CITY OF AUMSVILLE

WASTEWATER FACILITIES PLAN

June 1999

Prepared by:

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MUNICIPAL ENGINEERING AND LAND DEVELOPMENT SERVICES

Balfour Consulting, Inc. (**BCI**) would like to acknowledge and thank the following persons for their assistance in the preparation of this Wastewater Facilities Plan.

City of Aumsville

Special thanks to:

Mayor Harold White
Darlene Loyd, City Council President
Chester Bridges, City Councilor
Craig Chadwick, City Councilor
David Drews, City Councilor
Phil Gourley, City Councilor
Steve Oslie, Public Works Superintendent
Maryann Hills, City Administrator
Jerry Overgard, MWVCOG
Sam Healy, MWVCOG

Expires 12-31-99
Bryan C. Balfour, P.E.

Funding of this Wastewater Facilities Plan was provided through a grant from the Community Development Block Grant Program.

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CHAPTER 1 - INTRODUCTION

1.1 BACKGROUND

The City of Aumsville is located in northwestern Oregon, in the western corner of Marion County. It is fifteen minutes southeast of Salem along State Highway 22, and five minutes northwest of Stayton. Aumsville's population has now reached approximately 3,000 people.

In the late 1960's the City of Aumsville converted from a sewer system consisting of septic tanks with drain fields to a municipal sanitary sewer system, including a gravity collection system and facultative lagoon treatment plant. This system was comprised of approximately 830 feet of 6 inch line, 17,000 feet of 8 inch line, and 6,700 feet of 10 inch line. The pipelines are constructed of concrete pipe with rubber ring compression gaskets for joints.

The wastewater treatment facility now consists of the two original stabilization ponds, and two additional facultative lagoons that were added in the late 1970's. The system expansion doubled the plant capacity. The lagoons serve as detention basins between May 1 and October 31 of each year, as the City is not permitted to discharge wastewater into surface waters during this period. Treated effluent is discharged to Beaver Creek from November 1 through April 30 each year.

The City is currently experiencing capacity problems. In recent years the City was forced to discharge wastewater to Beaver Creek prior to November 1 due to high water levels in the lagoons. The City has implemented an infiltration and inflow (I&I) rehabilitation program that will help reduce flows. However, community growth and system deterioration will continue to exert pressures on the system.

1.2 PURPOSE

The purpose of this Plan is to evaluate the existing wastewater collection and treatment system, to identify the deficiencies, and to determine remedies that will allow the City to meet current and anticipated future regulations and allow for system growth.

1.3 SCOPE OF WORK

The scope of work covered in this Plan includes the tasks below.

Task 1 - Collect Background Data/Study Area Characterization
Task 2 - Existing System Description
Task 3 - Future Conditions Evaluation
Task 4 - Analysis of Alternatives
Task 5 - Analysis of Selected Alternative

Task 6 - Implementation Program and Schedule
Task 7 - City Staff and Council Presentation

Upon completion of the Plan and approval by the City, the Plan shall be submitted to the DEQ for review and approval.

The City Authorized Balfour Consulting, Inc., (BCI) to prepare this Wastewater Facilities Plan in April 1997.

1.4 ACKNOWLEDGMENTS

The time and efforts of many people contributed to this project. In particular, we wish to acknowledge the following individuals: Mayor Harold White; City Council President Darlene Loyd; City Councilor's Chester Bridges, Craig Chadwick, David Drews, and Phil Gourley; Steve Oslie, Public Works Superintendent; and Maryann Hills, City Administrator. Their leadership and foresight to provide a healthy environment for the citizens of Aumsville and the State of Oregon are gratefully appreciated.

We wish to also acknowledge Jerry Overgard and Sam Healy with the Mid-Willamette Valley Council of Governments; the Oregon Economic Development Department; and many others for their interest, guidance, and assistance during the course of this study. Funding for this Wastewater Facilities Plan was provided through a grant by the Community Development Block Grant program.

CHAPTER 2 - EXECUTIVE SUMMARY

The following summary of the Wastewater Facilities Plan for the City of Aumsville presents an overview of the conclusions and recommendations of the Plan.

2.1 CONCLUSIONS

2.1.1 Existing Wastewater System

- ***Service Area*** - The sewage service area encompasses the City of Aumsville and its Urban Growth Boundary.

- ***Collection System*** - The existing collection system consists of approximately 52,400 feet of buried piping, ranging in size from 6 to 10-inches in diameter. The majority of the collection pipes are 8-inch diameter. The existing 10-inch sewer main trunk line has a calculated maximum capacity of 2.16 MGD, compared to a projected design flow rate (20-year PIF) of 7.34 MGD.

  Although video tapes indicate the collection system is in relatively good shape with few leaks, limited flow monitoring data suggests a more significant I&I problem. The city should continue to move forward with its I&I reduction program.

- ***Treatment Plant Influent Pump Station*** - The City has one pump station, located at the wastewater treatment facility. This influent pump station has a capacity of 0.9 MGD with one pump operating, and 1.8 MGD with two pumps operating. The projected design flow (20-year PIF) is 7.34 MGD. The existing wetwell has insufficient capacity to handle future design flows.

- ***Wastewater Treatment Plant*** - The City operates a facultative lagoon wastewater treatment facility, consisting of two primary lagoons (7.6 and 6.7 acres), one secondary lagoon (7.8 acres), and one tertiary lagoon (6.3 acres). The effluent is disinfected with chlorine gas and routed through a contact chamber. The treatment facility was originally constructed in the late 1960's and expanded in the 1970's.

  Although the lagoon treatment plant is currently capable of meeting the 30/50 wastewater discharge limits stipulated in the City's NPDES permit, the plant has not had sufficient capacity to store all of the flow collected over the six-month dry weather period in the recent past.

- ***Effluent Disposal*** - During the wet season months (November through April), chlorinated effluent is discharged into Beaver Creek via a gravity outfall located adjacent to the treatment plant. During the remainder of the year (May through October), no discharge is permitted and the wastewater is stored in the treatment plant lagoons.

- ***Biosolids Management*** - Biosolids have not been removed from the primary lagoons since their last upgrade in the 1970's. Typically, sludge should be removed from primary facultative lagoons every 10 to 20 years.

2.1.2 Major System Needs and Deficiencies

- ***Collection System*** - In order to accommodate the projected future flows, replace the existing 10-inch diameter main trunk line sewer with a 24-inch diameter PVC sewer pipe between the influent lift station and manhole A-3.

  Conduct sewer system flow monitoring during significant rainfall with high groundwater events. Measure flow from manhole to manhole in high flow basins to identify specific pipe reaches within the system where high flows are found to exist and clarify where these problems can possibly be corrected.

- ***Treatment Plant Influent Pump Station*** - The influent lift station is currently at capacity and requires replacement. Install a pump station with a minimum capacity of 6.48 MGD, and a new 10 to 12-foot diameter wetwell. Install a new 16-inch forcemain to route the flow into the headworks.

- ***Wastewater Treatment Plant*** - The plant does not have sufficient capacity to meet the projected growth within the 20 year planning period, and has been unable to store all of the flow collected over the six-month dry weather period for two of the last three years. The lagoons have become so full that the integrity of the dikes was threatened and effluent was discharged to the river outside of the permitted discharge period. The City has received two notices of noncompliance (NONs) from the DEQ related to these discharges.

- ***Disinfection*** - The volume of the chlorine contact chamber is not sufficient to provide the required detention time for the current discharge flow rates. The receiving stream, Beaver Creek, is effluent dominated and may not provide sufficient dilution for a chlorinated discharge.

- ***Effluent Disposal (November-April)***- The outfall pipe discharges on the bank of the creek, above the normal water surface. The outfall to the creek must be upgraded to discharge below the normal water surface and to provide better mixing.

- ***Effluent Disposal (May - October)*** - The treatment facility has not had sufficient capacity to store all of the flow collected over the six-month dry weather period in the recent past. The City should immediately implement a reclaimed water irrigation system, coordinating with the DEQ to meet both current and future demands.

- ***Biosolids Management*** - Remove biosolids from the existing primary lagoons to accommodate construction of plant upgrades and to achieve the required storage and treatment capacity. Land apply the biosolids on nearby agricultural land.

2.1.3 Population

Aumsville has recently experienced significant growth. The population estimates used in this report project Aumsville's population to increase at an annual rate of 2.53 percent between 1997 and 2022, in accordance with Marion Count Board of Commissioners approved growth rate. Using the above growth projections and a 1997 sewered population of 2,820, future population projections are presented in Table 2-1.

Table 2-1: Current & Future Populations

Year	1997	2007	2022
Population	2,820	3,620	5,267

2.1.4 Present and Projected Wastewater Flows and Loads

Wastewater influent flow data from January 1993 to September 1997 were analyzed to determine present wastewater flows and mass loads. Future wastewater flows and mass loads were projected using the present data and anticipated population growth. Current and projected flows and mass loads are summarized in Table 2-2. Refer to Chapter 4 for a complete list of definitions for the terms listed in the tables on the next page.

2.1.5 Regulatory Requirements

The City operates its wastewater treatment system under the jurisdiction of NPDES waste discharge Permit No. 100881. The permit requires the City to comply with certain Waste Discharge limitations, as summarized in a Table 6-1. Future waste load limitations are anticipated to remain the same as those set forth in their current discharge permit. To satisfy these mass load limits future effluent quality from the WWTP must improve as influent flows increase. The projected monthly effluent quality required to handle future influent flows and still comply with the current mass load limits are presented in Table 2-3.

Table 2-2: Future Influent Flows & Loadings

Parameter	1996	2007	2022
Population	2585	3620	5267
Wastewater Flow, mgd			
ADF	0.464	0.648	0.945
ADWF	0.288	0.405	0.587
AWWF	0.640	0.894	1.304
MMDWF	0.358	0.500	0.729
MMWWF	0.694	0.970	1.414
PDF	1.590	2.226	3.239
PIF	3.600	5.043	7.335
Wastewater Loads, lb/d			
BOD_5			
Avg. Day	429.2	601.0	874.5
Max. Month	968.0	1355.5	1972.3
Max. Day	1127.2	1578.5	2296.7
TSS			
Avg. Day	374.6	524.5	763.2
Max. Month	914.2	1280.2	1862.7
Max. Day	1158.1	1621.8	2359.6

Table 2-3: Future Effluent Quality Requirements (Beaver Creek)

		Monthly Average Effluent Concentrations	
Year	Flow	BOD_5, mg/l	TSS, mg/l
2007	Winter (AWWF)	11.3	18.7
	Winter (MMWWF)	10.3	17.3
2022	Winter (AWWF)	7.7	12.9
	Winter (MMWWF)	7.1	11.9

2.1.6 Surface Water Quality Impact Analysis

Mixing zone and oxygen sag models were developed to assess the surface water quality in Beaver Creek below the City's effluent outfall. Parameters that were modeled include chlorine, ammonia, and dissolved oxygen. Due to a lack of available stream flow data, stream bed measurements and limited flow monitoring were used to estimate the low flow condition in Beaver Creek during the wet weather discharge period.

The results of the model indicate that Beaver Creek is effluent dominated for the assumed low flow condition. The modeled chlorine and ammonia river concentrations exceed DEQ water quality standards. The predicted dissolved oxygen deficit does not appear to be as significant. A comprehensive field study is recommended to obtain representative flow data and water quality samples.

2.1.7 Evaluation of Alternatives

Based on an analysis of the City's wastewater facilities, three (3) viable wastewater treatment plant alternatives were developed and evaluated. These alternatives are summarized as follows:

- ***Alternative 1 - Aerated Lagoons*** - Adding surface aerators to the existing WWTP system would allow for needed oxygenation capacity with a minimum of new construction. The construction cost would consist of the capital cost for the aerators, the installation cost for the placement and anchoring of these units as well as the purchase and installation of baffling (if necessary), and the construction of any modifications to the lagoons. New piping will also be needed between lagoons if they are upgraded.

- ***Alternative #2 - Earthen Basin Extended Aeration (Biolac Process)*** -The Earthen Basin Extended Aeration (EBEA) is a system that could be built as a modification to the existing lagoon process. EBEA uses retention times of 30 to 70 days in conjunction with floating fine bubble diffusers to aerate and mix the wastewater. An in-basin clarifier or a separate clarifier can be used for settling. The EBEA process can yield an effluent with less than 10 mg/l BOD_5 and 10 mg/l TSS.

- ***Alternative #3 - Sequencing Batch Reactor (SBR)*** - The SBR is a multi-stage, activated-sludge process that typically consists of screening of grit and non-organic material, SBR treatment, and disinfection. Aeration, sedimentation/clarification, and decant processes are combined in a single reactor, rather than in separate structures for each. The SBR process can yield an effluent with less than 10 mg/l BOD_5 and 10 mg/l TSS.

Each of the above alternatives include provisions for a new influent pump station, headworks improvements, an effluent filtration system, chlorination/de-chlorination disinfection facilities, a new discharge outfall into Beaver Creek, and a reclaimed water pump station and irrigation system.

A matrix evaluation was performed on each alternative with respect to cost, implementation capability, operation and maintenance characteristics, performance reliability, flexibility, energy use and resource recovery, and its likely ability to address future environmental regulations. Based on the matrix evaluation, Alternative #1 - Aerated Lagoons is considered the most viable alternative for the City.

2.1.8 Recommended Improvement Plan

The estimated project cost for the recommended improvements is presented below:

ITEM	TOTALS	
CAPITAL IMPROVEMENT COSTS		
Collection System	$287,000	
Influent Pump Station	$300,000	
Headworks	$344,000	
Secondary Treatment (Aerated Lagoons)	$1,099,000	
Effluent Filtration	$439,000	
Disinfection	$130,000	
Effluent Disposal	$448,000	
Biosolids	$200,000	
	TOTAL CAPITAL COST	$3,247,000
INDIRECT COSTS		
Construction Contingencies	$487,050	
Engineering and Construction Management	$649,400	
Legal & Administration	$162,350	
	TOTAL INDIRECT COST	$1298,800
LAND ACQUISITION		$320,000
	TOTAL ESTIMATED PROJECT COST	**$4,865,800**

2.1.9 Operation and Maintenance Costs

The O&M costs for the Recommended Improvement Plan are estimated to be $332,791 per year.

2.1.10 Financing

The capital improvements should be financed by a grant and loan from Rural Development (RD) and a grant from the Oregon Economic Development Department (OEDD). The City should qualify for a RD loan at the intermediate rate (currently 4.835%, 40-year term). Grant monies should be requested from OEDD, under the Oregon community Development Block Grant Program (OCDBG), and from RD. The majority of grant monies would likely come from RD.

The total debt service and O&M expenses may be paid from tax revenues generated from the issuance of G.O. bonds, or from user fees generated from facilities funded through revenue bonds. Sewer rates will likely need to increase from the current rate of $20 per month to between $35 and $40 per month to fund the Recommended Improvements, assuming significant funding assistance from OEDD and RD. Otherwise, if the Recommended Improvements had to be funded entirely with revenue bonds, sewer rate increases would be even higher.

The local share should be authorized by the issuance of bonds (general obligation or revenue). Prior to determining the bond issue amount for which the City should secure authorization, BCI recommends continuing working with the Mid-Willamette Council of Governments and OEDD to determine the best available funding mix possible.

2.1.11 Project Schedule

The City received its third notice of noncompliance (NONs) by the DEQ on March 31, 1999 for violating its permit between June 1998 and January 1999. The June 1998 violations were for discharging out of the permitted season, and is a Class I violation. In December 1998 and January 1999, the DEQ cited three Class II and three Class III violations for exceeding BOD and TSS limits. These most recent violations document that the facility is not just experiencing capacity problems, but also currently has treatment problems that must be addressed in the very near future.

As a result of the Class I violation, the DEQ will be referring the City to its enforcement section with the recommendation to issue a Notice of Permit Violation (NPV), which is a formal enforcement action requiring a response within 5 working days of receipt. If the facility is not operating in compliance with its permit, the City will be required to submit a written proposal to bring the facility into compliance with the permit and all applicable regulations which include:

- A detailed plan and time schedule for achieving compliance in the shortest practicable time.
- A description of the interim steps that will be taken to reduce the impact of the permit violations until the permitted facility is in compliance with the permit.

DEQ has suggested that the City consider a Mutual Agreement and Order (MAO), which would allow certain violations to continue until modified or new facilities are constructed pursuant to a negotiated schedule contained in the MAO. **BCI** recommends that the City enter into a MAO as suggested by the DEQ, and negotiate an acceptable implementation schedule the and reduce the chances of fines and other enforcement action.

The following is a proposed implementation schedule for planning purposes.

Phase I - Effluent Irrigation

▸ DEQ Approval of Facilities Plan	May 1999
▸ Submit Funding Application	June 1999
▸ Conduct Funding Meeting	June 1999
▸ Start Reclaimed Water Use Plan	August 1999
▸ Submit Reclaimed Water Use Plan for Approval	December 1999
▸ DEQ Approves Reclaimed Water Use Plan	February 2000
▸ Acquire Land and Easements (Reclaimed Water System)	March 2000
▸ Start Detailed Design of Reclaimed Water System	April 2000
▸ Submit 50% Complete Design of Reclaimed Water System to DEQ	August 2000
▸ Meet with DEQ Regarding 50% Submission	August 2000
▸ Complete Detailed Design of Reclaimed Water System	December 2000
▸ DEQ Approval of Reclaimed Water System	February 2001
▸ Advertise For Construction Bids (Reclaimed Water System)	February 2001
▸ Receive Construction Bids (Reclaimed Water System)	April 2001
▸ Award Contracts (Reclaimed Water System)	April 2001
▸ Start Construction (Reclaimed Water System)	May 2001
▸ Submit Draft O&M Manual	July 2001
▸ Approval of O&M Manual	August 2001
▸ Complete Construction (Reclaimed Water System)	September 2001

Phase II - Disinfection System

▸ Start Detailed Design of WWTP Improvements	October 1999
▸ Submit 50% Complete Design of Disinfection System	May 2000
▸ Complete WWTP Design	December 2000
▸ DEQ Approval of WWTP Plans & Specifications	March 2001
▸ Advertise for WWTP Construction Bids	March 2001
▸ Receive WWTP Construction Bids	April 2001
▸ Award Contracts	May 2001
▸ Start WWTP Construction	July 2001
▸ Submit Draft O&M Manual	March 2002
▸ Approval of O&M Manual	May 2002
▸ Complete Construction	July 2002
▸ Performance Certification	July 2003

Phase III - Short Term Treatment Plant Upgrade

▸ Begin Mixing Zone Study	July 1999
▸ Begin Sludge Management Plan	September 1999
▸ Submit Sludge Management Plan	May 2000
▸ Submit Draft Mixing Zone Study	May 2000
▸ Meet with DEQ on Above Item	June 2000

▸ Approval of Sludge Management Plan	August 2000
▸ Begin Re-phased Design Work	September 2000
▸ Submit Completed Mixing Zone Study	October 2000
▸ DEQ Approval of Mixing Zone Study	December 2000
▸ Submit 50% Completed Detailed Plans	December 2000
▸ Submit 100% Completed Detailed Plans	February 2001
▸ Advertise/Receive Construction Bids	April 2001
▸ Start Construction	June 2001
▸ Submit Draft O&M Manual	February 2002
▸ Approval of O&M Manual	May 2002
▸ Complete Construction	July 2002
▸ Performance Certification	July 2003

Phase IV - Long Term Treatment Plant Upgrade

▸ Begin Detailed Design Work	January 2003
▸ Submit 50% Complete Detailed Plan	June 2003
▸ Meet with DEQ	June 2003
▸ Submit 100% Complete Detailed Design	January 2004
▸ DEQ Approval of Detailed Design	March 2004
▸ Advertise for Construction Bids	April 2004
▸ Receive Construction Bids	May 2004
▸ Start Construction	July 2004
▸ Submit Draft O&M Manual	March 2005
▸ Approval of O&M Manual	May 2005
▸ Complete Construction	July 2005
▸ Performance Certification	July 2006

Phase V - Infiltration and Inflow Work

- On Going Activity

2.2 RECOMMENDATIONS

The following recommendations are made to the City Council to implement the WWTP Improvement Plan:

1. Adopt the Wastewater Facilities Plan.
2. Authorize the detailed design of the recommended Reclaimed Water System.
3. Submit any necessary applications to OEDD and RD requesting funding assistance to finance the project.
4. Submit any necessary environmental information to RD to allow the preparation of any environmental assessments.
5. Acquire land and easements (Reclaimed Water System).
6. Hold a public meeting to provide information to, and solicit input and support from, users of the City's wastewater facilities.
7. Secure the authority to issue revenue or general obligation bonds in the amount to be determined by RD.
8. Secure approval from DEQ for the Reclaimed Water Use Plan.
9. Secure approval from DEQ for the Wastewater Facilities Plan.
10. Receive construction bids and award contracts for Reclaimed Water System.
11. Construct the Reclaimed Water System.
12. Authorize the detailed design of the recommended WWTP improvements and the preparation of plans and specifications.
13. Secure any necessary special permits for construction of WWTP improvements.
14. Secure approval of WWTP plan review approval from DEQ and OEDD.
15. Receive WWTP construction bids and award contracts.
16. Construct the WWTP improvement projects.

CHAPTER 3 - PLANNING AREA CHARACTERISTICS

3.1 GENERAL

The City of Aumsville is located in northwestern Oregon, in the western corner of Marion County. It is fifteen minutes southwest of Salem on State Highway 22, and five minutes northwest of Stayton. The population of Aumsville has grown to over 2,800 people at the end of 1997. The local economy is driven by a combination of construction, lumber, light manufacturing, and retail and wholesale establishments.

A vicinity map for the City of Aumsville is presented in Figure 3-1.

3.2 TOPOGRAPHY

The City of Aumsville is generally flat, with elevations ranging from 350 to 365 feet above sea level. It is situated on a terrace that generally slopes in the westerly direction. More abrupt elevation changes occur to the north and northwest of the city. The present wastewater treatment facility is surrounded by a floodplain.

3.3 GEOLOGY

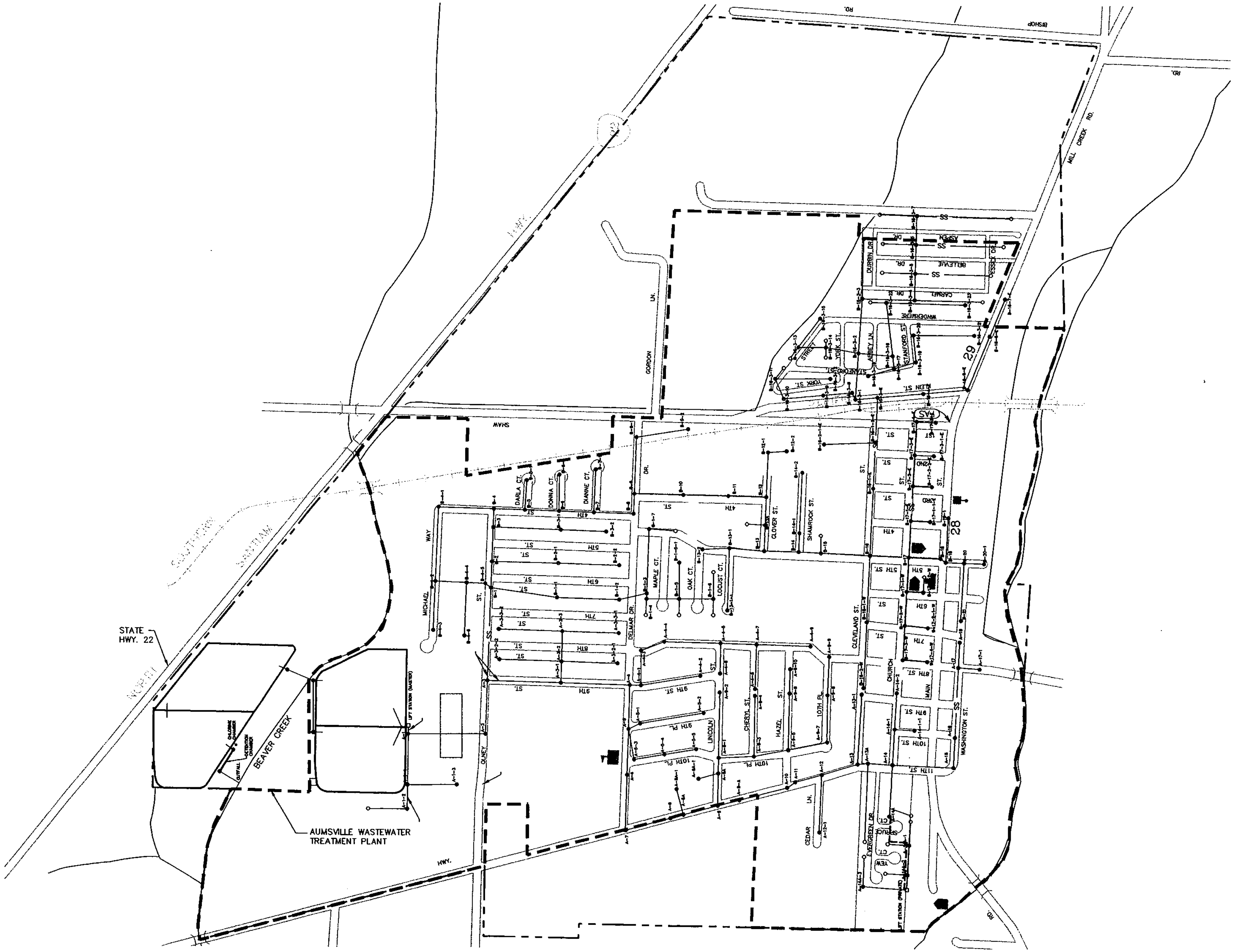
Aumsville is situated near the southern edge of the Pudding sub-basin in the Willamette Basin. The floor of this portion of the sub-basin consists of valley alluvial material. The silt, sand, and gravel of this formation constitute a major aquifer in the area of the Santiam alluvial fan.

3.4 NATURAL HAZARDS

Flooding is the most serious hazard in the area. Four small reservoir sites on Mill Creek and Beaver Creek are available for flood control storage. Flooding on the lower reaches of Mill Creek could not be effectively controlled by storage due to the sites being too far upstream.

3.5 WATER RESOURCES

Aumsville receives its water supply from a series of wells that tap a major aquifer of sand and gravel. Five wells are capable of producing 1.1 million gallons of water per day. The wells discharge directly, and without treatment, into the distribution system. Water storage is provided by an elevated reservoir with 100,000 gallons capacity and a newly constructed ground reservoir with one million gallon capacity.



LEGEND

URBAN GROWTH BOUNDARY

CITY LIMITS

GRAPHIC SCALE

0 200 400 800

(IN FEET)

REVISIONS:			
REV. NO.	DATE	DESCRIPTION	APPV'D

BALFOUR CONSULTING, INC.
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DATE:
DESIGN:
DRAWN: JDT
APPV'D: ERC

WASTEWATER FACILITIES PLAN
CITY OF AUMSVILLE, OREGON

VICINITY MAP
FIGURE 3–1

JOB NO: 140.01

SHEET

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3.6 LAND USE PLANNING

The type of development and pattern of existing land use in Aumsville is an important consideration in land use planning. The type and extent of existing land use activities aides in determining the location and amount of land required for future development.

In order to obtain an inventory of existing land uses, a survey was conducted in 1982 (updated, 1985). Acreage is tabulated according to seven use categories. Based on this survey, land use within the city is comprised of 53.7% Single-Family, 21.5% Streets & Right-of-way, 12.4% Public & Semi-Public, 6.7% Industrial, 4.4% Commercial, and 1.3% Multi-Family.

3.7 SOCIOECONOMIC ENVIRONMENT

3.7.1 Economic Conditions and Trends

Regional economic trends will heavily influence the population growth in Aumsville. State economic forecasters call for continued growth in the Willamette Valley over the next 5 to 10 years. Local manufacturing companies include NorPac Foods (600 employees), Blazer Industries (145 employees), Modern Building Systems, Inc., (100 employees), Bruce Packing Co., (70 employees), and Ektron Industries, Inc., (40 employees).

Aumsville is an attractive community in which to live, primarily due to its rural character and central location. Located eight miles from Salem, Aumsville offers good access to the I-5 corridor, State Highway 22.

Another significant growth driver for Aumsville is its relatively low housing costs. Based on 1990 data, the median housing cost is reported as $44,000 compared to $60,000 for Marion County. Combined with its short commute to Salem, City officials believe that the relatively low housing costs will result in significant growth rates over the planning period.

3.7.2 Population Analysis

In order to properly design Aumsville's wastewater system, a realistic population projection must be made. The projection is made based on past and present growth rates, growth in surrounding communities and similar areas, and possible future land and highway development. Aumsville's wastewater system will be designed for a 20 year population projection.

In 1980, the United States Census Bureau reported Aumsville's population as 1,432. In 1997 it had reached over 2,800, and is expected to continue to increase at an even higher growth rate. The annual growth rate in Marion County has been approximately 2.2% since 1970. Access to Aumsville will improve with on-going improvements to State Highway 22. Currently there is space within the UGB for considerable growth. There is a project with 80 units of housing pending, and another 40-acre development is being considered.

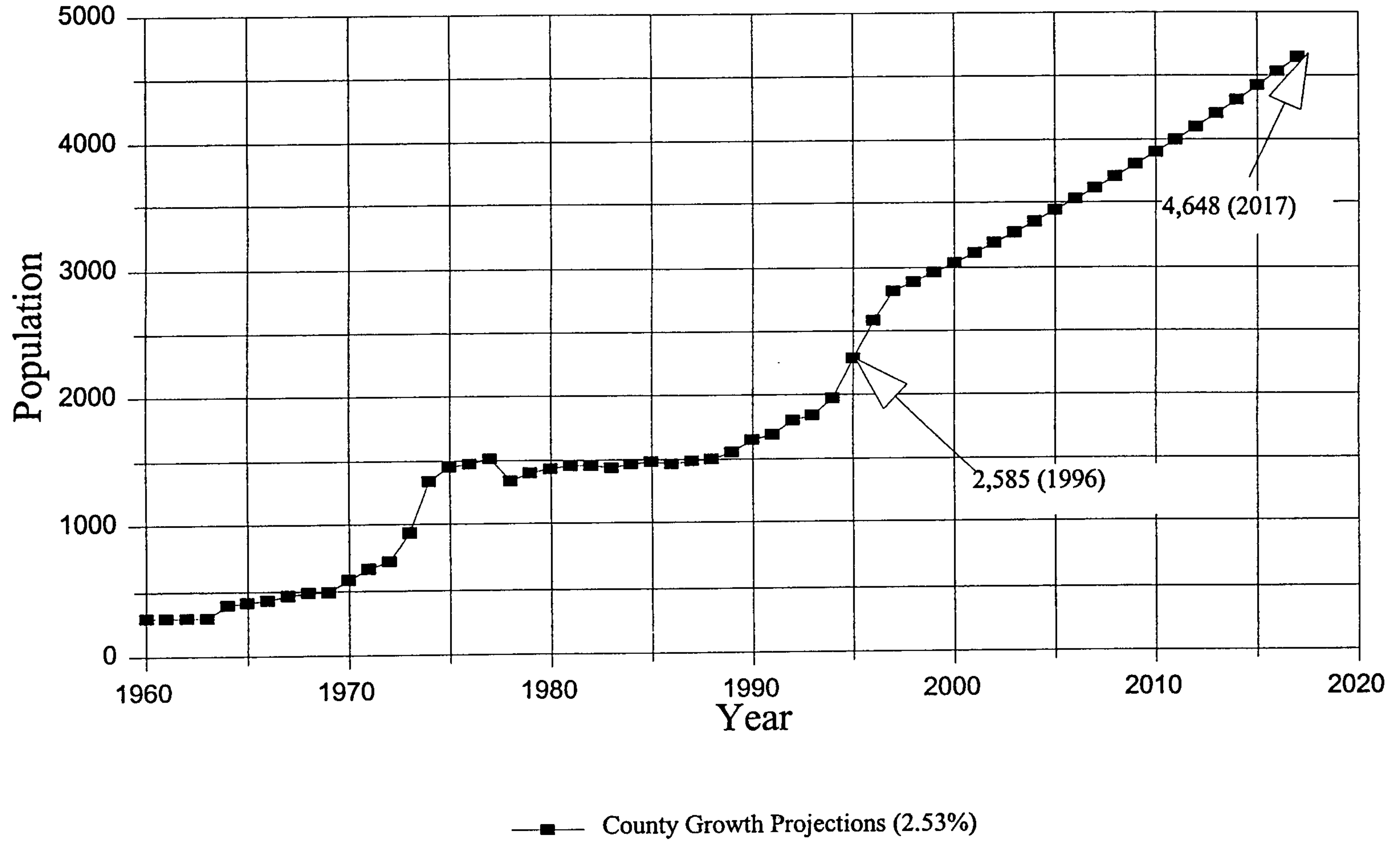
The population estimates used in this report project Aumsville's population to increase at an annual rate of 2.53 percent between 1997 and 2022, in accordance with Marion Count Board of Commissioners approved growth rate. Using the above growth projections and a 1997 sewered population of 2,820, future population projections are presented in Table 3-1 and shown in Figure 3-2.

Table 3-1: Current & Future Populations

Year	1997	2007	2022
Population	2,820	3,620	5,267

Figure 3-2: Population Projections

20-Year Timeline



CHAPTER 4 - WASTEWATER CHARACTERISTICS

4.1 DEFINITION OF TERMS

Aerobic - Microorganisms living in the presence of free oxygen.

Anaerobic - Microorganisms capable of living without the presence of free oxygen.

Anoxic Denitrification - The process by which nitrate nitrogen is converted biologically to nitrogen gas in the absence of oxygen.

Average Daily Flow (ADF) - The total influent flow in a time period divided by the total number of days in that time period.

Average Dry Weather Flow (ADWF) - The total influent flow during the summer months (May 1- October 31) divided by the number of days in that time period.

Biochemical Oxygen Demand (BOD)- The amount of oxygen required to stabilize the organic material in sewage by aerobic processes.

Biosolids - Solid and semi-solid residuals resulting from wastewater treatment operations. Sludge must periodically be removed from treatment systems.

Chlorine Residual - A residual of chlorine used in disinfecting wastewater. Chlorine residual can exist in two (2) forms: combined or free. The specific form is dependent on the rate of formation, which is controlled by the pH and temperature. A free chlorine residual is the most effective in achieving disinfection.

Fecal Coliform - Bacteria which are used as an indicator of fecal pollution.

Industrial Wastes - Waterborne wastes produced as a result of manufacturing or processing operations.

Infiltration - Water which 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.

Base Infiltration - Water which enters the sewerage system from the surrounding soil during periods of low groundwater levels.

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

Rainfall-Induced Infiltration - Additional infiltration which enters the sewerage system during and for several days after a period of rainfall. Rainfall often percolates into sewer ditches, especially ditches with granular backfill, and establishes a perched water table. This water then infiltrates into faulty sewers and manholes.

Inflow - Stormwater runoff which enters the sewerage system only during or immediately after rainfall. Points of entry may include connections with roof and area drains, storm drain connections, and holes in manhole covers in flooded streets.

Lagoon (Stabilization Pond) - A shallow basin constructed by excavating the ground and diking, for the purpose of treating raw sewage by storage under conditions that favor natural biological treatment and accompanying bacterial reduction.

Maximum Monthly Dry Weather Flow (MMDWF) - The flow associated with the average monthly flow in the rainiest summer month (May-October) with high groundwater. West of the Oregon Cascades, this month usually corresponds to May.

Maximum Monthly Wet Weather Flow (MMWWF) - The flow associated with the average monthly flow in the rainiest winter month (November-April) with high groundwater. West of the Oregon Cascades, this month usually corresponds to January.

Maximum Weekly Flow (MWF) - The greatest weekly average flow in a given time period.

Nitrification - The process by which ammonia nitrogen is converted to nitrite nitrogen, then to nitrate nitrogen by the Nitrosomonas and Nitrobacter bacteria.

NPDES - National Pollutant Discharge Elimination System.

Peak Daily Flow (PDF) - The maximum total daily flow occurring in a given time period.

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.

pH - The degree of acidity or alkalinity of waste water, 7.0 being neutral, a lower number being acidic, and a higher number being basic.

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

RAS - Return Activated Sludge.

Total Suspended Solids (TSS) - All of the solids in sewage that can be removed by settling or filtration. The quantity of TSS removed during treatment impacts the sizing of sludge handling and disposal processes, as well as the effectiveness of disinfection with chlorine.

WAS - Waste Activated Sludge.

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

4.2 WASTEWATER VOLUME

Wastewater flow rates are important factors in sizing wastewater collection and treatment components. In western Oregon, peak sewage flows are generally linked to rainfall. This usually occurs during the wet weather season when high groundwater coupled with a significant rainfall event lead to a massive influx of infiltration and inflow (I & I) into the collection system.

Infiltration becomes a problem when groundwater rises above the level of the sewage collection system, effectively submerging the pipe network. When this occurs, any deteriorated portions of the collection system are subject to leakage. This deterioration includes cracked or broken pipes, which is inevitable in any sewer system, and unsealed pipe or manhole joints. Infiltration is particularly a problem in older sewer systems which were constructed of materials with none or poor quality joint seals.

Inflow is the result of stormwater discharged into the collection system directly through roof and area drains, combined stormwater and sanitary sewer networks, and holes in manhole lids. Inflow occurs during and/or immediately following a storm event.

Due to this connection between rainfall and peak sewage flows, it is good practice to establish a relationship between actual sewage flow data and actual precipitation data. The DEQ has developed guidelines for calculating sewage flow rates in areas significantly impacted by precipitation, based on probability of system failure for both the dry (May 1 through October 31) and wet (November 1 through April 30) weather periods. In order to incorporate these impacts into flow calculations, the DEQ has defined the following terms: $MMDWF_{10}$, $MMWWF_5$, $PDAF_5$, and PIF_5. For the dry weather period, the DEQ has determined that a 10% probability of system failure is acceptable for a design basis, which correlates to an average of one system failure every 10 years. For the wet weather period, the DEQ has established a 20% probability of system failure as an acceptable design standard, which corresponds to an average of one system failure every 5 years. The regulations for the dry weather period are more stringent due to greater risk of damage to human health and the environment during the lower natural stream flows associated with this period.

Flows measured at the WWTP are directly influenced by the influent pump station and the accuracy of the metering device. The peak flow that can be discharged into the WWTP is approximately 1250 gpm (1.8 mgd) with both pumps operating. Under peak flow conditions with the lagoons at or approaching the maximum water surface elevation, the discharge from the parshall flume will become submerged, causing the flow meter to read high. A review of the daily flow data revealed that on 9 occasions the measured daily flow exceeded the estimated 2-pump capacity. These flow records were removed from the data set as part of this analysis. It was also noted that on 166 occasions the total daily flow exceeded the capacity of one pump. However, these records were not removed from the data set. The 166 data points represent 9.6% of the total set, and it was determined that without any other information to substantiate removal of this data, they would be used in all statistical evaluations. It is assumed that these flows are possibly higher than actual, and at most would cause the statistical flow evaluations to yield conservatively high flows.

Monthly influent flow data collected from January 1993 to February 1997 are summarized in Table 4-1. This flow data was used to determine both current design factors and for the twenty (20) year planning period. The following sections summarize the dry and wet weather flow evaluations.

4.2.1 Dry Weather Flow

Dry weather flows include the Average Dry Weather Flow (ADWF), Maximum Monthly Dry Weather Flow (MMDWF), and Average Annual Daily Flow (ADF). The ADWF is the average daily flow during the dry weather period. The $MMDWF_{10}$ is the monthly average flow, with a 10-year recurrence, in the wettest summer month with high groundwater, and consists of the domestic sewage and infiltration and inflow. In the Aumsville area, and in most of the areas west of the Oregon Cascades, the rainiest month during the dry weather period is May.

The ADWF is calculated by averaging the flow rates during the dry weather period. The $MMDWF_{10}$ is determined by plotting the total monthly precipitation values against the corresponding total monthly average influent flows for the dry weather months. A linear regression is completed, and the $MMDWF_{10}$ is the point corresponding with 5.07 inches of rainfall (see Figure 4-1), which is the monthly rainfall accumulation during May with a 10% probability of occurrence. The ADF is calculated by adding the total influent flow over the year divided by 365 days. Flows are summarized in Table 4-2.

Table 4-1: Monthly Average Influent Flow and Precipitation Data

	1993		1994		1995		1996		1997		Flow Averages (mgd)	Month
	Flow (mgd)	Precip (inches)	Flow (mgd)	Precip (inches)	Flow (mgd)	Precip (inches)	Flow (mgd)	Precip (inches)	Flow (mgd)	Precip (inches)		
Nov			0.241	0.056	0.614	0.420	0.622	0.319	0.655	0.353	0.533	Nov
Dec			0.363	0.266	0.654	0.261	0.778	0.312	1.003	0.575	0.700	Dec
Jan	1.017	0.172	0.506	0.211	0.656	0.326	0.810	0.418	0.800	0.321	0.758	Jan
Feb	0.699	0.089	0.458	0.219	0.601	0.195	0.870	0.489	0.582	0.167	0.642	Feb
Mar	0.966	0.182	0.415	0.136	0.462	0.178	0.452	0.182	0.672	0.302	0.574	Mar
Apr	1.198	0.349	0.383	0.116	0.409	0.195	0.516	0.225	0.442	0.187	0.627	Apr
Jun	0.784	0.174	0.206	0.084	0.235	0.107	0.252	0.040	0.312	0.113	0.369	Jun
Jul	0.371	0.051	0.165	0.004	0.208	0.020	0.219	0.040	0.231	0.031	0.241	Jul
Aug	0.314	0.050	0.161	0.000	0.189	0.100	0.201	0.012	0.201	0.031	0.216	Aug
Sep	0.287	0.001	0.160	0.051	0.184	0.111	0.211	0.101			0.211	Sep
Oct	0.278	0.067	0.191	0.198	0.320	0.186	0.293	0.252			0.270	Oct

Table 4-2: Wastewater Flow Summary

Flow Condition		Flow (mgd)	Flow (gpcd)	Peaking Factor
Average Daily Flow	ADF	0.464	179	NA
Average Dry Weather Flow	ADWF	0.288	112	0.62
Maximum Monthly Dry Weather Flow	MMDWF	0.358	138	0.77
Average Wet Weather Flow	AWWF	0.640	247	1.37
Maximum Monthly Wet Weather Flow	MMWWF	0.694	268	1.49
Peak Daily Flow	PDF	1.59	615	3.42
Peak Instantaneous Flow	PIF	3.6	1,393	7.76

4.2.2 Wet Weather Flow

Wet weather flows consist of the Average Wet Weather Flow (AWWF), the Maximum Monthly Wet Weather Flow (MMWWF), the Peak Daily Average Flow (PDAF), and the Peak Instantaneous Flow (PIF). The AWWF is an average of all the daily flow averages of only the wet weather months. The $MMWWF_5$ is similar to the $MMDWF_{10}$, except it uses the average of the wettest winter month with high groundwater, and is based on a 5-year recurrence. The $MMWWF_5$ is influenced by inflow and infiltration from the high groundwater levels, which are usually reached in January. The $MMWWF_5$ was determined using the same procedure as for the $MMDWF_{10}$. Figure 4-2 shows the $MMWWF_5$ The PDAF is determined from historical data, and is recorded as the maximum of all the average daily flows during the period of high ground water. The DEQ has also defined a $PDAF_5$ based on statistical evaluation using daily flow and precipitation. However, the calculation is very inconsistent, and will not be used as a design standard in this report. Following DEQ guidelines, the PIF can be determined in two ways. A diurnal peaking factor can be used, or it can be extrapolated from a known historical Peak Daily Average Flow. The latter method was applied for this plan, as no diurnal flow data is available. This extrapolation is shown in Figure 4-3. The flows are summarized in Table 4-2. Figure 4-2 shows the $MMWWF_5$ graph.

Figure 4-1: MMDWF

May through October, 1993 to 1997

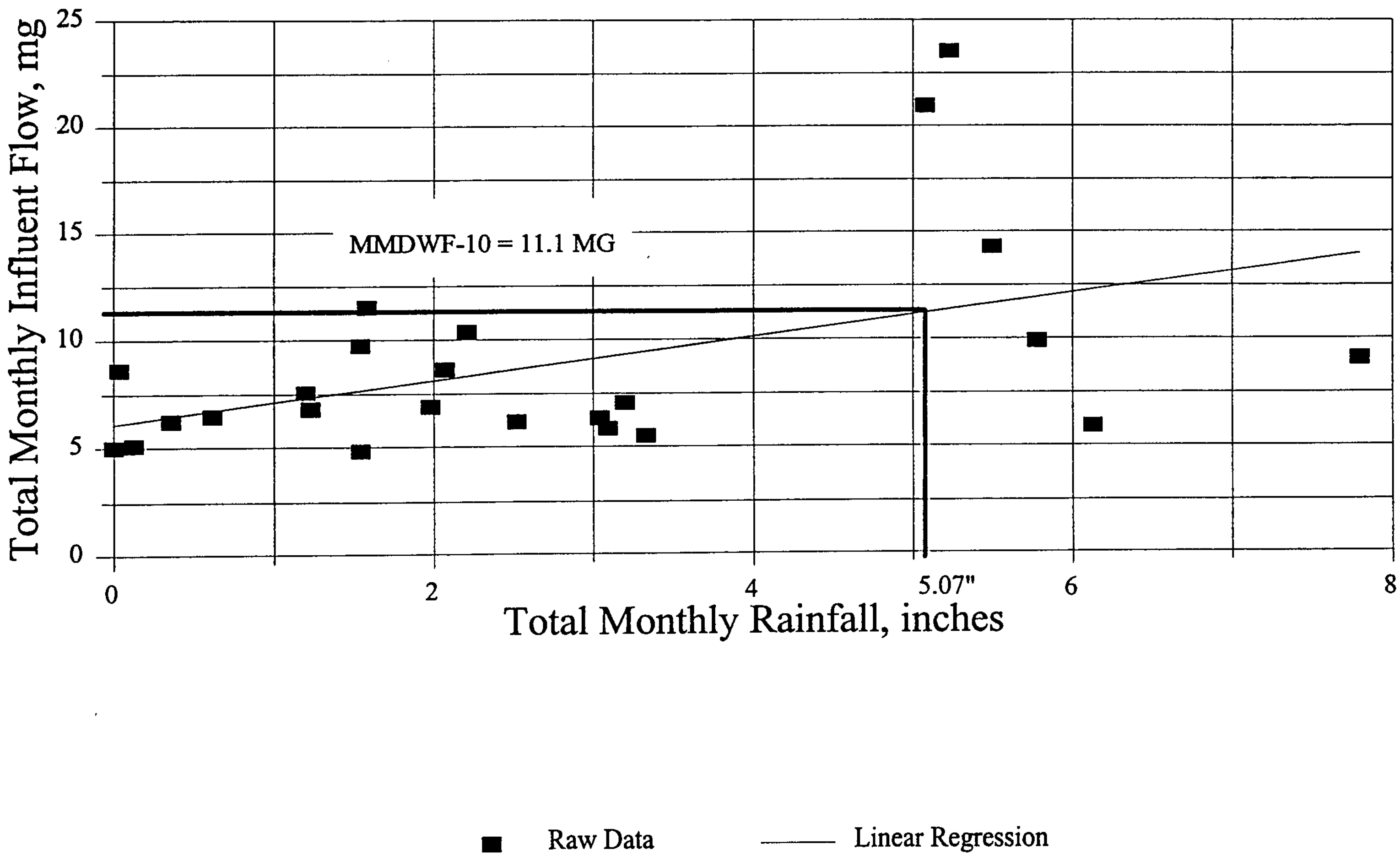


Figure 4-2: MMWWF

November through April, 1993 to 1997

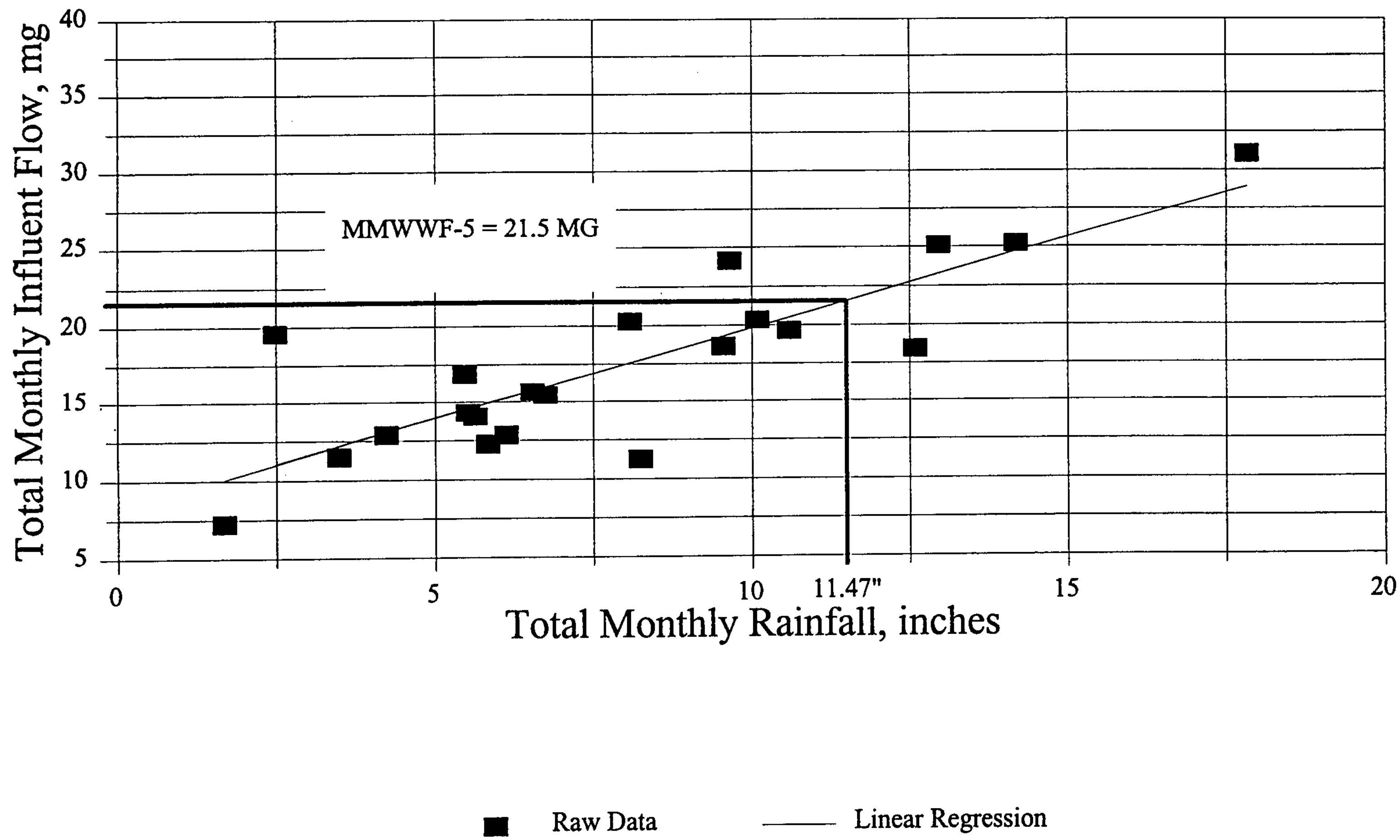
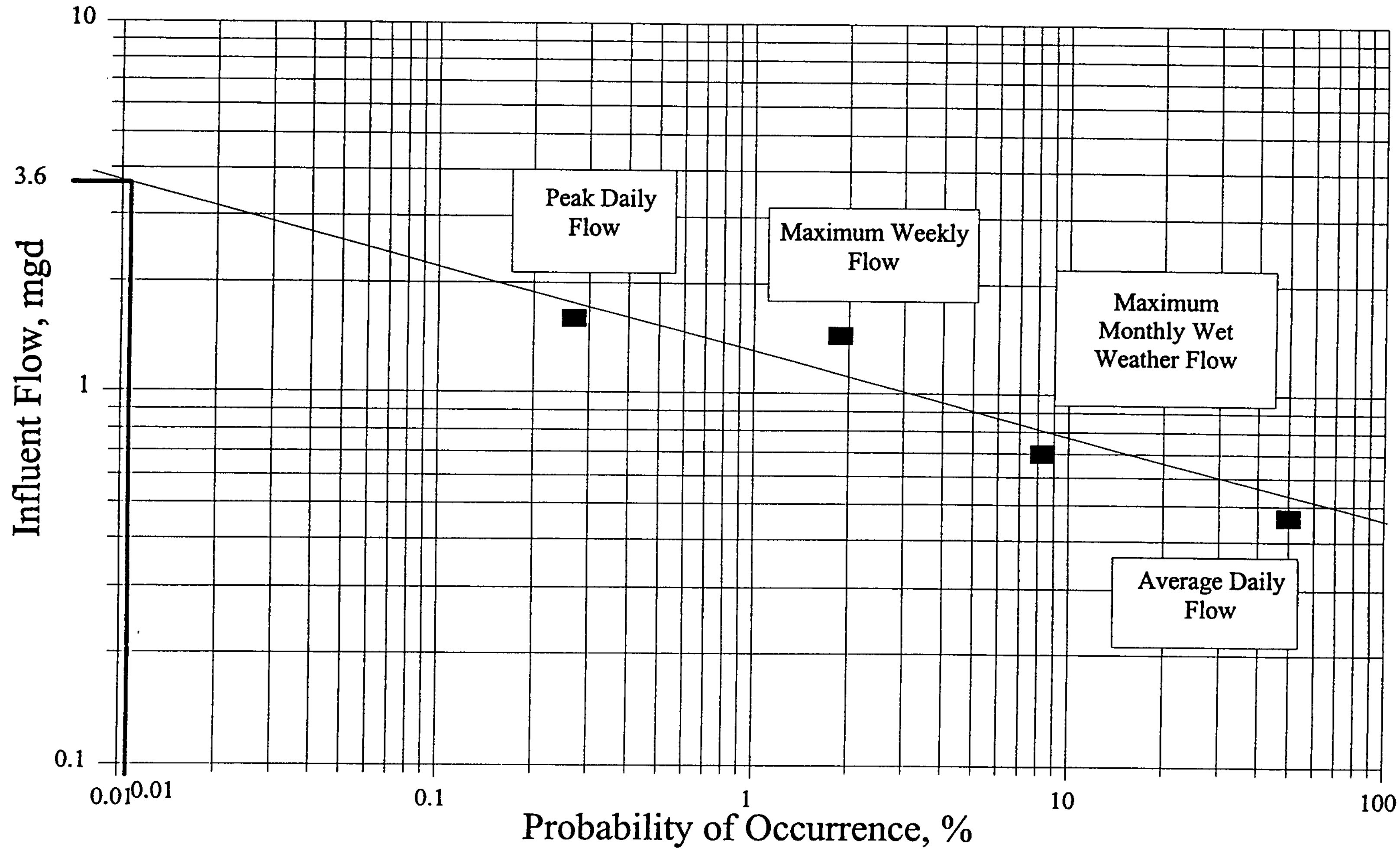


Figure 4-3: PIF

January 1993 to August 1997



4.2.3 Infiltration and Inflow (I&I) Flow

As discussed previously, I & I is that portion of sewage flow attributable to rainfall, directly through stormwater inflow and/or indirectly through infiltration of groundwater. The peak sewage flows that result from these intrusions can cause system failures as the collection and treatment systems may not be sized to adequately handle these flow rates. Consequently, raw sewage is bypassed into surface waters, creating a hazard to both human health and the environment.

Efforts to reduce I & I benefit a community in several ways. First, the capacity of the sanitary sewer system is used more efficiently for the collection, treatment and disposal of wastewater, which effectively reduces operations costs. It can also reduce the costs associated with any capital improvements to the system, particularly those associated with expanding the collection and treatment system, by minimizing the peak design flow rate. Second, reductions in I & I decrease the risk of raw sewage bypasses. This is very important in maintaining compliance with the community's NPDES discharge permit.

The City of Aumsville produces a per capita flow rate of 179 gpcd based on the ADF (see Table 4-2). According to the EPA, infiltration is defined as "excessive" if the highest average daily flow over a period of 7 to 14 days during a period of high groundwater and dry weather markedly exceeds 120 gpcd. Since the City of Aumsville's annual average flow rate, which is tempered by the lower flowrates of the dry weather period, is significantly greater than the 120 gpcd weekly or bi-weekly average identified by the EPA, there is strong evidence that the City is experiencing substantial infiltration and/or inflow problems.

Observations of the flow rate and precipitation data from the Daily Monitoring Reports for January 1993 to February 1997 show that the sewage flow rate response to a storm event generally does not occur until the day following the precipitation event. Since inflow is a direct contributor to the flow rate either during or immediately following a storm event, this delayed response indicates that the City is not currently experiencing excessive inflow to the sewage collection system.

4.2.4 Summary of Existing Flows

Flows at the treatment plant range from an Average Dry Weather Flow (ADWF) of 0.3 million gallons per day (mgd), to a Peak Instantaneous Flow (PIF) of 3.6 mgd. The relatively large difference between these flows suggests that the system is experiencing significant I&I. Another indication of an I&I problem is the fact that the average influent BOD concentration from August 1996 was 238 mg/l, compared to 33 mg/l in January 1997, and the average influent TSS concentration from August 1996 was 164 mg/l, as compared to 53 mg/l from January 1997. While the corresponding average BOD mass loading rate is somewhat lower in January 1997, this is still strong evidence that the wastewater is being diluted by I&I during the wet weather season.

Relatively high peaking factors such as these are common for older sewer systems in the Willamette Valley due to deterioration of the collection networks, excessive rainfall, and high groundwater conditions.

4.3 WASTEWATER COMPOSITION

4.3.1 Analysis of Plant Records

The City's Wastewater Discharge Permit requires the following monitoring program:

- BOD_5, TSS, and fecal coliform (once every two weeks)
- pH (three times per week)
- Chlorine residual (daily)

In addition, the discharge permit requires that the total usage of chlorine is recorded daily, and that the flow meter be calibrated annually. The City records this information in Daily Monitoring Reports (DMR's), which were used to evaluate the City's wastewater composition.

Wastewater composition data was evaluated for the period of January 1993 through February 1997. As shown in Table 4-3, the average influent values for BOD_5 and TSS are 147 mg/l and 121 mg/l, respectively, although they have reached as high as 486 mg/l and 354 mg/l, respectively. The typical composition of untreated domestic wastewater ranges from 100 to 400 mg/l BOD_5 and 100 to 350 mg/l TSS.

The City's permit also includes effluent discharge limits, which are summarized in Chapter 6. A review of the City's DMR's over the winter discharge months during the period of 1993 to 1997 revealed the following:

- Effluent BOD_5 concentrations have consistently been well below the permitted monthly discharge limit of 30 mg/L. The average effluent BOD_5 concentration is 6.1 mg/L, with a maximum concentration of 20 mg/L.

- Effluent BOD_5 mass loads have generally been below the permit limit of 84 lb/day, with an average of 43.6 lb/day. Occasional exceedances have occurred, with the maximum reported at 151.6 lb/day.

Table 4-3: Wastewater Characteristics

	Influent							Effluent							
	Flow	BOD5			TSS			Flow	BOD5			TSS			FC
Date Sampled	(mgd)	(mg/l)	(lb/d)	monthly lb/d**	(mg/l)	(lb/d)	monthly lb/d**	(mgd)	(mg/l)	Removal	(lb/d)	(mg/l)	Removal	(lb/d)	(#/100 ml)
06-Jan-93	1.620	44	587.7		36	486.4		1.330	5.0	88.5%	55.5	4.9	86.4%	54.4	<2
27-Jan-93	1.180	83	818.8	703.3	87	851.3	668.8	1.230	7.9	90.5%	81.0	16.4	81.0%	168.2	4
03-Feb-93	0.690	96	549.6	684.2	68	391.3	621.3	1.120	1.2	98.7%	11.2	24.0	64.7%	224.2	39
17-Feb-93	0.530	70	307.2	428.4	56	247.5	319.4	0.930	9.8	85.9%	76.0	12.0	78.6%	93.1	46
10-Mar-93	0.650	150	813.2	560.2	166	899.9	573.7	0.730	9.7	93.5%	59.1	11.0	93.4%	67.0	4
24-Mar-93	1.980	68	1122.9	968.0	48	792.6	846.3	1.410	9.9	85.5%	115.9	11.0	77.1%	129.4	29
07-Apr-93	1.000	79	656.4	889.6	70	583.8	688.2	1.270	4.2	94.6%	44.7	2.8	96.0%	29.7	<2
21-Apr-93	1.070	68	606.8	631.6	57	508.7	546.2	1.090	6.3	90.7%	57.3	1.5	97.4%	13.6	8
12-May-93	0.700	74	433.2	520.0	103	601.3	555.0	0.850	2.0	97.3%	14.5	3.9	96.2%	27.6	4
27-May-93	0.480	168	672.5	552.9	150	600.5	600.9	No Discharge							
09-Jun-93	1.060	128	1127.2	899.8	131	1158.1	879.3	0.530	6.4	95.0%	28.3	1.3	99.0%	5.7	<2
23-Jun-93	0.500	144	600.5	863.8	120	500.4	829.2	No Discharge							
09-Jul-93	0.380	205	649.7	625.1	174	551.4	525.9	No Discharge							
21-Jul-93	0.390	86	280.7	465.2	25	82.3	316.9	No Discharge							
13-Aug-93	0.310	221	570.1	425.4	315	814.4	448.3	No Discharge							
25-Aug-93	0.310	201	519.0	544.5	160	413.7	614.0	No Discharge							
07-Sep-93	0.340	177	501.9	510.5	216	612.5	513.1	No Discharge							
27-Sep-93	0.330	194	532.6	517.2	102	280.7	446.6	No Discharge							
03-Oct-93	0.260	246	533.4	533.0	188	407.7	344.2	No Discharge							
22-Oct-93	0.300	189	472.9	503.2	104	260.2	333.9	No Discharge							
17-Nov-93	0.290	306	740.1	606.5	264	638.5	449.4	0.620	3.1	99.0%	16.0	1.3	99.5%	6.9	4
24-Nov-93	0.160	347	463.0	601.6	72	96.1	367.3	0.480	2.0	99.4%	8.0	6.5	91.0%	26.0	7
08-Dec-93	0.480	127	508.4	485.7	60	240.2	168.1	0.550	4.2	96.7%	19.3	3.7	93.8%	17.0	3
22-Dec-93	0.280	110	256.9	382.6	45	105.1	172.6	0.650	1.9	98.3%	10.3	6.0	86.7%	32.5	4
14-Jan-94	0.420	112	392.3	324.6	40	140.1	122.6	0.360	4.3	96.2%	12.9	7.0	82.5%	21.0	7
26-Jan-94	0.470	75	295.2	343.7	63	246.9	193.5	0.540	7.9	89.5%	35.6	13.7	78.3%	61.7	27
09-Feb-94	0.250	165	344.0	319.6	50	104.3	175.6	0.360	8.6	94.8%	25.8	9.5	81.0%	28.5	24
24-Feb-94	0.660	197	1084.4	714.2	87	481.1	292.7	0.350	8.4	95.7%	24.5	13.0	85.1%	37.9	26
09-Mar-94	0.380	126	399.3	741.8	82	259.9	370.5	0.390	3.0	97.6%	9.8	17.0	79.3%	55.3	43
30-Mar-94	0.290	159	384.6	391.9	147	355.5	307.7	0.500	19.0	88.1%	79.2	20.0	86.4%	83.4	66
13-Apr-94	0.530	111	490.6	437.6	112	495.1	425.3	0.400	1.3	98.8%	4.3	5.3	95.3%	17.7	7
29-Apr-94	0.220	486	891.7	691.2	354	649.5	572.3	0.180	3.2	99.3%	4.8	2.0	99.4%	3.0	12
13-May-94	0.210	171	299.5	595.6	156	273.2	461.4	No Discharge							
25-May-94	0.200	316	527.1	413.3	130	216.8	245.0	No Discharge							
10-Jun-94	0.140	238	277.9	402.5	214	249.9	233.4	No Discharge							
29-Jun-94	0.180	209	313.8	295.8	92	138.1	194.0	No Discharge							
15-Jul-94	0.170	178	252.4	283.1	172	243.9	191.0	No Discharge							
27-Jul-94	0.160	122	162.8	207.6	110	146.8	195.3	No Discharge							
10-Aug-94	0.170	158	224.0	193.4	108	153.1	150.0	No Discharge							
29-Aug-94	0.160	232	309.6	266.8	172	229.5	191.3	No Discharge							
09-Sep-94	0.170	145	205.6	257.6	262	371.5	300.5	No Discharge							
21-Sep-94	0.160	234	312.2	258.9	174	232.2	301.8	No Discharge							
12-Oct-94	0.160	192	256.2	284.2	120	160.1	196.2	No Discharge							
26-Oct-94	0.170	237	336.0	296.1	154	218.3	189.2	No Discharge							
16-Nov-94	0.500	95	397.4	366.7	85	354.5	286.4	0.810	3.7	96.1%	25.0	1.3	98.5%	8.8	13
30-Nov-94	0.770	45	290.9	344.2	21	132.3	243.4	0.740	3.2	92.9%	19.7	9.0	56.3%	55.5	1
14-Dec-94	0.560	61	283.0	287.0	86	401.7	267.0	1.100	4.9	91.9%	45.0	14.5	83.1%	133.0	20
28-Dec-94	0.810	37	249.9	266.5	44	297.2	349.4	1.430	3.9	89.5%	46.5	9.2	79.1%	109.7	4
06-Jan-95	0.350	160	467.0	358.5	133	388.2	342.7	1.290	4.3	97.3%	46.3	10.2	92.3%	109.7	12
18-Jan-95	0.850	56	399.8	433.4	64	453.7	421.0	1.630	7.3	87.1%	99.2	8.6	86.6%	116.9	29
10-Feb-95	0.380	112	355.9	377.9	51	161.6	307.7	0.860	6.8	93.9%	48.8	5.2	89.8%	37.3	<2
22-Feb-95	0.690	66	379.8	367.9	65	375.8	268.7	0.920	4.7	92.9%	36.1	6.4	90.2%	49.1	<2
15-Mar-95	0.630	49	259.0	319.4	21	108.8	242.3	1.110	2.4	95.2%	22.0	3.2	84.5%	29.6	6
29-Mar-95	0.370	128	395.0	327.0	126	388.8	248.8	1.170	4.2	96.7%	41.0	1.0	99.2%	10.1	6
07-Apr-95	0.310	86	222.3	308.7	98	253.4	321.1	1.100	1.9	97.8%	17.2	1.6	98.4%	14.7	2
26-Apr-95	0.340	240	680.5	451.4	236	669.2	461.3	1.010	18.0	92.5%	151.6	0.8	99.7%	6.7	<2
10-May-95	0.360	118	354.3	517.4	147	441.4	555.3	No Discharge							
25-May-95	0.240	133	266.2	310.2	119	238.2	339.8	No Discharge							
09-Jun-95	0.220	240	440.4	353.3	240	440.4	339.3	No Discharge							
21-Jun-95	0.280	148	345.6	393.0	133	310.6	375.5	No Discharge							
07-Jul-95	0.210	230	402.8	374.2	250	437.9	374.2	No Discharge							
20-Jul-95	0.200	301	502.1	452.4	120	200.2	319.0	No Discharge							
11-Aug-95	0.200	149	248.5	375.3	173	288.6	244.4	No Discharge							
24-Aug-95	0.170	188	266.5	257.5	230	326.1	307.3	No Discharge							
07-Sep-95	0.180	193	289.7	278.1	172	258.2	292.2	No Discharge							
22-Sep-95	0.170	222	314.8	302.2	246	348.8	303.5	No Discharge							
05-Oct-95	0.240	107	214.2	264.5	122	244.2	296.5	No Discharge							
25-Oct-95	0.400	199	663.9	439.0	48	160.1	202.2	0.540	11.6	94.2%	52.2	2.5	94.8%	11.3	6
15-Nov-95	0.730	70	426.2	545.0	89	541.8	351.0	0.540	3.0	95.7%	13.5	0.9	99.0%	4.1	2
29-Nov-95	0.860	62	444.7	435.4	61	437.5	489.7	1.200	2.1	96.6%	21.0	2.4	96.1%	24.0	57
15-Dec-95	1.560	49	642.7	543.7	60	780.6	609.1	1.150	3.1	93.7%	29.7	3.0	95.0%	28.8	33
28-Dec-95	0.390	114	370.8	506.8	103	335.0	557.8	0.920	2.2	98.1%	16.9	3.0	97.1%	23.0	16
11-Jan-96	0.730	56	340.9	355.9	42	255.7	295.4	0.810	3.4	93.9%	23.0	5.8	86.2%	39.2	23

- BOD_5 removal efficiency has generally been above the permitted monthly minimum of 85 percent, with an average of 92.4 percent.

- Effluent TSS concentrations have been below the permitted monthly discharge limit of 50 mg/L, with an average effluent TSS concentration of 7.8 mg/L and a maximum concentration of 24 mg/L.

- Effluent TSS mass loads have generally complied with the permit limit of 140 lb/day, with an average of 58.2 lb/day and a maximum of 224.2 lb/day.

- TSS removal efficiency has generally been above the permitted monthly minimum of 75 percent, with an average of 87.7 percent.

- Fecal coliform samples have consistently fallen below the permitted monthly discharge limit of 200 organisms per 100 ml of wastewater. The average sampling result is 20.9 organisms per 100 ml of wastewater, with a maximum of 115 organisms per 100 ml of wastewater.

- The pH of the wastewater has been neutral, generally ranging between 6.9 and 7.2 pH units.

The historical effluent composition is summarized in Table 4-3 and Figures 4-4, 4-5, 4-6A, and 4-6B. A summary of the average influent wastewater characteristics for the dry and wet weather periods is presented in Table 4-4.

Figure 4-4: Effluent Concentrations

City of Aumsville, Oregon

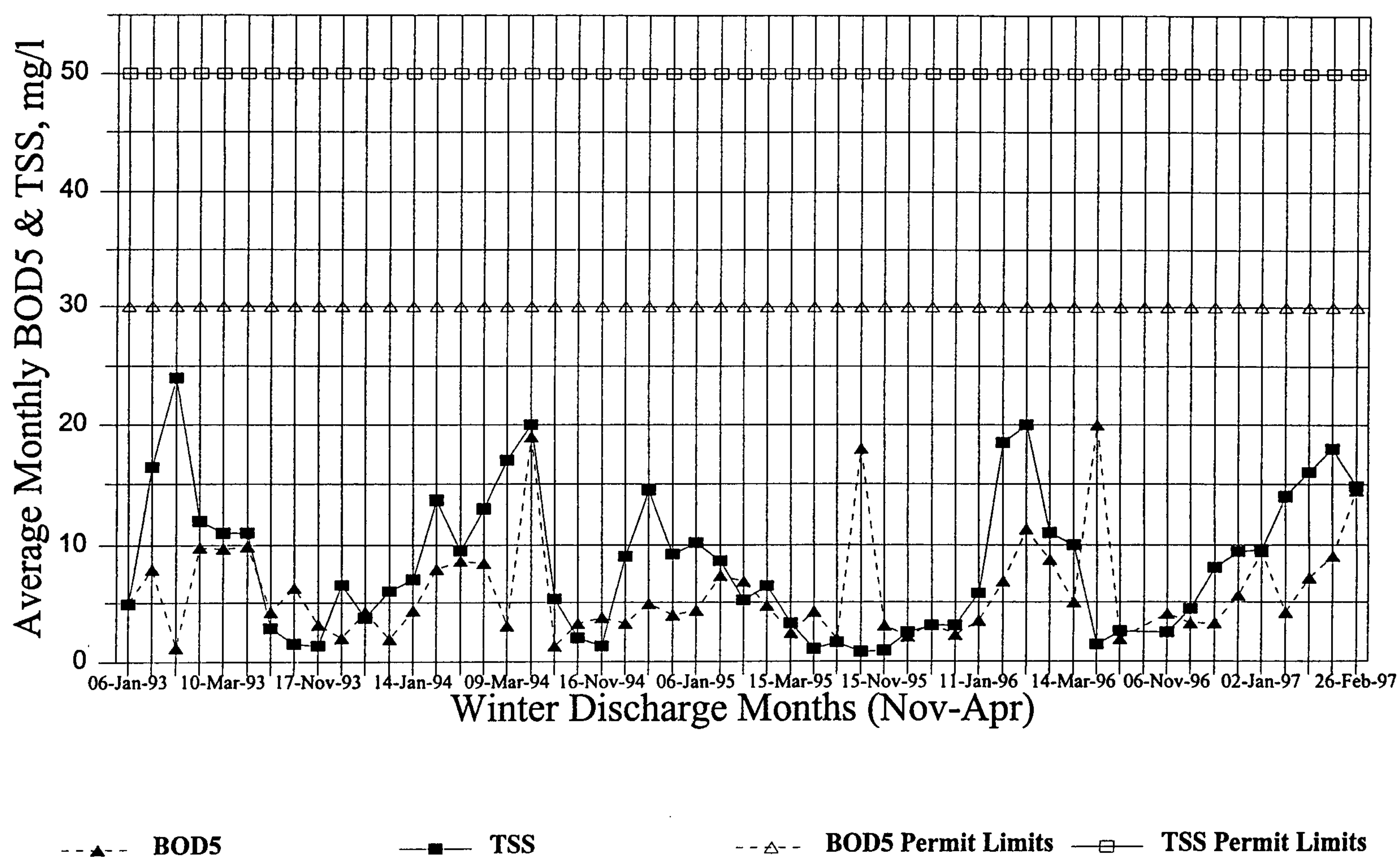


Figure 4-5: Effluent Percent Removal

City of Aumsville, Oregon

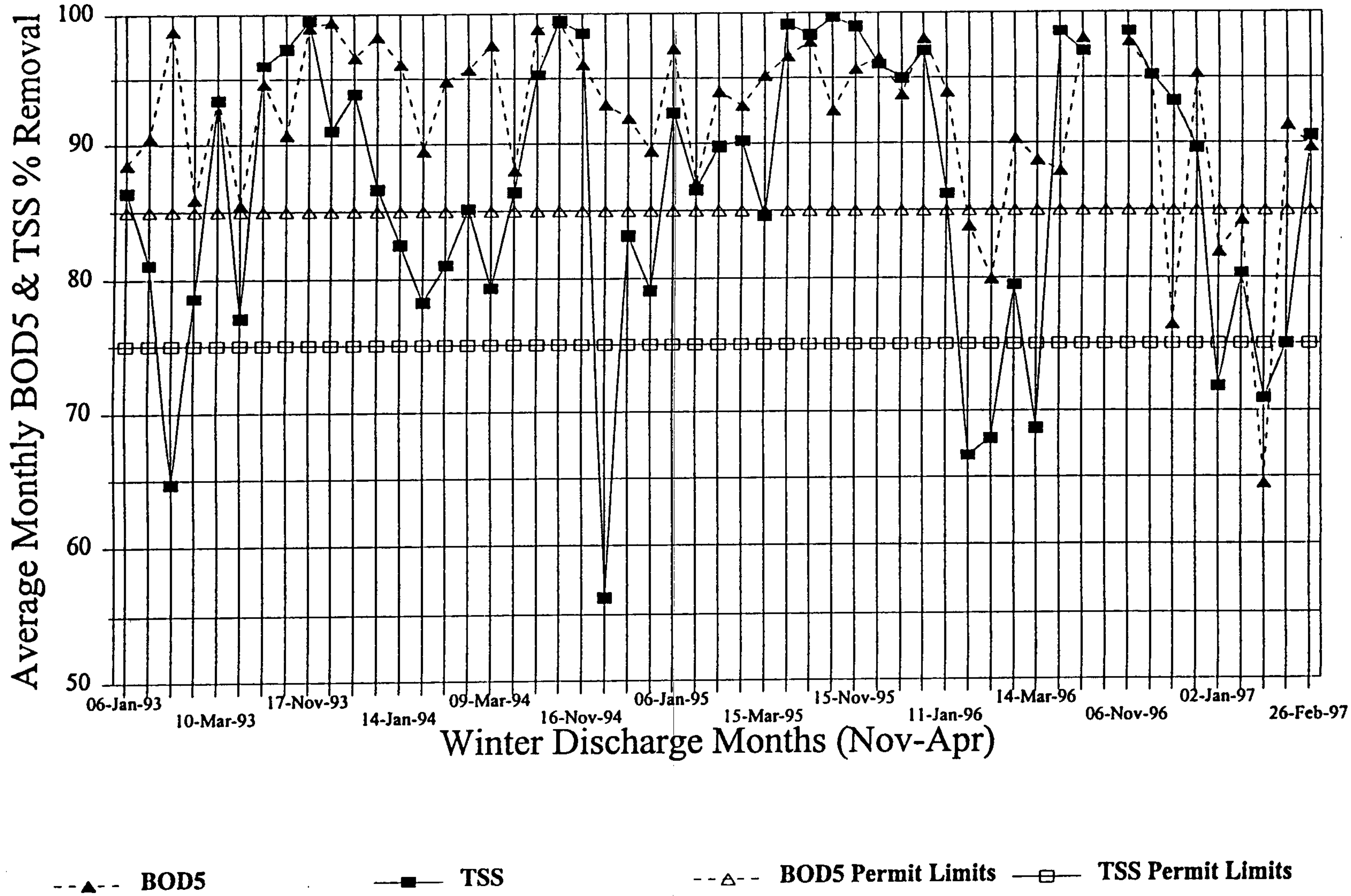


Figure 4-6A : BOD Effluent Mass Loads

City of Aumsville, Oregon

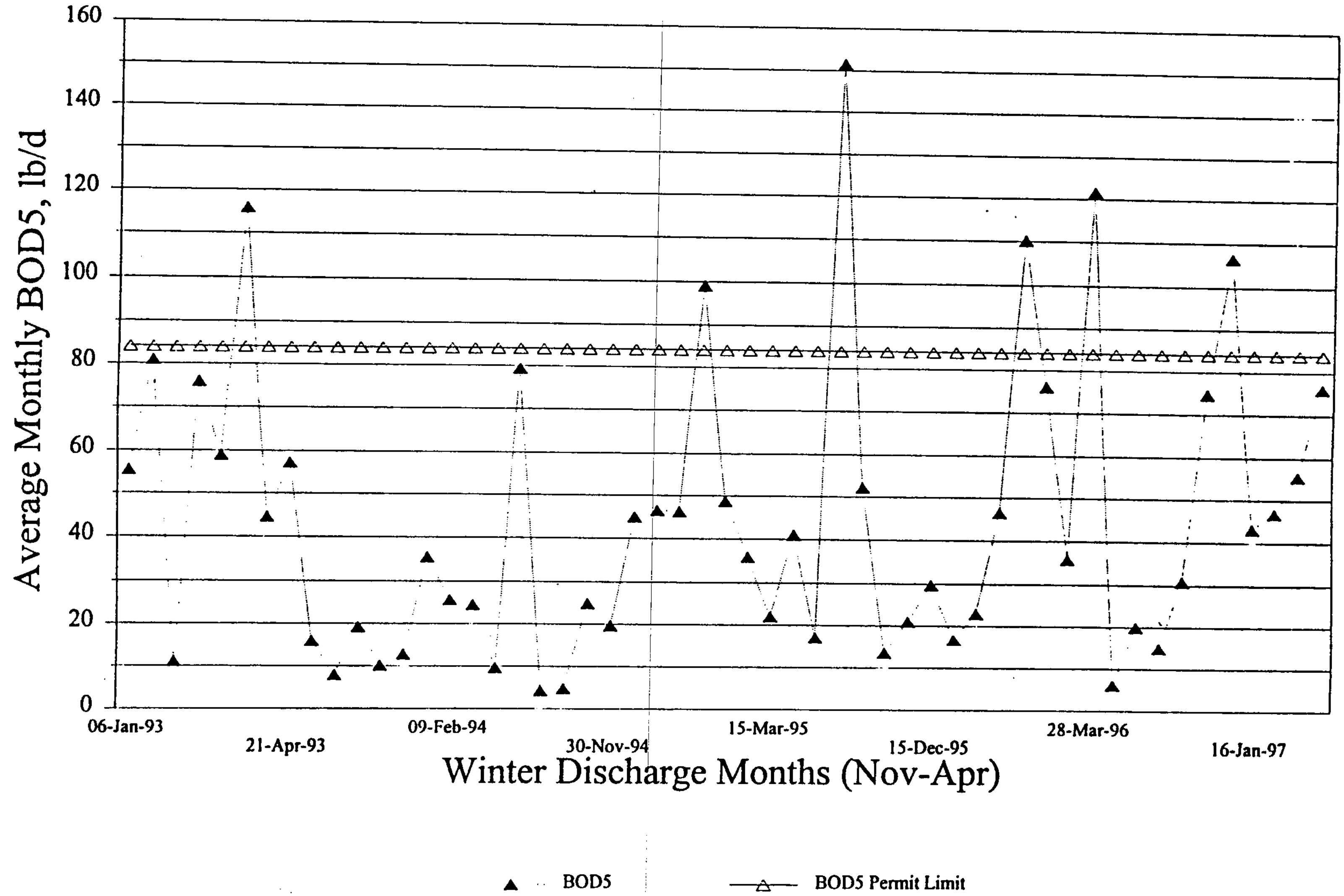


Figure 4-6B : TSS Effluent Mass Loads

City of Aumsville, Oregon

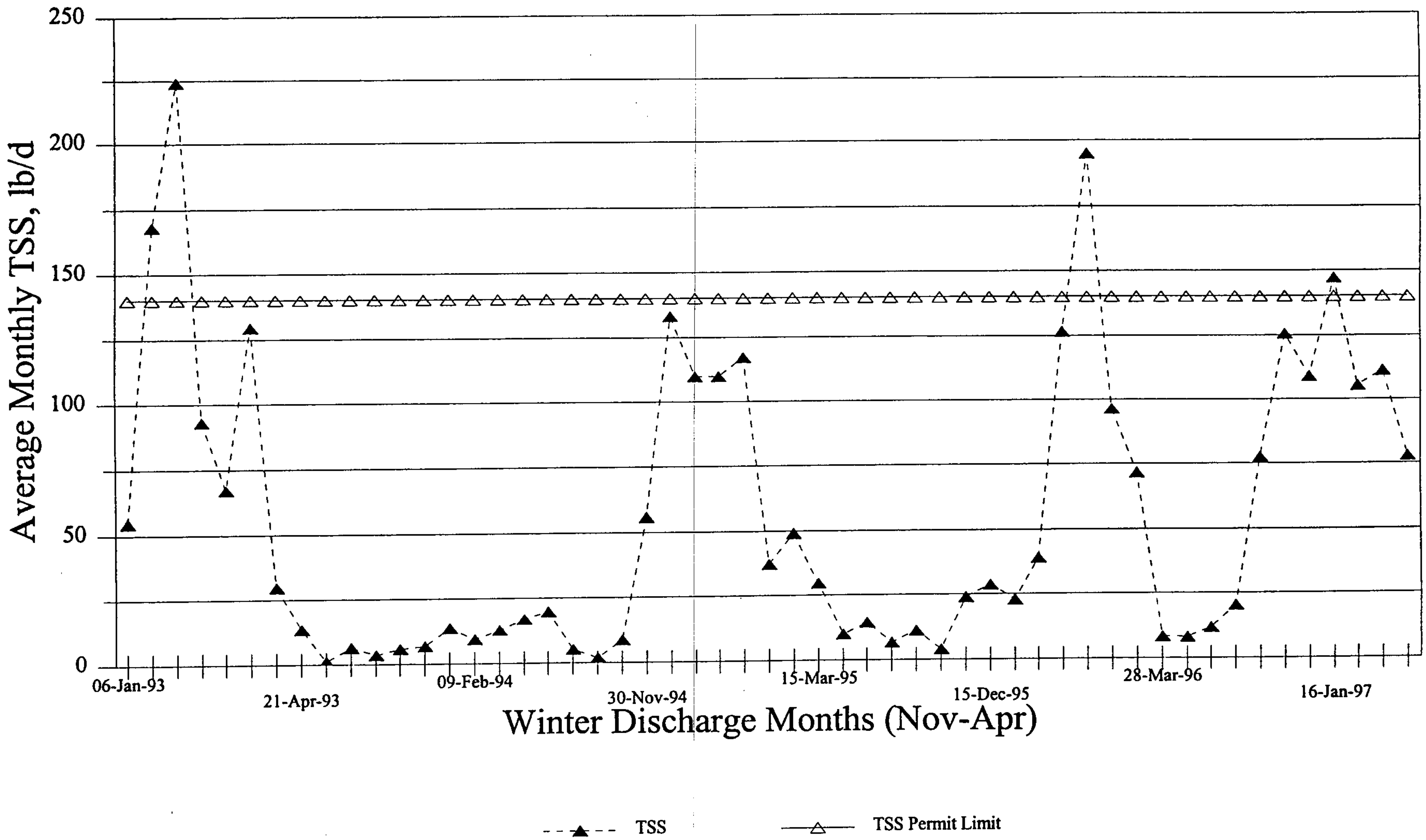


Table 4-4: Influent Wastewater Characteristics

Parameter	May - October		November - April	
	Average	Range	Average	Range
Flow, mgd	0.29	0.13 - 1.52	0.64	0.14 - 1.80
pH	7.5	6.4 - 8.8	7.0	6.1 - 8.3
BOD_5				
mg/l	188	74.2 - 316	108	13.6 - 486
lb/d	399	162.8 - 1127.2	457	88.9 - 1123
lb/cap/d	0.15		0.18	
TSS				
mg/l	155	25.3 - 315	88	20.6 - 354
lb/d	340	82.3 - 1158.1	407	96.1 - 982
lb/cap/d	0.13		0.16	

4.4 PROJECTED WASTEWATER CHARACTERISTICS

4.4.1 Wastewater Flows

Flow rates must be projected for various flow conditions in order to identify future system deficiencies and to size new components for the wastewater treatment system. A detailed evaluation of historical flow data has been previously discussed, including a summary of unit design factors. Projections for future flows are based on historical data, and an assumption that future I&I removal efforts will keep the current ratio of sewage to I&I constant despite future ageing of the collection system. Based on the I&I investigations conducted to date, it appears that infiltration is the predominant cause of the excessive flows. The majority of the identified inflow sources are on private sewers, and the City does not have direct control over the repairs. A summary of the estimated future flows is presented in Table 4-5.

4.4.2 Wastewater Composition

Fluctuation in organic mass loads will have a significant impact in selecting and sizing future treatment processes. The historical influent characteristics have been described in detail. The projected mass loads for the twenty (20) year planning process is summarized in Table 4-5. Note that only monthly and daily loadings are presented, since the City only reports the mass loads once every two (2) weeks.

Table 4-5: Future Influent Flows & Loadings

Parameter	1993-97	2007	2022
Population	2,585	3,620	5267
Wastewater Flow, mgd			
ADF	0.464	0.649	0.945
ADWF	0.288	0.405	0.587
AWWF	0.640	0.894	1.304
MMDWF	0.358	0.500	0.729
MMWWF	0.694	0.970	1.414
PDF	1.590	2.226	3.239
PIF	3.600	5.043	7.335
Wastewater Loads, lb/d			
BOD_5			
Avg. Day	429.2	601.0	874.5
Max. Month	968.0	1355.5	1972.3
Max. Day	1127.2	1578.5	2296.7
TSS			
Avg. Day	374.6	524.5	763.2
Max. Month	914.2	1280.2	1862.7
Max. Day	1158.1	1621.8	2359.6

Mass load projections were estimated using peaking factors calculated from the data in Table 4-3. Recommendations for improvements to the WWTP are based on the flow and mass load projections summarized in Table 4-5.

CHAPTER 5 - EXISTING WASTEWATER SYSTEM

5.1 HISTORY

The City of Aumsville converted from a sewer system consisting of septic tanks with drain fields to a municipal sanitary sewer system, including a gravity collection system and lagoon treatment plant, in the late 1960's. To accommodate the growth in the 1970's, the treatment plant was expanded by adding two new lagoons, which doubled the treatment capacity. The lagoons serve as storage basins between May 1 and October 31 of each year, as the City is not permitted to discharge wastewater into surface waters. Discharge of treated effluent to Beaver Creek is permitted from November 1 through April 30 of each year.

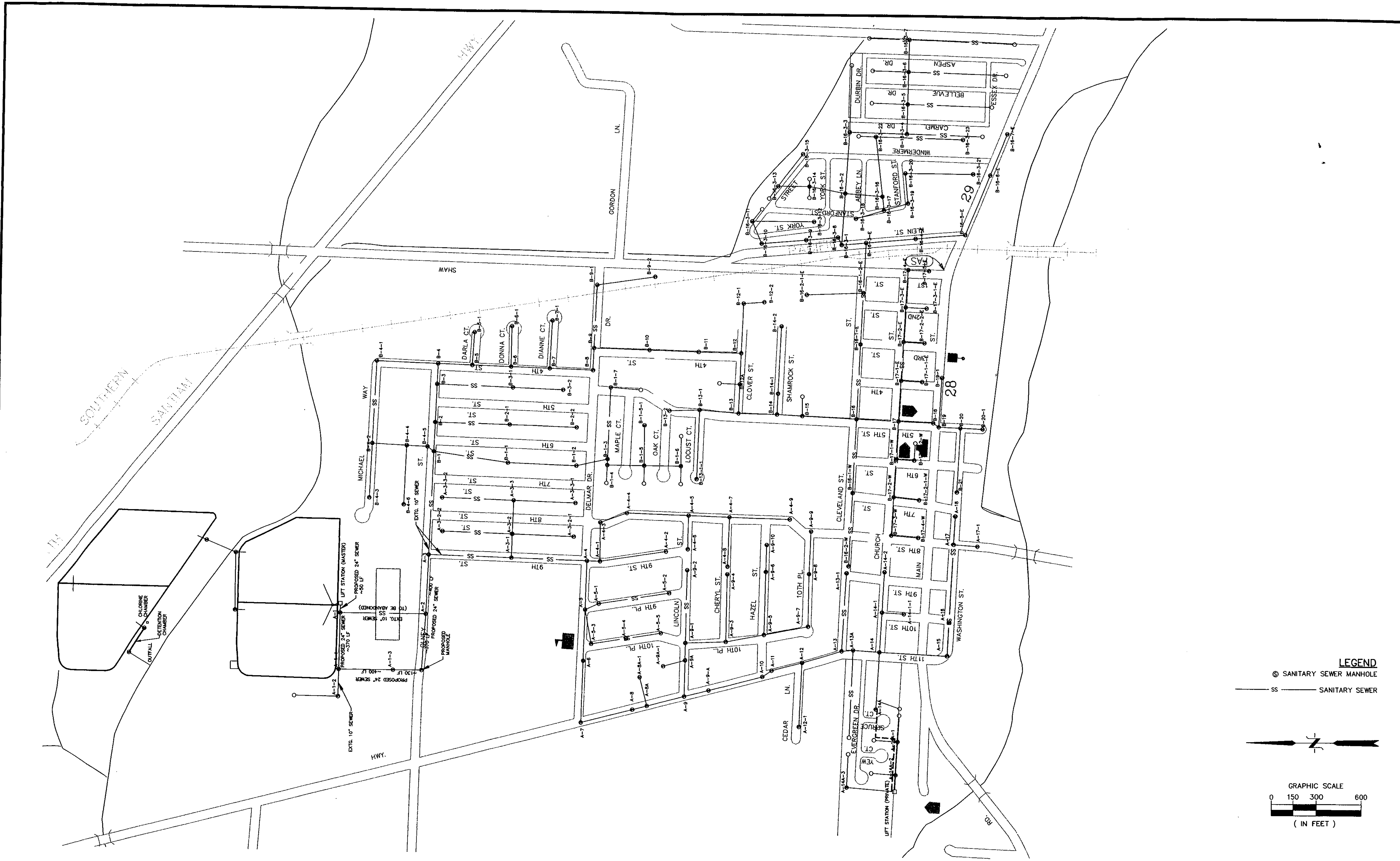
5.2 WASTEWATER COLLECTION SYSTEM

5.2.1 Gravity Collection System

The existing collection system consists of approximately 52,400 feet of buried piping, ranging in size from 6-to-10 inches in diameter. Most of the collection pipes, totaling 45,200 feet, are 8-inch diameter, and the 6-inch diameter pipes total approximately 5,000 feet. All are constructed of structurally sound rubber ring sealed concrete pipe. Approximately 2,200 feet of 10-inch diameter pipe comprises the main trunk line into the treatment plant. A map of the collection system is presented in Figure 5-1.

I&I appears to be a significant source of flow for the collection system. Numerous sections of the collection system surcharge during heavy rainfall events. The capacity of the 10-inch main trunk line, which passes under a manufacturing building, is estimated at 0.8 mgd under gravity-flow conditions. With a peak flow of 1.7 mgd, the sewer system periodically surcharges above street grade along Michael Way and bypasses sewage into the storm drainage system. However, the trunk line was evaluated to approximate its capacity under the surcharged condition for which the lowest manhole on Michael Way is near, but not quite at, overflow. This analysis resulted in a maximum capacity of 2.2 mgd, which indicates that the 10-inch trunk line should be capable of passing essentially all flowrates that the collection system is currently experiencing, as measured by the influent Parshall flume at the treatment plant. It appears the surcharges may be caused by the pump station being under sized. Reports indicate that surcharges occur only when one of the pumps in not operating.

The City has an on-going I&I reduction program, and to date the system has been partially smoke tested and actual sewer flow volumes have been field monitored at 19 key manholes throughout the system. In addition, many segments of the system have been TV inspected by Gelco Service, Inc.



REVISIONS:			
REV. NO.	DATE	DESCRIPTION	APPV'D

BALFOUR CONSULTING, INC.
MUNICIPAL ENGINEERING AND LAND DEVELOPMENT SERVICES
18605 WILLAMETTE DRIVE
WEST LINN, OR 97068
VOICE (503) 635 9293
FAX (503) 635 9294

DATE:	WASTEWATER FACILITIES PLAN	JOB NO: 140.01
DESIGN:	CITY OF AUMSVILLE, OREGON	
DRAWN: JDT	FIGURE 5–1	SHEET
APPV'D: ERC	COLLECTION SYSTEM MAP	

Z:\140\01\ACAD\FIG-5-1.DWG JDT 1/8/98

System repairs to date consist primarily of manhole sealing, and 30 manholes have been sealed over the last two years. The effectiveness of these repairs is difficult to measure with the limited quantifiable flow monitoring data available.

Available TV logs indicate that the collection system is generally in good shape, with only occasional leaking pipe sections and services. Leaks are generally difficult to see from a video examination unless the groundwater elevation is very high at the time of the taping. The large sewer flows that commonly occur during storm events usually drop quickly after the event, and a high percentage of these leaks become undetectable in as little as 36 hours after the storm event. The TV logs did, however, show several major pipe leaks which could be identified for repair.

Sewer flows were most recently measured during the night of January 29, 1992, by the City Engineer. This work was conducted between midnight and 4 a.m. so that the flows would represent system I&I due to limited domestic flow during this time period. In order to evaluate the available flow monitoring data, we prepared a detailed spread sheet. This spreadsheet is presented a Table 5-1, and includes the following information:

- Lineal feet of pipe above each key manhole for each pipe entering the manhole.
- Converts upstream pipes to inch-diameter-feet of pipe to facilitate analysis.
- Presents flow in gallons per day for each pipe entering a monitored manhole.
- Calculates upstream flow in gallons per foot of pipe and gallons per inch-diameter-foot.
- Provides data for pipe segments located between key manholes in order to isolate I&I within smaller sub-basins.
- Tabulates the number of feet of pipe leaking at five different rates for each major sub-basin.

The review of I&I data reveals that the City has a significant I&I problem. This is apparent by the significant increase in flow to the treatment facility during heavy rainfall events. The average annual flow of 179 gpcd is significantly higher than the average Oregon city, which ranges between 110 and 130 gallons per day. The PIF peaking factor of 7.8 is higher than the more typical peaking factors of 4 to 5.

The collection system spreadsheet analysis shows that 71 percent of the collection has I&I at or below 2 gallons per inch-diameter-foot per day, and 15 percent of the system has flows above 4 gallons per inch-diameter-foot per day. With additional selective flow monitoring, these problem

TABLE 5-1 : INFILTRATION AND INFLOW DATA ANALYSIS

City of Aumsville WWFP

Basin No.	Key Manhole #	Pipe into Manhole	Location	Pipe Size (in.)	Pump Sta. Impact	Upstream Pipe Length (feet)	Upstream Pipe Length (in.-dia.-ft.)	Measured Flow (gpd)	Upstream gal. per foot	Upstream gal./in. dia. ft.	Sub-basins Between Key Manholes: Upstream Pipe Length (feet)	Sub-basins Between Key Manholes: Upstream Pipe Length (in.-dia.-ft.)	Sub-basins Between Key Manholes: Measured Flow Difference (gpd)	Sub-basins Between Key Manholes: Upstream gal. per foot	Sub-basins Between Key Manholes: Upstream gal./in. dia. ft.	Lineal Footage in these I&I Rates: 0 to 1	1 to 2	2 to 3	3 to 4	4 to +
West Basins - A																				
	A-15		11th & Washington																	
		B		8	N	865	6,920	15,730	18.2	2.3								865		
		A		8		865	6,920	15,730	18.2	2.3										
	A-14		11th & Church																	
		B		8	N	645	5,160	13,460	20.9	2.6								645		
		C		8	N	1,315	10,520	43,950	33.4	4.2	450	3,600	28,220	62.7	7.8					450
		D		8	Y	1,910	15,280	115	0.1	0.0						1910				
		A				3,870	30,960	57,525	14.9	1.9										
	A-13		11th & Cleveland																	
		B		8	N	515	4,120	15,730	30.5	3.8									515	
		C		8	Y	4,705	37,640	97,460	20.7	2.6	835	6,680	39,935	47.8	6.0					835
		A				5,220	41,760	113,190	21.7	2.7										
	A-9		11th & Lincoln																	
		B		8	N	3,175	25,400	37,550	11.8	1.5							3175			
		C		8	Y	6,755	54,040	124,490	18.4	2.3	1,535	12,280	11,300	7.4	0.9	1535				
		A				9,930	79,440	162,040	16.3	2.0										
	A-4		9th & Delmar																	
		B		8	N	2,635	21,080	0	0.0	0.0						2635				
		C		10	Y	13,170	108,330	210,000	15.9	1.9	3,240	28,890	47,960	14.8	1.7		3240			
		A		10		15,805	129,410	210,000	13.3	1.6										
	A-3		9th & Olney																	
		B		10	N	33,380	273,970	569,330	17.1	2.1	690	6,900	25,408	36.8	3.7				690	
		C		10	Y	19,025	157,290	234,560	12.3	1.5	3,220	27,880	24,560	7.6	0.9	3220				
		A		10	Y	52,405	431,260	803,890	15.3	1.9										
East Basins - B																				
			Klein & Abbey																	
	B-16-3-E	B		8	N	10,275	82,200	720	0.1	0.0						10275				
		C		8	N	1,345	10,760	15,735	11.7	1.5							1345			
		A		8	N	11,620	92,960	16,455	1.4	0.2										
	B-16-1-2-E		2nd & Cleveland																	
		B		8	N	390	3,120	5,470	14.0	1.8							390			
		C		8	N	11,950	95,600	20,590	1.7	0.2	330	2,640	4,135	12.5	1.6		330			
		A		8	N	12,340	98,720	26,060	2.1	0.3										
	B-20		5th & Washington																	
		B		8	N	150	1,200	6,260	41.7	5.2										150
		C		8	N	430	3,440	464	1.1	0.1						430				
		A		8	N	580	4,640	6,724	11.6	1.4										

TABLE 5-1 : INFILTRATION AND INFLOW DATA ANALYSIS

City of Aumsville WWFP

Basin No.	Key Manhole #	Pipe Into Manhole	Location	Pipe Size (in.)	Pump Sta. Impact	Upstream Pipe Length (feet)	Upstream Pipe Length (in.-dia.-ft.)	Measured Flow (gpd)	Upstream gal. per foot	Upstream gal./in. dia. ft.	Sub-basins Between Key Manholes: Upstream Pipe Length (feet)	Sub-basins Between Key Manholes: Upstream Pipe Length (in.-dia.-ft.)	Sub-basins Between Key Manholes: Measured Flow Difference (gpd)	Sub-basins Between Key Manholes: Upstream gal. per foot	Sub-basins Between Key Manholes: Upstream gal./in. dia. ft.	Lineal Footage in these I&I Rates: 0 to 1	Lineal Footage in these I&I Rates: 1 to 2	Lineal Footage in these I&I Rates: 2 to 3	Lineal Footage in these I&I Rates: 3 to 4	Lineal Footage in these I&I Rates: 4 to +
	B-17-3-W		8th & Church																	
		B		8	N	170	1,360	0	0.0	0.0						170				
		C		8	N	445	3,560	28,640	64.4	8.0										445
		A		8	N	615	4,920	28,640	46.6	5.8										
	B-17-2-E		3th & Church																	
		B		8	N	865	6,920	54,180	62.6	7.8										865
		C		8	N	190	1,520	464	2.4	0.3						190				
		A		8	N	1,055	8,440	54,644	51.8	6.5										
	B-17		5th & Church																	
		B		8	N	1,780	14,240	65,140	36.6	4.6	725	5,800	10,496	14.5	1.8	725				
		C		8	N	955	7,640	15,730	16.5	2.1	375	3,000	9,006	24.0	3.0				375	
		D		8	N	1,350	10,800	37,550	27.8	3.5	735	5,880	8,910	12.1	1.5	735				
		A		8	N	4,085	32,680	118,420	29.0	3.6										
	B-16		5th & Cleveland																	
		B		8	N	13,120	104,960	50,690	3.9	0.5	780	6,240	24,630	31.6	3.9				780	
		C		8	N	4,370	34,960	124,190	28.4	3.6	285	2,280	5,770	20.2	2.5			285		
		D		8	N	975	7,800	31,520	32.3	4.0										975
		A		8	N	18,465	147,720	206,400	11.2	1.4									-	
	B-13		5th & Clover																	
		B		8	N	19,985	159,880	278,800	14.0	1.7	1,520	12,160	72,400	47.6	6.0					1520
		C		8	N	925	7,400	65,140	70.4	8.8										925
		A		8	N	20,910	167,280	343,940	16.4	2.1										
	B-12		4th & Clover																	
		B		8	N	470	3,760	2,440	5.2	0.6						470				
		C		8	N	21,455	171,640	204,200	9.5	1.2	545	4,360	(139,740)	-256.4	-32.1	545				
		A		10	N	21,925	175,400	206,640	9.4	1.2										
	B-9		4th & Delmar																	
		B		8	N	820	6,560	- ? -	0.0	0.0						820				
		C		10	N	22,925	185,400	214,560	9.4	1.2	1,000	10,000	7,920	7.9	0.8	1000				
		A		10	N	23,745	191,960	214,560	9.0	1.1										
	B-4		4th & Olney																	
		B		10	N	1,325	13,250	9,243	7.0	0.7						1325				
		C		10	N	25,745	210,350	514,780	20.0	2.4	2,000	18,390	300,220	150.1	16.3					2000
		A		10	N	27,070	223,600	524,023	19.4	2.3										
	B-1		6th & Olney																	
		B		8	N	595	4,760	3,032	5.1	0.6						595				
		C		10	N	29,355	240,390	490,000	16.7	2.0	2,285	16,790	(34,023)	-14.9	-2.0	2285				
		D		8	N	2,740	21,920	50,890	18.6	2.3								2740		
		A		8	N	32,690	267,070	543,922	16.6	2.0										
															Sub Total	28865	8480	4535	2360	8165
															Total					52405

areas could be isolated to specific pipe reaches between two manholes. Given the current data, all of the pipe within a sub-basin appears to contribute to the excessive I&I where in reality the problem may be limited to only one or two joints of bad pipe or leaking house services. Additional flow measurements combined with focused TV inspections conducted during high plant flow can pinpoint the location of significant leaks. This approach allows a spot repair to be made, which can result in very cost effective I&I removal.

5.2.2 Pump Stations

Currently there are no City-owned pump stations on the collection system. It is possible, however, that a pump station will be required to service certain areas within the UGB. The City should exercise care in controlling the design and construction of any pump stations that are associated with development projects. The units must be reliable, easy to maintain and not overloaded by subsequent expansions of the collection system.

5.3 WASTEWATER TREATMENT PLANT

5.3.1 Pretreatment

No pretreatment is conducted at the WWTP. Ektron, a metals finishing facility, is the only facility discharging an industrial wastewater in Aumsville. Ektron reportedly pretreats its wastewater prior to discharging to the collection system.

5.3.2 Influent Lift Station

Wastewater from the collection system is discharged into the WWTP through an influent lift station. The influent pump station is a duplex submersible station. The forcemain is comprised of two components: 6-inch ductile iron pipe and 8-inch asbestos-concrete pipe. Each pump forces wastewater through 26 feet of 6-inch ductile iron pipe, at which point they merge in a vault and connect to 25 feet of the 8-inch asbestos-concrete pipe. This 8-inch pipe discharges into a chamber directly upstream from the influent Parshall flume. The total static head overcome by the pump station is approximately 23 feet. Design and operating criteria for the influent pump station are summarized in Table 5-2.

The pump station has performed reasonably well in recent times, requiring only routine maintenance such as wet well cleaning and sealing to control grease buildup. The pump station has, however, reached its capacity. Both pumps are needed to handle major peak flows during rainfall events. If either or both pumps fail, surcharging and sewage overflows will occur on Michael Way. There is no direct bypass line from the pump station to the creek. The wet well appears to be in generally good condition, and could possibly be incorporated into future expansions. The pump station building which houses the generator, sampler and electrical equipment is cramped with little prospect to accommodate future expansion.

Table 5-2: WWTP Influent Pump Station Original Design Data

Influent Pump Station	Type	Duplex pump station with self-priming, non-clog, submersible, constant speed pumps.
	Capacity	630 gpm @ 27 feet TDH each
	Pump Horsepower, hp	7.5 each
	Level Control Type	Mercury level sensors
	Overflow Elevation	N/A
	Overflow Discharge	Manhole at west end of Michael Way
	Auxiliary Power Type	15 kW propane gas-powered generator @ WWTP
	Transfer Switch	Automatic
	Alarm Telemetry Type	Radio telemetry to the shop computer
	EPA Reliability Class	II
Forcemain	Type and Length	26 ft - 6" Ductile Iron Pipe (each pump), 25 ft -8" Asbestos-Concrete (common to both pumps). Continually ascending.
	Air Release Valve	None required.
	Discharge Location	Vertical chamber directly upstream from influent Parshall flume
	Sulfide Control System	None.

5.3.3 Influent Metering /Sampling

Metering

Wastewater flows from the influent pump station into the WWTP through a 6-inch Parshall flume. The pump station forcemain discharges at the base of a vertical chamber at the upstream end of the flume. The wastewater flows upward, then transitions to flow through the flume horizontally. This device measures all flow routed into the plant.

An ultrasonic transducer is used to measure the flow in the flume. According to the US Bureau of Reclamation Water Measurement Manual, a 6-inch parshall flume has a measurement range of 22.4 gpm at 0.10 feet of head to 1,750 gpm at 1.5 feet. The depths are for a free flow condition, measured at the standard location (i.e., H_a). Based on the WWTP Daily Monitoring Reports, influent flowrates have periodically exceeded this maximum. These exceedances indicate that the measurement capacity of the Parshall flume is being overwhelmed, likely due to a back-up condition on the divergent section of the flume.

Wastewater discharging from the flume drops vertically out of the end of the structure, then transitions to flow into both lagoons. The outlet structure does not promote laminar flow, and incurs significant head losses with respect to the available elevation between the flume and normal lagoon water levels. As long as pond levels do not rise beyond an elevation of approximately 348.50 (maximum WSE), flow will likely remain in a free flow condition and the accuracy may not be compromised. It should be noted that lagoon levels have frequently exceeded the maximum WSE elevation during recent winters. It was not possible to determine if any material that could cause additional energy losses or backups is present inside the discharge piping or at the outlet to either lagoon.

A water balance of the treatment plant lagoons is summarized in Section 6.4. The discrepancies in this evaluation provide further evidence of a flow measurement problem.

Care must be taken in interpreting or drawing conclusions from the peak flow rate evaluations that are based on the available data. A comprehensive analysis of the flume should be completed before commencing treatment facility designs in order to more accurately determine the nature of disparities between actual and measured flowrates.

Sampling

An American Sigma automatic sampler is used to collect flow-proportional composite and grab samples required in the City's NPDES discharge permit. For composite samples, the automatic sampler is currently programmed to collect 200 milliliters of sample for each 70,000 gallons of influent flow.

5.3.4 Primary Treatment

The primary treatment of wastewater is accomplished within two facultative lagoons. These types of lagoons operate with separate aerobic and anaerobic zones. The aerobic environment occurs in approximately the upper 80% of the lagoon, but must be adjusted according to the sludge level in the pond. All of the zones are measured from above the sludge level. The aerobic microorganisms use the soluble BOD and dissolved oxygen in their metabolic process. Photosynthesis and surface re-aeration provide oxygen for aerobic stabilization. The anaerobic zone, located in the lower depths of the lagoon, contains only anaerobic bacteria that use the settled wastes (BOD) and chemically bound oxygen for metabolism. Within this anaerobic zone a complex group of bacteria work together to further stabilize the wastes into stable compounds much like an anaerobic digester. Anaerobic stabilization occurs in two steps: (1) acid bacteria break down the complex organics by feeding on the soluble matter, with a conversion to organic acids; and (2) methane bacteria feed on the organic acids, with a conversion to carbon dioxide, ammonia, hydrogen sulfide, and methane gas. The liquid and gas byproducts of the anaerobic decomposition supply additional food to the aerobic bacteria. In turn, the aerobic bacteria provide byproducts that are used by the anaerobic bacteria.

Table 5-3 lists the critical dimensions for each lagoon cell at the WWTP. The two primary cells were originally designed for primary and secondary treatment. Modifications completed in 1978 added a new secondary treatment cell and a new tertiary treatment cell, and converted the existing secondary treatment cell into a primary treatment cell. Influent flow splits downstream of the parshall flume into separate 8-inch pipelines routed into each primary cell. The lagoon inlets are located towards the center of each cell. The cells are interconnected with an 10-inch diameter pipeline and can be operated either in series or parallel. Both cells have a design depth of 3.5 to 5 feet, with a 10 foot top width, with 2 feet of freeboard on the dikes. The dikes were constructed at 3:1 horizontal to vertical sideslopes, and the lagoons are clay lined. A limited layer of small diameter rip-rap remains on the dikes, with some evidence of bank erosion.

5.3.5 Secondary/Tertiary Treatment

Secondary and tertiary treatment of the wastewater is accomplished in two facultative lagoons located north of the primary cells. The primary and secondary/tertiary cells are separated by Beaver Creek, which flows southeast to northwest across the site. Wastewater is transferred between the primary and secondary cells via a 10-inch diameter cast iron pipeline routed beneath the creek. The creek crossing is accomplished with an inverted siphon. The secondary and tertiary lagoons are connected through a 10-inch diameter pipeline, and each cell operates in series. The dikes for both lagoons consist of a 10 foot top width, and 3:1 side slopes. Both cells have a design operating depth of 3.5 feet. The secondary/tertiary lagoons have a lower maximum water surface elevation than the primary cells, and flow from the primary lagoons is controlled by a manually-operated gate.

5.3.6 Lagoon Process Summary

Although there are a number of empirical design procedures for sizing and evaluating the performance of facultative lagoons, most facility designs follow well-documented standard guidelines developed by the EPA and the DEQ. The most typical design parameters are detention time (days) and BOD_5 loading (pounds/acre/day). The recommended detention time typically ranges from 25 to 180 days, with the EPA recommending a detention time of 120 to 180 days in the primary cell(s). The DEQ requires a minimum detention time of 60 days for the entire lagoon system. The recommended BOD_5 loading rates range from 20 to 180 lb/acre/day. The DEQ recommends a maximum design loading rate of 50 lb/acre/day for the primary cells, and an overall loading rate of 35 lb/acre/day.

Table 5-3: Existing Lagoon Dimensions

	Cell 1 (Primary)	Cell 2 (Primary)	Cell 3 (Secondary)	Cell 4 (Tertiary)
Average Area, acres	7.63	6.66	7.80	6.33
Invert WSE, feet msl	342.50	342.50	338.00	338.00
Minimum WSE, feet msl	345.00	345.00	340.50	340.50
Maximum WSE, feet msl	348.50	348.50	344.00	344.00
Top of Dike, feet msl	350.50	350.50	347.00	347.00
Available Volume, min. to max WSE, MG	8.701	7.595	8.895	7.219
Available Volume, I.E. to dike, MG	20.0	18.0	23.4	19.0

The current operating conditions are summarized in Table 5-4.

Table 5-4: Existing WWTP - Unit Process Summary

Influent Metering

Influent Flow Meter Type Size Capacity	 Parshall Flume 6-inch 1,750 gpm (2.52 mgd)

Wastewater Lagoons

Parameter	Cell 1 (Primary)	Cell 2 (Primary)	Cell 3 (Secondary)	Cell 4 (Tertiary)
Top of Dike, feet msl	350.50	350.50	347.00	347.00
Maximum WSE, feet msl	348.50	348.50	344.00	344.00
Minimum WSE, feet msl	345.00	345.00	340.50	340.50
Invert Elevation, feet msl	342.50	342.50	338.00	338.00
Operating Depth, feet	3.50	3.50	3.50	3.50
Average Surface Area, acres	7.63	6.66	7.80	6.33
Treatment Volume (between min. and max. WSE), MG	8.70	7.60	8.90	7.22
Storage Volume (between IE and top of dike), MG	20.0	18.0	23.4	19.0

Lagoon Performance Summary

Parameter	Primary Lagoons	Entire Lagoon System
BOD_5 loading (lb/ac/d) Recommended (DEQ) Current Average Annual Current Maximum Monthly	 25-50 30 68	 35 15 34
Hydraulic Detention Time (days) Recommended MMWWF MMDWF	 120-180 (EPA) 22 43	 60 (DEQ) 45 86

Table 5-4: Existing WWTP - Unit Process Summary	
Disinfection System	
Disinfectant	Chlorine Gas - (2) 150 Pound Cylinders
Chlorination Mechanism	Regal Chlorinator
Contact Chamber Type Total Length Diameter Volume Detention @ MMWWF	 Concrete Pipe 180 feet 5 feet 26,400 gallons 52 minutes
Mixer (Not in Service) Type Horsepower Speed	 Cleveland Model F 1.5 350 rpm
Effluent Disposal	
Summer Discharge (May 1 to October 31)	N/A
Winter Discharge (November 1 to April 30)	Beaver Creek
Effluent Flow Meter	Rectangular Weir with Stevens Model 61R
Creek Outfall	150 feet - 10" diameter DI

Over the past few years the ability of the lagoons to provide high quality effluent appears to be generally decreasing. Duckweed which once covered the primary ponds periodically during the summer is now gone and algae concentrations in the water appear to be thickening. These are indications that these two lagoons may be approaching their nutrient load capacity. It is possible that the long storage times during the summer are adversely impacting the primary cells, due to organic overload and little flow through the cells.

The plant has not had sufficient capacity to store all of the flow collected over the six-month dry weather period in the recent past. For two of the last three years, the lagoons have become so full that the integrity of the dikes was threatened. To alleviate pressure on the dikes, effluent was discharged to the river (directly or via the storm drain system) outside of the permitted discharge period.

The City received its third notice of noncompliance (NONs) by the DEQ on March 31, 1999 for violating its permit between June 1998 and January 1999. The June 1998 violations were for discharging out of the permitted season, and is a Class I violation. In December 1998 and January 1999, the DEQ cited three Class II and three Class III violations for exceeding BOD and TSS limits. These most recent violations document that the facility is not just experiencing capacity problems, but also currently has treatment problems that must be addressed in the very near future.

5.3.7 Disinfection System

Treated wastewater flows from the tertiary lagoon outlet structure, located in the southwest corner of the lagoon, through a 12-inch diameter cast iron pipe into a mixing vault, then into a chlorine contact chamber. The mixing vault is a 6-foot by 5-foot by 8-foot high concrete structure, with a propeller flash mixer. The contact chamber is discharged into a 60-inch diameter, 180 feet long concrete pipe. Chlorine gas is used to disinfect the wastewater effluent. Chlorine injectors are connected to 150-pound chlorine bottles stored in a building located adjacent to the contact chamber. The chlorine is injected as the water enters the mixer vault. The contact chamber provides 52 minutes of contact time at the current MMWWF. The outfall is an open pipe on the bank above the normal stream water surface.

5.3.8 Effluent Sampling/Metering

The disinfected effluent flows from the chlorine contact chamber over a flow measuring weir into a 12-inch outfall pipe and is discharged into Beaver Creek. Total flow is continuously recorded. An American Sigma automatic sampler is used to collect composite and grab samples required in the City's NPDES discharge permit.

5.3.9 Biosolids

Biosolids, also known as sludge, accumulate in each of the four (4) lagoons and stabilize through anaerobic treatment. This is the standard means of sludge storage in facultative lagoon wastewater treatment plants. Generally, sludge accumulates at a rate of 0.5 to 2.0 inches per year, and the sludge layer will vary in depth from the lagoon inlet to the perimeter of the cell. Sludge should be removed from a lagoon once every fifteen to thirty years.

To date, sludge has not been removed from any of the lagoons at the WWTP. The volume of sludge in the lagoons has reportedly not been measured. Sludge accumulation in the primary cells will reduce the lagoon volume available for storage and treatment. The accumulation of biosolids within the primary lagoons will need to be addressed in the near future.

5.4 EFFLUENT DISPOSAL

5.4.1 Winter Discharge

Effluent is discharged into Beaver Creek between November 1 and April 30. The City's current permit limits the effluent concentrations of BOD and TSS to less than 30 mg/l and 50 mg/l, respectively. While the discharge currently meets this criteria, an expansion of the treatment facilities could lead to more stringent permit limitations.

5.4.2 Summer Discharge

No discharge is permitted from May 1 to October 31. During this time, the lagoons serve as detention basins.

5.5 FINDINGS AND CONCLUSIONS

The overall performance of the WWTP is summarized in Table 5-4. Based on a review of the existing unit processes, a comparison to typical design factors, and an evaluation of the overall system, the following list of system deficiencies has been developed:

Collection System

- The existing 10-inch sewer main trunk line into the WWTP surcharges during large storm events, resulting in overflows in the vicinity of Michael Way. This sewer line has a calculated maximum capacity of 2.16 MGD, compared to a projected design flow rate (20-year PIF) of 6.48 MGD.

- Although video tapes indicate the collection system is in relatively good shape with few leaks, available flow monitoring data suggests a more significant I&I problem.

WWTP - Influent Pump Station

- The influent pump station has a capacity of 0.9 MGD with one pump operating, and 1.8 MGD with two pumps operating. The projected design flow (20-year PIF) is 6.48 MGD.

- The existing wetwell has insufficient capacity to handle future design flows.

WWTP - Influent Sampling/Metering

- The existing influent flow measuring system is over capacity during high flow events.

- Current flow measurement data may not be accurate due to surcharged conditions on the outlet side of the parshall flume during high-flow events.

WWTP - Lagoons

- The plant does not have sufficient hydraulic capacity to meet the projected growth within the 20 year planning period, and has been unable to store all of the flow collected over the six-month dry weather period for two of the last three years.

- The City's future mass load limitations are anticipated to remain the same as those set forth in their current discharge permit. Future effluent quality from the WWTP must improve significantly as influent flows increase, and the lagoons cannot provide sufficient treatment efficiency to satisfy these mass load limits.

- The piping between the primary and secondary /tertiary lagoons is unable to handle the peak flows into the plant, and additional capacity is needed.

WWTP - Disinfection

- The volume of the chlorine contact chamber is not sufficient to provide the required detention time for the current MMWWF and future projected flows.

- Future permit conditions will likely not allow chlorine discharges.

WWTP - Effluent Metering

- The existing effluent metering weir does not provide high flow measurement accuracy, and over time can lead to increased maintenance (i.e., solids retention, biological growth).

WWTP - Effluent Disposal (November - April)

- The existing outfall is an open pipe that discharges to the surface of Beaver Creek. The receiving stream is effluent dominated and may not provide significant dilution.

WWTP - Effluent Disposal (May - October)

- There is currently no provision for dry weather effluent disposal (i.e., reclaimed water irrigation). The treatment facility has not had sufficient capacity to store all of the flow collected over the six-month dry weather period in the recent past.

WWTP - Biosolids Management

- Biosolids have not been removed from the primary lagoons since their last upgrade in the 1970's. Typically, sludge must be removed from primary lagoons once every 15 to 30 years.

- Construction of plant upgrades will likely require the removal of biosolids.

CHAPTER 6 - PLANNING BASIS

6.1 DESIGN BASIS

The basis for the design of future improvements to the wastewater facilities includes both current and future regulatory requirements, as well as the technical design criteria. Each of these subjects is discussed in detail below.

6.1.1 Regulatory Requirements

The City operates its wastewater collection and treatment system under DEQ-issued NPDES Wastewater Discharge Permit #100881. A copy of the permit is included in Appendix B. The permit expired on March 31, 1997. DEQ personnel indicate that, due to staffing limitations, the agency is unsure of its schedule for permit renewal. Communities across the state are to submit applications for a new permit but continue to operate their facilities under their expired permits. For Aumsville, the permit renewal will likely be tied to planned improvements at the wastewater treatment plant.

Current Regulatory Requirements

The NPDES permit is divided into four sections, as follows:

- Schedule A - Waste Discharge Limitations
- Schedule B- Minimum Monitoring and Reporting Requirements
- Schedule C - Compliance Conditions and Schedules
- Schedule D - Special Conditions

Pertinent requirements are listed in Table 6-1. Table 6-2 summarizes the monitoring requirements.

Table 6-1: Effluent Discharge Limitations
November 1 to April 31 (Beaver Creek)

Parameter	NPDES Permit Limits
BOD_5	Average Concentration 30 mg/l monthly maximum 45 mg/l weekly maximum Average Mass Load Limits 84 lb/day monthly maximum 126 lb/day weekly maximum Maximum Mass Load Limits 168 lbs daily maximum Minimum % Removal 85% of monthly average concentration
TSS	Average Concentration 50 mg/l monthly maximum 80 mg/l weekly maximum Average Mass Load Limits 140 lb/day monthly maximum 224 lb/day weekly maximum Maximum Mass Load Limits 280 lbs daily maximum Minimum % Removal 75% of monthly average concentration
Fecal Coliform	Counts per 100 ml 200 monthly maximum 400 weekly maximum
pH	6.0 - 9.0
Discharge Flow Rate	0.633 mgd maximum

Table 6-2: NPDES Monitoring Requirements

Parameter	Influent/Effluent	Frequency
Flow Rate	Both	Daily
Flow Meter Calibration	Both	Annually
BOD_5	Both	2 per month
BOD5 % Removed	Effluent	1 per month
TSS	Both	2 per month
TSS % Removed	Effluent	1 per month
Fecal Coliform	Effluent	2 per month
pH	Both	3 per week
Quantity of Chlorine	Effluent	Daily
Chlorine Residual	Effluent	Daily

Future Regulatory Requirements

BCI personnel met with the DEQ to discuss expected regulatory requirements for future permits. The DEQ indicated that it will focus future regulatory requirements on the overall health and survival of aquatic organisms, in keeping with its statewide initiative. Based on these discussion, the anticipated changes to the City of Aumsville's permit include the following:

- Removing chlorine from surface water discharges, either through dechlorination or alternative means for disinfection.
- Changing the indicator organism from fecal coliform to e-coli.

With respect to mass loads, the DEQ generally does not allow increases. Oregon Administrative Rule (OAR) 340-41-026 (2) requires that, unless otherwise approved by the Environmental Quality Commission (EQC), growth and development shall be accommodated within existing permitted loads by the application of increased treatment and control efficiency. The EQC may, after full satisfaction of the intergovernmental coordination and public participation provisions of the confirming planning process, and with full considerations of OAR 340-41-026 (2, 3, 5), allow an increase in a discharger's mass load limits if the following is determined:

- No other reasonable alternatives exist except to lower water quality.

- The mass load increase is necessary and justifiable for economic or social development benefits and outweighs the environmental costs of lowered water quality.

- All water quality standards will be met and beneficial uses protected.

Given the current DEQ regulations, it is not anticipated that the City's mass loads will increase in the foreseeable future. As a result, the WWTP removal efficiency must improve in order to accommodate future increased flows. A summary of the effluent quality necessary to meet both the current discharge limits as well as the projected future flows is presented in Table 6-3.

Table 6-3: Future Effluent Quality Requirements (Beaver Creek)

		Monthly Average Effluent Concentrations	
Year	Flow	BOD_5, mg/l	TSS, mg/l
2007	AWWF	11.3	18.7
	MMWWF	10.3	17.3
2022	AWWF	7.7	12.9
	MMWWF	7.1	11.9

6.1.2 Design Criteria

The life of a wastewater system is impacted by several factors. The review and analysis completed to date determines the design considerations. The design criteria for expansion and/or improvements to the collection system and WWTP must take into account existing and projected loadings and flows, as discussed in earlier chapters, and regulatory requirements as presented herein. In developing alternate plans, the following general design considerations must be evaluated:

- **Design Period**: A collection and treatment system is designed to meet the immediate needs of the community, plus a reserve capacity for growth. 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 selected treatment alternative will be based on the effluent quality criteria shown in Table 6-3.

The design period will be twenty years for pump stations and the treatment facility and forty years for the collection system components.

- **Wastewater Treatment Facility**: The technology of wastewater treatment has changed over time, as have the regulatory requirements. The primary consideration will be the degree of treatment required to meet the discharge requirements and 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. Where possible, the operators must have the ability to alter plant flows around the major process units without significantly degrading effluent quality. If possible, the design should provide redundant units and multiple interconnections between units. 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 also allow the construction and connection of new process units as needed.

- **Reliability:** Reliability of treatment processes depends on the proper application of unit loading factors and the 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 should also be considered. Processes that require a high degree of manual labor, special schooling, and unique instrumentation should be avoided in most cases. Redundancy is a key factor in reliability. Back-up units for critical pieces of equipment are essential.

- **Operation:** Operation of wastewater systems entails considerable responsibility and cost, and provides public health benefits. These reasons require that personnel assigned to operate and maintain a treatment facility be appropriately trained and DEQ certified. The more sophisticated the process or equipment, the greater the level of expertise required. Qualified individuals are usually available in metropolitan areas, as is financial support for continued training. However, small communities can have more difficulty in securing and retaining the personnel required and budgeting the money required to pay them. Consequently, the selection of a treatment process or piece of equipment should reflect the regional and local training level of operations and maintenance personnel.

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

- **Other:** Consideration of site location, operational tasks, public perception, aesthetics, health and safety concerns, noise, odors, access to equipment, and hazards all have to be analyzed when assessing treatment alternatives.

6.2 BASIS OF COST ESTIMATE

In developing cost estimates, four components were included: construction; contingencies; and engineering, legal, and administrative services. The cost estimates presented herein are preliminary, and are based on the level and detail of planning presented in this plan. The estimates should be continually reviewed as the project(s) move through the design phase, and should be updated if necessary.

The economic evaluation of alternatives is based on a comparison of the present worth cost for each alternative. The present worth cost is affected by the O & M costs, potential for alternative technologies, land acquisition, and salvage value. The calculations are based on a discount rate of 8 percent and a 20-year life.

6.2.1 Construction Costs

Construction cost estimates are based on recent construction projects of similar nature, actual equipment and material costs, and published cost guides. For improvements to the existing system, the as-built drawings were used to estimate actual quantities, elevations and other pertinent design related issues. For each alternative, preliminary layouts were developed to establish the construction quantities.

6.2.2 Contingencies

At the current level of project planning, cost contingencies must be added to ensure that an adequate budget is provided. Such contingencies include potential variable costs that could result through deviations in the final quantities, market bidding conditions, potential adverse construction conditions, and any required special investigations and/or studies. Once the design stage of the project begins, the construction cost estimates will be refined based on the actual sizes of unit processes, pipelines and equipment. The cost estimates presented in this report include a contingency equal to 15 percent of estimated construction cost.

6.2.3 Engineering

Engineering services include pre-design, surveying, geotechnical investigations, special investigations, design, construction drawings, contract documents, bidding services, construction management, project inspection, construction staking, system start-up services, performance certification, and preparation of an operations and maintenance manual. Costs for these services typically range from 15 to 25 percent of the total construction cost. Cost estimates presented in this report include 20 percent of the estimated construction cost for engineering services.

6.2.4 Legal and Administrative

Legal and administrative costs cover such needs as administration of the funding, interest on interim financing, regulatory review fees, recording easements, legal advertising, and any legal assistance that may be required throughout the duration of the project. A cost equal to 5 percent of the estimated construction cost is included for legal and administrative services.

6.3 WATER QUALITY ANALYSIS

BCI performed a computer-based mathematical mixing zone study to evaluate the post-discharge concentrations of chlorine, ammonia, and dissolved oxygen in the defined mixing zone region of Beaver Creek. The methodology and results of the study are presented in the following sections.

6.3.1 Stream Flow Measurements

The DEQ requires that the 7Q10 flow rate in the receiving stream be used in evaluating potential impacts to the water quality of the receiving stream from wastewater discharges. The 7Q10 flow rate is defined as the 7-day average low-flow condition with a recurrence interval of 10 years. Since wastewater from the City of Aumsville is discharged into Beaver Creek only during the wet weather season (November through April), representative stream flow during this period should be used to approximate the 7Q10 flow condition.

The United States Geological Survey (USGS) does not maintain a stream flow monitoring station near the wastewater discharge outfall on Beaver Creek or Mill Creek, the receiving water that Beaver Creek empties into. Although a stream gauge was installed in Beaver Creek near the wastewater outfall in the mid-1970's, the discharge curve for this gauge does not appear to be accurate based on field measurements conducted as part of this study. As a result, long-term stream flow data is not available. Following discussions with the DEQ, a field survey of the streambed was conducted on September 26, 1997, and the measured flow was used as a basis to approximate the 7Q10 low flow condition. A proportional-area analysis was not conducted due to the relatively large distance between the outfall and nearest downstream monitoring station.

Streambed cross sections were surveyed at the outfall, 25-feet upstream of the outfall, and at stations 50, 100, and 150-feet downstream of the outfall. The width and depth of Beaver Creek ranged between 6 and 15 feet, and 0.3 and 1 foot, respectively. The velocity of the stream was measured using a propeller-driven Swoffer velocity meter, and ranged between 0.1 and 0.5 fps. The water depth at the outfall was measured at 0.6 feet, with a velocity of 0.4 fps. Flow rates were calculated for two of the cross sections and averaged, yielding a result of 1.6 cfs.

Since this flow rate estimate is based on data obtained during the dry weather period, a flow rate of 3.0 cfs was used for modeling purposes to more reasonably represent a 7Q10 flow rate during the discharge period.

Refer to Appendix C for stream data and related calculations.

Table 6-4: Beaver Creek Physical Data[1]

Station of Cross-Section[2]	Average Depth (ft)	Width (ft)	Area (ft^2)	Average Velocity (ft/s)	Flow Rate (cfs)
0+00	0.56	7.5	4.2	0.36	1.5
0+50	0.63	7.1	4.5	0.36	1.6

[1]Physical data for Beaver Creek based on a field survey conducted September 26, 1997.
[2]Stationing in downstream direction from WWTP effluent discharge outfall.

6.3.2 Water Quality Data

The mixing zone study conducted is limited to mathematical modeling only. No water quality sampling or field investigations were conducted as part of this study. Dissolved Oxygen (DO) and BOD_5 concentrations for Beaver Creek were estimated based on data from similar watersheds. Wastewater data was obtained from Discharge Monitoring Reports (DMRs) for the parameters of flow, BOD_5, and chlorine. A summary of water quality data used in the mixing zone study is presented in Table 6-5.

Table 6-5: Available Water Quality Data for City of Aumsville WWTP Discharge to Beaver Creek

Parameter	Value
Flow Rate	0.720 mgd[1]
BOD_5	6.1 mg/l[2]
Ammonia	5.04 mg/l[3]
Chlorine	0.14 mg/l[4]

[1]Average for the month of November, 1993-1996, from WWTP DMR's.
[2]Average for the years 1993-1996, from WWTP DMR's.
[3]From independent laboratory analysis, 12/3/97.
[4]Average for the years 1993-1996 from WWTP DMR's.

6.3.3 Effluent Flow Rate

During the permitted discharge period of November through April, the 7Q10 low flow for Beaver Creek will likely occur in the month of November for any given year. Therefore, the effluent flow rate used in the mixing zone study is the average discharge rate for the month of November for the years 1993-1996. The flow rate is 0.72 mgd. It should be noted that since the City's lagoon system is often reaching capacity at this time, the average discharge rate is very high as the City is discharging at the maximum rate possible to reduce lagoon water levels. Therefore this is a conservative value. At a flow rate of 3.0 cfs in Beaver Creek, this corresponds to a theoretical full dilution ratio of 2.7:1.

6.3.4 CORMIX Mixing Zone Model

Methods
The mixing zone evaluation for the parameters chlorine and ammonia was conducted using the Cornell Mixing Zone Expert System (CORMIX) model. The CORMIX model was developed by the USEPA Center For Exposure Assessment Modeling, in conjunction with Cornell University. The model is intended to analyze, predict, and design aqueous discharges into watercourses and places emphasis on the geometry and dilution of the initial mixing zone. The model is comprised of three modules that simulate the following discharge scenarios:

- Submerged single-point discharges (CORMIX 1)
- Submerged multi-port diffuser discharges (CORMIX 2)
- Buoyant surface discharges (CORMIX 3).

Given the geometry of the wastewater outfall, CORMIX 3 is most representative of the discharge into Beaver Creek and was used in the study.

Water quality standards for wastewater discharges into the Willamette River Basin are regulated by OAR Chapter 340, Division 41. According to OAR 340-41 Table 20, the acute freshwater quality criteria for chlorine is 0.019 mg/L and the chronic criteria is 0.011 mg/L. Although freshwater quality criteria for ammonia are temperature and pH dependent, a 1 mg/L standard is commonly applied at the edge of the mixing zone. In the CORMIX model, the acute criteria is represented as the Criterion Maximum Concentration (CMC) and the chronic criteria is represented as the Criterion Continuous Concentration (CCC).

The chronic water quality criteria (CCC) is applied at the edge of the Regulatory Mixing Zone (RMZ). An RMZ of 100 feet was established for the City of Aumsville by the DEQ as part of its NPDES permit for the wastewater discharge.

The acute water quality criteria is applied at the edge of the Zone of Initial Dilution (ZID). The ZID is the area immediately around the discharge where mixing and dilution are most influenced by the initial turbulence and buoyancy of the discharge. The CORMIX model refers to the ZID as the Toxic Dilution Zone (TDZ). According to the USEPA, the available dilution in the ZID (TDZ) and the distance to the point of compliance with the acute water quality criteria (CMC) is determined by the most restrictive of the following criteria:

- Ten percent of the distance from the edge of the outfall structure to the edge of the Regulatory Mixing Zone in any spatial direction.
- Fifty times the discharge length scale in any spatial direction.
- Five times the local water depth in any horizontal direction.

Results

Under the assumed flow conditions, the mixing zone is effluent-dominated. As a result, CORMIX will not determine the distance downstream required to achieve sufficient dilution of the wastewater. The model only predicted the concentration of the pollutant plume for the near-field region, which indicates that the discharge effluent is fully mixed with Beaver Creek at that point with no further dilution possible. Specifically, the plume concentration for total chlorine was 0.03 mg/l at the edge of the near-field region, a distance of 24 feet downstream from the outfall. This implies that the existing wastewater discharge exceeds the chronic water quality criteria (CCC) for chlorine within the 100-foot RMZ and the three USEPA criteria within the ZID (TDZ) for the acute water quality standard (CMC) for chlorine.

Toxicity standards for ammonia are pH and temperature dependent. Using a water temperature and pH of 10 degrees Celcius and 7.0, respectively, the CMC for ammonia is 25 mg/l, while the CCC is 2.2 mg/l. The plume concentration for ammonia was 1.2 mg/l at the edge of the near-field region, which is less than the CCC of 2.2 mg/l. This indicates that the existing wastewater discharge satisfies the chronic water quality criteria (CCC) for ammonia within the 100-foot RMZ. Also, since the CMC for ammonia is 25 mg/l and the discharge ammonia concentration is approximately 5.40 mg/l, it is intuitive that the acute standard for ammonia is met. Mixing zone modeling calculations and CORMIX results are presented in Appendix C.

Conclusions

The results of the model indicate that the total residual chlorine concentration downstream of the Beaver Creek outfall exceeds both the chronic water quality standard at the edge of the Regulatory Mixing Zone (RMZ) and the acute water quality standard within the Zone of Initial Dilution (ZID). While no CMC applies to ammonia, the model also predicted that the CCC for ammonia is not achieved within the RMZ. These results, however, are based on very limited stream flow data. It is recommended that the City conduct stream-flow monitoring over the course of several discharge periods in order to obtain accurate and representative stream flow data on which a more comprehensive mixing zone study could be based.

6.3.5 Oxygen Sag Modeling

Dissolved oxygen concentrations in Beaver Creek were modeled using the Streeter-Phelps oxygen sag equation. The model was applied using the assumed low flow condition for Beaver Creek of 3.0 cfs and the average effluent discharge rate of 0.72 mgd as reported in the DMRs. Complete input parameters for the Streeter-Phelps oxygen sag model are listed in Table 6-6.

Water quality standards for DO concentrations in water bodies located within the Willamette River Basin are regulated under OAR Chapter 340, Division 41. Based on a review of the water quality limited list published by the Oregon DEQ under Section 303(d) of the Clean Water Act, Beaver Creek is not designated as a "water quality limited" water body for DO.

The results of the model indicate that the maximum DO deficit of 3.8 mg/L occurs 77.2 miles downstream of the outfall. The resulting DO concentration in the river at the point of maximum deficit is 7.5 mg/L, or 66 percent of saturation. At a point 5 miles downstream of the outfall near the confluence of Beaver Creek and Mill Creek, the model predicts the DO deficit to be 2.1 mg/L. Dissolved oxygen model results are presented in Appendix C.

Conclusions

Based on the modeling results, the wastewater discharge into Beaver Creek at the assumed low flow condition does not appear to significantly impact the dissolved oxygen water quality of the stream. These results, however, are based on very limited stream flow data. It is recommended that the City conduct stream-flow monitoring over the course of several discharge periods in order to obtain accurate and representative stream flow data on which a more comprehensive oxygen sag study could be based.

Table 6-6 : Dissolved Oxygen Model Input Parameters

Model Input Parameter		Input
Stream	Elevation (ft)	150
	Flow (cfs)	3[1]
	Velocity (ft/s)	0.8[2]
	Dissolved Oxygen (mg/L)	11.3[3]
	BOD_5 (mg/L)	1.0[1]
	Reaeration Constant @ 20 °C (1/day)	0.3[1]
	Deoxygenation Constant @ 20 °C (1/day)	0.1[1]
	Temperature (°C)	10[4]
	Ammonia (mg/L)	0.1[1]
Effluent	Flow (MGD)	0.72
	Dissolved Oxygen (mg/L)	5.0[1]
	BOD_5 (mg/L)	6.1[1]
	Temperature (°C)	10.0[5]
	Ammonia (mg/L)	5.0[1]

[1]Based on January 5, 1998 phone conversation with the DEQ.
[2]Calculated by applying 3 cfs to the surveyed stream channel cross-section.
[3]90% of saturation at 10 degrees Celcius.
[4]Based on surface water temperatures for similar watersheds.
[5]Based on City of Aumsville, Oregon WWTP influent flow data.

6.4 LAGOON WATER BALANCE

A water balance was completed on the lagoon system to evaluate the dynamics of the inflow/storage/outflow process with time. The water balance is based on WWTP inflow and outflow, as measured by the influent and effluent flow metering devices, while accounting for precipitation and evaporation. Precipitation values are based on local daily rainfall data, and evaporation was approximated from the annual evaporation graph of the United States produced by the U.S. National Weather Service.

As shown in the table in Appendix D, the water balance cannot be an accurate reflection of existing conditions. The "Cumulative Stored" column, which is a running total of all inflows and outflows from 1993 to 1997, shows a cumulative stored volume of 143 million gallons at the end of February 1997. Since the maximum storage volume available for the entire lagoon system is approximately 79 million gallons, this value is not feasible.

It is unlikely that the precipitation and evaporation components of the water balance would cause a disparity of this magnitude. Therefore, the source of the imbalance is quite likely to be a flow measurement problem at the influent and/or effluent flow measurement device. The problems with the influent flow meter are discussed in more detail in Chapter 5.

CHAPTER 7 - DEVELOPMENT AND SCREENING OF ALTERNATIVES

The total wastewater system is made up of four (4) subsystems or components, as listed below: Deficiencies exist in each of the system components.

- Collection System,
- Treatment System,
- Sludge Disposal System, and
- Effluent Disposal System.

Through the analysis and evaluations completed in this study, several specific issues have been identified that affect both the wastewater collection and treatment system, including the following:

- The collection system experiences significant amounts of I&I during the wet weather months.

- The 10-inch trunk line into the WWTP does not have adequate capacity to convey the peak flows without overflowing.

- The WWTP is operating above the maximum hydraulic load.

- The primary lagoons are operating at or above the maximum organic load.

- The disinfection system does not provide adequate contact time at peak flows.

Some cost-effective solutions must be developed to assure that wastewater collection and treatment will comply with both existing and anticipated future discharge standards. Each component must be considered, and alternative approaches developed to address the identified needs. These alternatives must also assure that the plant will be of adequate size and condition to allow for population increases during the next 20-year period without major additional expense.

This chapter will develop and analyze the alternatives to correct the known and expected deficiencies. For those alternatives determined to be feasible for the City of Aumsville, detailed cost estimates will be developed for each basic system component. A cost-effective analysis and matrix evaluation will then be used to isolate the alternative most appropriate for implementation. After the alternatives have been defined, the interrelationship of all four (4) subsystems will be considered and system-wide improvement alternatives will be developed.

To begin the process of defining the final system improvement alternatives, various individual component alternatives are presented below which directly address known problems in the particular subsystem or component.

7.1 CONVEYANCE SYSTEM

7.1.1 Conveyance System Alternatives

The existing conveyance system and its needs were examined in earlier chapters. The evaluation led to the following conclusions:

- The review of I&I data reveals that the City has a significant I&I problem. This is apparent by the significant increase in flow to the treatment facility during heavy rainfall events. The average annual flow of 179 gallons per day per capita is significantly higher than the average Oregon city, which ranges between 110 and 130 gallons per day. The PIF peaking factor of 7.76 is higher than the more typical peaking factors of 4 to 5.

- The collection system analysis shows that 71 percent of the collection system has I&I at or below 2 gallons per inch-diameter-foot per day, and 15 percent of the system has flows above 4 gallons per inch-diameter-foot per day.

Alternatives for upgrading the City's conveyance system include: 1) no action, 2) upgrade the collection trunk line, 3) provide inflow and infiltration rehabilitation only, and 4) upgrade the trunk line and provide I & I rehabilitation.

No Action: The no action alternative is considered unacceptable if additional sewage flow is anticipated because many of the system components are currently at or above capacity during storm periods. If no action is taken, the City will most likely be unable to meet NPDES permit requirements.

Upgrade the Collection Trunk Line: The existing 10-inch diameter main collection trunk line appears to be capable of passing all flows that the collection system currently experiences; however, wastewater flow projections indicate that this trunk line will be overwhelmed in the near future. As discussed in Chapter 5, the maximum capacity of this pipeline under surcharged conditions is approximately 2.2 mgd, which is essentially equal to the Peak Daily Flow of 2.226 mgd anticipated for the year 2007. Therefore, this trunk line will require upgrading in the near future. Preliminary calculations show that a 24-inch diameter pipe installed from existing manhole A-3 to the influent lift station would be capable of passing projected future sewage flows. As discussed in Chapter 5, the pipe currently passes under a manufacturing facility. This alignment would need to be changed when the trunk line is replaced, and approximately 1,000 feet of additional pipe would be required to keep the trunk line in the public Right-of-Way and easements. However, another option would be to enter into discussions with the landowner to find an alternate solution.

Provide Inflow/Infiltration Rehabilitation Only: Inflow and Infiltration (I&I) has been documented as being excessive. A variety of methods can be utilized to remedy infiltration. Following is a brief summary of the different techniques. I&I removal using any of these techniques depends on the types, frequency, and location of problems. For systems that receive high I&I from within the collection system the removal rate can be much higher versus a system where more problems lie within the service laterals.

Chemical Grouting: Chemical grouting is commonly used to seal leaking joints in structurally sound pipe and manholes. The equipment used consists of a sealing packer and television (TV) camera pulled inside the sewer pipe with cables and winches, because the sealing is done inside the pipe. Excavation is not required unless unique problems develop.

The chemical grouts typically used are acrylamide, acrylate, or urethane gel. The chemicals necessary to form the gels are usually mixed in two separate tanks and pumped through separate hoses to the joint to be sealed. One tank is used to mix and dispense the grouting chemical, the other tank is used to mix and dispense a catalyst. The catalyst initiates a chemical reaction when mixed with the chemical grout. The materials are injected simultaneously into a leaking joint, a gel is formed and the leak is stopped. Urethane gel differs from acrylamide and acrylate gels in that water is the catalyst for the urethane gel material.

Chemical grouting does not improve the structural strength of the pipeline. This rehabilitation technology should not be used on pipes that are broken or deteriorated. If the groundwater table drops below the level of the pipe, the chemical grout may become dehydrated and its useful life shortened. When used appropriately, rehabilitation by chemical grouting has a useful life of 10 to 15 years.

The costs for chemical grouting vary depending upon the number of grouting locations and the quality of sealant used. The chemical grouting process generally includes pipeline cleaning, television inspection, testing all joints, sealing deficient joints, and sealing leaking manholes where needed. The television inspection will occasionally locate a section of pipe not repairable by chemical grouting. A point excavation is required to repair such a leak. Grouting must be repeated approximately every 10 years to control the quantity of I&I in the system because of the limited life of chemical grout. For portions of the system conducive to chemical grouting, one application performed initially and another at the end of 10 years should effectively seal the pipeline during the planning period. The cost for chemical grouting is estimated at $20.00 per lineal foot of pipeline, and includes pipeline cleaning, television inspection, testing and sealing, and an occasional point repair.

Pipe Replacement: Pipeline replacement by conventional excavation and backfill is normally done when the existing pipeline is deteriorated so badly that other methods of rehabilitation are not feasible. Replacement provides the opportunity to correct misalignments, increase the hydraulic capacity of the line by increasing the pipe diameter, repair service connections, or eliminate stormwater entry points such as catch basins. Replacing pipelines can also remove any "incidental" I&I (i.e., minor leaks that would not be cost-effective to remove). A rehabilitation alternative that is similar to complete pipe replacement are point repairs, which involve excavation, backfill, and pipe replacement for selected areas.

The advantage of pipe replacement is the service life gained with modern materials and methods is generally greater than 50 years. The cost of replacement is generally high. The replacement has associated inconveniences, and restoration requirements are bothersome in developed areas.

The costs for 8-inch pipeline replacement should be approximate $75 to $100 per lineal foot, and would include manholes, service lateral re-connection, and surfacing replacement. The cost would vary depending on the depth of excavation, number of manholes and service re-connections, and type of surface replacement. Typically, areas deemed cost-effective for pipeline replacement also include lateral replacement.

Pipe Bursting: Pipe bursting consists of expanding and breaking in-ground pipe and towing in segments of new polyethylene (PE) or PVC pipe. For the pipe-cracking operation, a modified soil displacement hammer is pulled through a pipe run via an aboveground winching system. Cutting blades of different size are fixed on the hammer to break the existing pipe. An expander fitted on the rear of the hammer enlarges the original bore so that pipe of equal or larger diameter can be pulled behind the pipe cracking process. The new pipe is fitted into the trailing end of the hammer unit. As the hammer advances through the old main, it cracks the pipe and the fragments are displaced laterally. Simultaneously, the new liner/pipe is then towed in. If a liner is required, the new conduit pipe is then towed in after the entire length of old main has been cracked and lined.

Pipe bursting is most often used under highways, railroads, and other structures where excavation is not possible or cost-effective. The service life is virtually identical to a new sewer pipe (50 years), since new pipe is actually being installed.

Pipe bursting technology is priced competitively with complete pipe replacement. Spot excavations are required to connect lateral services.

Sliplining: Sliplining is the method of inserting a slightly smaller new flexible pipeline, usually polyethylene, into the existing sewer pipe. This method is typically used where the existing sewer lines are extensively cracked such as in areas with unstable soil conditions, where the lines are badly deteriorating, or in lines with relatively flat grades. Sliplining will reduce the inside diameter of sewer pipe and reduce its flow capacity. Sliplining is generally used on mainlines larger than 8 inches in diameter.

Sliplining involves a minimum of excavations and the accompanying dewatering work. Excavations are required only at insertion pits and for service lateral re-connections. For this reason, sliplining is advantageous in inaccessible or difficult areas, or under landscaping or structures. Sliplining can be installed in existing pipelines having moderate horizontal or vertical deflections. Wastewater flow may be allowed to continue while sliplining operations occur.

The liner pipe is commonly pulled through the existing pipe with a winch assembly placed at a manhole and the liner pipe fed into the existing pipe through an insertion pit. The pipe is pulled by steel cable with the cable attached to a pulling head at the pipe end. The polyethylene pipe will stretch during pulling (one foot per 100 feet is common), and a relax time is required after pulling and before connection at manholes. Increased temperatures will also tend to stretch the pipe.

The service life of a sliplined sewer is similar to a new sewer replaced by conventional trench excavation and backfill, which is about 50 years. The new liner pipe is a pressure-capable pipe itself. A disadvantage of sliplining is that excavations are required at service laterals. This is often time consuming, labor intensive, and correspondingly expensive.

The cost for sliplining varies depending upon the number and depth of required excavations. Costs in excess of $85 per lineal foot of pipe are common.

Inversion Lining: Inversion lining installs a flexible lining material against the existing sewer pipe that is thermally hardened. Access to the sewer is through existing manholes. The liner is fastened to the end of the existing sewer pipe at a manhole. The liner is fed through the manhole and into the sewer pipe by filling the pipe and manhole with water. As water is pumped into the manhole, the flexible fabric is pushed through the pipe and inverted into place. The water is heated to cure and harden with thermo-setting resins.

Inversion lining is appropriate for pipelines requiring minor structural repair, misaligned pipelines, and for correcting corrosion problems. Because this method of rehabilitation does not require excavations, it may be used under highways and buildings. Service lateral connections are typically made with special cutters and sealers from inside the pipe. A television inspection of the existing sewer typically precedes the inversion lining work. The life of an inversion lined pipe has been claimed by the lining manufacturers to be 50 years. Installations with almost 30 years of service are known to exist.

The inversion lining will reduce the inside diameter of an 8-inch pipe by up to 3/4 inch depending on the service requirements. Flow capacity of the pipe may be impeded by the reduced pipe cross-sectional area, or increased by smoothing the flow channel.

Costs for inversion lining are generally well above the $85 per lineal foot estimate for complete pipe replacement.

7.1.2 Preferred Alternative

Collection system upgrades should start by conducting flow monitoring during significant rainfall with high groundwater events. The month of January is usually the best time to conduct these activities. Flows should be measured from manhole to manhole to identify specific pipe reaches within the system where high flows are found to exist and where these problems can possibly be corrected. Specifically, monitor the basins with leakage estimated to be above 2 gallons per inch-diameter-foot. In addition, flows from the new development east of the railroad tracks should be monitored. Monitoring must be done at night, during high groundwater periods. Monitoring should measure flows working upstream from key manholes until the I&I drops below 2 gallons per-inch-diameter-foot.

Televise the pipe within these identified reaches during high flow conditions to visually observe the specific sources of leakage. Chemically, grout repair lines with weeping joints or relatively minor leakage. Excavate and replace pipe sections that have large point source leaks, including lateral sewers or house services.

The recommended I&I repair program should be an annual, ongoing program of monitoring, televising, and repair for the indefinite future. The goal is to reduce I&I and retard the impacts of future system deterioration.

In addition to I&I work, the collection system trunk line should be upgraded.

7.2 TREATMENT - DISINFECTION

7.2.1 Alternatives

Disinfection of wastewater effluent is used to kill viruses and pathogens not removed through the secondary treatment process. The mechanisms include cell wall damage alteration of cell permeability, alteration of the colloidal nature of the protoplasm, and inhibition of enzyme activity. Disinfection removes the potential disease-causing organisms prior to discharge to the receiving water body, protecting those who would recreate in the water. Common disinfecting agents include chlorine gas, liquid chlorine, bromine, ozone, and ultraviolet light. In recent years chlorine compounds have come under closer scrutiny related to toxicity to aquatic life, and the potential to form chlorinated organic compounds, which are known to be carcinogenic.

As part of this facility plan, chlorine gas, liquid chlorine and ultraviolet light were considered in further detail.

Chlorine Gas

Chlorine gas is normally provided in 150 pound to one ton cylinders. The gas is an extremely volatile and hazardous chemical. Chlorine gas is used at all sizes of WWTPs, although it is most common at larger plants where it has proven more cost-effective to use. The equipment is reliable, and the process control well developed. Use of chlorine gas requires several equipment components, including, but not limited to, a chlorinator that meters the gas, an injector for creating a chlorine solution, a leak detector, multiple gauges, and safety equipment. Also, specific building improvements are required to meet current fire codes, including ventilation and system alarms.

Following chlorination, the wastewater would need to be dechlorinated to remove the chlorine residual. For gaseous chlorine systems, sulfur dioxide gas is commonly used to remove the chlorine residual. The dechlorination process is nearly instantaneous, and 1.45 parts of sulfur dioxide are typically needed to remove one part of chlorine. The equipment needed to dechlorinate is similar to application of gaseous chlorine.

Liquid Chlorine

Liquid chlorine is provided as calcium or sodium hypochlorite. The solution typically contains 10 to 15 percent chlorine, in 55-gallon drums. The injection system consists of a diaphragm or peristaltic pump, which has adjustable stroke length and speed control to control the injection rate. The solution is not considered as hazardous to handle, and does not pose the health risks of chlorine gas. The disadvantage is that the compound does have a shelf life, thus requiring routine deliveries to maintain a reliable product. It can also cause corrosion of metal surfaces. For high demand WWTP's, large storage systems would be necessary to maintain the supply.

Following chlorination, the wastewater would need to be dechlorinated to remove the chlorine residual. For liquid chlorine systems, liquid sodium bisulfite is commonly used to remove the chlorine residual. The dechlorination process is nearly instantaneous, and 1.45 parts of sodium bisulfite are needed to remove one (1) part chlorine. The equipment needed to dechlorinate is similar to application of liquid chlorine.

Ultraviolet Light

Disinfection with ultraviolet (UV) light occurs when the UV energy is absorbed in the DNA of the microorganisms, thereby preventing them from propagating. A low pressure mercury lamp is most frequently used in generating UV energy. The primary advantage of UV is that it leaves no residuals to affect aquatic life in receiving waters. The primary disadvantage is that anything in the water that can be seen with the naked eye will block the rays. Both high TSS and turbidity will reduce the effectiveness. To avoid this problem, wastewater is typically treated and filtered to less than 2 NTU. Lagoon facilities do not typically use UV for this reason.

7.2.2 Preferred Alternative

Liquid chlorine, with liquid sodium bisulfite to dechlorinate, is a relatively inexpensive and easy to operate disinfection system. The existing chlorine contact chamber could be upgraded with a second large diameter pipe coupled in series with the existing pipe, and the flash mixer could be repaired or replaced. The existing building would likely be sufficient for storage of all the chemicals, equipment and controls with little improvements. It is anticipated that the City would use one 55-gallon drum of liquid chlorine every 2-4 weeks in the winter, and less in the summer. To meet future NPDES wastewater discharge requirements, a dechlorination process must be added.

Disinfection using UV is appealing as no chemicals are required, dechlorination is not necessary, and there is no potential for the creation of disinfection by-products. However, the City will likely phase improvements to the WWTP, and it is uncertain that they will make the necessary liquid stream improvements required to apply UV before disinfection system improvements are implemented. The disinfection system is a component that will need attention sooner than other system components, due to inadequacies in the existing system and the anticipated chlorine removal requirements in the City's new permit. Finally, UV disinfection does not provide any residual that could keep irrigation sprinkler heads clean. As summer irrigation is very likely, this is an issue to consider.

The existing gas chlorination system needs to be upgraded to meet current codes for storage of chlorine gas, as well as for safety reasons. Also, the gas metering and injection is in need of significant upgrades. However, the WWTP operator and City engineer would like to continue use of chlorine gas. The existing chlorine contact chamber could be upgraded with a second large diameter pipe coupled in series with the existing pipe, and the flash mixer could be repaired or replaced. Therefore, based on the City's preference, future upgrades will incorporate new components for a gaseous chlorination system. To meet anticipated future NPDES wastewater discharge requirements, a dechlorination process must be installed. Sulfur dioxide would used for dechlorination.

7.3 EFFLUENT DISPOSAL

7.3.1 Wet Season Disposal Alternatives

Disposal alternatives during the wet weather season of November through April include the following:

- No action (i.e., continued discharge into Beaver Creek)
- Land application of effluent

No Action

No action would continue discharging into Beaver Creek. The mixing zone study presented in Chapter 6 showed that Beaver Creek appears to be effluent dominated, based on very limited stream flow data. The model results indicate that the total residual chlorine concentration downstream of the Beaver Creek outfall exceeds both the chronic water quality standard at the edge of the Regulatory Mixing Zone (RMZ) and the acute water quality standard within the Zone of Initial Dilution (ZID). And while no Criterion Maximum Concentration (CMC) applies to ammonia, the model also predicted that the Criterion Continuous Concentration (CCC) for ammonia is not achieved within the RMZ.

The future effluent quality would be improved from the current situation due to the addition of improvements identified in the recommended improvement plan. These improvements include the addition of dechlorination facilities, modifications to the outfall structure, and improved treatment efficiency.

Dissolved oxygen modeling presented in Chapter 6 revealed that the wastewater discharge into Beaver Creek at the assumed low flow condition does not appear to lower the DO concentration in the river to prohibitive levels. These results, too, are based on the limited amount of available stream flow data. The model indicates that a maximum DO deficit of 3.8 mg/L occurs 77.2 miles downstream of the outfall, with a resulting minimum DO concentration of 7.5 mg/L, or 66 percent saturation. At a point 5 miles downstream of the outfall near the confluence of Beaver Creek and Mill Creek, the model predicts the DO concentration in Beaver Creek to be 9.2 mg/L with an oxygen sag of 2.1 mg/L.

Land Application

As an alternative to discharging into Beaver Creek, effluent could theoretically be discharged through land application. This alternative, however, is not realistic during the wet weather months due to high precipitation, high groundwater tables, and low evapotranspiration rates in the Aumsville area during the winter. The DEQ regulations limit the land application of effluent to agronomic rates, matching the amount of effluent to the uptake rate of the crop. The local climate and soil conditions near Aumsville effectively preclude the agronomic addition of irrigation water during the winter season.

7.3.2 Preferred Alternative - Wet Season

The most viable wet weather effluent disposal alternative is to continue to discharge effluent into Beaver Creek at the existing outfall location. The outfall should be replaced with a larger diameter pipe and a new submerged single or multi-port diffuser.

7.3.3 Dry Season Disposal Alternatives

Effluent disposal alternatives during the dry weather season of May through October include the following:

- No Action
- Discharge to Beaver Creek
- Land Irrigation

A discussion of each of these alternatives are presented below.

No Action

There is currently no provision for dry weather effluent disposal (i.e., reclaimed water irrigation) in the City's NPDES permit. Influent flows during the dry weather season are treated and stored in the lagoons until the beginning of the permitted discharge period in November.

The treatment facility does not have capacity to store the future flows projected over the planning period, and has been unable to store all of the flow collected over the six-month dry weather period for two of the last three years. Due to these circumstances, the no action alternative is considered to be unacceptable.

Discharge to Beaver Creek

One alternative for summer effluent disposal is to discharge to Beaver Creek. This alternative would require modifying the City's existing NPDES permit which currently prohibits this activity. Although there is no flow monitoring data available for the dry weather season, flows in Beaver Creek are believed to be relatively small and insufficient to support an effluent discharge.

There is currently no provision in the City's discharge permit for future dry weather discharges to Beaver Creek. The DEQ personnel have indicated that such provisions for future permits are unlikely. Although Oregon Administrative Rules provide for hardship considerations and administrative challenges, the City would need to first conduct an extensive dry weather flow monitoring and mixing zone field study for Beaver Creek and commit significant resources to an administrative permitting process. Due to these circumstances, this alternative is not considered to be viable.

Land Irrigation

Another alternative for dry weather effluent disposal is to irrigate treated effluent on agricultural land at agronomic application rates. Oregon Administrative Rule - Chapter 340, Division 55 regulates this activity. Effluent irrigation which has become increasingly popular across the state.

The crops most typically irrigated with reclaimed water are pasture grass, alfalfa, and hybrid poplar trees. The cities of Veneta and Amity, Oregon, as well as many others, have used reclaimed water to spray irrigate fields containing pasture grass with good success. Recent research in this area has focused on hybrid poplar trees due to higher transpiration rates than traditional pasture grasses. A pilot project currently underway in Woodburn, Oregon disposes an average of 360,000 gallons per day of effluent on an area planted with hybrid poplar trees. Woodburn officials report that the trees have shown no signs of over watering and grew to a height of 13 feet in the first year of operation. Hybrid poplar trees are also being irrigated with municipal landfill leachate at the Lakeside Reclamation Landfill in Beaverton, Oregon and the Riverbend Landfill near McMinnville, Oregon. Reports from these facilities claim good success in their initial years of operation.

Hybrid poplar trees generally require less acreage than the more common pasture grasses. They have also been shown to remove potential pollutants from the soil profile, including hydrophobic organic compounds (i.e., benzene and chlorinated hydrocarbons), heavy metals, and other contaminants. Hybrid poplar trees grown under effluent disposal conditions could potentially be mechanically harvested every 6 to 8 years and sold for fiber to local or regional paper mills.

Estimated acreages required for the land application of Aumsville's WWTP effluent onto a poplar tree crop are presented in Table 7-1. These estimates are based on projections for future average dry weather flow rates (Chapter 4). The hydraulic loading rate (inches per month) is the amount of reclaimed water that is applied per unit area of land. The actual hydraulic loading rate used for the design of the reclaimed water irrigation system will be the more restrictive of either the soil permeability or the nitrogen concentration of water that percolates past the root zone. Because nitrogen concentrations in the WWTP effluent are believed to be small, this study assumes that soil permeability will be the limiting factor for the hydraulic loading rate.

The design time period for effluent disposal corresponds to the dry weather season extending from May 1 through October 31. Although this complete period is considered in this study, the climatic conditions near Aumsville are such that the month of October represents the interval with the lowest combined evapotranspiration rate (i.e., precipitation and evaporation). As a result, the area of irrigable land required for effluent disposal will be greatest during the month of October and these acreages are reflected in Table 7-1.

If possible, the City should consider procuring up to 80 acres of irrigable land to provide for at least 50 years of irrigation, due to the diminishing availability of land close to the WWTP. This level of planning will provide the maximum amount of operational flexibility and minimize on-going capital costs associated with effluent disposal.

Table 7-1: Projected Acreage Requirements For Land Application Of Effluent

Cultivated Crop	1997	2007	2022
Pasture Grass	20 - 25 acres	25-35 acres	40-45 acres
Poplar Trees	15 - 25 acres	20-30 acres	35-40 acres

The proximity of the irrigation site to the WWTP is a primary factor in selecting land. Effluent must be pumped from the WWTP to the irrigation site, and the pump station and transmission piping costs are minimized by selecting a site closest to the WWTP. Another primary factor is the cost of the land. If the City does not own enough irrigable land within close proximity to the WWTP, land acquisition costs can be significant. While irrigable land could be leased from private land owners, it is imperative that a lease provide the City with complete control over the land use (i.e., crops to be planted, no operational restrictions) and that the term of the lease be of sufficient length to protect the City's substantial capital investment in the irrigation system (i.e., minimum of 50 years). It should be noted that it may be difficult to successfully procure operational or environmental liability limitations through a leasing arrangement.

A cursory review of potential irrigation sites near the WWTP was conducted as part of this study. These activities included a review of available Marion County tax maps and discussions with City officials. Possible sites identified during this review are listed in Table 7-2.

Table 7-2: Potential Irrigation Sites

Owner	Map No.	Marion County Account No.	Approximate Acreage	Approximate Distance to WWTP Outfall (feet)
City of Aumsville	08-2W-24C Tax Lot 1700	57817369	10	800
Oregon Department of Transportation	08-2W-24 Tax Lot 1000	57635776	22	800
Alle & Bernice Dyk	08-2W-25C Tax Lot 100	57736000	58	5,000
Richard & Ruth Schaefer	08-2W-25B Tax Lot 300	57804001	33	2,000

In addition to the parcels listed above, there may be a few acres of irrigable land available on the WWTP site that could potentially be incorporated into an overall irrigation system.

7.3.4 Preferred Alternative - Dry Season

Irrigation with hybrid poplar trees is considered to the most viable dry weather effluent disposal alternative. Ideally, the City would use all of the dry weather effluent for irrigation in order to minimize the water level in the lagoons. Alternatively, some portion of the dry weather flow could be used for irrigation purposes if sufficient irrigable acreage is not available to the City.

7.4 BIOSOLIDS MANAGEMENT

Previously, generation of biosolids in the lagoon treatment process has been slow due to the anaerobic processes, and the lagoons have stored all solids. Biosolids will be generated in greater volumes with any of the liquid stream wastewater treatment alternatives considered herein. As part of this facilities plan, alternative means for storing, treating and disposing of biosolids must be considered.

Prior to disposal, biosolids must meet strict quality criteria, particularly for land application. They are regulated through 40 CFR 503. Biosolids cannot be land applied unless they meet:

- reduction of pathogens requirements

- reduction in vector attraction requirements
- trace element requirements

7.4.1 Treatment Requirements

Prior to disposal, biosolids must meet strict quality criteria, particularly for land application. They are regulated through 40 CFR 503. Biosolids cannot be land applied unless they meet:

- reduction of pathogens requirements
- reduction in vector attraction requirements
- trace element requirements

Pathogen Reduction Requirements

There are two classes of reduction; Class A and Class B. Class A is complete removal of all pathogens. Class B is a reduction in levels to the point that it is unlikely to pose a health risk when land applied.

Acceptable Class A processes include:

composting	beta ray irradiation
heat drying	gamma ray irradiation
heat treatment	pasteurization
thermophilic aerobic digestion	

Acceptable Class B alternatives include:

- **Alternative One** - Collect seven samples of the sewage sludge at the time it is used or disposed. The mean for these samples of the fecal coliform density shall be less than 2 million MPN per gram of total solids (dry weight basis), or less than 2 million CFU per gram of total solids (dry weight basis).
- **Alternative Two** - One of the following Processes to Significantly Reduce Pathogens (PSRP)shall be used.

  Aerobic Digestion - A solids retention time (SRT) of 40 days at 20 °C or 60 days at 15 °C.

Air Drying - Dried on sandbeds for a minimum of 3 months, 2 of which the ambient temperature must be greater than 0 °C.

Anaerobic Digestion - SRT of 15 days at between 35 °C and 55 °C, or 60 days at 20 °C.

Composting - A temperature raise to 40 °C or higher for 5 days.

Lime Stabilization - Add lime to raise the pH to 12 after 2 hours contact time.

- **Alternative Three** - Any process deemed equivalent to the aforementioned PSRPs by the permitting authority.

Vector Attraction Requirements

In order to land apply biosolids without any restrictions, they must meet one of the following criteria:

- Volatile solids in sludge shall be reduced by a minimum of 38%.

- If an anaerobic digestion process is used and the 38% reduction could not be met, digest a portion of the digested sludge in a bench-scale unit for 40 additional days at a temperature between 30 and 37 °C. If volatile solids are reduced by less than 17%, then vector attraction reduction has been achieved.

- If an aerobic digestion process is used and the 38% reduction could not be met, digest a portion of the digested sludge that has a percent solids of 2% or less in a bench-scale unit for 30 additional days at 20 °C. If volatile solids are reduced by less than 15%, then vector attraction has been achieved.

- If an aerobic digestion process is used, oxygen uptake rate shall be no greater than 1.5 mg oxygen per hour per gram of total solids at 20 °C.

- Aerobic processes shall treat the sludge for a minimum of 14 days at an average temperature of 45 °C and a minimum temperature of 40 °C.

- Alkali addition processes shall raise the pH level to no less than 12 for two hours, and maintain a pH level of 11.5 or greater for an additional 22 hours without adding additional alkali.

Trace Elements

Each of the following elements is given two limits for Class B biosolids; cumulative loading rates and maximum concentration. The biosolids' EQ rating is based on where it falls with respect to the two limits.

Arsenic - 41 kg/ha; 41 mg/kg	Mercury - 17 kg/ha; 17 mg/kg
Cadmium - 39 kg/ha; 39 mg/kg	Nickel - 420 kg/ha; 420 mg/kg
Copper - 1500 kg/ha;1500 mg/kg	Selenium - 100 kg/ha;100mg/kg
Lead - 300kg/ha; 300 mg/kg	Zinc - 2800 kg/ha; 2800 kg/ha

7.4.2 Biosolids Stabilization Alternatives

Biosolid stabilization is a key process in any wastewater treatment system. The objective is to stabilize the sludge for ultimate disposal or use, reducing pathogens, eliminating offensive odors, and minimizing the potential for putrefaction. The success of the treatment process is related to the effects on the volatile or organic fraction of the sludge. Survival of microorganisms will defeat the desired objectives in the treatment process. The most common means of sludge stabilization include anaerobic digestion, aerobic digestion, composting and lime stabilization.

Anaerobic Digestion: Anaerobic digestion is the most common and oldest form of sludge stabilization. The process is most commonly used in larger wastewater treatment systems. The sludge is treated in large tanks in the absence of oxygen. It is heat treated, and sometimes mixed, with the ultimate goal of converting solids to methane, carbon dioxide and water.

The process yields non-putrescible biosolids low in pathogens. It typically achieves good volatile solid destruction, is effective at reducing the total sludge mass, has low energy requirements, and the resulting solids can be used for agriculture application. The disadvantages are that it does require skilled operators, the methane-formers are slow growing, the process recovers slowly from upset, tank cleaning is difficult, and it has a high initial cost.

Aerobic Digestion: Aerobic digestion is most commonly used for smaller wastewater treatment systems. The process utilizes aerobic biological reactions to destroy the biologically degradable organic components of the sludge. It is very similar to the extended aeration suspended growth biological wastewater treatment process. The process typically takes place in a separate aerated basin.

Aerobic digestion generates a relatively inoffensive and biologically stable end product, is relatively simple to operate, has a low odor potential, and does not produce explosive gasses. The disadvantages include high power costs for aeration, a limited ability to consistently produce a product in compliance with the EPA standards, and, for mechanically aerated basins, mixed results in dewatering.

A similar alternative to this process is digestion in a facultative sludge lagoon (FSL). Solids from the biological wastewater treatment process are discharged into a lagoon where aerobic and anaerobic bacteria treat the sludge. In undertaking major wastewater system upgrades, a number of small communities have converted existing lagoons to FSLs. The FSLs have been found to overcome some of the disadvantages of aerobic treatment, resulting in the ability to produce a product to EPA standards. In an FSL, the solids retention time can easily meet the requirement for volatile solids destruction. The FSL can also provide additional curing and thickening through anaerobic digestion. Odors are minimized by maintaining a water cap over the sludge.

Composting: Composting has been receiving increased attention for the stabilization and ultimate disposal of sludge. The aerobic process converts sludge to a stable end product through use of bacteria and fungi. Properly composted sludge is a sanitary, nuisance free product. Approximately 20 to 30% of the volatile solids are converted to carbon dioxide and water. The process takes place at a temperature of 50 to 70 degrees Celsius, and pathogens are destroyed. The biosolids can be composted either separately or in combination with wood chips or other solids wastes.

Composting is accomplished through three primary means: aerated static pile, windrow, and in-vessel systems. The latter two are most applicable to sludge composting. Windrows consist of piles of a sludge and wood chip mixture, 3 to 6 feet high and 6 to 14 feet wide at the base. Rows are turned and mixed periodically, with no mechanical aeration required. Typically the material is composted for 2 to 3 weeks, and turned three to four times a week. At the end of this period, the compost must cure for approximately 30 days before it can be marketed. In-vessel composting occurs in an enclosed vessel, where mechanical systems control environmental conditions to speed up the process. Typically these systems yield a marketable material quicker than through the windrow process.

Composting eliminates the need for digestion, is a less capital intensive process, minimizes the need for land application area, and produces a useful product. However, the process does require a relatively large area, it can generate offensive odors, and requires a market for the ultimate product. The operation is considerably different than that of operating a wastewater treatment plant.

Lime Stabilization: Chemical stabilization of sludge has been a practical alternative for many years. Typically, lime (as $CaOH_2$ or CaO) is used to elevate the pH to chemically inactivate microorganisms. The process depends on maintaining the pH at a high enough level for a sufficient period of time to inactivate the organisms. Maintaining a pH of 12 for 2 to 3 hours is relatively common.

The process has a low capital cost, and is relatively easy to operate. It can also used as a emergency stabilization method. The disadvantages include the high pH necessary to inactivate the organisms and the requirement for handling of a caustic material that can cause burns. Also, it is less applicable to land application following treatment.

7.4.3 Preferred Alternative- Biosolids Treatment

The City currently has adequate lagoon space for biosolids treatment in the event a new mechanical wastewater treatment process is constructed. For an aerated lagoon upgrade alternative, the biosolids could be treated in the aerated and sedimentation lagoons. Construction of a separate aerobic or anaerobic digestion system would certainly be more costly, and add more complications to the overall system operation. The City does not have much room available for composting, and the potential for odors would cause problems with the surrounding residential neighborhoods. Lime stabilization is the most hazardous of the alternatives, and the end product may pose disposal difficulties..

Treatment through the lagoon process, either in a separate FSL or in an upgraded aerated lagoon process, is considered the most viable for the City.

7.4.4 Ultimate Use and Disposal of Biosolids

Generally, land application and landfilling have historically been the two alternatives for biosolids disposal. More recently the value of biosolids has been recognized, and an effort has been made to distribute and market it as organic supplement and fertilizer. With this increased attention to the ultimate use of biosolids has come increased attention from the EPA and the general public. It is common to meet with public opposition to the siting of a new disposal or processing facility, primarily due to concerns over odors and adverse health impacts. Each of the three alternatives is considered in detail herein.

Land Application: The concept of land application refers to the beneficial use of sludge through application to the land. Common sites for land application include agricultural land, pastures, tree farms, or mine reclamation. Land application does not simply infer beneficial use as related to crops or animal grazing.

In applying biosolids to the land, the sludge must first be classified as non-hazardous and meet criteria for maximum concentrations of trace metals, as established by the EPA and the DEQ. The sludge must be processed to significantly reduce pathogens. Sludge must also be applied at agronomic rates that are dependent on the crop and soil type. The nutrient content is typically the limiting factor in land application.

Land is the other significant parameter of concern in a successful land application program. Adequate land must be available, which is dependent upon the weather, time of year, crop type, and farming practices. Depending on the type of wastewater treatment process, and thus the quantity of sludge produced, an additional safety factor with respect to the land area may be desired. Also, it is required that storage space be available at the WWTP.

The primary advantage of land application is the beneficial re-use. The primary disadvantage includes the need to provide storage volume at the WWTP. In addition, the treatment requirements pose a hurdle, albeit one that must be overcome regardless of the disposal alternative selected.

Landfilling: Disposal of biosolids to a municipal solid waste landfill is another alternative. The costs of disposing of sludge are high, and generally force additional sludge dewatering to reduce volume. Landfill disposal is generally only considered when adequate landfill space is available at a nearby site and an area for land application is not an alternative. It is also an option for disposal during wet periods when land application is not viable. The DEQ regulations discourage landfilling if other viable alternatives exist.

Biosolids Marketing: Marketing of biosolids is a viable alternative for composted and heat-dried sludge. Typical end users include agricultural and horticultural industries, landscape contractors, and individual homeowners. The responsibility to develop a marketing strategy is left to the municipality. As with any marketing endeavor, the customer(s) must be identified, the product form clarified, a selling price established, and a distribution network established. A vendor can be hired to develop and implement the program, or the City can develop the program. Programs of this magnitude are typically undertaken by larger municipalities who generate large volumes of biosolids and have the resources to develop marketing and distribution programs.

7.4.5 Preferred Alternative- Biosolids Disposal

Land application is the recommended alternative for disposal of stabilized biosolids. The City will need to prepare a biosolids management plan when the existing lagoons are cleaned including characterization of the sludge and securing a land disposal site. The plan may need to be amended each time biosolids are disposed.

7.5 COMPLETE TREATMENT SYSTEM

The detailed alternatives for each component in the treatment system have been previously been developed and preferred alternatives selected, with the exception of the liquid treatment system. The purpose of this section is to summarize complete alternatives for the entire treatment train, and to make a final recommendation for improvements.

7.5.1 Common Improvements

In any expansion or replacement of the wastewater treatment system there are common improvements that must be completed, including upgrades to the collection system, influent pump station headworks, disinfection, effluent disposal, existing biosolids removal, and biosolids treatment and disposal. Each of these improvements is discussed in more detail below.

Collection System: A new 24- inch diameter trunk line must be constructed from manhole A-3 to the influent lift station.

Influent Pump Station: A new influent pump station must be constructed under any of the alternatives. The details were discussed in detail previously.

Headworks: Each of the alternatives would require a new screening system, a new influent flow meter, and new sampling equipment.

- **Screening:** Raw wastewater from the influent pump stations will be discharged through a self-cleaning mechanical screen. The screen would remove rags and large debris that could foul the mechanical equipment. The solids collected would be discharged to a dumpster, and periodically transferred to a local landfill. Drainage from the dewatering process would be discharged back into the influent flow stream.

- **Flow Monitoring:** Under any of the alternatives, the location of the existing parshall flume would not work. A new parshall flume system would be constructed following screening, with sufficient elevation to prevent surcharge at the outlet which would adversely affect the accuracy of the meter.

- **Sampling:** The City's existing sampling equipment could likely be incorporated into a new sampling system. The system would be set up to collect samples based on flow.

Disinfection: The existing disinfectant will be upgraded to liquid chlorine. In addition, the flash mixer must be upgraded and contact basin expanded. The effluent will need to be de-chlorinated prior to discharge to Beaver Creek.

Effluent Disposal: The treated effluent will continue to be discharged to Beaver Creek during the winter, with a new land application system constructed to dispose of wastewater during the summer.

A new outfall pipeline will be constructed to Beaver Creek. The crop for land application will be a hybrid poplar tree.

Existing Biosolids Removal: The existing lagoons have never been cleaned of biosolids. The volume of biosolids in the lagoons needs to be measured, and ultimately removed in any of the treatment plant upgrade alternatives.

Biosolids Treatment and Disposal: Treatment through the lagoon process, either in a separate FSL or in an upgraded aerated lagoon process, is considered the most viable for the City.

Miscellaneous: With any major upgrade to the WWTP, there are ancillary improvements that are necessary. Included would be additional lab equipment, general improvements to the maintenance facilities and buildings (such as shop lighting, a washer and dryer, more efficient HVAC, etc.), and building expansions to house control and electrical equipment.

7.5.2 Treatment - Liquid Stream

The identified needs of the Aumsville wastewater treatment system have been considered and detailed in the previous chapters. Now, treatment alternatives must be developed that will produce an effluent that meets the required discharge requirements. The alternatives for liquid stream treatment of the wastewater are affected by flow rates, influent mass loads, mass load limitations, and water quality limited parameters in the receiving water body. In the future, the City will be required to provide a higher quality secondary treated effluent to meet their existing load limitations.

In this section, the basic wastewater liquid stream treatment alternatives are examined and reviewed. From these basic alternatives, appropriate options are proposed for future plant upgrades. Using this analysis, a preferred alternative will be recommended.

7.5.2.1 General Alternatives

A broad range of alternatives must be considered in planning for major upgrades of wastewater systems. Generally, the alternatives include 1) no action, 2) expand the existing treatment plant, and 3) construction of a new treatment facility. Each of these general approaches is discussed in more detail below.

No Action: The No Action alternative must always be considered in any facilities plan to compare with the detailed options and help establish the need for action. Under this option, the City would continue to operate the existing WWTP as best as possible. However, this alternative is considered unacceptable considering the status of the current treatment facility. As discussed previously, the treatment plant is hydraulically overloaded during the wet season and will receive organic loads in excess of its design capacity in the near future. If no action is taken, the City could be faced with civil penalties. Further, the DEQ could deny new connections to the system due to the lack of capacity to treat additional flows. However, a moratoria for new construction is a prerogative of the City.

Expand Existing Wastewater Treatment Plant: Given the condition of the existing treatment facility, and the projected wastewater characteristics, it is feasible to consider expansion of the existing facility. However, to meet projected influent wastewater conditions and stay within the present mass load limits, the present treatment train must be upgraded to provide additional BOD capacity and produce a better quality effluent.

Primary and secondary treatment at the WWTP is accomplished by the four existing lagoons. In upgrading the WWTP, either additional lagoon capacity could be provided or the lagoons could be mechanically aerated. Additional lagoon capacity would increase both the hydraulic and BOD capacity without increasing the lagoon size significantly. Adding aeration would improve the operation and provide additional hydraulic and BOD treatment capacity. Aeration is typically more efficient at increasing BOD capacity.

Lagoon aeration is most commonly by either mechanical surface aerators or diffused air. Surface aerators agitate the wastewater mechanically to facilitate the transfer of air from the atmosphere. The aspirating mechanical aerator has been found to be successful in lagoon operation. It consists of a long hollow shaft with a motor at the surface and a propeller at the bottom. The propeller draws air through the shaft to form small bubbles. Operation and maintenance of surface aerators is straightforward. The main disadvantage of surface aerators is the power costs for operation. With diffused aeration, air is injected below the liquid surface through diffusers. Because of clogging of the diffusers and associated maintenance, surface aerators are more commonly used in lagoon systems.

The number and size of surface aerators depends upon the projected influent BOD loads, the horsepower required to ensure uniform oxygen dispersion, and the horsepower required to maintain partial or full suspension of solids. If aeration were added, both partially and completely mixed alternatives would be considered. Aerators would be spaced through the pond in proportion to the expected oxygen demand and mixing requirements. Depending on the final arrangement, parts of either one or both of the existing lagoons would be used for settling. Aeration could also be provided in the settling areas to control potential odors and algae problems.

Adding surface aerators to the existing WWTP system allows for needed oxygenation capacity with a minimum of new construction. Also, there is likely no need for acquisition of additional land, although this issue should be considered in the pre-design with respect to construction coordination. The construction cost would consist of the capital cost for the aerators, the installation cost for the placement and anchoring of these units as well as the purchase and installation of baffling (if necessary), and the construction of any modifications to the lagoons. New piping may also be needed between the two lagoons if they are upgraded. In addition, the headworks system would need to be upgraded, including pre-treatment screening, flow measurement, and sampling.

New Treatment Plant: Instead of upgrading the existing wastewater treatment plant, a new facility could be constructed at the present site. The components would include the following:

Headworks: The headworks consist of the following components: influent screening, grit removal, influent sampling and flow metering. Coarse screens or racks are utilized in the removal of large debris. Comminutors can also be used to grind up the coarse solids without removing them from the flow. Grit chambers or mechanical grit are used to remove grit (sand, gravel, cinders, and other heavy solid materials) so as to protect mechanical equipment and reduce heavy deposits in process equipment. The selection of preliminary treatment facilities depends on the nature and condition of the incoming wastewater and downstream treatment processes. Influent flow metering is necessary to space influent samplers and record flow.

Primary Treatment: Primary treatment involves removal of settleable solids and floating material in the influent wastewater to reduce BOD and TSS loadings on the secondary treatment process. Primary treatment is capable of removing from 50 to 70 percent of the influent TSS and 25 to 40 percent of the influent BOD. Since the need for primary treatment is dependent upon the type of secondary treatment utilized within the plant, primary treatment must be examined in conjunction with each of the secondary treatment alternatives considered.

Secondary Treatment: Secondary treatment is directed principally toward the removal of biodegradable material and is typically achieved through biological processes. The effluent from secondary treatment has little BOD and TSS, and may contain several milligrams of dissolved oxygen. Biological treatment processes can be classified in a number of ways such as by metabolic activity (i.e., aerobic, anoxic, or anaerobic) or location of the microorganisms (i.e., suspended or attached growth). Processes from both of these classifications can either be separate or combined into a single treatment system.

Two types of secondary treatment systems are discussed herein: suspended growth systems, and attached growth systems. Suspended growth processes, as typified by the activated sludge process, require the least space and are the most flexible in terms of operation. However, performance of suspended growth processes is more variable and operation is more complex than with attached growth systems. Attached growth systems, such as the trickling filter, are relatively simple to operate and are usually quite stable when treating waste with a consistent strength. Effluent quality from attached growth processes are generally not as good as from suspended growth processes, and oxygen transfer limitations serve to constrain the acceptable influent waste strength.

Representative biological treatment systems were identified and evaluated in further detail. These treatment systems included conventional activated sludge, extended aeration, oxidation ditch, sequencing batch reactor, and trickling filter / solids contact.

Conventional Activated Sludge: The activated sludge treatment system is a suspended growth treatment process. In the basin, the bacteria culture carries out the conversion of organic matter and oxygen to carbon dioxide and new bacterial cells. The aerobic environment is achieved by the use of diffused or mechanical aeration, which also serves to maintain the mixed liquor in suspension. The aeration basins can be designed to operate in any number of modes, including plug flow, complete mix, contact stabilization and extended aeration.

After a period of aeration, the mixture of new and old cells is discharged into a secondary clarifier, where the cells are allowed to settle from treated wastewater. A portion of the settled cells are returned to the aeration basin, while the remainder is wasted for additional solids treatment.

The process has been used extensively throughout the United States in many different forms, and is very flexible. This process offers the ability to adjust the mass of organisms in the basin plus other process variables. A properly designed activated sludge plant with secondary clarification designed for peak winter flows can produce a consistent effluent quality of 20 mg/l BOD and TSS or better.

Many versions of the activated sludge process are in use today, but they are all fundamentally the same. For a new activated sludge WWTP, the design should allow for operational flexibility, with options for complete mix, step feed, and contact stabilization. An inlet channel with multiple feed points would allow such flexibility.

The key components are summarized below. Generally, each component must be duplicated for redundancy, as required by the DEQ.

- Two aeration basins to biologically treat the wastewater.
- Two secondary clarifiers to settle out the solids generated in the aeration basin.
- Return Activated Sludge (RAS) and Waste Activated Sludge (WAS) pumping units.
- Sludge Stabilization.

The advantages of this system are as follows:

- The system has widespread use and flexibility.

- The treatment system can achieve very good removal rates.

The disadvantages of this system include:

- The operators would need to learn a more complex process of treatment and the City would require a higher level of operator certification.

- The operation and maintenance of an activated sludge process is consuming, costly and complicated.

- The cost for the required redundancy.

Extended Aeration: The extended aeration process is a modification of the activated sludge process. It has often been used for smaller wastewater treatment systems. In this system, the mean cell residence time (MCRT), or sludge age, is typically fifteen (15) to twenty-five (25) days. Comparatively, the MCRT in a conventional activated sludge process is three (3) to ten (10) days. The biomass operates in the endogenous growth phase, where the food is limited and the cells also consume each other as die-off occurs. This results in a stable sludge. The advantages include:

- High BOD removal.

- Nitrification often occurs.

- Process stability.

- Reduced sludge production.

The primary disadvantages include:

- Large aeration basins are required to maintain the MCRT.

- The systems are "mixing limited," which means that the basin energy demand is volume dependent and cannot be reduced proportionally with a decrease in incoming oxygen demand. The ultimate power requirements are based on the mixing requirements.

A modification of the process, known as Earthen Basin Extended Aeration (EBEA), has been developed to overcome these two disadvantages. It is a system that can be built as a modification to the traditional facultative lagoon process.

The Parkson Corporation has developed the EBEA process as a packaged treatment system. The system uses longer MCRT's (30 to 70 days) and floating fine bubble diffusers to aerate and mix the wastewater. Despite the increased MCRT, these systems are often more cost-competitive because they are built in earthen basins. The specially designed fine bubble aeration chains are installed perpendicular to the flow path and move across the basin surface providing increased mixing and more efficient oxygen transfer. The air can be controlled to provide anoxic conditions as well, allowing for denitrification.

For settling the MLSS, either an in-basin or separate clarifier can be constructed. The packaged system utilizes the in-basin alternative. Sludge recycling and wastage is completed through separate pumping systems. Control of the treatment system occurs through the use of timers to turn on/off pumps and blowers. Therefore, diurnal flow patterns can be handled more efficiently.

The EBEA process can yield an effluent with less than 10 mg/l BOD_5 and 10 mg/l TSS. The obvious advantage is the cost savings in using an existing earthen basin. The cost savings will not be as significant if a new basin must be built. The City will also be subject to a higher level of operator certification.

Oxidation Ditch: The oxidation ditch is also a modified activated sludge process. The system consists of a ring or oval shaped channel, and is equipped with mechanical aeration devices. These systems typically operate in an extended aeration mode with long detention time and solids retention times. Wastewater must be screened prior to entering the system. Primary clarification is not required. It is a good alternative for achieving nitrification. Typically this process is utilized for smaller communities where land area is limited.

The advantages of this system include:

- The process can produce effluents of 10 to 20 mg/l for BOD & TSS.
- The process if flexible.

The disadvantages are the same as listed under Conventional / Activated Sludge option.

Sequencing Batch Reactor: The sequencing batch reactor (SBR) process is a packaged activated sludge WWTP that has been applied effectively for small communities. The SBR is a batch flow, non-steady-state activated-sludge treatment system. The difference from a conventional activated sludge system is that aeration, sedimentation/clarification, and decant are combined in a single reactor rather than in separate structures for each.

A typical SBR process consists of screening of grit and non-organic material, SBR treatment, and disinfection. The SBR process employs a multistage cycle. The tank, which is partially filled with activated sludge (e.g., mixed liquor), is first filled with raw sewage. During the fill stage the influent is mixed with the settled biological solids remaining from the previous cycle. The tank may also be aerated during this stage. The second stage involves mixing the wastewater and mixed liquor while aerating to create oxidation of the organic matter. Aeration and mixing are stopped during the settle stage to allow the solids to settle. There is no flow through the tank during this settling period, which allows for very high quality separation.

The clarified supernatant is then drawn off the top during the fourth stage, passed through a disinfection facility, and discharged. Solids are periodically withdrawn from the bottom of the tank to maintain the MLSS concentration. The volume treated during each batch varies with influent flow because the batch processes operate within a fixed time frame. When the tank is filled, the raw sewage is automatically diverted to another SBR tank which has completed processing a batch and is ready to receive more raw influent.

The advantages of this upgrade are as follows:

- The process is very simple.
- No sludge recycle system is required.
- The process is controlled by a microprocessor based process controller, requiring minimum operator attention.
- The lack of secondary clarifiers and sludge recycle systems reduces site size requirements and construction costs.
- Since the biomass is acclimated to a wide range of dissolved oxygen and substrate concentrations, shock BOD loads have little or no effect on the process.

The disadvantages include:

- The process is highly automated, which can have serious consequences in the event of a system failure.
- The batch process requires a minimum of two (2) basins.

Trickling Filter/Solids Contact: The TFSC process consists of pretreatment of screening and grit removal, primary clarification, a trickling filter, an aerated solids contact tank, and secondary clarification. The process has been used to upgrade other similar plants where the trickling filter was no longer capable of producing the permit quality effluent.

In general, trickling filters can adequately remove soluble BOD_5 but have problems with dispersed solids in the effluent. In the TFSC process, the influent is subjected to primary treatment then is discharged to the trickling filter. The filter is loaded at the high end of typical organic and hydraulic loading rates, and is expected to reduce BOD_5 by approximately 65 percent. The biological solids formed on the trickling filter will periodically slough off and concentrate through sludge re-circulation in the contact tank. Hydraulic loading rates can range from 0.16 to 3.2 gpm per square foot, and organic loading rates can range from 30 to 500 pounds BOD_5 per cubic foot per day. Treatment efficiency will be reduced at the higher loading rates.

To overcome the performance limitations of the trickling filter and to meet the wastewater discharge permit, an aeration basin (termed a solids contact basin) is added following the trickling filter. The effluent from the tricking filter would contact with secondary return sludge in the aerated, short-detention-time tank. The mixed liquor suspended solids (MLSS) concentration would be maintained at 1,500 to 3,500 mg/l, and the MCRT would be 4.0 to 6.0 days. In addition to the aerobic removal of BOD_5, the solids would be flocculated, improving suspended solids and BOD_5 removal in the final clarifier.

The final unit in the TFSC process is the secondary clarifier. The clarifier serves to settle the flocculated solids, and the overflow would be the treated effluent. The solids settled in the bottom of the basin would be discharged from the tank, with a portion returned to the solids contact basin as return activated sludge (RAS), and the remainder wasted to the digester as waste activated sludge (WAS). To benefit from the TFSC process, aerated solids must be introduced into the secondary clarifier in a gently stirred, flocculated condition. Otherwise, the suspended solids become re-dispersed.

The advantages of the TFSC upgrade are as follows:

- This process is capable of consistently meeting secondary and some advanced secondary standards. Depending on the actual hydraulic loading to the trickling filter, final effluent concentrations less than 20 mg/l BOD_5 and TSS are typical, and can reach as low as 5 mg/l.

- The solids contact basin is considerably smaller than conventional activated sludge aeration basins, and thus the overall cost of operation would not increase as much as other more complete renovations.

- Depending on the flows into the plant and the performance of the solids contact basin, it would be possible to bypass the trickling filter at certain times of the year. This would allow for maintenance of the filter. Additionally, if the treatment efficiency could be maintained, bypassing the filter would also save operating money for power.

The disadvantages of this system include:

- The operators would need to learn a new process, and understand the operational characteristics of two separate wastewater treatment processes.

- The City would be subject to a higher level of operator certification.

- Redundancy requirements would make it necessary to provide two of each of the process units.

7.5.2.2 Development of Alternatives

A number of concepts for upgrading the wastewater treatment plant - liquid stream component have been discussed herein. From this list, the following alternatives are proposed for further consideration, as they are the most feasible alternatives given the existing system.

Alternative 1 - No Action Under this option, the City would continue to operate the existing wastewater treatment plant as best as possible. No construction work would be accomplished on the wastewater collection or treatment systems.

There are several problems with the existing treatment plant. If the plant continues to operate as it has in the past, the City will continue to have excursions outside their permit limitations and will be subject to sanctions and fines by the DEQ. In addition, the basic premise of this Facilities Plan is that by following the recommended improvements, the facilities would operate for a 20-year period without additional improvement beyond routine preventive maintenance, which could not be achieved with this alternative.

This alternative will not be considered further as a stand alone alternative. The use of the facility without other improvements will cause continued violations of discharge permit limits and in-stream water quality standards. The City in time would be placed under sanctions and be required to make improvements.

Alternative 2 - Upgrade Existing WWTP: Aerated Lagoons Adding aeration to the lagoons without return activated sludge would improve treatment efficiency for removal of BOD_5 and meet the future treatment requirements. A system could be designed to work within the existing lagoon plant, with dike modifications. Further, such a system would be flexible enough to allow for additional system improvements, including the addition of RAS capabilities, for upgrades beyond the current twenty (20) year planning period. The specific components and sizes would include the following:

- *Partially Mixed Aerated Basin #1:* A 5 to 6 acre, 12-foot-deep lagoon would be constructed in the existing primary lagoon #1 for partial mix aeration. Soluble, degradable organic matter would be converted to particulate matter, with a portion settling out in the basin and a portion discharged to the subsequent cell. A total of ninety-five (95) horsepower in mechanical surface aeration equipment would oxygenate the wastewater. The total influent BOD_5 would be reduced by approximately 75%, and the soluble effluent BOD_5 would be approximately 35 mg/l, based on the MMWWF in 2022. The design detention time would be approximately twelve (12) days.

- *Partially Mixed Aerated Basin #2:* A 5 to 6 acre, 12-foot deep lagoon would be constructed in the existing primary lagoon #2 for partial mix aeration. The soluble, degradable organic matter discharged from basin #1 would be converted to particulate matter. A portion of the particulate matter, both new and from basin #1, would settle out as the aeration would only provide for full mixing of the soluble material. The remaining suspended solids would be discharged from the lagoon. A total of 20 horsepower in mechanical, surface aeration would aerate the wastewater. Aspirating aerators would be preferred. The total BOD_5 discharged from the cell would be less than 10 mg/l, based on the MMWWF in 2022. The design detention time would be approximately twelve (12) days. Piping between the two (2) lagoons would allow for either parallel or series operation.

- *Settling Lagoon:* Treated wastewater would be discharged from the aeration basins into the existing tertiary lagoon, which would serve to settle out the suspended solids generated through the aerated lagoon process. The lower 2 to 3 feet would be used for sludge storage. The piping between the existing primary cells across the creek would also be increased in size.

- *Tertiary Treatment:* A filtration system will be constructed to remove excess TSS, including algae, to meet the year 2022 effluent TSS requirements. A filtration system is typically designed to treat peak hourly flows. In this case, the equalization through the lagoons will buffer peak flows. The proposed filtration system will be sized to treat the MMWWF in the year 2022.

Both of the new aerated lagoons can be constructed within the existing lagoon system with some diking modification to create new cells and increase the depth. The existing secondary lagoon would not be specifically used in this upgrade, however, it could serve as a surge basin, a storage basin for effluent irrigation, or as a sludge lagoon. The integrity of the liner should be maintained as the lagoon could serve in future upgrades. No dike modifications would be required to the existing tertiary lagoon.

Figure 7-1 shows a schematic diagram of the proposed treatment alternative. Table 7-3 presents the total system costs, including operation and maintenance costs, and salvage value.

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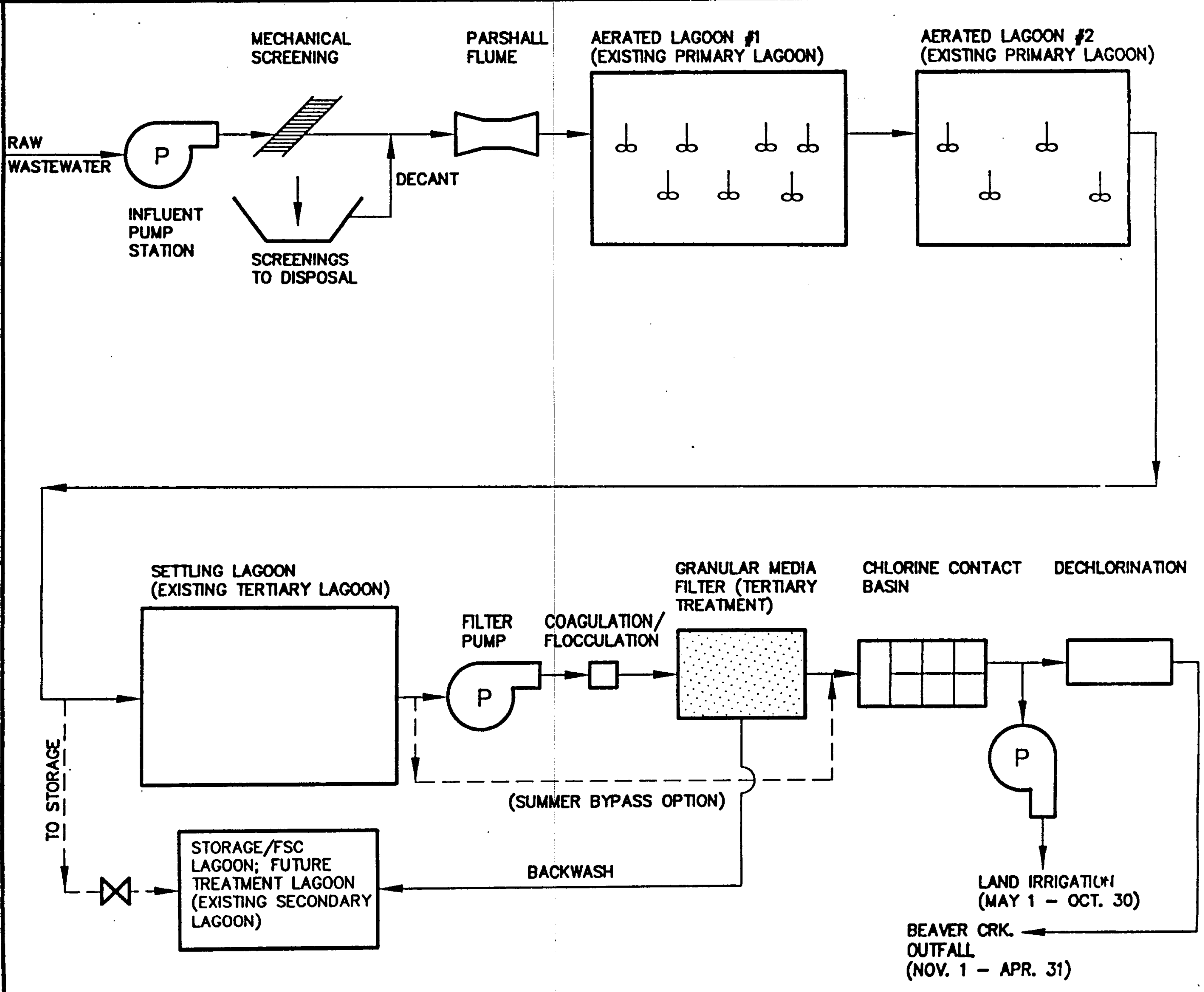

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AUMSVILLE WASTEWATER FACILITIES PLAN

FIGURE 7-1
AERATED LAGOON WWTP
SCHEMATIC DIAGRAM

TABLE 7-3

PRELIMINARY OPINION OF PROBABLE COSTS
ALTERNATIVE 2 - AERATED LAGOONS
1999 COST BASIS

ESTIMATE OF PROJECT COST					
ITEM	QUANTITY	UNITS	UNIT COST	AMOUNT	TOTALS
CAPITAL IMPROVEMENT COSTS					
COLLECTION SYSTEM				$287,000	
INFLUENT PUMP STATION				$300,000	
HEADWORKS				$344,000	
SECONDARY TREATMENT (Aerated lagoons)				$1,099,000	
EFFLUENT FILTRATION				$439,000	
DISINFECTION				$130,000	
EFFLUENT DISPOSAL				$448,000	
BIOSOLIDS				$200,000	
				TOTAL 1999 CONSTRUCTION COST =	$3,247,000
INDIRECT COSTS					
Construction Contingencies	15.00%			$487,050	
Engineering and Construction Management	20.00%			$649,400	
Legal & Administration	5.00%			$162,350	
				TOTAL INDIRECT COST =	$1,298,800
LAND ACQUISITION					$320,000
TOTAL PROJECT COST =					**$4,865,800**
ANNUAL OPERATIONS AND MAINTENANCE COSTS					
Personal Services	1	Annual	$101,650	$101,650	
Materials and Services	1	Annual	$181,600	$181,600	
Capital Outlay	1	Annual	$32,280	$32,280	
Operating Contingency	1	Annual	$17,261	$17,261	
TOTAL ANNUAL OPERATIONS & MAINTENANCE				$332,791	
PRESENT WORTH (8%/yr, 20 yrs)					$3,267,391
SALVAGE VALUE					
Collection	1	ls	$200,000	$200,000	
Treatment Plant	1	ls	$500,000	$500,000	
Land Disposal (tree harvest)	1	ls	$50,000	$50,000	
TOTAL SALVAGE VALUE				$750,000	
PRESENT WORTH (8%/yr, 20 yrs)					($160,911)
TOTAL PRESENT WORTH =					**$7,972,280**

Alternative 3 - New WWTP: Earthen Basin Extended Aeration The EBEA is considered a feasible alternative for upgrading the treatment plant. One of the existing lagoons could be modified to contain the new system. Raw wastewater would be pumped through the headworks, then allowed to gravity flow into the aeration basin. The system would be sized to treat the MMWWF and associated mass loads for the year 2022.

The specific components would be as follows:

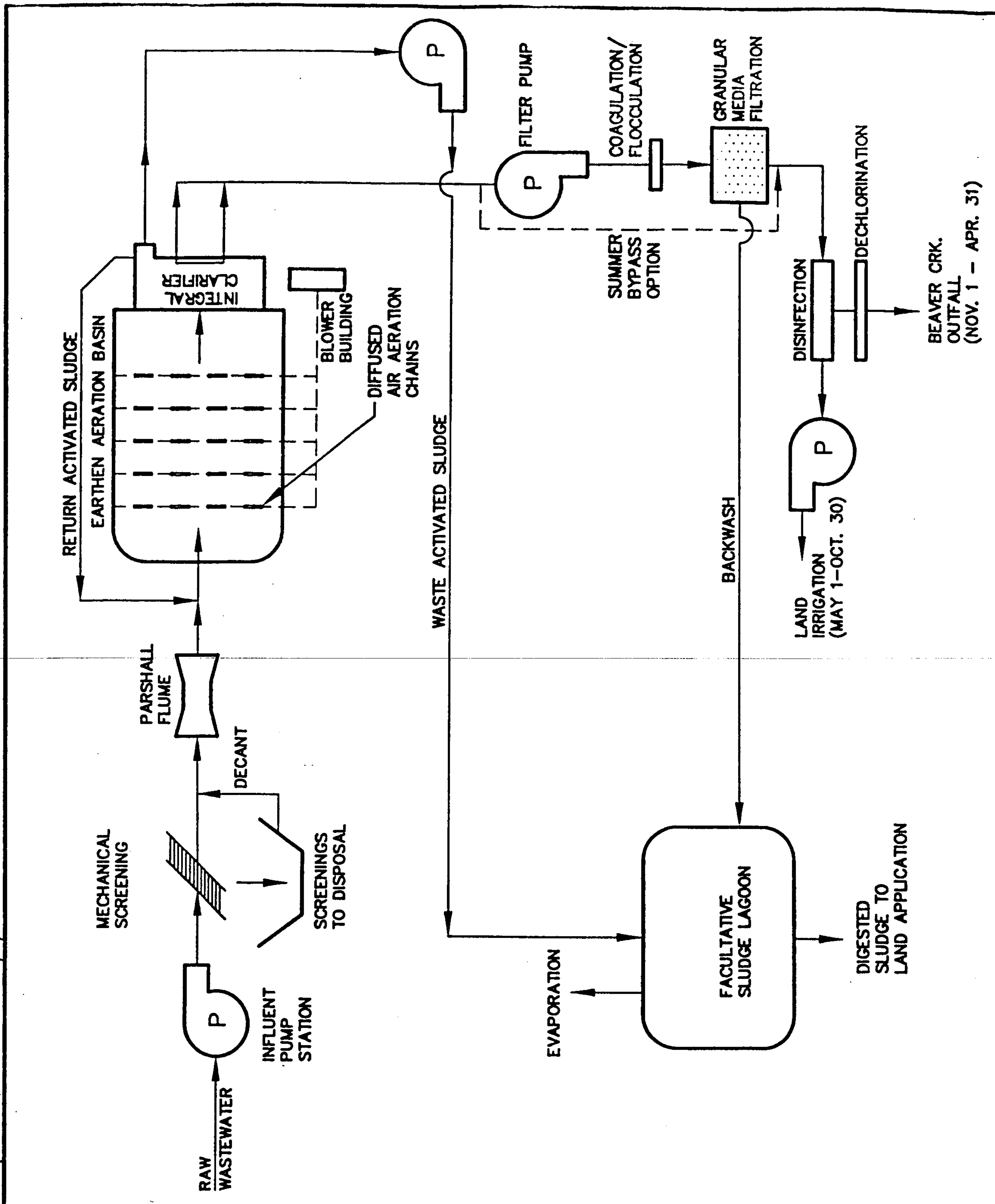
- *Aeration Basin*: An aeration basin would be constructed in the existing tertiary lagoons. The basin would have dimensions of approximately 180 feet by 160 feet, and would be 12 feet deep. Air would be supplied through approximately 100 horsepower in blowers.

- *Clarifier:* An in-basin clarifier would be used to settle out the biomass. Effluent would be discharged to the polishing filter, then to the chlorine contact basin. Sludge from the clarifier would be pumped either back to the aeration basin (RAS) or to a facultative sludge lagoon (WAS).

- *Control Building:* A new control building would be constructed to house the blowers and system controls.

- *Tertiary Treatment:* A filtration system will be constructed to remove excess TSS, including algae, to meet the year 2022 effluent TSS requirements. A filtration system is typically designed to treat peak hourly flows. In this case, the equalization through the lagoons will buffer peak flows. The proposed filtration system will be sized to treat the MMWWF in the year 2022.

Figure 7-2 shows a schematic diagram of the proposed treatment alternative. Table 7-4 presents the total system costs, including operation and maintenance costs, and salvage value.



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AUMSVILLE WASTEWATER FACILITIES PLAN
FIGURE 7–2
EARTHEN BASIN EXTENDED AERATION WWTP
SCHEMATIC DIAGRAM

TABLE 7-4

PRELIMINARY OPINION OF PROBABLE COSTS
ALTERNATIVE 3 - EARTHEN BASIN EXTENDED AERATION
1999 COST BASIS

ESTIMATE OF PROJECT COST					
ITEM	QUANTITY	UNITS	UNIT COST	AMOUNT	TOTALS
CAPITAL IMPROVEMENT COSTS					
COLLECTION SYSTEM				$287,000	
INFLUENT PUMP STATION				$300,000	
HEADWORKS				$344,000	
SECONDARY TREATMENT (EBEA)				$1,695,500	
EFFLUENT FILTRATION				$439,000	
DISINFECTION				$130,000	
EFFLUENT DISPOSAL				$448,000	
BIOSOLIDS				$200,000	
				TOTAL 1999 CONSTRUCTION COST =	$3,843,500
INDIRECT COSTS					
Construction Contingencies	15.00%			$576,525	
Engineering and Construction Management	20.00%			$768,700	
Legal & Administration	5.00%			$192,175	
				TOTAL INDIRECT COST =	$1,537,400
LAND ACQUISITION					$320,000
TOTAL PROJECT COST =					$5,700,900
ANNUAL OPERATIONS AND MAINTENANCE COSTS					
Personal Services	1	Annual	$121,650	$121,650	
Materials and Services	1	Annual	$191,200	$191,200	
Capital Outlay	1	Annual	$37,280	$37,280	
Operating Contingency	1	Annual	$22,261	$22,261	
TOTAL ANNUAL OPERATIONS & MAINTENANCE				$372,391	
PRESENT WORTH (8%/yr, 20 yrs)					$3,656,190
SALVAGE VALUE					
Collection	1	ls	$200,000	$200,000	
Treatment Plant	1	ls	$500,000	$500,000	
Land Disposal (tree harvest)	1	ls	$50,000	$50,000	
TOTAL SALVAGE VALUE				$750,000	
PRESENT WORTH (8%/yr, 20 yrs)					($160,911)
TOTAL PRESENT WORTH =					$9,196,179

Alternative 4 - New WWTP: Sequencing Batch Reactor (SBR) Plant This alternative would include a new mechanical treatment plant. Raw wastewater would be pumped through the headworks, then allowed to gravity flow into the aeration basin. The system would be sized to treat the MMWWF and associated mass loads for the year 2022. The components of this upgrade would include:

- *SBR:* Constructing a new SBR plant to treat the MMWWF and mass loads in 2022
- *Tertiary Treatment:* A filtration system will be constructed to remove excess TSS, including algae, to meet the year 2022 effluent TSS requirements. A filtration system is typically designed to treat peak hourly flows. In this case, the equalization through the lagoons will buffer peak flows. The proposed filtration system will be sized to treat the MMWWF in the year 2022.

A new SBR plant with effluent filtration would be a good choice if the City of Aumsville chooses to construct a new mechanical plant to replace the existing plant. This plant would easily meet the projected effluent discharge standards for BOD_5 and TSS. The process is also capable of both nitrification/denitrification, depending on the operation. This alternative would also meet all of the requirements of DEQ and would serve the City well for 20 years and beyond.

Figure 7-3 shows a schematic diagram of the proposed treatment alternative. Table 7-5 presents the total system costs, including operation and maintenance costs, and salvage value.