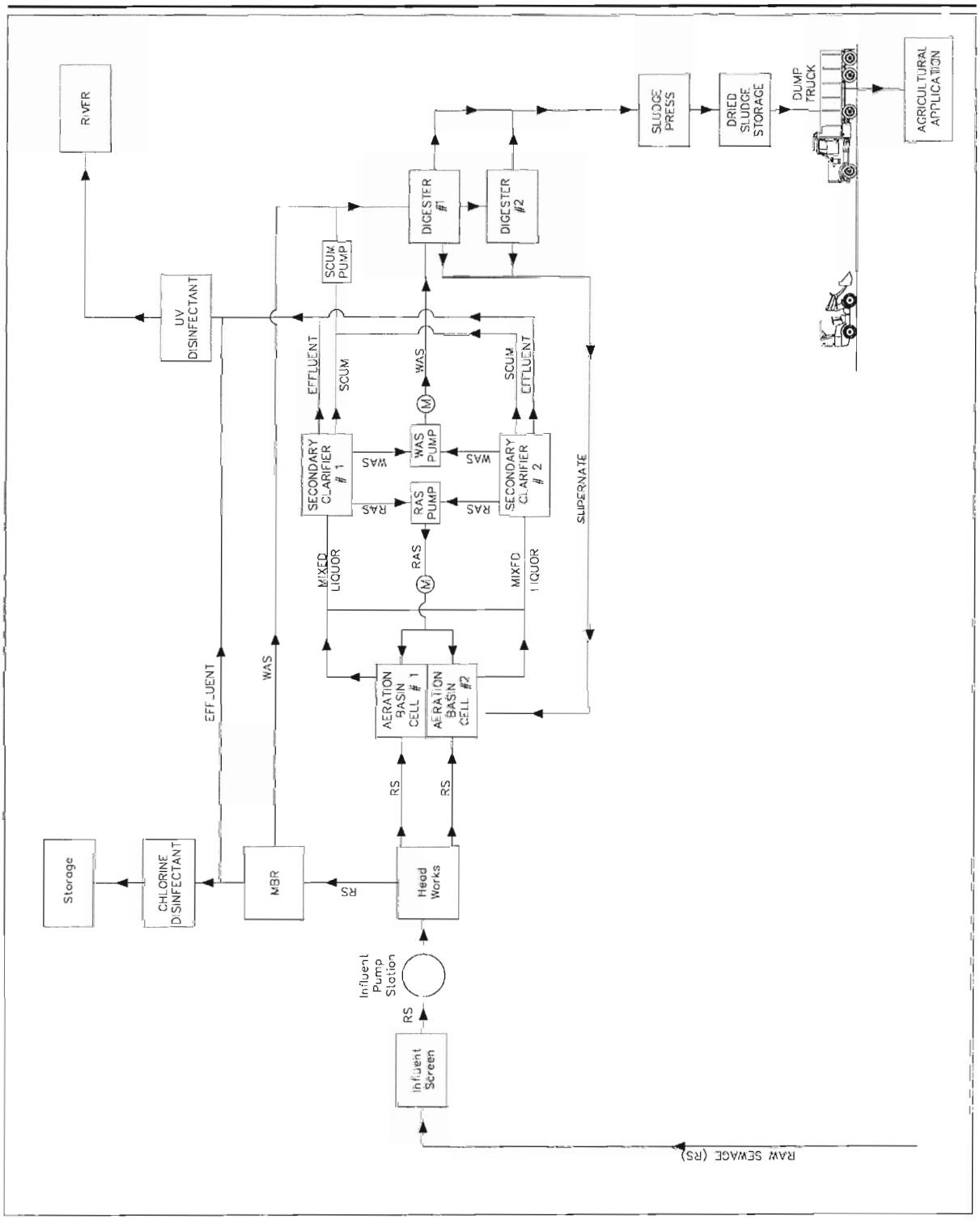


THE DYER PARTNERSHIP ENGINEERS & PLANNERS, INC.	***CITY OF SUTHERLIN WASTEWATER FACILITIES PLAN***	FIGURE NO.
DATE: JAN., 2010	***ALTERNATIVE NO.2 - CONVENTIONAL WITH FILTER***	***6.3.1A***
PROJECT NO.: 146.02		

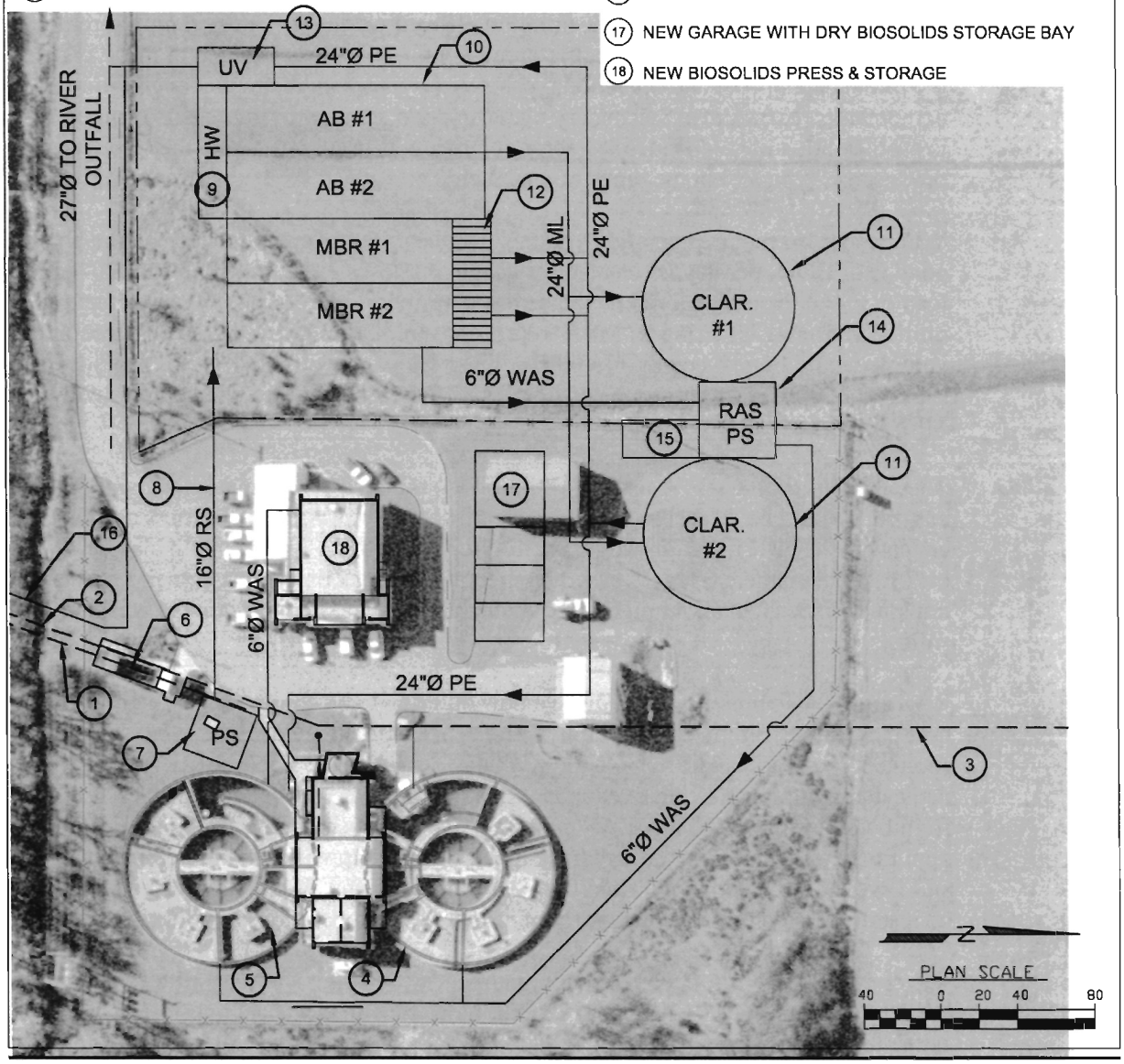
1. EXISTING 27"Ø INFLUENT TRUNK LINE
2. EXISTING IRRIGATION LINE TO GOLF COURSE
3. EXISTING IRRIGATION LINE TO FARM
4. EXISTING DIGESTER #2
5. EXISTING DIGESTER #1
6. NEW INFLUENT SCREEN IN EXISTING STRUCTURE
7. NEW INFLUENT PUMP STATION (PS)
8. NEW 16"Ø RAW SEWAGE (RS) LINE
9. NEW HEADWORKS (HW)
10. NEW 2 EACH AERATION BASIN
11. NEW SECONDARY CLARIFIER
12. NEW TERTIARY FILTER
13. NEW UV DISINFECTANT
14. NEW RAS/WAS PUMP STATION
15. NEW BLOWER BUILDING
16. TO RECYCLED WATER STORAGE POND
17. NEW GARAGE WITH DRY BIOSOLIDS STORAGE BAY
18. NEW BIOSOLIDS PRESS & STORAGE



THE DYER PARTNERSHIP ENGINEERS & PLANNERS, INC.	***CITY OF SUTHERLIN WASTEWATER TREATMENT PLAN***	FIGURE NO.
DATE: JAN., 2010	***ALTERNATE NO. 2 - CONVENTIONAL/FILTER***	***6.3.1B***
PROJECT NO.: 146.02		



THE DYER PARTNERSHIP ENGINEERS & PLANNERS, INC.	***CITY OF SUTHERLIN*** ***WASTEWATER FACILITIES PLAN***	FIGURE NO.
DATE: JAN., 2010 PROJECT NO.: 146.02	***ALTERNATIVE NO.2 - CONVENTIONAL WITH MBR***	***6.3.1C***



THE DYER PARTNERSHIP ENGINEERS & PLANNERS, INC.	***CITY OF SUTHERLIN WASTEWATER TREATMENT PLAN***	FIGURE NO.
DATE: JAN., 2010	***ALTERNATE NO. 2 - CONVENTIONAL / MBR***	***6.3.1D***
PROJECT NO.: 146.02		

An aeration basin can be designed to operate in a number of modes, such as complete mix, step feed, and contact stabilization, by providing an inlet channel with four discharge points (e.g. at the beginning, one eighth, one quarter, and one half the length of the basin). To meet current regulatory standards, two aeration basins would be required. Return activated sludge from the clarifiers would be mixed with the influent at the head of the aeration basins. Following the aeration basins, two circular clarifiers would be required to satisfy the redundancy requirements. Pumping facilities would be required for the transport of RAS and WAS. The existing aeration basin and clarifiers would be converted to aerobic digester and sludge storage.

Key advantages of conventional activated sludge systems include their widespread use (and thus familiarity) and flexibility. Conventional activated sludge systems are capable of producing BOD_5 and TSS concentration effluent quality generally ranging from 10 to 20 mg/l, depending upon the specific process train. The disadvantage of these systems is the initial capital cost and operational costs associated with the redundant treatment components (two primary/secondary clarifiers, two aeration basins). Nitrogen reduction is problematic unless an extended aeration process is used. As noted, the effluent produced is typically in the range of 10-20 mg/l.

With respect to required aeration volume, a value of 1,000 cubic feet of volume per 15 pounds of BOD_5 per day is selected for design purposes. The projected year 2040 BOD_5 load of 2,460 lbs/day will require an aeration basin volume of 164,000 cubic feet (1.3 million gallons). Utilizing an MBR to process summertime flow rates, the aeration basin volume would be reduced by 136,000 gallons. Blowers would have to be provided to supply 2,330 SCFM of air to the diffusers (1,740 SCFM of air to the diffusers with an MBR).

The clarifiers with Class A tertiary filtration would have 5,000 square feet, and both new clarifiers would be 80 feet in diameter. Depth should be 14 feet. Clarifiers used in concert with MBR filtration would each have 4,400 square feet and both clarifiers would be 75 feet in diameter. This is based on meeting the peak daily flow demand with both units in service at a maximum overflow rate of 1500 gal/sf/day to prevent washout of solids.

RAS and WAS pumping will require a new building to be constructed, which would also house the aeration blowers. For conventional aeration, the return activated sludge pumps should be sized to handle 50% of the peak day design flow. Each clarifier should have a dedicated pump with a single redundant pump and the ability to adjust valves and pump from both clarifiers with any RAS pump. WAS pumps should be sized in the 30 to 40 gpm range. Each clarifier should have a dedicated pump with a single redundant pump, and have the ability to pump from both clarifiers with any WAS pump. Blowers would be installed in the existing blower room of the WWTP operations building.

The construction cost of conventional treatment with Class A tertiary filtration, aeration basins, clarifiers, RAS/WAS pump station, blowers with electrical improvements is estimated to be approximately $ 6,250,000.

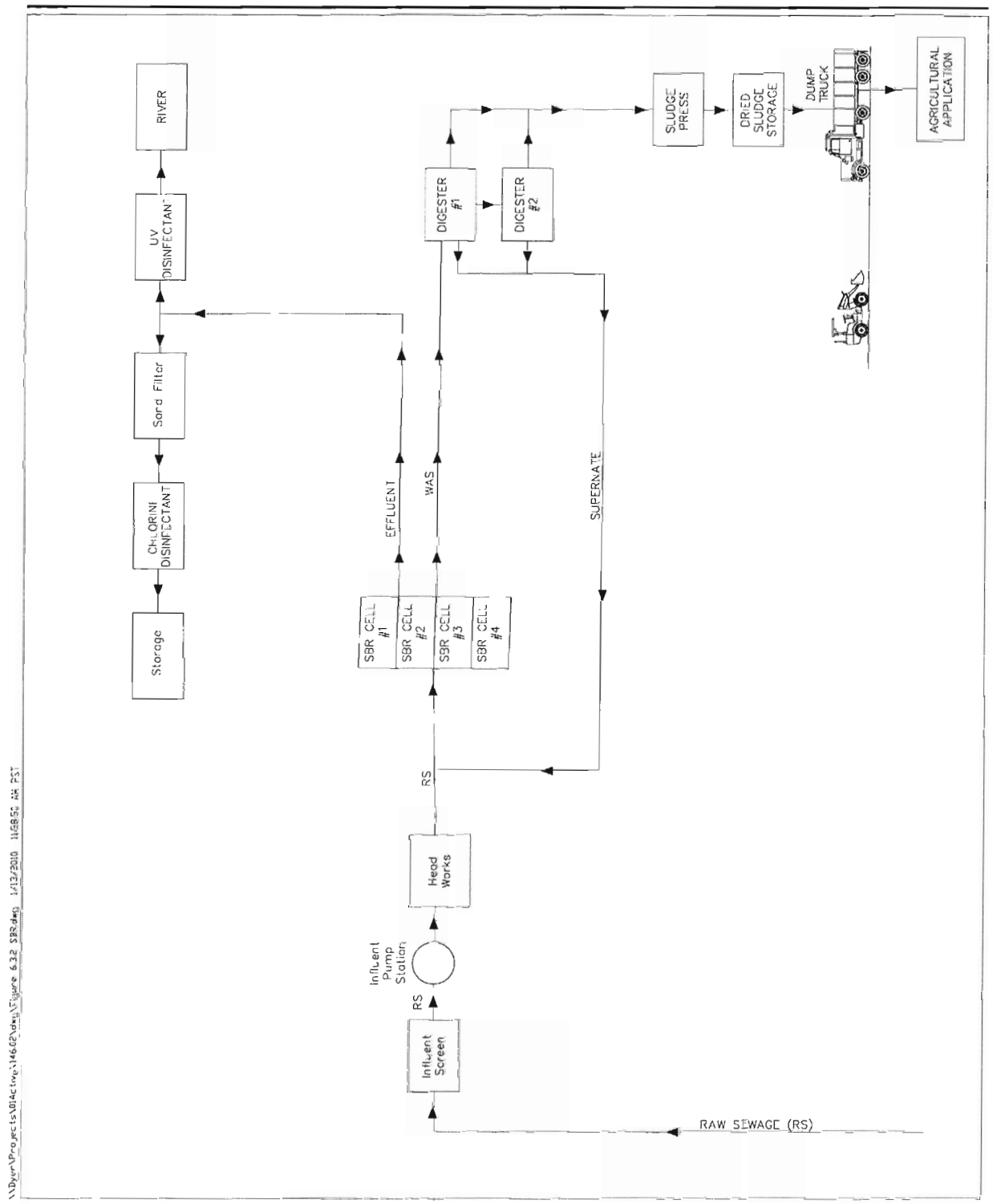
The construction cost of an MBR coupled with downsized conventional treatment is estimated to be approximately $9,280,000.

3. **New Sequencing Batch Reactor (SBR).** A schematic of this process may be seen by reference to Figures 6.3.2A and 6.3.2C. The difference between the two is that one utilizes a tertiary filter and the other utilizes an MBR to achieve Class A effluent. Component layouts as related to the two processes are shown in Figures 6.3.2B and 6.3.2D, respectively. The SBR process is operated in a periodic manner (batch) and utilizes a single tank for both biochemical oxidation and sedimentation of suspended microorganisms. Because of the batch operation, there is normally more than one SBR tank utilized for a continuous wastewater stream.

   A typical operation sequence consists of four steps: fill, react, settle, and decant. The treatment sequence begins with the introduction of wastewater into a partially filled tank containing settled mixed liquor from a previous cycle (fill phase). In the react phase, aeration and mixing are provided for a length of time sufficient to produce an effluent of the desired quality. With some SBR units, the fill and react phases are combined in an aerated fill step. After the react phase, the mixed liquor is then allowed to operate in a quiescent settling phase, after which the clear supernatant liquid is then subsequently decanted as effluent. Once the decant phase is completed, the system is ready for refilling. During the settle and decant phases in one tank, the other tank is undergoing the fill and react phases. Because the process continually runs through cycles, automated controls are utilized to operate the treatment process.

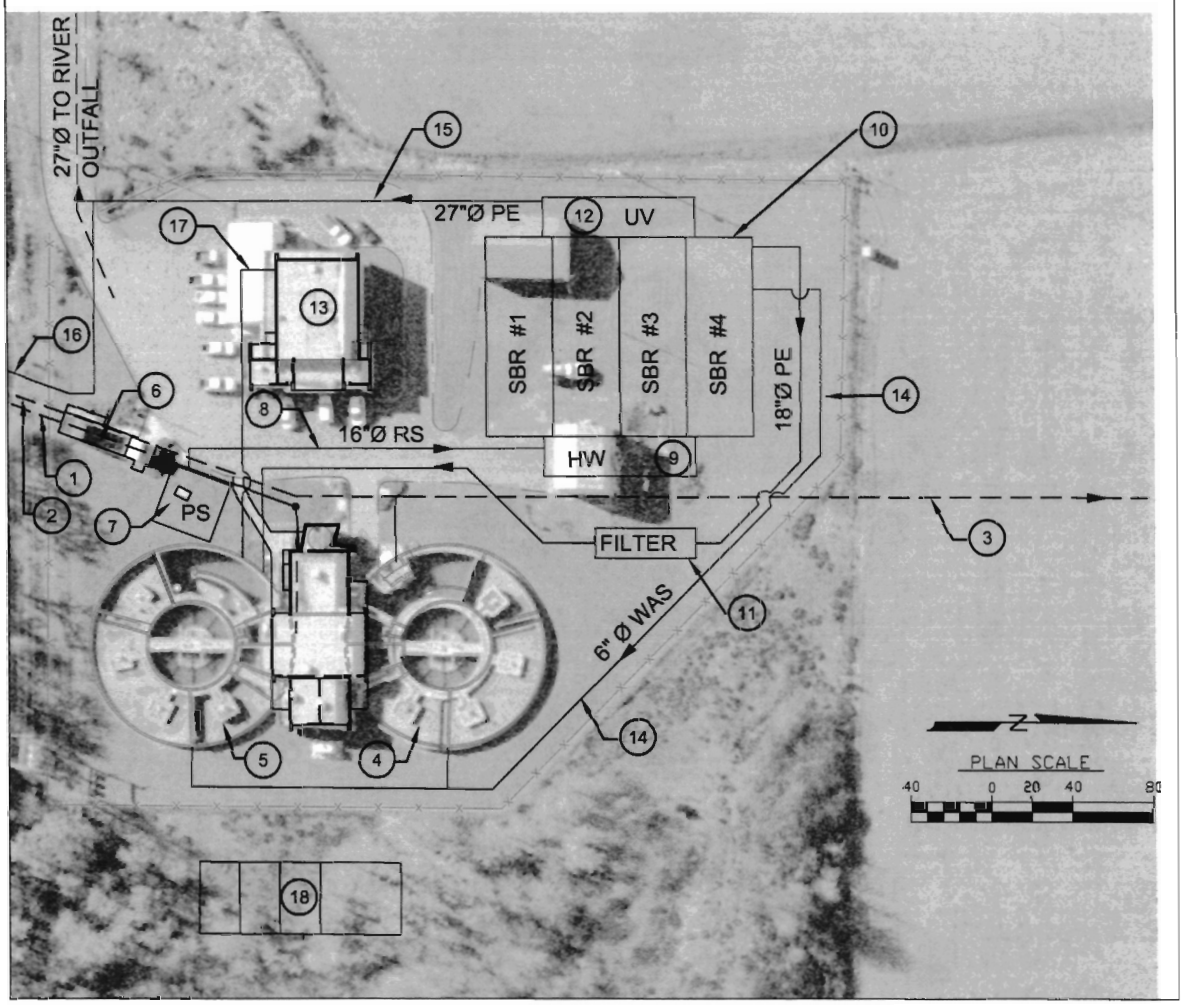
   Key advantages of the SBR process include its simplicity and reliability, high operational flexibility, capability of very high and consistent effluent quality due to quiescent batch settling (<10 mg/l BOD_5, TSS), requirement of less operator attention than most other mechanical systems, and ideal suitability to wide flow variations. The existing contact stabilization treatment units would be converted to aerobic digestion and sludge storage.

   After preliminary treatment, the wastewater would then flow by gravity to a flow splitter that would divide the flow entering the SBR basins. The four-cell design under consideration is based on the type of SBR that contains a baffled inlet, which allows inlet flow even during the settling cycle without impairing treatment effectiveness. For maintenance purposes, all the inlet flows can be diverted to the other basins. There are a number of SBR vendors in the United States, some of which use a pure batch system and others that are designed for continuous feed.

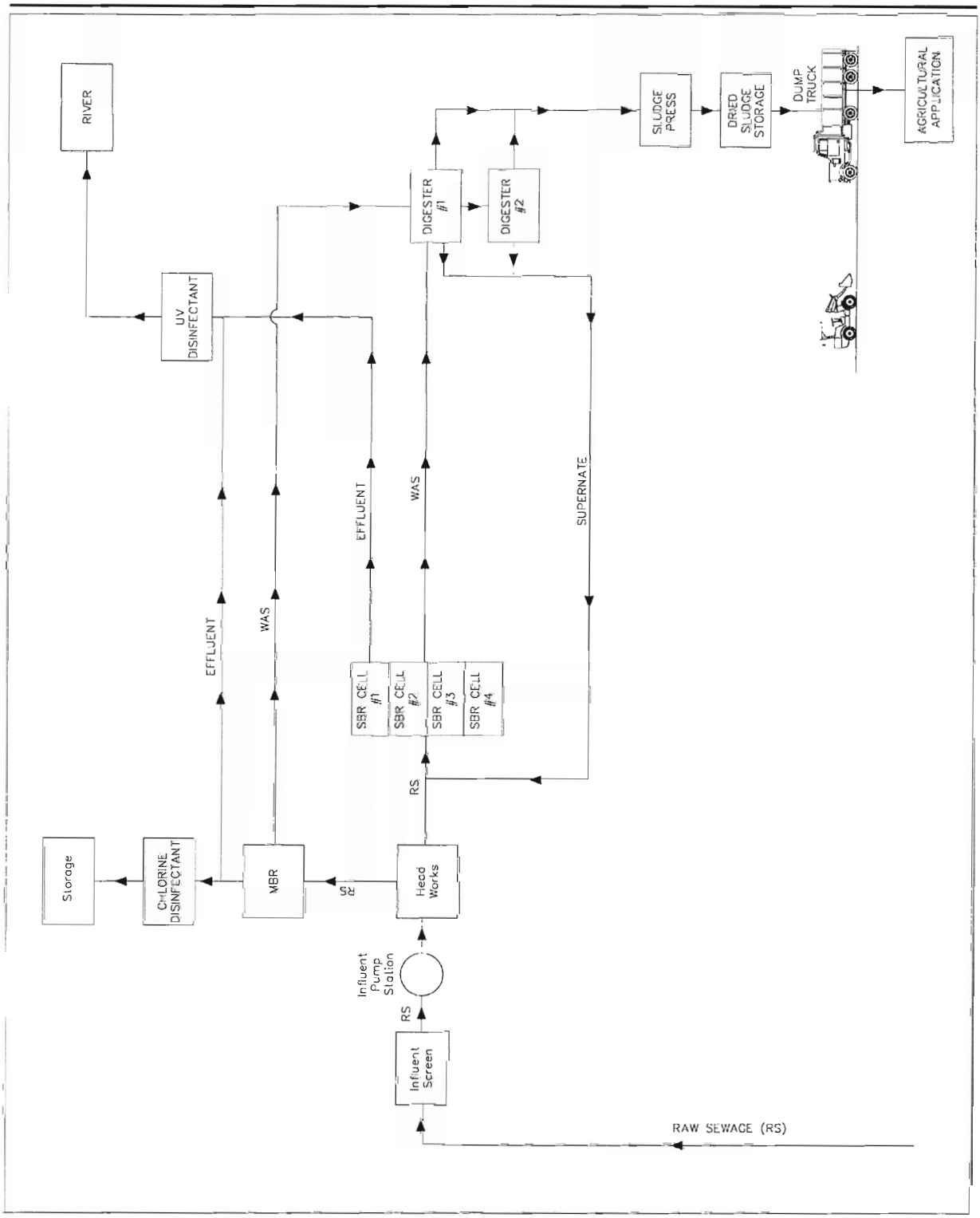


THE DYER PARTNERSHIP ENGINEERS & PLANNERS, INC.	***CITY OF SUTHERLIN WASTEWATER FACILITIES PLAN***	FIGURE NO.
DATE: JAN., 2010	***ALTERNATIVE NO. 3 - SBR WITH FILTER***	***6.3.2A***
PROJECT NO.: 146.02		

1. EXISTING 27"Ø INFLUENT LINE
2. EXISTING IRRIGATION LINE TO GOLF COURSE
3. EXISTING IRRIGATION LINE TO FARM
4. EXISTING DIGESTER #2
5. EXISTING DIGESTER #1
6. NEW INFLUENT SCREENS IN EXISTING STRUCTURE
7. NEW INFLUENT PUMP STATION
8. NEW 16"Ø RAW SEWAGE (R/S)
9. NEW HEADWORKS (HW)
10. NEW 4 CELLED SBR
11. NEW TERTITARY FILTER
12. NEW UV DISINFECTANT
13. RECONSTRUCT EXISTING FILTER BUILDING INTO BIOSOLIDS PROCESS BUILDING.
14. NEW 6"Ø WAS LINE
15. NEW 27"Ø EFFLUENT LINE TO RIVER
16. TO RECYCLED WATER STORAGE POND
17. NEW 6"Ø BIOSOLIDS LINE
18. NEW GARAGE WITH DRY BIOSOLIDS STORAGE BAY

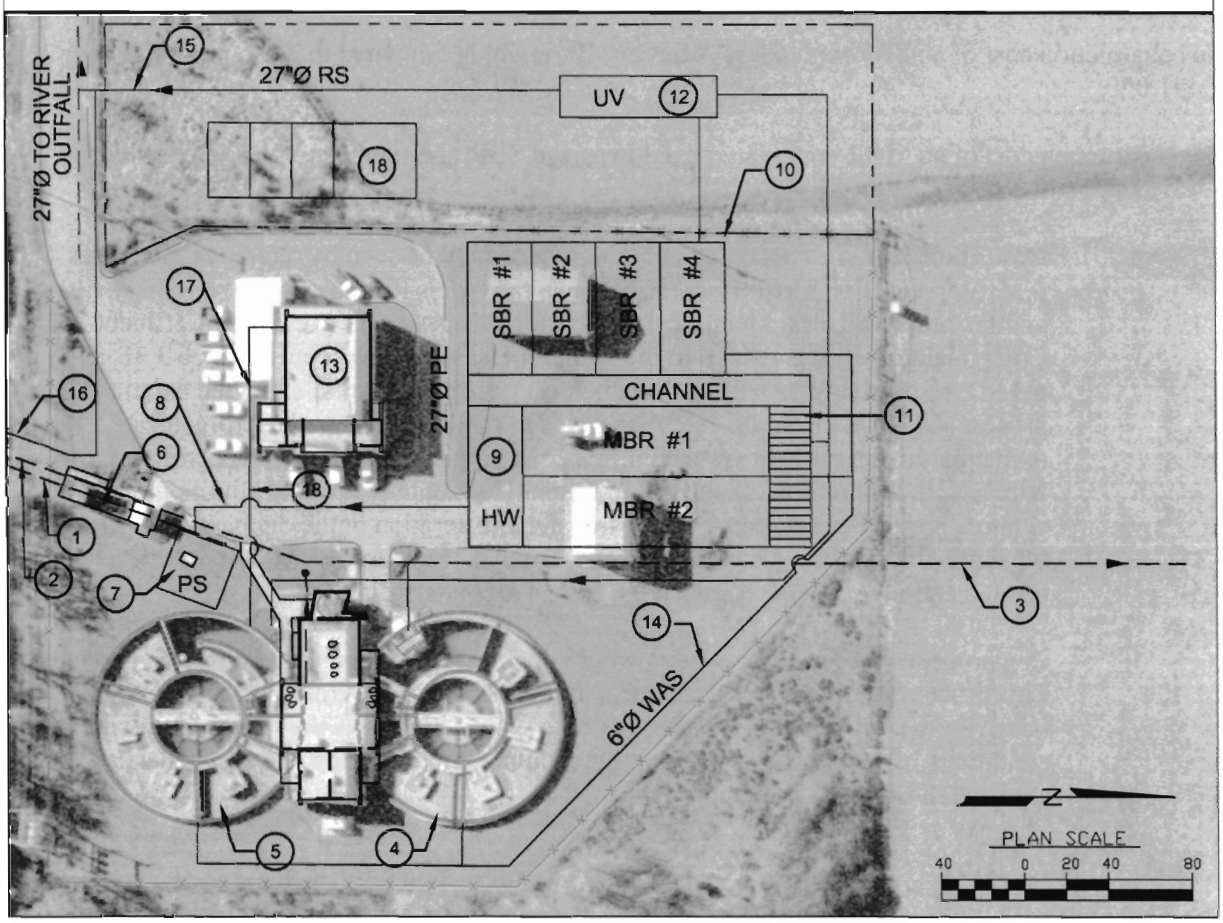


THE DYER PARTNERSHIP ENGINEERS & PLANNERS, INC.	***CITY OF SUTHERLIN WASTEWATER TREATMENT PLAN***	FIGURE NO.
DATE: May, 2010	***ALTERNATE NO. 3 - SBR/FILTER***	***6.3.2B***
PROJECT NO.: 146.02		



THE DYER PARTNERSHIP ENGINEERS & PLANNERS, INC.	***CITY OF SUTHERLIN*** ***WASTEWATER FACILITIES PLAN***	FIGURE NO. ***6.3.2C***
DATE: JAN., 2010 PROJECT NO.: 146.02	***ALTERNATIVE NO. 3 - SBR WITH MBR***	

1. EXISTING 27"Ø INFLUENT LINE
2. EXISTING IRRIGATION LINE TO GOLF COURSE
3. EXISTING IRRIGATION LINE TO FARM
4. EXISTING DIGESTER #2
5. EXISTING DIGESTER #1
6. NEW INFLUENT SCREENS IN EXISTING STRUCTURE
7. NEW INFLUENT PUMP STATION
8. NEW 16"Ø RAW SEWAGE (RS)
9. NEW HEADWORKS (HW)
10. NEW 4 CELLED SBR
11. NEW MBR
12. NEW UV DISINFECTANT
13. RECONSTRUCT EXISTING FILTER BUILDING INTO BIOSOLIDS STORAGE BUILDING.
14. NEW 6"Ø WAS LINE
15. NEW 27"Ø EFFLUENT LINE TO RIVER
16. TO RECYCLED WATER STORAGE POND
17. NEW 6"Ø BIOSOLIDS LINE
18. NEW GARAGE WITH DRY BIOSOLIDS STORAGE BAY



THE DYER PARTNERSHIP ENGINEERS & PLANNERS, INC.	***CITY OF SUTHERLIN WASTEWATER TREATMENT PLAN***	FIGURE NO.
DATE: JAN., 2010	***ALTERNATE NO. 3 - SBR/MBR***	***6.3.2D***
PROJECT NO.: 146.02		

Each SBR basin would consist of a concrete rectangular structure with a top water and bottom water level of 18 and 12.5 feet (approximate), respectively. In each basin, there would be a pre-react zone or mixers installed to inhibit filamentous growth that causes sludge bulking. Since each basin acts as an aeration basin and clarifier, no return activated sludge equipment is required. Scum would be removed by floating skimmers in each basin and sent back to the preliminary treatment process for dewatering and compaction. The waste sludge is pumped and measured from each basin to the aerobic digester with small submersible pumps. Common wall construction would be utilized to minimize the SBR footprint and construction costs.

Since the SBR is a batch or semi-batch process, the effluent flow rate during a decant cycle is greater than the influent flows. The use of four cells provides adequate equalization allowing uniform downstream flows. The overall dimensions for the SBR treatment with Class A tertiary filtration are 100 feet long by 33 feet wide by 21.5 feet high for each of the four units. The overall dimensions for the SBR treatment unit treating the liquid stream in parallel with an MBR are 80 feet long by 30 feet wide by 21.5 feet high for each of the four units.

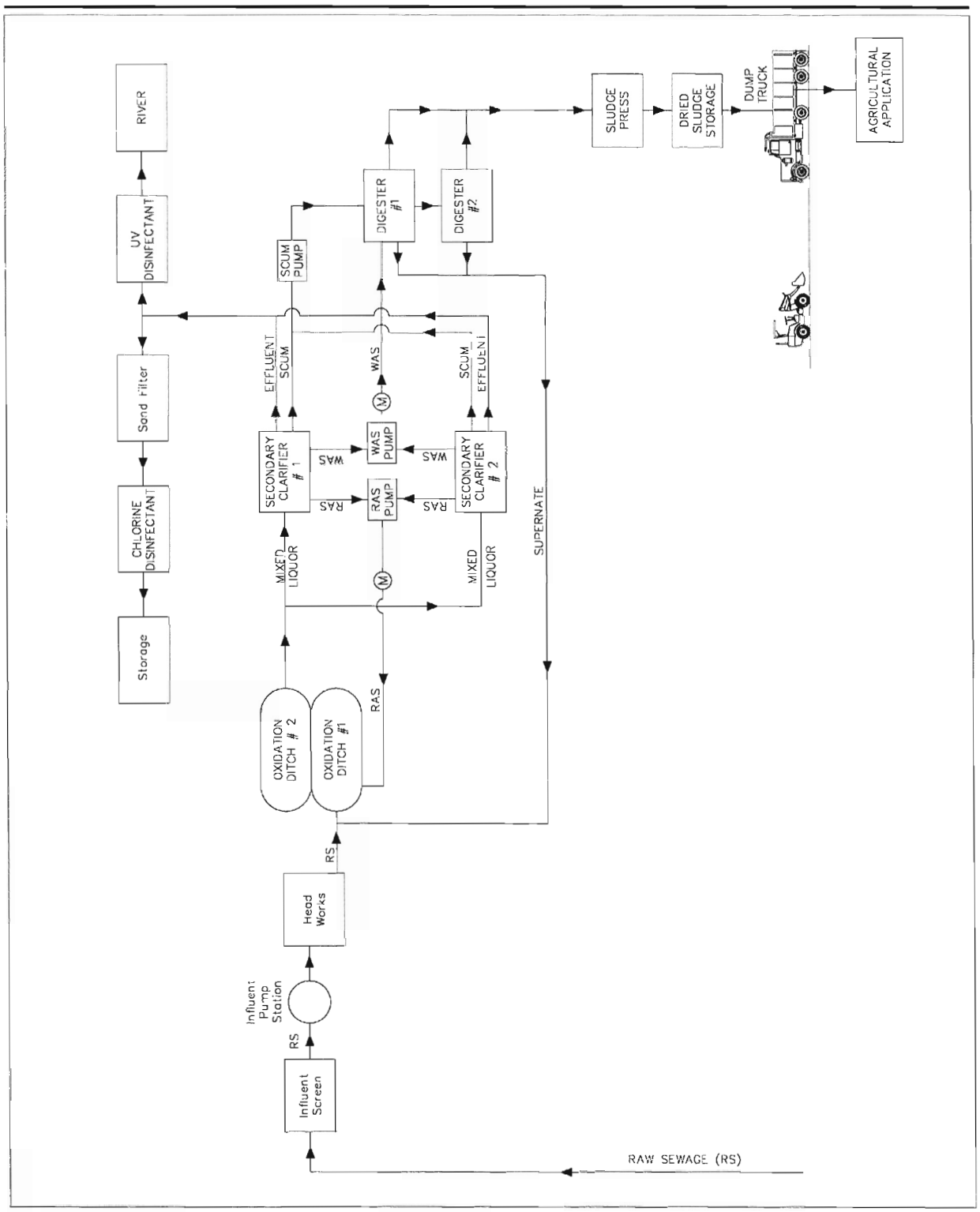
Compared with the other biological alternatives investigated, SBRs eliminate the need for RAS/WAS pumping facilities, separate clarifiers and associated yard piping.

The construction cost of an SBR with Class A tertiary filtration, is estimated to be approximately $5,491,000.

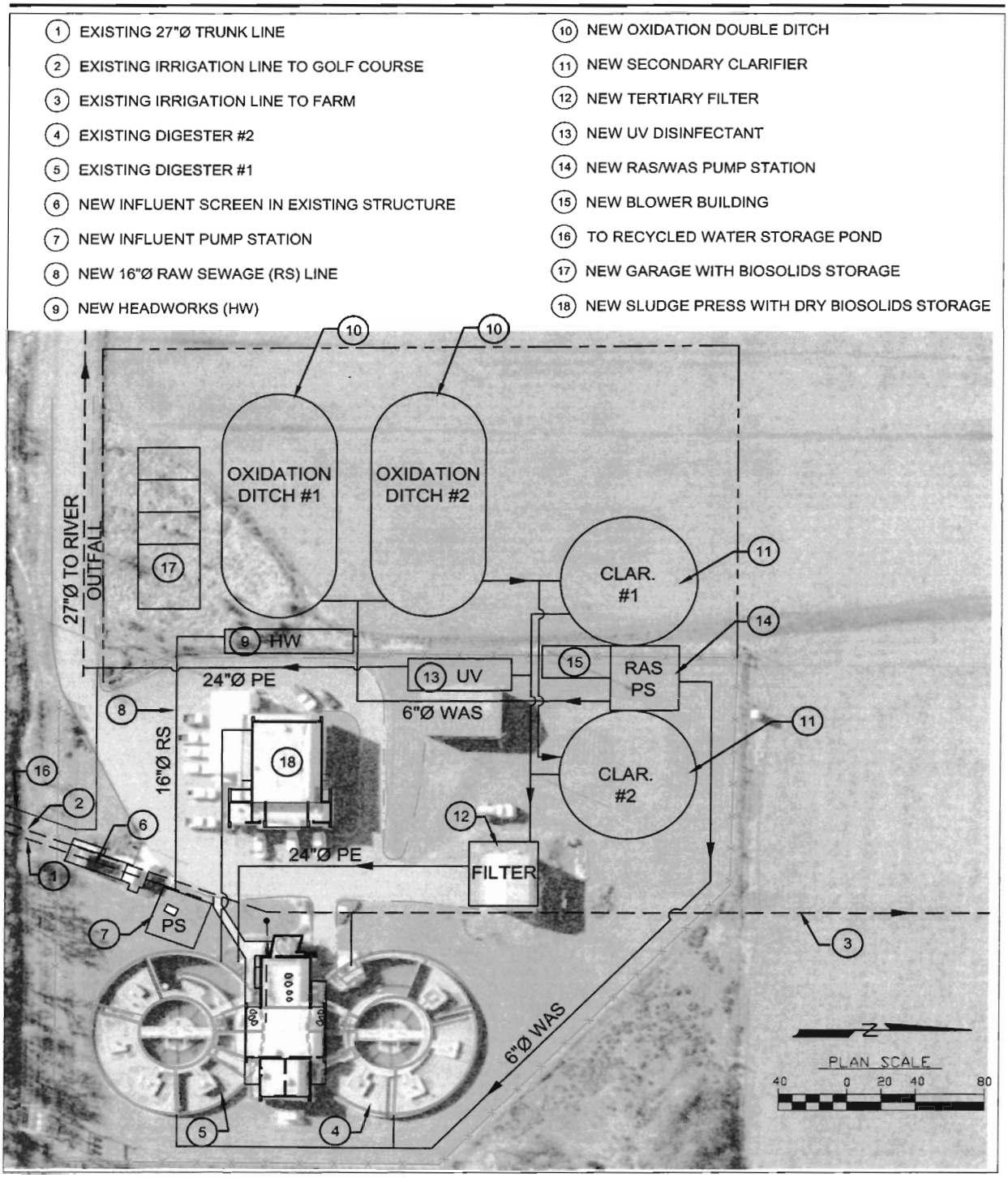
The construction cost of an MBR coupled with a downsized SBR is estimated to be approximately $7,461,000.

4. **Oxidation Ditch.** A schematic of this process may be seen by reference to Figures 6.3.3A and 6.3.3C. The difference between the two is that one utilizes a tertiary filter and the other utilizes a Membrane Bioreactor (MBR) to achieve Class A effluent. Component layouts as related to the two processes are shown in Figures 6.3.3B and 6.3.3D, respectively. An oxidation ditch is a modified form of extended aeration. The process utilizes long solids retention times to remove biodegradable organics. A typical oxidation ditch treatment system consists of a single or multi-channel configuration with a ring, oval or horseshoe-shaped basin. Horizontally or vertically-mounted aerators provide circulation, oxygen transfer and aeration in the ditch. The process is typically preceded by screening and grit removal. Flow to the oxidation ditch is aerated and mixed with return sludge from a secondary clarifier.

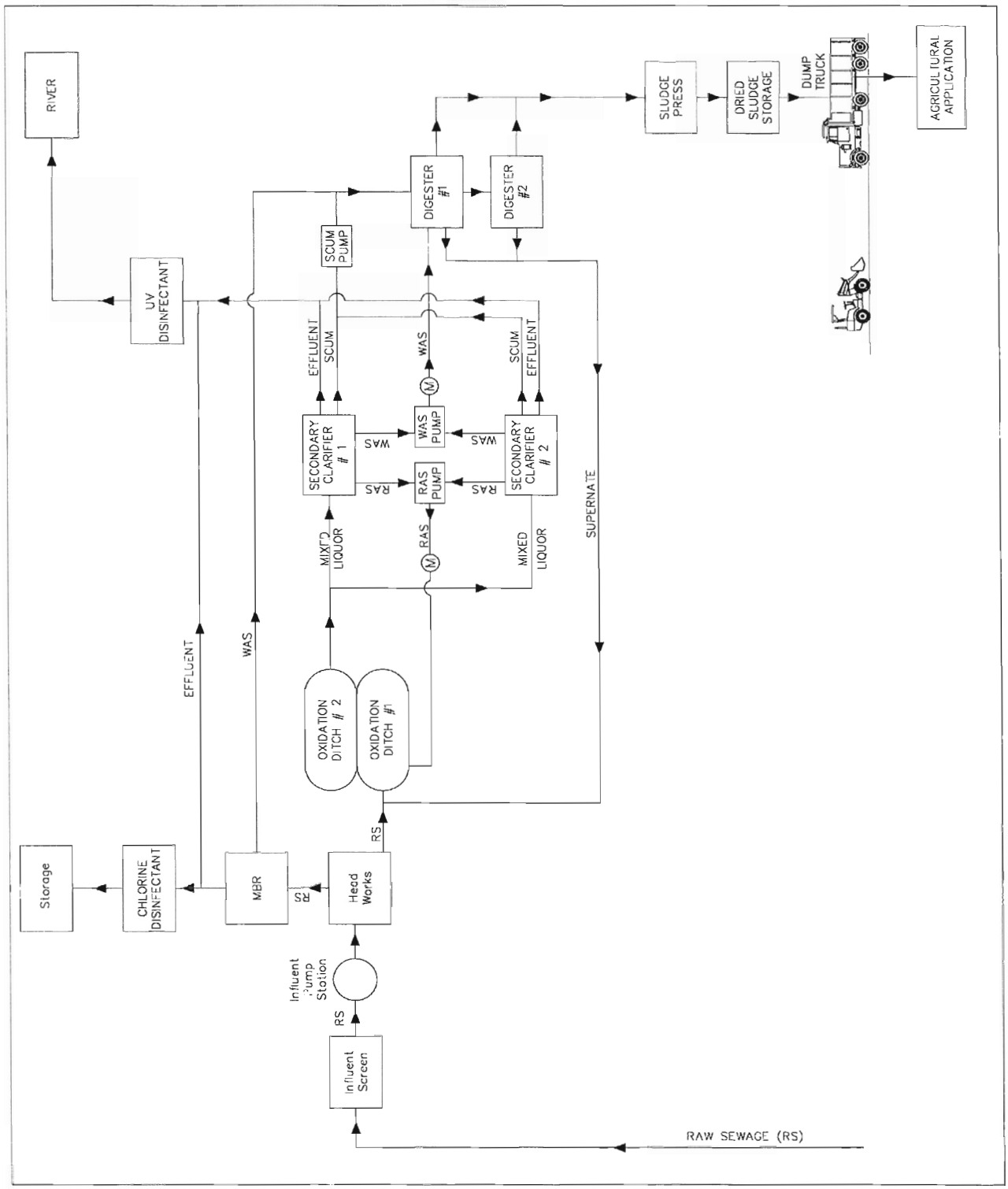
   The proposed double ditch system would operate in a Phased Isolation Ditch process. This process alternates flow patterns and process conditions (aerobic and anoxic) occurring within the oxidation ditches. The system alternates the flow and process conditions in the tanks. As a result, the volume of nitrification and denitrification can be adjusted to fit the actual conditions.



THE DYER PARTNERSHIP ENGINEERS & PLANNERS, INC.	***CITY OF SUTHERLIN*** ***WASTEWATER FACILITIES PLAN***	FIGURE NO. ***6.3.3A***
DATE: JAN., 2010 PROJECT NO.: 146.02	***ALTERNATIVE NO.4 - OXIDATION DITCH WITH FILTER***	

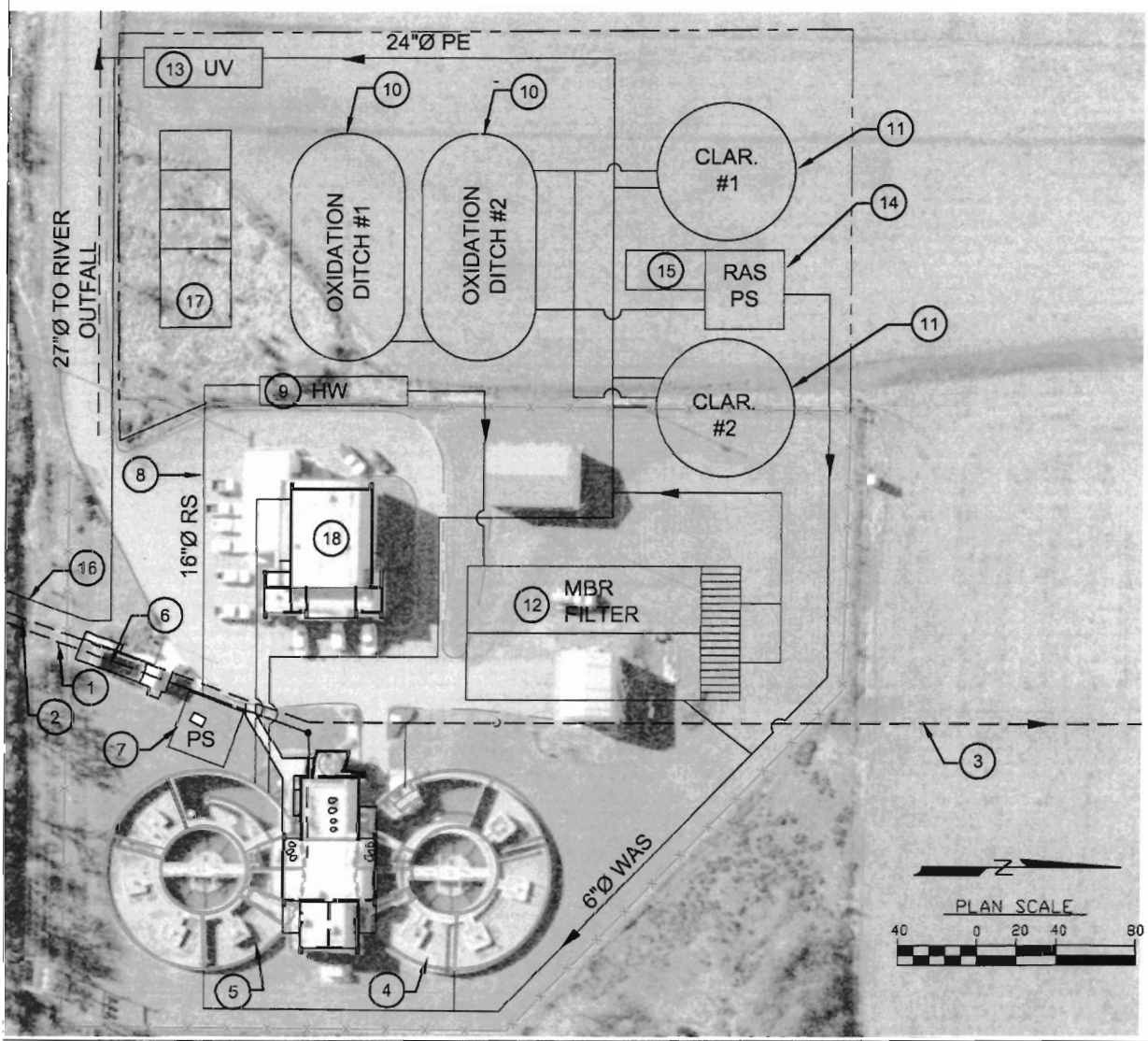


THE DYER PARTNERSHIP ENGINEERS & PLANNERS, INC.	***CITY OF SUTHERLIN WASTEWATER TREATMENT PLAN***	FIGURE NO.
DATE: JAN., 2010 PROJECT NO.: 146.02	***ALTERNATE NO. 4 - OXIDATION WITH FILTER***	***6.3.3B***



THE DYER PARTNERSHIP ENGINEERS & PLANNERS, INC.	***CITY OF SUTHERLIN WASTEWATER FACILITIES PLAN***	FIGURE NO.
DATE: JAN., 2010	***ALTERNATIVE NO.4 - OXIDATION DITCH WITH MBR***	***6.3.3C***
PROJECT NO.: 146.02		

1. EXISTING 27"Ø TRUNK LINE
2. EXISTING IRRIGATION LINE TO GOLF COURSE
3. EXISTING IRRIGATION LINE TO FARM
4. EXISTING DIGESTER #2
5. EXISTING DIGESTER #1
6. NEW INFLUENT SCREEN IN EXISTING STRUCTURE
7. NEW INFLUENT PUMP STATION
8. NEW 16"Ø RAW SEWAGE (RS) LINE
9. NEW HEADWORKS (HW)
10. NEW OXIDATION DOUBLE DITCH
11. NEW SECONDARY CLARIFIER
12. NEW TERTIARY FILTER
13. NEW UV DISINFECTANT
14. NEW RAS/WAS PUMP STATION
15. NEW BLOWER BUILDING
16. TO RECYCLED WATER STORAGE POND
17. NEW GARAGE WITH BIOSOLIDS STORAGE
18. NEW SLUDGE PRESS WITH DRY BIOSOLIDS STORAGE



THE DYER PARTNERSHIP ENGINEERS & PLANNERS, INC.	***CITY OF SUTHERLIN WASTEWATER TREATMENT PLAN***	FIGURE NO.
DATE: JAN., 2010	***ALTERNATE NO. 4 - OXIDATION WITH MBR***	***6.3.3D***
PROJECT NO.: 146.02		

The advantages of this system include its reliability and performance with respect to hydraulic surge and shock load. This is due to the system's characteristic flow attenuation due to volume and complete mix characteristics. The system also produces less sludge than other biological processes because of extended biological activity during the activated sludge process. The system is also very energy efficient with respect to conventional activated sludge processes. Disadvantages include high effluent suspended solids concentrations and the large land area required with respect to other processes.

The two-cell design considered for purposes of comparison in this study is based on the type of oxidation ditch that contains brush rotors. For maintenance purposes, all the inlet flow can be diverted to one of the cells during the lower flow season. There are various types of aeration and drive equipment available.

Each oxidation ditch cell would consist of an elongated concrete basin structure with a side water depth of 11 feet. Each basin would be 144 feet long by 73 feet wide. The waste sludge is pumped and measured from each basin to the aerobic digester with small submersible pumps. Common wall construction would be utilized to minimize the oxidation ditch footprint and construction costs.

The Oxidation Ditch alternative with Class A tertiary filtration has an estimated construction cost of $7,494,000, including all necessary WAS and RAS pumps, aeration devices, piping and clarifier equipment.

The capital cost of an MBR coupled with downsized Oxidation Ditches, clarifiers and RAS/WAS pumping station is estimated to be approximately $10,740,000.

The costs are developed in Appendix C and summarized in Table 6.3.1. Life cycle costs are summarized in Table 6.3.2.

TABLE 6.3.1
TREATMENT ALTERNATIVES CONSTRUCTION COST SUMMARY

Description		Filter with			MBR with		
		Oxidation Ditch	Conventional	SBR	Oxidation Ditch	Conventional	SBR
	Est. Cost	$ 3,348,000	$ 2,104,000	$ 4,487,000	$ 3,248,000	$ 1,788,000	$ 2,927,000
Clarifiers w filter	$ 2,624,000	●	●				
Clarifiers w MBR	$ 2,440,000				●	●	
MBR	$ 4,534,000				●	●	●
Tertiary Filter	$ 1,004,000	●	●	●			
Ras/Was Building	$ 518,000	●	●		●	●	
Total Basic Construction		$ 7,494,000	$ 6,250,000	$ 5,491,000	$ 10,740,000	$ 9,280,000	$ 7,461,000

Tertiary Treatment Alternatives

Each of the filters evaluated is sized based on a MMDWF of 1.88 MGD and on the ability to meet the requirements for Class A recycled effluent.

1. **Upflow Sand Filter.** An upflow gravity filter utilizing the moving bed design provides a continuous supply of filtered water without the interruptions of backwash cleaning cycles. Influent enters the center of the filter through a feed chamber and flows downward through the central feed chamber and radial arm system. Filtered water is

collected in the effluent nozzle located at the top of the filter cell, after passing through a minimum of 40 inches of filter media. Solids captured in the filter bed are drawn downward with the sand into the suction of an airlift pump. The turbulent, upward flow in the airlift provides a scrubbing action that effectively separates the sand and solids before discharging into the filter washbox. The washbox is a baffled chamber that allows for gravity separation of the cleaned sand and the concentrated waste solids. This process is accomplished by utilizing filtrate water to clean the contaminated sand. From here, the regenerated sand is returned to the top of the filter bed, and the solids, or "reject", are piped to a suitable disposal point. The SSF filters will produce a filtrate with <5 mg/l TSS, <2 NTU.

Filtration would be accomplished within three circular tanks, each having overall dimensions of 144 inches inside diameter,17 feet 4 inches of height and an overall area of 113 square feet for each tank. The airlift sand cleaning system will be capable of continuously backwashing and internally recycling the filter sand once every 8 to 12 hours. The estimated construction cost of these units is approximately $1,078,000.

2. **Disk Filter.** The influent flows by gravity into the filter discs from the center drum. Solids are separated from the water by the filter media mounted on the two sides of the discs, which are partially submerged. With this arrangement, the solids are retained within the filter discs while only the clean water flows to the outside of the discs and into the collection tank. This allows for the effective removal of large solids and floatable material. Maintenance is reduced since there is no accumulation of solids in the tank. During normal operation, the discs remain static until the water level in the inlet channels rises to a specific point, which then automatically initiates the backwash cycle. The filtered effluent provides a perfect source of backwash water, eliminating the need for a separate source of cleaning water or an additional clean water collection tank. Clean effluent is pumped to the backwash spray header and nozzles, washing solids into the collection trough as the discs rotate. The filter will produce a filtrate with <5 mg/l TSS, <2 NTU.

   The estimated construction cost of these units is approximately $1,004,000.

3. **MBR Biology and Filtration.** The MBR process is a suspended growth-activated sludge system that utilizes microporous membranes for solid/liquid separation in lieu of secondary clarifiers or tertiary filters, thus resulting in a smaller footprint. The facilities used are a concrete rectangular tank divided into three cells (influent equalization, anoxic zone, and aerated/filtration zone), pumps, electrically actuated valves, blowers, level controls, a programmable logic controller (PLC), and ultra filtration membrane filter. The screened raw wastewater is discharged to the anoxic cell. The raw wastewater is mixed with recirculated mixed liquor in the anoxic cell and then flows to the aeration cell. In the aeration cell, the wastewater is aerated through a grid of fine bubble diffusers connected to positive displacement blowers. The micro-filtration membranes are immersed directly in the aerated mixed liquor and are either gravity fed or connected to the suction side of a centrifugal pump. The wastewater is biologically treated in the anoxic/aerobic bioreactor and the clean permeate is drawn through the membranes and discharged to the disinfection system.

   MBRs are easy to operate, require less maintenance, have less disinfection requirements, and produce a superior effluent quality. Effluent quality is less sensitive to operations, and precise control of the sludge residence time/mixed liquor suspended solids/food to

microorganisms ratio ((F:M)/(MLSS)/(SRT)) is not as important. MBR technology puts much greater distance between reclamation and the risk of microbial disease. Pathogens are not just reduced by a highly selective chemical or photochemical reaction; they are rejected by size exclusion. Effluent suspended solids are in the less than 5 mg/L range and turbidity is <1 NTU. Effluent quality is very suitable for Class A recycled water applications.

MBRs are proposed which would treat the entire summertime flow. Each MBR train has a pre-anoxic cell, a post-anoxic cell, an aeration cell and a membrane cell. The total concrete structure would be 68 feet wide by 140 feet long. The estimated capital construction cost of this unit is approximately $4,534,000.

Selection of Treatment Alternative

The six liquid stream treatment schemes are: 1. Conventional with tertiary filtration; 2. SBR with tertiary filtration; 3. Oxidation Ditch with tertiary filtration; 4. MBR with conventional treatment; 5. MBR with an SBR; and 6. MBR with an Oxidation Ditch. These alternatives were examined and compared for suitability and cost. Life Cycle Cost comparisons for liquid stream treatment alternatives are summarized and presented in Table 6.3.2.

TABLE 6.3.2
PRESENT WORTH COSTS FOR LIQUID STREAM TREATMENT ALTERNATIVES

Item	Conventional with Filter	SBR with Filter	Oxidation with Filter	MBR with Conventional	MBR with SBR	MBR with Oxidation
Capital	$6,250,000	$5,491,000	$7,494,000	$9,280,000	$7,461,000	$10,740,000
O&M P.W. (1)	$1,441,000	$1,499,000	$2,450,000	$3,350,000	$3,408,000	$4,359,000
Salvage Value	($50,000)	($50,000)	($50,000)	($50,000)	($50,000)	($50,000)
Total	$7,641,000	$6,940,000	$9,894,000	$12,580,000	$10,819,000	$15,049,000

(1) O & M Present Worth costs for alternatives are developed in Appendix C.

Non-cost Matrix Evaluation
For the matrix evaluation, a rating system was employed to compare the alternatives. This rating system consists of a three-point scale, three being the best and one, the worst. Two or more alternatives may have the same rating for a particular parameter. The ratings are subjective. The ratings for the matrix evaluation are summarized in Table 6.3.2.

Flexibility
Flexibility is an important characteristic of liquid stream alternatives, since large variations in flow may be encountered during the wet weather season. An SBR system eliminates the need for a return activated sludge system, since it is not needed in the process. The SBR system provides treatment and settling within a single tank, which is a distinct flexibility advantage over other considered alternatives. In addition, high flows can be treated simply by cycle timing control.

Reliability
The oxidation ditch and the sequencing batch reactor (SBR) are considered the most reliable, requiring less moving parts and simpler infrastructure.

Operability
Each alternative uses equipment and processes that are familiar to the plant operators. The oxidation ditch is considered to be the simplest to operate. The oxidation ditch would require less operator attention. The current WWTP staff has the required skill level for either operation.

Ability to Construct
The oxidation ditch requires the largest amount of usable land and larger footprint. The SBR is considered the simplest to construct.

Environmental Factors
Each alternative poses equal potential for environmental impacts.

Community Impact
There is no real difference between these options.

Expandability
The SBR is the most expandable alternative, as common walls may be utilized for expansion.

TABLE 6.3.3
MATRIX EVALUATION OF LIQUID STREAM TREATMENT ALTERNATIVES

Parameter	Conventional with Filter	SBR with Filter	Oxidation with Filter	MBR with Conventional	MBR with SBR	MBR with Oxidation
Flexibility	2	2	2	2	2	2
Reliability	2	3	3	2	2	2
Operability	2	2	3	1	1	2
Ability to Construct	3	3	2	1	2	1
Environmental Factors	2	2	2	3	3	3
Community Impact	3	3	3	3	3	3
Expandability	2	3	1	1	2	1
Total	16	18	16	13	15	14

The results above indicate Alternate 3 (Figure No. 6.3.2B), SBR with Filter, should be selected based on both cost and operations considerations.

6.4 Disinfection Alternatives

During winter months effluent is discharged to Calapooya Creek, which is classified as waters of the state and is located within the Umpqua River Basin. Calapooya Creek is water quality limited and the Umpqua Basin TMDL limits contaminants such as chlorine. Either chlorine disinfection with dechlorination or ultraviolet light disinfection would be the appropriate technology when discharging to waters of the state where no chlorine residual is allowed.

Minimum chlorine contact times, based on DEQ guidelines, are 15 minutes at peak hourly flow (PIF) and 20 minutes at peak daily average flow (PDAF) and 60 minutes at the average dry weather flow (ADWF), whichever results in the greater treatment capacity. A minimum length to width ratio of 40:1 is required. The existing chlorination tank is 105,500 gallons with some baffling. Existing contact time for 2012 PDAF is 14 minutes, and for 2012 PIF, it is 10 minutes. The chlorine contact tank is undersized for the 2012 flows. In addition, the existing chlorine contact chamber has no redundancy. If the year-round

chlorine disinfection is selected, a new chlorine contact tank, new disinfection equipment, and a new dechlorination chamber would be required.

Currently winter effluent is chlorine disinfected, then dechlorinated and discharged to Calapooya Creek. Due to health concerns, chlorine disinfection and the potential toxicity of residual chlorine in receiving streams is not desirable. Chlorine is a toxic substance. Overdosing during dechlorination may also be toxic for aquatic organisms in receiving streams. Ultraviolet (UV) disinfection is preferred to chlorine disinfection when discharging to waters of the state.

During summer months, effluent may be recycled as irrigation water and/or stored for wintertime disposal. The irrigation option requires 200 acres of crop land. Open storage ponds are subject to poor water quality created by algae growth (which clogs sprinklers), breeding of vectors, slimes, aesthetically displeasing deposits and odors. Chlorine residual can effectively improve a storage reservoir's water quality. Therefore chlorine-based disinfection is recommended to treat summertime effluent.

The four basic alternatives examined were: No Action, Chlorine Disinfection with Dechlorination, Ultraviolet Light Disinfection (UV), and a combination of summertime chlorine disinfection and wintertime UV.

Basic Alternatives

1. **No action.** With this alternative, the wintertime effluent will continue to be disinfected via the existing gas chlorination system. Chlorine gas costs have recently quadrupled for the City and delivery and logistics have become less reliable. This has resulted in new risks and uncertainties for the City regarding the usage of chorine gas.

   The advantages of a standard chlorine system include the fact that it is an established technology, effective and reliable. The disadvantages are that residuals are toxic to aquatic life. Acquisition of the chemical is increasingly difficult and it may react with chemicals in the wastewater to produce toxic gases. Chlorine use presents safety, maintenance and fire hazards to personnel. In addition, chlorine gas is becoming increasingly costly.

2. **Hypochlorite Solution Disinfection.** This alternative is the injection of a Hypochlorite disinfection Solution (HS) into the effluent at an appropriate treatment dose. Hypochlorite solution is either purchased in bulk and stored on-site in tanks, or is manufactured by On-Site Generation (OSG) as an oxidant solution. The process of on-site generation uses salt, water, and power to create a chlorine-based disinfectant (or oxidant), thereby eliminating the transport and storage of hazardous chemicals. The salt feedstock is fully converted, resulting in negligible addition of sodium to the treated water and no negative impact to irrigated landscapes. A hypochlorite solution or OSG oxidant solution is typically inject directly into the effluent stream as it enters a contact chamber or storage pond.

   The advantages of a hypochlorite solution-based system are: 1) improved operator safety; 2) greener applications; and 3) cost savings. HS systems are less sophisticated systems, very reliable, and are easier to operate and maintain. OSG systems produce a non-hazardous oxidant solution due to the lower chlorine concentrations. OSG systems typically produce a disinfection solution at a much lower cost than the purchase and delivery of a HS. The mixed oxidant solution has additional advantages for irrigation

applications. The residual is effective in reducing bio-film on irrigation pipe walls and improving reservoir water quality.

The disadvantages of bulk purchasing of hypochlorite solution (HS) are cost and storage. To make an economic purchase the solution should be purchased in large volumes; however, the storage life is only around 30 days. The disadvantage of an OSG system is the requirement to use soft water for generation of the disinfectant solution. Soft water, however, causes electrochemical cells to develop scales over time, impeding the ability of the cells to generate chlorine, which could eventually destroy the electrodes. OSG produces hydrogen which is colorless, odorless, and flammable. Hydrogen is lighter than air; consequently, good ventilation is required.

Construction cost for an HS injection system, including equipment, pumps, large storage tank, building and contact chamber is $165,000. An OSG-based system is estimated at $275,000.

Wintertime use of chlorine-based disinfection requires a chlorine contact chamber with sufficient size and redundancy to allow continuous operations during maintenance and cleaning. The wintertime option would require a new chlorine contact chamber at an estimated amount of $892,000. This is in addition to the chlorination equipment cost. A summary of capital costs and life cycle costs is shown in Table 6.4.1.

3. **Ultraviolet disinfection.** Ultraviolet disinfection (UV) is a process in which ultraviolet energy is introduced into water or wastewater for the destruction of microorganisms. An ultraviolet system would consist of a concrete or stainless steel channel, ultraviolet lighting banks and weirs or weighted control gates to ensure that the lighting banks are always submerged.

The effectiveness of this process is dependent upon the penetration of the rays into water. Wastewater characteristics such as transmittance, suspended solids concentration, and presence of constituents that can absorb UV light decrease the intensity of light within the reactor and hence adversely affect performance. The key advantages of UV disinfection are that no residual is left in the treated effluent that could affect aquatic life in the receiving waters, and there are no hazardous materials for operators to handle. A disadvantage is that the performance of UV disinfection can be affected by characteristics of the wastewater stream such as color, suspended and colloidal solids, and chemical compounds (e.g. iron). Another disadvantage in association with pond storage of UV-treated effluent is the lack of chlorine residual. Chlorine residual will discourage algae growth and vectors. UV disinfection is generally not acceptable for use on storage pond systems due to transmittance interference caused by algae.

The main components of a UV disinfection system include mercury arc lamps, a reactor, and ballast. The source of UV radiation is either from low-pressure or medium-pressure mercury arc lamps. The optimum wavelength to effectively inactivate microorganisms is in the range of 250 to 270 nanometers (nm). Low-pressure lamps, which are mostly used at small facilities, emit essentially monochromatic light at a wavelength of 253.7 nm. Medium-pressure lamps are generally used for large facilities and have approximately 15 to 20 times the germicidal UV intensity of low-pressure lamps. However, the medium-pressure amps are more costly and operate at higher temperatures with higher energy consumption.

Other differences between low-pressure and medium-pressure lamp systems include methods of installation and cleaning. Low-pressure lamp systems are installed in concrete or fabricated steel open channels. Medium-pressure systems can either be installed in open channels or closed vessels (horizontal or vertical). Automatic lamp cleaning is possible with medium lamp systems, which reduces the labor costs as compared to manual cleaning for low-pressure systems.

Installation of a new UV disinfection system to serve the wintertime effluent flows is estimated at $298,000 for the structure and $231,000 for the equipment, totaling $529,000.

4. **HS or OSG Combined with UV disinfection**. This alternative is the application of UV disinfection in the wintertime (discharge to Calapooya Creek) with either a HS or OSG during the summertime (discharge to the storage pond and land applied). During summertime, the HS or OSG disinfection solution would be injected directly into the force main leading to the storage pond, thus eliminating the need for the chlorine contact chamber. The advantage of this approach is that during wintertime, effluent is of higher quality and environmentally friendly. There is no chemical residual. During the summer when effluent is stored in a pond, the HS or OSG residual will help reduce the growth of algae. The disadvantage is the cost of installing, operating and maintaining two disinfection systems.

   A combined HS/UV disinfection system has HS equipment cost of $165,000 and UV equipment cost of $231,000 for a total equipment cost of $396,000. A combined HS/UV disinfection system has HS structure of $85,000 and UV structure cost of $231,000 for total structures costs of $316,000. The total construction cost for a summertime HS system combined with a wintertime UV system is $779,000.

   A combined OSG/UV disinfection system has OSG equipment cost of $275,000 and UV equipment cost of $231,000 for a total equipment cost of $506,000. A combined HS/UV disinfection system has OSG structure of $85,000 and UV structure cost of $231,000 for a total structures cost of $316,000. The total construction cost for a summertime HS system combined with a winter time UV system is $889,000.

Selection

Costs comparisons for the effluent disinfection options are presented in Table 6.4.1.

TABLE 6.4.1
PRESENT WORTH COSTS FOR DISINFECTION ALTERNATIVES [1]

	Year-Round HS Disinfection	Year-Round OSG Disinfection	Year-Round UV Disinfection	HS/UV Disinfection	OSG/UV Disinfection
Capital					
Equipment	$ 165,000	$ 275,000	$ 231,000	$ 396,000	$ 506,000
Structure	$ 892,000	$ 892,000	$ 298,000	$ 316,000	$ 316,000
O&M	$ 245,000	$ 220,000	$ 210,000	$ 309,000	$ 303,000
Total	**$ 1,302,000**	**$ 1,387,000**	**$ 739,000**	**$ 1,021,000**	**$ 1,125,000**

(1) Present worth costs were based on 3.2 percent
(2) Interest over 20 years

For the matrix evaluation, a rating system was employed to compare the alternatives. This rating system consists of a three-point scale, three being the best and one, the worst. These ratings are subjective evaluations. Two or more alternatives may have the same rating for a particular parameter. The ratings for the matrix evaluation are summarized in Table 6.4.2.

The parameters that were considered are described as follows:

Flexibility
Flexibility is an important characteristic of disinfection alternatives, since large variations in flow may be encountered during the wet weather season and effluent storage is required during the summertime. Hypochlorite solution (or OSG) disinfection is effective and easily adjusted for varying conditions.

Capacity
Hypochlorite disinfection may be easily adjusted for varying flow rates, while UV systems have fixed capacity.

Reliability
The effectiveness of UV systems is dependent upon the effluent turbidity and flow rate.

Operability
A UV system requires more operator time and cleaning. Hypochlorite solution disinfection requires more testing. Operation demands are higher when operating a wintertime (UV) and summertime HS (or OSG) combination.

Ability to Construct
Wintertime Hypochlorite (or OSG) disinfection requires the construction of a dual channel contact chamber. A chlorine contact chamber has baffling, interior concrete channel walls, requires twice the size due to redundancy and is therefore more difficult to construct, UV and OSG systems are more sophisticated, also requiring more effort during construction. A combined UV and HS system is the easiest to construct.

Environmental Factors
Hypochlorite disinfection systems are increasingly scrutinized regarding their impact on the environment, including residuals in receiving streams. However, hypochlorite-based systems with a residual help control algae growth and vectors in the storage ponds. UV systems do not require chemical handling and effluent is considered more environmentally friendly to receiving streams.

Community Impact
UV systems are considered to have fewer impacts to the community, as discussed in Environmental Factors above.

TABLE 6.4.2
MATRIX EVALUATION OF DISINFECTION ALTERNATIVES

Parameter	HS or OSG Disinfection	UV Disinfection	OSG / UV Disinfection	HS / UV Disinfection
Flexibility	3	3	2	2
Capacity	3	3	2	2
Reliability	3	3	2	2
Operability	2	2	2	2
Ability to Construct	2	2	3	3
Environmental Factors	2	3	3	3
Community Impact	2	2	3	3
Total	**17**	**18**	**17**	**15**

The recommended alternative is year-round UV disinfection due to flexibility, reliability and environmental impacts.

6.5 Recycled Water

The City's NPDES permit prohibits discharge from June 1 through October 31 and conditional release in October and May. The Umpqua Basin TMDL report (which addresses bacteria, temperature, nutrients, and bio-criteria) determined Calapooya Creek is water quality limited and established Waste Load Allocations (WLA). At the City's request, DEQ collected additional data to support a WLA during October. Discharge to Calapooya Creek is allowed in October conditionally, based upon a minimum stream flow requirement. See Section 5. This WWFP assumes that there is inadequate creek flow and that an October release will not be allowed.

Currently, recycled water is stored in a 2-acre golf course pond and irrigated over 95 acres. Current setbacks allow for use of Class A recycled water. In 2012 the June through October recycled water volume was 99 million gallons. Thirty nine million gallons was irrigated on the golf course and 60 million gallons was released to Calapooya Creek.

Land application area and/or storage pond sizing was determined by calculating a water balance with estimated average dry weather in the year 2037. A water balance analysis found that October disposal determines the sizing of the facilities. An 8-foot side water depth was utilized in determining a pond's surface area. Land application (irrigation) requires a 200-acre irrigation site coupled with a 30-acre storage pond. Using a storage pond only requires a minimum of 60 acres. If Fords Pond (95 acres) is utilized, the pond side water depth would be four feet. A minimum depth of six feet is recommended for control of algae.

Alternative cost estimates are included in Appendix C.

Options:

1. **No action, continue to land apply at the golf course.** The golf course facilities are undersized for current conditions. This course of action is not in compliance with the NPDES permit.

2. **Storage and Land Application (Irrigation).** The golf course has approximately 95 acres of land that is currently being irrigated. An additional 105 acres of irrigation land along with 30-acre storage pond is required to dispose of the year 2037 dry weather

flows. Considering setbacks and pond berm construction, this analysis assumes an additional 160 acres is required. This option is dependent on construction cost as well as the ability to acquire developable land in the vicinity of the wastewater treatment plant. The estimated cost including land acquisition, effluent pump stations, two miles of force main and storage pond development is $4,714,000.

3. **Pond Storage.** Class A effluent may be stored entirely in a pond and released in November. The size of the pond would be 95 acres, reaching up to four feet deep by the end of October. Fords Pond is one potential site that is located within 2,000 lineal feet of the treatment plant. Effluent would be pumped to the pond from June through October and metered out over the winter. The advantages of utilizing Fords Pond are that it's a permitted use, is close to the treatment facility, and does not require dredging. Fords Creek would be intercepted and diverted south, along Church Road then along the toe of the existing berm to the pond outfall. This option is dependent on construction cost as well as the ability to acquire the property. The estimated cost including land acquisition, effluent pump stations, force main, and storage pond development is $3,742,000.

Selection

Class A effluent disposal is dependent on the availability of nearby property for storage and possible irrigation. The analysis assumes pond storage and release at a metered rate over the winter. For budgetary purposes $3,742,000 is utilized.

6.6 Biosolids Treatment, Storage and Disposal Alternatives

Biosolids originate as leftover waste materials, domestic septage and sewage sludge, which are generated from sewage treatment. Presently biosolids produced at the WWTP are aerobically digested. Excess biosolids are transported to a licensed private treatment facility near Roseburg and ultimately land applied on a DEQ-approved site. Sutherlin does not have the digester capacity necessary to meet the existing sludge load based on the current design. The minimum mean cell residence time is determined by the following formula: (Time @ T°C)/(40d) = 1.08(2—T). This report utilizes 60-day mean residence time for comparison purposes. Selection of the most viable biosolids stabilization alternative is dependent upon the selected ultimate use and disposal of the biosolids. The following is a discussion of the biosolids stabilization and ultimate use/disposal alternatives.

Biosolids Stabilization

Biosolids stabilization is a treatment process which converts sludge generated in the liquid stream treatment process to a stable product for ultimate disposal or use. This process reduces pathogens and vector attraction in the sludge and produces a less odorous product. The most common biosolids stabilization processes used in small communities are facultative sludge lagoons, aerobic digestion, anaerobic digestion, and lime stabilization. While not typically utilized in small communities, composting is considered a potential stabilization alternative. The use of facultative sludge lagoons was not considered a viable option for biosolids stabilization since these facilities require relatively large amounts of constructible land, which is at a premium in the vicinity of the WWTP. The use of anaerobic digestion has been ruled out because of the complexity of operation, the problem of methane disposal and the potential for odor production.

The Sutherlin WWTP currently uses aerobic digestion for sludge stabilization, followed by storage, land application or hauling to a private disposal facility on an as-needed basis. The existing aeration basin

provides additional sludge storage but the volume of this basin combined with the aerobic digester is not adequate for existing loads. Excess biosolids must be hauled to a private facility.

Adequate digestion is achieved based on test results of the specific oxygen uptake rate (SOUR) test and fecal coliform counts. They do not meet volatile solids reduction time (less than 60 percent) due to the lack of detention times in the digesters. The solids retention time (SRT) is approximately 30 days, which is half the recommended 60 days. The existing digesters are approximately 50 percent undersized for existing loads.

Currently, the City is utilizing a combination of Biosolids Land Application and disposal at a private facility (Heard Farm). Biosolids are land applied at four different sites, the furthest away being approximately 40 miles from the treatment facility. On average, 990,000 gallons of biosolids are land applied each year. An average of 590,000 gallons are disposed of each year at a tipping fee of $0.07 per gallon. Several community rates have been raised to $0.12 per gallon.

With the recommended new secondary treatment system, additional digester and storage space will be required. Each of the existing treatment units will be converted to digestion. The volume of the existing treatment units is 978,000 gallons (existing contact tank, stabilization tank, and secondary clarifier). Utilizing an average of 1% solids (12,380 gallons per day by 2040) the mean residence time would be 78 days. Concentrating the sludge to 2% solids will generate 2,259,000 gallons of biosolids per year.

Biosolids Storage

Biosolids can be stored within the wastewater treatment process units, biosolids treatment process units, or in separate specially designed tanks. Wastewater treatment units can store biosolids for short-term storage (a few hours to 24 hours). For longer detention times, biosolids treatment units, such as aerobic or anaerobic digesters, and facultative sludge lagoons are used for storage. Separate tanks are usually used for obtaining longer detention times. These separate holding tanks often use mixing and/or aeration to prevent septicity, odors, and solids suspension. Mixing may be accomplished using diffused air and top-entry or submersible mechanical mixers. Other odor control measures include either chemical addition of chlorine, hydrogen peroxide or iron salts, and maintenance of an aerobic surface layer (e.g. facultative sludge lagoon).

Storage is required to hold biosolids until they can be disposed of. Disposal by land application can typically be completed between the months of May through October. Land use, such as harvesting hay, can reduce this window to August through October. For purposes of this study, storage facilities are required for a minimum of eight months. With a new treatment process, the existing tank processes can be converted to sludge storage. Long-term storage of biosolids that are not completely dried can create serious odor problems.

Basic Alternatives

1. **Facultative Sludge Lagoons**. Typically in small communities, facultative sludge lagoons have been recommended and implemented for biosolids stabilization and storage. However, the use of a facultative sludge lagoon in Sutherlin for biosolids is not considered viable due to lack of appropriate sites and lack of available property in the vicinity of the treatment plant.

2. **Drying Beds**. Drying beds are contained structures with the floor sloping to a drain system. A layer of gravel is built up over the drains, and a layer of sand is applied over the gravel; the surfaces of the beds are flooded with digested biosolids. The liquid content of the biosolids drains through sand and gravel and is returned to the headworks

of the plant. Dewatered biosolids are scraped off after each application, along with the top layer of the sand, using a small front-end loader. The biosolids are hauled by dump truck and disposed of by landfill or land applied. The solids content of the finished biosolids may vary from 15% to 70%, with 16% used as an estimate for study purposes. However, aerobic digested sludge is more difficult to dewater than anaerobic digested sludge and requires a much longer time to dewater. Covered beds would be required due to the volume and duration of rainfall in the area. Sutherlin would require 29,300 square feet of covered sludge drying beds based on provision of 2.5 square feet/capita for sludge dewatering. This would require a footprint of 170 feet by 170 feet.

Disadvantages include odor concerns and multiple handling of the material; it must be spread, scraped up, loaded into a truck and then tilled in at the land application site. Use of the drying beds also requires access to a small front-end loader, dump truck and manure spreader. Drying beds are less efficient in Sutherlin's wet climate. The estimated cost for this improvement is approximately $2,174,000.

3. **Dewatering Press.** A dewatering press is used to decrease the total volume and moisture content of sludge, reducing the required storage space and the number of trips eventually necessary to haul biosolids offsite. Digested sludge is treated with polymer to allow flocculation and easier dewatering. The press produces liquid pressate, which is pumped back to the headworks for further treatment, and a dewatered sludge with a solids content of approximately 14 percent solids. A storage area for the thickened biosolids would be required. Removal of the thickened sludge would require a front loader or other mechanical means of loading and a manure spreader for land application, or land fill disposal. Disposal is not weather-dependent and can occur year-round.

   A press suitable for a community the size of Sutherlin can process sludge at about 375 pounds of dry solids per hour. The projected 2037 solids load for the WWTP is about 1,700 pounds per day, depending on the biological process selected. A dewatering press would need to run about 32 hours per week to dewater the digester biosolids to 14 percent. The resulting "cake" could then be stored in a sludge press building until land application is possible. Press operation is considered reliable, but is dependent on the level of maintenance and skill of the operator. In the case of Sutherlin, the liquid sludge storage prior to dewatering greatly increases the reliability of the process. As previously noted, provision of a press will allow conversion of the existing aeration basin for liquid sludge store prior to drying. If a dewatering press is not provided, additional liquid sludge storage is required. The estimated cost for the dewatering press/dry storage is $1,160,000.

4. **Tank Storage and liquid disposal.** Tanks for holding liquid biosolids need to be large enough to get through the period between land application seasons. Currently Sutherlin land applies 990,000 gallons of biosolids per year and disposes of 590,000 gallons at a private facility. By the year 2037 the treatment plant is estimated to generate 2,259,000 gallons of 2% liquid biosolids per year. The 990,000 gallons of land application biosolids will be digested and stored in the existing treatment tanks. By the year 2040 an additional 1,269,000 gallons of 2% bisolids will require storage or be disposed of on a year-round basis. Utilizing the existing process tanks as sludge storage provides 995,000 gallons of storage.

   The advantage of tank storage of liquid sludge is that there is minimal labor involved in the use of a storage tank and aeration equipment, and a certain amount of aerobic

digestion would continue. The disadvantage of tank storage is the volume of material to be managed, the larger amount of aeration required to maintain storage, and hauling cost for disposal of large volumes of liquid biosolids.

The plant expansion will allow the existing treatment basins (stabilization tank, contact tank, and clarifier) to be converted to digestion and some sludge storage. The estimated cost of tankage to store 1,269,000 gallons of 2% liquid biosolids is $1,419,000. This excludes the operational cost of liquid biosolids disposal.

Disposal of Biosolids

Disposal alternatives for WWTP biosolids were evaluated with respect to regulatory requirements for pollutant limits (i.e. 40 CFR Part 503, Subpart B) and to agronomic rates for the on-site vegetation (i.e. nitrogen). The Part 503 rule requires that biosolids be land applied at a rate that is equal to or less than the agronomic rate for nitrogen at the application site. Additional Part 503 requirements include the following (EPA 1995):

- Biosolids cannot be land applied unless trace element concentrations in the sludge are below ceiling concentrations specified in Part 503.

- Biosolids must meet either (1) the pollutant concentration limits specified in Table 3 of Part 503 or (2) the Part 503 cumulative pollutant loading rate (CPLR) limits for bulk biosolids.

The ultimate end use or disposal of biosolids is often the area of greatest uncertainty in sludge handling because of its dependency on solids marketability, land availability, and regulatory requirements. Another important consideration of the potential end use or disposal option is public acceptance. The reluctance of the public to accept a biosolids disposal or processing facility in their area generally stems from concerns about odors and adverse health impacts. Potentially viable options for use and disposal of biosolids include disposal at a landfill, land application, and distribution by marketing. The WWTP improvements proposed will provide the opportunity to use existing digestion tankage for sludge storage.

Basic Alternatives

1. **Land Application.** Land application refers to any beneficial use project that applies biosolids to the land. Such land sites include primary agricultural land, pastures, tree farms, and old mines. Any biosolids to be land applied must be classified as non-hazardous and meet criteria for maximum allowable concentrations of trace metals (e.g. cadmium, copper, lead, nickel and zinc). For application to agricultural lands, all biosolids must undergo treatment by a process to significantly reduce pathogens. In addition to evaluating biosolids with respect to environmental suitability, a land application program will depend on the nutrient content of the biosolids, the land to which it will be applied, and the crops to be grown on the land. For most biosolids produced and land applied, the limiting factor is the nutrient content of the biosolids when it is applied as a fertilizer for a particular crop.

   Land application is dependent on site conditions and weather. Prolonged heavy rains could greatly reduce the flexibility of liquid land application. The dry sludge land application alternative is relatively unlimited and offers the most flexibility. In addition, land application sites are becoming more difficult to locate and maintain as long-term disposal sites.

The anticipated land application program would operate (by city staff) for a minimum of four months and land apply 990,000 gallons per year. The remaining 1,269,000 gallons would be disposed of at a private facility such as Heard Farm an $0.07 per gallon "tipping fee". The key advantages of local land application are the ability to utilize wastewater biosolids for a beneficial use.

2. **Landfill Disposal.** Landfill disposal is a generally less desirable alternative than land application for beneficial use. If a suitable site is convenient, a commercial facility may be used for the disposal of biosolids if facility operator and regulatory officials permit this practice. The economics of hauling biosolids usually indicates that the dewatering for volume reduction will result in justifiable savings. While this process is more expensive and does not take advantage of the beneficial uses of biosolids, disposal at a landfill is a viable option when weather conditions or regulatory requirements limit land application.

   The City can haul to a Douglas County landfill, provided the digested biosolids are dewatered and meet certain testing criteria. The disposal fee is $60 per ton.

3. **Distribution and Marketing of Biosolids**. Compost and heat-dried (Class A) biosolids may be distributed and marketed to end-users such as the agricultural and horticultural industries, landscape contractors, and homeowners. Each municipality must develop its particular distribution and marketing strategy based on surveys of potential users and competing products. Some municipalities have chosen to market the product through a broker or distributor. Items such as product quality, selling price, storage, responsibility for unsold product, and other risk-sharing decisions should be included in any contracts. Promotional and demonstration programs are usually required to promote public attention and acceptance, and inform potential users of the product's potential use and availability.

   The distribution and marketing of processed wastewater biosolids is usually only done by larger municipalities (e.g. Portland, Newberg) that produce considerable amounts of biosolids. These municipalities usually have the resources to successfully develop a product market. Sutherlin currently produces Class B biosolids and would need to further process the waste to achieve Class A. Class A material could be used directly by the City for fertilizing plantings in parks, at local schools and on other municipal property. Surplus could be given away to the public or farmers.

   EPA-approved methods of achieving Class A biosolids include composting, irradiation and heat treatment. Sutherlin lacks adequate space for composting, the public acceptance for irradiation, or an inexpensive energy source for heat treatment. With the current economic and regulatory climate, producing Class A biosolids is not cost-effective and this method is excluded from further consideration

Estimated capital and Operation and Maintenance (O&M) costs were compiled for biosolids storage, dewatering, holding, and land application in Table 6.6.1.

TABLE 6.6.1
PRESENT WORTH COSTS FOR BIOSOLIDS STORAGE ALTERNATIVES

	Drying Beds	Dewatering Press	Storage Tank
Captial Cost	$2,126,000	$1,069,000	$1,419,000
O&M P.W. (1)	$492,000	$345,000	$253,000
Salvage Value	($30,000)	($15,000)	($25,000)
Disposal Cost(1)			
Land App.			$1,702,000
Land Fill	$1,714,000	$1,714,000	
Total	$4,302,000	$3,113,000	$3,349,000

(1) – Present worth costs were based on 3.2 percent interest over 20 years.

Selection

For the matrix evaluation, a rating system was employed to compare the alternatives. This rating system consists of a three-point scale, three being the best and one, the worst. Two or more alternatives may have the same rating for a particular parameter. The ratings for the matrix evaluation are summarized in Table 6.6.2.

TABLE 6.6.2
MATRIX EVALUATION SLUDGE DISPOSAL ALTERNATIVES

Parameter	Drying Beds	Dewatering Press	Storage Tank
Flexibility	2	3	2
Capacity	2	3	2
Reliability	2	2	2
Operability	2	3	3
Ability to Construct	1	2	2
Envir. Factors	2	3	2
Community Impact	2	2	2
Total	13	18	15

Based on the above analysis, screw press with dry storage and land application is considered the highest-ranking alternative.

6.7 Summary of Complete Wastewater Treatment Alternative

In the sections above, alternatives were presented and evaluated for each component of the wastewater treatment facility. The recommendations are to continue to use the existing site and to construct an SBR facility with a new headworks, UV disinfection system for wintertime flows, HS disinfection for summertime flows, and pond storage for summer effluent. The existing donut plant would be reused for digester space and the clarifier would be used for future biosolids storage. A further evaluation of the selected plan is presented in Section 7.

Recommended Plan

Section

7

Section 7

Recommended Plan

Improvements will be completed in two phases. Phase 1 improvements will bring the Wastewater Treatment Plant in compliance with the NPDES permit and corrects deficiencies at the Everett Avenue Pump Station. Phase 2 will be collection improvements such as I/I repair projects and increasing conveyance capacity.

7.1 Existing Piping System Improvements

It is recommended that work continue to identify and correct Inflow and Infiltration (I/I) in the existing system as follows:

1. The City should institute a TV program for the entire collection system over a five-year period (20% per year) and continue to repeat the TVing every five-year period.

2. Serious maintenance and repair issues should be identified in the I/I inspection program and should be corrected as funding sources become available.

3. A new sewer system evaluation survey (SSE) should be completed at least every ten years. A SSE should include line grit removal and cleaning, inspection of the lines with a TV camera, physical inspection of manholes, performance of flow testing at structures, smoke testing of lines, mapping results, development of I/I repair capital improvement projects (Phase 2) and performance of a cost-effective analysis.

7.2 Collection System Expansion

It is recommended that collection system expansion, as developed in Section 6, continue as required following the treatment plant improvements. The timing of the expansion is dependent upon the actual growth rate within the city.

7.3 Pump Station Recommendations

Plant Influent Pump Station

Construction of a new plant influent pump station is recommended in Phase 1. The existing Plant Influent Pump Station located inside the operations building will remain operational during construction of the new treatment facilities and will be decommissioned when the new treatment units, new plant influent pump station and new force main become operational. Estimated construction cost is approximately $816,000.

Everett Avenue Pump Station

It is recommended that a new pump station (wet well with submersible pumps) be constructed adjacent to the existing station. Detailed recommendations and improvement are provided in the *Everett Avenue Pump Station and Force Main Evaluation*. See Appendix E. The total costs of the new wet well and pumps are estimated at $925,000.

7.4 Treatment Facility Expansion

The existing flows exceed the hydraulic capacity of the Wastewater Treatment Plant (WWTP) and mass loads are at treatment capacity. The plant cannot meet current NPDES requirements on a consistent basis. The recommendation is to construct a new WWTP with the capacity to meet current and future projected flows for the year 2037. This should be accomplished by building an SBR biological treatment system, new headworks, new sludge press and a new UV treatment system. The existing clarifier, aeration basin and aerobic digester would be modified and reused as would the existing sludge storage basin. The existing outfall would continue to be used.

Costs have been included for the city to purchase additional properties. A plan view of the proposed wastewater treatment plant expansion was previously shown in Figure 6.3.2B. Capacities of the existing components to be reused and proposed components for a complete new wastewater treatment system are summarized in Table 7.4.1. The proposed plant expansion is sized for the projected population in 2037.

TABLE 7.4.1
SUTHERLIN WWTP COMPONENT DESIGN SPECIFICATIONS

Component	Type	New or Existing	Capacity Proposed
Plant Influent Lift Station	Submersible Pumps	New	9.0 MGD
Influent Flow Meter	Magnetic	New	9.0 MGD
Effluent Flow Meter	Magnetic	New	9.0 MGD
Influent Screen	Perforated Plate/Climber in-channel	New	9.0 MGD
Grit Removal	Centrifugal Vortex-powered	New	9.0 MGD
Grit Washer	Screw Classifier	New	9.0 gpm
SBR	Continuous Flow through	New	7.0 MGD
Blower	Positive Displacement	New	840 SCFM @ 8.1 psi
WAS Pump	Submersible	New	110 gpm
Decanter	Per Manufacturer	New	25' Weir x 4
UV Disinfection	Low Pressure High Intensity	New	6.25 MGD
Outfall	24" Concrete River Outfall	Existing	9.0 MGD
Aerobic Digester 1	Aerobic Digester	Existing	85,000 gallons
Aeration/Mixer	PD Blower	New	15 HP
Transfer Pump	Submersible Sludge	New	50 gpm
Aerobic Digester 2	Aerobic Digester	Existing	85,000 gallons
Aeration/Mixer	PD Blower	New	15 HP
Transfer Pump	Submersible Sludge	New	50 gpm
Sludge Storage Basins	Existing Treatment Units-2	Converted	740,000 gallons
Aeration/Mixer	PD Blower	New	15 HP
Transfer Pump	Submersible Sludge	New	30 gpm
Dewatering Press	12" diameter	New	375 lbs/hr
Dried Sludge Transport	End Dump	New	8-10 cy

Truck			
Generator	Diesel	New	400 kW

7.5 Biosolids Disposal

A large concern facing the WWTP operating staff is the timely removal and disposal of biosolids. The conversion of the contact tanks and stabilization tanks to aerobic digestion and the clarifier basins to digestion and sludge storage will provide sludge treatment for the design period. Storage will be accomplished by utilizing a press for dewatering and by providing dry storage in a bay adjacent to the public works garage. The improvements will be enough to avoid wet-weather application of sludge under any circumstances.

7.6 Project Cost Summary for Wastewater Treatment Plant

Capital costs for the recommended projects are summarized in Table 7.6.1. The project costs now are estimated to total approximately $21 million. The comparative cost differentials of the alternatives as presented in Table 6.6.1 are assumed to have remained generally the same.

TABLE 7.6.1
WWTP & WW SYSTEM IMPROVEMENTS COST ESTIMATE

Item	Cost
Influent Pump Station	$ 816,000
Headworks with Influent Screen	$ 1,247,000
SBR	$ 4,487,000
Tertiary Filtration	$ 1,004,000
UV Disinfection	$ 529,000
Recycled Water Disinfection System	$ 250,000
Convert Existing Tanks to Digestions	$ 775,000
Biosolids Processing Facility	$ 1,160,000
Site Structures	$ 555,000
Recycled Water Storage Pond	$ 742,000
Misc. Equipment	$ 380,000
Site Improvements and Yard Piping	$ 723,000
Basic WWTP Construction Total	**$ 12,568,000**
Engineering – Design – Bidding Services	$ 1,267,000
Engineering – Construction Services	$ 1,267,000
Value Engineering	$ 85,000
Contingency 15%	$ 1,900,000
Environmental Report	$ 45,000
Fords Pond Land Acquisition	$ 3,000,000
Review Fees	$ 10,000
Admin / Legal	$ 75,000
WWTP Project Total	**$ 20,317,000**

Everett Avenue Pump Station Improvements	$	925,000
Project Total	**$**	**21,242,000**

Yearly operation and maintenance costs for the WWTP will increase due to an increase in sludge production (from the new treatment plant) and the addition of the summer effluent storage pond disinfection. An estimate for the new WWTP operation and maintenance costs is provided in Table 7.6.2

TABLE 7.6.2
WWTP ESTIMATED YEARLY O&M

Item		Cost
Current Operations Materials and Services	$	586,650
Internal Cost Recovery	$	206,981
Personnel	$	408,134
STEP System Maintenance	$	32,000
Added Biosolids Press and Effluent Pond Disinfection	$	27,000
Total Yearly O&M	**$**	**1,260,765**
Annual Debt Payment	**$**	**83,429**

7.7 Project Schedule

In order to meet the conditions of the MAO and NPDES permits, wastewater treatment plant improvements as outlined in this facility plant should be constructed as soon as possible. Phasing is not a viable alternative for treatment improvements considering the extent and interrelatedness of required treatment improvements.

Until the wastewater treatment plant improvements are completed and the city is able to meet NPDES requirements, the significant expansion of sewer collection service is restricted under the MAO. It is anticipated that modest expansion of the collection as discussed in Chapter 6.1 under New Sewer Extensions will follow in a phased manner during the study period.

The following is a tentative schedule identifying the key activities and approximate implementation dates for the improvements.

Project Schedule

- Facilities Plan Completed for WWTP — October 2013
- Facilities Plan Approved by City Council — November 2013
- DEQ Approval of Final Facilities Plan — January 2014
- Secure Funding — September 2014
- Value Engineering — October 2014
- Start WWTP Improvements
  - Site Surveys & Locates — October 2014
  - Predesign Report — December 2014
  - Geotechnical Report — May 2014
  - DEQ Approval of Predesign Report and Environmental Assessment — February 2015

- Environmental Assessment — May 2015
- Design of WWTP project: August 2015
  - 30% DEQ Plan Review — January 2015
  - 50% DEQ Plan Review — March 2015
  - 70% DEQ Plan and Tech. Specification Review — May 2015
  - 100% Full Completion — August 2015

- DEQ Approval of Plans — October 2015
- Advertise for Bids — November 2015
- WWTP Improvement Construction — January 2016 – April 2017
- Facility Commissioning — May 2017
- Performance Evaluation — June 2017-June 2018

Financing

Section

8

Section 8

Financing

Most communities are unable to finance major infrastructure improvements without some form of governmental funding assistance such as low-interest loans or grants. In this Section, a number of major Federal and State funding programs and local funding mechanisms that are appropriate for the recommended improvements are discussed. A recommended financing strategy for the proposed infrastructure system improvements is also presented, along with a discussion of the potential impact to rate payers.

8.1 Grant and Loan Programs

Some level of outside funding assistance in the form of grants or low-interest loans may be necessary to make the proposed improvement projects affordable for the city of Sutherlin and its citizens. The amount and types of outside funding will dictate the amount of local funding that the City must secure. In evaluating grant and local programs, the major objective is to select a program, or a combination of programs, which are most applicable and available for the intended project.

A brief description of the major Federal and State funding programs that are typically utilized to assist qualifying communities in the financing of infrastructure improvement programs is given below. Each of the government assistance programs has particular prerequisites and requirements. These assistance programs promote such goals as aiding economic development, benefiting areas of low to moderate-income families, and providing for specific community improvement projects. With each program having its specific requirements, not all communities or projects may qualify for each of these programs.

Economic Development Administration (EDA) Public Works Grant Program

The EDA Public Works Grant Program, administered by the U.S. Department of Commerce, is aimed at projects which directly create permanent jobs or remove impediments to job creation in the project area. Thus, to be eligible for this grant, a community must be able to demonstrate the potential to create jobs from the project. Potential job creation is assessed with a survey of businesses to demonstrate the prospective number of jobs that might be created if the proposed project were completed.

Proposed projects must be located within an EDA-designated Economic Development District. Priority consideration is given to projects that improve opportunities for the establishment or expansion of industry and that create or retain private sector jobs in both the near-term and the long-term. Communities that can demonstrate that the existing system is at capacity (i.e. moratorium on new connections) have a greater chance of being awarded this type of grant. EDA grants are usually in the range of 50 to 80 percent of the project cost; therefore some type of local funding is also required. Grants typically do not exceed one million dollars.

Water and Waste Disposal Loans and Grants (Rural Development)

The Rural Development Administration (Rural Development) manages the loans and grants for wastewater programs that used to be overseen by the Farmers Home Administration. The Rural Utilities

Service (RUS) is one of three entities that comprise the USDA's Rural Development mission area. The RUS supports various programs that provide financial and technical assistance for development and operation of safe and affordable water supply systems and sewer and other forms of waste disposal facilities.

Rural Development has the authority to make loans to public bodies and non-profit corporations to construct or improve essential community facilities. Grants are also available to applicants who meet the median household income (MHI) requirements. Eligible applicants must have a population of less than 10,000. Priority is given to public entities in areas smaller than 5,500 people to restore a deteriorating water supply, or to improve, enlarge, or modify a water facility and/or inadequate waste facility. Preference is given to requests that involve the merging of small facilities and those serving low-income communities.

In addition, borrowers must meet the following stipulations:

- Be unable to obtain needed funds from other sources at reasonable rates and terms,
- Have legal capacity to borrow and repay loans, to pledge security for loans, and to operate and maintain the facilities or services,
- Be financially sound and able to manage the facility effectively,
- Have financially sound facility based on taxes, assessments, revenues, fees, or other satisfactory sources of income to pay all facility costs including operation and maintenance, and ability to retire the indebtedness and to maintain a reserve,
- Water and waste disposal systems must be consistent with any development plans of state, multi-jurisdictional area, county, or municipality in which the proposed project is located. All facilities must comply with federal, state, and local laws including those concerned with zoning regulations, health and sanitation standards, and the control of water pollution.

Loan and grant funds may be used for the following types of improvements:

- Construct, repair, improve, expand, or otherwise modify waste collection, pumping, treatment, or other disposal facilities. Facilities to be financed may include such items as sewer lines, treatment plants (including stabilization ponds), storm sewer facilities, sanitary landfills, incinerators, and necessary equipment.
- Legal and engineering costs connected with the development of facilities,
- Other costs related to the development of the facility, including the acquisition of rights-of-way and easements, and the relocation of roads and utilities,
- Finance facilities in conjunction with funds from other agencies or those provided by the applicant.

Interim commercial financing will normally be used during construction and Rural Development funds will be available when the project is completed. If interim financing is not available or if the project cost is less than $50,000, multiple advances of Rural Development funds may be made as construction progresses.

The maximum term on all loans is 40 years. However, no repayment period will exceed any statutory limitation on the organization's borrowing authority, nor the useful life of the improvement of the facility to be financed. Interest rates are set quarterly and are based on current market yields for municipal obligations. Current interest rates may be obtained from any Rural Development office.

The following rates currently apply for the Rural Development program:

Market rate. Those applicants whose median household income (MHI) in the service area is more than the $41,230 (Oregon non-metropolitan MHI) pay the market rate. The market rate is currently 3.375%.

Intermediate rate. The intermediate rate is paid by those applicants whose MHI of the service area is less than $41,230 but greater than $32,984. The intermediate rate is currently 2.75%.

Poverty line rate. Those applicants whose MHI of the service area is below $32,984 (80% of the non-metropolitan MHI) pay the lowest rate. Improvements must also be to correct a regulatory violation or health risk issue to qualify for this lowest rate. The current poverty line rate is 2.0%.

Maximum grant amounts, based on MHI, are provided in Table 8.1.1. The grants are calculated on the basis of eligible costs that do not include the costs attributable to reserve capacity or interim financing. In addition, grant funds cannot be used to reduce total user costs below that of comparable communities funded by RUS. Currently, RUS will not provide grant funding if the total of operation, maintenance costs, and debt servicing for a community is under the threshold of $43 per EDU per month.

TABLE 8.1.1
MAXIMUM RURAL DEVELOPMENT GRANT FUNDS
BASED ON MEDIAN HOUSEHOLD INCOME

Median Household Income (MHI)	Maximum Grant [(a)]	Interest Rate [(b)]
<$32,984	75%	2.0%
$32,984 - $41,230	45%	2.627%
>$41,230	0%	3.375%

[(a)] MHI<32,984 may be considered for a grant up to 75% of eligible project cost if the project is needed to alleviate a health or sanitary problem.
[(b)] Rates are current as of this quarter.

The 2006-2010 American Community Survey 5-Year lists Sutherlin as having an MHI of $39,171. At this MHI, the City may not be eligible for a grant. The City may be eligible for a RUS loan at a loan rate of 2.625%. However, this rate is dependent on the existence of a documented regulatory violation or health risk.

There are other restrictions and requirements associated with these loans and grants. If the City becomes eligible for grant assistance, the grant will apply only to eligible project costs. Additionally, grant funds are only available after the City has incurred long-term debt resulting in an annual debt service obligation equal to 0.5% of the MHI. In addition, an annual funding allocation limits the Rural Development funds. To receive a Rural Development loan, the City must secure bonding authority, usually in the form of general obligation or revenue bonds.

Applications for financial assistance are made at area offices of Rural Development. For additional information on Rural Development loans and grant programs call 1-541-673-0136 or visit the RUS website at http://www.rurdev.usda.gov/UWEP_HomePage.hmtl. The Oregon Rural Development website is http://www.rurdev.usda.gov/OR_Home.html.

Technical Assistance and Training Grants (TAT)

Available through the USDA Rural Utilities Service (RUS) as part of Water and Waste Disposal programs, TAT grants are intended to provide technical assistance and training to associations on a wide range of issues relating to the delivery of water and waste disposal services.

Rural communities with populations of less than 10,000 persons are eligible, along with private, nonprofit organizations that have been granted tax-exempt status by the IRS.

TAT funds may be used for the following activities:

- Identify and evaluate solutions to water and/or waste-related problems of associations in rural areas.
- Assist entities with preparation of applications for Water and Waste Disposal loans and grants.
- Provide training to association personnel in order to improve the management, operation and maintenance of water and/or waste disposal facilities.
- Pay expenses related to providing the technical assistance and/or training.

Grants may be made for up to 100% of the eligible project costs. Applications are filed with any USDA Rural Development office. For additional information on Rural Development loans and grant programs, call 1-541-673-0136 or visit the RUS website at http://www.rurdev.usda.gov/UWP-wwtat.htm.

(Oregon) Community Development Block Grant (CDBG) Program

The Community Development Block Grant Program section of the Infrastructure Finance Authority (IFA) administers the CDBG Program. Grants and technical assistance are available to develop livable urban communities for persons of low and moderate incomes by expanding economic opportunities and providing housing and suitable living environments.

Non-metropolitan cities and counties in rural Oregon can apply for and receive grants. Oregon tribes, urban cities (Ashland, Bend, Corvallis, Eugene, Gresham, Hillsboro, Medford, Portland, Salem and Springfield) and counties (Clackamas, Multnomah, Washington) receive funds directly from HUD.

All projects must meet one of three national objectives:

1. The proposed activities must benefit low- and moderate-income individuals.
2. The activities must aid in the prevention or elimination of slums or blight.
3. There must be an urgent need that poses a serious and immediate threat to the health or welfare of the community.

Funding amounts are based on:

- The applicant's need;
- the availability of funds; and
- other restrictions defined in the program's guidelines.

The following are the maximum grants possible for any individual project, by category:

- Economic Development: $750,000
- Microenterprise: $100,000
- Public Works
  - Water and Wastewater Improvements: $2,000,000 except preliminary/engineering planning grants: $150,000
  - Downtown Revitalization: $400,000
  - Off-site Infrastructure: $225,000
- Community/Public Facilities: $1,500,000
- Community Capacity/Technical Assistance: no specific per-award-limit but limited overall funds
- Emergency Grants: $500,000
- Regional Housing Rehabilitation: $400,000
- Emergency Projects: $500,000

For additional information on the CDBG programs, call 1-866-467-3466 or visit the IFA website at http://www.orinfrastructure.org/Learn-About-Infrastructure-Programs/Interested-in-a-Community-Development-Project/Community-Development-Block-Grant/.

Oregon Special Public Works Fund

The Special Public Works Fund (SPWF) provides funds for publically-owned facilities that support economic and community development in Oregon. Special Public Works Funds provide funding for construction and/or improvement of infrastructure needed to support industrial, manufacturing and certain types of commercial development. Funds are available to public entities for:

- Planning,
- designing,
- purchasing,
- improving and constructing publically owned facilities,
- replacing publically owned essential community facilities, and
- emergency projects as a result of a disaster.

Public agencies that are eligible to apply for funding are:

- Cities,
- counties,
- county service districts (organized under ORS Chapter 451),
- tribal councils,
- ports,
- districts as defined in ORS 198.010, and
- airport districts (ORS 838).

Facilities and infrastructure projects that are eligible for funding are:

- Airport facilities,

- buildings and associated equipment,
- restoration of environmental conditions on publically owned industrial lands,
- port facilities, wharves and docks,
- the purchase of land, rights of way and easements necessary for a public facility,
- telecommunications facilities,
- railroads,
- roadways and bridges,
- solid waste disposal sites,
- storm drainage systems,
- wastewater systems, and
- water systems.

Loans
Loans for development (construction) projects range from less than $100,000 to $10 million. The Infrastructure Finance Authority offers very attractive interest rates that reflect tax-exempt market rates for highly qualified borrowers. Initial loan terms can be up to 25 years or the useful life of the project, whichever is less.

Grants
Grants are available for construction projects that create or retain traded-sector jobs. They are limited to $500,000 or 85 percent of the project cost, whichever is less, and are based on up to $5,000 per eligible job created or retained.

Limited grants are available to plan industrial site development for publically owned sites and for feasibility studies.

For additional information on IFA programs, call 1- 503-986-0123 or visit the IFA website at http://www.orinfrastructure.org/Learn-About-Infrastructure-Programs/Interested-in-a-Community-Development-Project/Special-Public-Works-Fund/.

Water/Wastewater Financing Program

Water/Wastewater Financing is available for construction and/or improvement of water and wastewater systems to meet state and federal standards. This loan program funds the design and construction of public infrastructure needed to ensure compliance with the Clean Water Act.

The public entities that are eligible to apply for the program are:

- Cities,
- counties,
- county service districts (organized under ORS Chapter 451),
- tribal councils,
- ports, and
- special districts as defined in ORS 198.010.

The proposed project must be owned and operated by a public entity as listed above. Allowable funded project activities may include:

- Reasonable costs for construction improvement or expansion of drinking water system, wastewater system or stormwater system,
- water source, treatment, storage and distribution,

- wastewater collection, treatment and disposal facilities,
- stormwater system,
- purchase of rights of way and easements necessary for construction,
- design and construction engineering, or
- planning/technical assistance for small communities.

To be eligible for funding:

- A system must have received, or is likely to soon receive, a Notice of Non-Compliance by the appropriate regulatory agency or is for a facility plan or study required by a regulatory agency; and

- A registered Professional Engineer will be responsible for the design and construction of the project.

Funding and Uses
Loan and grant amounts are determined by a financial analysis of the applicant's ability to afford a loan (debt capacity, repayment sources and other factors).

Loans
Program guidelines, project administration, loan terms and interest rates are similar to the Special Public Works Fund program. The maximum loan term is 25 years or the useful life of the infrastructure financed, whichever is less. The maximum loan amount is $10 million per project through a combination of direct and/or bond-funded loans. Recently IFA is offering a lower, reduced interest rates for municipalities whose household income is less than the statewide median income. The current terms of this loan are for 25 years at 1.94 percent interest.

Loans are generally repaid with utility revenues or voter-approved bond issues. A limited tax general obligation pledge also may be required. "Credit worthy" borrowers may be funded through the sale of state revenue bonds.

Grants
Grant awards up to $750,000 may be awarded based on a financial review.

An applicant is not eligible for grant funds if the applicant's annual median household income is equal or greater than 100 percent of the state average median household income for the same year.

Funding for Technical Assistance
The Infrastructure Finance Authority offers technical assistance with financing for municipalities with populations of less than 15,000. The funds may be used to finance preliminary planning, engineering studies and economic investigations.

Technical assistance projects must be in preparation for a construction project that is eligible and meets the established criteria.

- Grants up to $20,000 may be awarded per project.
- Loans up to $50,000 may be awarded per project.

Interested applicants should contact the Oregon Business Development Department (OBDD) prior to submitting an application. Applications are accepted year-round. For additional information on this IFA program, call 1-503-986-0123 or visit the IFA website at http://www.orinfrastructure.org/Learn-About-

Infrastructure-Programs/Interested-in-a-Water-or-Wastewater-Improvement-Project/water-wastewater-financing/.

Department of Environmental Quality, Clean Water State Revolving Fund (CWSRF)

The Clean Water State Revolving Fund loan program is administered by DEQ and provides low-cost loans for the planning, design or construction of various projects that prevent or mitigate water pollution. The Oregon Department of Environmental Quality administers the program. Eligible agencies include Indian tribal governments, cities, counties, sanitary districts, soil and water conservation districts, irrigation districts, various special districts and certain intergovernmental entities.

Six different types of loans are available within the program, including loans for planning, design, construction, emergencies, urgent repairs and local community projects. A portion of the fund is reserved for small communities, planning or green projects.

Interest rates for the loan program change quarterly based on a percentage of the national municipal bond rate. Those percentages vary from 25 percent to 65 percent of the bond rate. For example, with a quarterly bond rate of 4.1 percent, CWSRF interest rates range from 1.0 percent to 2.7 percent depending on the length of the loan repayment period.

Current CWSRF interest rates range from just over 1% to 2.63%, depending on the term of the loan. Different interest rates and other financial requirements apply to different loans. Rates are adjusted quarterly, based on the average Bond Buyer Rates of the previous quarter as published by the Federal Reserve. When a loan is signed, the interest rate is fixed for the life of the loan. Below are 2013 first quarter interest rates for Small Communities with less than statewide MMHI.

TABLE 8.1.2
CWSRF INTEREST RATES

Loan Type	Repayment Term	Annual Interest Rate
Planning	5 years	0.94%
Design/Construction	5 years	0.94%
Design/Construction	10 years	2.06%
Design/Construction	15 years	2.25%
Design/Construction	30 years	2.39%

All eligible proposed projects are ranked based on their application information. Points are assigned based on specific ranking criteria, which include: 1) the anticipated benefit for water quality or public health; 2) potential water quality or public health consequences of not funding the project; and 3) other considerations such as education and outreach. The DEQ website lists detailed ranking criteria.

The Intended Use Plan is one part of Oregon's annual SRF capitalization grant application. This plan includes lists of eligible projects ranked in priority order. Projects which are allocated funds are placed on the Funded List. Unfunded projects are placed on the Planning List to receive funds if any of the Funded List projects do not complete the loan process. Projects identified on the Funded List from prior years which have not been initiated are placed on a Supplemental List.

Obtaining SRF funding requires the submission of an application, preparation of an environmental report on the project, a land use compatibility statement from a county planning official, and a copy of the user charge system. There are additional requirements depending on the nature of the project. An applicant needs to contact a DEQ Project Officer to discuss the project and find out what is required.

For additional information on this and other DEQ programs, call 1-800-452-4011 or visit the DEQ website at http://www.deq.state.or.us/wq/loans/loans.htm.

Oregon Department of Energy, Small Scale Energy Loan Program (SELP)

The SELP program offers loans to projects whose purpose is to promote energy conservation and renewable energy resource development. Eligible applicants include cities, counties, special districts, individuals, and non-profit groups. Loans will cover up to 100% of construction costs, including engineering, fees, and studies. The finished project must at least break even in power costs.

The program offers low-interest loans for projects that:

- Conserve natural gas, electricity, oil, or other source of energy,
- produce energy from renewable resources such as water, wind, geothermal, solar, biomass, waste materials or waste heat,
- use recycled materials to create products.

Interested parties should contact the Oregon Office of Energy for details. For additional information on the Office of Energy programs, call 1-503-378-4040 or visit the Office of Energy website at http://oregon.gov/ENERGY/.

8.2 Local Funding Sources

The amount and type of local funding obligations for infrastructure improvements will depend, in part, on the amount of grant funding anticipated and the requirements of potential loan funding. Local revenue sources for capital expenditures include *ad valorem* taxes, various types of bonds, wastewater service charges, connection fees, and system development charges. Local revenue sources for operating costs include *ad valorem* taxes and wastewater service charges. The following sections identify those local funding sources and financing mechanisms that are most common and appropriate for the improvements identified in this study.

General Obligation Bonds

A general obligation (G.O.) bond is backed by the full faith and credit of the issuer. For payment of the principal and interest on the bond, the issuer may levy *ad valorem* general property taxes. Such taxes are not needed if revenue from assessments, user charges or other sources are sufficient to cover debt service. Oregon Revised Statutes limit the maximum term to 40 years for cities. Except in the event that Rural Development Administration will purchase the bonds, the realistic term for which general obligation bonds should be issued is 15 to 20 years. Under the present economic climate, the lower interest rates will be associated with the shorter terms.

Financing of wastewater system improvements by general obligation bonds is usually accomplished by the following procedure:

- Determination of the capital costs required for the improvement,
- An election authorizing the sale of general obligation bonds,
- Following voter approval, the bonds are offered for sale,
- The revenue from the bond sale is used to pay the capital costs associated with the projects.

From a fundraising viewpoint, general obligation bonds are preferable to revenue bonds in matters of simplicity and cost of issuance. Since the bonds are secured by the power to tax, these bonds usually command a lower interest rate than other types of bonds. General obligation bonds lend themselves readily to competitive public sale at a reasonable interest rate because of their high degree of security, their tax-exempt status, and their general acceptance.

These bonds can be revenue-supported, wherein a portion of the user fee is pledged toward payment of the debt service. Using this method, the need to collect additional property taxes to retire the obligated bonds is eliminated. Such revenue-supported general obligation bonds have most of the advantages of revenue bonds, but also maintain the lower interest rate and ready marketability of general obligation bonds. Because the users of the water system pay their share of the debt load based on their water usage rates, the share of that debt is distributed in a fare and equitable manner.
Advantages of general obligation bonds over other types of bonds include:

- The laws authorizing general obligation bonds are less restrictive than those governing other types of bonds.
- By the levying of taxes, the debt is repaid by all property benefited and not just the system users.
- Taxes paid in the retirement of these bonds are IRS-deductible.
- General obligation bonds offer flexibility to retire the bonds by tax levy and/or user charge revenue.

The disadvantage of general obligation bond debt is that it is often added to the debt ratios of the underlying municipality, thereby restricting the flexibility of the municipality to issue debt for other purposes. Furthermore, general obligation bonds are normally associated with the financing of facilities that benefit an entire community and must be approved by a majority vote, and often necessitate extensive public information programs. A majority vote often requires waiting for a general election in order to obtain an adequate voter turnout. Waiting for a general election may take years, and too often a project needs to be undertaken in a much shorter amount of time.

Revenue Bonds

Revenue bonds offer some advantages over general obligation bonds and are becoming a more frequently used option. Revenue bonds are payable solely from charges made for the services provided. These bonds cannot be paid from tax levies or special assessments; their only security is the borrower's promise to operate the system in a way that will provide sufficient net revenue to meet the debt service and other obligations of the bond issue.

Many communities prefer revenue bonding, as opposed to general obligation bonding, because it insures that no tax will be levied. In addition, debt obligation will be limited to system users since repayment is derived from user fees. Another advantage of revenue bonds is that they do not count against a municipality's direct debt, but instead are considered "overlapping debt." This feature can be a crucial advantage for a municipality near its debt limit or for the rating agencies, which consider very closely the amount of direct debt when assigning credit ratings. Revenue bonds also may be used in financing projects extending beyond normal municipal boundaries. These bonds may be supported by a pledge of revenues received in any legitimate and ongoing area of operation, within or outside the geographical boundaries of the issuer.

Successful issuance of revenue bonds depends on the bond market evaluation of the revenue pledged. Revenue bonds are most commonly retired with revenue from user fees. Recent legislation has eliminated the requirement that the revenues pledged to bond payment have a direct relationship to the services financed by revenue bonds. Revenue bonds may be paid with all or any portion of revenues derived by a public body or any other legally available monies. In addition, if additional security to finance revenue bonds is needed, a public body may mortgage grant security and interests in facilities, projects, utilities or systems owned or operated by a public body.

Normally, there are no legal limitations on the amount of revenue bonds to be issued, but excessive issue amounts are generally unattractive to bond buyers because they represent high investment risks. In rating revenue bonds, buyers consider the economic justification for the project, reputation of the borrower, methods and effectiveness for billing and collecting, rate structures, provision for rate increases as needed to meet debt service requirements, track record in obtaining rate increases historically, adequacy of reserve funds provided in the bond documents, supporting covenants to protect projected revenues, and the degree to which forecasts of net revenues are considered sound and economical.

Municipalities may elect to issue revenue bonds for revenue-producing facilities without a vote of the electorate (ORS 288.805-288.945). In this case, certain notice and posting requirements must be met and a 60-day waiting period is mandatory. A petition signed by 5% of the municipality's registered voters may cause the issue to be referred to an election.

Capital Construction (Sinking) Fund

Sinking funds are often established by budgeting for a particular construction purpose. Budgeted amounts from each annual budget are carried in a sinking fund until sufficient revenues are available for the needed project. Such funds can also be developed with revenue derived from system development charges or serial levies.

The disadvantage of a sinking fund is that it is usually too small to undertake any significant projects. Also, setting aside money generated from user fees without a designated and specified need is not generally accepted in a municipal budgeting process.

System Development Charges

A system development charge (SDC) is essentially a fee collected as each piece of property is developed, and which is used to finance the necessary capital improvements and municipal services required by the development. Such a fee can only be used to recover the capital costs of infrastructure. Operating, maintenance, and replacement costs cannot be financed through system development charges.

Two types of charges are permitted under the Oregon Systems Development Charges Act, improvement fees and reimbursement fees. SDCs charged before construction are considered improvement fees and are used to finance capital improvements to be constructed. After construction, SDCs are considered reimbursement fees and are collected to recapture the costs associated with capital improvements already constructed or under construction. A reimbursement fee represents a charge for utilizing excess capacity in an existing facility paid for by others. The revenue generated by this fee is typically used to pay back existing loans for improvements.

Under the Oregon SDC Act, methodologies for deriving improvement and reimbursement fees must be documented and available for review by the public. A capital improvement plan must also be prepared which lists the capital improvements that may be funded with improvement fee revenues, and the estimated cost and timing of each improvement. Thus, revenue from the collection of SDCs can only be used to finance specific items listed in a capital improvement plan. In addition, SDCs cannot be assessed on portions of the project paid for with grant funding.

Ad Valorem Taxes

Ad valorem property taxes are often used as a revenue source for utility improvements. Property taxes may be levied on real estate, personal property or both. Historically, *ad valorem* taxes were the traditional means of obtaining revenue to support all local governmental functions.

A marked advantage of these taxes is the simplicity of the system; it requires no monitoring program for developing charges, additional accounting and billing work is minimal, and default on payments is rare. In addition, *ad valorem* taxation provides a means of financing that reaches all property owners that benefit from a water system, whether a property is developed or not. The construction costs for the project are shared proportionally among all property owners based on the assessed value of each property.

Ad valorem taxation, however, is less likely to result in individual users paying their proportionate share of the costs as compared to their benefits. Public hearings an election with voter approval would be required to implement *ad valorem* taxation.

User Fees

User fees can be used to retire general obligation bonds, and are commonly the sole source of revenue to retire revenue bonds and to finance operation and maintenance. User fees represent monthly charges of all residences, businesses, and other users that are connected to the wastewater system. These fees are established by resolution and may be modified, as needed, to account for increased or decreased operating and maintenance costs. User fees may be based on a metered volume of water consumption and/or on the type of user (e.g. residential, commercial, schools etc.).

Assessments

Under special circumstances, the beneficiary of a public works improvement may be assessed for the cost of a project. For example, the City may provide some improvements or services that directly benefit a particular development. The City may choose to assess the industrial or commercial developer to provide up-front capital to pay for the administered improvements.

8.3 Financing Strategy

A financing strategy or plan must provide a mechanism to generate capital funds in sufficient amounts to pay for the proposed improvements over the relatively short duration in design and construction, generally two years. The financing strategy must also identify the manner in which annual revenue will be generated to cover the expense for long-term debt repayment and the on-going operation and maintenance of the system.

The objectives of a financial strategy include the following:

- Identify the capital improvement cost for the project and the estimated expense for operation and maintenance.
- Evaluate the potential funding sources and select the most viable program.
- Determine the availability of outside funding sources and identify the local cost share.
- Determine the cost to system users to finance the local share and the annual cost for operation and maintenance.

Project Expenses

A total of $1.08 million for the Everett Avenue Pump Station and $19.7 million for the WWTP capital improvement project costs were identified in Section 7. The identified projects expand, replace, or repair existing equipment and facilities and are expected to slightly increase the annual operations and maintenance costs to the City.

Funding Sources

With any of the proposed funding sources within the financial strategy, the City is advised to confirm specific funding amount s with the appropriate funding agencies prior to making local financing arrangements.

Most of the grant programs require that the project address a DEQ-issued violation, potential violation or order before the project is eligible for funding.

It is recommended that the City undertake efforts to secure funding in the form of grants and loans. Rural Development looks closely at sewer user rates and expects local rates to be at or above that of similar communities before the project becomes eligible for grants. Sewer collections for the 2013 budget year were $33.50 per single family residentual unit and 3,210 EDUs. Table 8.3.1 summarizes the city's 2013 wastewater rate structure.

TABLE 8.3.1
SUTHERLIN WASTEWATER RATE STRUCTURE

Rate Description	No. of Units/ Customers	Current Rate	Current EDUs	Notes
Residential Units	2,240	$ 33.50	2,240	Single residential / mobile home park space
Out of City	3	$ 67.00	6	Per Service
Motels/Offices	1	$ 2.77	7	Per 1,000 gallons of consumption
RV Dump Station	1	$ 86.00	3	Per Service
Schools in Session	5	$ 1.18	39	Per student and staff
Summer School	5	$ 41.50	2	Per School for 3 months
Churches/Grange	22	$ 31.50	20	Per Customer
Industrial	4	$ 29.25	9	Plus $8.75 for each add. fixture,
Laundry/Carwash	4	$ 3.11	9	Per 1,000 gallons of consumption
Commercial Multi/EDU	67	$ 37.25	67	Plus $8.75 for each add. fixture
Commercial EDU's (1 or 2)	118		132	Commercial 1 - $37, Commercial 2 - $54.75
Commercial Multi EDU/Single	1		13	Plus $8.75 for each add. fixture
GV Corp 10 EDUs	1	$397.00	12	Per Service
Union Gap SE per Res	1	$ 38.50	89	Per residential connection
RV Park	2	$ 1.67	8	Per 1,000 gallons of consumption
Multi-Res Complexes	86	$ 2.77	112	Per 1,000 gallons of consumption
Residential Units	12	$ 33.50	387	Master Metered
Motels	2	$ 2.77	11	Per 1,000 gallons of consumption
RV Park	1	$ 1.67	12	Per 1,000 gallons of consumption
RV Dump Station	1	$ 86.00	3	
Churches/Grange	1	$ 31.50	1	
Industrial	2	$ 29.25	3	Plus $8.75 for each additional fixture
Multi-Res Complexes	7	$ 2.77	23	Per 1,000 gallons of consumption
Combo Rate	1		4	Misc Accounts
Grand Totals	2,974		3,210	

Note: Rates do not reflect additional surcharge for maintenance of STEP systems.

The City signed a Mutual Agreement and Order (MAO) with DEQ to provide temporary operating limits and to settle current violations. While the MAO makes the City eligible for grant funding from the standpoint of addressing current violations, the current low average cost of sewer services per EDU makes it unclear whether or not it will qualify to receive Rural Development grants. It is recommended that the City attend a one-stop meeting after regulatory approval of the final WWFP in order to assess the funding environment at that time.

Table 8.3.2 presents various potential funding scenarios. These scenarios show the various terms associated with the various potential funding sources. Table 8.3.2 provides the anticipated rate for each existing rate type for each funding scenario that was discussed at a one-stop meeting in Salem, Oregon on January 14, 2014. A new sewer rate study may be necessary in order to clarify the rate structure and to ensure that system costs are fairly allocated between each rate type.

TABLE 8.3.2

Project Description	Cost		Funding Source	
Everett Avenue Pump Station	$925,000			
WWTP Expansion	$20,317,000			
Total	**$21,242,000**		See Options Below	
	Loan Amount	**Grant**	**Years**	**Interest Rate**
Scenario #1: IFA Loan & RD				
RD	$12,000,000		40	2.625%
IFA - WW	$9,492,000	$300,000	25	3.180%
Scenario #2: DEQ & IFA				
DEQ - CWSRF	$18,242,000		30	2.390%
IFA – WW	$3,000,000	$300,000	25	3.180%
Scenario #3: DEQ & IFA				
IFA – WW	$3,000,000		25	3.180%
IFA – WW	$7,700,000	$3,000,000	25	3.180%
DEQ - CWSRF	$10,242,000		30	2.390%

User Rates

This discussion will consider the least costly potential alternative. Once the City has determined what funding is available, the current rate structure should be revised to determine the actual impact to ratepayers. All grants, loans, existing debts and reserves, and surpluses should be taken into account when calculating the final impact to ratepayers.

Table 8.3.3 below lists potential funding scenarios.

TABLE 8.3.3
WWTP EXPANSION FUNDING SOURCE USER RATES

Source of Funding	Loan Cost/EDU per Month	O & M Cost/EDU per Month	Required Min. Rate per Month
Scenario #1: IFA Loan & RD	$ 25.33	$ 33.64	$ 58.96
Scenario #2: DEQ & IFA	$ 25..45	$ 33.64	$ 59.09
Scenario #3: DEQ & IFA	$ 27.75	$ 33.64	$ 61.39

With any of the proposed funding sources within the financial strategy, the City is advised to confirm specific funding amounts with the appropriate funding agencies prior to making local financing arrangements. Sutherlin may qualify for significant grant funding. A one-stop meeting with funding agencies is recommended as soon as the City has made a firm commitment as to schedule and the extent of capital improvements. The total anticipated sewer cost per EDU is dependent on the amount of grant funding available. User rates may vary from $59 per EDU per month to $61.39 per EDU per month.

GLOSSARY

Glossary

The terms used in this study are defined below.

Average Dry Weather Flow (ADWF) - the average flow measured during a dry weather season, usually May 1 to October 31, and during low groundwater levels that occur on a daily basis. During periods of little or no precipitation, wastewater flow is composed primarily of sanitary sewage and commercial and/or industrial wastes. Base infiltration may be present.

Average Wet Weather Flow (AWWF) – the average flow measured during the wet season, usually November 1st to April 30th. This value may be utilized as a basis for higher winter mass load limits.

Base Infiltration - water that enters the sewage system from the surrounding soil during periods of low groundwater levels.

Biochemical Oxygen Demand (BOD) - a measure of wastewater strength in terms of the quantity of oxygen required for biological oxidation of the organic matter contained in wastewater. The BOD loading imposed on a treatment plant influences both the type and degree of treatment, which must be provided to produce the required effluent quality. All references to BOD in this report are with respect to five-day BOD and 20° Celsius.

DEQ - the Oregon State Department of Environmental Quality.

Discharge Monitoring Report (DMR) – the standard form required by the Oregon Department of Environmental Quality (DEQ) for the recording and reporting of influent and effluent volumes and characteristics along with other data pertaining to the wastewater system.

Excessive Infiltration and Inflow (I/I) - portion of infiltration and/or inflow which can be removed from the sewage system through rehabilitation at less cost than continuing to transport or treat that portion of I/I.

Industrial Wastes - waterborne wastes produced as the result of manufacturing or processing operations.

Infiltration - water that enters the sewage system from the surrounding soil. Common points of entry include broken pipe and defective joints in pipe and manhole walls. Although generally limited to sewers laid below the normal groundwater level, infiltration also occurs as a result of rain or irrigation water soaking into the ground and entering mains, manholes, and even shallow house sewer laterals with defective joints or other faults.

Inflow - water that enters the sewage system from surface runoff. Inflow may enter the sewer system through manhole covers, exposed broken pipes and defective pipe joints, cross

connections between storm sewers and sanitary sewers, and illegal connections of roof and area drains.

Maximum Monthly Dry Weather Flow (MMDWF) - the monthly average flow that has only twenty-percent probability of being experienced during May to October in any given year. In other words, this flow represents the wettest dry weather season monthly average flow that is anticipated to have a five-year recurrence interval. For western Oregon, May is usually the month, which has the highest dry weather flow.

Maximum Monthly Wet Weather Flow (MMWWF) - the monthly average flow that has only twenty-percent probability of being experienced during November to April in any given year. This flow represents the wettest wet season monthly average flow that is anticipated to have a five-year recurrence interval. For western Oregon, January is usually the month that has the highest wet weather flow.

"mg/l" - means milligrams per liter.

Peak Instantaneous Flow (PIF) - the highest hourly flow measured during wet weather. The addition of increased I/I during periods of high groundwater levels and rainfall may produce flows several times greater than the ADWF. This value determines the hydraulic capacity of major process units, sewers, channels, and pumps.

Rain Induced Infiltration - portion of infiltration due to leakage of percolating rainwater into collection system defects that lie near the ground surface.

Residual - the amount of chlorine in mg/l left in treated effluent at discharge.

Sanitary Sewage - waterborne wastes principally derived from the sanitary conveniences of residences, business establishments, and institutions.

Total Suspended Solids (TSS) - a measure of the quantity of suspended material contained in the wastewater. The quantity of TSS removed during treatment influences the sizing of sludge handling and disposal processes, as well as the effectiveness of disinfection with chlorine.

Wastewater - total fluid flow in a sewerage system. Wastewater may include sanitary sewage, industrial wastes, and infiltration and inflow (I/I).

REFERENCES

References

EPA, Wastewater Technology Fact Sheet, *Chlorine Disinfection*, Washington D.C., September 1999.

EPA, *Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability*, Washington D.C., EPA 430-99-74-001, July 1974.

EPA, *Design Manual Municipal Wastewater Disinfection*, Cincinnati OH, October 1986.

Everett Avenue Pump Station & Force Main Evaluation Study, The Dyer Partnership, June, 2010.

Jon Gasik, Oregon DEQ, Personal communications.

Federal Emergency Management Agency (FEMA), *The National Flood Insurance Program Community Status Book*, Available URL: http://www.fema.gov/cis/or.pdf [Accessed 20 January 2005].

Natural Resources Conservation Service, United States Department of Agriculture. Official Soil Series Descriptions [Online WWW]. Available URL: http://soils.usda.gov/soils/technical/classification/osd/index.html Accessed 20 January 2005].

Oregon Department of Geology and Mineral Industries, "Relative Earthquake Hazard Maps for Selected Urban Areas in Western Oregon," Interpretive Map Series IMS-9, 2000.

Oregon Economic & Community Development Department website, www.econ.state.or.us, April, 2002.

State of Oregon DEQ, *Quality Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon*, July 2000.

United State Census website, http://factfinder.census.gov.

APPENDICES

APPENDIX A - PERMITS



NPDES Permit

Mutual Agreement and Order (MAO)

Mutual Agreement and Order, Amendment No. 1

Whole Effluent Toxicity Test (Sept. 13, 2012)

City of Sutherlin, Excess Thermal Load Limits

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM
WASTE DISCHARGE PERMIT

Department of Environmental Quality
Western Region – Salem Office
750 Front Street NE, Suite 120, Salem, OR 97301-1039
Telephone: (503) 378-8240

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

ISSUED TO:
Sutherlin, City of
126 E. Central
Sutherlin, OR 97479

SOURCES COVERED BY THIS PERMIT:

Type of Waste	Outfall Number	Outfall Location
Treated Wastewater	001	R.M. 9.8
Reclaimed Water Reuse	002	Irrigation

FACILITY TYPE AND LOCATION:
Activated Sludge – Extended Aeration Package Plant
City of Sutherlin
4601Stearns Lane, Sutherlin, OR 97479
Treatment System Class: Level III
Collection System Class: Level II

RECEIVING STREAM INFORMATION:
Basin: Umpqua
Sub-Basin: Umpqua
Receiving Stream: Calapooya Creek
LLID: 1234686433656 - 9.8 - D
County: Douglas

EPA REFERENCE NO: OR-002084-2
Issued in response to Application No. 979880 received September 15, 2005. This permit is issued based on the land use findings in the permit record.

John W Blanchard for

John J. Rusciguo, Water Quality Manager
Western Region North

October 24, 2006
Date

PERMITTED ACTIVITIES

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

Page
Schedule A - Waste Discharge Limitations not to be Exceeded 2
Schedule B - Minimum Monitoring and Reporting Requirements 4
Schedule C - Compliance Conditions and Schedules 8
Schedule D - Special Conditions 9
Schedule F - General Conditions 13

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

SCHEDULE A

Waste Discharge Limitations not to be exceeded after permit issuance.

a. Treated Effluent Outfall 001

(1) June 1 - October 31: No discharge to waters of the State (unless approved in writing by the Department)

(2) November 1 - November 30:

- The weekly and daily mass load limitations that follow are based on stream flow in Calapooya Creek measured from the stream gauge near the Rochester Bridge in Sutherlin (See Notes 1 and 2):

Parameter	Stream Flow	Average Effluent Concentrations Monthly Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
BOD_5 and TSS	<45 cfs	30 mg/L 45 mg/L	440	170	260
BOD_5 and TSS	45 cfs-60 cfs	30 mg/L 45 mg/L	440	320	480
BOD_5 and TSS	>60 cfs-96 cfs	30 mg/L 45 mg/L	440	440	660
BOD_5 and TSS	> 96 cfs	30 mg/L 45 mg/L	440	660	880

December 1- May 31: Discharge in May allowed only while the daily flow in Calapooya Creek exceeds 82 cfs:

Parameter	Average Effluent Concentrations Monthly	Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
BOD_5	30 mg/L	45 mg/L	560	840	1100
TSS	30 mg/L	45 mg/L	560	840	1100

* Average dry weather design flow to the facility equals 1.3 MGD. Winter mass load limits based upon average wet weather design flow of the facility equaling 2.22 MGD. The daily mass load limit is suspended on any day in which the flow to the treatment facility exceeds 2.6 MGD (twice the design average dry weather flow).

(3)

Other parameters (November – May)	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 3)
pH	Shall be within the range of 6.0 – 9.0
BOD_5 and TSS Removal Efficiency	Shall not be less than 85% monthly average for BOD_5 and 85% monthly for TSS.
Ammonia-N	Shall not exceed 21 mg/L daily maximum and 7.8 mg/L monthly average.
Total Chlorine Residual	Shall not exceed 0.04 mg/L daily maximum and 0.01 mg/L monthly average (See Note 4)

(4) No wastes may be discharged or activities conducted that cause or contribute to a violation of water quality standards in OAR 340-041 applicable to the Umpqua basin except as provided for in OAR 340-045-0080 and the following regulatory mixing zone:

The allowable mixing zone is that portion of Calapooya Creek contained within a band extending out no more than 1/2 the stream width and extending from a point ten (10) feet upstream of the outfall to a point thirty (30) feet downstream from the outfall. The Zone of Immediate Dilution (ZID) shall be defined as that portion of the allowable mixing zone that is within three (3) feet of the point of discharge.

(5) Raw sewage discharges are prohibited to waters of the State except as allowed Schedule F, Section B, Condition 6 of this permit. If an overflow occurs between May 22 and June 1, and if the permittee demonstrates to the Department's satisfaction that no increase in risk to beneficial uses occurred because of the overflow, no violation will be triggered if the storm associated with the overflow was greater than the one-in-five-year, 24-hour duration storm.

b. Reclaimed Wastewater Outfall 002

(1) No discharge to state waters is permitted. All reclaimed water shall be distributed on land, for dissipation by evapotranspiration and controlled seepage by following sound irrigation practices so as to prevent:

a. Prolonged ponding of treated reclaimed water on the ground surface;

b. Surface runoff or subsurface drainage through drainage tile;

c. The creation of odors, fly and mosquito breeding or other nuisance conditions;

d. The overloading of land with nutrients, organics, or other pollutant parameters; and,

e. Impairment of existing or potential beneficial uses of groundwater.

(2) Prior to land application of the reclaimed water, it shall receive at least level II treatment as defined in OAR 340-055 to:

Reduce Total Coliform to 240 organisms per 100 mL in two consecutive samples, and a 7-day median of 23 organisms per 100 mL.

(3) Irrigation shall conform to the irrigation management plan approved by the Department.

c. 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. Stream flow shall be a weekly average of daily values in Calapooya Creek as measured at the former USGS stream gage that is located near the Rochester Bridge in Sutherlin.

2. Calapooya Creek is water quality limited for fecal coliform bacteria and dissolved oxygen during the period of discharge. This permit may be reopened to include any Waste Load Allocation (WLA), best management practice or any other condition required by the Total Maximum Daily Load (TMDL) for this water body.

3. 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.

4. When the total residual chlorine limitation is lower than 0.10 mg/L, the Department will use 0.10 mg/L as the compliance evaluation level (i.e. daily maximum concentrations below 0.10 mg/L will be considered in compliance with the limitations).

SCHEDULE B

1. **Minimum Monitoring and Reporting Requirements** (unless otherwise approved in writing by the Department).
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 grab and composite samples and measurements are taken from the influent chamber between the grit chamber and the influent screen

Item or Parameter	Minimum Frequency	Type of Sample
BOD_5	2/Week	Composite
TSS	2/Week	Composite
pH	3/Week	Grab

b. Treated Effluent Outfall 001

The facility effluent grab and composite samples and measurements are taken from the effluent chlorine contact chamber.

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD)	Daily	Measurement
Flow Meter Calibration	Annual	Verification
pH	3/Week	Grab
Temperature	3/week	Grab
BOD_5	2/Week	Composite
TSS	2/Week	Composite
E. coli	2/Week	Grab (See Note 1)
Ammonia (NH3-N)	Once every 2 Weeks	Composite
Total Chlorine Residual	Daily	Grab
Quantity Chlorine Used	Daily	Measurement
Pounds Discharged (BOD_5 and TSS)	Weekly	Calculation
Average Percent Removed (BOD_5 and TSS)	Monthly	Calculation
Dissolved Oxygen	Once every 2 Weeks	Grab
Toxics:		
Metals (Sb, As, Be, Cd, Cr, Cu, Pb, Hg, Ni, Se, Ag, Tl, and Zn), Cyanide, Total Phenolic Compounds and Hardness	Semi-annually	24-hour Composite (See Note 2 and 3)
Organic Priority Pollutants	(See Note 4)	24-hour Composite
Whole Effluent Toxicity (See Note 5)	Annually	Acute and Chronic

c. Biosolids Management

Item or Parameter	Minimum Frequency	Type of Sample
Sludge 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) Sludge 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 Digester withdrawal line (See Note 6)
Solids pumped from treatment system	Each Occurrence	Date, volume (gallons), percent solids, hauler, transfer point
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 sludges were applied recorded on site location map.
Record of % volatile solids reduction accomplished through stabilization	Monthly when land applying	Calculation (See Note 7)
Record of digestion days (mean cell residence time)	Monthly when land applying	Calculation (See Note 8)
Daily Minimum Sludge Temperature	Daily when land applying	Record

d. Reclaimed Wastewater Outfall 002

Item or Parameter	Minimum Frequency	Type of Sample
Quantity Irrigated (inches/acre)	Daily	Measurement
Flow Meter Calibration	Annually	Verification
Quantity Chlorine Used	Daily	Measurement
Total Chlorine Residual	Daily	Grab
pH	2/Week	Grab
Total Coliform	1/Week	Grab
Nutrients (TKN, NO_2+NO_3-N, NH_3, Total Phosphorus)	Quarterly	Grab

e. Receiving Stream (Calapooya Creek)

Item or Parameter	Minimum Frequency	Type of Sample
Metals (Sb, As, Be, Cd, Cr, Cu, Pb, Hg, Ni, Se, Ag, Tl, and Zn), and cyanide.	Semi-annually (See Note 9)	24-hour Composite

2. **Reporting Procedures**

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

b. State monitoring reports shall 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 shall also identify each system classification as found on page one of this permit.

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

3. **Report Submittals**

a. The permittee shall have in place a program to identify and reduce inflow and infiltration into the sewage collection system. An annual report shall be submitted to the Department by February 1 each year which details sewer collection maintenance activities that reduce inflow and infiltration. The report shall state those activities that have been done in the previous year and those activities planned for the following year.

b. For any year in which biosolids are land applied, a report shall be submitted to the Department by February 19 of the following year that describes solids 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).

c. By no later than January 15 of each year, the permittee shall submit to the Department an annual report describing the effectiveness of the reclaimed water system to comply with approved reclaimed water use plan, the rules of Division 55, and the limitations and conditions of this permit applicable to reuse of reclaimed water.

NOTES:

1. *E. coli* monitoring must be conducted according to any of the following test procedures as specified in **Standard Methods for the Examination of Water and Wastewater, 19th Edition**, or according to any test procedure that has been authorized and approved in writing by the Director or an authorized representative:

Method	Reference	Page	Method Number
mTEC agar, MF	Standard Methods, 18th Edition	9-29	9213 D
NA-MUG, MF	Standard Methods, 19th Edition	9-63	9222 G
Chromogenic Substrate, MPN	Standard Methods, 19th Edition	9-65	9223 B
Colilert QT	Idexx Laboratories, Inc.		

2. For two years after permit issuance, monitoring for metals, cyanide, total phenolic compounds and hardness must be conducted on the effluent at the specified frequency. Monitoring for arsenic must be conducted in accordance with EPA Method 1632 with a practical quantification level (PQL) of 5 ng/L or less. Monitoring for cadmium, silver and thallium must be conducted using a method with a PQL of 0.1 µg/L or lower. Monitoring for lead must be conducted using a method with a PQL of 0.2 µg/L or lower. Monitoring for mercury must be conducted in accordance with EPA Method 1631E with a method detection limit of 5 ng/L. Monitoring for all other toxic parameters must be performed using a method capable of achieving the PQLs in Appendix B, Table 8 of the Department's Internal Management Directive for conducting a Reasonable

Potential Analysis for Toxic Pollutants. Changes in monitoring methods or PQLs must be approved in writing by the Department. Effluent TSS must be monitored simultaneously. For all tests, the method detection limit and method reporting limit must be reported along with the sample result. After two years of monitoring, monitoring of the effluent may be eliminated unless otherwise notified in writing by the Department. The Department will review the data and may require additional monitoring for specific pollutants of concern or reopen the permit to incorporate permit limits.

3. For effluent cyanide samples, at least six discrete grab samples must be collected over the operating day. Each aliquot must not be less than 100 mL and must be collected and composited into a larger container which has been preserved with sodium hydroxide for cyanide samples to insure sample integrity.

4. The permittee must perform testing for organic pollutants as required in Part D of EPA Form 2A. The testing includes all pollutants included under volatile organic, acid extractable and base-neutral compounds. The monitoring needs to be conducted using EPA Methods 624 for volatile organic compounds, EPA Method 625 for semi-volatile organic compounds and Polycyclic Aromatic Hydrocarbons, and EPA Method 608 for pesticides. Three scans are required during the 4 ½ years after permit issuance. Two of the three scans must be performed no fewer than 4 months and no more than eight months apart. The effluent samples must be 24-hour daily composites, except where sampling volatile compounds. In this case, six discrete samples (not less than 40 mL) collected over the operating day are acceptable. The permittee must take special precautions in compositing the individual grab samples for the volatile organics to insure sample integrity (i.e. no exposure to the outside air). Alternately, the discrete samples collected for volatiles may be analyzed separately and averaged.

5. Beginning no later than calendar year 2007, the permittee must conduct Whole Effluent Toxicity (WET) testing for a period of four years in accordance with the frequency specified above. If the WET tests show that the effluent samples are not toxic at the dilutions determined to occur at the Zone of Immediate Dilution and the Mixing Zone, no further WET testing will be required during this permit cycle. Note that four WET test results will be required along with the next NPDES permit renewal application.

6. Composite samples from the digester withdrawal line shall consist of at least four aliquots of equal volume collected over an 8 hour period and combined.

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

7. 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 secondary clarifier solids) and a representative composite sample of solids exiting each digester withdrawal line (as defined in Note 7 above).

8. The days of digestion shall be calculated by dividing the digester volume (one-tme measurement, gallons) by the volumetric flow rate through the digester (measured on a continuous basis in gpd).

9. For the first two years after permit issuance, the permittee shall perform metals monitoring in Calapooya Creek on a semi-annual basis for the pollutants listed. After all monitoring is complete, the Department will perform a new RPA to determine if the discharge has the potential to cause or contribute to an excursion above the state water quality criteria. If there is a reasonable potential, then the Department may modify the permit as necessary to include the proper permit limits. If no reasonable potential exists, then the monitoring requirement will be discontinued for the remainder of the permit period.

SCHEDULE C

Compliance Schedules and Conditions

1. By no later than January 21, 2007, the permittee must submit to the Department a report which either identifies known sewage overflow locations and a plan for estimating the frequency, duration and quantity of sewage overflowing, or confirms that there are no overflow points. The report must also provide a schedule to eliminate the overflow(s), if any.

2. The permittee is expected to meet the compliance dates which have been established in this schedule. Either prior to or no later than fourteen days following any lapsed compliance date, the permittee shall submit to the Department a notice of compliance or noncompliance with the established schedule. The Director may revise a schedule of compliance if he/she determines good and valid cause resulting from events over which the permittee has little or no control.

SCHEDULE D

Special Conditions

1. 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 solids 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 Douglas County. All currently approved sites are located in Douglas County. No new public notice is required for the continued use of these currently approved sites. 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. For proposed new application sites that are deemed by the DEQ to be sensitive with respect to residential housing, runoff potential or threat to groundwater, an opportunity for public comment shall be provided in accordance with OAR 340-050-0030.

2. 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.

3. **Whole Effluent Toxicity Testing**

   a. The permittee must conduct whole effluent toxicity (WET) tests as specified in Schedule B of this permit.

   b. WET tests may be dual end-point tests, only for the fish tests, in which both acute and chronic end-points can be determined from the results of a single chronic test (the acute end-point must be based upon a 48-hour time period).

   c. Acute Toxicity Testing - Organisms and Protocols

      (1) The permittee must conduct 48-hour static renewal tests with the *Ceriodaphnia dubia* (water flea) and the *Pimephales promelas* (fathead minnow).

      (2) The presence of acute toxicity will be determined as specified in **Methods for Measuring the Acute Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms,** Fourth Edition, EPA/600/4-90/027F, August 1993.

      (3) An acute WET test will be considered to show toxicity if there is a statistically significant difference in survival between the control and 100 percent effluent, unless the permit specifically provides for a Zone of Immediate Dilution (ZID) for toxicity. If the permit specifies such a ZID, acute toxicity will be indicated when a statistically significant difference in survival occurs at dilutions greater than that which is found to occur at the edge of the ZID.

   d. Chronic Toxicity Testing - Organisms and Protocols

      (1) The permittee must conduct tests with: *Ceriodaphnia dubia* (water flea) for reproduction and survival test endpoint, *Pimephales promelas* (fathead minnow) for growth and survival test endpoint, and *Raphidocelis subcapitata* (green alga formerly known as *Selanastrum capricornutum*) for growth test endpoint.

(2) The presence of chronic toxicity must be estimated as specified in **Short-Term Methods for Estimating the Chronic Toxicity of Effluents and Receiving Waters to Freshwater Organisms**, Third Edition, EPA/600/4-91/002, July 1994.

(3) A chronic WET test will be considered to show toxicity if a statistically significant difference in survival, growth, or reproduction occurs at dilutions greater than that which is known to occur at the edge of the mixing zone. If there is no dilution data for the edge of the mixing zone, any chronic WET test that shows a statistically significant effect in 100 percent effluent as compared to the control will be considered to show toxicity.

e. Quality Assurance

(1) Quality assurance criteria, statistical analyses and data reporting for the WET tests must be in accordance with the EPA documents stated in this condition and the Department's **Whole Effluent Toxicity Testing Guidance Document**, January 1993.

f. Evaluation of Causes and Exceedances

(1) If toxicity is shown, as defined in sections c.(3) or d.(3) of this permit condition, another toxicity test using the same species and Department approved methodology must be conducted within two weeks, unless otherwise approved by the Department. If the second test also indicates toxicity, the permittee must follow the procedure described in section f.(2) of this permit condition.

(2) If two consecutive WET test results indicate acute and/or chronic toxicity, as defined in sections c.(3) or d.(3) of this permit condition, the permittee must evaluate the source of the toxicity and submit a plan and time schedule for demonstrating compliance with water quality standards. Upon approval by the Department, the permittee must implement the plan until compliance has been achieved. Evaluations must be completed and plans submitted to the Department within 6 months unless otherwise approved in writing by the Department.

g. Reporting

(1) Along with the test results, the permittee must include: 1) the dates of sample collection and initiation of each toxicity test; 2) the type of production; and 3) the flow rate at the time of sample collection. Effluent at the time of sampling for WET testing should include samples of required parameters stated under Schedule B, condition 1. of this permit.

(2) The permittee must make available to the Department, on request, the written standard operating procedures they, or the laboratory performing the WET tests, are using for all toxicity tests required by the Department.

h. Reopener

(1) If WET testing indicates acute and/or chronic toxicity, the Department may reopen and modify this permit to include new limitations and/or conditions as determined by the Department to be appropriate, and in accordance with procedures outlined in Oregon Administrative Rules, Chapter 340, Division 45.

4. The permittee shall meet the requirements for use of reclaimed water under Division 55, including the following:

   a. All reclaimed water shall be managed in accordance with the approved Reclaimed Water Use Plan. No substantial changes shall be made in the approved plan without written approval of the Department.

   b. No reclaimed water shall be released by the permittee to another person, as defined in Oregon Revised Statute (ORS) 468.005, for use unless there is a valid contract between the permittee and that person that meets the requirements of OAR 340-055-0015(9).

   c. The permittee shall notify the Department within 24 hours if it is determined that the treated effluent is being used in a manner not in compliance with OAR 340-055. When the Department offices are not open, the permittee shall report the incident of noncompliance to the Oregon Emergency Response System (Telephone Number 1-800-452-0311).

   d. No reclaimed water shall be made available to a person proposing to recycle unless that person certifies in writing that they have read and understand the provisions in these rules. This written certification shall be kept on file by the sewage treatment system owner and be made available to the Department for inspection.

5. Unless otherwise approved in writing by the Department, a deep-rooted, permanent grass cover shall be maintained on the land irrigation area at all times. Grass shall be periodically cut and removed to ensure maximum evapotranspiration and nutrient capture.

6. The permittee shall comply with Oregon Administrative Rules (OAR), Chapter 340, Division 49, "Regulations Pertaining To Certification of Wastewater System Operator Personnel" and accordingly:

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

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

   b. The permittee's wastewater system may not be without supervision (as required by Special Condition 6.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 in the proper classification and at grade level I or higher.

   c. 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.

   d. 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, 400 East Scenic Drive, Suite 307, The Dalles, OR 97058. This requirement is in addition to the reporting requirements contained under Schedule B of this permit.

c. 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 justification for the time needed, a schedule for recruiting and hiring, the date the system supervisor availability ceased and the name of the alternate system supervisor(s) as required by 6.b. above.

7. The permittee shall not be required to perform a hydrogeologic 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.

8. The permittee shall notify the appropriate DEQ Office 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 F
NPDES GENERAL CONDITIONS – DOMESTIC FACILITIES

SECTION A. STANDARD CONDITIONS

1. Duty to Comply with Permit
The permittee must comply with all conditions of this permit. Failure to comply with any permit condition is a violation of the Clean Water Act, Oregon Revised Statutes (ORS) 468B.025, and 40 Code of Federal Regulations (CFR) Section 122.41(a), and grounds for an enforcement action. Failure to comply is also grounds for the Department to modify, revoke, or deny renewal of a permit.

2. Penalties for Water Pollution and Permit Condition Violations
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. Additionally 40 CFR 122.41 (A) provides that any person who violates any permit condition, term, or requirement may be subject to a federal civil penalty not to exceed $25,000 per day for each violation.

Under ORS 468.943 and 40 CFR 122.41(a), 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.

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. Additionally, under 40 CFR 122.41(a) any 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 federal civil penalty not to exceed $100,000, and up to 6 years in prison.

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

4. Duty to Reapply

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

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

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

a. Violation of any term, condition, or requirement of this permit, a rule, or a statute
b. Obtaining this permit by misrepresentation or failure to disclose fully all material facts
c. A change in any condition that requires either a temporary or permanent reduction or elimination of the authorized discharge
d. The permittee is identified as a Designated Management Agency or allocated a wasteload under a Total Maximum Daily Load (TMDL)
e. New information or regulations
f. Modification of compliance schedules
g. Requirements of permit reopener conditions
h. Correction of technical mistakes made in determining permit conditions
i. Determination that the permitted activity endangers human health or the environment
j. Other causes as specified in 40 CFR 122.62, 122.64, and 124.5

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 for toxic pollutants within the time provided in the regulations that establish those standards or prohibitions, even if the permit has not yet been modified to incorporate the requirement.

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

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

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

SECTION B. OPERATION AND MAINTENANCE OF POLLUTION CONTROLS

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

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

3. Bypass of Treatment Facilities

a. Definitions
(1) "Bypass" means intentional diversion of waste streams from any portion of the treatment facility. The term "bypass" does not apply if the diversion does not cause effluent limitations to be exceeded, provided the diversion is to allow essential maintenance to assure efficient operation or the diversion is due to nonuse of nonessential treatment units or processes at the treatment facility.
(2) "Severe property damage" means substantial physical damage to property, damage to the treatment facilities or treatment processes that 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 unless:
(a) Bypass was necessary to prevent loss of life, personal injury, or severe property damage;
(b) 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
(c) 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.

Upset

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

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

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

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

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

6. Overflows from Wastewater Conveyance Systems and Associated Pump Stations

a. Definitions
(1) "Overflow" means the diversion and discharge of waste streams from any portion of the wastewater conveyance system including pump stations, through a designed overflow device or structure, other than discharges to the wastewater treatment facility.
(2) "Severe property damage" means substantial physical damage to property, damage to the conveyance system or pump station which causes them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of an overflow.
(3) "Uncontrolled overflow" means the diversion of waste streams other than through a designed overflow device or structure, for example to overflowing manholes or overflowing into residences, commercial establishments, or industries that may be connected to a conveyance system.

b. Prohibition of storm related overflows. Storm related overflows of raw sewage are prohibited to waters of the State. However, the Environmental Quality Commission (EQC) recognizes that it is impossible to design and construct a conveyance system that will prevent overflows under all storm conditions. The State of Oregon has determined that all wastewater conveyance systems should be designed to transport storm events up to a specific size to the treatment facility. Therefore, such storm related overflows will not be considered a violation of this permit if:
(1) The permittee has conveyance and treatment facilities adequate to prevent overflows except during a storm event greater than the one-in-five-year, 24-hour duration storm from November 1 through May 21 and except during a storm event greater than the one-in-ten-year, 24-hour duration storm from May 22 through October 31. However, overflows during a storm event less than the one-in-five-year, 24-hour duration storm from November 1 through May 21 are also not permit violations if, the permittee had separate sanitary and storm sewers on January 10, 1996, had experienced sanitary sewer overflows due to inflow and infiltration problems, and has submitted an acceptable plan to the Department to address these sanitary sewer overflows by January 1, 2010;
(2) The permittee has provided the highest and best practicable treatment and/or control of wastes, activities, and flows and has properly operated the conveyance and treatment facilities in compliance with General Condition B.1.;
(3) The permittee has minimized the potential environmental and public health impacts from the overflow; and
(4) The permittee has properly maintained the capacity of the conveyance system.

c. Prohibition of other overflows. All overflows other than stormwater-related overflows (discussed in Schedule F, Section B, Condition 6.b.) are prohibited unless:
(1) Overflows were unavoidable to prevent an uncontrolled overflow, loss of life, personal injury, or severe property damage;
(2) There were no feasible alternatives to the overflows, such as the use of auxiliary pumping or conveyance systems, or maximization of conveyance system storage; and
(3) The overflows are the result of an upset as defined in General Condition B.4. and meeting all requirements of this condition.

d. Uncontrolled overflows are prohibited where wastewater is likely to escape or be carried into the waters of the State by any means.

e. Reporting required. Unless otherwise specified in writing by the Department, all overflows and uncontrolled 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. Reports concerning storm related overflows must include information about the amount and intensity of the rainfall event causing the overflow.

7. Public Notification of Effluent Violation or Overflow
If effluent limitations specified in this permit are exceeded or an overflow occurs, upon request by the Department, the permittee must take such steps as are necessary to alert the public about the extent and nature of the discharge. 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. Removed Substances
Solids, sludges, filter backwash, or other pollutants removed in the course of treatment or control of wastewaters must be disposed of in such a manner as to prevent any pollutant from such materials from entering waters of the state, causing nuisance conditions, or creating a public health hazard.

SECTION C. MONITORING AND RECORDS

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

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

3. Monitoring Procedures
Monitoring must be conducted according to test procedures approved under 40 CFR part 136, unless other test procedures have been specified in this permit.

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

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

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

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

8. Retention of Records
Except for records of monitoring information required by this permit related to the permittee's sewage sludge use and disposal activities, which shall be retained for a period of at least five years (or longer as required by 40 CFR part 503). The permittee must retain records of all monitoring information, including: all calibration, 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 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 representative 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.

SECTION D. REPORTING REQUIREMENTS

1. Planned Changes
The permittee must comply with OAR chapter 340, division 52, "Review of Plans and Specifications" and 40 CFR Section 122.41(l) (1). Except where exempted under OAR chapter 340, division 52, no construction, installation, or modification involving disposal systems, treatment works, sewerage systems, or common sewers may be commenced until the plans and

specifications are submitted to and approved by the Department. The permittee must give notice to the Department as soon as possible of any planned physical alterations or additions to the permitted facility.

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

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

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

5. Twenty-Four Hour Reporting
The permittee must report any noncompliance that may endanger health or the environment. Any information must be provided orally (by telephone) within 24 hours, unless otherwise specified in this permit, from the time the permittee becomes aware of the circumstances. 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).

A written submission must also be provided within 5 days of the time the permittee becomes aware of the circumstances. Pursuant to ORS 468.959 (3) (a), if the permittee is establishing an affirmative defense of upset or bypass to any offense under ORS 468.922 to 468.946, delivered written notice must be made to the Department or other agency with regulatory jurisdiction within 4 (four) calendar days of the time the permittee becomes aware of the circumstances. The written submission 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;
d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance; and
e. Public notification steps taken, pursuant to General Condition B.7

The following must be included as information that must be reported within 24 hours under this paragraph:

f. Any unanticipated bypass that exceeds any effluent limitation in this permit;
g. Any upset that exceeds any effluent limitation in this permit;
h. Violation of maximum daily discharge limitation for any of the pollutants listed by the Department in this permit; and
i. Any noncompliance that may endanger human health or the environment.

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 this permit. The permittee must also furnish to the Department, upon request, copies of records required to be kept by this permit.

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

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

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

federal civil penalty not to exceed $10,000 per violation, or by imprisonment for not more than 6 months per violation, or by both.

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

SECTION E. DEFINITIONS

1. *BOD* means five-day biochemical oxygen demand.
2. *CBOD* means five day carbonaceous biochemical oxygen demand
3. *TSS* means total suspended solids.
4. *"Bacteria"* includes but is not limited to fecal coliform bacteria, total coliform bacteria, and E. coli bacteria.
5. *FC* means fecal coliform bacteria.
6. *Total residual chlorine* means combined chlorine forms plus free residual chlorine
7. *Technology based permit effluent limitations* means technology-based treatment requirements as defined in 40 CFR Section 125.3, and concentration and mass load effluent limitations that are based on minimum design criteria specified in OAR Chapter 340, Division 41.
8. *mg/l* means milligrams per liter.
9. *kg* means kilograms.
10. m^3/d means cubic meters per day.
11. *MGD* means million gallons per day.
12. 24-hour *Composite sample* means a sample formed by collecting and mixing discrete samples taken periodically and based on time or flow. The sample must be collected and stored in accordance with 40 CFR part 136.
13. *Grab sample* means an individual discrete sample collected over a period of time not to exceed 15 minutes.
14. *Quarter* means January through March, April through June, July through September, or October through December.
15. *Month* means calendar month.
16. *Week* means a calendar week of Sunday through Saturday.
17. *POTW* means a publicly owned treatment works.

BEFORE THE ENVIRONMENTAL QUALITY COMMISSION

OF THE STATE OF OREGON

IN THE MATTER OF:	)	MUTUAL AGREEMENT AND ORDER
CITY OF SUTHERLIN,	)	
Permittee	)	NO. WQ/M-WR-05-054 DOUGLAS COUNTY

WHEREAS:

1. On January 22, 2001, the Department of Environmental Quality (Department or DEQ) issued National Pollutant Discharge Elimination System (NPDES) Waste Discharge Permit Number 101993 (Permit) to City of Sutherlin (Permittee). The Permit authorizes the Permittee to construct, install, modify or operate wastewater treatment control and disposal facilities (facilities) and discharge adequately treated wastewaters to the Calapooya Creek, waters of the state, during winter months of the year, and to irrigate the Oak Hill Golf Course during late spring through early fall months in conformance with the requirements, limitations and conditions set forth in the Permit. The Permit expires on December 31, 2005.

2. Condition 1.a.(1) of Schedule A of the Permit prohibits discharge of treated wastewater to waters of the state from June 1 through October 31 of each year.

3. During the time period the permit has been in effect, Permittee has allowed reclaimed water from the Oak Hill Golf Course irrigation storage ponds to discharge to Cook Creek, which is a point not authorized by the permit. This release of treated effluent to waters of the state has resulted in a Class I Notice of Noncompliance being issued to the City of Sutherlin.

4. DEQ and the Permittee recognize that the Permittee does not have to sufficient land with sufficient water storage capacity to irrigate the amounts of reclaimed water necessary to keep the storage ponds from overflowing. Until new or modified facilities are constructed and put into full operation, Permittee will continue to violate the permit conditions at times.

5. Permittee presently is capable of treating its effluent so as to meet the following effluent limitations, measured as specified in the Permit:

Outfall 001 (Calapooya Creek)

(1) June 1 – October 31: Shall minimize discharge by utilizing the reclaimed water system to the maximum extent possible. When discharging, shall comply with the following interim limits:

Parameter	Average Effluent Concentrations Monthly	Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
BOD_5	30 mg/L	45 mg/L	120	220	350
TSS	30 mg/L	45 mg/L	120	220	350

* Average dry weather design flow to the facility equals 0.7 MGD.

(2) Permittee shall comply with all other permit limitations including the year-round parameters and Outfall 002 (Reclaimed Water Reuse) requirements.

6. The Department and Permittee recognize that the Environmental Quality Commission has the power to impose a civil penalty and to issue an abatement order for violations of conditions of the Permit. Therefore, pursuant to ORS 183.415(5), the Department and Permittee wish to settle those past violations referred to in Paragraph 3 and to limit and resolve the future violations referred to in Paragraph 4 in advance by this Mutual Agreement and Order (MAO).

7. This MAO is not intended to settle any violation of any interim effluent limitations set forth in Paragraph 5 above. Furthermore, this MAO is not intended to limit, in any way, the Department's right to proceed against Permittee in any forum for any past or future violations not expressly settled herein.

NOW THEREFORE, it is stipulated and agreed that:

8. The Environmental Quality Commission shall issue a final order:

A. Requiring Permittee to comply with the following schedule:

(1) By no later than May 1, 2006, Permittee shall submit an Inflow Infiltration (I/I) Study for Department review. This I/I Study must identify areas of excessive I/I in the sanitary sewer collection system and propose a plan and schedule to correct the collection system deficiencies. The I/I Study must also identify all inflow sources and propose a plan to eliminate these sources.

(2) By no later than December 1, 2006, Permittee shall submit a draft Wastewater Facilities Plan for Department review. This plan must conform to the requirements of the Clean Water Act State Revolving Fund and contain a schedule for the required improvements. Improvements to the reclaimed water system must be given highest priority and may be scheduled ahead of other improvements.

(3) By no later than 6 months after receiving comments on the draft facilities plan, Permittee shall submit a revised final plan in compliance with DEQ comments.

(4) By no later than 9 months after Department approval of the Wastewater Facilities Plan, Permittee shall complete funding arrangements for the treatment plant upgrades. Permittee shall notify the Department no later than 8 months after Department approval of the facilities plan if funding schedule can not be met.

(5) By no later than 12 months after Department approval of the facilities plan, Permittee shall complete and submit to the Department pre-design engineering report and a draft revised Reclaimed Water Use Plan for Department review.

(6) By no later than 6 months after Department approval of the pre-design engineering report, Permittee shall submit to the Department, final engineering plans and specifications for providing wastewater control facilities as needed to assure that the Permittee can continuously comply with permit conditions and all other applicable water quality standards.

(7) By no later than 14 months following approval of final plans and specifications, Permittee shall complete construction of the plan upgrades and implement the

Reclaimed Water Use Plan.

B. Requiring Permittee to meet the interim effluent limitations set forth in Paragraph 5 above until implementation of the reclaimed water plan required by paragraph 8.A.4. above.

C. Requiring Permittee, should Permittee fail to comply with the above schedule, to cease allowing new connections to Permittee's sewage collection system upon written requirement of the Department.

D. Requiring Permittee, upon receipt of a written Penalty Demand Notice from the Department, to pay the following civil penalties:

(1) $250 for each day of each violation of the compliance schedule set forth in Paragraph 8A.

(2) $100 for each violation of each daily or weekly average waste discharge limitation set forth in Paragraph 5.

(3) $500 for each violation of each monthly average waste discharge limitation set forth in Paragraph 5.

9. If any event occurs that is beyond Permittee's reasonable control and that causes or may cause a delay or deviation in performance of the requirements of this MAO, Permittee shall immediately notify the Department verbally of the cause of delay or deviation and its anticipated duration, the measures that have been or will be taken to prevent or minimize the delay or deviation, and the timetable by which Permittee proposes to carry out such measures. Permittee shall confirm in writing this information within five (5) working days of the onset of the event. It is Permittee's responsibility in the written notification to demonstrate to the Department's satisfaction that the delay or deviation has been or will be caused by circumstances beyond the control and despite due diligence of Permittee. If Permittee so demonstrates, the Department shall extend times of performance of related activities under this MAO as appropriate. Circumstances or events beyond Permittee's control include, but are not limited to, acts of nature,

unforeseen strikes, work stoppages, fires, explosion, riot, sabotage, or war. Increased cost of performance or consultant's failure to provide timely reports may not be considered circumstances beyond Permittee's control.

10. Regarding the violations set forth in Paragraphs 3 and 4 above, which are expressly settled herein without penalty, Permittee and the Department hereby waive any and all of their rights to any and all notices, hearing, judicial review, and to service of a copy of the final order herein. The Department reserves the right to enforce this order through appropriate administrative and judicial proceedings.

11. Regarding the schedule set forth in Paragraph 8A above, Permittee acknowledges that Permittee is responsible for complying with that schedule regardless of the availability of any federal or state grant monies.

12. The terms of this MAO may be amended by the mutual agreement of the Department and Permittee.

13. The Department may amend the compliance schedule and conditions in this MAO upon finding that such modification is necessary because of changed circumstances or to protect public health and the environment. The Department shall provide Permittee a minimum of thirty (30) days written notice prior to issuing an Amended Order modifying any compliance schedules or conditions. If Permittee contests the Amended Order, the applicable procedures for conduct of contested cases in such matters shall apply.

14. This MAO shall be binding on the parties and their respective successors, agents, and assigns. The undersigned representative of each party certifies that he or she is fully authorized to execute and bind such party to this MAO. No change in ownership or corporate or partnership status relating to the facility shall in any way alter Permittee's obligations under this MAO, unless otherwise approved in writing by DEQ.

15. All reports, notices and other communications required under or relating to this MAO should be directed to Paul Kennedy, DEQ Roseburg Regional Office, 725 SE Main Street,

Roseburg, Oregon 97470, phone number (541) 440-3338 x. 228. The contact person for Permittee shall be Mike Gray, 126 East Central, Sutherlin OR 97479 phone number (541) 459-3542.

16. Permittee acknowledges that it has actual notice of the contents and requirements of the MAO and that failure to fulfill any of the requirements hereof would constitute a violation of this MAO and subject Permittee to payment of civil penalties pursuant to Paragraph 8D above.

17. Any stipulated civil penalty imposed pursuant to Paragraph 8D shall be due upon written demand. Stipulated civil penalties shall be paid by check or money order made payable to the "Oregon State Treasurer" and sent to: Business Office, Department of Environmental Quality, 811 S.W. Sixth Avenue, Portland, Oregon 97204. Within 21 days of receipt of a "Demand for Payment of Stipulated Civil Penalty" Notice from the Department, Permittee may request a hearing to contest the Demand Notice. At any such hearing, the issue shall be limited to Permittee's compliance or non-compliance with this MAO. The amount of each stipulated civil penalty for each violation and/or day of violation is established in advance by this MAO and shall not be a contestable issue.

18. Providing Permittee has paid in full all stipulated civil penalties pursuant to Paragraph 17 above, this MAO shall terminate 60 days after Permittee demonstrates full compliance with the requirements of the schedule set forth in Paragraph 8A above.

PERMITTEE

OCT 24 2006
Date

Arthur J. Schmidt
City Manager, City of Sutherlin

DEPARTMENT OF ENVIRONMENTAL QUALITY

OCT 24 2006

Date

Kerri L. Nelson, Western Region Administrator

FINAL ORDER

IT IS SO ORDERED:

ENVIRONMENTAL QUALITY COMMISSION

OCT 24 2006

Date

Kerri L. Nelson, Western Region Administrator
Department of Environmental Quality
Pursuant to OAR 340-11-136(1)

Oregon
John A. Kitzhaber, MD, Governor

Department of Environmental Quality
Headquarters
811 SW Sixth Avenue
Portland, OR 97204-1390
(503) 229-5696
FAX (503) 229-6124
TTY: 711

April 17, 2012

City of Sutherlin
c/o Brian Elliott, Wastewater Division Supervisor
126 E. Central Avenue
Sutherlin, OR 97479

Re: Amendment
Mutual Agreement and Order
No. WQ/M-WR-05-054

Dear Mr. Elliott:

Please find enclosed a fully executed Amendment to the referenced Mutual Agreement and Order. If you have any questions, please contact me at 503-229-5950.

Thank you for your cooperation.

Sincerely,

Jeff Bachman
Office of Compliance and Enforcement

Enclosure
cc: Jon Gasik, Western Region, Medford Office, DEQ

BEFORE THE ENVIRONMENTAL QUALITY COMMISSION

OF THE STATE OF OREGON

IN THE MATTER OF:	)	AMENDMENT NO. 1
	)	
CITY OF SUTHERLIN, Wastewater Treatment Facility	)	MUTUAL AGREEMENT AND ORDER NO. WQ/M-WR-05-054
	)	
Permittee.	)	DOUGLAS COUNTY

WHEREAS:

1. On October 24, 2006, the Department of Environmental Quality (DEQ) and Permittee entered into Mutual Agreement and Order (MAO) No. WQ/M-WR-05-054.

2. Paragraph 12 of MAO states: "The terms of this MAO may be amended by the mutual agreement of the Department and Permittee."

3. Also on October 24, 2006, DEQ renewed permittee's National Pollutant Discharge Elimination System Permit No. 101993. The renewed permit contained new daily and monthly average effluent limits on ammonia. In the permit evaluation report, DEQ stated:

> "A file review of effluent monitoring data shows that the ammonia concentrations do not regularly exceed these limits. The permittee should be able to meet these limits with proper operation and maintenance and the Department considers these limits to be protective of the water quality standard for Ammonia-N."

For this reason, no compliance schedule for ammonia was provided in the permit. Since that time the City has properly operated and maintained the system and yet is unable to consistently meet the ammonia limits. The City has exceeded the monthly average limit of 7.8 mg/L on the following occasions:

Month	Reported Value
January 2006	10.2
February 2006	8.8
March 2006	8.6
April 2006	8.7
May 2006	13.2
June 2006	19.2

July 2006	25.5
August 2006	11.3
September 2006	17.8
October 2006	8.8
July 2007	8.2
August 2007	9.8
September 2007	8.4
February 2008	10
March 2008	11.2
July 2008	11.4
August 2008	10.9
September 2008	12.0
October 2008	8.9
April 2009	8.6
May 2009	8.6
June 2009	19.0
July 2009	24.5
August 2009	22.5
September 2009	14.5
March 2010	8.4
April 2010	11.5
June 2010	8.0
July 2010	13.0
August 2010	12.8
September 2010	8.5
January 2011	9.2
February 2011	9.9
May 2011	8.5
June 2011	9.6
July 2011	14.0
August 2011	19.0
September	26.0
October 2011	19.0
November 2011	15.5
December 2011	12.8

The City has exceeded the daily maximum limit of 21 mg/L on the following occasions:

Date	Reported Value
July 10, 2006	22.9 mg/L
July 24, 2006	28.1 mg/L
August 8, 2006	22.1 mg/L
September 4, 2006	25 mg/L
June 29, 2009	23 mg/L
July 29, 2009	29 mg/L
August 10, 2009	25 mg/L
September 5, 2011	22 mg/L
September 19, 2011	30 mg/L

4. DEQ and the Permittee recognize that until new or modified facilities are constructed and put into full operation, Permittee will continue to violate the ammonia limit at times.

5. Permittee presently is capable of treating its effluent so as to meet the following effluent limitations, measured as specified in the Permit:

Pollutant	**Limitation**
Ammonia-N	Shall not exceed 30 mg/L daily maximum concentration.
Ammonia-N	Shall not exceed 27 mg/L monthly average concentration.

NOW THEREFORE, pursuant to ORS 183.415(5), the Department and Permittee wish to settle those past violations referred to in Paragraph 3 and to limit and resolve the future violations referred to in Paragraph 4 in advance by this amendment to MAO No. WQ/M-WR-05-054.

5. Paragraph 5 of MAO No. WQ/M-WR-05-054 is amended to include the interim ammonia limits in Paragraph 5 above.

6. Paragraph 8.A. is amended to add milestone 7 as follows: "By no later than fifteen (15) months after initiation of operation of the new facilities, Permittee must submit a report summarizing the performance testing conducted during the first twelve (12) months of operation and an evaluation of whether the actual performance meets expectations. If needed, the report must also include a list of recommended performance improvements."

7. Paragraph 15 is replaced with the following: "All reports, notices and other communications required under or relating to this MAO should be directed to DEQ Eugene Regional Office, 165 East 7th Avenue, Suite 100, Eugene, OR 97401, phone number (541) 687-7331. The contact for Permittee is the City Manager, 126 East Central, Sutherlin OR 97479 phone number (541) 459-3542."

7. Paragraph 18 is replaced with the following: "This MAO will terminate the day after the final task in Paragraph 7 is completed. However, Permittee remains liable for stipulated penalties

for any violations of the MAO occurring during the period the MAO was in effect and demanded pursuant to Paragraph 15."

9. The past violations listed in Paragraph 3 above and the potential future violations referred to in Paragraph 4 above are resolved without penalty.

10. All other terms and conditions of MAO No. WQ/M-WR-05-054 remain unchanged.

PERMITTEE

4/4/2012
Date

Ron Garzini, City Manager interim
City of Sutherlin, Oregon

DEPARTMENT OF ENVIRONMENTAL QUALITY and
ENVIRONMENTAL QUALITY COMMISSION

4/13/12
Date

Leah E. Koss, Manager
Office of Compliance and Enforcement
on behalf of DEQ pursuant to OAR 340-012-0170
on behalf of the EQC pursuant to OAR 340-011-0505

OREGON DEPARTMENT OF ENVIRONMENTAL QUALITY
Memorandum
WESTERN REGION - MEDFORD

State of Oregon Department of Environmental Quality

To: Sutherlin STP Permit Development File **Date:** 1/8/2009

From: Jonathan Gasik, MS, PE, Senior Engineer
Western Region Medford Office

Calapooya Creek Analysis
Reserve Capacity Allowance of Calapooya Creek TMDL TP
Sutherlin STP - File No. 86662

BACKGROUND

On October 24, 2006, the DEQ issued NPDES Waste Discharge Permit Number 101993 (Permit) to City of Sutherlin (City). The Permit authorizes the City to construct, install, modify or operate wastewater treatment control and disposal facilities (facilities) and discharge adequately treated wastewaters to the Calapooya Creek, waters of the state, during winter months of the year, and to irrigate the Oak Hill Golf Course during late spring through early fall months in conformance with the requirements, limitations and conditions set forth in the Permit. The Permit prohibits discharge of treated wastewater to waters of the state from June 1 through October 31 of each year. However, the City has allowed reclaimed water from the Oak Hill Golf Course irrigation storage ponds to discharge to Cook Creek, which is a point not authorized by the permit. The City does not have sufficient land with sufficient water storage capacity to irrigate the amounts of reclaimed water necessary to keep the storage ponds from overflowing. Until new or modified facilities are constructed and put into full operation, the City will continue to violate the permit conditions at times. To resolve this issue, the DEQ and City entered into Mutual Agreement and Order (MAO) number WQ/M-WR-05-054 which requires the City to upgrade the treatment system.

Paragraph 8.A.1. of the MAO is as follows:

> By no later than 12 months after the Calapooya Creek analysis is completed, Permittee shall submit a draft Wastewater Facilities Plan for Department review. This plan must conform to the requirements of the Clean Water Act State Revolving Fund and contain a schedule for the required improvements. Improvements to the reclaimed water system must be given highest priority and shall be scheduled ahead of other improvements.

This memo is intended to serve as the "Calapooya Creek Analysis" referred to in the MAO.

SOUTH UMPQUA TMDL

The South Umpqua Total Maximum Daily Load (TMDL) was approved by EPA on April 13, 2007 and contains Waste Load Allocation (WLAs) for total phosphorus for point sources in the Umpqua Basin. The wasteload allocation for Sutherlin is applicable June 1 though October 31 and is as follows:

> Currently, neither wastewater treatment plant (WWTP) is allowed to discharge during the summer period except under extreme rainfall events. Therefore the WWTP WLAs are zero. Point sources may, at their option, collect additional data, perform the necessary

evaluations, and request a portion of the reserve capacity. Similarly, additional analysis could determine monthly or flow based loading capacities.

While the City has adequate land at the Oak Hill Golf Course to irrigate all the recycled water from June through September, there is limited demand for recycled water during October. Therefore, the City has requested that the DEQ perform analyses on their behalf and develop flow based effluent limits for October.

DISCUSSION

In October 2006, DEQ collected additional data to support calibration of the EPA-supported Qual2k mathematical model. The calibrated model was used to predict effluent soluble phosphorus (ortho-phosphorus) concentrations that would result in no measurable decrease in instream dissolved oxygen at varying streamflow conditions for the October "shoulder" month. DEQ is confident that the following flow-based WLAs would result in no measureable decrease in dissolved oxygen in the Calapooya Creek during the month of October:

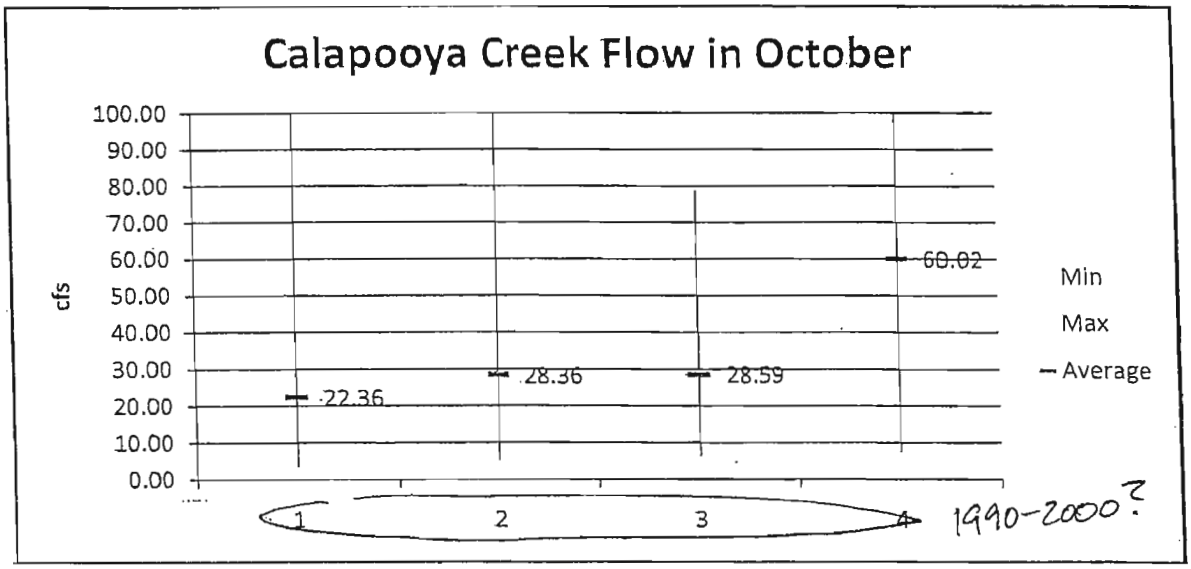
Creek Flow (Qr) *	WLA
< 10 cfs	Zero
> 10 cfs and < 250 cfs	=Qr*0.175 ppd
> 250 cfs	44 Ppd

* - Based on USGS Gage #14320700 Calapooya Creek near Oakland

For facilities that propose to meet these limits utilizing biological nutrient removal (BNR) and targeting an effluent phosphorus concentration, DEQ has implemented the WLA as a monthly median based on the median monthly stream flow. This allows for the many variables that contribute to proper functioning of BNR systems, such as consistency of influent flows, influent BOD concentrations, influent phosphorus concentrations, temperature, and antecedent conditions. However, Sutherlin proposes instead to meet these limits by reducing the effluent flow, so that the loading is reduced. Because of the potential for periodic high concentrations of phosphorus in the creek, the use of monthly median values would not be as protective of water quality.

The growth of algae is dependent upon temperature, solar radiation, and nutrient availability. Assuming that temperature and solar radiation are at optimum levels and the nutrient loading is variable, it will take several days of higher nutrient loading to develop a significant biomass. For this reason, it is acceptable to implement the WLA as a weekly average.

The following chart summarizes weekly average creek flow data from the USGS stream flow gage number 14320700 Calapooya Creek near Oakland for the years 1990 through 2000 for the month of October:



This charts shows that the weekly average stream flow typically does not drop below 10 cfs during October, except during dry years. Therefore, a flow-based effluent limit based on a weekly average would allow some discharge during most years while being protective of water quality.

Ortho-phosphorus (the soluble form and a portion of total phosphorus) was the focus of the modeling exercise to predict no measureable decrease in dissolved oxygen. For permitting purposes, total phosphorus wasteload allocations are used. The ratio of total phosphorus to ortho-phosphorus was determined in development of the South Umpqua TMDL was 1 : 0.7. Applying this ratio to the above formula, the total phosphorus WLA becomes: $WLA_{TP} = Qr*0.25$. The following table shows the amount of total phosphorus that could be discharged during an average October with no measureable decrease in dissolved oxygen. The table also shows the amount of treated wastewater that could be discharged, assuming the total phosphorus concentration is 4.0 mg/L:

Week	Low flow			Average			High Flow		
	Qr	TP ppd	Qe	Qr	TP ppd	Qe	Qr	TPeff	Qe
1.00	3.66	0.00	0.00	22.36	5.59	0.17	121.86	30.46	0.91
2.00	5.27	0.00	0.00	28.36	7.09	0.21	142.57	35.64	1.07
3.00	6.34	0.00	0.00	28.59	7.15	0.21	78.57	19.64	0.59
4.00	7.37	0.00	0.00	60.02	15.01	0.45	327.14	81.79	2.45
total			0.00			8.66			42.51

The net irrigation need in October during a dry year is 2.32 inches (19 out of 20), an average year is 0.31 inches (5 out of 10), and zero during a wet year. The Oak Hill Golf Course currently irrigates approximately 95 acres. The current design average dry weather design flow is 1.3 mgd, or about 40 million gallons for October. Based on these assumptions, the following table provides estimated storage requirements:

Condition	STP flow	Irrigation Flow	Effluent Discharged	Storage need
Dry Year	40	6.0 (18.4 ac ft)	0	34 million gallons
Average	40	0.8 (2.45 ac ft)	8.66	31.3 million gallons
Wet Year	40	0	42	zero

Therefore, under this current proposal, the City would need about 43 million gallons of storage capacity during a dry year, if the treatment plant were to operating at design capacity. However, the treatment plant is currently operating well below ADWDF (17 million gallons in October 2008). To address long term planning needs, the City may wish to explore options of expanding the irrigation system and/or other opportunities for recycled water use. The City may also wish to consider nutrient removal alternatives.

SUMMARY

When renewed, the proposed effluent limits for the City of Sutherlin's NPDES permit should be as follows:

1. Treated Effluent Outfall 001

   a. June 1 – September 30: No discharge to waters of the state

   b. October:

Parameter	Weekly Average Creek Flow (Qr) at USGS Gage Calapooya Creek near Oakland 14310000	Weekly Average Limit lb/day
Total Phosphorus	< 10	0
	> 10 and < 250	= Qr*0.25
	> 250	44

Oregon
John A Kitzhaber, M.D., Governor

Department of Environmental Quality
Western Region Salem Office
750 Front Street NE, Suite 120
Salem, OR 97301-1039
(503) 378-8240
FAX (503) 373-7944
TTY (503) 378-3684

September 13, 2012

Mr. Dan Huff, Community Development Director
City of Sutherlin
126 E. Central Ave.
Sutherlin, OR 97479

Re: ***NPDES*** Permit Application
File Number: 86662 / Permit Number: 101993
Facility: Sutherlin STP, Sutherlin, Oregon
Douglas County

Dear Mr. Huff:

On March 27, 2012, I sent you a letter stating that DEQ is willing to reschedule the permit renewal to 2014 contingent upon the City characterizing the wastewater discharge. The letter included an attachment that included details on the toxics monitoring and requested Whole Effluent Toxicity (WET) tests.

Since that time, DEQ has reconsidered the need to perform WET tests concurrently with the Tier 1 toxics monitoring. Since the City has already completed 4 WET tests, no additional WET tests are needed at this time. DEQ is reviewing the results of WET tests already preformed and will inform the City later if additional WET testing is needed.

Additionally, after reviewing the list of **Tier 1 pollutants** in the attachment previously provided with commercial laboratories, DEQ has revised the list. DEQ has also reconsidered the need for Sutherlin to perform pesticide and PCB analysis. Please see the attached revised Attachment.

Please perform this monitoring in September 2012, February 2013, September 2013, and February 2014.

Should you have any questions about the content of this letter, please feel free to contact Paul Kennedy at (541) 687-7439 or Jon Gasik at 541-776-6242.

Sincerely,

Steve Schnurbusch
Acting WQ Manager
Western Region, Salem Office

Enclosure: Revised Attachment A: Additional Wastewater Characterization

ec: Carrie Everett,
Paul Kennedy, ODEQ Eugene Office
Jonathan Gasik, MS, PE, ODEQ Medford Office

Attachment A
Additional Wastewater Characterization

The City of Sutherlin must submit a summary report (in both electronic and paper versions) and copies of the laboratory analytic reports for all pollutants below by May 1, 2014. The electronic summary report must be on a DEQ-approved template.

Signatory Requirement: The report must be signed and certified in accordance with 40 CFR § 122.22.

Monitoring Procedures: Whenever possible, the permittee must use a test method, as indicated in 40 CFR § 136.3. When no analytical method is approved under 40 CFR part 136, any suitable method may be used, but a description of the method must be provided. The permittee must meet the Quantification Limits (QL) noted in the tables below.

The permittee must ensure that all monitoring analysis reports contain both the QL and the detection level as defined below:

a. Detection Level: Same as the Method Detection Limit (MDL) derived using 40 CFR part 136 Appendix B.

b. Quantitation Limit (QL): Same as the Method Reporting Limit (MRL). It is the lowest level at which the entire analytic system must give a recognizable signal and acceptable calibration for the analyte. It is equivalent to the concentration of the lowest calibration standard, assuming that all method-specified sample weights, volumes, and cleanup procedures have been employed.

Effluent Toxics Characterization Monitoring: The permittee must take effluent samples at Outfall 001 and measurements of the pollutant parameters listed below in accordance with the following table. These must be 24-hour composite samples collected in September 2012, February 2013, September 2013, and February 2014.

Metals[1], Cyanide, Total Phenols and Hardness (ug/l unless otherwise specified)					
Pollutant	CAS[3]	QL	Pollutant	CAS	QL
Aluminum	7429905	1.0	Nickel	7440020	10
Antimony	7440360	0.10	Selenium	7782492	2.0
Arsenic (Total)	7440382	0.50	Silver	7440224	1.0
Beryllium	7440417	0.10	Thallium	7440280	0.10
Cadmium	7440439	0.10	Zinc	7440666	5.0
Chromium (Total)	7440473	0.40	Cyanide (Free)[2]	57125	10
Copper	7440508	10	Cyanide (Total)[2]	57125	5.0
Iron	7439896	100	Total Phenolic Compounds		--
Lead	7439921	5	Hardness (Total as CaCO3)		--
Mercury	7439976	0.010			

1 All metals must be analyzed for total recoverable concentration unless otherwise specified.
2 For effluent cyanide samples, at least six discrete grab samples must be collected over the operating day. Each aliquot must not be less than 100 mL and must be collected and composited into a larger container which has been preserved with sodium hydroxide to insure sample integrity.
3 Chemical Abstract Service

Attachment A
Additional Wastewater Characterization

Volatile Organic Compounds (ug/l unless otherwise specified)					
Pollutant	CAS	QL	Pollutant	CAS	QL
acrolein	107028	5.0	1,1-dichloroethylene	75354	0.50
acrylonitrile	107131	5.0	1,2-dichloropropane	78875	0.50
benzene	71432	0.50	1,3-dichloropropylene	542756	0.50
bromoform	75252	0.50	ethylbenzene	100414	0.50
carbon tetrachloride	56235	0.50	methyl bromide	74839	0.50
chlorobenzene	108907	0.50	methyl chloride	74873	0.50
chlorodibromomethane	124481	0.50	methylene chloride	75092	0.50
chloroethane	75003	0.50	1,1,2,2-tetrachloroethane	79345	0.50
2-chloroethylvinyl ether	110758	5.0	tetrachloroethylene	127184	0.50
chloroform	67663	0.50	toluene	108883	0.50
dichlorobromomethane	75274	0.50	1,1,1-trichloroethane	71556	0.50
1,1-dichloroethane	75343	0.50	1,1,2-trichloroethane	79005	0.50
1,2-dichloroethane	107062	0.50	trichloroethylene	79016	0.50
1,2-trans-dichloroethylene	156605	0.50	vinyl chloride	75014	0.50
The permittee must collect six discrete samples (not less than 40 mL) over the operating day with the results averaged.					

Acid-extractable Compounds (ug/l unless otherwise specified)					
Pollutant	CAS	QL	Pollutant	CAS	QL
p-chloro-m-cresol (Methyl-4,6-dinitrophenol 2)	59507	1.0	2-nitrophenol	88755	2.0
2-chlorophenol	95578	1.0	4-nitrophenol	100027	5.0
2,4-dichlorophenol	120832	1.0	pentachlorophenol	87865	2.0
~~2,4-dimethylphenol~~	105679	5.0	phenol	108952	1.0
2,4-dinitro-o-cresol	534521	2.0	2,4,5-trichlorophenol	95954	2.0
dinitrophenols	25550597	n/a	2,4,6-trichlorophenol	88062	1.0
2,4-dinitrophenol	51285	5.0			

Base-Neutral Compounds (ug/l unless otherwise specified)					
Pollutant	CAS	QL	Pollutant	CAS	QL
acenaphthene	83329	1.0	3,3-Dichlorobenzidine	91941	1.0
acenaphthylene	208968	1.0	diethyl phthalate	84662	1.0
anthracene	120127	1.0	dimethyl phthalate	131113	1.0
benzidine	92875	10	2,4-dinitrotoluene	121142	1.0
benzo(a)anthracene	56553	1.0	2,6-dinitrotoluene	606202	1.0
benzo(a)pyrene	50328	1.0	1,2-diphenylhydrazine	122667	5.0
3,4- Benzo(b)fluoranthene	205992	1.0	fluroranthene	206440	2.0
benzo(ghi)perylene	191242	1.0	fluorene	86737	1.0
benzo(k)fluoranthene	207089	1.0	hexachlorobenzene	118741	1.0
bis(2-chloroethoxy)methane	111911	2.0	hexachlorobutadiene	87683	2.0
bis(2-chloroethyl)ether	111444	1.0	hexachlorocyclopentadiene	77474	2.0
bis(2-chloroisopropyl)ether	108601	2.0	hexachloroethane	67721	2.0
~~Bis (Chloromethyl) ether~~	542881	na	indeno(1,2,3-cd)pyrene	193395	1.0
bis (2-ethylhexyl)phthalate	117817	1.0	isophorone	78591	10
4-bromophenyl phenyl ether	101553	1.0	napthalene	91203	1.0

Attachment A
Additional Wastewater Characterization

Base-Neutral Compounds (ug/l unless otherwise specified)					
Pollutant	CAS	QL	Pollutant	CAS	QL
butylbenzyl phthalate	85687	1.0	nitrobenzene	98953	1.0
2-chloronaphthalene	91587	1.0	N-nitrosodimethylamine	62759	1.0
4-chlorophenyl phenyl ether	7005723	1.0	N-nitrosodi-n-propylamine	621647	2.0
chrysene	218019	1.0	N-nitrosodiphenylamine	86306	1.0
di-n-butyl phthalate	84742	1.0	Pentachlorobenzene	608935	10
di-n-octyl phthalate	117817	1.0	phenanthrene	85018	1.0
dibenzo(a,h)anthracene	53703	1.0	pyrene	129000	1.0
1,2-Dichlorobenzene (o)	95501	0.50	1,2,4-trichlorobenzene	128821	5.0
1,3-Dichlorobenzene (m)	541731	0.50	Tetrachlorobenzene,1,2,4,5	95943	1.0
1,4-Dichlorobenzene (p)	106467	0.50			

~~Organochlorine Pesticides (ug/l unless otherwise specified)~~					
~~Pollutant~~	~~CAS~~	~~QL)~~	~~Pollutant~~	~~CAS~~	~~QL~~
~~Aldrin~~	~~309002~~	~~0.010~~	~~Endosulfan (total)~~	~~115297~~	~~0.010~~
~~BHC (Technical)~~	~~608731~~	~~0.010~~	~~Endrin~~	~~72208~~	~~0.010~~
~~BHC-alpha~~	~~319846~~	~~0.010~~	~~Endrin Aldehyde~~	~~7421934~~	~~0.010~~
~~BHC-beta~~	~~319857~~	~~0.010~~	~~Heptachlor~~	~~76448~~	~~0.010~~
~~BHC-delta~~	~~319868~~	~~0.010~~	~~Heptachlor Epoxide~~	~~1024573~~	~~0.010~~
~~BHC-gamma~~	~~58899~~	~~0.010~~	~~Methoxychlor~~	~~72435~~	~~0.010~~
~~Chlordane~~	~~57749~~	~~0.10~~	~~Toxaphene~~	~~8001352~~	~~0.50~~
~~DDD 4,4'~~	~~72548~~	~~0.010~~	~~PCB--Aroclor 1016~~	~~12674112~~	~~0.50~~
~~DDE 4,4'~~	~~72559~~	~~0.010~~	~~PCB--Aroclor 1221~~	~~11104282~~	~~0.50~~
~~DDT 4,4'~~	~~50293~~	~~0.010~~	~~PCB--Aroclor 1232~~	~~11141165~~	~~0.50~~
~~Dieldrin~~	~~60571~~	~~0.010~~	~~PCB--Aroclor 1242~~	~~53469219~~	~~0.50~~
~~Endosulfan alpha-~~	~~959988~~	~~0.010~~	~~PCB--Aroclor 1248~~	~~12672296~~	~~0.50~~
~~Endosulfan beta-~~	~~33213659~~	~~0.010~~	~~PCB--Aroclor 1254~~	~~11097691~~	~~0.50~~
~~Endosulfan Sulfate~~	~~1031078~~	~~0.010~~	~~PCB--Aroclor 1260~~	~~11096825~~	~~0.50~~

DEQ-DC1

Attachment A
Additional Wastewater Characterization

The City of Sutherlin must submit a summary report (in both electronic and paper versions) and copies of the laboratory analytic reports for all pollutants below by May 1, 2014. The electronic summary report must be on a DEQ-approved template.

Signatory Requirement: The report must be signed and certified in accordance with 40 CFR § 122.22.

Monitoring Procedures: Whenever possible, the permittee must use a test method, as indicated in 40 CFR § 136.3. When no analytical method is approved under 40 CFR part 136, any suitable method may be used, but a description of the method must be provided. The permittee must meet the Quantification Limits (QL) noted in the tables below.

The permittee must ensure that all monitoring analysis reports contain both the QL and the detection level as defined below:

a. Detection Level: Same as the Method Detection Limit (MDL) derived using 40 CFR part 136 Appendix B.

b. Quantitation Limit (QL): Same as the Method Reporting Limit (MRL). It is the lowest level at which the entire analytic system must give a recognizable signal and acceptable calibration for the analyte. It is equivalent to the concentration of the lowest calibration standard, assuming that all method-specified sample weights, volumes, and cleanup procedures have been employed.

Effluent Toxics Characterization Monitoring: The permittee must take effluent samples at Outfall 001 and measurements of the pollutant parameters listed below in accordance with the following table. These must be 24-hour composite samples collected in September 2012, February 2013, September 2013, and February 2014.

Metals[1], Cyanide, Total Phenols and Hardness (ug/l unless otherwise specified)					
Pollutant	CAS[3]	QL	Pollutant	CAS	QL
Aluminum	7429905	1.0	Nickel	7440020	10
Antimony	7440360	0.10	Selenium	7782492	2.0
Arsenic (Total)	7440382	0.50	Silver	7440224	1.0
Beryllium	7440417	0.10	Thallium	7440280	0.10
Cadmium	7440439	0.10	Zinc	7440666	5.0
Chromium (Total)	7440473	0.40	Cyanide (Free)[2]	57125	10
Copper	7440508	10	Cyanide (Total)[2]	57125	5.0
Iron	7439896	100	Total Phenolic Compounds		--
Lead	7439921	5	Hardness (Total as CaCO3)		--
Mercury	7439976	0.010			

1 All metals must be analyzed for total recoverable concentration unless otherwise specified.
2 For effluent cyanide samples, at least six discrete grab samples must be collected over the operating day. Each aliquot must not be less than 100 mL and must be collected and composited into a larger container which has been preserved with sodium hydroxide to insure sample integrity.
3 Chemical Abstract Service

Attachment A
Additional Wastewater Characterization

Volatile Organic Compounds (ug/l unless otherwise specified)					
Pollutant	CAS	QL	Pollutant	CAS	QL
acrolein	107028	5.0	1,1-dichloroethylene	75354	0.50
acrylonitrile	107131	5.0	1,2-dichloropropane	78875	0.50
benzene	71432	0.50	1,3-dichloropropylene	542756	0.50
bromoform	75252	0.50	ethylbenzene	100414	0.50
carbon tetrachloride	56235	0.50	methyl bromide	74839	0.50
chlorobenzene	108907	0.50	methyl chloride	74873	0.50
chlorodibromomethane	124481	0.50	methylene chloride	75092	0.50
chloroethane	75003	0.50	1,1,2,2-tetrachloroethane	79345	0.50
2-chloroethylvinyl ether	110758	5.0	tetrachloroethylene	127184	0.50
chloroform	67663	0.50	toluene	108883	0.50
dichlorobromomethane	75274	0.50	1,1,1-trichloroethane	71556	0.50
1,1-dichloroethane	75343	0.50	1,1,2-trichloroethane	79005	0.50
1,2-dichloroethane	107062	0.50	trichloroethylene	79016	0.50
1,2-trans-dichloroethylene	156605	0.50	vinyl chloride	75014	0.50
The permittee must collect six discrete samples (not less than 40 mL) over the operating day with the results averaged.					

Acid-extractable Compounds (ug/l unless otherwise specified)					
Pollutant	CAS	QL	Pollutant	CAS	QL
p-chloro-m-cresol (Methyl-4,6-dinitrophenol 2)	59507	1.0	2-nitrophenol	88755	2.0
2-chlorophenol	95578	1.0	4-nitrophenol	100027	5.0
2,4-dichlorophenol	120832	1.0	pentachlorophenol	87865	2.0
~~2,4-dimethylphenol~~	105679	5.0	phenol	108952	1.0
2,4-dinitro-o-cresol	534521	2.0	2,4,5-trichlorophenol	95954	2.0
dinitrophenols	25550597	n/a	2,4,6-trichlorophenol	88062	1.0
2,4-dinitrophenol	51285	5.0			

Base-Neutral Compounds (ug/l unless otherwise specified)					
Pollutant	CAS	QL	Pollutant	CAS	QL
acenaphthene	83329	1.0	3,3-Dichlorobenzidine	91941	1.0
acenaphthylene	208968	1.0	diethyl phthalate	84662	1.0
anthracene	120127	1.0	dimethyl phthalate	131113	1.0
benzidine	92875	10	2,4-dinitrotoluene	121142	1.0
benzo(a)anthracene	56553	1.0	2,6-dinitrotoluene	606202	1.0
benzo(a)pyrene	50328	1.0	1,2-diphenylhydrazine	122667	5.0
3,4- Benzo(b)fluoranthene	205992	1.0	fluroranthene	206440	2.0
benzo(ghi)perylene	191242	1.0	fluorene	86737	1.0
benzo(k)fluoranthene	207089	1.0	hexachlorobenzene	118741	1.0
bis(2-chloroethoxy)methane	111911	2.0	hexachlorobutadiene	87683	2.0
bis(2-chloroethyl)ether	111444	1.0	hexachlorocyclopentadiene	77474	2.0
bis(2-chloroisopropyl)ether	108601	2.0	hexachloroethane	67721	2.0
~~Bis (Chloromethyl) ether~~	542881	na	indeno(1,2,3-cd)pyrene	193395	1.0
bis (2-ethylhexyl)phthalate	117817	1.0	isophorone	78591	10
4-bromophenyl phenyl ether	101553	1.0	napthalene	91203	1.0

Attachment A
Additional Wastewater Characterization

Base-Neutral Compounds (ug/l unless otherwise specified)					
Pollutant	CAS	QL	Pollutant	CAS	QL
butylbenzyl phthalate	85687	1.0	nitrobenzene	98953	1.0
2-chloronaphthalene	91587	1.0	N-nitrosodimethylamine	62759	1.0
4-chlorophenyl phenyl ether	7005723	1.0	N-nitrosodi-n-propylamine	621647	2.0
chrysene	218019	1.0	N-nitrosodiphenylamine	86306	1.0
di-n-butyl phthalate	84742	1.0	Pentachlorobenzene	608935	10
di-n-octyl phthalate	117817	1.0	phenanthrene	85018	1.0
dibenzo(a,h)anthracene	53703	1.0	pyrene	129000	1.0
1,2-Dichlorobenzene (o)	95501	0.50	1,2,4-trichlorobenzene	128821	5.0
1,3-Dichlorobenzene (m)	541731	0.50	Tetrachlorobenzene,1,2,4,5	95943	1.0
1,4-Dichlorobenzene (p)	106467	0.50			

~~Organochlorine Pesticides (ug/l unless otherwise specified)~~					
~~Pollutant~~	~~CAS~~	~~QL)~~	~~Pollutant~~	~~CAS~~	~~QL~~
~~Aldrin~~	~~309002~~	~~0.010~~	~~Endosulfan (total)~~	~~115297~~	~~0.010~~
~~BHC (Technical)~~	~~608731~~	~~0.010~~	~~Endrin~~	~~72208~~	~~0.010~~
~~BHC-alpha~~	~~319846~~	~~0.010~~	~~Endrin Aldehyde~~	~~7421934~~	~~0.010~~
~~BHC-beta~~	~~319857~~	~~0.010~~	~~Heptachlor~~	~~76448~~	~~0.010~~
~~BHC-delta~~	~~319868~~	~~0.010~~	~~Heptachlor Epoxide~~	~~1024573~~	~~0.010~~
~~BHC-gamma~~	~~58899~~	~~0.010~~	~~Methoxychlor~~	~~72435~~	~~0.010~~
~~Chlordane~~	~~57749~~	~~0.10~~	~~Toxaphene~~	~~8001352~~	~~0.50~~
~~DDD 4,4'~~	~~72548~~	~~0.010~~	~~PCB- Aroclor 1016~~	~~12674112~~	~~0.50~~
~~DDE 4,4'~~	~~72559~~	~~0.010~~	~~PCB- Aroclor 1221~~	~~11104282~~	~~0.50~~
~~DDT 4,4'~~	~~50293~~	~~0.010~~	~~PCB- Aroclor 1232~~	~~11141165~~	~~0.50~~
~~Dieldrin~~	~~60571~~	~~0.010~~	~~PCB- Aroclor 1242~~	~~53469219~~	~~0.50~~
~~Endosulfan alpha-~~	~~959988~~	~~0.010~~	~~PCB- Aroclor 1248~~	~~12672296~~	~~0.50~~
~~Endosulfan beta-~~	~~33213659~~	~~0.010~~	~~PCB- Aroclor 1254~~	~~11097691~~	~~0.50~~
~~Endosulfan Sulfate~~	~~1031078~~	~~0.010~~	~~PCB- Aroclor 1260~~	~~11096825~~	~~0.50~~

DEQ-DC1

Analytical Laboratory & Consultants, Inc.
361 West Fifth Avenue • Eugene, OR 97401
541-485-8404 • 1-800-262-5973
Fax: 541-484-5995

Oregon ORELAP Accredited
ORELAP ID OR100012 EPA ID OR00016
Accredited in accordance with NELAC
Accredited analytes marked "¥"

ANALYSIS REPORT

Attention	Brian Elliott	Lab Order No.	1209643
Client	City of Sutherlin	Date Received	9/26/12
	126 East Central Avenue		
	Sutherlin, OR 97479		
Project ID			

CASE NARRATIVE

This report presents the results of the analyses of the sample(s) received on the date above and assigned the listed ALC lab report numbers. Test results relate only to the parameters tested and to the samples as received by the laboratory. This report shall not be reproduced, except in full, without written consent of Analytical Laboratory & Consultants, Inc.

All analyses were performed according to the Analytical Laboratory & Consultants, Inc. Quality Assurance Program.
All QA/QC requirements were met except as noted below.
Analytical comments are noted with data flags on the reports and/or below.

For the analyses performed by ALC: No anomalies associated with the analysis of these sample(s) were observed.

Trace Metals by EPA 200.7/200.8 were analyzed by Neilson Research Corporation, Medford, OR.
No anomalies associated with the analysis of these sample(s) were reported except as noted in the NRC Case Narrative or qualified with data flags on the NRC report. For NRC Case Narrative see ALC Report Page 5. For sample ID Key see ALC Report Page 3. For results see ALC Report Page 6.

Volatile Organic Compounds EPA 624 were analyzed by Neilson Research Corporation, Medford OR.
No anomalies associated with the analysis of these sample(s) were reported except as noted in the NRC Case Narrative or qualified with data flags on the NRC report. For NRC Case Narrative see ALC Report Page 5. For sample ID Key see ALC Report Page 3. For results see ALC Report Pages 7-8.

Semivolatile Organic Compounds EPA 625 were analyzed by Neilson Research Corporation, Medford OR.
No anomalies associated with the analysis of these sample(s) were reported except as noted in the NRC Case Narrative or qualified with data flags on the NRC report. For NRC Case Narrative see ALC Report Page 5. For sample ID Key see ALC Report Page 3. For results see ALC Report Pages 9-11.

Total Mercury by EPA 1631E was analyzed by ALS Environmental, Kelso, WA.
No anomalies associated with the analysis of these sample(s) were reported except as noted in the ALS Case Narrative or qualified with data flags on the ALS report. For ALS Case Narrative see ALC Report Page 35. For sample ID Key see ALC Report page 35. For results see ALC Report Page 42

Approved Sandra Nordman Laboratory Supervisor Date 10/24/12

Analytical Laboratory & Consultants, Inc.
361 West Fifth Avenue • Eugene, OR 97401
541-485-8404 • 1-800-262-5973
Fax: 541-484-5995

Oregon ORELAP Accredited
ORELAP ID OR100012 EPA ID OR00016
Accredited in accordance with NELAC
Accredited analytes marked "¥"

ANALYSIS REPORT

Attention	Brian Elliott	Lab Order No.	1302284
Client	City of Sutherlin	Date Received	2/13/13
	126 East Central Avenue		
	Sutherlin, OR 97479		
Project ID			

CASE NARRATIVE

This report presents the results of the analyses of the sample(s) received on the date above and assigned the listed ALC lab report numbers. Test results relate only to the parameters tested and to the samples as received by the laboratory. This report shall not be reproduced, except in full, without written consent of Analytical Laboratory & Consultants, Inc.

All analyses were performed according to the Analytical Laboratory & Consultants, Inc. Quality Assurance Program.
All QA/QC requirements were met except as noted below.
Analytical comments are noted with data flags on the reports and/or below.

For the analyses performed by ALC: No anomalies associated with the analysis of these sample(s) were observed.

Trace Metals by EPA 200.7/200.8 were analyzed by Neilson Research Corporation, Medford, OR.
No anomalies associated with the analysis of these sample(s) were reported except as noted in the NRC Case Narrative or qualified with data flags on the NRC report. For NRC Case Narrative see ALC Report Page 5. For sample ID Key see ALC Report Page 3. For results see ALC Report Page 6.

Volatile Organic Compounds EPA 624 were analyzed by Neilson Research Corporation, Medford OR.
No anomalies associated with the analysis of these sample(s) were reported except as noted in the NRC Case Narrative or qualified with data flags on the NRC report. For NRC Case Narrative see ALC Report Page 5. For sample ID Key see ALC Report Page 3. For results see ALC Report Pages 7-8.

Semivolatile Organic Compounds EPA 625 were analyzed by Neilson Research Corporation, Medford OR.
No anomalies associated with the analysis of these sample(s) were reported except as noted in the NRC Case Narrative or qualified with data flags on the NRC report. For NRC Case Narrative see ALC Report Page 5. For sample ID Key see ALC Report Page 3. For results see ALC Report Pages 9-10.

Total Mercury by EPA 1631E was analyzed by ALS Environmental, Kelso, WA.
No anomalies associated with the analysis of these sample(s) were reported except as noted in the ALS Case Narrative or qualified with data flags on the ALS report. For ALS Case Narrative see ALC Report Page 35. For sample ID Key see ALC Report page 34. For results see ALC Report Page 41.

Approved Sandra Nordman Laboratory Supervisor Date 3/4/13

State of Oregon
Department of
Environmental
Quality

OREGON DEPARTMENT OF ENVIRONMENTAL QUALITY
Memorandum
WESTERN REGION - MEDFORD

To: **Sutherlin STP File** **Date: DATE**

From: Jonathan Gasik, MS, PE, Senior Engineer
Western Region Medford Office

Subject: City of Sutherlin Excess Thermal Load Limits

WQ File No. 86662

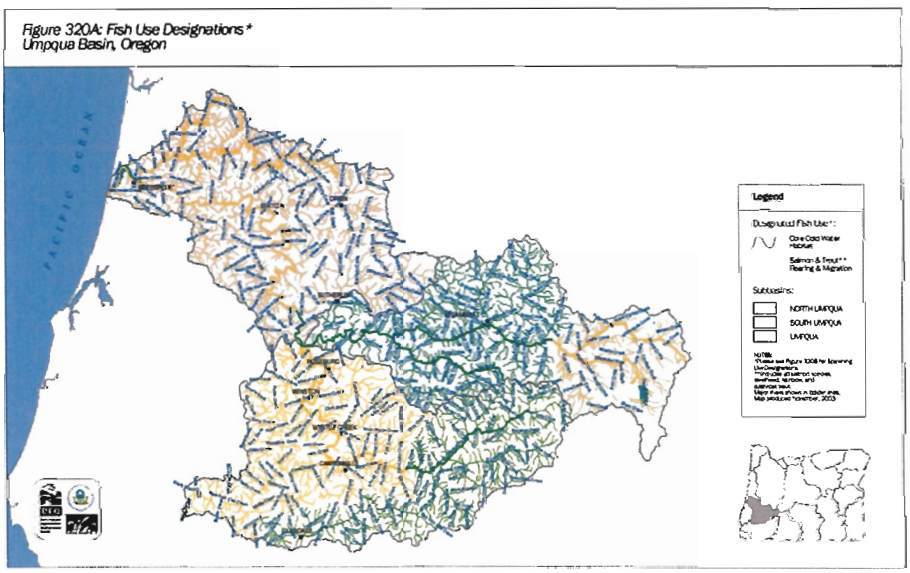
PURPOSE

In April 2008, DEQ issued the Temperature Standard Implementation Internal Management Directive (Temperature IMD). This memo will use the methods described in the IMD to calculate Excess Thermal Loads and evaluate whether the Sutherlin WWTF can immediately comply with the calculated Excess Thermal Load Limits.

BACKGROUND

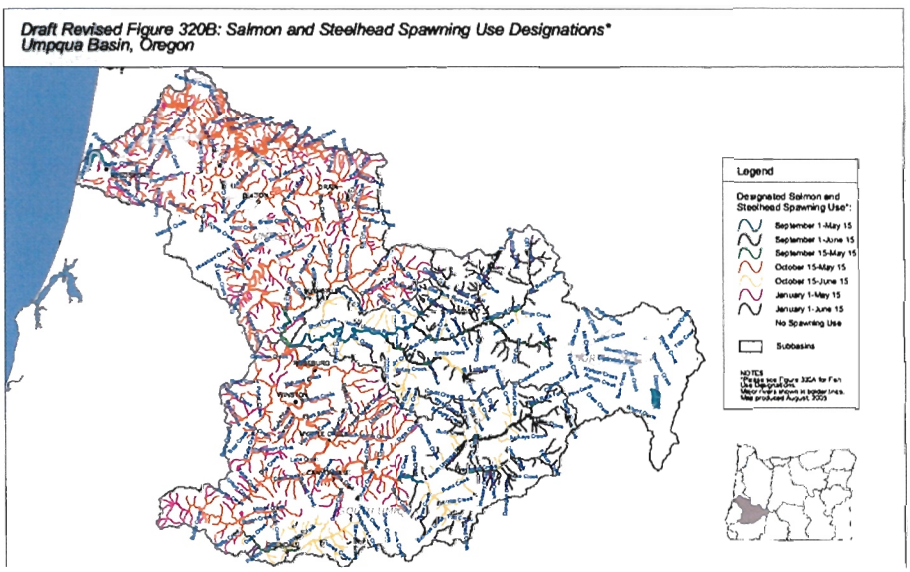
Applicable Temperature Criteria

The Sutherlin WWTF discharges into Calapooya Creek at River Mile 9.8. The discharge is within the Umpqua Basin and Umpqua sub-basin. DEQ utilizes Fish Use Designation and Salmon and Steelhead Spawning Use Designations maps to identify applicable temperature criteria for each basin. The Umpqua Basin maps are contained in OAR 340-041, Figures 320A (Fish Use Designation) and 320B (Steelhead Spawning Use Designations).



From the Fish Use Designation map above, Calapooya Creek has salmon and trout rearing and migration uses. The applicable biologically based numeric criteria (BBNC) is 18°C during periods when spawning does not occur.

According to the salmon and steelhead spawning use designation map below, spawning occurs in Calapooya Creek from October 15 through May 15. During this time, the applicable BBNC is 13°C.





Integrated Report Assessment Database and 303(d) List

The Integrated Report Assessment Database contains information on water quality in Oregon's surface waters and includes waters identified as water quality limited that need TMDLs (303(d) list). The following information is from the 2010 integrated report. There is no other information on the status of the temperature standard:

River Mile	Pollutant	Season	Status	2010 Assessment Action
0 to 36.1	Temperature	Year Around (Non-spawning)	Cat 4A: Water quality limited, TMDL approved	Delisted - TMDL approved

While the 2010 Integrated Report did not address the spawning criterion, the assessment information will be updated to reflect the data and applicable criteria the next time the report is generated.

Temperature TMDL

In October 2006, DEQ finalized the Umpqua Basin TMDL. EPA approved this TMDL on April 13, 2007. The Umpqua Basin TMDL allocates the Sutherlin WWTF a 0.10°C increase above the applicable criterion in Calapooya Creek. This allowance over the criterion is known as the human use allowance (HUA).

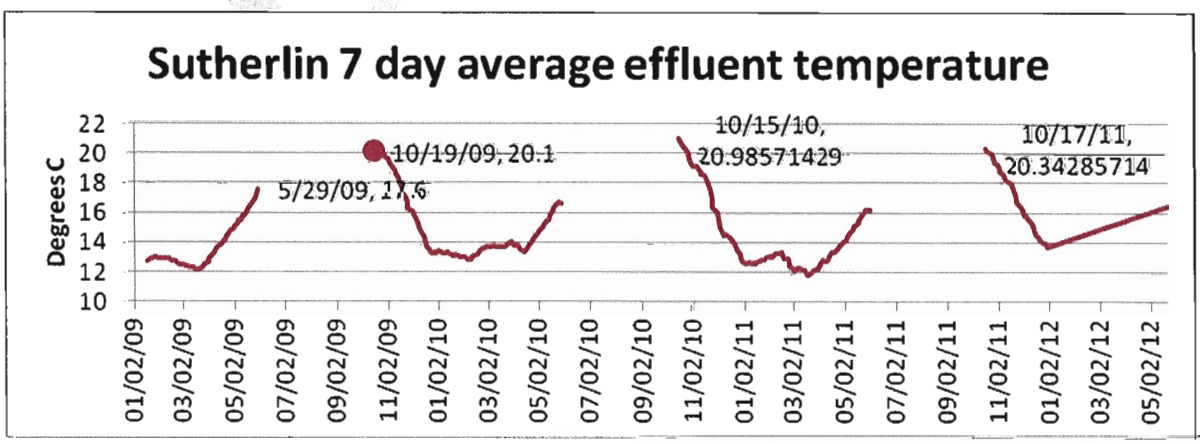
The TMDL only addresses the non-spawning impairment and is therefore only applicable May 16 through October 14.

As part of the TMDL process, the natural thermal potential (NTP) temperature of Calapooya Creek was modeled using a computer program (Heat Source). The critical case (mid-July) NTP at RM 9.8 is about 26°C. Additionally, the current conditions are similar to the NTP at this point in Calapooya Creek. As long as the system is not dramatically altered, it is likely that current conditions will also be similar to NTP for other periods of the year. Therefore, NTP can be adequately estimated using current conditions for the purposes of determining waste load allocations (WLAs) for times other than the critical period at this point.

AVAILABLE DATA

Effluent Flow and Temperature

The city of Sutherlin wastewater treatment plant collects daily effluent flow data. From January 2009 through December 2011, the wastewater treatment plant also collected effluent temperature and dissolved oxygen data three times per week. This data is used in this analysis. The following chart is a summary of this data:

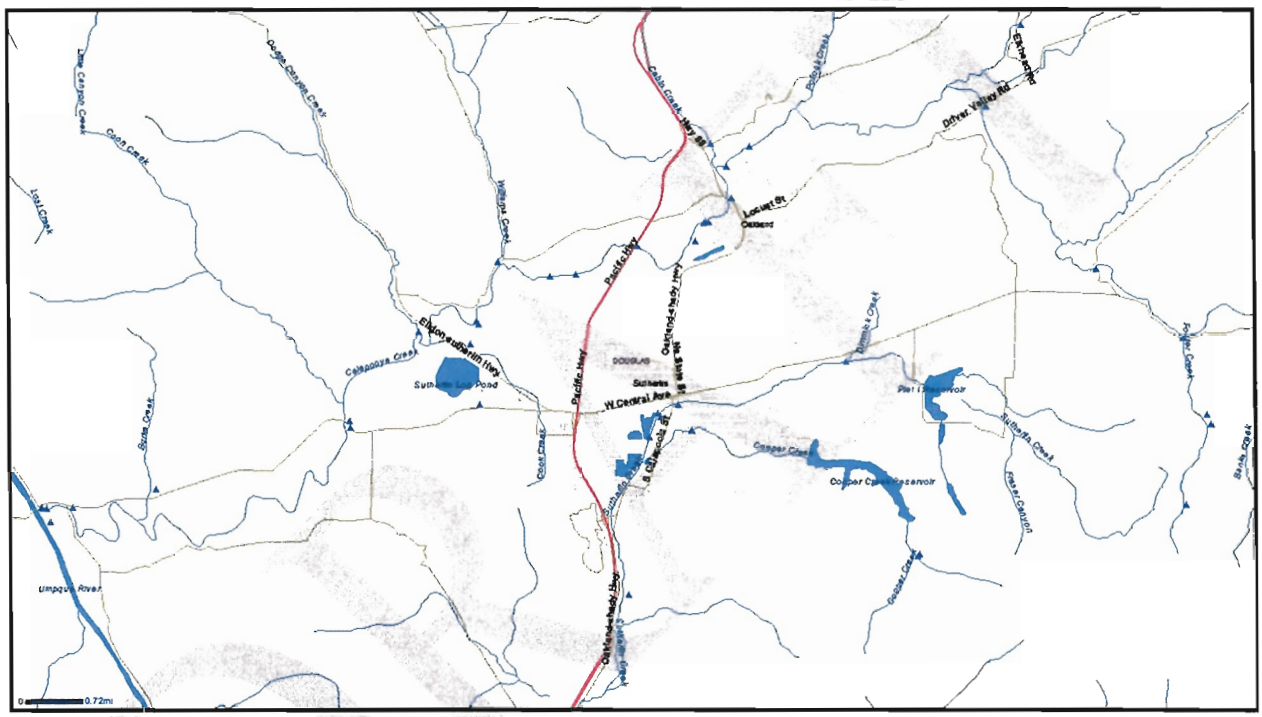


Stream Flow

Between October 1, 1955 and September 30, 2001, the USGS maintained a stream gage on Calapooya Creek (14320700, Calapooya Creek at Oakland). Douglas County now maintains this stream gage. The data from this gage is used in this analysis. The EPA program DFLOW is used to calculate various design flows, such as the seven-day average flow with a 10 year recurrence interval (7Q10).

Stream Temperature

Oregon DEQ has several ambient water quality monitoring sites on Calapoola Creek:



The following table lists the stations on Calapooya Creek in the vicinity of Sutherlin where temperature has been monitored:

Station	LOCATION_DESCRIPTION	BEGIN_DATE	END_DATE	Number of Results	Latitude	Longitude	Location *
10996	Calapooya Creek at Umpqua	2/15/1972 9:50	9/10/2012 11:13	13215	43.3664	-123.46	1
12808	Calapooya Creek at Fort McKay Road (Stephens)	1/23/1997 15:30	7/15/1998 12:40	42	43.3834	-123.393	2

33226	Calapooya Creek at Rochester Covered Bridge	5/15/2006 18:50	10/19/2006 9:25	205	43.402	-123.363	3
12806	Calapooya Creek 8 meters downtream of Sutherlin WWTF outfall	1/1/1997	10/8/1998 15:45	1424	43.4015	-123.363	4
12805	Calapooya Creek 20 meters upstream of Sutherlin WWTF outfall	1/1/1997	7/15/1998 11:30	1418	43.3978	-123.364	5
13072	Calapooya Creek 0.5 miles downstream of Strauss Ranch House	11/6/1997 10:50	11/6/1997 11:00	18	43.4105	-123.346	6
13071	Calapooya Creek 100 feet upstream of Strauss Ranch House	11/6/1997 10:00	11/6/1997 10:10	18	43.4109	-123.339	7
12807	Calapooya Creek at I-5 Bridge	1/23/1997 14:35	10/15/2002 14:25	297	43.4163	-123.324	8
11307	Calapooya Creek downstream of Oakland	4/10/1973 10:30	7/19/1977 9:30	53	43.3822	-123.393	9
12802	Calapooya Creek 90 meters downstream of Oakland WWTF outfall	1/1/1997	10/8/1998 14:40	1405	43.4208	-123.309	10
12801	Calapooya Creek 9 meters upstream of Oakland WWTF outfall	1/1/1997	1/24/1997 10:30	1367	43.4211	-123.308	11

- - downstream to upstream

Station 10992 "Calapooya Creek at Umpqua" contains the most temperature data and will be used in this analysis.

Mixing Zone Study

On October 9, 2008, DEQ staff conducted a mixing zone study. This study determined that the minimum dilution at the edge of the ZID is about 2 and the average dilution at the edge of the mixing zone is about 8. The stream flow at the time was about 19 cfs.

Thermal Effluent Limit Determination and Calculations

The Temperature IMD contains procedures for implementing the temperature standard in NPDES permits. The first step is to determine whether a permit limit is needed based on meeting the applicable criteria and/or the thermal plume requirements.

Reasonable Potential Analysis

Permit writers must perform a "reasonable potential analysis" to determine which pollutants in the discharge have a reasonable potential to violate water quality standards. Limits must be included in the permit for those pollutants that have a reasonable potential to violate standards. "Limitations must control all pollutants …which the Director determines are or may be discharged at a level which will cause, have the reasonable potential to cause, or contribute to an excursion above any State water quality standard, including State narrative criteria for water quality." (40CFR122.44(d)(1)(i)).

This critical low flow is the lowest seven day average that has a 10 year recurrence (7Q10). Sutherlin has a land application system and is prohibited from discharging between July 1 and October 31. Therefore, the critical low flow period for the non-spawning season is May 16 – May 31. Likely, discharge is prohibited during October, so the critical low flow period for the spawning season is November 1 – May 15. DEQ uses the EPA program DFLOW to calculate the 7Q10 stream flows during these time periods

The following table summarizes the results of this review:

Time Period	Criterion °C	Effluent Temp °C	Effluent Flow CFS	Receiving Water 7Q10 CFS	ΔT at edge of MZ	ΔT with 25% of 7Q10 Flow	Reasonable Potential?
Non-Spawning [1]	18°C	17.6 [1]	1.66 (1.07 mgd)	34.3 [1]	NA	NA	No
Spawning [2]	13°C	19.6	2.60 (1.68 mgd)	19.3 [2]	0.82°C	2.3°C	Yes

1- Based on May 16 – May 31
2- Based on Nov 1 – May 15

For the non-spawning period, there is no reasonable potential to exceed the criteria because the maximum 7-day average effluent temperate is less than the criteria. For the spawning period, there is a reasonable potential for the discharge to cause an exceedance of the temperature criteria and DEQ will establish an excess thermal load limit.

EXCESS THERMAL LOAD LIMITS

As described in the IMD, prior to issuance of a TMDL, the excess thermal load (ETL) limit is calculated using the following equation:

ETL limit = Q_{ps} D ΔT_{ps} c

Where, Qps = Point Source Effluent Design Flow (cfs)
D = dilution using the smaller of 25% of stream flow or MZ study results.
ΔT_{ps} = Allowed temperature increase at the edge of MZ
C = conversion factor (2.45 million kcal·s/ °C·ft3·day)

Spawning Season (October 15 – May 15)

The critical low flow during this time period (excluding October 16-31) is 19.3 (USGS stream gage 14320700, Calapooya Creek at Oakland). From the reasonable potential analysis calculations above, the MZ study dilution is greater than the dilution based on 25% of 7Q10 flow. Therefore, the spawning season pre-TMDL critical low-flow scenario ETL limit is as follows:

ETL limit = Q_{ps} D ΔT_{ps} c = 2.6*2.86*0.3*2.45 = 5.46 million kcals/day

D = (Qps + 0.25*Qr)/Qps = (2.6 +0.25*19.3)/2.6 = 2.86

The Umpqua Basin TMDL does not apply during the spawning season.

Actual Stream Flow-based Limit:

The ETL limit may also be based on the actual stream flow. In this case, Sutherlin would need to collect and report stream flow data, and calculate the ETL limit on a running 7 day basis using the equations above.

EXCESS THERMAL LOAD (ETL)

The ETL calculated using the following equation:

Excess Thermal Load = Qps*ΔT*c

Where, Qps = point source effluent flow
ΔT = difference between the effluent temperature and the applicable criterion (Te-Tcriterion)
C = unit conversion factor (2,446,665 kcal·s/ °C·ft3·day)

The ETL is calculated as a weekly average by using the seven day average of the effluent flow and the seven day average of the daily maximum effluent temperatures.

Spawning Season (October 15 – May 15)

Using the equation above and the average wet weather flow, the ETL would be:

Excess Thermal Load = Qps*ΔT*c = 2.6*(19.3-13)*2.45 = 40 million kcals/day

While this is well in excess of the November critical case excess thermal load limit, November flows are typically lower than the average wet weather flow. The following table shows estimated critical case low flow scenarios by month:

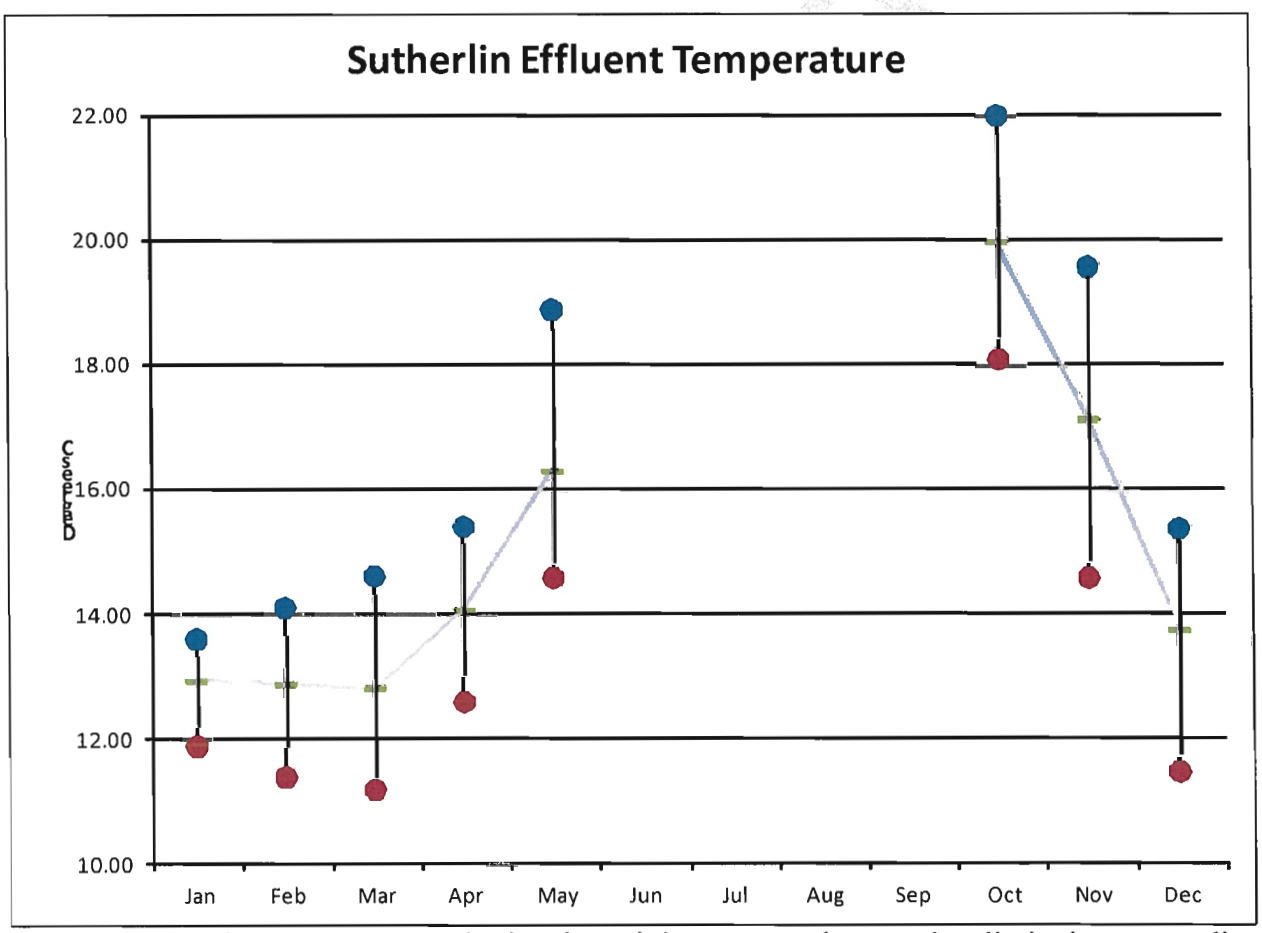
Month	Qr (cfs)	Qe (cfs)	Criteron (°C)	Max Te (°C)	ETL Limit (mKcals/d)	ETL (mKcals/d)	ETLL - ETL (mKcals/d)
Nov	19.2	1.67	13.0	19.63	4.76	27.12	-22.37
Dec	47.0	1.98	13.0	15.87	10.09	13.92	-3.83
Jan	139.0	2.32	13.0	13.36	27.25	2.03	25.22

Feb	186.0	1.95	13.0	13.67	35.61	3.22	32.40
Mar	165.0	2.38	13.0	14.03	32.07	6.00	26.07
Apr	113.0	2.13	13.0	14.90	22.33	9.89	12.43
May	78.8	1.45	13.0	17.59	15.55	16.33	-0.78

From the above table, additional thermal reductions will be needed to meet the temerpature limits during November and potentially December.

Thermal Plume Analysis

The following chart shows Sutherlin's minimum, maximum, and average effluent temperatures:



In addition to the temperature criteria, the mixing zone rule contains limitations regarding thermal plumes. In accordance with OAR 340-041-0053(2)(d), temperature mixing zones and thermal effluent limits will be established as necessary to prevent or minimize the following adverse effects on salmonids:

A) Impairment of an active salmonid spawning area where spawning redds are located or likely to be located.
B) Acute impairment or instantaneous lethality.
C) Thermal shock caused by sudden increase in water temperature.
D) Migration blockage caused by temperature differential between the plume and the receiving stream.

OAR 340-041-0053(2)(d)(A) applies to active salmonids spawning areas where spawning redds are located or likely to be located. As discussed above, according to the approved use designation maps, salmon and steelhead spawning is not a designated use in Calapooya Creek in the area of the Outfall, although spawning is a designated use in the main stem Umpqua 9.8 river miles downstream. However, Oregon Department of Fish and Wildlife has recently informed DEQ that spawning dose occur in the lower section of the Calapooya near Rochester bridge.

Impairment of spawning is prevented or minimized by limiting potential fish exposure to temperatures above 13 degrees Celsius [OAR 340-041-0053(2)(d)(A)]. Additional information is needed to determine the potential impacts.

Active Acute impairment or instantaneous lethality is prevented or minimized by limiting potential fish exposure to temperatures of 32°C or more to less than two seconds. Acute impairment requirements are met because all of the effluent temperature data is below this temperature. The highest effluent temperature recorded is 22° C on 10/1/2010.

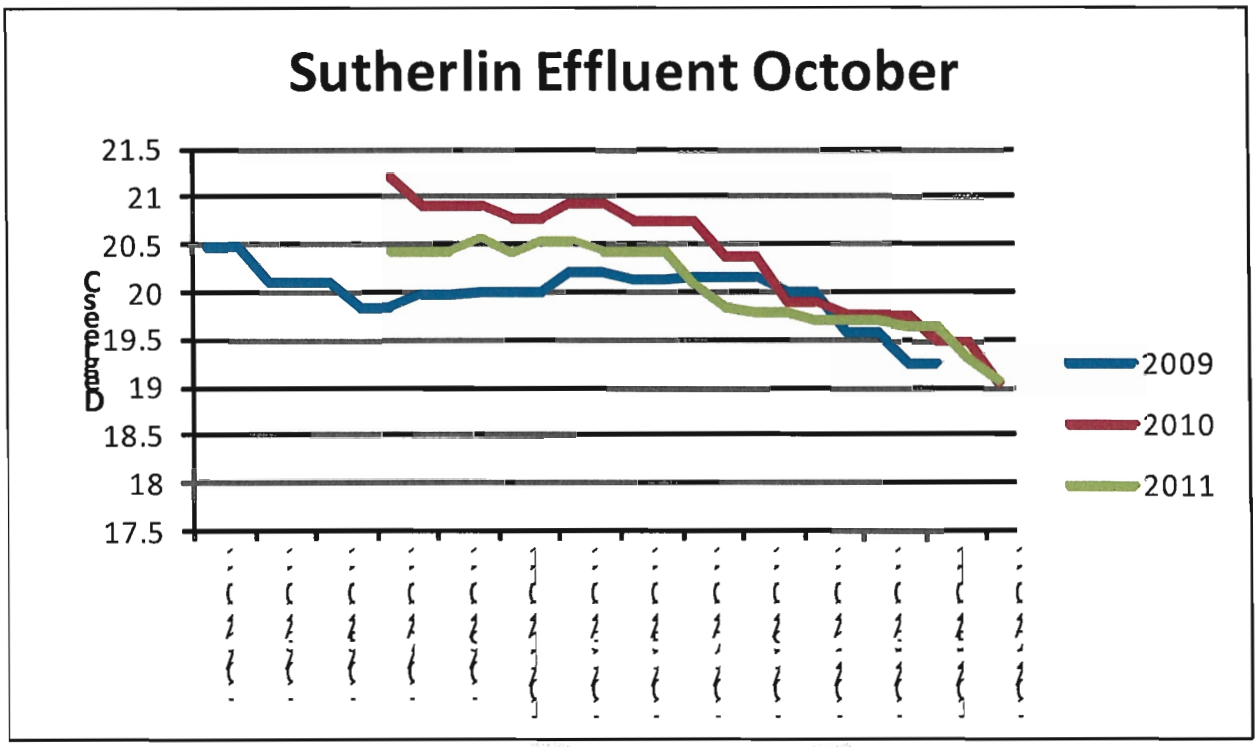
Thermal shock is prevented or minimized by limiting potential fish exposure to temperatures of 25°C or more to less than 5 percent of the cross section of the water body, unless the upstream temperature is above 23°C. This requirement is met because all of the effluent temperature data is below this temperature. The highest effluent temperature recorded is 22° C on 10/1/2010.

Migration blockage is prevented or minimized by limiting potential fish exposure to temperatures of 21°C or more to less than 25 percent of the cross section of the water body, unless the upstream temperature is above 21°C. This requirement is met because all of the effluent temperature data is below this temperature during the permitted discharge period. The highest effluent temperature recorded during this period is 19.2° C on 11/1/2009.

October Discharge Evaluation

The City of Sutherlin has asked DEQ to evaluate the possibility of an October discharge.

The following chart shows the change in discharge temperature in October:



Using calculations similar to those above, thermal reductions and/or thermal trading would also be needed to allow an October discharge:

Month	Qr (cfs)	Qe (cfs)	Criteron (°C)	Te (°C)	ETL Limit (mKcals/d)	ETL (mKcals/d)	ETLL - ETL (mKcals/d)
Oct 15-31	5.3	1.03	13.0	20.00	1.73	17.66	-15.93

APPENDIX B – MISCELLANEOUS MAPS

B

FEMA Map

Isopluvial Maps

Monthly Precipitation

Hazard Maps

Soil Survey, Douglas County

Wetland Maps

Collection System Maps



MAP SCALE 1" = 1000'

500 0 1000 2000

FEET

MET

NFIP

PANEL 0990F

FIRM

FLOOD INSURANCE RATE MAP

DOUGLAS COUNTY, OREGON AND INCORPORATED AREAS

PANEL 990 OF 3100

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
DOUGLAS COUNTY	410059	0990	F
SUTHERLIN, CITY OF	410275	0990	F

NATIONAL FLOOD INSURANCE PROGRAM

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
41019C0990F

EFFECTIVE DATE
FEBRUARY 17, 2010

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

LEGEND

SPECIAL FLOOD HAZARD AREAS (SFHAs) SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Areas formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X	Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X	Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D	Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

FLOOD INSURANCE RATE MAP

DOUGLAS COUNTY, OREGON

AND INCORPORATED AREAS

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

1% Annual Chance Floodplain Boundary

0.2% Annual Chance Floodplain Boundary

Floodway boundary

Zone D boundary

CBRS and OPA boundary

Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.

513 — Base Flood Elevation line and value; elevation in feet*

(EL 987) — Base Flood Elevation value where uniform within zone; elevation in feet*

*Referenced to the North American Vertical Datum of 1988

A — A — Cross section line

23 - - - - - 23 — Transect line

45° 02' 08", 93° 02' 12" — Geographic coordinates referenced to the North American Datum of 1983 (NAD 83) Western Hemisphere

3100000 FT — 5000-foot ticks: Oregon State Plane South Zone (FIPS Zone 3602), Lambert Conformal Conic projection

4889000m N — 1000-meter Universal Transverse Mercator grid values, zone 10N

DX5510 × — Bench mark (see explanation in Notes to Users section of this FIRM panel)

M1.5 — River Mile

MAP REPOSITORIES

Refer to Map Repositories list on Map Index

EFFECTIVE DATE OF COUNTYWIDE FLOOD INSURANCE RATE MAP

FEBRUARY 17, 2010

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

For community map revision history prior to countywide mapping, refer to the Community Map History table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

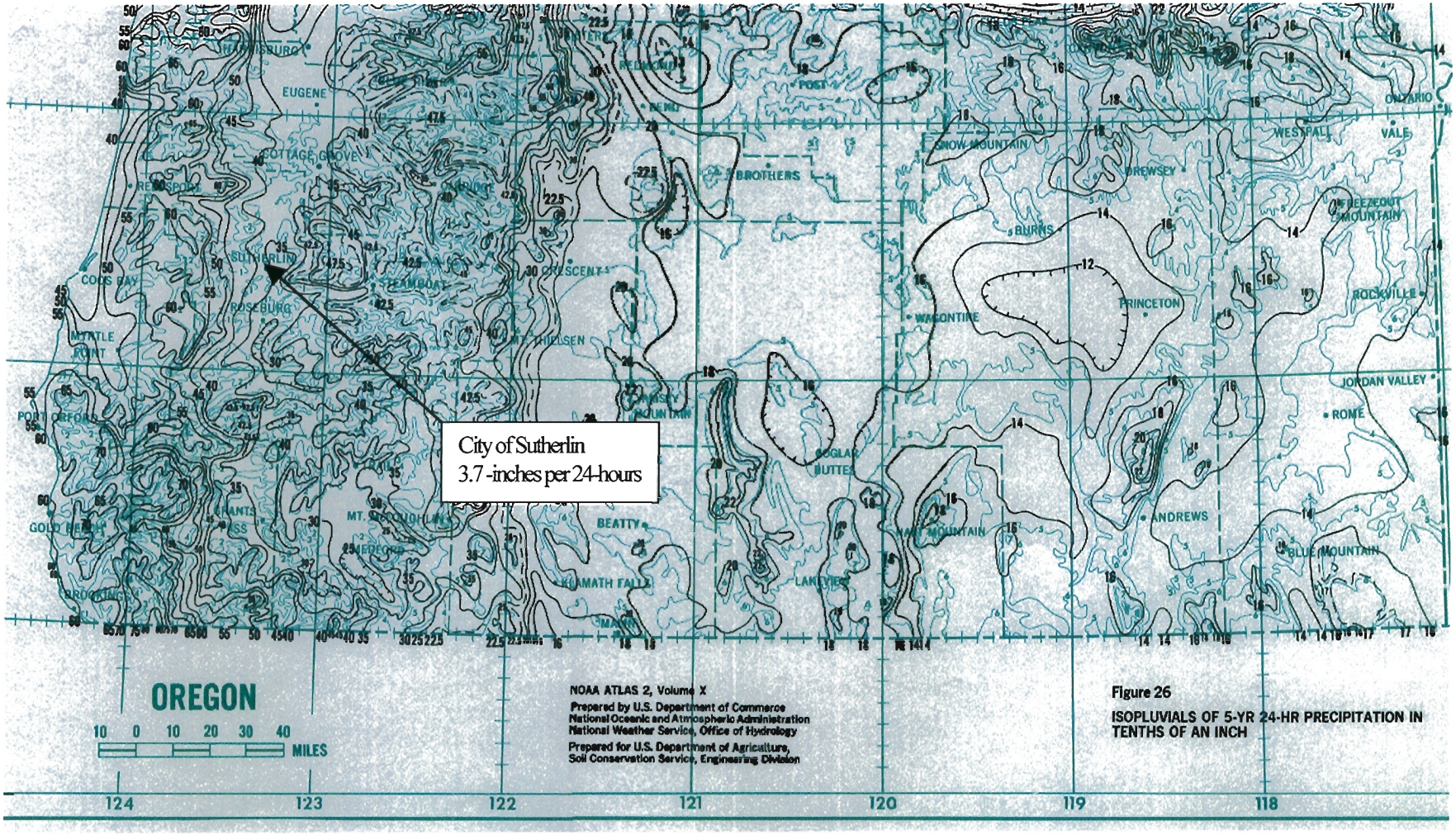
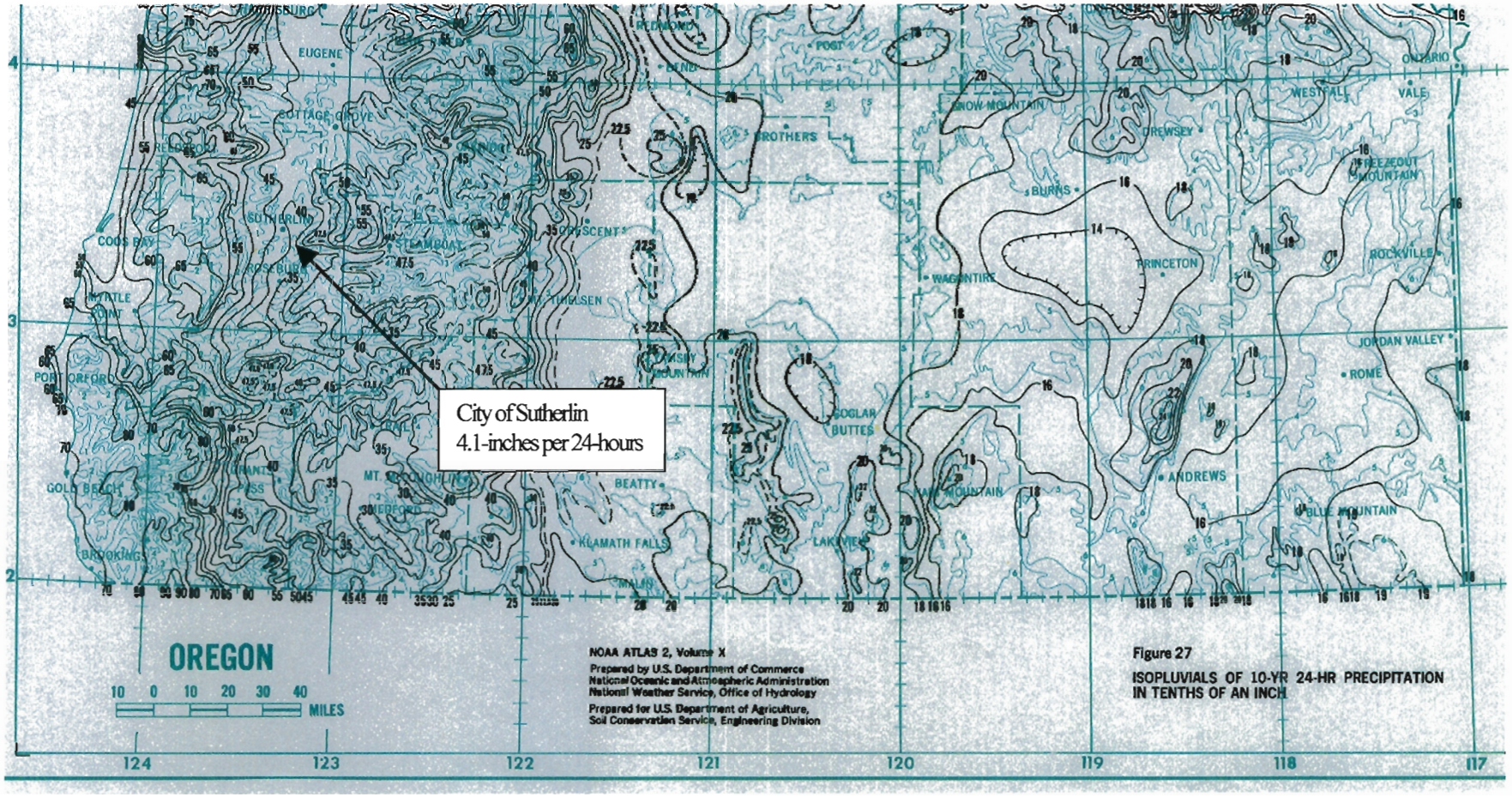
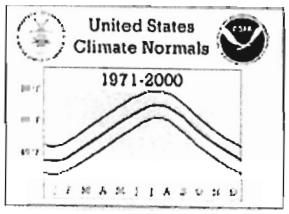


Figure 26

ISOPLUVIALS OF 5-YR 24-HR PRECIPITATION IN TENTHS OF AN INCH


City of Sutherlin
4.1-inches per 24-hours
OREGON
10 0 10 20 30 40
MILES
NOAA ATLAS 2, Volume X
Prepared by U.S. Department of Commerce
National Oceanic and Atmospheric Administration
National Weather Service, Office of Hydrology
Prepared for U.S. Department of Agriculture,
Soil Conservation Service, Engineering Division
Figure 27
ISOPLUVIALS OF 10-YR 24-HR PRECIPITATION
IN TENTHS OF AN INCH
124
123
122
121
120
119
118
117
4
3
2



Monthly Precipitation Probabilities and Quintiles, 1971-2000

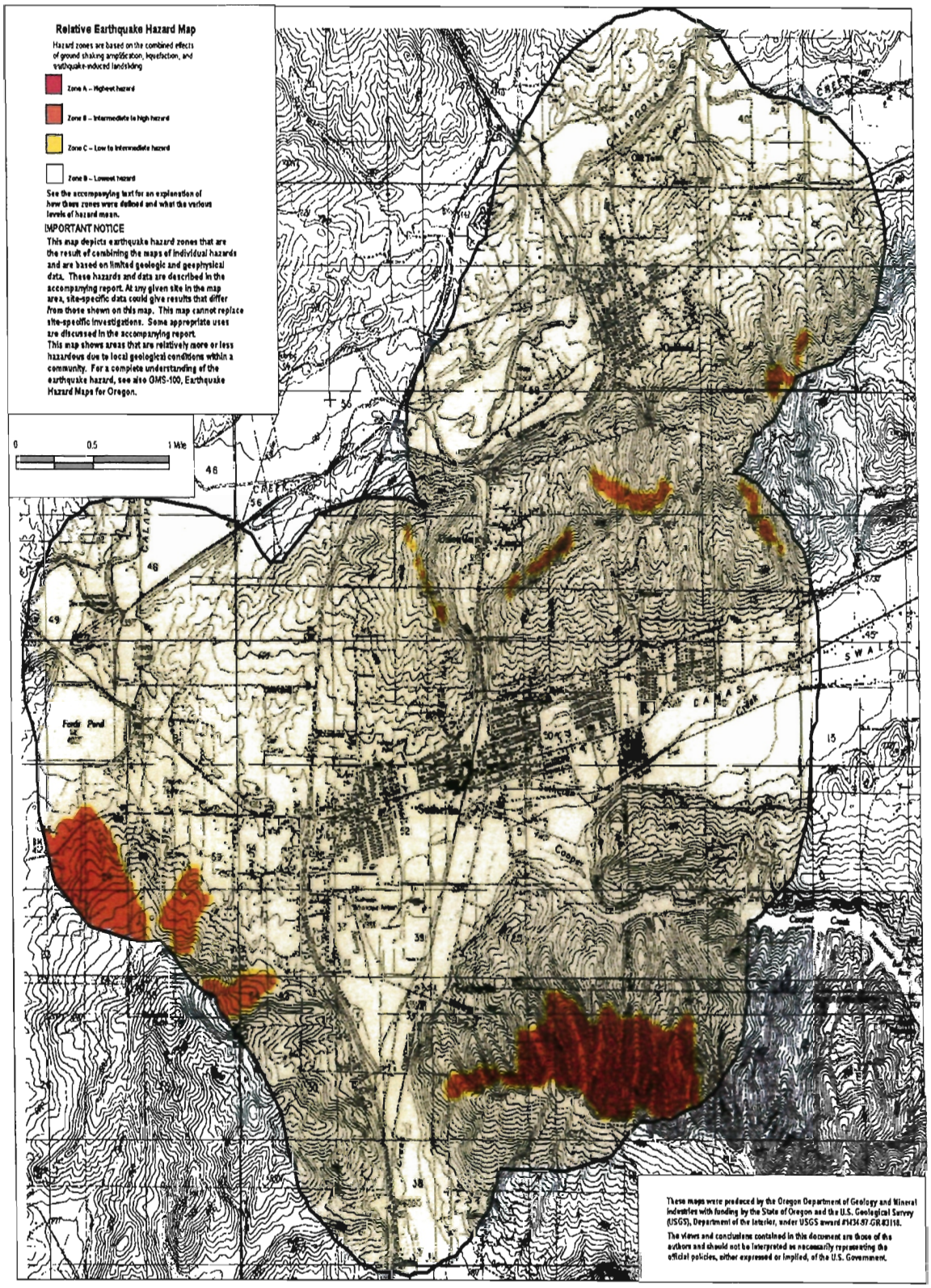
CLIMATOGRAPHY OF THE UNITED STATES NO. 81, Supplement No. 1

Monthly Precipitation (inches)

Station Description	Quin	Prob	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	ANN
357310 REO	Q0	MIN	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	
ROME 2 NW		0.1	0.00	0.09	0.24	0.25	0.10	0.10	0.00	0.00	0.00	0.00	0.23	0.10	5.19
OR	Q1	0.2	0.18	0.17	0.37	0.39	0.29	0.26	0.00	0.00	0.00	0.13	0.35	0.19	6.07
	Q2	0.4	0.46	0.35	0.59	0.62	0.67	0.54	0.09	0.00	0.22	0.31	0.55	0.41	7.39
		0.5	0.61	0.45	0.70	0.74	0.89	0.69	0.20	0.08	0.34	0.41	0.64	0.54	8.01
	Q3	0.6	0.78	0.57	0.82	0.87	1.14	0.86	0.34	0.19	0.48	0.53	0.75	0.70	8.67
	Q4	0.8	1.28	0.93	1.16	1.22	1.89	1.34	0.79	0.53	0.91	0.87	1.04	1.19	10.33
		0.9	1.75	1.26	1.46	1.54	2.61	1.79	1.25	0.87	1.34	1.19	1.29	1.66	11.70
	Q5	MAX	3.30	1.68	1.86	2.74	4.99	2.11	2.56	1.88	2.37	2.33	1.53	2.32	
357331 RBG	Q0	MIN	0.58	1.02	1.25	0.88	0.27	0.13	0.00	0.00	0.00	0.06	1.09	0.84	
ROSEBURG KQEN		0.1	1.74	1.62	1.91	1.23	0.49	0.18	0.00	0.00	0.00	0.41	1.89	1.65	23.68
OR	Q1	0.2	2.47	2.21	2.40	1.60	0.75	0.31	0.00	0.00	0.12	0.73	2.68	2.45	26.64
	Q2	0.4	3.73	3.20	3.18	2.22	1.25	0.57	0.12	0.12	0.44	1.38	4.07	3.89	30.98
		0.5	4.39	3.70	3.57	2.53	1.53	0.72	0.22	0.25	0.66	1.76	4.79	4.67	32.99
	Q3	0.6	5.13	4.26	3.99	2.87	1.84	0.90	0.34	0.44	0.93	2.21	5.60	5.54	35.08
	Q4	0.8	7.14	5.77	5.08	3.77	2.72	1.42	0.76	1.13	1.78	3.54	7.81	7.96	40.27
		0.9	8.94	7.09	6.01	4.55	3.52	1.91	1.21	1.91	2.66	4.80	9.78	10.17	44.47
	Q5	MAX	9.92	9.75	6.99	5.61	6.33	2.67	2.98	3.30	3.70	4.66	15.91	15.77	
357500 SLE	Q0	MIN	0.41	0.75	0.89	0.62	0.05	0.26	0.00	0.00	0.00	0.12	1.13	1.26	
SALEM MCNARY AP		0.1	2.04	1.88	1.92	1.11	0.63	0.44	0.01	0.00	0.10	0.68	2.44	2.59	28.58
OR	Q1	0.2	2.88	2.62	2.49	1.50	0.94	0.66	0.06	0.04	0.27	1.12	3.36	3.51	32.07
	Q2	0.4	4.37	3.89	3.40	2.16	1.52	1.04	0.22	0.24	0.69	1.98	4.92	5.06	37.01
		0.5	5.15	4.54	3.86	2.50	1.82	1.25	0.34	0.38	0.96	2.46	5.73	5.85	39.28
	Q3	0.6	6.02	5.27	4.36	2.87	2.17	1.48	0.48	0.57	1.30	3.02	6.63	6.72	41.89
	Q4	0.8	8.40	7.26	5.68	3.87	3.14	2.13	0.95	1.16	2.34	4.62	9.04	9.07	47.51
		0.9	10.52	9.02	6.82	4.75	4.03	2.72	1.43	1.77	3.39	6.12	11.18	11.13	52.23
	Q5	MAX	10.89	13.01	8.56	5.72	5.56	4.19	2.63	2.56	3.98	7.96	15.23	15.01	
357698 SXT	Q0	MIN	0.18	0.28	1.43	0.73	0.11	0.10	0.00	0.00	0.00	0.01	0.74	0.49	
SEXTON SUMMIT		0.1	0.98	1.52	1.86	1.02	0.36	0.28	0.00	0.00	0.00	0.31	1.31	1.27	20.99
OR	Q1	0.2	1.65	2.14	2.38	1.35	0.56	0.42	0.04	0.00	0.13	0.66	2.08	2.02	24.45
	Q2	0.4	3.00	3.23	3.23	1.90	0.93	0.67	0.15	0.13	0.50	1.50	3.57	3.47	29.68
		0.5	3.77	3.80	3.65	2.18	1.14	0.81	0.22	0.25	0.74	2.04	4.40	4.28	32.14
	Q3	0.6	4.66	4.43	4.10	2.48	1.37	0.96	0.31	0.42	1.04	2.69	5.34	5.20	34.74
	Q4	0.8	7.24	6.16	5.29	3.30	2.02	1.38	0.59	1.04	2.00	4.71	8.03	7.82	41.30
		0.9	9.68	7.71	6.32	4.01	2.62	1.76	0.86	1.73	2.98	6.74	10.53	10.26	46.70
	Q5	MAX	17.33	8.49	7.36	4.30	4.06	2.80	1.81	4.23	4.26	6.59	24.09	11.79	
360106 ABE	Q0	MIN	0.67	1.01	0.97	0.61	1.43	0.97	0.33	0.76	1.10	1.10	0.68	0.62	
ALLENTOWN LEHIGH VLY AP		0.1	1.20	1.35	1.77	1.32	2.03	1.68	1.44	1.42	1.51	1.45	1.37	1.08	36.37
PA	Q1	0.2	1.71	1.71	2.23	1.82	2.64	2.24	2.06	2.06	2.15	1.91	1.90	1.58	39.13
	Q2	0.4	2.61	2.28	2.97	2.68	3.63	3.17	3.17	3.20	3.27	2.67	2.83	2.47	43.04
		0.5	3.08	2.57	3.34	3.13	4.13	3.64	3.74	3.80	3.85	3.06	3.30	2.95	44.80
	Q3	0.6	3.60	2.88	3.73	3.62	4.68	4.16	4.39	4.47	4.51	3.48	3.83	3.48	46.60
	Q4	0.8	5.04	3.69	4.76	4.95	6.11	5.54	6.17	6.33	6.30	4.59	5.27	4.95	50.98
		0.9	6.33	4.38	5.64	6.13	7.36	6.75	7.77	8.01	7.91	5.56	6.55	6.28	54.43
	Q5	MAX	8.42	5.44	6.59	7.87	10.62	8.58	9.26	9.42	11.57	7.49	9.69	7.89	
360130 AOO	Q0	MIN	0.27	0.64	0.90	0.79	1.14	0.82	1.20	0.02	0.31	0.73	0.64	0.59	
ALTOONA BLAIR CO AP		0.1	0.75	1.00	1.60	1.15	2.19	1.51	1.66	0.86	1.15	0.95	0.99	1.00	29.52
PA	Q1	0.2	1.09	1.31	2.02	1.62	2.65	2.05	2.16	1.28	1.60	1.35	1.45	1.34	31.77
	Q2	0.4	1.71	1.84	2.67	2.44	3.35	2.97	2.99	2.05	2.36	2.07	2.27	1.89	34.97
		0.5	2.04	2.11	3.00	2.87	3.69	3.44	3.41	2.47	2.76	2.44	2.71	2.17	36.40
	Q3	0.6	2.41	2.40	3.35	3.34	4.06	3.96	3.86	2.93	3.20	2.86	3.20	2.48	37.88
	Q4	0.8	3.43	3.18	4.26	4.64	4.98	5.36	5.07	4.24	4.40	4.02	4.57	3.30	41.46
		0.9	4.35	3.85	5.04	5.80	5.76	6.60	6.11	5.43	5.46	5.05	5.81	4.02	44.29
	Q5	MAX	6.91	4.40	5.60	7.48	6.34	11.46	8.74	7.07	8.51	7.02	10.90	4.29	
360865 BFD	Q0	MIN	1.29	0.75	0.50	1.30	0.94	1.55	1.50	1.75	1.77	0.98	0.40	1.21	
BRADFORD RGNL AP		0.1	1.63	1.45	1.50	1.90	1.82	2.31	2.22	1.91	2.33	1.63	1.56	2.25	35.99
PA	Q1	0.2	2.12	1.83	2.04	2.33	2.35	3.09	2.73	2.44	2.83	2.04	2.09	2.71	39.25
	Q2	0.4	2.92	2.42	2.96	3.00	3.21	4.39	3.53	3.29	3.62	2.70	2.99	3.42	43.91
		0.5	3.33	2.72	3.42	3.32	3.65	5.04	3.93	3.71	4.01	3.03	3.44	3.77	46.03
	Q3	0.6	3.77	3.04	3.94	3.67	4.12	5.76	4.35	4.17	4.42	3.38	3.94	4.14	48.21
	Q4	0.8	4.93	3.87	5.33	4.57	5.36	7.70	5.43	5.37	5.47	4.29	5.29	5.08	53.54
		0.9	5.94	4.58	6.55	5.33	6.43	9.39	6.35	6.40	6.36	5.07	6.47	5.87	57.78
	Q5	MAX	8.13	5.29	6.72	6.23	8.84	11.21	10.09	8.55	7.57	6.81	8.93	7.28	
362260 DUJ	Q0	MIN	1.05	0.98	1.69	1.18	1.42	0.86	2.15	0.67	0.73	0.73	0.71	2.02	
DUBOIS JEFFERSON CO AP		0.1	1.36	1.40	2.01	1.82	1.72	1.91	1.70	1.61	1.97	1.35	1.35	1.99	35.61
PA	Q1	0.2	1.77	1.75	2.44	2.23	2.25	2.66	2.25	2.16	2.49	1.75	1.80	2.36	38.00
	Q2	0.4	2.43	2.31	3.11	2.89	3.13	3.95	3.16	3.10	3.31	2.39	2.55	2.93	41.35
		0.5	2.76	2.59	3.44	3.21	3.57	4.63	3.62	3.58	3.71	2.72	2.93	3.21	42.85
	Q3	0.6	3.11	2.89	3.79	3.56	4.04	5.37	4.12	4.10	4.15	3.07	3.35	3.50	44.38
	Q4	0.8	4.06	3.66	4.68	4.44	5.31	7.40	5.47	5.51	5.29	4.00	4.46	4.24	48.09
		0.9	4.88	4.32	5.43	5.19	6.41	9.21	6.64	6.74	6.27	4.81	5.44	4.85	51.00
	Q5	MAX	6.49	5.47	6.70	5.18	9.10	14.68	10.32	10.08	8.31	6.61	8.72	7.01	

STATE OF OREGON
DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES
JOHN D. BEAULIEU, STATE GEOLOGIST

Sutherlin-Oakland Urban Area



These maps were produced by the Oregon Department of Geology and Mineral Industries with funding by the State of Oregon and the U.S. Geological Survey (USGS), Department of the Interior, under USGS award #1434-97-GR-03118.

The views and conclusions contained in this document are those of the authors and should not be interpreted as necessarily representing the official policies, either expressed or implied, of the U.S. Government.

STATE OF OREGON
DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES
JOHN D. BEAULIEU, STATE GEOLOGIST

Sutherlin-Oakland Urban Area

Relative Earthquake Hazard Maps
for Selected Urban Areas in Western Oregon

By Ian P. Madin and Zhenming Wang

SUTHERLIN-OAKLAND

Relative Amplification Hazard Map

Hazard zones are based on the degree to which ground shaking from a given earthquake is likely to be amplified

- Highest amplification hazard (UBC soil type E)
- Medium amplification hazard (UBC soil type D)
- Low amplification hazard (UBC soil type C)
- No amplification hazard (UBC soil type B)

See the accompanying text for an explanation of how these zones were defined and what the various levels of hazard mean.

IMPORTANT NOTICE

This map depicts only amplification hazard zones that are based on limited geologic and geophysical data as described in the accompanying report. At any given site in the map area, the maps for other types of hazards may show different hazard levels and need to be taken into consideration along with this map. This map cannot replace site-specific investigations. Some appropriate uses are discussed in the accompanying report.

0 0.5 1 Mile

These maps were produced by the Oregon Department of Geology and Mineral Industries with funding by the State of Oregon and the U.S. Geological Survey (USGS), Department of the Interior, under USGS award #1434-97-GR-03118.

The views and conclusions contained in this document are those of the authors and should not be interpreted as necessarily representing the official policies, either expressed or implied, of the U.S. Government.

STATE OF OREGON
DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES
JOHN D. BEAULIEU, STATE GEOLOGIST

Sutherlin-Oakland Urban Area

Relative Liquefaction Hazard Map

Hazard zones are based on the likelihood that liquefaction will occur in a given earthquake

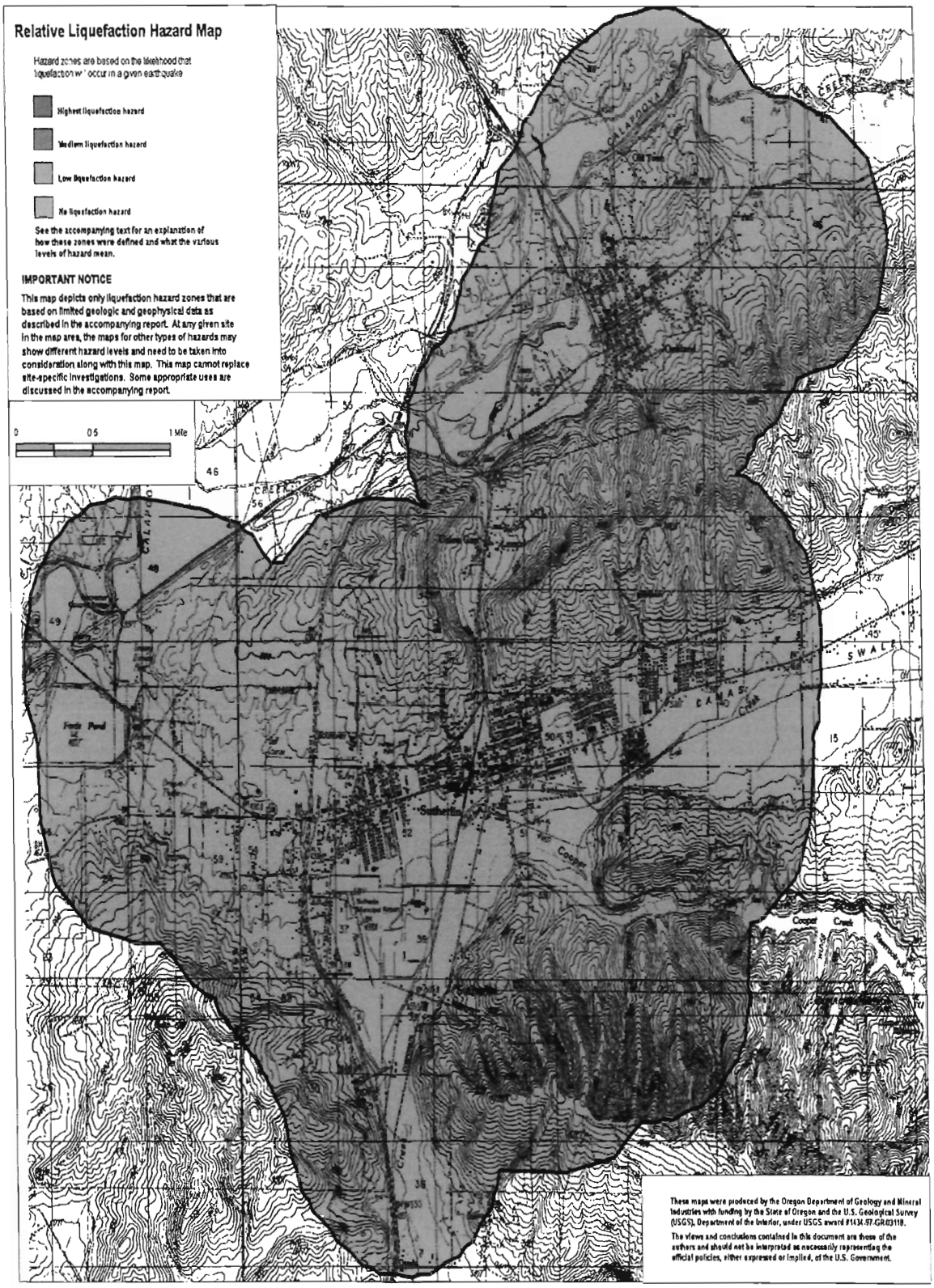
- Highest liquefaction hazard
- Medium liquefaction hazard
- Low liquefaction hazard
- No liquefaction hazard

See the accompanying text for an explanation of how these zones were defined and what the various levels of hazard mean.

IMPORTANT NOTICE

This map depicts only liquefaction hazard zones that are based on limited geologic and geophysical data as described in the accompanying report. At any given site in the map area, the maps for other types of hazards may show different hazard levels and need to be taken into consideration along with this map. This map cannot replace site-specific investigations. Some appropriate uses are discussed in the accompanying report.

0 0.5 1 Mile

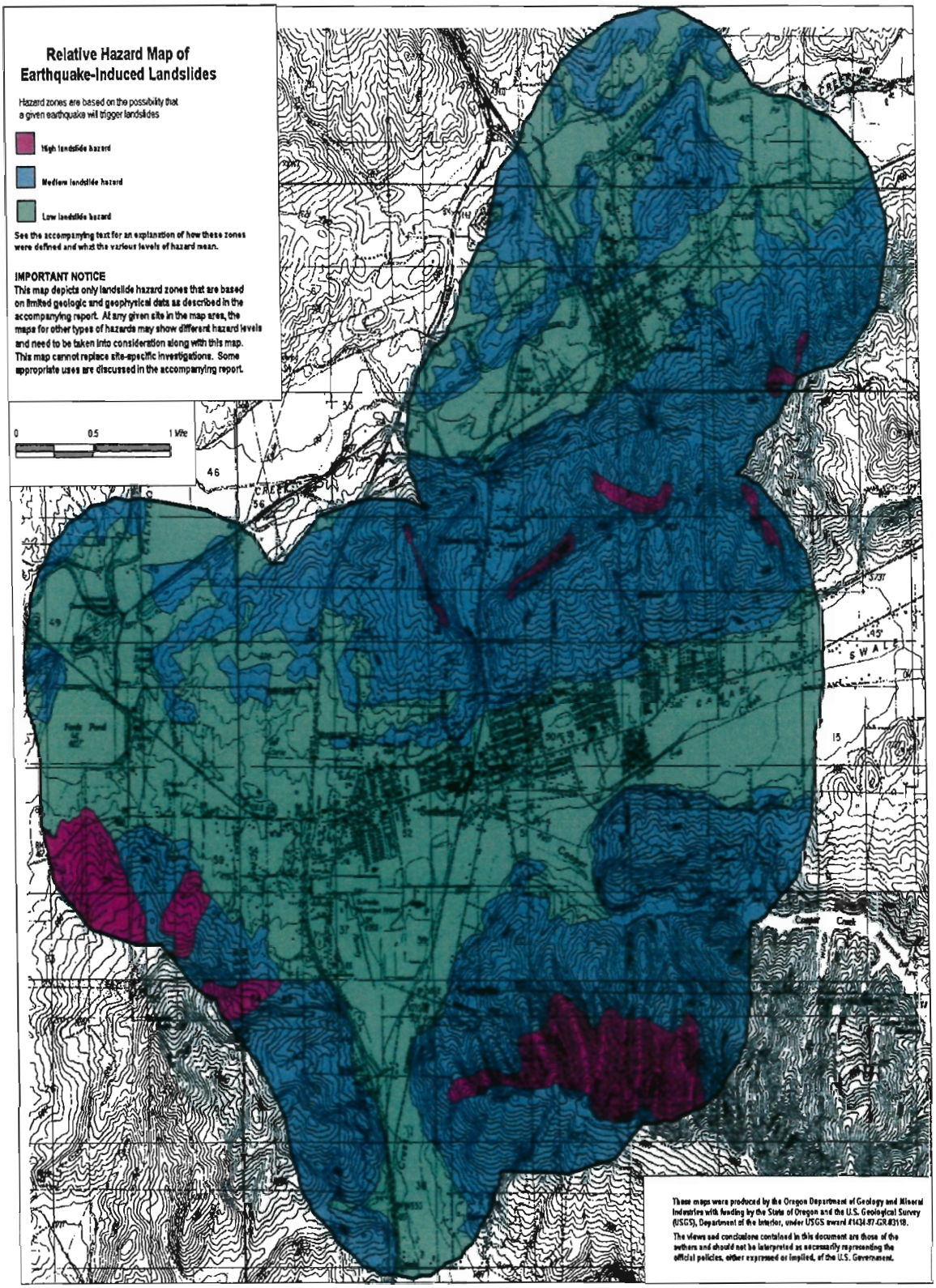


These maps were produced by the Oregon Department of Geology and Mineral Industries with funding by the State of Oregon and the U.S. Geological Survey (USGS), Department of the Interior, under USGS award #1434-97-GR03118.

The views and conclusions contained in this document are those of the authors and should not be interpreted as necessarily representing the official policies, either expressed or implied, of the U.S. Government.

STATE OF OREGON
DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES
JOHN D. BEAULIEU, STATE GEOLOGIST

Sutherlin-Oakland Urban Area



Map Unit Symbol	Map Unit Name
1C	ABEGG VERY GRAVELLY SANDY LOAM, 2 TO 12 PERCENT SLOPES
1D	ABEGG VERY GRAVELLY SANDY LOAM, 12 TO 20 PERCENT SLOPES
1E	ABEGG VERY GRAVELLY SANDY LOAM, 20 TO 30 PERCENT SLOPES
2E	ABSAQUIL-BLACHLY-MCDUFF COMPLEX, 3 TO 30 PERCENT SLOPES
3E	ABSAQUIL-HONEYGROVE-MCDUFF COMPLEX, 3 TO 30 PERCENT SLOPES
4E	ACKER GRAVELLY LOAM, 12 TO 30 PERCENT SLOPES
5F	ACKER-NORLING COMPLEX, 30 TO 60 PERCENT NORTH SLOPES
6F	ACKER-NORLING COMPLEX, 30 TO 60 PERCENT SOUTH SLOPES
7F	ACKER-NORLING COMPLEX, HIGH ELEVATION, 30 TO 60 PERCENT SLOPES
8E	ATRING GRAVELLY LOAM, 12 TO 30 PERCENT SLOPES
8F	ATRING GRAVELLY LOAM, 30 TO 60 PERCENT SLOPES
8G	ATRING GRAVELLY LOAM, 60 TO 90 PERCENT SLOPES
9G	ATRING VERY GRAVELLY LOAM, HIGH ELEVATION, 60 TO 90 PERCENT SLOPES
10E	ATRING-LARMINE COMPLEX, 12 TO 30 PERCENT SLOPES
10F	ATRING-LARMINE COMPLEX, 30 TO 60 PERCENT SLOPES
10G	ATRING-LARMINE COMPLEX, 60 TO 90 PERCENT SLOPES
11F	ATRING-LARMINE-ROCK OUTCROP COMPLEX, 30 TO 60 PERCENT SLOPES
11G	ATRING-LARMINE-ROCK OUTCROP COMPLEX, 60 TO 90 PERCENT SLOPES
12F	ATRING-VERMISA COMPLEX, 30 TO 60 PERCENT NORTH SLOPES
12G	ATRING-VERMISA COMPLEX, 60 TO 90 PERCENT NORTH SLOPES
13G	ATRING-VERMISA COMPLEX, 60 TO 90 PERCENT SOUTH SLOPES
14A	BANNING LOAM, 0 TO 3 PERCENT SLOPES
14C	BANNING LOAM, 3 TO 12 PERCENT SLOPES
14D	BANNING LOAM, 12 TO 20 PERCENT SLOPES
15A	BASHAW CLAY, 0 TO 1 PERCENT SLOPES
16C	BATEMAN SILT LOAM, 3 TO 12 PERCENT SLOPES
16E	BATEMAN SILT LOAM, 12 TO 30 PERCENT SLOPES
16F	BATEMAN SILT LOAM, 30 TO 60 PERCENT SLOPES
17	BEACHES
18E	BEAL LOAM, 12 TO 30 PERCENT SLOPES
19F	BEEKMAN-VERMISA COMPLEX, 30 TO 60 PERCENT SOUTH SLOPES
19G	BEEKMAN-VERMISA COMPLEX, 60 TO 90 PERCENT SOUTH SLOPES
20G	BEEKMAN-VERMISA COMPLEX, 60 TO 90 PERCENT NORTH SLOPES
21C	BELLPINE SILT LOAM, 3 TO 12 PERCENT SLOPES
21E	BELLPINE SILT LOAM, 12 TO 30 PERCENT SLOPES
21F	BELLPINE SILT LOAM, 30 TO 60 PERCENT SLOPES
22C	BELLPINE CLAY LOAM, 3 TO 12 PERCENT SLOPES
22E	BELLPINE CLAY LOAM, 12 TO 30 PERCENT SLOPES
23F	BELLPINE-WINDYGAP COMPLEX, 30 TO 60 PERCENT SLOPES
24C	BICKFORD SILT LOAM, 3 TO 12 PERCENT SLOPES
25C	BIGDUTCH GRAVELLY LOAM, 3 TO 12 PERCENT SLOPES
25E	BIGDUTCH GRAVELLY LOAM, 12 TO 30 PERCENT SLOPES
25F	BIGDUTCH GRAVELLY LOAM, 30 TO 60 PERCENT SLOPES
26E	BLACHLY SILTY CLAY LOAM, 3 TO 30 PERCENT SLOPES
26F	BLACHLY SILTY CLAY LOAM, 30 TO 60 PERCENT SLOPES
27G	BOHANNON-PREACHER-DAMEWOOD COMPLEX, 60 TO 90 PERCENT SLOPES
28A	BRAGTON MUCK, 0 TO 1 PERCENT SLOPES
29A	BRAND SILTY CLAY LOAM, 0 TO 3 PERCENT SLOPES
30D	BUCKEYE LOAM, 2 TO 20 PERCENT SLOPES
30E	BUCKEYE LOAM, 20 TO 30 PERCENT SLOPES
30F	BUCKEYE LOAM, 30 TO 60 PERCENT SLOPES

31E	BUCKSHOT SANDY LOAM, 3 TO 30 PERCENT SLOPES
32F	BUCKSHOT SANDY LOAM, 30 TO 60 PERCENT NORTH SLOPES
33G	BUCKSHOT-STINGER COMPLEX, 60 TO 90 PERCENT NORTH SLOPES
34A	CAMAS-NEWBERG COMPLEX, 0 TO 3 PERCENT SLOPES
35A	CENTRAL-POINT LOAM, 0 TO 3 PERCENT SLOPES
36E	CHAMATE EXTREMELY GRAVELLY LOAM, 3 TO 30 PERCENT SLOPES
36F	CHAMATE EXTREMELY GRAVELLY LOAM, 30 TO 60 PERCENT SLOPES
37A	CHAPMAN-CHEHALIS COMPLEX, 0 TO 3 PERCENT SLOPES
38F	CHIMNEY-ROCK VERY GRAVELLY LOAM, 30 TO 60 PERCENT NORTH SLOPES
38G	CHIMNEY-ROCK VERY GRAVELLY LOAM, 60 TO 90 PERCENT NORTH SLOPES
39F	CHIMNEY-ROCK VERY GRAVELLY LOAM, 30 TO 60 PERCENT SOUTH SLOPES
39G	CHIMNEY-ROCK VERY GRAVELLY LOAM, 60 TO 90 PERCENT SOUTH SLOPES
40F	CLEVESCOVE-SALANDER COMPLEX, 30 TO 70 PERCENT SLOPES
41E	CLIMAX CLAY, 12 TO 30 PERCENT SLOPES
41F	CLIMAX CLAY, 30 TO 60 PERCENT SLOPES
42B	COBURG SILTY CLAY LOAM, 0 TO 5 PERCENT SLOPES
43A	COBURG SILTY CLAY LOAM, FLOODED, 0 TO 3 PERCENT SLOPES
44A	CONSER SILTY CLAY LOAM, 0 TO 3 PERCENT SLOPES
45A	COQUILLE SILT LOAM, 0 TO 1 PERCENT SLOPES
46A	COQUILLE SILT LOAM, PROTECTED, 0 TO 1 PERCENT SLOPES
47E	CRATER LAKE SANDY LOAM, 3 TO 30 PERCENT SLOPES
47F	CRATER LAKE SANDY LOAM, 30 TO 60 PERCENT SLOPES
48C	CURTIN CLAY, 3 TO 12 PERCENT SLOPES
48D	CURTIN CLAY, 12 TO 20 PERCENT SLOPES
49F	DAMEWOOD-BOHANNON-UMPCOOS COMPLEX, 30 TO 60 PERCENT SLOPES
49G	DAMEWOOD-BOHANNON-UMPCOOS COMPLEX, 60 TO 90 PERCENT SLOPES
50E	DARBY SILTY CLAY LOAM, 12 TO 30 PERCENT SLOPES
50F	DARBY SILTY CLAY LOAM, 30 TO 60 PERCENT SLOPES
51C	DEBENGER-BRADER COMPLEX, 2 TO 12 PERCENT SLOPES
51E	DEBENGER-BRADER COMPLEX, 12 TO 30 PERCENT SLOPES
51F	DEBENGER-BRADER COMPLEX, 30 TO 60 PERCENT SLOPES
52G	DICECREEK-BELLPINE-WINDYGAP COMPLEX, 30 TO 60 PERCENT SOUTH SLOPES
53E	DICKERSON LOAM, 3 TO 30 PERCENT SLOPES
53G	DICKERSON LOAM, 30 TO 90 PERCENT SLOPES
54E	DICKERSON-ROCK OUTCROP COMPLEX, 3 TO 30 PERCENT SLOPES
54G	DICKERSON-ROCK OUTCROP COMPLEX, 30 TO 90 PERCENT SLOPES
55G	DICKERSON-ROCK OUTCROP-CHIMNEYROCK COMPLEX, 60 TO 90 PERCENT NORTH SLOPES
56G	DICKERSON-ROCK OUTCROP-CHIMNEYROCK COMPLEX, 60 TO 90 PERCENT SOUTH SLOPES
57E	DIGGER-BOHANNON COMPLEX, 3 TO 30 PERCENT SLOPES
57F	DIGGER-BOHANNON COMPLEX, 30 TO 60 PERCENT SLOPES
58G	DIGGER-BOHANNON-UMPCOOS COMPLEX, 60 TO 90 PERCENT SLOPES
59G	DIGGER-PREACHER COMPLEX, 60 TO 90 PERCENT SLOPES
60F	DIGGER-UMPCOOS-ROCK OUTCROP COMPLEX, 30 TO 60 PERCENT SLOPES
60G	DIGGER-UMPCOOS-ROCK OUTCROP COMPLEX, 60 TO 90 PERCENT SLOPES
61C	DIXONVILLE SILTY CLAY LOAM, 3 TO 12 PERCENT SLOPES
61E	DIXONVILLE SILTY CLAY LOAM, 12 TO 30 PERCENT SLOPES
62F	DIXONVILLE SILTY CLAY LOAM, 30 TO 60 PERCENT NORTH SLOPES
63F	DIXONVILLE SILTY CLAY LOAM, 30 TO 60 PERCENT SOUTH SLOPES
64F	DOMPIER-ZING-BEAL COMPLEX, 30 TO 60 PERCENT SLOPES
65F	DUBAKELLA VERY STONY CLAY LOAM, 30 TO 60 PERCENT NORTH SLOPES
66F	DUBAKELLA VERY STONY CLAY LOAM, 30 TO 60 PERCENT SOUTH SLOPES
67E	DUBAKELLA-PEARSOLL COMPLEX, 5 TO 30 PERCENT SLOPES

68F	DUBAKELLA-PEARSOLL COMPLEX, 30 TO 70 PERCENT NORTH SLOPES
69C	DUMONT GRAVELLY LOAM, 2 TO 12 PERCENT SLOPES
69E	DUMONT GRAVELLY LOAM, 12 TO 30 PERCENT SLOPES
70F	DUMONT GRAVELLY LOAM, 30 TO 60 PERCENT NORTH SLOPES
71F	DUMONT GRAVELLY LOAM, 30 TO 60 PERCENT SOUTH SLOPES
72E	DUMONT GRAVELLY LOAM, GRANITIC SUBSTRATUM, 3 TO 30 PERCENT SLOPES
73E	DUMONT-ZING COMPLEX, 2 TO 30 PERCENT SLOPES
74	DUNELAND
75C	DUPEE SILTY CLAY LOAM, 3 TO 12 PERCENT SLOPES
75E	DUPEE SILTY CLAY LOAM, 12 TO 30 PERCENT SLOPES
76E	EDENBOWER CLAY, 3 TO 30 PERCENT SLOPES
77D	EIGHTLAR VERY GRAVELLY SILTY CLAY LOAM, 5 TO 20 PERCENT SLOPES
78A	EVANS LOAM, 0 TO 3 PERCENT SLOPES
79E	FERNHAVEN GRAVELLY LOAM, 3 TO 30 PERCENT SLOPES
79F	FERNHAVEN GRAVELLY LOAM, 30 TO 50 PERCENT SLOPES
79G	FERNHAVEN GRAVELLY LOAM, 50 TO 75 PERCENT SLOPES
80E	FERNHAVEN-DIGGER COMPLEX, 3 TO 30 PERCENT SLOPES
80F	FERNHAVEN-DIGGER COMPLEX, 30 TO 60 PERCENT SLOPES
81A	FOEHLIN GRAVELLY LOAM, 0 TO 3 PERCENT SLOPES
81C	FOEHLIN GRAVELLY LOAM, 3 TO 12 PERCENT SLOPES
82C	FORDICE VERY COBBLY LOAM, 0 TO 12 PERCENT SLOPES
83A	GLIDE FINE SANDY LOAM, 0 TO 3 PERCENT SLOPES
84E	GRAVECREEK GRAVELLY LOAM, 12 TO 30 PERCENT SLOPES
85F	GRAVECREEK GRAVELLY LOAM, 30 TO 60 PERCENT NORTH SLOPES
85G	GRAVECREEK GRAVELLY LOAM, 60 TO 80 PERCENT NORTH SLOPES
86F	GRAVECREEK GRAVELLY LOAM, 30 TO 60 PERCENT SOUTH SLOPES
86G	GRAVECREEK GRAVELLY LOAM, 60 TO 80 PERCENT SOUTH SLOPES
87E	GREENGULCH-CEDARGROVE COMPLEX, 3 TO 30 PERCENT SLOPES
88F	GREENGULCH-CEDARGROVE COMPLEX, 30 TO 60 PERCENT NORTH SLOPES
89F	GREENGULCH-CEDARGROVE COMPLEX, 30 TO 60 PERCENT SOUTH SLOPES
90E	GUSTIN-ORFORD COMPLEX, 3 TO 30 PERCENT SLOPES
91G	HARRINGTON-KILCHIS-ROCK OUTCROP COMPLEX, 60 TO 100 PERCENT SLOPES
92A	HECETA FINE SAND, 0 TO 2 PERCENT SLOPES
93C	HECETA-WALDPORT COMPLEX, 0 TO 12 PERCENT SLOPES
94E	HEMCROSS-KLISTAN COMPLEX, 3 TO 30 PERCENT SLOPES
95F	HILLTISH VERY GRAVELLY SANDY LOAM, 30 TO 60 PERCENT NORTH SLOPES
95G	HILLTISH VERY GRAVELLY SANDY LOAM, 60 TO 90 PERCENT NORTH SLOPES
96F	HILLTISH VERY GRAVELLY SANDY LOAM, 30 TO 60 PERCENT SOUTH SLOPES
96G	HILLTISH VERY GRAVELLY SANDY LOAM, 60 TO 90 PERCENT SOUTH SLOPES
97E	HONEYGROVE GRAVELLY CLAY LOAM, 3 TO 30 PERCENT SLOPES
97F	HONEYGROVE GRAVELLY CLAY LOAM, 30 TO 60 PERCENT SLOPES
98E	HONEYGROVE-GUSTIN COMPLEX, 3 TO 30 PERCENT SLOPES
99E	HONEYGROVE-PEAVINE COMPLEX, 3 TO 30 PERCENT SLOPES
99F	HONEYGROVE-PEAVINE COMPLEX, 30 TO 60 PERCENT SLOPES
100E	HONEYGROVE-SHIVIGNY COMPLEX, 3 TO 30 PERCENT SLOPES
101F	HONEYGROVE-SHIVIGNY-GUSTIN COMPLEX, 30 TO 60 PERCENT NORTH SLOPES
102F	HONEYGROVE-SHIVIGNY-GUSTIN COMPLEX, 30 TO 60 PERCENT SOUTH SLOPES
103G	HUMMINGTON VERY GRAVELLY LOAM, 60 TO 90 PERCENT NORTH SLOPES
104F	ILLAHEE-MELLOWMOON-SCAREDMAN COMPLEX, 30 TO 60 PERCENT NORTH SLOPES
105F	ILLAHEE-MELLOWMOON-SCAREDMAN COMPLEX, 30 TO 60 PERCENT SOUTH SLOPES
106G	ILLAHEE-ROCK OUTCROP COMPLEX, 60 TO 90 PERCENT NORTH SLOPES
107G	ILLAHEE-SCAREDMAN COMPLEX, 60 TO 90 PERCENT NORTH SLOPES

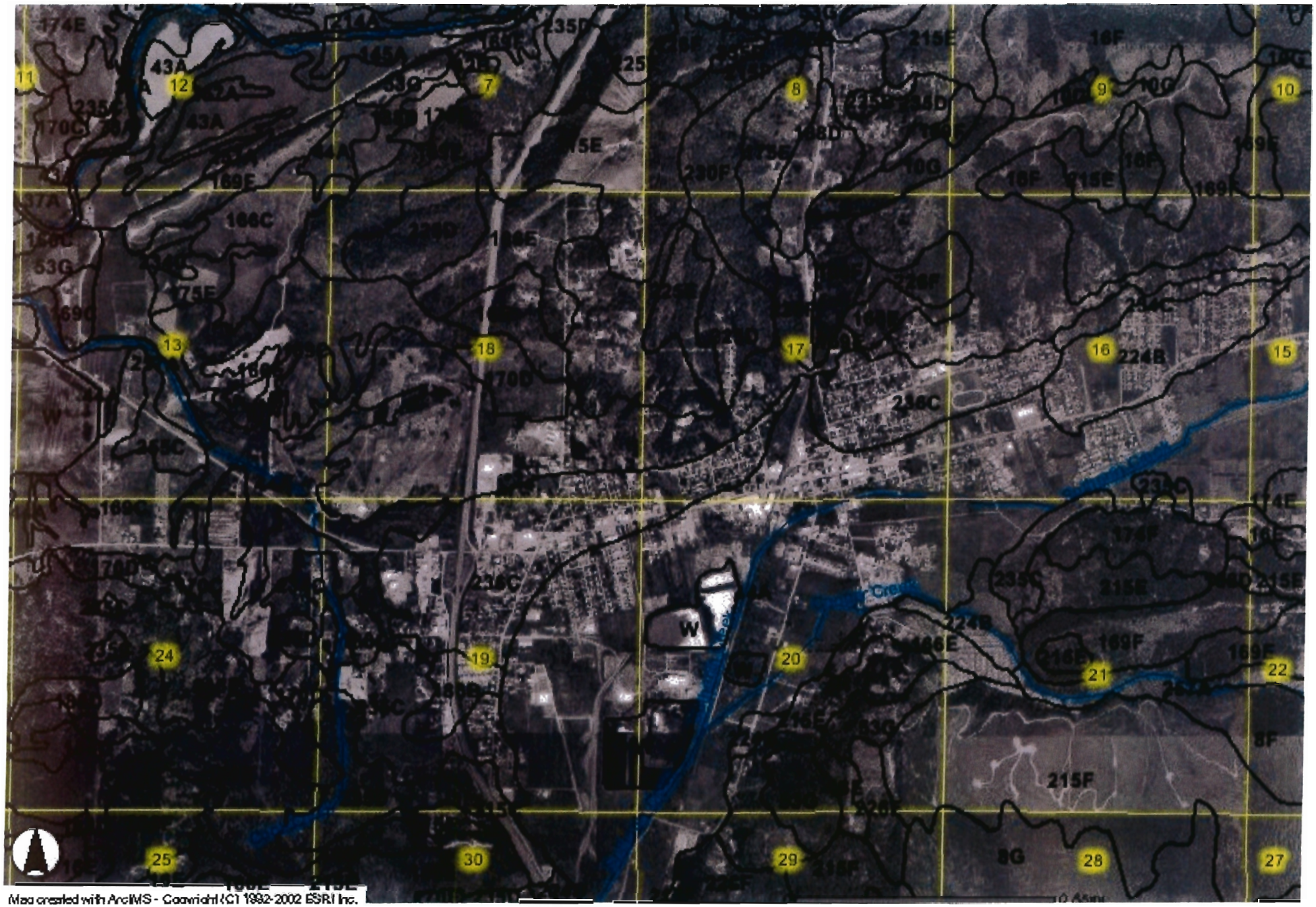
108G	ILLAHEE-SCAREDMAN COMPLEX, 60 TO 90 PERCENT SOUTH SLOPES
109E	JAYAR VERY GRAVELLY LOAM, 12 TO 30 PERCENT SLOPES
110F	JAYAR VERY GRAVELLY LOAM, 30 TO 70 PERCENT NORTH SLOPES
111F	JAYAR VERY GRAVELLY LOAM, 30 TO 70 PERCENT SOUTH SLOPES
112A	JIMBO-HAFLINGER COMPLEX, 0 TO 3 PERCENT SLOPES
113C	JORY SILTY CLAY LOAM, 2 TO 12 PERCENT SLOPES
113D	JORY SILTY CLAY LOAM, 12 TO 20 PERCENT SLOPES
113E	JORY SILTY CLAY LOAM, 20 TO 30 PERCENT SLOPES
113F	JORY SILTY CLAY LOAM, 30 TO 60 PERCENT SLOPES
114F	JORY-RITNER COMPLEX, 30 TO 60 PERCENT SLOPES
115C	JOSEPHINE GRAVELLY LOAM, 3 TO 12 PERCENT SLOPES
115E	JOSEPHINE GRAVELLY LOAM, 12 TO 30 PERCENT SLOPES
116F	JOSEPHINE-SPEAKER COMPLEX, 30 TO 60 PERCENT NORTH SLOPES
117F	JOSEPHINE-SPEAKER COMPLEX, 30 TO 60 PERCENT SOUTH SLOPES
118E	KANID VERY GRAVELLY LOAM, 12 TO 30 PERCENT SLOPES
118F	KANID VERY GRAVELLY LOAM, 30 TO 70 PERCENT SLOPES
119F	KANID-ATRING COMPLEX, 30 TO 60 PERCENT SOUTH SLOPES
119G	KANID-ATRING COMPLEX, 60 TO 90 PERCENT SOUTH SLOPES
120G	KANID-ATRING COMPLEX, 60 TO 90 PERCENT NORTH SLOPES
121G	KILCHIS-HARSLOW-ROCK OUTCROP COMPLEX, 60 TO 100 PERCENT SOUTH SLOPES
122E	KINNEY-KLICKITAT COMPLEX, 3 TO 30 PERCENT SLOPES
123A	KIRKENDALL-NEKOMA COMPLEX, 0 TO 3 PERCENT SLOPES
124F	KLICKITAT-HARRINGTON COMPLEX, 30 TO 60 PERCENT NORTH SLOPES
124G	KLICKITAT-HARRINGTON COMPLEX, 60 TO 90 PERCENT NORTH SLOPES
125G	KLICKITAT-HARRINGTON COMPLEX, 60 TO 90 PERCENT SOUTH SLOPES
126F	KLICKITAT-KINNEY COMPLEX, 30 TO 60 PERCENT SLOPES
127F	KLISTAN-HEMCROSS COMPLEX, 30 TO 60 PERCENT SLOPES
128F	LADERLY VERY GRAVELLY LOAM, 30 TO 60 PERCENT NORTH SLOPES
129G	LADERLY-ROMANOSE COMPLEX, 60 TO 90 PERCENT NORTH SLOPES
130E	LEMPIRA GRAVELLY LOAM, 3 TO 30 PERCENT SLOPES
131F	LEMPIRA-LLAHEE COMPLEX, 30 TO 60 PERCENT NORTH SLOPES
132E	LEOPOLD CLAY LOAM, 3 TO 30 PERCENT SLOPES
132F	LEOPOLD CLAY LOAM, 30 TO 60 PERCENT SLOPES
133E	LETTIA GRAVELLY LOAM, 3 TO 30 PERCENT SLOPES
134F	LETTIA GRAVELLY LOAM, 30 TO 60 PERCENT NORTH SLOPES
135F	LETTIA GRAVELLY LOAM, 30 TO 60 PERCENT SOUTH SLOPES
136E	LETTIA-BEAL-ZING COMPLEX, 3 TO 30 PERCENT SLOPES
137F	LETTIA-BEAL-ZING COMPLEX, 30 TO 60 PERCENT NORTH SLOPES
138F	LETTIA-BEAL-ZING COMPLEX, 30 TO 60 PERCENT SOUTH SLOPES
139F	LETTIA-TISHAR COMPLEX, 30 TO 70 PERCENT NORTH SLOPES
140F	LETTIA-TISHAR COMPLEX, 30 TO 70 PERCENT SOUTH SLOPES
141C	LINT SILT LOAM, 0 TO 12 PERCENT SLOPES
141D	LINT SILT LOAM, 12 TO 20 PERCENT SLOPES
142G	LITTLESAND-NONPAREIL-ROCK OUTCROP COMPLEX, 60 TO 90 PERCENT SOUTH SLOPES
143G	LITTLESAND-ROSEHAVEN-ATRING COMPLEX, 60 TO 90 PERCENT SLOPES
144A	MALABON SILTY CLAY LOAM, 0 TO 3 PERCENT SLOPES
145A	MALABON SILTY CLAY LOAM, FLOODED, 0 TO 3 PERCENT SLOPES
146E	MCCOMAS VERY GRAVELLY LOAM, 3 TO 30 PERCENT SLOPES
147F	MCDUFF-ABSAQUIL-BLACHLY COMPLEX, 30 TO 60 PERCENT SLOPES
148F	MCDUFF-ABSAQUIL-HONEYGROVE COMPLEX, 30 TO 60 PERCENT SLOPES
149E	MCMULLIN-RESTON COMPLEX, 3 TO 30 PERCENT SLOPES
149F	MCMULLIN-RESTON COMPLEX, 30 TO 75 PERCENT SLOPES

150F	MCMULLIN-RESTON-ROCK OUTCROP COMPLEX, 30 TO 75 PERCENT SLOPES
151A	MCNAB CLAY LOAM, 0 TO 3 PERCENT SLOPES
152E	MCNAB-WINDYGAP COMPLEX, 3 TO 30 PERCENT SLOPES
153D	MEDA LOAM, 2 TO 20 PERCENT SLOPES
154B	MEDFORD CLAY LOAM, 0 TO 7 PERCENT SLOPES
154C	MEDFORD CLAY LOAM, 7 TO 15 PERCENT SLOPES
155E	MELLOWMOON-ILLAHEE COMPLEX, 3 TO 30 PERCENT SLOPES
156G	MILLICOMA-REEDSPORT COMPLEX, 60 TO 90 PERCENT SLOPES
157A	NATAL CLAY LOAM, 0 TO 3 PERCENT SLOPES
158A	NATROY CLAY, 0 TO 2 PERCENT SLOPES
159C	NEKIA SILTY CLAY LOAM, 2 TO 12 PERCENT SLOPES
159D	NEKIA SILTY CLAY LOAM, 12 TO 20 PERCENT SLOPES
159E	NEKIA SILTY CLAY LOAM, 20 TO 30 PERCENT SLOPES
160E	NEKIA-JORY COMPLEX, 30 TO 60 PERCENT SLOPES
161A	NEKOMA-GARDINER COMPLEX, 0 TO 3 PERCENT SLOPES
162A	NESTUCCA SILT LOAM, 0 TO 3 PERCENT SLOPES
163C	NETARTS FINE SAND, 2 TO 12 PERCENT SLOPES
163E	NETARTS FINE SAND, 12 TO 30 PERCENT SLOPES
164A	NEWBERG FINE SANDY LOAM, 0 TO 3 PERCENT SLOPES
165A	NEWBERG LOAMY SAND, 0 TO 3 PERCENT SLOPES
166C	NONPAREIL LOAM, 3 TO 12 PERCENT SLOPES
166E	NONPAREIL LOAM, 12 TO 30 PERCENT SLOPES
167G	NONPAREIL LOAM, 60 TO 90 PERCENT NORTH SLOPES
168G	NONPAREIL LOAM, 60 TO 90 PERCENT SOUTH SLOPES
169C	NONPAREIL-OAKLAND COMPLEX, 3 TO 12 PERCENT SLOPES
169E	NONPAREIL-OAKLAND COMPLEX, 12 TO 30 PERCENT SLOPES
169F	NONPAREIL-OAKLAND COMPLEX, 30 TO 60 PERCENT SLOPES
170C	OAKLAND SILT LOAM, 3 TO 12 PERCENT SLOPES
170D	OAKLAND SILT LOAM, 12 TO 20 PERCENT SLOPES
170E	OAKLAND SILT LOAM, 20 TO 30 PERCENT SLOPES
171F	OAKLAND SILT LOAM, 30 TO 60 PERCENT NORTH SLOPES
172F	OAKLAND SILT LOAM, 30 TO 60 PERCENT SOUTH SLOPES
173E	OAKLAND-DUPEE COMPLEX, 12 TO 30 PERCENT SLOPES
174E	OAKLAND-NONPAREIL-SUTHERLIN COMPLEX, 12 TO 30 PERCENT SLOPES
174F	OAKLAND-NONPAREIL-SUTHERLIN COMPLEX, 30 TO 60 PERCENT SLOPES
175E	OAKLAND-SUTHERLIN COMPLEX, 12 TO 30 PERCENT SLOPES
176F	ONEONTA-HUMMINGTON COMPLEX, 30 TO 60 PERCENT NORTH SLOPES
177F	ONEONTA-HUMMINGTON COMPLEX, 30 TO 60 PERCENT SOUTH SLOPES
178E	ONEONTA-KEEL COMPLEX, 3 TO 30 PERCENT SLOPES
179E	ORFORD GRAVELLY SILT LOAM, 3 TO 30 PERCENT SLOPES
179F	ORFORD GRAVELLY SILT LOAM, 30 TO 60 PERCENT SLOPES
180E	ORFORD GRAVELLY LOAM, 3 TO 30 PERCENT SLOPES
180F	ORFORD GRAVELLY LOAM, 30 TO 60 PERCENT SLOPES
181F	ORFORD-GUSTIN COMPLEX, 30 TO 60 PERCENT SLOPES
182F	ORFORD-MCDUFF COMPLEX, 30 TO 60 PERCENT SLOPES
183B	PACKARD GRAVELLY LOAM, 0 TO 5 PERCENT SLOPES
184A	PACKARD GRAVELLY LOAM, FLOODED, 0 TO 3 PERCENT SLOPES
185D	PANTHER SILTY CLAY LOAM, 4 TO 20 PERCENT SLOPES
186F	PEARSOLL-DUBAKELLA COMPLEX, 30 TO 70 PERCENT SOUTH SLOPES
187E	PEEL CLAY LOAM, 3 TO 30 PERCENT SLOPES
188D	PENGRA SILT LOAM, 2 TO 20 PERCENT SLOPES
188E	PENGRA SILT LOAM, 20 TO 30 PERCENT SLOPES

189E	PHILOMATH-DIXONVILLE COMPLEX, 3 TO 30 PERCENT SLOPES
189F	PHILOMATH-DIXONVILLE COMPLEX, 30 TO 70 PERCENT SLOPES
190E	PHILOMATH-EDENBOWER COMPLEX, 12 TO 30 PERCENT SLOPES
190F	PHILOMATH-EDENBOWER COMPLEX, 30 TO 60 PERCENT SLOPES
191	PITS
192E	POLLARD GRAVELLY LOAM, 3 TO 30 PERCENT SLOPES
193F	POLLARD GRAVELLY LOAM, 30 TO 60 PERCENT NORTH SLOPES
194F	POLLARD GRAVELLY LOAM, 30 TO 60 PERCENT SOUTH SLOPES
195E	PREACHER LOAM, 0 TO 30 PERCENT SLOPES
195F	PREACHER LOAM, 30 TO 50 PERCENT SLOPES
195G	PREACHER LOAM, 50 TO 75 PERCENT SLOPES
196E	PREACHER-BLACHLY COMPLEX, 12 TO 30 PERCENT SLOPES
197E	PREACHER-BOHANNON COMPLEX, 3 TO 30 PERCENT SLOPES
197F	PREACHER-BOHANNON COMPLEX, 30 TO 60 PERCENT SLOPES
198F	PREACHER-BOHANNON-BLACHLY COMPLEX, 30 TO 70 PERCENT SLOPES
199G	PREACHER-BOHANNON-DIGGER COMPLEX, 60 TO 90 PERCENT SLOPES
200F	PREACHER-BOHANNON-XANADU COMPLEX, 30 TO 60 PERCENT SLOPES
201A	QUOSATANA SILT LOAM, 0 TO 3 PERCENT SLOPES
202B	REDBELL SILT LOAM, 0 TO 5 PERCENT SLOPES
203F	REEDSPORT-MILLICOMA COMPLEX, 30 TO 60 PERCENT SLOPES
204G	REMOTE-DIGGER COMPLEX, 60 TO 90 PERCENT SLOPES
205E	RITNER GRAVELLY SILTY CLAY LOAM, 12 TO 30 PERCENT SLOPES
206F	RITNER GRAVELLY SILTY CLAY LOAM, 30 TO 60 PERCENT NORTH SLOPES
207G	RITNER GRAVELLY SILTY CLAY LOAM, 60 TO 90 PERCENT NORTH SLOPES
208G	RITNER-JORY COMPLEX, 60 TO 90 PERCENT SLOPES
209F	RITNER-SAHAPTIN COMPLEX, 30 TO 60 PERCENT SOUTH SLOPES
209G	RITNER-SAHAPTIN COMPLEX, 60 TO 90 PERCENT SOUTH SLOPES
210	RIVERWASH
211	ROCK OUTCROP
212G	ROCK OUTCROP-UMPCOOS COMPLEX, 60 TO 110 PERCENT SLOPES
213G	ROMANOSE- LADERLY COMPLEX, 60 TO 90 PERCENT SOUTH SLOPES
214A	ROSEBURG LOAM, 0 TO 3 PERCENT SLOPES
215C	ROSEHAVEN LOAM, 3 TO 12 PERCENT SLOPES
215E	ROSEHAVEN LOAM, 12 TO 30 PERCENT SLOPES
215F	ROSEHAVEN LOAM, 30 TO 60 PERCENT SLOPES
216E	ROSEHAVEN-ATRING COMPLEX, 12 TO 30 PERCENT SLOPES
216F	ROSEHAVEN-ATRING COMPLEX, 30 TO 60 PERCENT SLOPES
217E	SALANDER SILT LOAM, 12 TO 30 PERCENT SLOPES
217F	SALANDER SILT LOAM, 30 TO 60 PERCENT SLOPES
218G	SCAREDMAN-LIMPY-ROCK OUTCROP COMPLEX, 60 TO 100 PERCENT SOUTH SLOPES
219E	SHARPSHOOTER LOAM, 3 TO 30 PERCENT SLOPES
220F	SHARPSHOOTER LOAM, 30 TO 60 PERCENT NORTH SLOPES
220G	SHARPSHOOTER LOAM, 60 TO 90 PERCENT NORTH SLOPES
221F	SHARPSHOOTER LOAM, 30 TO 60 PERCENT SOUTH SLOPES
221G	SHARPSHOOTER LOAM, 60 TO 90 PERCENT SOUTH SLOPES
222F	SHIVIGNY-HONEYGROVE COMPLEX, 30 TO 60 PERCENT NORTH SLOPES
222G	SHIVIGNY-HONEYGROVE COMPLEX, 60 TO 90 PERCENT NORTH SLOPES
223F	SHIVIGNY-HONEYGROVE COMPLEX, 30 TO 60 PERCENT SOUTH SLOPES
224B	SIBOLD FINE SANDY LOAM, 0 TO 5 PERCENT SLOPES
225D	SPEAKER LOAM, 2 TO 20 PERCENT SLOPES
225E	SPEAKER LOAM, 20 TO 30 PERCENT SLOPES
226F	SPEAKER LOAM, 30 TO 60 PERCENT NORTH SLOPES

227F	SPEAKER LOAM, 30 TO 60 PERCENT SOUTH SLOPES
228G	SPEAKER-BEEKMAN-JOSEPHINE COMPLEX, 60 TO 90 PERCENT NORTH SLOPES
229G	SPEAKER-BEEKMAN-NONPAREIL COMPLEX, 60 TO 90 PERCENT SOUTH SLOPES
230E	SPEAKER-NONPAREIL COMPLEX, 3 TO 30 PERCENT SLOPES
230F	SPEAKER-NONPAREIL COMPLEX, 30 TO 60 PERCENT SLOPES
231G	STACKYARDS EXTREMELY GRAVELLY LOAM, 60 TO 90 PERCENT NORTH SLOPES
232G	STEINMETZ-SITKUM COMPLEX, 60 TO 90 PERCENT NORTH SLOPES
233G	STEINMETZ-SITKUM COMPLEX, 60 TO 90 PERCENT SOUTH SLOPES
234C	STOCKEL FINE SANDY LOAM, 3 TO 12 PERCENT SLOPES
235C	SUTHERLIN SILT LOAM, 3 TO 12 PERCENT SLOPES
235D	SUTHERLIN SILT LOAM, 12 TO 20 PERCENT SLOPES
235E	SUTHERLIN SILT LOAM, 20 TO 30 PERCENT SLOPES
235F	SUTHERLIN SILT LOAM, 30 TO 50 PERCENT SLOPES
236C	SUTHERLIN-OAKLAND COMPLEX, 3 TO 12 PERCENT SLOPES
237E	SVENSEN LOAM, 3 TO 30 PERCENT SLOPES
237F	SVENSEN LOAM, 30 TO 60 PERCENT SLOPES
238F	SVENSEN-MILLICOMA-REEDSPORT COMPLEX, 35 TO 75 PERCENT SLOPES
239E	SWEETBRIAR SILTY CLAY LOAM, 3 TO 30 PERCENT SLOPES
240F	SWEETBRIAR SILTY CLAY LOAM, 30 TO 60 PERCENT NORTH SLOPES
241F	SWEETBRIAR SILTY CLAY LOAM, 30 TO 60 PERCENT SOUTH SLOPES
242F	TEMPLETON-MILLICOMA COMPLEX, 30 TO 50 PERCENT SLOPES
242G	TEMPLETON-MILLICOMA COMPLEX, 50 TO 75 PERCENT SLOPES
243G	TETHRICK-SISKIYOU COMPLEX, 60 TO 90 PERCENT NORTH SLOPES
244G	TETHRICK-SISKIYOU COMPLEX, 60 TO 90 PERCENT SOUTH SLOPES
245E	THISTLEBURN CLAY LOAM, 3 TO 30 PERCENT SLOPES
246E	THISTLEBURN-ILLAHEE-TELEMON COMPLEX, 3 TO 30 PERCENT SLOPES
246F	THISTLEBURN-ILLAHEE-TELEMON COMPLEX, 30 TO 60 PERCENT SLOPES
247F	THREEFORKS LOAM, 30 TO 60 PERCENT NORTH SLOPES
247G	THREEFORKS LOAM, 60 TO 90 PERCENT NORTH SLOPES
248G	THREEFORKS LOAM, 60 TO 90 PERCENT SOUTH SLOPES
249E	TISHAR-JORY COMPLEX, 3 TO 30 PERCENT SLOPES
249F	TISHAR-JORY COMPLEX, 30 TO 60 PERCENT SLOPES
250E	TISHAR-MCGINNIS COMPLEX, 3 TO 30 PERCENT SLOPES
251F	TISHAR-MCGINNIS COMPLEX, 30 TO 60 PERCENT NORTH SLOPES
251G	TISHAR-MCGINNIS COMPLEX, 60 TO 80 PERCENT NORTH SLOPES
252F	TISHAR-MCGINNIS COMPLEX, 30 TO 60 PERCENT SOUTH SLOPES
252G	TISHAR-MCGINNIS COMPLEX, 60 TO 80 PERCENT SOUTH SLOPES
253F	UMPCOOS-ROCK OUTCROP-DAMEWOOD COMPLEX, 30 TO 60 PERCENT SLOPES
253G	UMPCOOS-ROCK OUTCROP-DAMEWOOD COMPLEX, 60 TO 90 PERCENT SLOPES
254G	VENA-ROCK OUTCROP COMPLEX, 60 TO 100 PERCENT SLOPES
255C	VENETA LOAM, 0 TO 12 PERCENT SLOPES
255D	VENETA LOAM, 12 TO 20 PERCENT SLOPES
256G	VERMISA-ROCK OUTCROP COMPLEX, 60 TO 100 PERCENT SOUTH SLOPES
257A	WALDO SILTY CLAY LOAM, 0 TO 3 PERCENT SLOPES
258C	WALDPORT FINE SAND, 0 TO 12 PERCENT SLOPES
258E	WALDPORT FINE SAND, 12 TO 30 PERCENT SLOPES
258F	WALDPORT FINE SAND, 30 TO 70 PERCENT SLOPES
259E	WALDPORT FINE SAND, THIN SURFACE, 0 TO 30 PERCENT SLOPES
260A	WASSON LOAM, 0 TO 3 PERCENT SLOPES
261A	WILLANCH FINE SANDY LOAM, 0 TO 3 PERCENT SLOPES
262C	WINDYGAP SILT LOAM, 2 TO 12 PERCENT SLOPES
262E	WINDYGAP SILT LOAM, 12 TO 30 PERCENT SLOPES

262F	WINDYGAP SILT LOAM, 30 TO 60 PERCENT SLOPES
263C	WINDYGAP CLAY LOAM, 2 TO 12 PERCENT SLOPES
263E	WINDYGAP CLAY LOAM, 12 TO 30 PERCENT SLOPES
264E	WINDYGAP-BELLPINE COMPLEX, 12 TO 30 PERCENT SLOPES
265F	WINDYGAP-BELLPINE COMPLEX, 30 TO 60 PERCENT NORTH SLOPES
266F	WINDYGAP-BELLPINE COMPLEX, 30 TO 60 PERCENT SOUTH SLOPES
267C	WINTLEY SILT LOAM, 0 TO 12 PERCENT SLOPES
268E	WOLFPEAK SANDY LOAM, 3 TO 30 PERCENT SLOPES
269F	WOLFPEAK SANDY LOAM, 30 TO 60 PERCENT NORTH SLOPES
270F	WOLFPEAK SANDY LOAM, 30 TO 60 PERCENT SOUTH SLOPES
271E	WOLFPEAK-BEAL-ZING COMPLEX, 3 TO 30 PERCENT SLOPES
272E	XANADU GRAVELLY LOAM, 3 TO 30 PERCENT SLOPES
272F	XANADU GRAVELLY LOAM, 30 TO 60 PERCENT SLOPES
273G	XERORTHENTS-ROCK OUTCROP COMPLEX, 30 TO 80 PERCENT SLOPES
274A	YACHATS FINE SANDY LOAM, 0 TO 3 PERCENT SLOPES
275D	YONCALLA SILTY CLAY LOAM, 2 TO 20 PERCENT SLOPES
275E	YONCALLA SILTY CLAY LOAM, 20 TO 30 PERCENT SLOPES
276E	ZALEA- PYRADY COMPLEX, 15 TO 30 PERCENT SLOPES
277E	ZING LOAM, 0 TO 30 PERCENT SLOPES
278E	ZING-LETTIA COMPLEX, 3 TO 30 PERCENT SLOPES
279E	ZING-SWEETBRIAR COMPLEX, 3 TO 30 PERCENT SLOPES
279F	ZING-SWEETBRIAR COMPLEX, 30 TO 60 PERCENT SLOPES
DA	DENIED ACCESS
W	WATER



Map created with ArcIMS - Copyright (C) 1992-2002 ESRI Inc.


PEMB
PSSB
PFOC
Wetland Types
Estuarine and Marine Deepwater
Estuarine and Marine Wetland
Freshwater Emergent Wetland
Freshwater Forested/Shrub Wetland
Freshwater Pond
Lake
Other
Riverine
Status Map
Digital (vector data)
Scan (raster data)
Non-Digital (hardcopy only)
No Data
Image State of Oregon
© 2009 Google
© 2009 Europa Technologies

FIGURE 3.2.1B

FIGURE 3.2.1D

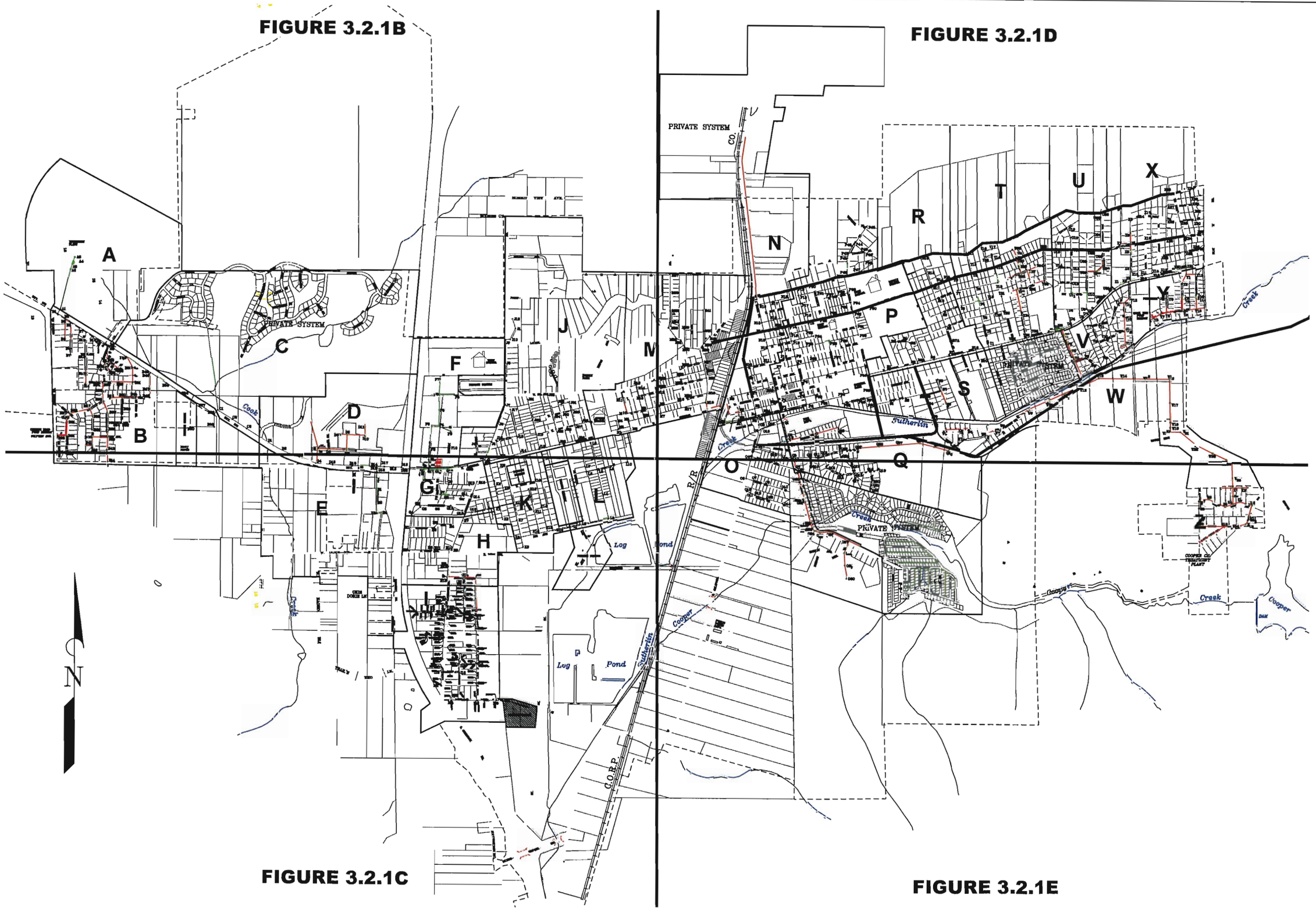


FIGURE 3.2.1C

FIGURE 3.2.1E

FIGURE NO. 3.2.1A

CITY OF SUTHERLIN
WASTEWATER FACILITIES PLAN

EXISTING COLLECTION SYSTEM MAP

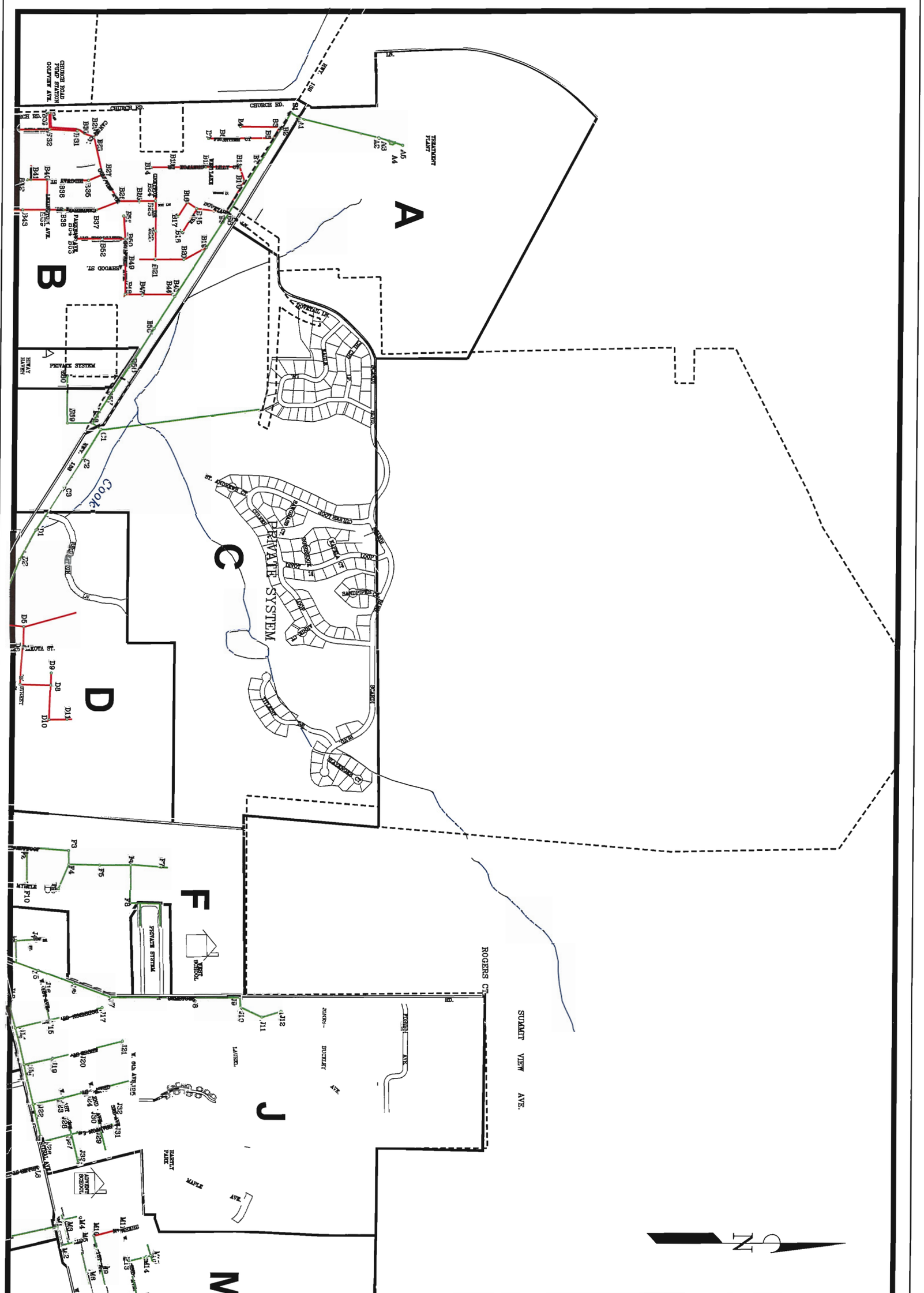
THE DYER PARTNERSHIP
ENGINEERS & PLANNERS

DATE: JAN., 2010

PROJECT NO.: 146.02

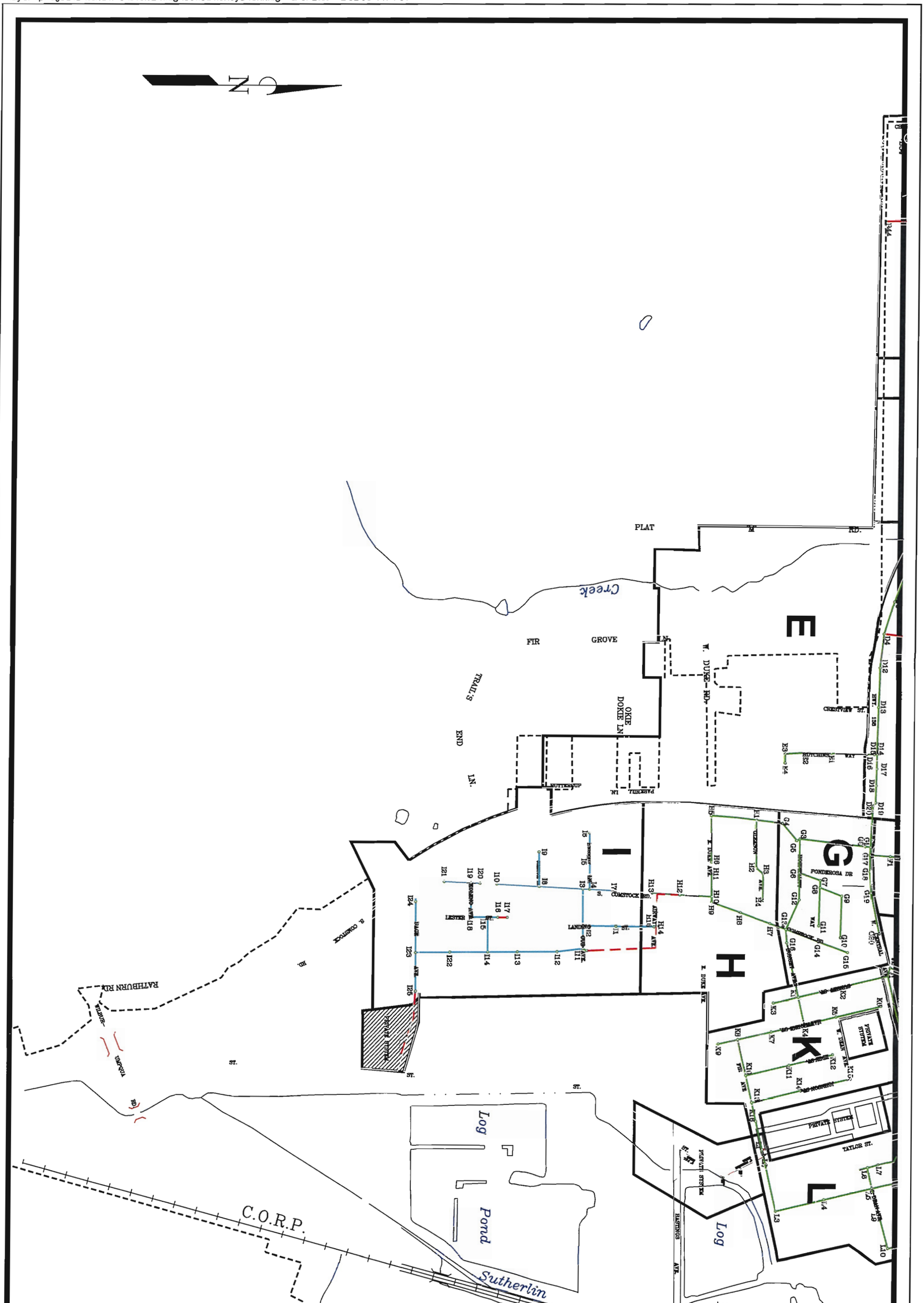
\\Dyer\projects\01Active\146.02\dwg\CollectionSystem.dwg 1/8/2010 2:52:36 PM PST

\\Dyer\projects\01Active\146.02\dwg\CollectionSystem.dwg 1/8/2010 2:52:36 PM PST



THE DYER PARTNERSHIP ENGINEERS & PLANNERS	CITY OF SUTHERLIN WASTEWATER FACILITIES PLAN	FIGURE NO.
DATE: JAN., 2010	EXISTING COLLECTION SYSTEM MAP	3.2.1B
PROJECT NO.: 146.02		

\\Dyer\projects\01Active\146.02\dwg\CollectionSystem.dwg 1/8/2010 2:52:36 PM PST



THE DYER PARTNERSHIP ENGINEERS & PLANNERS	CITY OF SUTHERLIN WASTEWATER FACILITIES PLAN	FIGURE NO.
DATE: JAN., 2010	EXISTING COLLECTION SYSTEM MAP	3.2.1C
PROJECT NO.: 146.02		

PRIVATE SYSTEM

CO.

N

R

T

U

X

P

S

V

W

Y

Creek

Sutherlin

PRIVATE SYSTEM

NOT A THROUGH STREET

QUAIL RUN PUMP STATION

FIGURE NO.
3.2.1D

CITY OF SUTHERLIN
WASTEWATER FACILITIES PLAN

EXISTING COLLECTION SYSTEM MAP

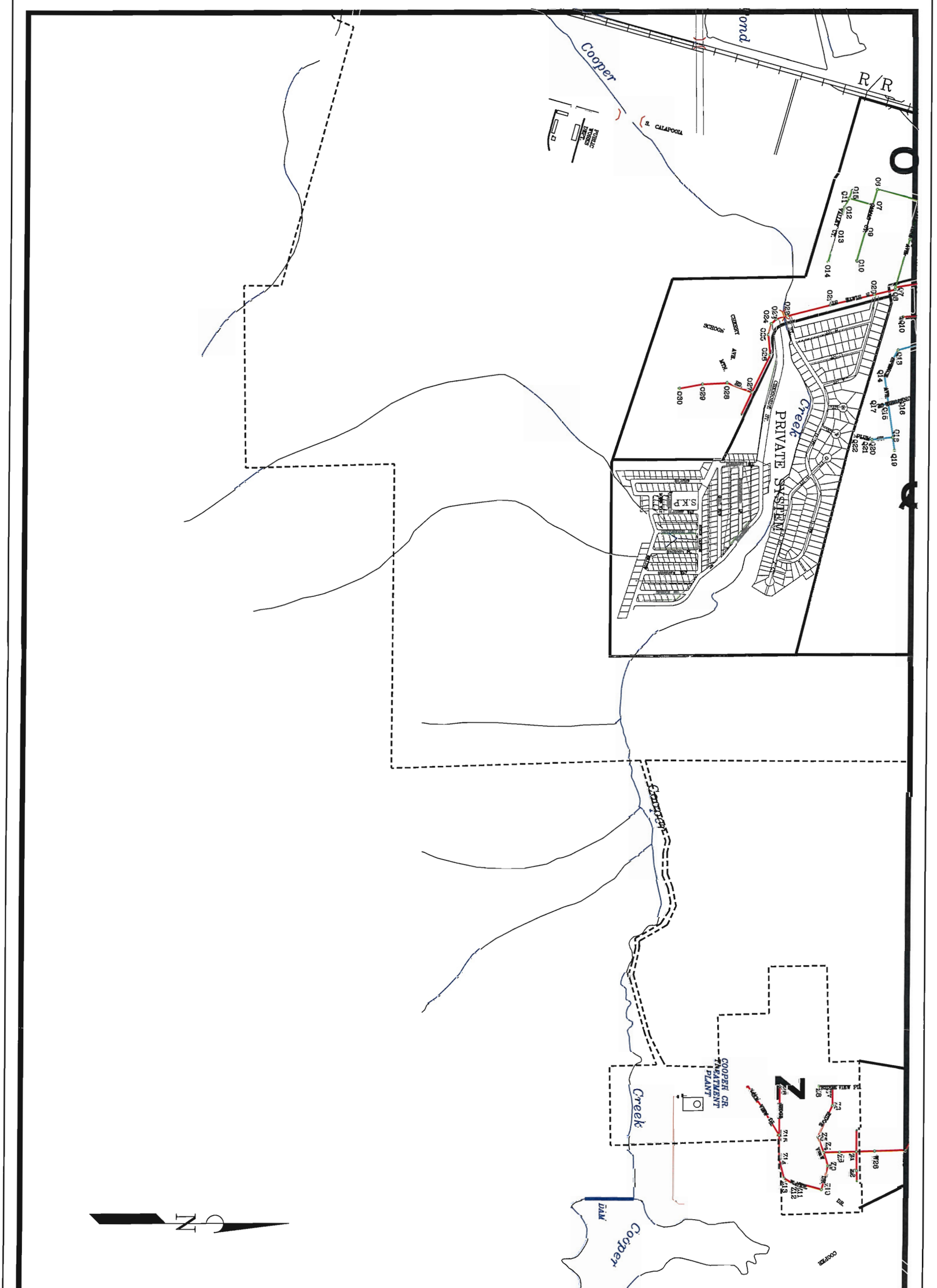
THE DYER PARTNERSHIP
ENGINEERS & PLANNERS

DATE: JAN., 2010

PROJECT NO.: 146.02

\\Dyer\projects\01Active\146.02\dwg\CollectionSystem.dwg 1/8/2010 2:52:36 PM PST

\\Dyer\projects\01Active\146.02\dwg\CollectionSystem.dwg 1/8/2010 2:52:36 PM PST



THE DYER PARTNERSHIP ENGINEERS & PLANNERS	CITY OF SUTHERLIN WASTEWATER FACILITIES PLAN	FIGURE NO.
DATE: JAN., 2010	EXISTING COLLECTION SYSTEM MAP	3.2.1E
PROJECT NO.: 146.02		

APPENDIX C – COST ESTIMATES

C

Cost Estimates

City of Sutherlin Draft

10/23/13

		County:	Douglas			
Compliance Issue? (Y/N)						
Number Jobs Created						
					Per EDU	
		# EDUs*	3,210	Current Single Family Rate	33.50	
		Annual O&M	1,260,765	O&M per EDU per mo.	32.73	
Total Project Cost:		Current debt pmt	83,429	Existing debt pmt/EDU/mo.	2.17	34.90
$ 21,242,000						

Financing Scenarios

Type	Award	select	Loan	Grant	Annual Debt Payment	Rate *	Term (years)	Total of payments	Monthly Debt per EDU	Debt Cost - EDU/Mo.	
IFA Loan & Grant											
IFA, CDBG				2,000,000							
IFA, WW			18,492,000	750,000	1,177,462	3.95%	25	29,436,543	30.57	65.46	
RD											
USDA, RUS		x	19,242,000	2,000,000	799,149	2.75%	40	31,965,976	20.75	55.64	
IFA and DEQ											
IFA CBDG				2,000,000							
DEQ CWSRF			19,242,000	-	1,172,240	1.96%	20	23,444,799	30.43	65.33	
										selected	55.64

Notes: *Estimated rates; Direct rates are set quarterly. Oregon Bond Bank rates are set at time of the Bond Sale. ** USDA/RUS Payment includes 10% Annual Debt Service; ***DEQ Rate is "an estimate of the Effective rate" reflecting an annual fee; **These Scenarios are ESTIMATES ONLY and should not be considered actual rates, costs or funding commitments.**

Alternatives Construction Cost Summary

Description	Est. Cost	Tertiary Filter			MBR		
		Oxidation Ditch	Conventional Aeration Basin	SBR	Oxidation Ditch	Conventional Aeration Basin	SBR
		$ 3,348,000	$ 2,104,000	$ 4,487,000	$ 3,248,000	$ 1,788,000	$ 2,927,000
Influent Pump Station	$ 816,000	●	●	●	●	●	●
Headworks with Influent Screen	$ 1,247,000	●	●	●	●	●	●
Clarifiers with filter	$ 2,624,000	●	●				
Clarifiers with MBR	$ 2,440,000				●	●	
MBR	$ 4,534,000				●	●	●
Tertiary Filter	$ 1,004,000	●	●	●			
UV Disinfection	$ 529,000	●	●	●	●	●	●
Recycled Water Disinfection	$ 250,000	●	●	●	●	●	●
RAS/WAS Building	$ 518,000	●	●		●	●	
Convert Existing Tanks to Digestion	$ 775,000	●	●	●	●	●	●
Bio-solids Process Facility	$ 1,160,000	●	●	●	●	●	●
Site Structures	$ 555,000	●	●	●	●	●	●
Recycled Water Storage Pond	$ 742,000	●	●	●	●	●	●
Misc. Equipment	$ 380,000	●	●	●	●	●	●
Site Work	$ 723,000	●	●	●	●	●	●
Total Basic Construction		$ 14,421,000	$ 13,427,000	$ 12,668,000	$ 17,917,000	$ 16,457,000	$ 14,638,000
Engineering, Design, Bidding Services		$ 1,442,000	$ 1,343,000	$ 1,267,000	$ 1,792,000	$ 1,646,000	$ 1,464,000
Engineering Construction Services		$ 1,442,000	$ 1,343,000	$ 1,267,000	$ 1,792,000	$ 1,646,000	$ 1,464,000
Value Engineering		$ 85,000	$ 85,000	$ 85,000	$ 85,000	$ 85,000	$ 85,000
Contingency @ 15%		$ 2,163,000	$ 2,014,000	$ 1,900,000	$ 2,688,000	$ 2,469,000	$ 2,196,000
Environmental Report		$ 45,000	$ 45,000	$ 45,000	$ 45,000	$ 45,000	$ 45,000
Fords Pond Land Acquisition		$ 3,000,000	$ 3,000,000	$ 3,000,000	$ 3,000,000	$ 3,000,000	$ 3,000,000
Review Fees		$ 10,000	$ 10,000	$ 10,000	$ 10,000	$ 10,000	$ 10,000
Admin / Legal		$ 75,000	$ 75,000	$ 75,000	$ 75,000	$ 75,000	$ 75,000
WWTP Project Total		**$ 22,683,000**	**$ 21,342,000**	**$ 20,317,000**	**$ 27,404,000**	**$ 25,433,000**	**$ 22,977,000**
Everette Avenue Pump Station		$ 925,000	$ 925,000	$ 925,000	$ 925,000	$ 925,000	$ 925,000
WWFP Total		**$ 23,608,000**	**$ 22,267,000**	**$ 21,242,000**	**$ 28,329,000**	**$ 26,358,000**	**$ 23,902,000**

Alternatives Operations Cost Summary

Description		Filter			MBR		
		Oxidation Ditch	Conventional Aeration Basin	SBR	Oxidation Ditch	Conventional Aeration Basin	SBR
	P.W. Operational	$ 2,318,000	$ 1,309,000	$ 1,367,000	$ 2,318,000	$ 1,309,000	$ 1,367,000
Influent Screen	$ 120,000	●	●	●	●	●	●
Influent Pump Station	$ 382,000	●	●	●	●	●	●
Headworks	$ 72,000	●	●	●	●	●	●
Clarifier - 6.25 mgd	$ 209,000	●	●				
Clarifier - 5.5 mgd	$ 209,000				●	●	
MBR	$ 2,041,000				●	●	●
Tertiary Filter	$ 132,000	●	●	●			
Chlorination	$ 42,000	●	●	●	●	●	●
UV Disinfection	$ 135,000	●	●	●	●	●	●
Ras/Was	$ 320,000	●	●		●	●	
Sludge/ Biosolids	$ 345,000	●	●	●	●	●	●
	O&M Cost	$ 4,075,000	$ 3,066,000	$ 2,595,000	$ 5,984,000	$ 4,975,000	$ 4,504,000

Description		Filter			MBR		
		Oxidation Ditch	Conventional Aeration Basin	SBR	Oxidation Ditch	Conventional Aeration Basin	SBR
	Est. Cost	$ 3,348,000	$ 2,104,000	$ 4,487,000	$ 3,248,000	$ 1,788,000	$ 2,927,000
Clarifiers w filter	$ 2,624,000	●	●				
Clarifiers w MBR	$ 2,440,000				●	●	
MBR	$ 4,534,000				●	●	●
Tertiary Filter	$ 1,004,000	●	●	●			
Ras/Was Building	$ 518,000	●	●		●	●	
Total Basic Construction		$ 7,494,000	$ 6,250,000	$ 5,491,000	$ 10,740,000	$ 9,280,000	$ 7,461,000

Treatment Alternatives Operations Cost Summary

Description		Filter			MBR		
		Oxidation Ditch	Conventional Aeration Basin	SBR	Oxidation Ditch	Conventional Aeration Basin	SBR
	P.W. Operational	$ 2,318,000	$ 1,309,000	$ 1,367,000	$ 2,318,000	$ 1,309,000	$ 1,367,000
MBR	$ 2,041,000				●	●	●
Tertiary Filter	$ 132,000	●	●	●			
	O&M Cost	2,450,000	1,441,000	1,499,000	4,359,000	3,350,000	3,408,000

Disinfection Alternatives Present Worth

	Year Round HS Disinfection	Year Round OSG Disinfection	HS/UV Disinfection	OSG/UV Disinfection
Capital				
Equipment	$ 165,000	$ 275,000	$ 396,000	$ 506,000
Structure	$ 892,000	$ 892,000	$ 383,000	$ 383,000
O&M	$ 245,000	$ 220,000	$ 309,000	$ 303,000
Total	**$ 1,302,000**	**$ 1,387,000**	**$ 1,088,000**	**$ 1,192,000**

City of Sutherlin				Common Impr. used in multiple trains
Construction Cost Estimate			**Date:**	**10/8/2013**
Influent Screen				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Site Preparation	LS	1	$ 48,000	$ 48,000
Mechanical Screening Unit	LS	2	$ 175,000	$ 350,000
Retrofit Existing Channel	EA	1	$ 20,000	$ 20,000
Retrofit grating	LS	1	$ 8,000	$ 8,000
New Access Stairs	LS	1	$ 10,000	$ 10,000
Sampler Stations	LS	1	$ 15,000	$ 15,000
Electrical Modification	LS	1	$ 15,000	$ 15,000
Vac Truck Dump Station and Screen	LS	1	$ 45,000	$ 45,000
Instrumentation	LS	1	$ 15,000	$ 15,000
Total				**$ 526,000**

Influent Pump Station				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Site Preparation	LS	1	$ 67,000	$ 67,000
Dewatering and Shoring	LS	1	$ 35,000	$ 35,000
Foundation Stabilization	LS	1	$ 40,000	$ 40,000
New Control Enclosure	SF	250	$ 150	$ 37,500
Submersible five-plex Pumps (Rail Guides, Piping, Valves)	LS	1	$ 205,000	$ 205,000
Redundant High Flow pump	LS	1	$ 60,000	$ 60,000
Wet Well Excavation & backfill	CY	210	$ 125	$ 26,250
Wet Well Concrete Structure	CY	50	$ 1,200	$ 60,000
Wetwell Lid with Hatch (Traffic Rated, New Concrete Pad)	LS	4	$ 12,500	$ 50,000
Control Panel with Controls (NEMA 4X)	LS	1	$ 60,000	$ 60,000
Level Control (Pressure Transducer, Float Backup)	LS	1	$ 10,000	$ 10,000
Magnetic Meter	LS	1	$ 25,000	$ 25,000
AC Pavement Access	TON	50	$ 130	$ 6,500
Check Valve Vault (Traffic Rated)	LS	1	$ 15,000	$ 15,000
Pump Station Building	SF	300	$ 200	$ 60,000
Boom Crane (w/ Winch, Platform Mnt.)	EA	1	$ 20,000	$ 20,000
Bollards	EA	4	$ 1,000	$ 4,000
Electrical	LS	1	$ 35,000	$ 35,000
Total				**$ 816,000**

Headworks				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Site Preparation	SF	1,130	$ 15	$ 16,950
Excavation	CY	190	$ 20	$ 3,800
Gravel under structure	CY	130	$ 50	$ 6,500
Backfill - Structural	CY	70	$ 50	$ 3,500
Foundation Concrete	CY	50	$ 900	$ 45,000
Structural Concrete	CY	100	$ 1,200	$ 120,000
Grit Removal / Classifing & Washing / Dewatering	EA	1	$ 377,000	$ 377,000
Slide Gates	LS	1	$ 40,000	$ 40,000
Chute & Dumpster	LS	1	$ 3,000	$ 3,000
Stairs	LS	1	$ 5,000	$ 5,000
Walkway	LF	60	$ 80	$ 4,800
Piping and Connections	LS	1	$ 30,000	$ 30,000
Electrical	LS	1	$ 30,000	$ 30,000
Instrumentation	LS	1	$ 25,000	$ 25,000
Misc. grate	SF	150	$ 45	$ 6,750
Handrails	LF	60	$ 60	$ 3,600
Total				**$ 721,000**

RAS/WAS Pump Station				
Description	Unit	Quantity	Unit Cost	Total Cost
Site Preparation	SF	1,600	$ 15	$ 24,000
RAS Pumps	EA	2	$ 37,500	$ 75,000
WAS Pumps	EA	2	$ 22,500	$ 45,000
8" RAS mag meters	EA	2	$ 16,000	$ 32,000
6" WAS mag meters	EA	2	$ 15,000	$ 30,000
6" DI sta. pipe	LF	180	$ 180	$ 32,400
DI fittings lbs	LF	2,500	$ 4	$ 10,000
Misc. Hardware	LS	1	$ 8,500	$ 8,500
Pump monorail	LS	2	$ 16,000	$ 32,000
Pump Hoists	EA	2	$ 8,500	$ 17,000
Control panel	LS	1	$ 30,000	$ 30,000
Instrumentation	LS	1	$ 25,000	$ 25,000
Electrical	LS	1	$ 32,000	$ 32,000
Building (25' x 25')	SF	625	$ 200	$ 125,000
Total				$ 518,000

Tertiary Filtration - Up flow sand filters				
	Unit	Qnt.	Unit Cst.	Tot. Cost
Site Preparation	SF	1,600	$ 15	$ 24,000
Excavation	CY	310	$ 20	$ 6,200
Gravel under structure	CY	120	$ 50	$ 6,000
Concrete Slab	CY	90	$ 900	$ 81,000
Backfill - structural	CY	60	$ 35	$ 2,100
Electrical	LS	1	$ 32,000	$ 32,000
Pump Station	LS	1	$ 40,000	$ 40,000
Piping and gates	LS	1	$ 45,000	$ 45,000
Mag Meter	EA	3	$ 15,000	$ 45,000
Instrumentation	LS	1	$ 5,000	$ 5,000
Galvanized Stairs and access platform	LS	1	$ 35,000	$ 35,000
Equipment	LS	1	$ 756,300	$ 756,300
Total				$ 1,078,000

Tertiary Filtration - Disk Filter				
	Unit	Qnt.	Unit Cst.	Tot. Cost
Site Preparation	SF	1,750	$ 15	$ 26,250
Excavation	CY	260	$ 20	$ 5,200
Gravel under structure	CY	130	$ 50	$ 6,500
Backfill - structural	CY	70	$ 35	$ 2,450
Btm Concrete Slab	CY	50	$ 900	$ 45,000
Concrete Wall	CY	70	$ 1,200	$ 84,000
Electrical	LS	1	$ 32,000	$ 32,000
Pump Station	LS	1	$ 40,000	$ 40,000
Piping and gates	LS	1	$ 40,000	$ 40,000
Mag Meter	EA	3	$ 11,000	$ 33,000
Instrumentation	LS	1	$ 25,000	$ 25,000
Galvanized Stairs, access platform and handrail	LS	1	$ 40,000	$ 40,000
Equipment	LS	1	$ 625,000	$ 625,000
Total				$ 1,004,000

Clarifiers 80' dia.- 6.25 mgd				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Site Preparation	LS	1	$ 20,000	$ 20,000
Excavation	CY	820	$ 30	$ 24,600
Backfill (struc.)	CY	410	$ 35	$ 14,350
Gravel under struc	CY	200	$ 50	$ 10,000
Concrete Slab	CY	280	$ 1,200	$ 336,000
Outer Walls	CY	150	$ 1,200	$ 180,000
Outer Footings	CY	80	$ 1,200	$ 96,000
Center Sludge Ring Foundation	CY	10	$ 1,200	$ 12,000
Launder Concrete	CY	50	$ 1,500	$ 75,000
Equipment	LS	1	$ 193,900	$ 193,900
Electrical	LS	1	$ 120,000	$ 120,000
Instrumentation	LS	1	$ 50,000	$ 50,000
Misc. Equipment	LS	1	$ 50,000	$ 50,000
Wall fittings pipe	LS	1	$ 5,000	$ 5,000
Stairs	LF	20	$ 300	$ 6,000
Subtotal for single unit				$ 1,431,420
Total for 2 Clarifiers				**$ 2,624,000**

Clarifiers 75' dia.- 5.25 mgd				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Site Preparation	LS	1	$ 20,000	$ 20,000
Excavation	CY	730	$ 30	$ 21,900
Backfill (struc.)	CY	360	$ 35	$ 12,600
Gravel under struc	CY	170	$ 50	$ 8,500
Concrete Slab	CY	250	$ 1,200	$ 300,000
Outer Walls	CY	140	$ 1,200	$ 168,000
Outer Footings	CY	80	$ 1,200	$ 96,000
Center Sludge Ring Foundation	CY	10	$ 1,200	$ 12,000
Launder Concrete	CY	40	$ 1,500	$ 60,000
Equipment	LS	1	$ 179,000	$ 179,000
Electrical	LS	1	$ 120,000	$ 120,000
Instrumentation	LS	1	$ 50,000	$ 50,000
Misc. Equipment	LS	1	$ 50,000	$ 50,000
Wall fittings pipe	LS	1	$ 5,000	$ 5,000
Stairs	LF	20	$ 300	$ 6,000
Subtotal for single unit				$ 1,330,800
Total for 2 Clarifiers				**$ 2,440,000**

UV Disinfection	Unit	Qnt.	Unit Cost	Total Cost
Site Preparation	SF	920	$ 15	$ 13,800
Excavation	CY	110	$ 20	$ 2,200
Gravel under structure	CY	70	$ 50	$ 3,500
Concrete - foundation	CY	30	$ 1,200	$ 36,000
Concrete - structure	CY	40	$ 1,200	$ 48,000
Backfill - structural	CY	40	$ 30	$ 1,200
Electrical	LS	1	$ 28,000	$ 28,000
Piping and gates	LS	1	$ 40,000	$ 40,000
Instrumentation	LS	1	$ 25,000	$ 25,000
Shelter over outdoor UV channel	SF	500	$ 200	$ 100,000
Subtotal				**$ 298,000**
Equipment	LS	1	$ 231,000	$ 231,000
Total				**$ 529,000**

City of Sutherlin	Common Impr. used in multiple trains
Construction Cost Estimate	Date: 10/8/2013

HS Chlorination of Reclaimed water				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Electrical	LS	1	$ 15,000	$ 15,000
Piping & Fittings	LS	1	$ 15,000	$ 15,000
Instrumentation	LS	1	$ 15,000	$ 15,000
Structure	SF	200	$ 200	$ 40,000
Subtotal				**$ 85,000**
Chlorination Equipment (pumps, tanks, etc.)	LS	1	$ 165,000	$ 165,000
Total				**$ 250,000**

OSG Chlorination of Reclaimed water				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Electrical	LS	1	$ 15,000	$ 15,000
Piping & Fittings	LS	1	$ 15,000	$ 15,000
Instrumentation	LS	1	$ 15,000	$ 15,000
Structure	SF	200	$ 200	$ 40,000
Subtotal				**$ 85,000**
Chlorination Equipment (pumps, tanks, etc.)	LS	1	$ 275,000	$ 275,000
Total				**$ 360,000**

HS Chlorination / Dechlorination Year Round				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Site Preparation including Demoliton	LS	1	$18,000	$ 18,000
Excavation	CY	390	$20	$ 7,800
Backfill Sturcture	CY	210	$50	$ 10,489
Concrete Slab	CY	120	$900	$ 108,000
Concrete Exterior Walls	CY	40	$1,200	$ 48,000
Concrete Interior Walls	CY	70	$1,200	$ 84,000
New Inlet Structure	LS	1	$2,000	$ 2,000
Drainage and cleaning sumps	EA	6	$2,000	$ 12,000
Misc Grating	SF	500	$30	$ 15,000
Misc Piping	LF	100	$80	$ 8,000
New Handrail	LF	250	$30	$ 7,500
New injection piping and equipment	LS	1	$20,000	$ 20,000
New flow pace meter	LS	1	$7,000	$ 7,000
Instrumentation	LS	1	$20,000	$ 20,000
Electrical	LS	1	$15,000	$ 15,000
Dechlorination Equipment (bldg., pumps, tanks, etc.)	LS	1	$63,100	$ 63,100
subtotal	**EA**	**2**		**$ 892,000**
Chlorination Equipment (pumps, tanks, etc.)	LS	1	$ 165,000	$ 165,000
Total				**$ 1,057,000**

City of Sutherlin			Common Impr. used in multiple trains	
Construction Cost Estimate			**Date:**	**10/8/2013**
OSG Chlorination / Dechlorination Year Round				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Site Preparation including Demoliton	LS	1	$18,000	$ 18,000
Excavation	CY	390	$20	$ 7,800
Backfill Sturcture	CY	210	$50	$ 10,489
Concrete Slab	CY	120	$900	$ 108,000
Concrete Exterior Walls	CY	40	$1,200	$ 48,000
Concrete Interior Walls	CY	70	$1,200	$ 84,000
New Inlet Structure	LS	1	$2,000	$ 2,000
Drainage and cleaning sumps	EA	6	$2,000	$ 12,000
Misc Grating	SF	500	$30	$ 15,000
Misc Piping	LF	100	$80	$ 8,000
New Handrail	LF	250	$30	$ 7,500
New injection piping and equipment	LS	1	$20,000	$ 20,000
New flow pace meter	LS	1	$7,000	$ 7,000
Instrumentation	LS	1	$20,000	$ 20,000
Electrical	LS	1	$15,000	$ 15,000
Dechlorination Equipment (bldg., pumps, tanks, etc.)	LS	1	$63,100	$ 63,100
subtotal	**EA**	**2**		**$ 892,000**
Chlorination Equipment (pumps, tanks, etc.)	LS	1	$ 275,000	$ 275,000
Total				**$ 1,167,000**

Existing Treatment Unit Conversion to Digestion				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Sludge transfer Pumps	EA	8	$ 15,000	$ 120,000
Electrical	LS	1	$ 75,000	$ 75,000
Instrumentation	LS	1	$ 15,000	$ 15,000
Piping Modifications	LS	1	$ 30,000	$ 30,000
Aerators	EA	6	$ 20,000	$ 120,000
Piping for air	LS	1	$ 40,000	$ 40,000
Sludge Hauilng During Construction	LS	1	$ 40,000	$ 40,000
Demolition of old equipment	LS	2	$ 12,500	$ 25,000
Misc. Equipment	LS	1	$ 10,000	$ 10,000
Decanters	EA	8	$ 15,000	$ 120,000
Air Blowers	EA	6	$ 30,000	$ 180,000
				$ 775,000

Biosolids Press (& filter room conversion)				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Site Preparation / Demo of existing Filter	LS	1	$ 30,000	$ 30,000
Structure fill of basement area	CY	230	$ 40	$ 9,200
Concrete slab, floor and walls	CY	106	$ 700	$ 74,200
Structural Modifications and garage door	LS	1	$ 15,000	$ 15,000
Galvanized steel stairs	EA	1	$ 5,000	$ 5,000
New Filter Press	EA	1	$ 543,000	$ 543,000
Conveyor	EA	1	$ 120,000	$ 120,000
Electrical	LS	1	$ 45,000	$ 45,000
Instrumentation	LS	1	$ 15,000	$ 15,000
Piping	LS	1	$ 40,000	$ 40,000
Sludge Pumps	EA	2	$ 32,000	$ 64,000
Polymer Equip.	LS	1	$ 60,000	$ 60,000
Biosolids Dry Storage	SF	1,400	$ 100	$ 140,000
Total				**$ 1,160,000**

City of Sutherlin	Common Impr. used in multiple trains
Construction Cost Estimate	**Date: 10/8/2013**

Sludge Drying Beds

	Unit	Qnt.	Unit Cst.	Tot. Cost
Site Preparation	LS	1	$101,000.00	$101,000.00
Walls	CY	70	$800.00	$56,000.00
Footing	CY	160	$800.00	$128,000.00
Excavation	CY	2800	$15.00	$42,000.00
liner	SF	36000	$1.50	$54,000.00
graval sand	CY	2000	$50.00	$100,000.00
perf. collection pipe	LF	2400	$20.00	$48,000.00
distribution pipe	LF	120	$25.00	$3,000.00
stop logs	EA	20	$600.00	$12,000.00
Structural Cover	SF	36000	$45.00	$1,620,000.00
Misc. items	LS	1	$5,000.00	$5,000.00
lights	LS	1	$5,000.00	$5,000.00
Total				$2,174,000.00

Sludge Storage 95' dia. - 1,269,000 gallons

Description	Unit	Quantity	Unit Cost	Total Cost
Site Preparation	LS	1	$ 100,000	$ 100,000
Foundation Stablizaton	LS	1	$ 50,000	$ 50,000
Excavation	CY	1,200	$ 30	$ 36,000
Backfill (struc.)	CY	100	$ 35	$ 3,500
Gravel under struc	CY	900	$ 50	$ 45,000
Concrete Slab	CY	600	$ 900	$ 540,000
Outer Walls	CY	300	$ 1,200	$ 360,000
Outer Footings	CY	70	$ 1,200	$ 84,000
Elevated walkway	CY	60	$ 1,500	$ 90,000
WAS Pump	LS	1	$ 35,000	$ 35,000
Electrical	LS	1	$ 10,000	$ 10,000
Decanter	LS	1	$ 20,000	$ 20,000
Misc. Equipment	LS	1	$ 10,000	$ 10,000
Wall fittings pipe	LS	1	$ 5,000	$ 5,000
Handrail & Stairs	LS	1	$ 30,000	$ 30,000
Total				**$ 1,419,000**

Misc. Site Work

Description	Unit	Quantity	Unit Cost	Total Cost
Site Preparation	SY	4,000	$ 18	$ 72,000
Roadway Excavation	CY	2,600	$ 20	$ 52,000
Drive and parking Aggregate Base	CY	2,000	$ 50	$ 100,000
Yard Piping 24" diameter	LF	430	$ 125	$ 53,750
Yard Piping 16" diameter	LF	1,150	$ 100	$ 115,000
Yard Piping 6" diameter	LF	800	$ 50	$ 40,000
Potable Water System	LS	1	$ 35,000	$ 35,000
Non Potable Water System	LS	1	$ 40,000	$ 40,000
New Compound Lighting	LS	1	$ 50,000	$ 50,000
Erosion Control	LS	1	$ 10,000	$ 10,000
Final Landscaping	LS	1	$ 45,000	$ 45,000
Drainage	LS	1	$ 50,000	$ 50,000
Fencing	LF	1,000	$ 60	$ 60,000
Total				**$ 723,000**

City of Sutherlin	Common Impr. used in multiple trains
Construction Cost Estimate	**Date: 10/8/2013**

Public Works Buildings				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Public Works Garage	SF	2,200	$ 150	$ 330,000
Remodel and enlarge Lab	LS	1	$ 100,000	$ 100,000
Blower Building	SF	625	$ 200	$ 125,000
Total				$ 555,000

30 Acre Recycled Water Storage Pond + Golf Course Irrigation + 105 acres of new Irrgation				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Levee site preparation	LS	1	$ 20,000	$ 20,000
Site topsoil stripping	CY	80,000	$ 5	$ 400,000
Berm Construction	CY	57,600	$ 15	$ 864,000
Pond Lining				
Pond Underdrain	LF	70,000	$ 15	$ 1,050,000
Pond Anchor Trench	LF	4,800	$ 5	$ 24,000
Pond Lining Fine Grading	LS	1	$ 10,000	$ 10,000
HDPE Lining Placement	SF	1,440,000	$ 0.65	$ 936,000
Inlet & Outlet Structures	LS	1	$ 20,000	$ 20,000
Outfall Pipe to Creek	LF	1,000	$ 50	$ 50,000
Electrical	LS	1	$ 12,000	$ 12,000
Force Main	LF	2,400	$ 50	$ 120,000
Pump Station	LS	1	$ 200,000	$ 200,000
Irrigation Equipment	AC	105	$ 1,500	$ 157,500
Wetlands Mitigation	LS	1	$ 50,000	$ 50,000
Land Acquistion	AC	160	$ 5,000	$ 800,000
Total				$ 4,714,000

Fords Pond Recycled Water Storage				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Site preparation	LS	1	$ 20,000	$ 20,000
60-Inch Culverts	LF	900	$ 300	$ 270,000
Inlet & Outlet Structures	LS	1	$ 80,000	$ 80,000
Electrical	LS	1	$ 12,000	$ 12,000
Force Main	LF	80	$ 2,000	$ 160,000
Pump Station	LS	1	$ 200,000	$ 200,000
Land Acquistion	LS	1	$ -	$ -
Total				$ 742,000

Note: add $3,000,000 for land acquistion

Miscellaneous Equipment				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
New Television Monitoring Equipment	LS	1	$ 40,000	$ 40,000
Standby Generator (400kw) and Trans Switch	LS	1	$ 190,000	$ 190,000
Liquid Sludge Transport Truck w/ Spreader	LS	1	$ 150,000	$ 150,000
Total				$ 380,000

City of Sutherlin				Common Impr. used in multiple trains
Construction Cost Estimate			**Date:**	**10/8/2013**
MBR				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Site Preparation	SF	10,660	$ 15	$ 159,900
Excavation	CY	1,180	$ 30	$ 35,400
Backfill (struc.)	CY	150	$ 50	$ 7,500
Gravel under struc	CY	730	$ 50	$ 36,500
Btm. slab in tank	CY	350	$ 900	$ 315,000
Outer Walls	CY	310	$ 1,200	$ 372,000
Outer Footings	CY	90	$ 1,200	$ 108,000
Interior Walls	CY	200	$ 900	$ 180,000
Interior Wall Footings	CY	40	$ 1,200	$ 48,000
Outlet weir	CY	20	$ 900	$ 18,000
Walkway Slab	CY	40	$ 1,500	$ 60,000
Sidewalk	CY	50	$ 500	$ 25,000
Equipment	LS	1	$ 2,833,600	$ 2,833,600
Controls	LS	1	$ 85,000	$ 85,000
Electrical	LS	1	$ 110,000	$ 110,000
Mechanical	LS	1	$ 90,000	$ 90,000
Hand rails	LF	550	$ 75	$ 41,250
Stairs	LF	30	$ 300	$ 9,000
Total				**$ 4,534,000**

City of Sutherlin				Aeration Basin / Filter
Construction Cost Estimate			**Date:**	**6/24/2013**
Aeration Basin				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Site Preparation	SF	11,480	$ 3	$ 34,440
Excavation	CY	1,700	$ 20	$ 34,000
Backfill (struc.)	CY	150	$ 35	$ 5,250
Gravel under struc	CY	780	$ 50	$ 39,000
Btm. Concrete slab in tank	CY	560	$ 900	$ 504,000
Concrete Footings	CY	190	$ 1,200	$ 228,000
Outer Outer Walls	CY	320	$ 1,200	$ 384,000
Center Cell Concrete Divider Wall	CY	160	$ 900	$ 144,000
Center Recirculation Channel	CY	80	$ 1,500	$ 120,000
Concrete Bridge Slab	CY	80	$ 1,500	$ 120,000
Concrete Outlet weir struc..	CY	10	$ 900	$ 9,000
Misc. conc. struc.	CY	10	$ 900	$ 9,000
Sidewalk	CY	10	$ 500	$ 5,000
Diffused Aerators	LS	2.0	$ 48,000	$ 96,000
Blowers	EA	2.0	$ 60,000	$ 120,000
Electrical	LS	1.0	$ 40,000	$ 40,000
Instrumentation	LS	1.0	$ 35,000	$ 35,000
Mechanical	LS	1.0	$ 121,000	$ 121,000
Hand rails	LF	720.0	$ 70	$ 50,400
Stairs	LF	30.0	$ 200	$ 6,000
Total Cost				$ 2,104,000

City of Sutherlin				Aeration Basin & MBR
Construction Cost Estimate			**Date:**	**6/24/2013**

Aeration Basin				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Site Preparation	SF	8,650	$ 3	$ 25,950
Excavation	CY	1,280	$ 30	$ 38,400
Backfill (struc.)	CY	130	$ 50	$ 6,500
Gravel under struc	CY	580	$ 50	$ 29,000
Btm. Concrete slab in tank	CY	420	$ 1,200	$ 504,000
Concrete Footings	CY	170	$ 1,200	$ 204,000
Outer Outer Walls	CY	270	$ 900	$ 243,000
Center Cell Concrete Divider Wall	CY	90	$ 900	$ 81,000
Center Recirculation Channel	CY	60	$ 1,500	$ 90,000
Concrete Bridge Slab	CY	70	$ 1,500	$ 105,000
Concrete Outlet weir struc..	CY	10	$ 900	$ 9,000
Misc. conc. struc.	CY	10	$ 900	$ 9,000
Sidewalk	CY	10	$ 500	$ 5,000
Diffused Aerators	LS	2.0	$ 48,000	$ 96,000
Blowers	EA	2.0	$ 60,000	$ 120,000
Electrical	LS	1.0	$ 40,000	$ 40,000
Instrumentation	LS	1.0	$ 35,000	$ 35,000
Mechanical	LS	1.0	$ 93,500	$ 93,500
Hand rails	LF	620.0	$ 70	$ 43,400
Stairs	LF	30.0	$ 350	$ 10,500
Total Cost				$ 1,788,000

City of Sutherlin				SBR with Filter
Construction Cost Estimate			**Date:**	**6/24/2013**
SBR				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Site Preparation	SF	16,130	$ 3	$ 48,390
Excavation	CY	1,790	$ 30	$ 53,700
Backfill (struc.)	CY	170	$ 35	$ 5,950
Gravel under struc	CY	1,190	$ 50	$ 59,500
Outer Concrete Walls	CY	350	$ 1,200	$ 420,000.00
Outer Concrete Footings	CY	160	$ 1,200	$ 192,000.00
Interior Concrete Walls	CY	330	$ 900	$ 297,000.00
Interior Concrete Wall Ftgings	CY	50	$ 1,200	$ 60,000.00
Btm. Concrete slab in tank	CY	730	$ 900	$ 657,000.00
Walkway Concrete Slab	CY	170	$ 1,500	$ 255,000.00
Outlet Concrete weir	CY	100	$ 900	$ 90,000.00
Sidewalk	CY	40	$ 500	$ 20,000.00
Equipment	LS	1.0	$ 2,000,000	$ 2,000,000.00
Controls	LS	1.0	$ 85,000	$ 85,000.00
Electrical	LS	1.0	$ 50,000	$ 50,000.00
Hand rails	LF	970	$ 75	$ 72,750.00
Equalization tank Cover	LS	1.0	$ 20,000	$ 20,000.00
Grates	LS	1.0	$ 90,000	$ 90,000.00
Stairs	LF	30.0	$ 350	$ 10,500.00
Total SBR				$ 4,487,000.00

City of Sutherlin				SBR with MBR
Construction Cost Estimate			**Date:**	**6/24/2013**

SBR

Description	Unit	Quantity	Unit Cost	Total Cost
Site Preparation	SF	8,800	$ 3	$ 26,400
Excavation	CY	980	$ 30	$ 29,400
Backfill (struc.)	CY	130	$ 50	$ 6,500
Gravel under struc	CY	650	$ 50	$ 32,500
Outer Concrete Walls	CY	270	$ 1,200	$ 324,000.00
Outer Concrete Footings	CY	80	$ 1,200	$ 96,000.00
Interior Concrete Walls	CY	160	$ 900	$ 144,000.00
Interior Concrete Wall Ftgings	CY	50	$ 1,200	$ 60,000.00
Btm. Concrete slab in tank	CY	70	$ 900	$ 63,000.00
Walkway Concrete Slab	CY	130	$ 1,500	$ 195,000.00
Outlet Concrete weir	CY	100	$ 900	$ 90,000.00
Sidewalk	CY	30	$ 500	$ 15,000.00
Equipment	LS	1.0	$ 1,568,000	$ 1,568,000.00
Controls	LS	1.0	$ 85,000	$ 85,000.00
Electrical	LS	1.0	$ 50,000	$ 50,000.00
Mechanical	LS	1.0	$ 85,000	$ 85,000.00
Hand rails	LF	620	$ 75	$ 46,500.00
Stairs	LF	30.0	$ 350	$ 10,500.00
Total SBR				$ 2,927,000.00

City of Sutherlin				Oxidation Ditch
Construction Cost Estimate				**Date: 6/24/2013**
Oxidation Ditch				
Description	**Unit**	**Quantity**	**Unit Cost**	**Total Cost**
Site Clring & Prep	SF	20000.0	$3.00	$60,000
Excavation	CY	2220.0	$30.00	$66,600
Backfill (struc.)	CY	190.0	$35.00	$6,650
Gravel under struc	CY	813.0	$50.00	$40,650
Concrete	CY	1580.0	$1,200.00	$1,896,000
Equipment	LS	1.0	$1,031,000.00	$1,031,000
Electrical	LS	1.0	$50,000.00	$50,000
Mechanical	LS	1.0	$90,000.00	$90,000
Controls	LS	1.0	$60,000.00	$60,000
Hand rails	LF	400.0	$75.00	$30,000
Stairs	LF	48.0	$350.00	$16,800
Oxidation Ditch Total Cost				$3,348,000

Influent Screen Power

2 units
3.5 HP per unit
125.328 kw h per day

Influent Screen

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Equipment Power	Kwh	15,270	$	0.10	$	1,527
2	Parts & Repair	LS	1	$	1,000	$	1,000
3	Operation Labor	Hr	78	$	28	$	2,184
4	Solids Disposal	Loads	52	$	40	$	2,080
5	Maintenance Labor	Hr	52	$	28	$	1,456
			Operations & Maintenance			**$**	**8,247**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost			**$**	**120,000**

Influent Pump Station Power consumption information calculated

Average Pump Rate MGD	1.38		
Average Pump Rate GPM	958.33		
50 Head ft.45% eff.	24.1	HP used ave.	SBR
KW req'd for average pumping	18.0	KW	SBR
	157938	kwh/yr	SBR

Influent Pump Station

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Influent Pump Power Demands 65'	Kwh	157,938	$	0.10	$	15,794
4	Controls, inst. power	Kwh	800	$	0.10	$	80
5	Parts & Repair	LS	1	$	5,500	$	5,500
6	Monitoring & Operation	Hr	100	$	28	$	2,800
7	Maintenance Labor	Hr	70	$	28	$	1,960
		SBR	**Operations & Maintenance**			**$**	**26,134**
			Interest Rate				3.2%
			Project life				20
		SBR	**Present Value O&M Cost**			**$**	**382,000**

Headworks Power

1 units
0.5 HP per unit
8.952 kw h per day

Headworks

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Equipment Power	Kwh/day	3,267	$	0.10	$	327
2	Parts & Repair	LS	1	$	650	$	650
3	Operation Labor	Hr	78	$	28	$	2,184
4	Solids Disposal	Hr	52	$	28	$	1,456
5	Maintenance Labor	Hr	10	$	28	$	280
			Operations & Maintenance			**$**	**4,897**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost			**$**	**72,000**

Clarifier Power

2 units
0.75 HP per unit
26.856 kw h per day

Clairifiers Annual & Present Worth O&M

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Electrical Power Demands *	Kwh	9,802	$	0.10	$	980
2	Parts & Repair	LS	1	$	4,000	$	4,000
3	Monitoring & Operation	Hr	300	$	28	$	8,400
4	Maintenance Labor	Hr	34	$	28	$	952
			Operations & Maintenance			**$**	**14,332**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost			**$**	**209,000**

MBR Electrical Power Demands		totals	
PA Blower	50 hp 3 each		
MBR Blower	30 hp 4 each		
Feed Forward Pump	40 hp 4 each	160	
Basin Mixing Pump	8 hp 2 each	16	
		176	hp
		3151.104	kwh/day

MBR Annual & Present Worth O&M

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Electrical Power Demands	Kwh	1,150,153	$	0.10	$	115,015
2	Parts & Repair	LS	1	$	2,000	$	2,000
3	Monitoring & Operation	Hr	500	$	28	$	14,000
4	Maintenance Labor	Hr	312	$	28	$	8,736
			Operations & Maintenance			**$**	**139,751**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost			**$**	**2,041,000**

Tertiary Filter

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Equipment Power	Kwh/day	9,000	$	0.10	$	900
2	Parts & Repair	LS	1	$	1,200	$	1,200
3	Operation Labor	Hr	208	$	28	$	5,824
5	Maintenance Labor	Hr	40	$	28	$	1,120
			Operations & Maintenance			**$**	**9,044**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost			**$**	**132,000**

Disinfection Alternatives

Full Year H/S (Chlorine) / Dechlorination Disinfection Annual & Present Worth O&M

Item	Operations & Maintenance Costs	Unit	Quantity	Unit Cost	Total Cost
1	Electricity	kWh	1,752	$0.10	$ 175
2	Labor @ 5 hrs per week	Hr	260	$28.00	$ 7,280
3	Hypochlorite Solution	lb	5,991	$0.50	$ 2,995
4	Sulphur Dioxide 150 lb cylinders	lb	7,488	$0.40	$ 2,995
5	Eductors Power @ 1.5 Hp	kWh	9,290	$0.10	$ 929
6	Repair Parts	LS	1	$1,500.00	$ 1,500
7	Reagents	LS	1	$900.00	$ 900
			Annual Project Total		**$ 16,775**
			Interest Rate		3.2%
			Project life		20
			Present Value O&M Cost		**$ 245,000**

Full Year OSG (Chlorine) / Dechlorination Disinfection Annual & Present Worth O&M

Item	Operations & Maintenance Costs	Unit	Quantity	Unit Cost	Total Cost
1	Electricity	kWh	1,752	$0.10	$ 175
2	Labor @ 5 hrs per week	Hr	260	$28.00	$ 7,280
3	Salts	ton	7	$100.00	$ 749
4	Sulphur Dioxide 150 lb cylinders	lb	7,488	$0.40	$ 2,995
5	Eductors Power @ 1.5 Hp	kWh	9,290	$0.10	$ 929
6	Repair Parts	LS	1	$2,000.00	$ 2,000
7	Reagents	LS	1	$900.00	$ 900
			Annual Project Total		**$ 15,028**
			Interest Rate		3.2%
			Project life		20
			Present Value O&M Cost		**$ 220,000**

Full Year UV Disinfection Annual & Present Worth O&M

Item	Description	Unit	Quantity	Unit Cost	Total Cost
1	Energy to Lamps	kWh	70,080	$ 0.10	$ 7,008
2	Weekly Lamp Cleaning	Hr	104	$ 28	$ 2,912
3	Lamp Replacement	EA	23	$ 45	$ 1,035
4	Ballast Replacement	EA	3	$ 180	$ 540
5	Quartz Replacement	EA	5	$ 45	$ 225
6	Monitoring & Operation	Hr	104	$ 22	$ 2,288
7	Semi Annual Basin Cleaning	Hr	16	$ 22	$ 352
			Operations & Maintenance		**$ 14,360**
			Interest Rate		3.2%
			Project life		20
			Present Value O&M Cost		**$ 210,000**

6 months UV Disinfection Annual & Present Worth O&M

Item	Description	Unit	Quantity	Unit Cost	Total Cost
1	Energy to Lamps	kWh	35,040	$ 0.10	$ 3,504
2	Weekly Lamp Cleaning	Hr	104	$ 28	$ 2,912
3	Lamp Replacement	EA	23	$ 45	$ 1,035
4	Ballast Replacement	EA	2	$ 180	$ 360
5	Quartz Replacement	EA	2	$ 45	$ 90
6	Monitoring & Operation	Hr	52	$ 22	$ 1,144
7	Semi Annual Basin Cleaning	Hr	8	$ 22	$ 176
			Operations & Maintenance		**$ 9,221**
			Interest Rate		3.2%
			Project life		20
			Present Value O&M Cost		**$ 135,000**

6 months (Summertime only) HS (Chlorine) / Dechlorination Disinfection Annual & Present Worth O&M

Item	Operations & Maintenance Costs	Unit	Quantity	Unit Cost	Total Cost
1	Electricity	kWh	1,752	$0.10	$ 175
2	Labor @ 5 hrs per week	Hr	150	$28.00	$ 4,200
3	Hypochlorite Solution	lb	2,462	$0.50	$ 1,231
4	Sulphur Dioxide 150 lb cylinders	lb	7,488	$0.40	$ 2,995
5	Eductors Power @ 1.5 Hp	kWh	9,290	$0.10	$ 929
6	Repair Parts	LS	1	$1,500.00	$ 1,500
7	Reagents	LS	1	$900.00	$ 900
			Annual Project Total		**$ 11,931**
			Interest Rate		3.2%
			Project life		20
			Present Value O&M Cost		**$ 174,000**
				Winter UV O&M	$ 135,000
				Total HS + UV O&M =	**$ 309,000**

6 months (Summertime only) OSG (Chlorine) / Dechlorination Disinfection Annual & Present Worth O&M

Item	Operations & Maintenance Costs	Unit	Quantity	Unit Cost	Total Cost
1	Electricity	kWh	1,752	$0.10	$ 175
2	Labor @ 5 hrs per week	Hr	150	$28.00	$ 4,200
3	Salts	ton	3	$100.00	$ 308
4	Sulphur Dioxide 150 lb cylinders	lb	7,488	$0.40	$ 2,995
5	Eductors Power @ 1.5 Hp	kWh	9,290	$0.10	$ 929
6	Repair Parts	LS	1	$2,000.00	$ 2,000
7	Reagents	LS	1	$900.00	$ 900
			Annual Project Total		**$ 11,507**
			Interest Rate		3.2%
			Project life		20
			Present Value O&M Cost		**$ 168,000**
				Winter UV O&M	$ 135,000
				Total OSG + UV O&M =	**$ 303,000**

UV Disinfection Annual & Present Worth O&M

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Energy to Lamps	kWh	35,040	$	0.10	$	3,504
2	Weekly Lamp Cleaning	Hr	104	$	28	$	2,912
3	Lamp Replacement	EA	23	$	45	$	1,035
4	Ballast Replacement	EA	2	$	180	$	360
5	Quartz Replacement	EA	2	$	45	$	90
6	Monitoring & Operation	Hr	52	$	22	$	1,144
7	Semi Annual Basin Cleaning	Hr	8	$	22	$	176
			Operations & Maintenance			**$**	**9,221**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost			**$**	**135,000**

Summertime H/S Disinfection Onsite Generation Annual & Present Worth O&M

Item	Operations & Maintenance Costs	Unit	Quantity	Unit Cost	Total Cost
1	Electricity	kWh	768	$0.10	$77
2	Labor @ 4.5x per week	Hr	99	$28.00	$2,772
3	Hypochlorite Solution	lb	5600	$0.50	$2,800
4	Sulfur Dioxide 150 lb cylinders	lb	1120	$0.35	$392
5	Gas Eductors Power @ 1.5 Hp	kWh	4072	$0.10	$407
6	Repair Parts	LS	1	$600.00	$600
7	Reagents	LS	1	$200.00	$200
			Annual Project Total		**$2,849**
			Interest Rate		3.2%
			Project life		20
			Present Value O&M Cost		**$ 42,000**

RAS / WAS Pumping

RAS/WAS Power consumption information calculated

	RAS Pumps Return Rate of Q	
	Design Flow MGD RAS	0.86 MGD RAS
		600 GPM RAS
28	Design Head ft RAS - 50% eff	7.6 HP
	WAS Pumps	120.0 gpm
50	Design Head ft WAS - 30% eff	4.5 HP
	Scum / Sump Pumping	4.5 HP

RAS / WAS Pumping and Building Annual & Present Worth O&M

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	RAS Pump Power Demands	Kwh	49,836	$	0.10	$	4,984
2	WAS Pump Power Demands	Kwh	2,472	$	0.10	$	247
3	Scum / Sump Pumping	Kwh	2,472	$	0.10	$	247
4	Lights, HVAC, controls, inst. power	Kwh	12,000	$	0.10	$	1,200
5	Parts & Repair	LS	1	$	4,000	$	4,000
6	Monitoring & Operation	Hr	200	$	25	$	5,000
7	Maintenance Labor	Hr	250	$	25	$	6,250
			Operations & Maintenance			**$**	**21,928**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost			**$**	**320,000**

Sludge Alternatives

Dewatering Press Power Calculations

5	hp
375	lbs per hour
620,000	Lbs per year
1,653	operating hours per year
6,167	kwh per year

Dewatering Press Annual& Present Worth O&M

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Energy to Press	kWh	6,167	$	0.10	$	617
2	Energy to Misc. Elec. Load	kWh	2,000	$	0.10	$	200
3	Polymers	LS	1	$	8,000	$	8,000
4	Weekly Cleaning Labor	Hr	104	$	28	$	2,912
5	Monitoring & Operation	Hr	260	$	28	$	7,280
6	Maintenance Labor	Hr	20	$	55	$	1,100
7	Press Parts	LS	1	$	3,500	$	3,500
			Operations & Maintenance			**$**	**23,609**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost			**$**	**345,000**

Drying Beds Annual& Present Worth O&M

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Front end loader & truck O&M	Hr	208	$	130	$	27,040
2	Sludge Removal Labor	Hr	104	$	28	$	2,912
3	Weekly Cleaning Labor	Hr	10	$	28	$	280
4	Monitoring & Operation	Hr	104	$	28	$	2,912
5	Maintenance Labor	Hr	10	$	55	$	550
			Operations & Maintenance			**$**	**33,694**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost			**$**	**492,000**

Wet Tank Storage Annual& Present Worth O&M

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Energy to Aeration	kWh	30,967	$	0.10	$	3,097
2	Sludge Removal Labor	Hr	104	$	28	$	2,912
3	Weekly Cleaning Labor	Hr	52	$	28	$	1,456
4	Monitoring & Operation	Hr	52	$	28	$	1,456
5	Maintenance Labor	EA	80	$	55	$	4,400
			Operations & Maintenance			**$**	**13,321**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost			**$**	**194,562**
			Operations & Maintenance			**$**	**17,317**
			Present Value O&M Cost			**$**	**253,000**

Sludge Disposal Alternatives

Liquid Sludge Land Application Annual& Present Worth O&M (based on 990,000 gallons per year)

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Energy to Pump to Tanker	kWh	551	$	0.10	$	55
2	Tanker expense per trip	trip	330	$	65	$	21,450
3	Labor for spreading	Hr	660	$	28	$	18,480
4	Monitoring & Operation	Hr	36	$	28	$	1,008
5	Maintenance Labor	Hr	26	$	28	$	728
			Operations & Maintenance			**$**	**41,721**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost			**$**	**609,000**

Liquid Sludge Haul to Disposal Facility (1,269,000 gallons per year)

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Disposal Tipping Fee	gallons	1,269,000	$	0.08	$	101,520
4	Monitoring & Operation	Hr	127	$	28	$	3,553
5	Maintenance Labor	Hr	10	$	28	$	280
			Operations & Maintenance			**$**	**105,353**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost			**$**	**1,539,000**

Liquid Sludge Land Application Annual& Present Worth O&M (based on 2,259,000 gallons per year)

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Energy to Pump to Tanker	kWh	551	$	0.10	$	55
2	Tanker expense per trip	trip	750	$	65	$	48,750
3	Labor for spreading	Hr	1,500	$	28	$	42,000
4	Monitoring & Operation	Hr	36	$	28	$	1,008
5	Maintenance Labor	Hr	26	$	28	$	728
			Operations & Maintenance			**$**	**92,541**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost			**$**	**1,352,000**
			Land Acquistion			$	350,000
			Total present worth costCost			**$**	**1,702,000**

Liquid Sludge Haul to Disposal Facility (based on year round disposal)

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Disposal Tipping Fee	gallons	2,259,000	$	0.08	$	180,720
4	Monitoring & Operation	Hr	226	$	28	$	6,325
5	Maintenance Labor	Hr	10	$	28	$	280
			Operations & Maintenance			**$**	**187,325**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost			**$**	**2,736,000**

Dry Sludge Land Fill Annual & Present Worth O&M

Item	Description	Unit	Quantity	Unit Cost		Total Cost
1	Tipping Fee	ton	1,380	60	$	82,800
1	Front end loader op. expense	load event	160	50	$	8,000
2	Truck expense	Hr	320	50	$	16,000
3	Labor for driving	Hr	320	28	$	8,960
4	Monitoring & Operation	Hr	32	28	$	896
5	Maintenance Labor	Hr	24	28	$	672
			Operations & Maintenance		**$**	**117,328**
			Interest Rate			3.2%
			Project life			20
			Present Value O&M Cost		**$**	**1,714,000**

Liquid Stream Alternatives

SBR Electrical Power Demands
1886 Average kwh per day
688390

SBR Annual & Present Worth O&M

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Electrical Power Demands	Kwh	688,390	$	0.10	$	68,839
2	Parts & Repair	LS	1	$	2,000	$	2,000
3	Monitoring & Operation	Hr	500	$	28	$	14,000
4	Maintenance Labor	Hr	312	$	28	$	8,736
			Operations & Maintenance			**$**	**93,575**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost				**$ 1,367,000**

Conventional Treatment Power consumption information calculated

	BOD loading	2288 lb BOD/day
1.3	lb O2/1lb BOD	2974 lb O2/day
2.5	lb O2 per HP-HR	1190 HP-HR
	Aeration HP required	49.6 HP
	Mixing HP per aeration requirement	49.6 HP
	Total HP requirement	99.1 HP
0.747	kwh/HP-Hr	74.1 Kwh
	kw use/day.	1777.5 Kwh/day

Conventional Annual & Present Worth O&M

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Electrical Power Demands	Kwh	648,788	$	0.10	$	64,879
2	Parts & Repair	LS	1	$	2,000	$	2,000
3	Monitoring & Operation	Hr	500	$	28	$	14,000
4	Maintenance Labor	Hr	312	$	28	$	8,736
			Operations & Maintenance			**$**	**89,615**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost				**$ 1,309,000**

OxidationTreatment Power consumption information calculated

288	O2/1lb BOD per hour	6912 lb O2/day
2.5	lb O2 per HP-HR	2765 HP-HR
	Aeration HP required	200.0 HP
	Submerged Mixing HP	15.0 HP
	Total HP requirement	215.0 HP
0.747	kwh/HP-Hr	160.6 Kwh
	kw use/day.	3854.5 Kwh/day

Oxidation Ditch Annual & Present Worth O&M

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Electrical Power Demands *	Kwh	1,406,900	$	0.10	$	140,690
2	Parts & Repair	LS	1	$	2,000	$	2,000
3	Monitoring & Operation	Hr	364	$	28	$	10,192
4	Maintenance Labor	Hr	208	$	28	$	5,824
			Operations & Maintenance			**$**	**158,706**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost				**$ 2,318,000**

General Site and Housekeeping at Plant

Item	Description	Unit	Quantity		Unit Cost		Total Cost
1	Controls, Inst. power, HVAC, lights	Kwh	20,000	$	0.10	$	2,000
2	Parts & Repair	LS	1	$	5,000	$	5,000
3	Record keeping/ Reports	Hr	260	$	28	$	7,280
4	Lab work	Hr	312	$	28	$	8,736
5	Maintenance Labor	Hr	312	$	28	$	8,736
			Operations & Maintenance			**$**	**31,752**
			Interest Rate				3.2%
			Project life				20
			Present Value O&M Cost			**$**	**464,000**

City of Sutherlin 10/22/2013

Wastewater Collection System Improvements

The Dyer Partnership Engineers & Planners, Inc.

Everett Avenue Pump Station

Everett Avenue Pump Station

Item	Description	Unit	Quantity	Unit Cost	Total Cost
1	Const. Facilities & Temporary Controls	LS	1	$ 55,000	$ 46,000
2	Demolition & Site Preparation	LS	1	$ 45,000	$ 32,000
3	25hp submersible pumps (w/ Soft Start)	EA	3	$ 34,000	$ 102,000
4	Demo Existing Equipment / Piping	LS	1	$ 18,000	$ 18,000
5	Temporary Sheet Pile	LS	1	$ 21,000	$ 21,000
6	12-feet Φ Wet Well, 25' Depth w/ Traffic Rated Hatch	LS	1	$ 100,000	$ 100,000
7	Valve Vault 14'x8', Piping & Appurtenanced	LS	1	$ 26,000	$ 26,000
8	Valve Vault Hatch (Traffic Rated)	LS	1	$ 9,000	$ 9,000
9	15" Gravity Sewer Pipe - (Depth >10')	LF	60	$ 150	$ 9,000
10	New Fence and Gate	LS	1	$ 9,000	$ 9,000
11	Jib Crane	LS	1	$ 15,000	$ 15,000
16	Concrete Pad for Generator	CY	10	$ 800	$ 8,000
17	Eyewash Station	EA	1	$ 2,000	$ 2,000
18	Fill Existing Wet Well & Pipes Abandon (slurry)	LS	1	$ 5,000	$ 5,000
19	Level Control (pressure Transducer, Float Backup)	LS	1	$ 10,000	$ 10,000
20	Instrumentation & Controls (Alarm System)	LS	1	$ 35,000	$ 35,000
21	New 480V Power Drop & Connection	LS	1	$ 20,000	$ 20,000
22	New External Lighting	LS	1	$ 5,000	$ 5,000
23	Electrical	LS	1	$ 50,000	$ 50,000
24	Mechanical	LS	1	$ 60,000	$ 60,000

Construction Subtotal	**$**	**582,000**
Land Purchase/Survey	$	45,000
Engineering	$	121,000
Design Review	$	5,000
Contingency	$	92,000
Predesign Report	$	25,000
Geotech Report	$	15,000
Environmental Report	$	35,000
Permitting / Easement	$	5,000
Pump Station Total	**$**	**925,000**

Flood Profiles in the Calapooya Creek Basin, Oregon

U.S. GEOLOGICAL SURVEY
OPEN-FILE REPORT 82-439

Prepared in cooperation with
DOUGLAS COUNTY

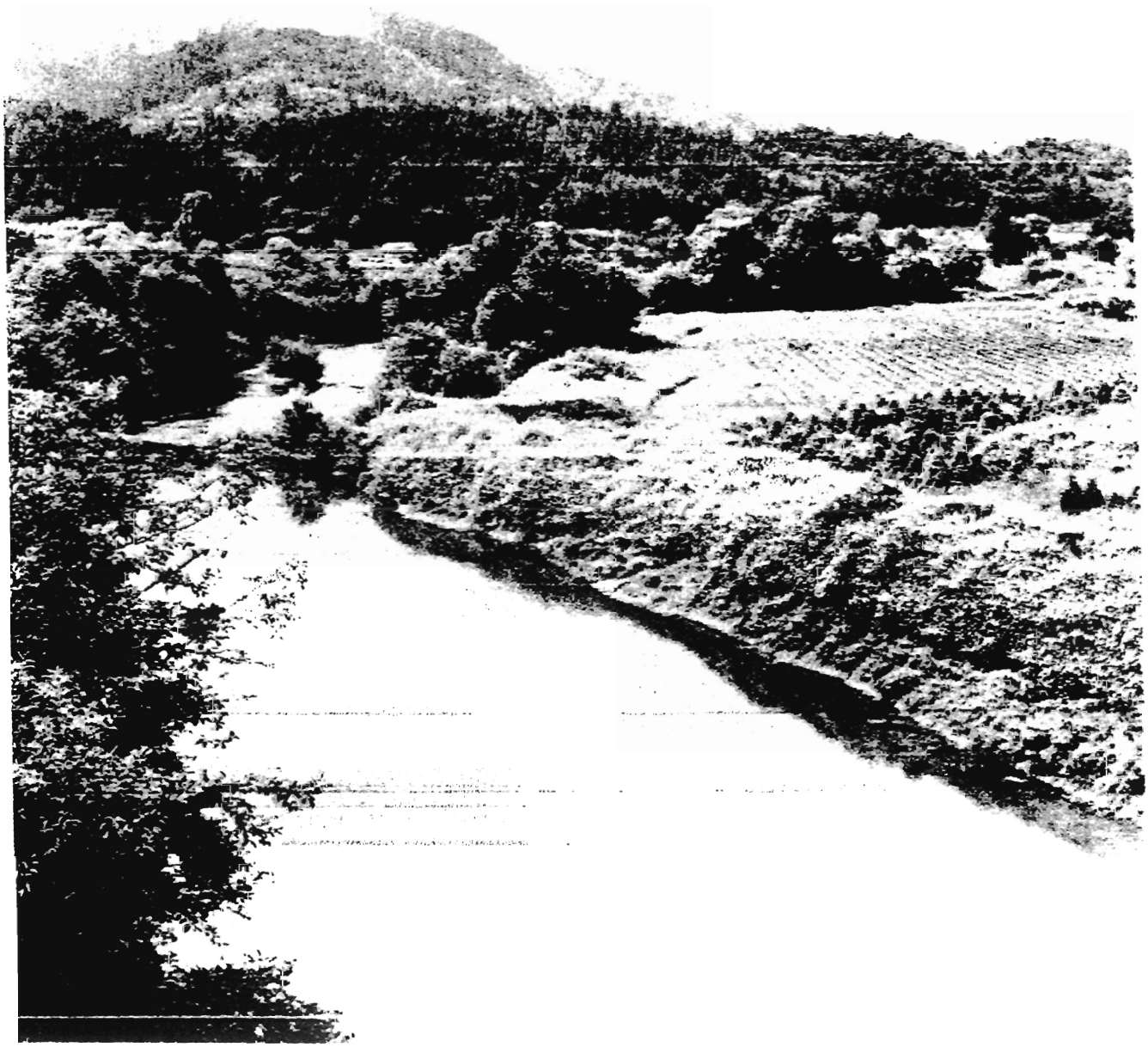


Table 2.--Calapooya Creek profile elevations--Continued

Cross Section		Elevation, in feet above sea level (NGVD)					
Number	Distance [1]	2-year flood	10-year flood	50-year flood	100-year flood	500-year flood	1961 flood [2]
--	37,860	Coon Creek		--	--	--	--
35	38,306	373.3	377.4	379.2	379.9	381.7	378.0
36	39,122	373.9	378.3	380.2	380.9	382.6	378.9
37	39,896	374.6	379.3	381.4	382.2	384.1	380.0
38	40,829	375.4	380.7	383.0	383.7	385.5	381.5
39	41,710	376.6	382.2	384.5	385.3	387.1	383.0
40	42,577	377.9	383.1	385.4	386.2	388.0	383.9
41	43,530	379.4	384.5	386.8	387.6	389.4	385.3
42	44,955	381.3	385.9	388.0	388.9	390.7	386.6
--	45,430	Dodge Canyon		--	--	--	--
43	47,857	382.2	386.5	388.7	389.6	391.5	387.2
44	48,117	382.3	386.6	388.8	389.7	391.6	387.3
--	48,261	State Highway No. 138		--	--	--	--
45	48,565	382.5	387.1	390.0	390.9	392.6	388.1
46	50,011	383.2	387.6	390.4	391.3	393.0	388.5
47	51,109	384.0	388.4	391.0	391.9	393.6	389.2
48	52,842	387.1	391.9	394.6	395.4	397.3	392.9
49	53,208	388.1	392.3	394.9	395.7	397.6	393.2
--	53,262	County Road No.10 and gaging station 14-3207.00 (14 feet downstream)					--
50	53,472	388.6	393.0	395.7	396.5	398.3	394.0
51	54,386	389.1	393.5	396.3	397.2	399.0	394.5
52	55,406	390.3	394.4	397.1	397.9	399.7	395.3
53	56,524	392.0	395.7	398.0	398.8	400.6	396.5
54	57,711	392.6	396.4	398.6	399.3	401.1	396.9
--	58,020	Williams Creek		--	--	--	--
55	59,582	393.6	397.4	399.5	400.2	401.8	398.1
56	60,804	394.8	398.4	400.2	400.8	402.4	399.0
57	61,850	395.6	399.4	401.0	401.6	403.0	399.9
58	62,869	397.1	401.6	403.0	403.4	404.6	402.1
59	63,879	398.4	403.6	405.4	406.0	407.4	404.2
60	64,941	399.4	404.8	407.0	407.7	409.1	405.6
61	66,225	401.0	405.8	407.9	408.6	410.1	406.5
62	67,111	401.8	406.3	408.5	409.3	410.9	407.0
63	67,668	402.0	406.4	408.6	409.4	411.0	407.1
64	68,217	402.4	406.6	408.9	409.7	411.4	407.4
--	68,383	Interstate Highway I-5		--	--	--	--
65	68,768	403.2	408.2	411.5	412.8	416.1	409.3
66	69,446	404.4	408.8	411.9	413.2	416.3	409.8

WWTP

[1] Measured in feet upstream from confluence with Umpqua River.

[2] November 23, 1961.

FLOOD PROFILES IN THE CALAPOOYA CREEK BASIN, OREGON

--

By John Friday

--

ABSTRACT

Water-surface profiles were computed for a 19.4-mile reach of Calapooya Creek in Douglas County, Oreg. The data will enable the county to evaluate flood hazards in the flood-prone areas in the reach. Profiles for floods having recurrence intervals of 2, 10, 50, 100, and 500 years are shown in graphic and tabular form. A floodway, allowing encroachment of the 100-year flood, was designed with a maximum 1.0-foot surcharge limitation. A profile for a flood that occurred in November 1961 is also presented. All data were derived from a digital computer model developed for the study.

INTRODUCTION

This study was made at the request of the Douglas County Water Resources Survey. The flood-profile data will enable the county to evaluate the flood hazards in the lower reaches of Calapooya Creek. A knowledge of these elevations and their related flood boundaries is essential for administering land-use measures in the flood plains. A well-managed flood plain can reduce the risk of losses of life and property by controlling the location of future developments in flood-prone areas.

Wherever possible, this study has been made compatible with a Flood Insurance Study (FIS) completed for the unincorporated areas of Douglas County (U.S. Federal Emergency Management Agency, 1978). In that study, data for Calapooya Creek were derived by approximate methods, based in part on flood elevations computed in an earlier profile study of the Umpqua River (Oster, 1972). The data presented herein supersede the approximate study and follows a format suitable for a subsequent Flood Insurance Study should the need arise.

SCOPE OF STUDY

The study reach extends from the mouth of Calapooya Creek to a point 639 ft upstream from Oldham Creek (river mile 19.4). See figure 1. Water-surface elevations were computed for floods that would occur on the average of once every 2, 10, 50, 100, and 500 years; a profile documenting a flood that occurred in November 1961 was also a requirement of this study. A floodway analysis was made to determine the areas that must be kept free from encroachment in order to pass the 100-year flood without exceeding a specified 1.0-foot surcharge.

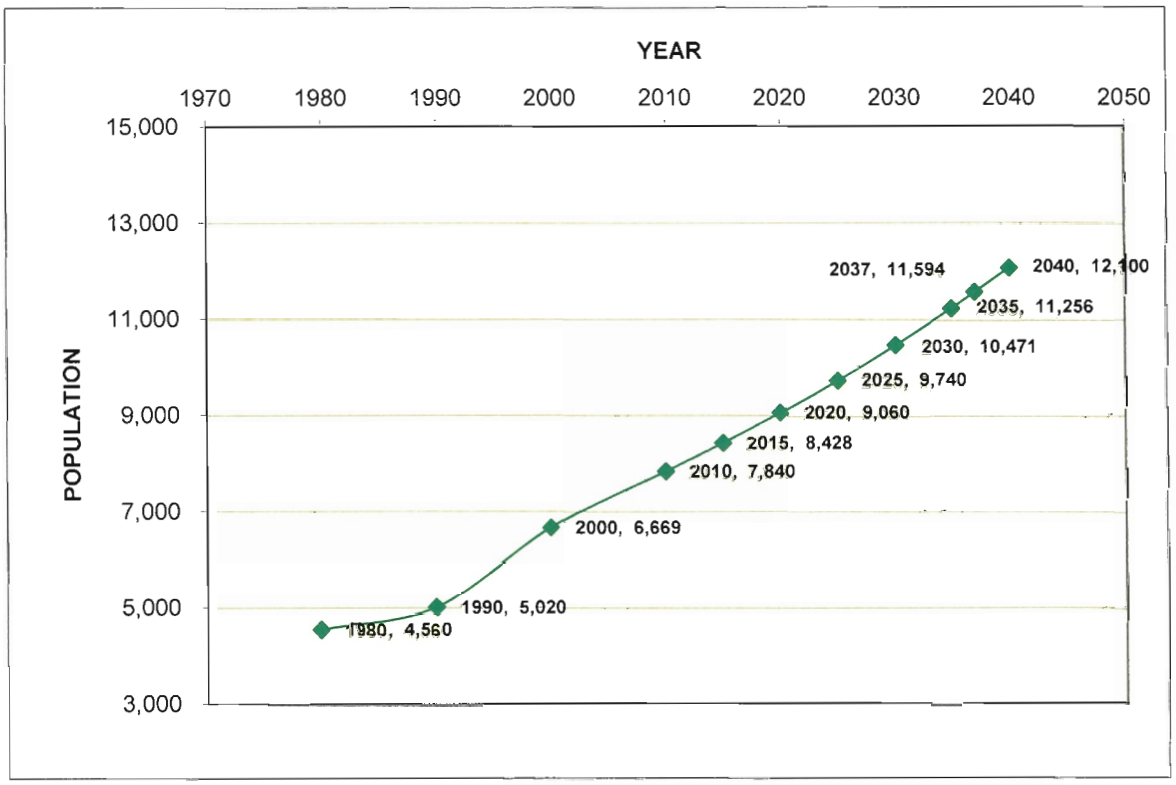
The accuracy limit of all profile elevations is ±0.5 ft; all elevations are referenced to the National Geodetic Vertical Datum* of 1929 (NGVD).

* ADD 3ft TO NGVD TO OBTAIN NAVD 88

Population Forecast based on Historic Growth Rate

By: Tom Hart Date: 29-Mar-13

1. Historic population and forecasted estimate population graph



2. Historical Population Growth Table

Year	Douglas County	City
1980	93,748	4,560
1990	94,649	5,020
2000	100,397	6,669
2010	107,690	7,840
2015	112,043	8,428
2020	117,632	9,060
2025	123,341	9,740
2030	129,062	10,471
2035	134,371	11,256
2040	140,619	12,100

2. Calculations

	Douglas County			City		
Year	Pop	Increase	Annual Rate	Pop	Increase	Annual Rate
1980	93,748			4,560		
1990	95,050	1,302	0.14%	5,020	460	0.92%
2000	100,579	5,529	0.55%	6,669	1,649	2.47%
2010	107,690	7,111	0.70%	7,840	1,171	1.50%
2015	110,549	2,859	**0.52%**	8,428	588	**1.50%**
2020	115,841	5,292	**0.91%**	9,060	632	**1.50%**
2025	121,208	5,367	**0.89%**	9,740	680	**1.50%**
2030	125,929	4,721	**0.75%**	10,471	731	**1.50%**
2035	129,695	3,766	**0.58%**	11,256	785	**1.50%**
2037				11,594	338	**1.50%**
2040	132,840	3,145	**0.47%**	12,100	844	**1.50%**

Notes:

1 Douglas County 1980 thru 2050 population is taken from the Office of Ecomnomic Analysis, Department of Administration Services, State of Oregon, Release Date March 28, 2013

2 City of Sutherlin population from 1980 to 2010 was prepared by the Population Research Center, PSU March 2012

3 City of Sutherlin Population projection from 2010 thru 2040 is based on the 2000 to 2010 growth rate of 1.5%

4 Potential annexation to the city limits may add 300 people.

Forecasts of Oregon's County Populations and Components of Change, 2010 - 2050

Prepared by Office of Economic Analysis, Department of Administrative Services, State of Oregon

Release date: March 28, 2013

	Estimate							PSU'S ESTIMATES		FORECAST							
Area Name	1980	1985	1990	1995	2000	2005	2010	2011	2012	2015	2020	2025	2030	2035	2040	2045	2050
Oregon	2,642,128	2,672,648	2,860,375	3,184,369	3,431,100	3,626,900	3,837,300	3,857,625	3,883,735	4,001,600	4,252,100	4,516,200	4,768,000	4,995,200	5,203,000	5,398,800	5,588,500
Douglas	93,837	91,956	95,050	98,874	100,579	104,253	107,690	107,795	108,195	110,562	116,113	121,715	126,642	130,550	133,786	136,710	139,675

POPULATION CHANGE

	Estimate								FORECAST							
	1980-1985	1985-1990	1990-1995	1995-2000	2000-2005	2005-2010	2010-2011	2011-2012	2010-2015	2015-2020	2020-2025	2025-2030	2030-2035	2035-2040	2040-2045	2045-2050
Oregon	30,520	187,727	323,994	246,731	195,800	210,400	20,325	26,110	164,300	250,500	264,100	251,800	227,200	207,800	195,801	189,700
Douglas	-1,881	3,094	3,824	1,705	3,674	3,437	105	400	2,872	5,551	5,602	4,927	3,907	3,236	2,925	2,964

ANNUALIZED GROWTH RATES

	Estimate								FORECAST								
Area Name	1980-1985	1985-1990	1990-1995	1995-2000	2000-2005	2005-2010	2010-2011	2011-2012	2010-2015	2015-2020	2020-2025	2025-2030	2030-2035	2035-2040	2040-2045	2045-2050	2010-2012
Oregon	0.23%	1.36%	2.15%	1.49%	1.11%	1.13%	0.53%	0.68%	0.84%	1.21%	1.21%	1.09%	0.93%	0.82%	0.74%	0.69%	0.60%
Douglas	-0.40%	0.66%	0.79%	0.34%	0.72%	0.65%	0.10%	0.37%	0.53%	0.98%	0.94%	0.79%	0.61%	0.49%	0.43%	0.43%	0.23%

PLANNING DEPARTMENT

Room 106 • Justice Building • Douglas County Courthouse
Roseburg, Oregon 97470

Agency Coordination • Administrative • Long Range • Support Services
(541) 440-4289 • (541) 440-6266 Fax

On-Site Services (541) 440-6183
Community Services (541) 464-6443
(541) 464-6429 Fax

March 28, 2012

Dan Huff, Community Development Director
City of Sutherlin
126 E. Central Ave
Sutherlin, Oregon 97479

RE: Population Forecast Coordination

Dear Dan:

The 2009 coordinated population growth allocation for the City of Sutherlin was 1.5%. As part of that coordination, the County recognized that cities, when completing a capital facility plan, typically consider a thirty-year period to budget for infrastructure improvements. For capital facilities planning, the City of Sutherlin may consider a capital facilities population forecast of 2.0%.

As you are aware, the 2009 & 2010 County Population forecasts were subject to appeal and remand. The last twenty-year population forecast update acknowledged for the County was in 2000. At that time, the coordinated rate for the City of Sutherlin (2000 County population forecast) was a growth rate of 2.7 percent.

The City of Sutherlin has proposed to use the 1.5% population forecast as part of its present capital facilities planning process. The County has no objections to the City using that rate of growth. We will update the coordinated growth rates for cities in the Douglas County Comprehensive Plan Population Element when we move forward with our Population Element plan amendment process.

Sincerely,

John J. Boyd AICP
Senior Planner

c Keith L. Cubic, Planning Director

H:\LongRange\Projects\Pop_Forecast\Pop2012\Sutherlin_032712.wpd

Table 4. Populations for Oregon and Its Counties and Incorporated Cities: July 1, 2010 - July 1, 2011 estimates; and Census Counts 1990-2010

Prepared by Population Research Center, PSU, March 2012.

County and Cities	July 1 Population Estimates		April 1 Census Population		
	2011	2010rev	2010	2000	1990
OREGON	3,857,625	3,837,300	3,831,074	3,421,399	2,842,321
Unincorporated	51,920	51,240	51,188	48,898	46,638
DOUGLAS	107,795	107,690	107,667	100,399	94,649
Canyonville	1,885	1,885	1,884	1,293	1,219
Drain	1,155	1,155	1,151	1,021	1,086
Elkton	195	195	195	147	172
Glendale	875	875	874	855	707
Myrtle Creek	3,440	3,440	3,439	3,419	3,063
Oakland	925	925	927	954	844
Reedsport	4,145	4,150	4,154	4,378	4,796
Riddle	1,185	1,185	1,185	1,014	1,143
Roseburg	21,690	21,660	21,181	20,017	17,069
Sutherlin	7,880	7,840	7,810	6,669	5,020
Winston	5,385	5,385	5,379	4,613	3,773
Yoncalla	1,055	1,050	1,047	1,052	919
Unincorporated	57,980	57,945	58,441	54,967	54,838

Please use caution when comparing the population estimates of the unincorporated county areas, and note that the population estimates for the unincorporated areas represent revised estimates rather than estimates resulting from measured direct change during the year. The unincorporated area is a residual of the county total and the sum of the cities; thus the unincorporated area population is affected by those cities not submitting to PRC the completed annual form since the population of a city is held constant to the previous year if no recent form data are received. Additionally, annexations affect the unincorporated population.

Brian Elliott

From: Dan Huff
Sent: Wednesday, November 21, 2012 9:28 AM
To: Jerry Gillham; v.luther@ci.sutherlin.or.us; Debbie Hamilton (d.hamilton@ci.sutherlin.or.us); Kirk Sanfilippo; Ron Harker (r.harker@ci.sutherlin.or.us); Billie Wright; Brian Elliott; Randy Harris; Chris Berquist
Subject: Sutherlin Population

Sutherlin's new population estimate as determined by Portland State University Center for Population Research is **7,905**

Record e-mail

From: LeBombard, Josh [mailto:josh.lebombard@state.or.us]
Sent: Friday, May 31, 2013 10:14 AM
To: Tom Hart
Cc: 'ISAZA Jaime'
Subject: RE: City of Sutherlin Population Projection.

Tom,

... The population forecast that you prepared for the City of Sutherlin is acceptable.

Cheers,
Josh

Josh LeBombard | Southern Oregon Regional Representative
Community Services Division
Oregon Dept. of Land Conservation and Development
Southern Oregon Regional Solution Center
c/o Jobs Council, Southern Oregon University
100 E Main Street, Suite A | Medford, OR 97501
Cell: (541) 414-7932
josh.lebombard@state.or.us | www.oregon.gov/LCD

From: Tom Hart [thart@dyerpart.com]
Sent: Thursday, May 23, 2013 10:58 AM
To: LeBombard, Josh
Cc: 'ISAZA Jaime'
Subject: City of Sutherlin Population Projection.

Josh LeBombard | Southern Oregon Regional Representative
Community Services Division
Oregon Dept. of Land Conservation and Development
Southern Oregon Regional Solution Center

Hi Josh:

I have updated Sutherlin's population projections based on your annexation comments and Kanhaiya Vaidya (Office of Economic Analysis) comments. Kanhaiya thought a projection based on city historical trends would be more appropriate rather than a Douglas County Safe Harbor projection. Attached is the revised estimate.

DEQ requires DLCD acknowledgement than this Sutherlin population projection is acceptable to DLCD. Please confirm the estimate is acceptable for a WWFP.

Thanks,

Tom Hart
The Dyer Partnership

City of Sutherlin WWFP

2009 Sludge Projections

By: Tom Hart Date: 3-May-10

1 Pounds of BOD removed by WWTP based on DMR's

Influent BOD mg/l - Effluent BOD **x** flow in mgd x 8.34 **=** lbs/day BOD removed

*	152	14	0.93	=	1,065 lbs BOD removed
	Influent BOD	Eff. BOD	flow		

*Based on 2009 average from **2006-2009 DMR's Summary**

2 Solids produced in lbs/day

lbs/day of BOD removed **x** process conversion factor **=** lbs of solids produced

Process Conversion Factors

Process	lbs soilds / lb BOD Removed
Primary	1.7
Activated Sludge with Primary	0.7
Activated Sludge without Primary	
Conventional	0.85
Step Feed	0.85
Extended Aeration	0.65
Oxidation Ditch	0.65
Contact Stablization	1
Acitvated Sludge - SBR	0.8

Contact Stablization Process Converstion Factor = 1.00

then : 1,065 lbs of solids produced

3 Daily Wasting Rates

lbs/day of solids produced / (waste sludge concentration in mg/l **x** 8.34) **x** 1,000,000 **=**

1,065	6,571	19,435 gallons per day of waste
lbs solids produced	waste conc. Mg/l from **2006 - 2009 DMR's Summary**	

City of Sutherlin WWFP

2009 Sludge Projections

By: Tom Hart Date: 3-May-10

4 Digester Sludge Retention Detention Time (SRT)

Digester capacity in gallons / waste activated sludge = SRT

197,400	19,435	0.5
digester cap. in gallons	waste gallons/d	Was/Ras Ratio

9,717 WAS gal/day

20 **SRT in days**

Note: assumes no exporting to an offsite treatement facility, nor decanting

5 lbs of dry solids removed per day and year (in Digester)

1,065 lbs BOD produced	0.65 .Reduction factor (Aeration Digester)	**372.8**	**lbs/day removal**
		68.0	tons/year removed

City of Sutherlin WWFP

2006-2009 DMR Summary / Mass Balance

By: Tom Hart

Date: 1-May-09

	DMR's:						Mass Balance:						
2006	Plant Flows daily Average	Influent BOD's mg/l	Effluent BOD's mg/l	Influent TSS's mg/l	Effluent TSS's mg/l	Biosolids Hauled gallons	Daily Sludge out lbs/day	Daily Sludge produced lbs/day (ext. air)	Daily Sludge produced lbs/day (cont stab)	Aeration Waist TSS's mg/l	Est. Gal. Waste per day	Est. Avg. Decant gal/day	Est. SRT in digester in days
January	2.582	143	13.7	61	12.3	0	2784	1810	2784	7500	28934	14467	14.2
February	1.334	108	11	133	11	97000	1079	701	1079	8299	10135	5067	40.7
March	1.515	102	10.9	109	8.6	28800	1151	748	1151	7870	11399	5700	36.1
April	1.220	111	8.4	119	10	25600	1044	679	1044	7450	10921	5461	37.7
May	0.934	149	19	183	17.1	60800	1013	658	1013	7526	10487	5243	39.3
June	0.840	171	14.8	218	12.1	89600	1094	711	1094	9030	9445	4722	43.6
July	0.690	190	18.8	175	15.9	86400	985	640	985	8288	9264	4632	44.5
August	0.652	237	13.7	225	13.3	89600	1214	789	1214	8605	10998	5499	37.5
Sept.	0.645	209	11.6	266	13.1	192000	1062	690	1062	6846	12089	6044	34.1
Oct.	0.687	224	13.7	244	19.1	86400	1205	783	1205	5588	16806	8403	24.5
Nov.	1.733	101	7	115	6.9	16000	1359	883	1359	6929	15282	7641	27.0
Dec.	1.928	91	7.3	107	8.9	73600	1346	875	1346	7878	13315	6657	30.9
Totals	14.760					**845800**							
Averages	**1.230**	**153**	**12**	**163**	**12**	**70483**	**1278**	**831**	**1278**	**7651**	**16601**	8300	**34**
Min	0.645	91	7	61	7	0	985	640	985	5588	12797	6399	14
Max	**2.582**	237	19	266	19	192000	2784	1810	2784	9030	36167	18084	44

City of Sutherlin WWFP

2006-2009 DMR Summary / Mass Balance

By: Tom Hart

Date: 1-May-09

	DMR's:						Mass Balance:						
2007	Plant Flows daily Average	Influent BOD's mg/l	Effluent BOD's mg/l	Influent TSS's mg/l	Effluent TSS's mg/l	Biosolids Hauled gallons	Daily Sludge out lbs/day	Daily Sludge produced lbs/day (ext. air)	Daily Sludge produced lbs/day (cont stab)	Aeration Waist TSS's mg/l	Est. Gal. Waste per day	Est. Avg. Decant gal/day	Est. SRT in digester in days
January	1.574	101	7.5	110	7.6	44800	1227	798	1227	9365	10215	5107	40.3
February	1.658	123	6.8	123	6.6	76800	1607	1044	1607	9830	12739	6370	32.3
March	1.397	133	7.4	130	5.8	92800	1463	951	1463	10243	11135	5567	37.0
April	1.070	175	6.6	171	2.4	0	1503	977	1503	9870	11866	5933	34.7
May	0.831	190	8.2	212	6.2	57600	1260	819	1260	8868	11073	5537	37.2
June	0.692	246	6.6	258	5.8	156800	1382	898	1382	9156	11761	5880	35.0
July	0.681	227	11.8	315	10.6	86400	1222	794	1222	8823	10797	5398	38.2
August	0.596	255	11.9	368	10.9	176000	1208	785	1208	8574	10984	5492	37.5
Sept.	0.593	295	10.8	390	12.8	89600	1406	914	1406	8118	13494	6747	30.5
Oct.	0.864	182	10.4	177	8.6	186800	1237	804	1237	6826	14118	7059	29.2
Nov.	1.172	143	11	149	6.5	83200	1290	839	1290	6664	15090	7545	27.3
Dec.	1.915	82	10.3	88	5.8	54400	1145	744	1145	7043	12672	6336	32.5
Totals	13.043					**1105200**							
Averages	**1.087**	**179**	**9**	**208**	**7**	**92100**	**1329**	**864**	**1329**	**8615**	**17265**	8633	**34**
Min	0.593	82	7	88	2	0	1145	744	1145	6664	14875	7437	27
Max	**1.915**	295	12	390	13	186800	1607	1044	1607	10243	20871	10436	40

City of Sutherlin WWFP

2006-2009 DMR Summary / Mass Balance

By: Tom Hart

Date: 1-May-09

2008	Plant Flows	Influent BOD's	Effluent BOD's	Influent TSS's	Effluent TSS's	Biosolids Hauled	Daily Sludge out lbs/day	Daily Sludge produced lbs/day (ext. air)	Daily Sludge produced lbs/day (cont stab)	Aeration Waist TSS's mg/l	Est. Gal. Waste per day	Est. Avg. Decant gal/day	Est. SRT in digester in days
January	2.093	86	9.5	85	5.8	86400	1335	868	1335	7800	13343	6671	30.9
February	1.351	126	8	125	30	0	1330	864	1330	7847	13205	6603	31.2
March	1.330	126	11	137	11.2	0	1276	829	1276	7663	12974	6487	31.8
April	0.934	159	9.8	169	30	0	1162	755	1162	7467	12131	6065	34.0
May	0.698	187	11	203	10.5	0	1025	666	1025	7481	10674	5337	38.6
June	0.618	233	11.7	219	30	83200	1141	741	1141	8149	10909	5454	37.8
July	0.561	199	19.2	246	19.2	92800	841	547	841	7875	8326	4163	49.5
August	0.549	227	16.5	291	1.8	348800	964	626	964	7452	10080	5040	40.9
Sept.	0.530	230	6.4	290	14.8	86400	988	642	988	6595	11680	5840	35.3
Oct.	0.551	192	9.8	277	10.3	185600	837	544	837	4895	13331	6665	30.9
Nov.	0.902	121	9.4	155	6.8	0	840	546	840	5861	11164	5582	36.9
Dec.	1.517	103	9.6	149	7	92800	1182	768	1182	6913	13322	6661	30.9
Totals	11.634					**976000**							
Averages	**0.970**	**166**	**11**	**196**	**15**	**81333**	**1077**	**700**	**1077**	**7167**	**13985**	6993	**36**
Min	0.530	86	6	85	2	0	837	544	837	4895	10876	5438	31
Max	**2.093**	233	19	291	30	348800	1335	868	1335	8149	17346	8673	49

City of Sutherlin WWFP

2006-2009 DMR Summary / Mass Balance

By: Tom Hart

Date: 1-May-09

	DMR's:						Mass Balance:						
2008-2009	Plant Flows daily Average	Influent BOD's mg/l	Effluent BOD's mg/l	Influent TSS's mg/l	Effluent TSS's mg/l	Biosolids Hauled gallons	Daily Sludge out lbs/day	Daily Sludge produced lbs/day (ext. air)	Daily Sludge produced lbs/day (cont stab)	Aeration Waist TSS's mg/l	Est. Gal. Waste per day	Est. Avg. Decant gal/day	Est. SRT in digester in days
Jan. 2009	1.155	106	8	151	6	92800	944	661	944	7460	10621	5311	38.8
Feb. 2009	1.209	113	9	141	8	0	1049	734	1049	6850	12849	6424	32.1
March.2009	1.463	72	9	109	9	73600	769	538	769	6777	9520	4760	43.3
April.2009	0.927	129	11	165	5	0	912	639	912	6238	12275	6137	33.6
May. 2009	0.812	138	11	198	9	89400	860	602	860	6572	10984	5492	37.5
June. 2009	0.661	178	18	201	20	96000	882	617	882	7104	10421	5211	39.5
July. 2009	0.564	219	27	302	21	182400	903	632	903	7815	9700	4850	42.5
Aug. 2009	0.591	251	34	257	17	345600	1070	749	1070	6819	13165	6583	31.3
Sept. 2009	0.579	223	17	233	11	92800	995	696	995	5550	15044	7522	27.4
Oct. 2009	0.725	173	9	200	6	115200	992	694	992	4895	17003	8502	24.2
Nov. 2008	0.902	121	9.4	155	6.8	0	840	588	840	5861	12023	6011	34.3
Dec. 2008	1.517	103	9.6	149	7	92800	1182	827	1182	6913	14347	7174	28.7
Totals	11.1050					**1180600**							
Averages	**0.9254**	**152**	**14**	**188**	**10**	**98383**	**950**	**665**	**950**	**6571**	**13285**	6642	**34**
Min	0.5640	72	8	109	5	0	769	538	769	4895	10753	5377	24
Max	**1.5170**	251	34	302	21	345600	1182	827	1182	7815	16530	8265	43

City of Sutherlin WWFP

2035 Sludge Projections

By: Tom Hart Date: 13-May-10

1 Influent BOD in lbs per day

Influent BOD mg/l x Flow mgd x 8.34 = lbs/day of BOD

*	170	1.38	8.34	=	**1,949** lbs BOD per day
	Influent BOD	flow			

*Based on Chapter 4 Unit Design Factors for average Waste Water Composition

2 Effluent BOD in lbs per day

Effluent BOD mg/l x Flow mgd x 8.34 = lbs/day of BOD

10	1.38	8.34	=	**115** lbs BOD per day
Effluent BOD	flow			

3 Sludge to Digester Biological Calculation

Amount of Sludge Produced, lbs per day = (Influent BOD - Effluent BOD) x Yield Factor

$$\text{Volume of Sludge Produced, gpd} = \frac{\text{Amount of Sludge produced}}{\text{Solids Fraction} \times 8.34 \text{ lbs per million gal.}}$$

Process Conversion Factors

Process	lbs soilds / lb BOD Removed
Primary	1.7
Activated Sludge with Primary	0.7
Activated Sludge without Primary	
Conventional	0.85
Step Feed	0.85
Extended Aeration	0.65
Oxidation Ditch	0.65
Contact Stablization	1
Acitvated Sludge - SBR	0.8

Given:

Design Month Influent BOD Loading	1,949	lbs per day
Effluent BOD, lbs per day	115	lbs per day
Process BOD conversion factor	0.8	(SBR)
Solids Fraction	0.0085	

Calc'd:

Amount of Sludge Produced	1,468	lbs per day
Volume of Sludge Produced (was)	**20,706**	**gpd**

4 Digester and Sludge available volumes

Refurbish existing donuts and convert existing contact tank, digester tanks and stabilization tanks to sludge digester tanks.

			SWD		
Digester Tank:	950	sf per unit	14	99,497	gallons per unit
Stabilization Tank:	1,900	sf per unit	14	198,995	gallons per unit
Contact Tank:	950	sf per unit	12	85,283	gallons per unit
Clarifier Tank:	1,380	sf per unit	11	113,562	gallons per unit
				497,337	gal. per treatment unit
				2	Units
				994,674	total gallons

5 Digester Sludge Retention Detention Time (SRT) without decanting / thickening

Digester capacity in gallons / waste activated sludge = SRT

994,674	20,706		**48**	**SRT in days**
digester cap. in gallons	waste gallons/d			

City of Sutherlin WWFP

2035 Sludge Projections
based on availble exisiting tanks

By: Tom Hart Date: 15-May-10

1 Digester and Sludge Storage Volume

Refurbish existing donuts and convert existing contact tank, digester tanks and stabilization tanks to sludge digester tanks.

			SWD		
Digester Tank (DI):	950	sf per unit	14	99,497	gallons per unit
Stabilization Tank (ST):	1,900	sf per unit	14	198,995	gallons per unit
Contact Tank (CT):	950	sf per unit	12	85,283	gallons per unit
Clarifier Tank (CL):	1,380	sf per unit	11	113,562	gallons per unit
				497,337	TOTAL GALLONS per unit

2 Sludge Thickening and Sludge Retention Time

Digester 1

City of Coquille WWTP Sludge Storage Quantity			Volume for Storage								
DECANT THICKENING			DI 1	ST 1	CT 1	CL 1	DI 2	ST 2	CT 2	CL 2	Sub-Total
YEAR 2004			gallons	gallons	gallons	gallons	gallons	gallons	gallons	gallons	gallons
			99,497	198,995	85,283	113,562	99,497	198,995	85,283	113,562	994,674
Daily Generation from SBR			Fill Time								
WAS	Solid	Solids	AB	SCL	AD	CL 1	SS	SS	CL 1	CCL	Total
gallons	%	pounds	days	days	days	days	days	days	days	days	days
20,706	0.85%	1,468	5				5				10

Stablizaton Tank 1

30% Reduction of Solids from SBR Generation

Daily Generation from SBR			Fill Time								
Sludge	Solid	Solids	DI 1	ST 1	CT 1	CL 1	DI 2	ST 2	CT 2	CL 2	Sub-Total
gallons	%	pounds	days	days	days	days	days	days	days	days	days
9,856	1.25%	1,027		20				20			40

Contact Tank 1

35% Reduction of Solids from SBR Generation

Daily Generation from SBR			Fill Time								
Sludge	Solid	Solids	DI 1	ST 1	CT 1	CL 1	DI 2	ST 2	CT 2	CL 2	Sub-Total
gallons	%	pounds	days	days	days	days	days	days	days	days	days
7,803	1.50%	976			11				11		22

Clarifier Tank 1

40% Reduction of Solids from SBR Generation

Daily Generation from SBR			Fill Time								
Sludge	Solid	Solids	DI 1	ST 1	CT 1	CL 1	DI 2	ST 2	CT 2	CL 2	Sub-Total
gallons	%	pounds	days	days	days	days	days	days	days	days	days
5,559	2.00%	927				20				20	40
							Total Days of Sludge Storage:				112

Future Tank

40% Reduction of Solids from SBR Generation

Daily Generation from SBR			Fill Time								
Sludge	Solid	Solids								CL 2	Sub-Total
gallons	%	pounds	Future Tank Volume =		900000	Gallons				days	days
5,559	2.00%	927								162	162
							Total Days of Sludge Storage w/ new tank:				274

City of Sutherlin
Wastewater Treatment Plant
Existing Units
Design Data

			2009	2035	
Population Equivalents			7950	11708	
Gallons/day @ 100 gal/cap/day					
BOD - lbs/day/capita		0.15			
BOD in total lbs/day					
Aerobic Digester		each unit	(2)		
Liquor Depth	SWD ft.	14			
Volume	Gallons	98,700			
Cu. Ft. /capita	cu.ft.	3			
Aeration Tower	CFM	92.5			
Aeration Tower with 4"ø air supply pipe					
Contact Chamber (activated sludge)		each unit	(2)		
Liquor Depth	SWD ft.	12			
Volume	Gallons	85,000			
Retention Time	Hours	1.5			
Aeration Capacity	CFM	85			
Aeration Tower with 3"ø air supply pipe					
Reaeration Chamber		each unit	(2)		
Liquor Depth	SWD ft.	14			
Volume	Gallons	199000			
Retention Time	Hours	7.2			
Aeration Capacity	CFM	85			
2 Aeration Towers with 3"ø air supply pipe					
Secondary Clarifier		each unit	(2)		
Diameter	ft.	40			
Liquor Depth	SWD	11			
Volume	Gallons	103000			
Retention Time	Hours	1.9			Assumes one unit is off
Surface Area	sq. ft.	1300			Assumes one unit is off
Surface Settling Rate	gal/sq.ft./day	980			Assumes one unit is off
Weir Overflow Rate	gal/ft/day	10110			Assumes one unit is off
Influent & Outfall pipe size - inches		27			

Wastewater Reuse Calculations

(combination storage pond and irrigation)

By: Tom Hart Date: 6/25/2013

Per the NPDES permit effluent discharge to Calapooia Creek will not be allowed from June 1st to October 31st. Assume the Irrigation season is from June 1st to October 31st. Based on 2040 Average Dryweatehr Flows flows of 1.11 MGD.

Imput Data

Irrigation Field Area (FA):	200 **Acres Req.d with 20% SF=**		240.00
Pond Area (PA):	**Acres =**	30	
	PA =	1,306,800 **sf**	

Computational Table 1
Reuse Water Inflow ADWF: 1.11 MGD

Month	Days	Average Rainfall, inches	Pastures ET* & Pond Evaporation, inches/mth	Inflow Gallons Wastewater gallons	Inflow Gallons Field Rainwater	Inflow Gallons Pond Rainwater
(1)	(2)	(3)	(4)	(5)	(6)	(7)
May						
June	30.00	1.86	4.96	33,300,000	10,102,000	1,515,300
July	31.00	0.52	6.65	34,410,000	2,824,200	423,600
August	31.00	0.16	5.59	34,410,000	869,000	130,300
September	30.00	0.53	3.89	33,300,000	2,896,600	434,500
October	31.00	3.32	1.97	34,410,000	18,049,700	2,707,500
Total	153.00	6.40	23	169,830,000	34,741,500	5,211,200

Computational Table 2
Reuse Water Ouflow

	Inflow, gal.	Outflow, gal Pond	Outflow, gal ET, Crop	Storage Accumlative Gallons	Storage Pond Depth, ft
	(9) (5+6+7)	(10) (4 x PA)	(11) (4 x FA)	(12) (9+12-10-11)	(13) (12 / PA)
May					
June	44,917,300	4,040,800	26,938,800	13,937,700	1.40
July	37,657,800	5,417,600	36,117,500	10,060,400	1.00
August	35,409,300	4,554,100	30,360,400	10,555,200	1.10
September	36,631,100	3,169,100	21,127,400	22,889,800	2.30
October	55,167,200	1,604,900	10,699,500	65,752,600	6.70
Total	209,782,700	18,786,500	125,243,600		

Notes:

1. ET Crop rate based on 9 out of 10 year probalblilty
2. Pond evaporation rate assumed to be equivalent to ET Crop rate
3. Equirpment spray loss is considered neglible
4. Use average summer rainfall of years 2010, 2011, 2012

Wastewater Reuse Calculations

(summer storage pond required)

By: Tom Hart **Date: 6/25/2013**

Per the NPDES permit effluent discharge to Calapooia Creek will not be allowed from June 1st to October 31st. Assume the Irrigation season is from June 1st to October 31st. Based on 2040 Average Dryweatehr Flows flows of 1.11 MGD.

Imput Data

			0.00
Pond Area (PA):	**Acres =**	60	
	PA =	2,613,600 **sf**	

Computational Table 1
Reuse Water Inflow ADWF: 1.11 MGD

Month	Days	Average Rainfall, inches	Pastures ET* & Pond Evaporation, inches/mth	Inflow Gallons Wastewater gallons	Inflow Gallons Field Rainwater	Inflow Gallons Pond Rainwater
(1)	(2)	(3)	(4)	(5)	(6)	(7)
May						
June	30.00	1.86	4.96	33,300,000	N/A	3,030,600
July	31.00	0.52	6.65	34,410,000	N/A	847,300
August	31.00	0.16	5.59	34,410,000	N/A	260,700
September	30.00	0.53	3.89	33,300,000	N/A	869,000
October	31.00	3.32	1.97	34,410,000	N/A	5,414,900
Total	153.00	6.40	23	169,830,000	0	10,422,500

Computational Table 2
Reuse Water Ouflow

	Inflow, gal.	Outflow, gal Pond	Outflow, gal ET, Crop	Storage Accumlative Gallons	Storage Pond Depth, ft
	(9) (5+6+7)	(10) (4 x PA)	(11) (4 x FA)	(12) (9+12-10-11)	(13) (12 / PA)
May					
June	36,330,600	8,081,600	0	28,249,000	1.40
July	35,257,300	10,835,300	0	52,671,000	2.70
August	34,670,700	9,108,100	0	78,233,600	4.00
September	34,169,000	6,338,200	0	106,064,400	5.40
October	39,824,900	3,209,800	0	142,679,500	7.30
Total	180,252,500	37,573,000	0		

Notes:

1. ET Crop rate based on 9 out of 10 year probalblilty
2. Pond evaporation rate assumed to be equivalent to ET Crop rate
3. Equirpment spray loss is considered neglible
4. Use average summer rainfall of years 2010, 2011, 2012

Wastewater Reuse Calculations

(summer storage pond required)

By: Tom Hart Date: 6/25/2013

Per the NPDES permit effluent discharge to Calapooia Creek will not be allowed from June 1st to October 31st. Assume the Irrigation season is from June 1st to October 31st. Based on 2040 Average Dryweatehr Flows flows of 1.11 MGD.

Imput Data

Pond Area (PA):	Acres =	95
	PA =	4,138,200 sf

Computational Table 1
Reuse Water Inflow ADWF: 1.11 MGD

Month	Days	Average Rainfall, inches	Pastures ET* & Pond Evaporation, inches/mth	Inflow Gallons Wastewater gallons	Inflow Gallons Field Rainwater	Inflow Gallons Pond Rainwater
(1)	(2)	(3)	(4)	(5)	(6)	(7)
May						
June	30.00	1.86	4.96	33,300,000	N/A	4,798,500
July	31.00	0.52	6.65	34,410,000	N/A	1,341,500
August	31.00	0.16	5.59	34,410,000	N/A	412,800
September	30.00	0.53	3.89	33,300,000	N/A	1,375,900
October	31.00	3.32	1.97	34,410,000	N/A	8,573,600
Total	153.00	6.40	23	169,830,000	0	16,502,300

Computational Table 2
Reuse Water Ouflow

	Inflow, gal.	Outflow, gal Pond	Outflow, gal ET, Crop	Storage Accumlative Gallons	Storage Pond Depth, ft
	(9) (5+6+7)	(10) (4 x PA)	(11) (4 x FA)	(12) (9+12-10-11)	(13) (12 / PA)
May					
June	38,098,500	12,795,900	0	25,302,600	0.80
July	35,751,500	17,155,800	0	43,898,300	1.40
August	34,822,800	14,421,200	0	64,299,900	2.10
September	34,675,900	10,035,500	0	88,940,300	2.90
October	42,983,600	5,082,300	0	126,841,600	4.10
Total	186,332,300	59,490,700	0		

Notes:

1. ET Crop rate based on 9 out of 10 year probalblilty
2. Pond evaporation rate assumed to be equivalent to ET Crop rate
3. Equirpment spray loss is considered neglible
4. Use average summer rainfall of years 2010, 2011, 2012

CITY OF SUTHERLIN

WASTEWATER COLLECTION SYSTEM ANALYSIS

SMOKE TESTING RESULTS

PROJECT NO. 146.00C

OCTOBER 2012



EXPIRES: 12/31/2012

City of Sutherlin
Wastewater Collection System Analysis - Smoke Testing Results
Project No. 146.00C

Smoke testing of the City of Sutherlin's Wastewater Collection System was completed with a two-man crew from The Dyer Partnership Engineers & Planners, Inc., and employees from the city of Sutherlin. The smoke testing exercise was completed from October 16-19, and October 23, 2012. Smoke was introduced into the system at a total of 60 manholes. The results were compiled into the following report, complete with pictures and sketched maps of the findings.

Smoke Testing Results Summary

Smoke was discovered inside structures at the following locations:

- 267 Fifth Avenue
- 1826 Garnet Court
- 232 Quail Run
- 624 Arvilla Way
- 988 Twin Pine Park
- Tech Built, Inc. on Oak Street
- True Value Center on Central Avenue
- Laundry room of Kim's Court Trailer Park (State Street and Second Avenue)

The remaining deficiencies that were discovered are summarized as follows:

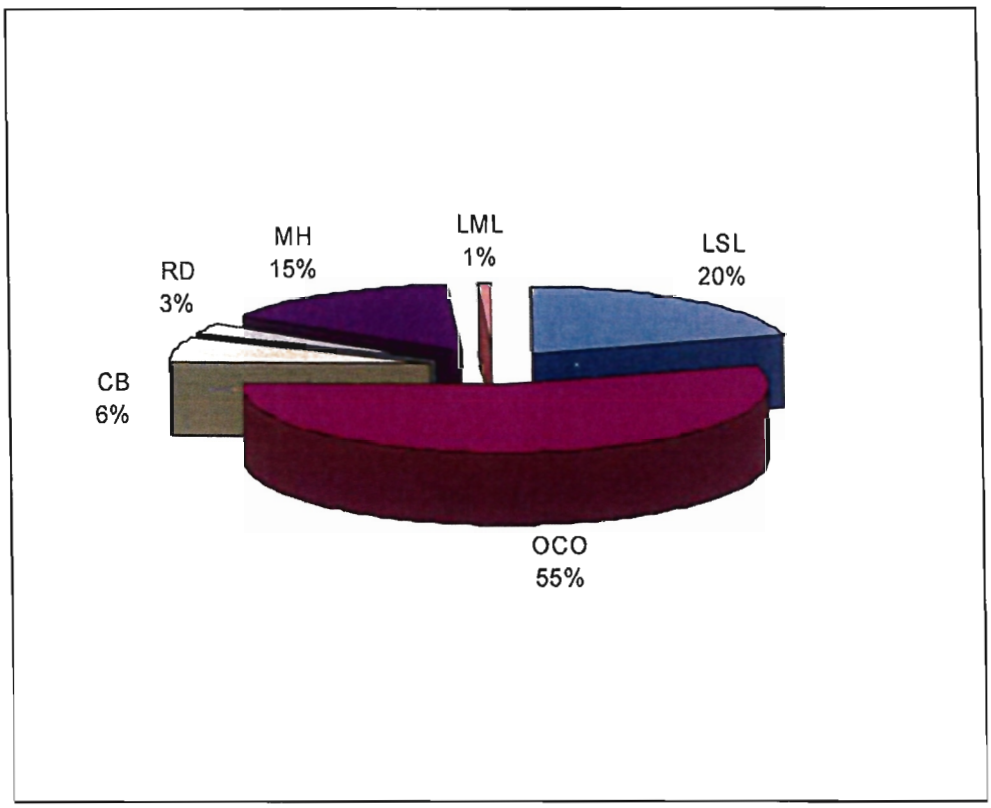
Type of Deficiency	Number of Issues
Leaking Service Lateral (LSL)	28
Open Cleanout (OCO)	49
Broken Cleanout (OCO)	28
Catch Basin (OCO)	5
Area Drain (OCO)	3
Roof Drain (RD)	4
Manhole (MH) - Needs to be re-grouted	21
Leaking Main Line (LML)	1
	Total: 139

The following is a list of manholes not reported within the analysis that will need to be re-grouted, located and uncovered, or raised.

Re-grout: D1, B57, I13, I14, I19, H12, D3, L7

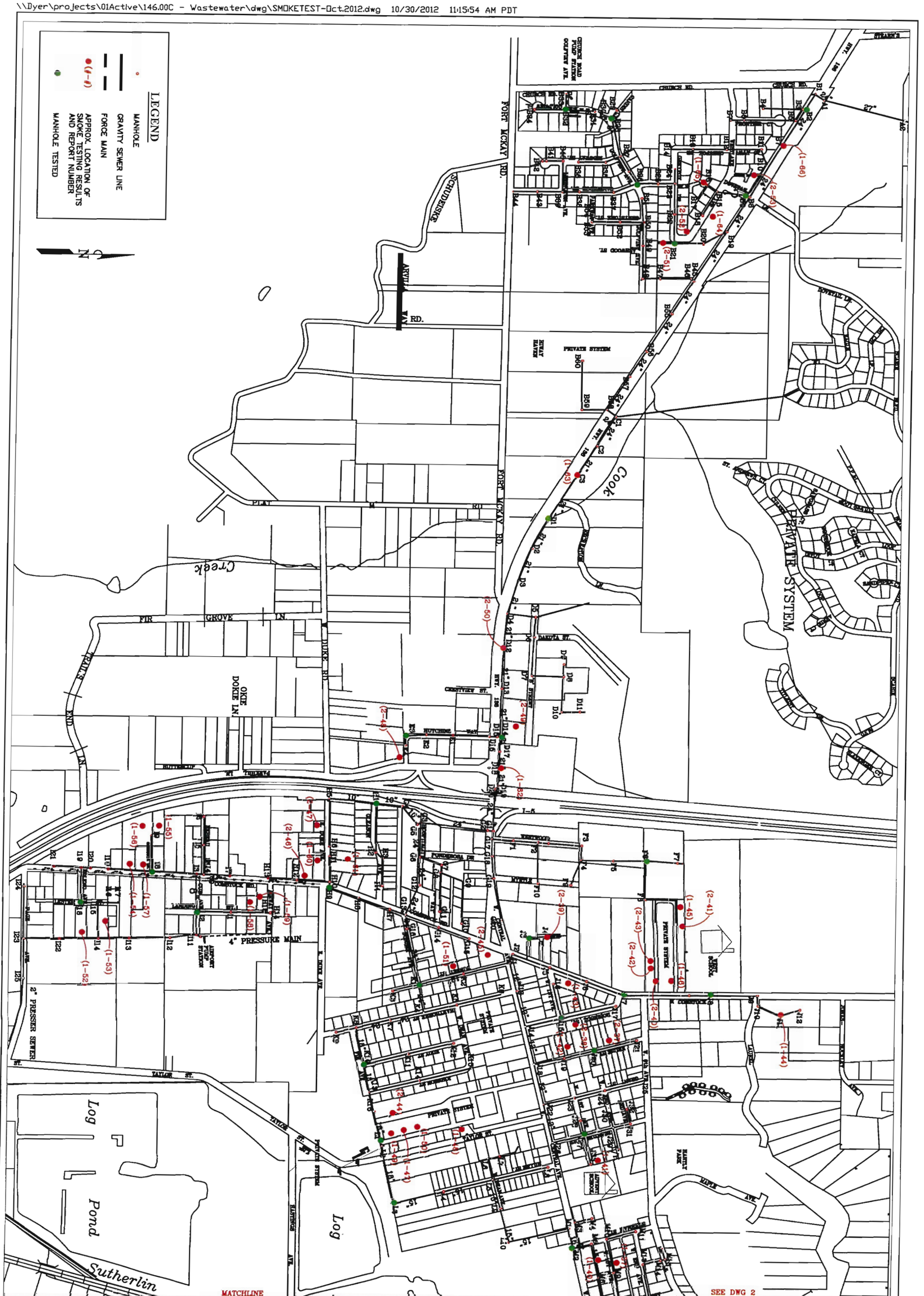
Locate/uncover: D11, D7, D8, D9, K6, G15, I23, I24, I25, H6, H11, D8, X9, U3, P47, T23, R12, P11, P51, P20, L5, L8, J1, J2, J5, F2, F7, M41, M19, J12, and MH on W. Duke Rd.

Raised: B2, A1, G13, D12, F4, A2



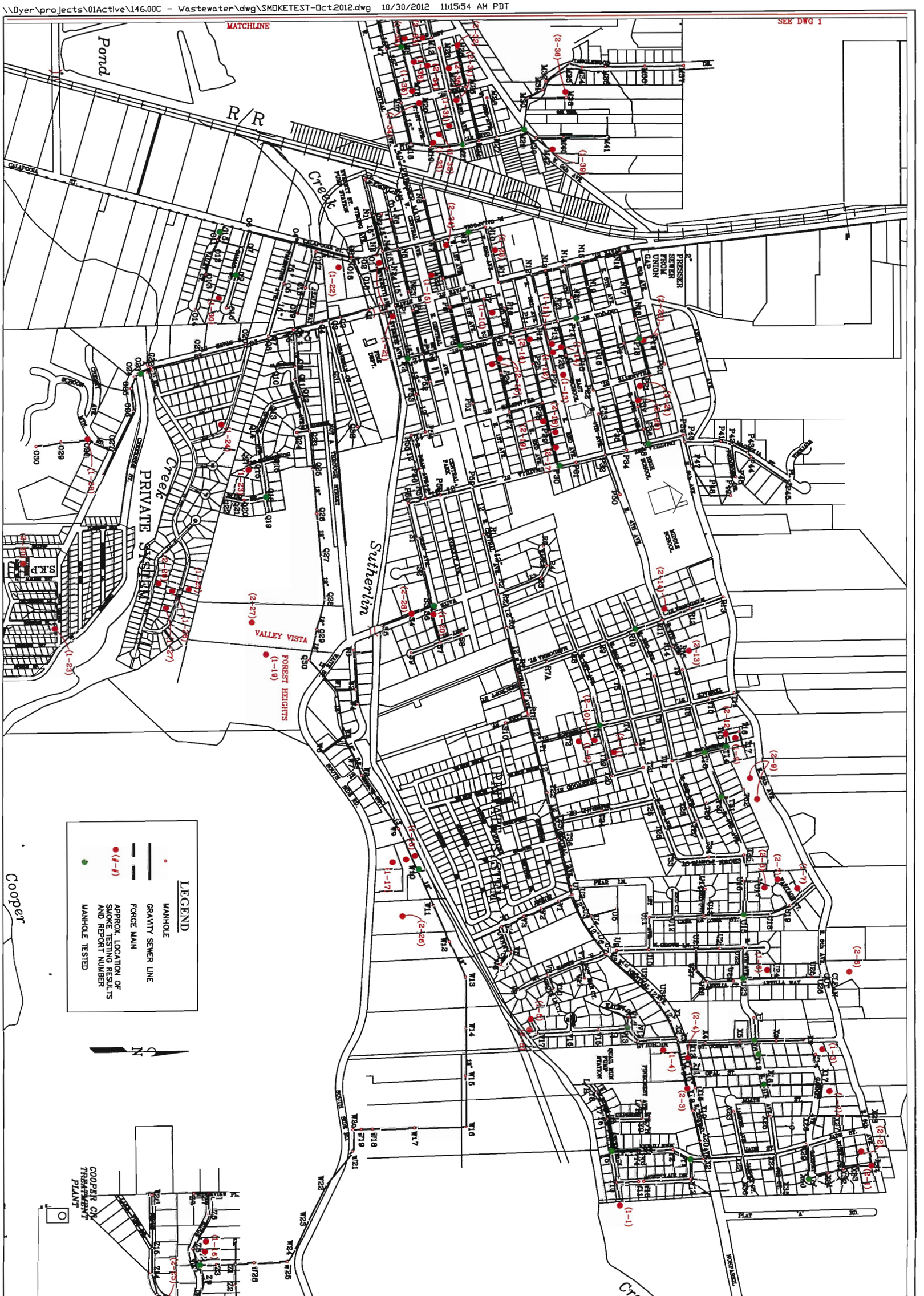
Deficiency Type	Report #
Leaking Lateral Service (LSL)	(1-9), (1-11), (1-12), (1-14), (1-15), (1-18), (1-24), (1-25), (1-31), (1-37), (1-40), (1-41), (1-42), (1-45), (1-47), (2-8), (2-15), (2-16), (2-18), (2-21), (2-24), (2-32), (2-33), (2-48), (2-51)
Open Cleanout (OCO)	(1-1), (1-2), (1-3), (1-4), (1-6), (1-10), (1-15), (1-16), (1-19), (1-22), (1-27), (1-28), (1-36), (1-46), (1-48), (1-49), (1-50), (1-53), (1-56), (1-60), (1-61), (1-64), (1-65), (2-2), (2-6), (2-9), (2-11), (2-15), (2-17), (2-19), (2-26), (2-27), (2-30), (2-37), (2-38), (2-39), (2-42), (2-43), (2-44), (2-46), (2-47), (2-49), (2-53)
Broken Cleanout (OCO)	(1-7), (1-13), (1-17), (1-21), (1-23), (1-26), (1-30), (1-32), (1-38), (1-43), (1-51), (1-52), (1-54), (1-55), (1-57), (1-58), (1-59), (2-5), (2-14), (2-22), (2-29), (2-31), (2-35), (2-36), (2-41), (2-52)
Catch Basin (CB)	(1-33), (2-13), (2-23), (2-40), (2-45)
Area Drain (CB)	(1-8), (1-9), (1-53)
Roof Drain (RD)	(1-9), (1-46), (2-10)
Manholes Need to be Raised (LMH)	(1-34), (1-35), (1-66), (2-50)
Manholes Need to be Regrouted (LMH)	(1-20), (1-28), (1-29), (1-39), (1-44), (1-62), (1-63), (2-1), (2-3), (2-4), (2-12), (2-25), (2-28)
Leaking Main Line (LML)	(2-20)
Lamp Lid (LMH)	(1-5), (2-7)

\\Dyer\projects\01Active\146.00C - Wastewater\dwg\SMOKETEST-Oct.2012.dwg 10/30/2012 11:15:54 AM PDT



THE DYER PARTNERSHIP ENGINEERS & PLANNERS	CITY OF SUTHERLIN FLOW MAPPING	FIGURE NO.
DATE: OCTOBER, 2012	WEST BASIN MAP	1
PROJECT NO.: 146.00C		

\\Dyer\projects\01Active\146.00C - Wastewater\dwg\SMOKETEST-Oct.2012.dwg 10/30/2012 11:15:54 AM PDT



THE DYER PARTNERSHIP ENGINEERS & PLANNERS	CITY OF SUTHERLIN FLOW MAPPING	FIGURE NO.
DATE: OCTOBER, 2012	EAST SIDE BASIN MAP	2
PROJECT NO.: 146.00C		

City of Sutherlin

EVERETT AVENUE PUMP STATION AND FORCE MAIN EVALUATION

DRAFT

JUNE 2010



The Dyer Partnership
Engineers & Planners, Inc.

1330 Teakwood Avenue
Coos Bay, Oregon 97420
(541) 269-0732 ◘ Fax (541) 269-2044
www.dyerpart.com

Project No. 146.19
759 West Central Avenue
Sutherlin, Oregon 97479
(541) 459-4619 ◘ Fax (541) 459-4679

EVERETT AVENUE PUMP STATION AND FORCE MAIN EVALUATION

CITY OF SUTHERLIN
Douglas County, Oregon

Project No. 146.19

June 2010



Table of Contents

SECTION 1 - EXECUTIVE SUMMARY

SECTION 2 - INTRODUCTION

2.1 Background and Need 2-1
2.2 Objectives 2-1
2.3 Scope of Services 2-1
2.4 Authorization 2-2
2.5 Acknowledgements 2-2

SECTION 3 - EVALUATION OF ALTERNATIVES

3.1 Force Main Relocation Alignment Alternatives 3-1
Force Main Relocation Alignment Alternative No. 1
Force Main Relocation Alignment Alternative No. 2
3.2 Everett Avenue Pump Station Improvements and Alternatives 3-5
Background
Alternatives
Everett Avenue Pump Station Improvements

SECTION 4 – SUMMARY & RECOMMENDATIONS

4.1 Summary & Recommendations 4-1
Everett Avenue Pump Station Improvements
Force Main Relocation Improvements
4.2 Future Considerations 4-2

LIST OF TABLES & FIGURES

Table 3.1.1 Everett Avenue Pump Station Improvements Force Main Relocation Alternative No. 1 Preliminary Engineering Cost Estimate..... 3-3
Table 3.1.2 Everett Avenue Pump Station Improvements Force Main Relocation Alternative No. 2 Preliminary Engineering Cost Estimate..... 3-4
Table 3.2.1 Existing Everett Avenue Pump Station Pump Station Design Data 3-9
Table 3.2.2 Everett Avenue Pump Station Improvements Preliminary Engineering Cost Estimate..... 3-13

Figure 3.1.1 Force Main Relocation Alternatives 3-2
Figure 3.2.1 Existing Pump Station 3-6
Figure 3.2.2 Existing Collection System Basins Serviced by Pump Station 3-7
Figure 3.2.3 Inside Everett Avenue Pump Station Building Pump Piping and Diesel Generator..... 3-5
Figure 3.2.4 Inside Everett Avenue Pump Station Building Existing Pumps..... 3-8
Figure 3.2.5 Proposed Everett Avenue Pump Station Improvements 3-12

APPENDIX

A – Calculations

B – Equipment

Section

1

Executive Summary

Section 1

Executive Summary

Everett Avenue Pump Station was constructed over thirty years ago, in 1978. The pump station services the entire eastern portion of the City east of the rail line that bisects it, as shown in Figure 3.2.2. City Staff have done an excellent job of maintaining the pump station regardless of its current operational state and lack of performance. Current peak wet weather flows exceed the operational capacity of the pump station and considerable operational deficiencies create additional difficulties and maintenance issues.

The objective of this study was to evaluate three potential improvement alternatives for the aging Everett Avenue Pump Station and two alternatives for the relocation of the force main to a downstream location. The three pump station alternatives include the rehabilitation of the existing pump station and the construction of a new pump station at either a new location or in a location directly adjacent to the existing pump station.

The recommended pump station alternative is to construct a new pump station directly adjacent to the existing pump station. Improvements to the pump station would include a new wet well with a traffic-rated hatch, new jib crane, three new submersible pumps, level control, standby generator with automatic transfer switch, new control panel and other miscellaneous improvements. The existing building and diesel fuel tank would continue to be utilized in a different capacity. The proposed pump station improvements are estimated to cost approximately $1.03 million, including costs for construction, engineering, required reports, permitting/easement, land acquisition and contingency.

Two alternatives were considered for the relocation of the existing force main. The intention of the relocation is to move the discharge location from Everett Avenue Pump Station to a downstream location with greater hydraulic capacity than the current discharge point at N.W. Oak Street (between West Central Avenue and West First Avenue). This relocation would help to prevent collection system surcharging and Sanitary Sewer Overflows (SSO).

Force Main Relocation Alternative No. 2 is the recommended force main alternative for this project. This alternative investigated the relocation of the Everett Avenue Pump Station force main, including a significant section be constructed through horizontal directional drilling (HDD). This alternative required that a significantly shorter pipeline to be constructed, which resulted in significantly lower costs as compared to Alternative No. 1. The selected force main relocation alternative is illustrated in Figure 3.1.1 and is estimated to cost approximately $1.75 million, including costs for construction, engineering, required reports, permitting/easements and contingency. The estimated cost for Alternative No. 1 is approximately $2.37 million.

Section

2

Introduction

Section 2

Introduction

2.1 Background and Need

Everett Avenue Pump Station has exceeded its design life, which has resulted in time-consuming operations and maintenance. The pumps have been rebuilt multiple times and parts have become unavailable. Pumps continuously seize with rags and other material and have to be taken apart before placing back into operation. Other major maintenance and control issues exist as well, including the malfunctioning level control and alarm system.

The existing sewer force main from Everett Avenue Pump Station discharges to a manhole in the alleyway directly north of the intersection of Central Avenue and Oak Street. The existing gravity sewer line in this area surcharges during times of high groundwater and rain events. Changing the location of the force main discharge will improve the performance of the sewer collection system in this area.

Everett Avenue Pump Station requires equipment upgrades, new wet well, pumps, level control, alarm system, generator and other equipment. This study evaluates whether existing system elements, including the existing building and diesel fuel tank, may be utilized as part of the improvements. Layout drawings, equipment lists and alternative cost estimates are provided. Consideration has been given to the possibility of relocating the pump station as compared to utilization of the existing site.

Multiple potential options for a new sanitary sewer force main from Everett Avenue Pump Station were considered, two of which are presented in this study. These two options include two alternative routings of the force main to the existing 24-inch line beginning on Hawthorn Street. A layout plan with cost estimates is provided.

2.2 Objectives

The objective of this study is to evaluate the existing Everett Avenue Pump Station and its force main. Possible improvement scenarios for the pump station are discussed, and recommended improvements are developed, including layout drawings and project cost estimates. Two alternatives were investigated for the relocation of the force main to a downstream location with greater hydraulic capacity than the current discharge point at N.W. Oak Street. This would help prevent collection system surcharging and Sanitary Sewer Overflows (SSO). Recommendations are provided for improvements to the force main, including an alignment drawing and project cost estimates.

2.3 Scope of Services

The scope of services for this study is to evaluate improvement alternatives and develop improvement recommendations for the Everett Avenue Pump Station. Two alternatives were evaluated for the relocation of the force main, and improvement recommendations have been provided for improvements to the pump station. The existing condition of the pump station is also discussed.

2.4 Authorization

The City of Sutherlin authorized The Dyer Partnership Engineers & Planners, Inc. to prepare a study evaluating improvement alternatives for Everett Avenue Pump Station on February 19, 2008.

2.5 Acknowledgements

We wish to acknowledge the efforts of Robb Corbett, City Manager; Dan Huff, Public Works Director; Kyle Woodward, Wastewater Division Supervisor; and the City's administrative staff.

Section

3

Evaluation of Alternatives

Section 3

Evaluation of Alternatives

The pump station's deficiencies and alternatives for improvements are presented in this section. Section 4 provides a summary and recommendations for improvements.

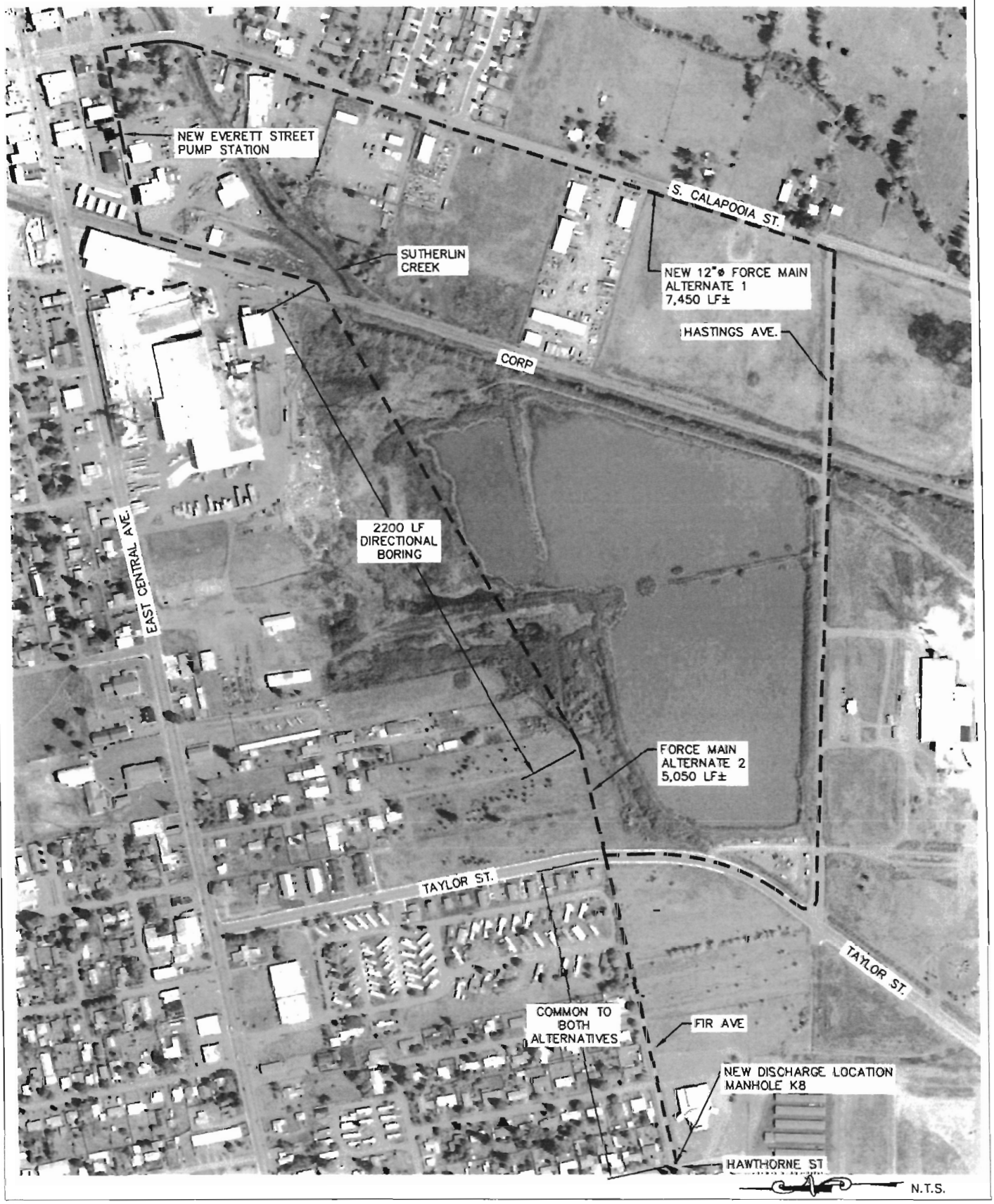
3.1 Force Main Relocation Alignment Alternatives

This section discusses two alternatives that were considered for the relocation of the force main from Everett Avenue Pump Station, as shown in Figure 3.1.1. As discussed in more detail in Section 3.2, the collection system immediately downstream of the existing force main discharge location lacks adequate capacity to service peak wet weather flows. The relocation of the force main discharge to a downstream location, with greater hydraulic capacity, would improve the functionality of the conveyance system. The collection system immediately downstream of the existing force main discharge location is more than adequate to handle gravity wastewater flows generated from adjacent areas.

Force Main Relocation Alignment Alternative No. 1

The first alternative considered for the relocation of the force main follows conventional trenched construction techniques. It begins at the discharge of the new pump station, travels eastward along Everett Avenue, then southward along South Calapooia Street (Old Highway 99 North). The alignment then turns west on Hastings Avenue, north on Taylor Street, then west along the intersection with the eastward extended line of Fir Avenue. The force main would discharge into Manhole K8 at the intersection of Fir Avenue and Hawthorn Street. The alignment for this alternative is shown in Figure 3.1.1.

Survey data was collected for existing ground and sewer line infrastructure along the alternative path. The total length of this force main is approximately 7,450 feet. A minimum flow rate of 1,250 gpm must be met in order to ensure minimum cleaning velocity of 3.5 feet per second in the force main. At this flow rate, the resulting frictional pipe losses would require that 35 hp submersible pumps be installed. The approximate total dynamic head would be 58 feet. For peak wet weather flows of approximately 2,100 gpm, the total dynamic head would be approximately 115 feet. This relatively high head loss might normally indicate that the pipe diameter should be increased to reduce frictional losses, however, this would reduce the cleaning velocity below the minimum required. Other operational solutions exist for this force main issue. Intermittent dual pump operational cycles can be implemented in order to ensure cleaning velocities are met. This solution could result in installation of a smaller, dry-weather pump instead of three large pumps. This will be considered in more detail during pre-design and after more accurate and up-to-date flow analyses are conducted. This force main would also result in substantially longer turnover times; it would hold approximately 45,000 gallons of wastewater. The new pump station may need to cycle between 7 and 10 times in order to turn over the wastewater in the force main to reduce the potential for hydrogen sulfide buildup. Preliminary costs for the proposed improvements are listed in Table 3.1.1.



THE DYER PARTNERSHIP ENGINEERS AND PLANNERS	***CITY OF SUTHERLIN*** ***EVERETT AVENUE PUMP STATION & FORCE MAIN STUDY***	FIGURE NO. ***3.1.1***
DATE: JUN 2010	***FORCE MAIN RELOCATION ALTERNATIVES***	
PROJECT NO.: 146.19		

x:\Projects\146.19 Everett PS & Force Main Evaluation\CAD\dwg\everett force main study.dwg, 6/15/2010 4:50:14 PM

Table 3.1.1 – Everett Avenue Pump Station Improvements
Force Main Relocation Alternative No. 1
Preliminary Engineering Cost Estimate

Item	Description	Unit	Qty.	Unit Cost	Total Cost
1	Construction Facilities & Temporary Controls	LS	1	$ 120,000	$ 120,000
2	Demolition & Site Preparation	LS	1	$ 80,000	$ 80,000
3	12" Fusible PVC Force Main Piping, Class B Backfill	EA	7,450	$ 135	$ 1,005,750
4	Fittings	EA	15	$ 2,500	$ 37,500
5	Cleanout Assemblies	EA	10	$ 4,500	$ 45,000
6	Field Connection to Existing Stuboff	LS	1	$ 8,500	$ 8,500
7	Asphalt Pavement Removal & Replacement	LF	7,450	$ 25	$ 186,250
8	New 5' Manhole On Hawthorne > 10' Depth	EA	1	$ 8,500	$ 8,500
9	Asphalt Concrete Repair	TON	20	$ 320	$ 6,400
10	Landscaping & Restoration	LS	1	$ 25,000	$ 25,000
11	Rammed Railroad Crossing	LS	1	$ 100,000	$ 100,000
				Construction Subtotal	**$ 1,622,900**
				Land Purchase Allowance	$ 25,000
				Engineering	$ 290,000
				DEQ Review Fee	$ 5,000
				Contingency	$ 325,000
				Predesign Report	$ 15,000
				Geotech Report	$ 25,000
				Environmental Report	$ 35,000
				Permitting / Easement	$ 25,000
				Project Total	**$ 2,367,900**

Force Main Relocation Alignment Alternative No. 2

The second alternative considered for the relocation of the force main follows a hybrid project that would include both conventional trenched and horizontal directional drilling construction methods. It begins at the discharge of the new pump station, travels west, then south along the eastern boundary of the Central Oregon & Pacific Railroad right of way. Prior to the alignment intersection with Sutherlin Creek, directional drilling methods would be utilized to construct a portion of the line underneath private property. The directional drilled portion of the force main would terminate at the approximate location of the extension of Miller Street and Fir Avenue. The line would then continue west with a straight alignment along Fir Avenue to the new discharge Manhole K8 located at the intersection of Fir Avenue and Hawthorn Street.

The total length of this force main would be approximately 5,050 feet. In order to ensure minimum cleaning velocity in the force main, a minimum flow rate of 1,250 gpm would be necessary. At this flow rate, the resulting frictional pipe losses would require that 30 hp submersible pumps be installed. The approximate total dynamic head would be 48 feet. For peak wet weather flows of approximately 2,100 gpm, the total dynamic head would be approximately 85 feet. This relatively high head loss might normally indicate that the pipe diameter should be increased to reduce frictional losses; however, this would reduce the cleaning velocity below the minimum required. Other operational solutions exist for this force main issue. Intermittent dual pump operational cycles can be implemented in order to ensure cleaning velocities are met. This solution could result in installation of a smaller, dry-weather pump instead of three large pumps. This will be considered in more detail during pre-design and after more accurate and up-to-date flow analyses are conducted. This longer force main would also result in

substantially longer turnover times in the force main. This force main would hold approximately 30,000 gallons of wastewater. The new pump station may need to cycle between 5 and 7 times in order to turn over the wastewater in the force main, to reduce the potential for hydrogen sulfide buildup.

Preliminary costs for the proposed improvements are listed in Table 3.1.2, located at the end of this section.

Table 3.1.2 – Everett Avenue Pump Station Improvements
Force Main Relocation Alternative No. 2
Preliminary Engineering Cost Estimate

Item	Description	Unit	Qty.	Unit Cost	Total Cost
1	Construction Facilities & Temporary Controls	LS	1	$ 95,000	$ 95,000
2	Demolition & Site Preparation	LS	1	$ 75,000	$ 75,000
	Force Main from Wet Well to Pond Boring Site				
3	12" Fusible PVC Force Main Piping, Class B BF	LF	1,200	$ 90	$ 108,000
4	Fittings	EA	8	$ 3,500	$ 28,000
5	Cleanout Assemblies	EA	2	$ 5,500	$ 11,000
				Subtotal	**$ 147,000**
	Pond Boring Section				
6	14" HDPE Direction Drilled Pipeline Under Log Pond	LF	2,200	$ 250	$ 550,000
7	Cutoff Walls with Required Fittings	EA	2	$ 12,500	$ 25,000
8	HDPE Fittings	EA	6	$ 4,500	$ 27,000
9	Cleanout Assemblies	EA	4	$ 4,500	$ 18,000
				Subtotal	**$ 620,000**
	Force Main - West of Pond to 24" MH @ Hawthorne				
10	12" Fusible PVC Force Main Piping, Class BF	EA	1,650	$ 90	$ 148,500
11	New 4' Manhole On Hawthorne > 10'	EA	1	$ 3,500	$ 3,500
12	Fittings	EA	6	$ 3,500	$ 21,000
13	Cleanout Assemblies	EA	2	$ 4,500	$ 9,000
14	Field Connection to Existing Stuboff	LS	1	$ 8,500	$ 8,500
15	Asphalt Pavement Removal & Replacement	LF	350	$ 35	$ 12,250
16	Asphalt Concrete Repair	TON	15	$ 320	$ 4,800
17	Landscaping & Restoration	LS	1	$ 6,000	$ 6,000
				Subtotal	**$ 213,550**
				Construction Subtotal	**$ 1,150,550**
				Land Purchase Allowance	*$ 45,000*
				Engineering	$ 208,000
				Design Review	$ 5,000
				Contingency	$ 230,000
				Predesign Report	$ 25,000
				Geotech Report	$ 25,000
				Environmental Report	$ 35,000
				Permitting / Easement	$ 25,000
				Project Total	**$ 1,748,550**

3.2 Everett Avenue Pump Station Improvements and Alternatives

The following subsection discusses the existing condition of the Everett Avenue Pump Station and considers the necessary improvements required to bring the pump station into compliance with Oregon Department of Environmental Quality (DEQ) regulations. The site plan of the existing pump station is shown in Figure 3.2.1.

Background

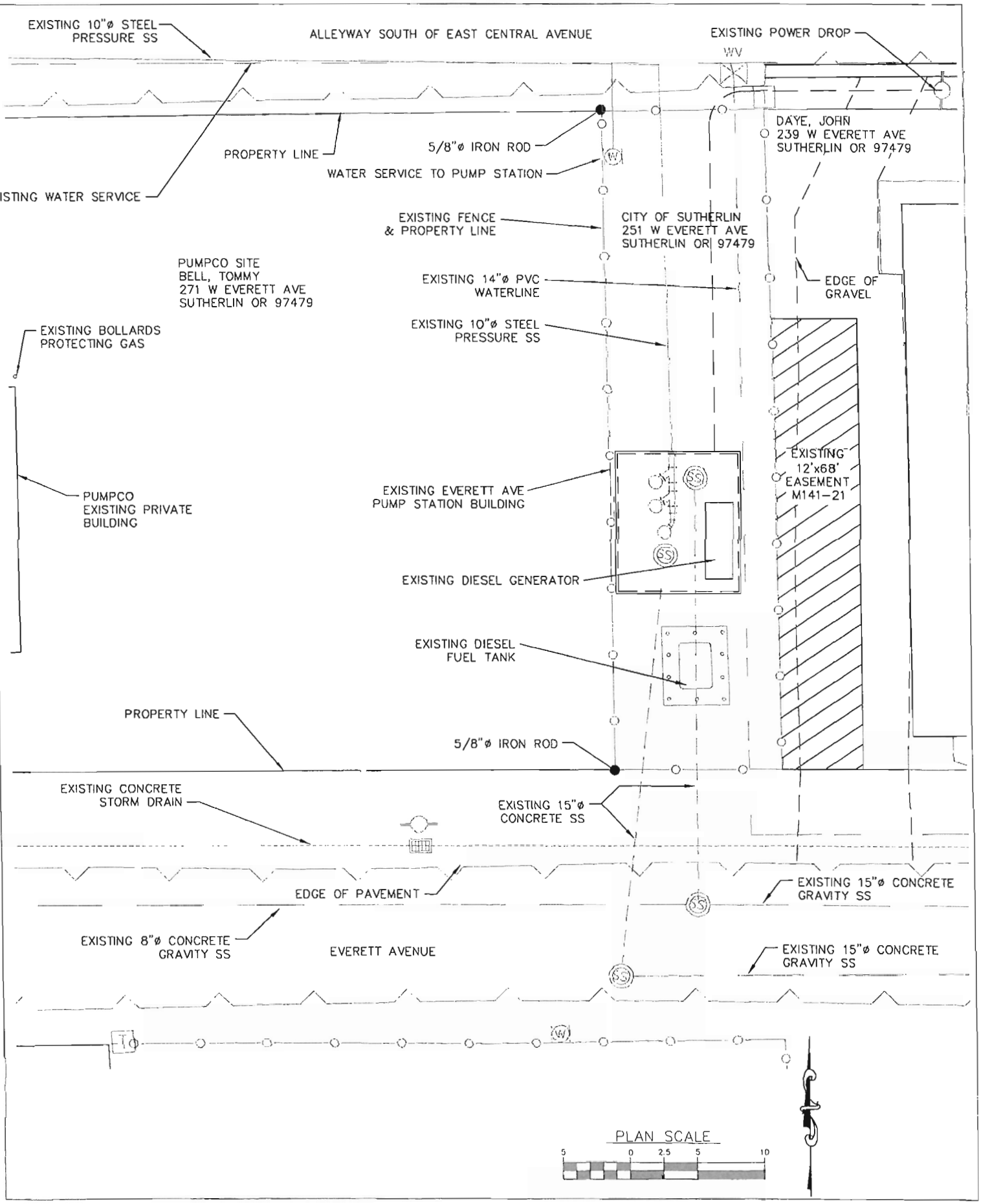
Everett Avenue Pump Station was constructed in 1978 and is located in the southwest corner of Subbasin N near the intersection of Front Street and Everett Avenue. It services the entire eastern portion of the City of Sutherlin east of the Central Oregon and Pacific Railroad line that bisects the City. This includes sewer flows from both public and private systems, including the Union Gap area. More detailed information is provided in the City's recently submitted draft Wastewater Facilities Plan (Dyer, 2010). The eastern portion of the City's sanitary conveyance system that is served by Everett Avenue Pump Station is illustrated in Figure 3.2.2. The pump station collects flows from Subbasin N through Subbasin Z and transmits flows to Subbasin M through a 10-inch diameter, 890-foot long steel pressure main. The force main exits the pump station, crosses the rail line along Central Avenue and discharges into Manhole M18, which is located in the alleyway directly north of the intersection of West Central Avenue and Oak Street.

The pump station is supported by a standby diesel generator that is housed inside the existing building. This can be seen in Figure 3.2.3, which shows the pump header piping on the left and the existing diesel generator on the right. The generator is relatively old, but has not caused significant operational problems since it has been well maintained by City staff. The existing diesel fuel tank is in good condition as was shown by a recent servicing and inspection.

During high flow events, the station is not able to pump the required volume of sewage; this has caused overflows in the past. In addition, the receiving gravity conveyance lines surcharge during pump operation. Manholes O7, M7, M17 and M18 commonly overflow during periods of high ground water and heavy rain, as indicated in Figure 3.2.2. The overflows have been documented in correspondence to DEQ. These alternatives are discussed in Section 3.1.

FIGURE 3.2.3
INSIDE EVERETT AVENUE PUMP STATION BUILDING
PUMP PIPING AND DIESEL GENERATOR





THE DYER PARTNERSHIP ENGINEERS AND PLANNERS	***CITY OF SUTHERLIN*** ***EVERETT AVENUE PUMP STATION & FORCE MAIN STUDY***	FIGURE NO. ***3.2.1***
DATE: JUN 2010	***EXISTING PUMP STATION***	
PROJECT NO.: 146.19		

X:\Projects\146.19 Everett PS & Force Main Evaluation\CAD\dwg\everett force main study.dwg, 6/15/2010 4:27:20 PM

2"ø PRESSURE SEWER FROM UNION GAP

N

R

T

U

X

M

E. 8TH STREET

N. STATE STREET

P

E. 8TH STREET

NONPAREIL

MH M7

MH M17

MH M18

TO WASTEWATER TREATMENT PLANT

PRESSURE SS DISCHARGE

EXISTING PRESSURE SS

EXISTING EVERETT AVENUE PUMP STATION

CENTRAL AVE

Y

V

Sutherlin Creek

PRIVATE SYSTEM

Creek

S

O

CORP

MH 07

W

Q

Log Pond

S. CALAPOOIA ST

Cooper Creek

PRIVATE SYSTEM

LEGEND

- CONCRETE
- A/C
- PVC
- PRESSURE SEWER
- CITY LIMITS
- WASTEWATER BASIN BOUNDARIES

Z

Creek

Cooper

FIGURE NO. 3.2.2

CITY OF SUTHERLIN

EVERETT AVENUE PUMP STATION & FORCE MAIN STUDY

EXISTING COLECTION SYSTEM BASINS SERVICED BY PUMP STATION

THE DYER PARTNERSHIP ENGINEERS & PLANNERS

DATE: JUN 2010

PROJECT NO.: 146.19

X:\Projects\146.19 Everett PS & Force Main Evaluation\CAD\dwg\everett force main study.dwg, 6/15/2010 3:53:07 PM

Drawdown testing of the pumps was conducted as part of the recent wastewater facilities planning process. The drawdown data is included in Appendix A although the data is no longer accurate, as is discussed in the following paragraph. The draft plan was submitted to DEQ and the City for review as of the date of this study. Drawdown testing must be conducted during the pre-design process and then compared to total pump running hours in order to gain a greater understanding of the current condition of the pumps and of flows. The two previous wet weather seasons were particularly dry as compared to previous years. This testing will be important to gain a more recent and accurate assessment of current wet weather wastewater flows. It is estimated that current peak wet weather flow can reach as high as approximately three million gallons per day. Due to surcharging of the inflow line (thus detaining influent), the actual inflow rate cannot be accurately measured and is unknown at this time. Recent inversion lining and lateral repairs along Everett Avenue are anticipated to reduce peak and average wet weather flows into the pump station. The actual impacts have not yet been observed. Peak flows were estimated by comparing recent drawdown testing results for each pump, then applying pump running hours at those flow rates during recorded peak flow conditions.

The existing pumps are relatively old and the City is no longer able to find parts to repair the pumps. An image of the existing pumps is shown in Figure 3.2.4. Pump station design data is summarized in Table 3.2.1. Pump No. 1 was rebuilt in 2004 and Pump No. 3 was rebuilt in August 2009. The pumps present significant operational and maintenance difficulties, including seizing, or ragging, and must be taken apart weekly for service. An example of the extent of operating and maintenance problems can be seen in January 2008. A significant maintenance issue occurred in 11 out of the 31 days of the month for the pump station. The pump station during this period of time relied mainly on the newly replaced Pump No. 1 to pump the flows entering the wet well. Pump No. 1 seized, due to ragging, 9 times on 9 different days. It is typical for City staff to de-rag pumps in the station two or more times per week. This requires a significant level of effort to accomplish. It is important to note that during times when pumps are out of service for the de-ragging process, peak flow continues to enter the wet well. This presents a significant level of risk for the overall operation of the pump station. Many other deficiencies include old vacuum canisters that are no longer supported, grease issues, and control and alarm deficiencies.

FIGURE 3.2.4
INSIDE EVERETT AVENUE PUMP STATION BUILDING
EXISTING PUMPS



TABLE 3.2.1
EXISTING EVERETT AVENUE PUMPS STATION
PUMP STATION DESIGN DATA

Pump Station Item	Pump Station Data
Location	Front St. & Everett St.
Date Built	1978[1]
Last Upgrade	1978
Type	Site Built
EPA Classification	I
Brand	Cornell 4NHTVM5-8 (1ea.) & 6NHGVM-8 (2ea.)
Level Control	Mercury Switch (float)
Pump Type	Vacuum Primed
Pump Installation Year	1978
Motor Size	One 5 HP & Two 15 HP
VFD	No
Design Rated Flow (GPM) @ (TDH ft.)	400 Pump 1 @ (24) 1080 Pump 2 & 3 @ (50) 1500 Pump 2 & 3 @ (36)
Flow Monitoring	None
Overflow Point	Nearest Manhole
Overflow Elevation	N/A (system manholes as noted)
Overflow Discharge Stream	Sutherlin Creek
Time to Overflow[2]	3 hr 20 min.
Alarm Type	Auto loop dedicated phone line
AUXILIARY POWER	
Type	Stationary Inside Bldg. – Onan Diesel
Aux. Power Output	50 kW
Station Requirement	50 kW
Diesel Fuel Tank Capacity	500 gallon
Run Hours per Tank	48
ELEVATIONS & DEPTHS	
Building Slab / Ground	515.48' / 515.4'
Wet well Floor	18' deep

Alternatives

Three alternatives are discussed in this section and the recommended alternative, which includes a layout drawing and project cost estimate, is developed. The first alternative was for the rehabilitation of the pump station. The second alternative discusses the construction of a new pump station at a different location. The third alternative is for the construction of a new pump station directly adjacent to the existing pump station. Each of these potential alternatives for the improvement of Everett Avenue Pump Station was discussed during preliminary conversations with City Staff.

The first alternative included the potential for rehabilitation of the existing pump station. Pump station rehabilitation is an appropriate method to improve pump station performance and to meet regulatory requirements. The existing wet well is undersized for the peak flow entering the collection system. The operational volume is inadequate and larger pumps will not fit into the wet well. The wet well is difficult to service and has grease and solids build-up as a result. Access to the wet well is inside the building, which creates a classified area. The existing electrical system and controls do not meet the Class 1 Division 2 requirements. If it were possible to utilize the existing wet well, there would be no

cost-effective or feasible means by which to keep the pump station operational while the improvements occurred.

A separate alternative considered building the new pump station in a different location within the City. The purpose behind this alternative was to service other areas that might potentially develop in the future with closer proximity to the pump station. In order for this to occur, a new gravity sewer main would need to be constructed along South Calapooia Road to capture wastewater flows that currently enter the existing pump station. A new force main would also need to be constructed along the path considered for Alternative 1 in Section 3.1.

The third alternative includes the requirements for building a new pump station directly adjacent to the existing pump station, as discussed below.

Everett Avenue Pump Station Improvements

General
The new pump station would include new 30 hp triplex submersible pumps with variable frequency drives installed into a new 12-feet wide rectangular wet well. The new wet well would be constructed directly adjacent to the existing pump station, as shown in Figure 3.2.5. Other improvements for the new pump station would include a new jib crane, fixed diesel generator with automatic transfer switch, level control, alarm system, wet well baffle, new 480 volt power service and other instrumentation and controls. The existing building would be utilized to house the control system and the existing diesel fuel tank would be utilized as well. Pump sizing is critical to ensure that peak flow conditions can be met, while at the same time ensuring that minimum force main cleaning velocities are met during the dry weather season.

Pumping & Level Control
Each new pump would be 30 hp, run at 1745 rpm and be rated to move approximately 1,250 gpm (1.8 MGD) at 48 feet TDH. Peak wet weather flows and pump requirements must be verified during the pre-design process, prior to actual pump selection. It may be possible to install a smaller pump to handle dry-weather flows and to intermittently conduct dual pumping operational cycles to ensure force main cleaning requirements are met. This will be reconsidered during pre-design. The pumps would operate on three-phase 480-volt power. A new jib crane would be utilized to remove the pumps when service is required. The pump characteristics curve is located in the Appendix. The force main from the station would be reconstructed as discussed in Section 3.1. The existing 10-inch steel force main from the station would be maintained until the construction is completed, then would be abandoned in place. The new wet well level would be controlled using a pressure transducer with high and low level float sensors as back up.

Standby Power System
The existing diesel standby generator would no longer be utilized and a new fixed diesel generator would be constructed as part of the improvements. The existing diesel fuel tank would be reconnected to the new diesel generator, as it is in good condition. The new 100kW fixed diesel power generator, with sound enclosure and automatic transfer switch would provide standby power to the pumping station. The transfer switch would be housed in a new NEMA 4X control panel. The actual output of the generator must be verified after final selection of the pumps is made during the pre-design process.

Plumbing System
The pump station improvements would include all necessary piping, valves and vault required to connect to the new force main. This would include a new traffic rated wet well hatch and valve vault. The new valve vault would contain isolation valves, pigging port and backflow prevention in the form of a flapper

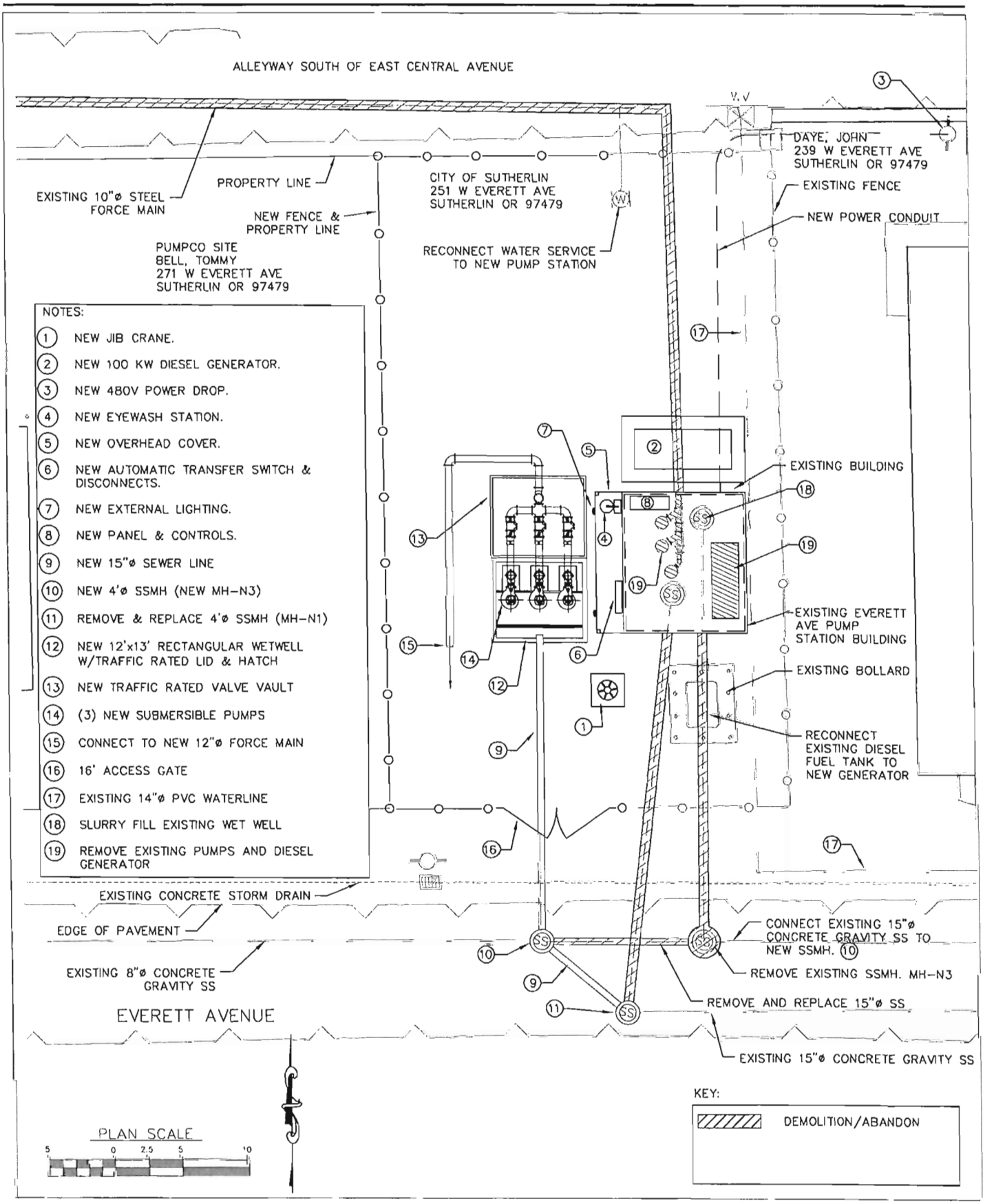
check valve for each pump. The new pressure piping would be connected to the new force main outside the new valve vault.

Electrical Systems & Controls

The new control panel for the pump station would be housed inside the existing pump station building. A new 480V power drop would be added. The panel would house controls and the existing building would hold the new automatic transfer switch for standby power. Preliminary costs for the proposed improvements are listed in Table 3.2.2.

Site Improvements

Site improvements for the new pump station would include a new fence and gate, an enlarged piece of public land, new lighting and minimal landscaping.



THE DYER PARTNERSHIP ENGINEERS AND PLANNERS	***CITY OF SUTHERLIN*** ***EVERETT AVENUE PUMP STATION & FORCE MAIN***	FIGURE NO.
DATE: JUN 2010	***PROPOSED EVERETT AVENUE PUMP STATION IMPROVMENTS***	*3.2.5*
PROJECT NO.. 146.19		

x:\Projects\146.19 Everett PS & Force Main Evaluation\CAD\dwg\everett force main study.dwg, 6/15/2010 4:29:33 PM

Table 3.2.2 – Everett Avenue Pump Station Improvements
Preliminary Engineering Cost Estimate

Item	Description	Unit	Qty.	Unit Cost	Total Cost
1	Construction Facilities & Temporary Controls	LS	1	$ 55,000	$ 55,000
2	Demolition & Site Preparation	LS	1	$ 45,000	$ 45,000
3	30hp submersible pumps with VFD's	EA	3	$ 30,000	$ 90,000
4	Demo Existing Equipment / Piping	LS	1	$ 15,000	$ 15,000
5	Temporary Sheet Pile	LS	1	$ 20,000	$ 20,000
6	12-feet Φ Wet Well, 25' Depth w/ Traffic Rated Hatch	LS	1	$ 95,000	$ 95,000
7	Valve Vault 14'x8', Piping & Appurtenances	LS	1	$ 25,000	$ 25,000
8	Valve Vault Hatch (Traffic Rated)	LS	1	$ 8,500	$ 8,500
9	15" Gravity Sewer Pipe - (Depth >10')	LF	70	$ 120	$ 8,400
10	New Fence and Gate	LS	1	$ 8,500	$ 8,500
11	Jib Crane	LS	1	$ 12,000	$ 12,000
12	Core Drill Holes In Vault, Foundation & Wet Well	LS	1	$ 5,000	$ 5,000
13	New 4' Manhole On Everett Street > 10'	EA	1	$ 3,500	$ 3,500
14	Manhole Remove & Backfill Class C - Everett Avenue	EA	1	$ 10,000	$ 10,000
15	Diesel Generator w/ Transfer Switch / Disconnect	LS	1	$ 80,000	$ 80,000
16	Concrete Pad for Generator	CY	10	$ 90	$ 900
17	Eyewash Station	EA	1	$ 1,000	$ 1,000
18	Fill Existing Wet Well & Pipes Abandon (slurry)	LS	1	$ 5,000	$ 5,000
19	Level Control (Pressure Transducer, Float Backup)	LS	1	$ 8,500	$ 8,500
20	Instrumentation & Controls (Alarm System)	LS	1	$ 35,000	$ 35,000
21	New 480V Power Drop & Connection	LS	1	$ 15,000	$ 15,000
22	New External Lighting	LS	1	$ 2,500	$ 2,500
23	Electrical	LS	1	$ 45,000	$ 45,000
24	Mechanical	LS	1	$ 50,000	$ 50,000
				Construction Subtotal	**$ 643,800**
				Land Purchase Allowance	$ 45,000
				Engineering	$ 129,000
				DEQ Review Fee	$ 5,000
				Contingency	$ 123,500
				Pre-design Report	$ 25,000
				Geotechnical Report	$ 15,000
				Environmental Report	$ 35,000
				Permitting / Easement	$ 10,500
				Project Total	**$ 1,031,800**

Section

4

Summary & Recommendations

Section 4

Summary & Recommendations

In this section, the improvement alternatives presented in Section 3 are discussed and recommendations are presented for the Everett Avenue Pump Station and Force Main Relocation Improvements.

4.1 Summary & Recommendations

Three potential improvement alternatives for the aged existing Everett Avenue Pump Station were discussed in the study. These included the rehabilitation of the existing pump station, the construction of a new pump station in either a different location or directly adjacent to the existing pump station.

Two alternatives were considered for the relocation of the existing force main. The intention of the relocation is to move the discharge location from the Everett Avenue Pump Station to a downstream location with greater hydraulic capacity. This will help to prevent collection system surcharging and Sanitary Sewer Overflows (SSO).

Everett Avenue Pump Station Improvements

Construction of a new pump station directly adjacent to the existing pump station is the most cost-effective and prudent alternative. The existing pump station is undersized and not capable of supporting the required equipment upgrades. This alternative will allow the City to maintain the operation of the existing pump station until the new pump station is ready to place into service. A preliminary layout and a cost estimate were provided for this alternative in Figure 3.1.5. The pump station improvements will include a new wet well with a traffic-rated hatch, new jib crane, three new submersible pumps, level control, standby generator with automatic transfer switch, new control panel and other miscellaneous improvements. The existing building and diesel fuel tank will continue to be utilized. The proposed pump station improvements are estimated to cost approximately $1.03 million.

Force Main Relocation Improvements

Force Main Relocation Alternative No. 1 required a significantly longer pipeline to be constructed, which resulted in much higher costs as compared to Alternative No. 2. Larger pumps, generator and wet well would be required for the new pump station as well. This alternative is not recommended for this project. This force main relocation alternative is illustrated in Figure 3.2.1 and is estimated to cost approximately $2.38 million.

Force Main Relocation Alternative No. 2 is the recommended alternative for this project. This alternative included a long section of pipeline to be constructed through horizontal directional drilling (HDD) methods. This alternative requires that less pipeline be constructed, resulting in significantly lower costs as compared to Alternative No. 1. This force main relocation alternative is illustrated in Figure 3.2.1 and is estimated to cost approximately $1.75 million.

4.2 Future Considerations

The City of Sutherlin's Everett Avenue Pump Station was constructed in 1978. The domestic wastewater flows and inflow and infiltration entering the collection system exceed the operational capacity of the existing pump station. There are no plans to extend the urban growth boundaries and, therefore, future additions to flows should be offset by the City's aggressive inflow and infiltration reduction program. The new pump station and force main recommended by this study will provide long years of excellent operational performance for this portion of the wastewater conveyance system.

Appendix A – Calculations

City of Sutherlin
Everett Avenue PS and Force Main Evaluation Study
Pump Cycle Time Calculations
Project No. 146.19
Friday, June 04, 2010

Pump Sizing **Force Main Relocation Alternative No. 1**			**Notes**
Airport Pump Station Submersible Pumps			
TDH =	58	feet	2-35hp Pumps Running
Design Flow =	2,100	gpm	
Total Wetwell Operational Volume =	7,000	gallons	12' x 13' x 6' (Operational Depth)
Time to Fill Op Volume at PIF =	3.33	minutes	With Pumps Off
Pump Flow Volume =	2,500	gpm	With 2 Pumps Running
Net Outflow From Wetwell =	400.00	gpm	Net Outflow is Pump Flow Minus Max Pump Output
Time to Empty Wetwell =	17.50	minutes	At PIF with One 2 Pumps On
Starts Per Hour =	2.88		

Pump Sizing **Force Main Relocation Alternative No. 2**			**Notes**
Airport Pump Station Submersible Pumps			
TDH =	48	feet	2-30hp Pumps Running
Design Flow =	2,100	gpm	
Total Wetwell Operational Volume =	7,000	gallons	12' x 13' x 6' (Operational Depth)
Time to Fill Op Volume at PIF =	3.33	minutes	With Pumps Off
Pump Flow Volume =	2,500	gpm	With 2 Pumps Running
Net Outflow From Wetwell =	400.00	gpm	Net Outflow is Pump Flow Minus Max Pump Output
Time to Empty Wetwell =	17.50	minutes	At PIF with One 2 Pumps On
Starts Per Hour =	2.88		

City of Sutherlin
Everett Avenue PS & Force Main Evaluation Study
Friction Loss Calculation
Project No. 146.19
Friday, June 04, 2010

Multiple Flow Rates

Force Main Relocation Alternative No. 1

Diameter (in)	Diameter (ft)	C factor	Length (ft)	Static
12	1	120	7450	25

Flowrate (CFS)	Velocity (ft/s)	Flowrate (GPM)	Static	Head Loss	TDH
2.23	2.8	1000	25.0	22.0	47.0
2.45	3.1	1100	25.0	26.3	51.3
2.79	3.5	1250	25.0	33.3	58.3
2.90	3.7	1300	25.0	35.8	60.8
3.12	4.0	1400	25.0	41.1	66.1
3.34	4.3	1500	25.0	46.7	71.7
3.56	4.5	1600	25.0	52.6	77.6
3.79	4.8	1700	25.0	58.8	83.8
4.01	5.1	1800	25.0	65.4	90.4
4.23	5.4	1900	25.0	72.3	97.3
4.46	5.7	2000	25.0	79.5	104.5
4.68	6.0	2100	25.0	87.0	112.0
4.90	6.2	2200	25.0	94.8	119.8
5.12	6.5	2300	25.0	102.9	127.9
5.35	6.8	2400	25.0	111.3	136.3
5.57	7.1	2500	25.0	120.1	145.1

Force Main Relocation Alternative No. 2

Diameter (in)	Diameter (ft)	C factor	Length (ft)	Static
12	1	120	5050	25

Flowrate (CFS)	Velocity (ft/s)	Flowrate (GPM)	Static	Head Loss	TDH
2.23	2.8	1000	25.0	14.9	39.9
2.45	3.1	1100	25.0	17.8	42.8
2.79	3.5	1250	25.0	22.6	47.6
2.90	3.7	1300	25.0	24.3	49.3
3.12	4.0	1400	25.0	27.8	52.8
3.34	4.3	1500	25.0	31.6	56.6
3.56	4.5	1600	25.0	35.6	60.6
3.79	4.8	1700	25.0	39.9	64.9
4.01	5.1	1800	25.0	44.3	69.3
4.23	5.4	1900	25.0	49.0	74.0
4.46	5.7	2000	25.0	53.9	78.9
4.68	6.0	2100	25.0	59.0	84.0
4.90	6.2	2200	25.0	64.3	89.3
5.12	6.5	2300	25.0	69.8	94.8
5.35	6.8	2400	25.0	75.5	100.5
5.57	7.1	2500	25.0	81.4	106.4

City of Sutherlin
Wastewater Facilities Plan
Project No. 146.19
Drawdown Tests

Pump #1 is being replaced at the time of this test, no test data was taken.
Engineering data for this pump is available.
Flow rate for pumps is decreased by approximately 1/2 when jammed with rags.

Everett Avenue Pump Station	
Approx. Inflow Total (gpm)	72.12
Approx. Inflow Total (ft^3/s)	0.16
Wetwell Diameter (ft)	17

Pump No.	Trial #1		Trial #2	
2	Level Change (in.)	12	Level Change (in.)	12
	Time (s)	579	Time (s)	726
	Volume Change (ft^3)	226.98	Volume Change (ft^3)	226.98
	Volume Change (gal)	1697.81	Volume Change (gal)	1697.81
	Volume Flowrate (ft^3/s)	0.55	Volume Flowrate (ft^3/s)	0.47
	Volume Flowrate (gal/min)	248.06	Volume Flowrate (gal/min)	212.43
3	Level Change (in.)	12	Level Change (in.)	12
	Time (s)	755	Time (s)	550
	Volume Change (ft^3)	226.98	Volume Change (ft^3)	226.98
	Volume Change (gal)	1697.81	Volume Change (gal)	1697.81
	Volume Flowrate (ft^3/s)	0.46	Volume Flowrate (ft^3/s)	0.57
	Volume Flowrate (gal/min)	207.05	Volume Flowrate (gal/min)	257.34

Inflow Evaluation*

	Trial #1	Trial #2	Average
Wetwell Level Change (ft)	2.23	1.77	
Time (s)	2700	3000	
Wetwell Diameter (ft)	17	17	
Volume Change (ft^3)	506.17	401.75	
Volume Change (gal)	3786.12	3005.12	
Volume Flowrate (ft^3/s)	0.19	0.13	0.16
Volume Flowrate (gal/min)	84.14	60.10	72.12

*This evaluation was done by averaging the inflow over time.
This was done since the inflow varies widely over short time periods.
Longer time periods were used.

Force Main Velocity:

Diameter 10 inch
Length 890 feet

230.24682
232.1904185

CalculatedVelocity
Average cfs = 0.5156175 ft.^3/sec
Pipe Area = 0.5451389 sf
V = q/a = 0.9458462 ft.^2/s

Appendix B – Equipment

FLYGT

PERFORMANCE CURVE

PRODUCT	TYPE
NP3171.185	MT

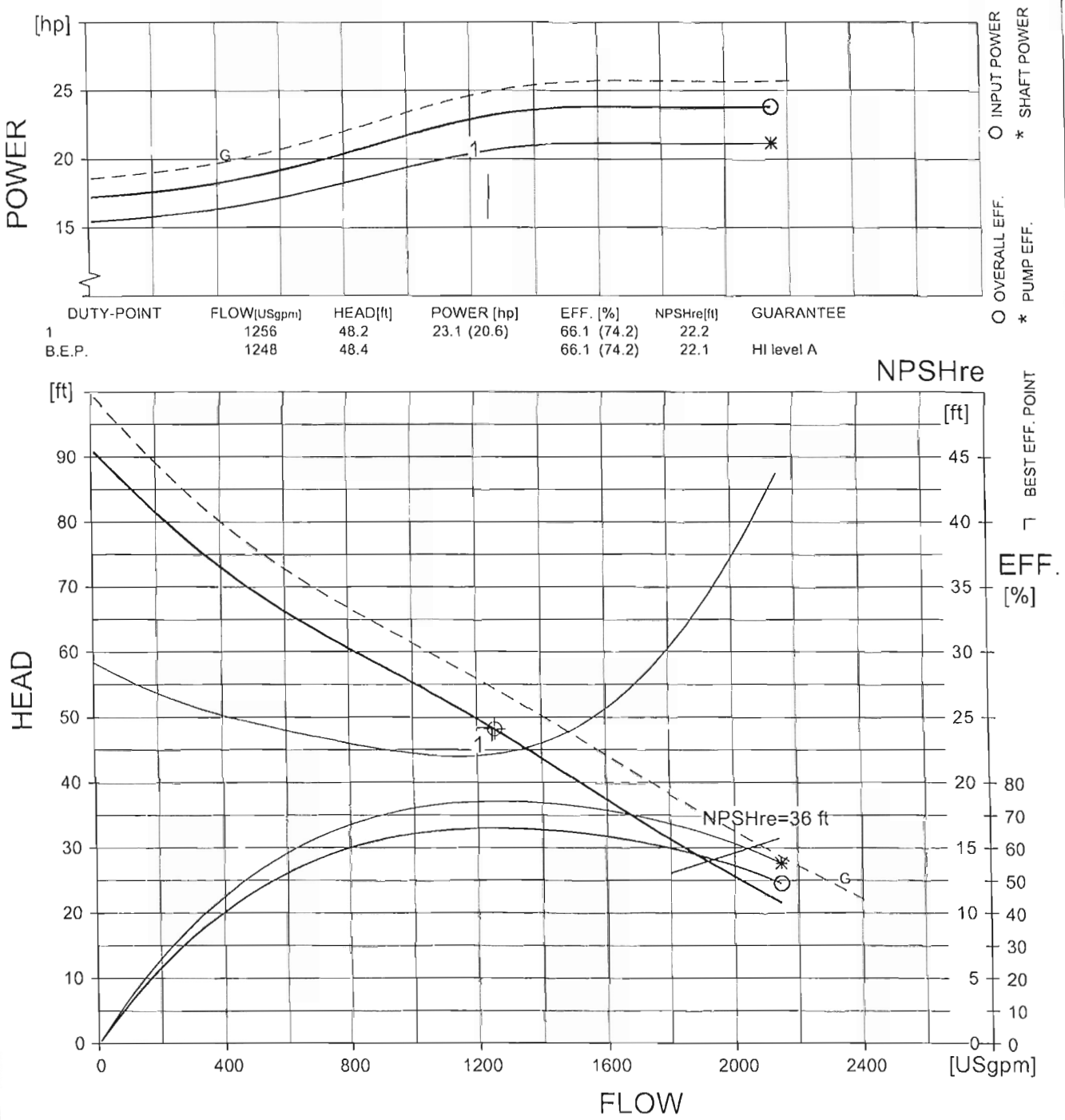
DATE	PROJECT	CURVE NO	ISSUE
2010-06-04		63-436-00-3030	2

	1/1-LOAD	3/4-LOAD	1/2-LOAD
POWER FACTOR	0.87	0.83	0.73
EFFICIENCY	89.0 %	90.0 %	90.5 %
MOTOR DATA	---	---	---

RATED POWER	30	hp
STARTING CURRENT ...	231	A
RATED CURRENT ...	36	A
RATED SPEED	1755	rpm
TOT.MOM.OF INERTIA ...	0.15	kgm2
NO. OF BLADES	2	

IMPELLER DIAMETER			
234 mm			
MOTOR #	STATOR	REV	
25-17-4AA	01D	11	
FREQ.	PHASES	VOLTAGE	POLES
60 Hz	3	460 V	4
GEARTYPE	RATIO		
---	---		

COMMENTS	INLET/OUTLET	IMP. THROUGHLET
	-/ 6 inch	---



DUTY-POINT	FLOW[USgpm]	HEAD[ft]	POWER [hp]	EFF. [%]	NPSHre[ft]	GUARANTEE
1	1256	48.2	23.1 (20.6)	66.1 (74.2)	22.2	
B.E.P.	1248	48.4		66.1 (74.2)	22.1	HI level A

NPSHre = NPSH3% + min. operational margin

Performance with clear water and ambient temp 40 °C

GUARANTEE BETWEEN LIMITS (G) ACC. TO

HI level A

FLYPS3.1.6.2 (20060531)

FLYGT

PERFORMANCE CURVE

PRODUCT	TYPE
NP3202.180	MT

DATE	PROJECT	CURVE NO	ISSUE
2010-06-04		63-642-00-5040	5

	1/1-LOAD	3/4-LOAD	1/2-LOAD
POWER FACTOR	0.86	0.83	0.75
EFFICIENCY	89.0 %	89.5 %	89.0 %
MOTOR DATA	---	---	---

RATED POWER	35	hp
STARTING CURRENT ...	246	A
RATED CURRENT ...	43	A
RATED SPEED	1170	rpm
TOT.MOM.OF INERTIA ...	0.39	kgm2
NO. OF BLADES	2	

IMPELLER DIAMETER: 334 mm

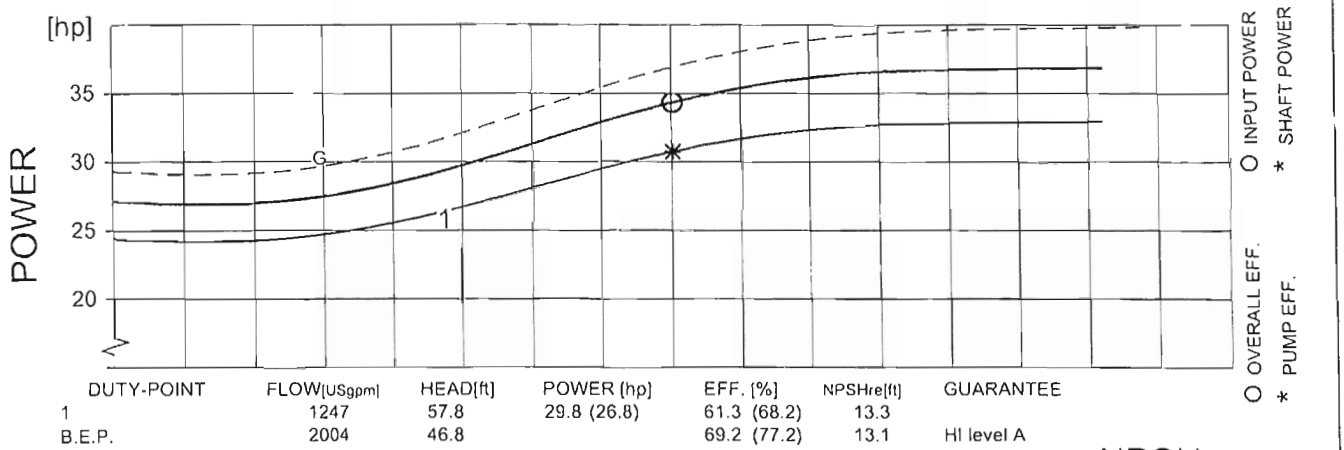
MOTOR #	STATOR	REV
30-18-6AA	01D	10

FREQ.	PHASES	VOLTAGE	POLES
60 Hz	3	460 V	6

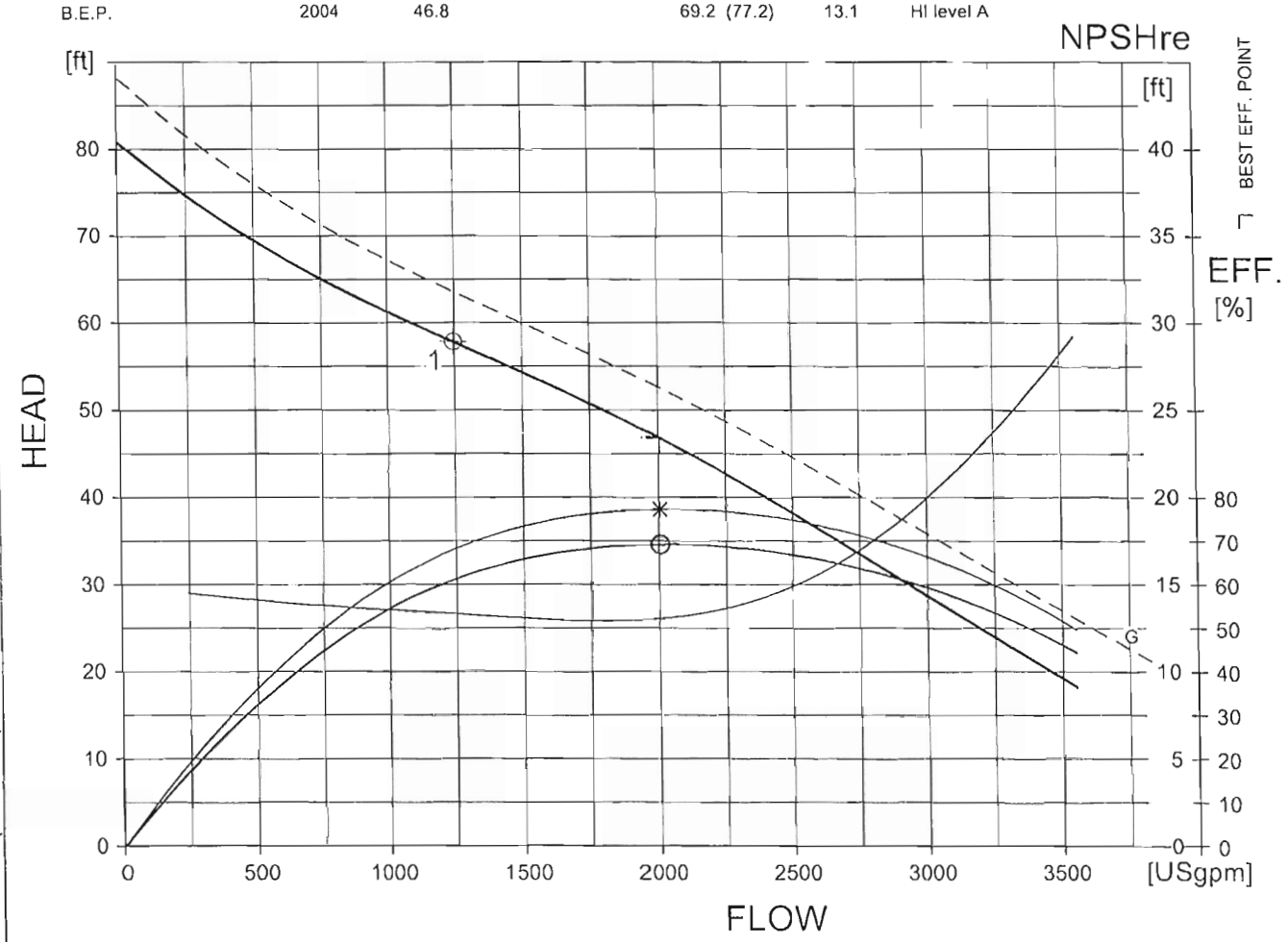
GEARTYPE	RATIO
---	---

COMMENTS

INLET/OUTLET	IMP. THROUGHLET
-/ 8 inch	---



DUTY-POINT	FLOW[USgpm]	HEAD[ft]	POWER [hp]	EFF. [%]	NPSHre[ft]	GUARANTEE
1	1247	57.8	29.8 (26.8)	61.3 (68.2)	13.3	
B.E.P.	2004	46.8		69.2 (77.2)	13.1	HI level A



NPSHre = NPSH3% + min. operational margin

Performance with clear water and ambient temp 40 °C

GUARANTEE BETWEEN LIMITS (G) ACC. TO

HI level A

FLYPS3.1.6.2 (20060531)