Public Works Director's Copy

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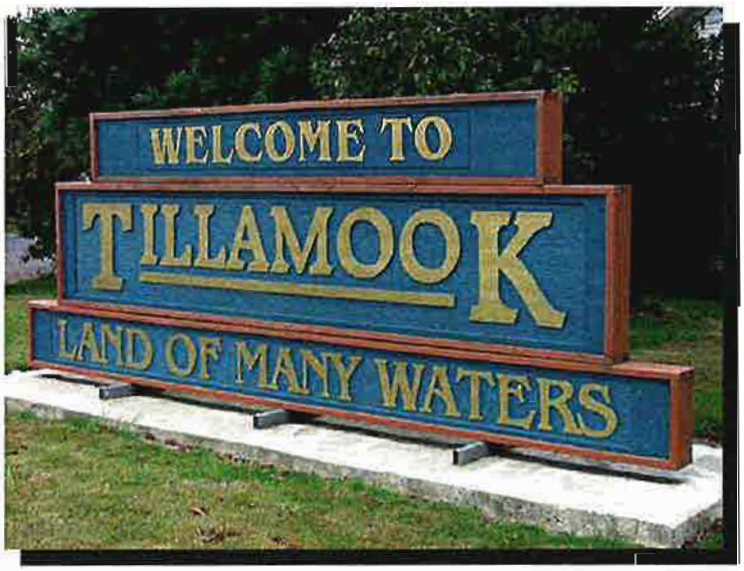
Wastewater Facility Plan
Tillamook, Oregon

August 2005



Prepared for

City of Tillamook
210 Laurel Avenue
Tillamook, Oregon 97141

K/J Project No. 036403.00

FINAL

BEFORE THE ENVIRONMENTAL QUALITY COMMISSION

OF THE STATE OF OREGON

IN THE MATTER OF:	)	MUTUAL AGREEMENT
	)	AND ORDER
CITY OF TILLAMOOK	)	NO. WQ/M-NWR-02-144
PERMITTEE	)	TILLAMOOK COUNTY
	)	

WHEREAS:

1. On January 6, 2003, the Department of Environmental Quality (Department or DEQ) issued National Pollutant Discharge Elimination System Permit Number 101239 (Permit) to the City of Tillamook (Permittee). The Permit authorizes the Permittee to discharge adequately treated wastewater into the Trask River, waters of the state, in conformance with the requirements, limitations and conditions set forth in the Permit.

2. Condition 1 of Schedule A of the Permit does not allow the Permittee to exceed the following waste discharge limitations after the Permit issuance date:

Outfall Number 001

(1) May 1-October 31

Parameter	Average Effluent Concentrations (mg/L)		Effluent Loadings (lbs)		
	Monthly	Weekly	Monthly Average	Weekly Average	Daily Maximum
BOD	20	30	177	265	354
TSS	20	30	177	265	354
FC/100	200	400			

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approved a Corrective Action Plan in June 4, 2001 for additional facilities that are intended to ensure permit compliance. The Department required that the Permittee expand the CAP to improve reliability and redundancy. Permittee is in the process of completing plans and specifications under the expanded Corrective Action Plan.

4. DEQ and the Permittee recognize that until Permittee completes the actions required by this Mutual Agreement and Order (MAO), Permittee will continue to violate the Permit and Oregon law as described in Paragraph 2.

5. The Department and Permittee recognize that the Environmental Quality Commission has the power to impose a civil penalty and to issue an abatement order for violations of conditions of the Permit. Therefore, pursuant to ORS 183.415(5), the Department and Permittee wish to settle those past violations referred to in Paragraph 2; and to limit and resolve the future violations referred to in Paragraph 4 in advance by this MAO as follows:

i) violations of applicable summer limitations until May 1, 2003, and

ii) violations of applicable winter limitations until May 1, 2004.

6.. This MAO is not intended to limit, in any way, the Department's right to proceed against Permittee in any forum for any past or future violations not expressly settled herein.

NOW THEREFORE, it is stipulated and agreed that:

7. The Environmental Quality Commission shall issue a final order:

A. Requiring Permittee to comply with the following schedule:

(1) By February 1, 2003, submit a draft updated plant operations and maintenance manual meeting the DEQ guidelines.

(2) By May 1, 2003, complete the construction of the modifications to the facilities and repairs to the wastewater treatment facilities, including the RBC air piping, as required in the approved plans and specifications for the expanded Corrective Action Plan.

(3) Within 60 days after receiving comments from DEQ on the draft updated plant operations and maintenance manual, submit a complete final version of the operations and

(2) November 1-April 30:

Parameter	Average Effluent Concentrations (mg/L)		Effluent Loadings (lbs)		
	Monthly	Weekly	Monthly Average	Weekly Average	Daily Maximum
BOD	45	67.5	500	900	1200
TSS	45	67.5	500	900	1200
BOD and TSS removal efficiency			Shall not be less than 85% monthly average*		
*Note: When the monthly average flow exceeds 1.5 MGD the percent removal shall not apply.					

The Permittee shall operate the wastewater treatment facility as effectively as possible during those times when the inflow exceeds 1.5 mgd. All BOD/TSS sample results shall be reported on the Permittee's discharge monitoring report. The discharge monitoring report shall contain information on progress with influent monitoring.

E. Requiring Permitee, upon receipt of a written Penalty Demand Notice from the Deparment to pay the following civil penalties:

(1) $250.00 for each violation of the compliance schedule set forth in Paragraph 7.A.

(2) $100.00 for each violation of each daily or weekly average waste discharge limitation set for the in Paragraph7. C.and D.

(3) $ 500.00 or each violation of each monthly average waste discharge limitation set forth in Paragraph 7.C. and D.

8. If any event occurs that is beyond Permittee's reasonable control and that causes or may cause a delay or deviation in performance of the requirements of this MAO, Permittee shall immediately notify the Department verbally of the cause of delay or deviation and its anticipated duration, the measures that have been or will be taken to prevent or minimize the delay or deviation, and the timetable by which Permittee proposes to carry out such measures. Permittee

unless otherwise approved in writing by DEQ.

13. All reports, notices and other communications required under or relating to this MAO should be directed to Beth Moore, DEQ Water Quality Northwest Regional Office, 2020 SW Fourth Ave., Portland, Oregon 97034 phone number (503) 229-5586.

14. Permittee acknowledges that it has actual notice of the contents and requirements of the MAO and that failure to fulfill any of the requirements hereof would constitute a violation of this MAO and subject Permittee to payment of civil penalties pursuant to Paragraph 6.D. above.

15. Any stipulated civil penalty imposed pursuant to Paragraph 6.D. shall be due upon written demand. Stipulated civil penalties shall be paid by check or money order made payable to the "Oregon State Treasurer" and sent to: Business Office, Department of Environmental Quality, 811 S.W. Sixth Avenue, Portland, Oregon 97204. Within 21 days of receipt of a "Demand for Payment of Stipulated Civil Penalty" Notice from the Department, Permittee may request a hearing to contest the Demand Notice. At any such hearing, the issue shall be limited to Permittee's compliance or non-compliance with this MAO. The amount of each stipulated civil penalty for each violation and/or day of violation is established in advance by this MAO and shall not be a contestable issue.

16. Providing Permittee has paid in full all stipulated civil penalties pursuant to Paragraph 15 above, this MAO shall terminate 60 days after Permittee demonstrates full compliance with the requirements of the schedule set forth in Paragraph 6A above.

PERMITTEE

12/30/02
Date

Robert McPheeters
Mayor
City of Tillamook

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ORDINANCE NO. 1202

AN ORDINANCE OF THE CITY OF TILLAMOOK ADOPTING A NEW WASTEWATER FACILITIES PLAN

WHEREAS, the City of Tillamook is required by the Oregon Department of Environmental Quality (DEQ) under its current National Pollutant Discharge Elimination System permit to review its wastewater collection system and treatment plant periodically and develop a new wastewater facility plan document, and

WHEREAS, the city has prepared a new Wastewater Facility Plan document whose scope includes an evaluation of current and future needs of the city's wastewater facilities as well as a projection of future wastewater flows and loads, proposed programs and facility improvements, and a Capital Improvement Plan (CIP).

NOW, THEREFORE, based on the foregoing, the City of Tillamook ordains as follows:

Section 1: The Wastewater Facility Plan document prepared for the City of Tillamook by Kennedy/Jenks Consultants and dated August 2005 along with any subsequent amendments is hereby adopted and incorporated hereto by reference.

Section 2: This new Wastewater Facility Plan document will supersede the Tillamook City Wastewater Facilities Plan dated January 1976.

Section 3: All ordinances inconsistent with this specific wastewater facility plan document are hereby repealed.

PASSED FIRST READING by the Council this 19th day of September, 2005.

PASSED SECOND READING by the Council this 3rd day of October, 2005.

APPROVED by the Mayor this 3rd day of October, 2005.

Robert McPherson
Mayor

ATTEST:
Bernadette M Sorensen
City Recorder

Wastewater Facility Plan

Tillamook, Oregon

REGISTERED PROFESSIONAL ENGINEER
71094PE
OREGON
July 9, 2002
HARRY E. RITTER
EXPIRES: 10/31/2006

August 2005

K/J 036403.00

Prepared for:

City of Tillamook
210 Laurel Avenue
Tillamook, Oregon 97141

Prepared by:

Kennedy/Jenks Consultants
200 S.W. Market Street, Suite 500
Portland, Oregon 97201-5715
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Section 1: Executive Summary

1.1 Purpose

This facility plan was prepared to evaluate the City of Tillamook's (City) wastewater collection system and treatment plant. The scope of this document includes an evaluation of current and future needs, a projection of future flows and loads, proposed programs and facility improvements, and a Capital Improvement Plan (CIP). This document is intended to comply with the Oregon Department of Environmental Quality (DEQ) requirements for wastewater facility plans.

1.2 Study Area and Population

The City's wastewater system serves nearly all of the homes and businesses within the city limits as well as a portion of the developed area outside the city's limits but within the urban growth boundary (UGB).

The estimated population within the City limits is approximately 4,375. Based on the growth rates used in the Comprehensive Plan, and assuming that service is provided to all development within the UGB, the 2025 population served by the wastewater system is projected to be 7,515.

1.3 Existing Wastewater Facilities

The existing facilities include a collection system and a wastewater treatment plant (WWTP). The collection system consists of gravity sewers, 63 septic tank effluent pumping (STEP) units, one primary pump station and four smaller pump stations. The WWTP provides for primary and secondary treatment, disinfection and residual solids processing. Effluent discharge is to the Trask River.

1.3.1 Existing Collection System

The existing collection system consists of approximately 20 miles of gravity pipe and pressure mains, five pump stations, and a STEP system serving an area along north Highway 101. The older part of the system, constructed between 1920 and 1950, consists primarily of clay pipe. Subsequent portions of the collection system were constructed primarily of concrete pipe. Polyvinyl chloride pipe was used in the most recent extensions. In general, the collection system is in poor condition due to its age. Major problems are as follows:

- Infiltration resulting from leaky and misaligned pipe joints is severe. Peak flows under wet weather conditions may be up to ten times the average flow.
- Hydraulic bottlenecks and insufficient capacity result in overflowing manholes under wet weather conditions.

- The major pump station (12th Street Pump Station) does not meet current requirements for capacity and reliability.
- The STEP units along Highway 101 are old, non-uniform, and maintenance intensive.
- Grease accumulation associated with commercial food establishments causes capacity and maintenance problems.

The consequences of these problems are excess wet-weather flow, periodic sewage overflows, intensive maintenance requirements, and the inability to provide sufficient sewer capacity to serve future development within the UGB.

1.3.2 Existing Wastewater Treatment Plant

The existing WWTP has a dry weather permit capacity of 1.06 million gallons per day (MGD) and a wet weather permit capacity of 2.12 MGD. Liquid processes at the plant include influent pumping, grit removal, comminution, primary clarification, fixed-film biological treatment, secondary clarification, disinfection, and dechlorination. Final effluent is discharge to the Trask River. Residual solids treatment includes anaerobic sludge digestion, sludge storage, and land application of Class B biosolids.

The existing WWTP unit processes were evaluated and scored as follows:

- Good condition - few or no repairs needed:
  - Built in 1998: headworks and North Primary Clarifiers
  - Built in 1997: dechlorination facilities
  - Built in 1978: Administration/Lab/Workshop building.
- Fair condition - repairs needed:
  - Built in 1978: RBCs, secondary clarifiers, and outfall
  - Built in 1969: chlorine contact tank.
- Poor condition - major repairs required:
  - Built in 1957: primary digester
  - Built in 1948: influent pump station, sludge holding tanks.

1.4 Population and Wastewater Flows/Loads

Population within the UGB was projected using the estimated annual growth rates obtained from Tillamook County. Wastewater flow and load projections were based on historical WWTP

operating records from 1999 through 2005. The DEQ guideline for wet weather flow prediction was used to predict future flows.

Operating records from May 2003 to May 2005 were evaluated following the original draft submittal of this Facility Plan. These records indicate similar flows but higher loadings than are shown in Section 6. The revised loading data is reflected in Table 1.1. See Appendix K for the memorandum discussing the updated loading data.

Table 1.1: Summary of Design Population, Flows and Loads

	Year	2003	2005	2010	2015	2020	2025
Population (number of persons connected to the wastewater facility)							
Persons Served	Number	4,625	5,116	6,450	6,795	7,165	7,515
Dry Weather Flow (DWF in MGD)							
Average	ADWF	0.55	0.61	0.77	0.82	0.86	0.90
Maximum Month	MMDWF	1.50	1.59	1.90	1.96	2.02	2.08
WET Weather Flow (WWF in MGD)							
Average	AWWF	1.86	1.97	2.31	2.38	2.46	2.53
Maximum Month	MMWWF	3.17	3.31	3.73	3.82	3.92	4.01
Peak Flow (PF In MGD)							
Day	PDF	5.97	6.16	6.75	6.89	7.03	7.16
Instantaneous	PIF	8.47	8.72	9.47	9.65	9.83	10.01
Average Influent Loading							
TSS	lb/d	1,250	1,381	1,742	1,834	1,934	2,029
BOD_5	lb/d	1,250	1,381	1,742	1,834	1,934	2,029

Key:

ADWF = Average Dry Weather Flow
MMDWF = Maximum Month Dry Weather Flow
AWWF = Average Wet Weather Flow
MMWWF = Maximum Month Wet Weather Flow
PDF = Peak Day Flow
PIF = Peak Instantaneous Flow
TSS = Total suspended solids
BOD_5 = 5-Day Biochemical oxygen demand

1.5 Basis of Planning

1.5.1 Effluent Discharge Requirements

Current permit limits are summarized on Table 1.2.

Table 1.2: NPDES Permit Effluent Discharge Limitations

Parameter	Average Effluent Concentrations Monthly (mg/l)	Weekly (mg/l)	Monthly Average (lb/day)[a]	Weekly Average (lb/day)[a]	Daily Maximum (lbs)[a]
May 1 – October 31					
BOD_5	20	30	177	265	354
TSS	20	30	177	255	354
November 1 – April 30					
BOD_5	30	45	530	795	1060
TSS	30	45	530	795	1060
Notes	[a] The dry weather mass load limits are based on average dry weather flow of 1.06 MGD. The winter mass load limits are based on average wet weather flow of 2.12 MGD. The daily mass load limit is suspended on any day in which the flow exceeds 2.12 MGD.				

1.5.2 Water Quality Study

A Water Quality Study, completed in 2003 resulted in the following findings:

1. Outfall location and Type – Steps should be taken to improve the existing outfall. A preliminary estimate indicates that a dilution factor of 19 is possible by relocating the outfall to the bottom of the river near the bank.

2. Ammonia - With an increased dilution factor, concerns over chronic toxicity of ammonia will be reduced although monitoring or limits on ammonia could still be imposed.

3. Temperature – Based on existing data and temperature standards, there does not appear to be a temperature issue.

4. BOD Dilution – With the outfall at its present configuration, minimum dilution requirements stipulated by Oregon Administrative Rules could require a treated effluent BOD of 6.5 mg/l or less during worse case conditions. With an improved outfall, it may be possible to increase the dilution by 3-fold thereby increasing the effluent BOD limit to 19 mg/l under worse case conditions.

1.6 Alternative Evaluations and Recommendations

1.6.1 Collection System Alternatives

Based on the existing condition and future capacity requirements of the collection system, the following improvements are recommended:

- Develop and implement an annual program of smoke testing and video tape inspection of sewer lines. Prioritize and correct deficiencies in the most cost effective manner possible.
- Develop and implement a Capacity, Management, Operation, and Maintenance (cMOM) program.
- Develop, implement, and enforce a city-wide grease management ordinance.
- Reconnect appropriate businesses along 1st Street to the sewer on Front Street and decommission the existing bypass pipes to eliminate the wet-weather overflows.
- Replace the manhole at 1st Street and Stillwell Avenue.
- Construct a parallel interceptor along the south end of Evergreen Drive to Maple Lane.
- Upgrade the STEP system components in Sub Basin 5
- Replace the Southside Interceptor
- Upgrade the 12th Street Pump Station
- Replace the 12th Street Interceptor
- Construct the Marlof and Brookfield Interceptor to serve the Fairview Area.

1.6.2 Wastewater Treatment Plant Alternatives

The following seven secondary treatment alternatives were evaluated:

1. Selector Activated Sludge Basin in parallel with existing RBCs (SAS/RBC)
2. Biotower before existing RBCs (BT/RBC)
3. Oxidation Ditch without Primary Clarification (OXDnoPC)
4. Additional RBCs with replacement of existing RBC equipment (RBC/RBC)
5. Oxidation Ditch with Primary Clarification (OXDwPC)
6. Sequencing Batch Reactors with Primary Clarification (SBRs)

7. Nitrifying Selector Activated Sludge Basin with Primary Clarification (NSAS)

1.6.2.1 Apparent Best Process

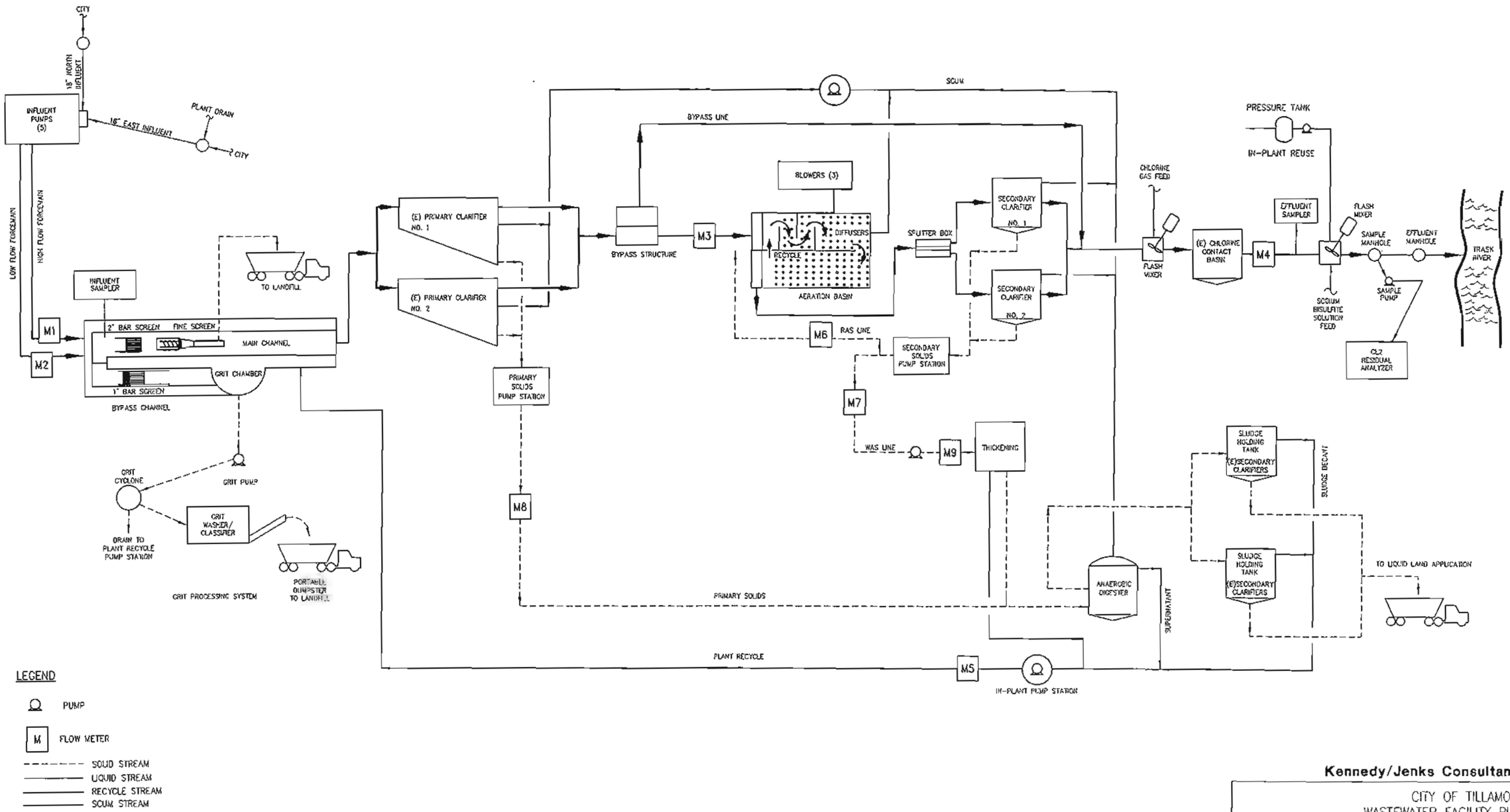
The Apparent Best Process is based on the Nitrifying Selector Activated Sludge (NSAS) treatment alternative. The recommendations for expansion of the WWTP include the following elements:

- New Influent Pump Station
- Headworks Improvements including a new influent screen and bypass channel
- New Activated Sludge Process – including:
  - Flexible Aeration Basin with dual parallel tanks
  - Selector zones for control of filamentous sludge
  - Diffused aeration equipment and blowers
  - Maximum side water depth to minimize land area requirements
- Two new Secondary Clarifiers with a new RAS and WAS pumping facility
- New chlorine contact tank with dual basins and chemical flash mixers
- Reconstructed submerged effluent outfall
- New Solids Handling Facility including a mechanical thickening device, a new anaerobic digester, a septage receiving station, and conversion of existing basins for sludge storage
- New Sludge Hauling Truck
- Consideration of a City-owned site for land application of Class B biosolids

A flow schematic of the Apparent Best Process based WWTP upgrade is shown on Figure 1.1.

1.7 Capital Improvement Plan

Recommendations for the wastewater collection system and WWTP improvements were compiled into the CIP for a three-phase project as shown in Table 1.3. Timing for the construction of each phase is estimated to occur in approximately five-year increments.



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CITY OF TILLAMOOK
WASTEWATER FACILITY PLAN

SIMPLIFIED FLOW SCHEMATIC OF
APPARENT BEST UPGRADE

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FIGURE 1.1

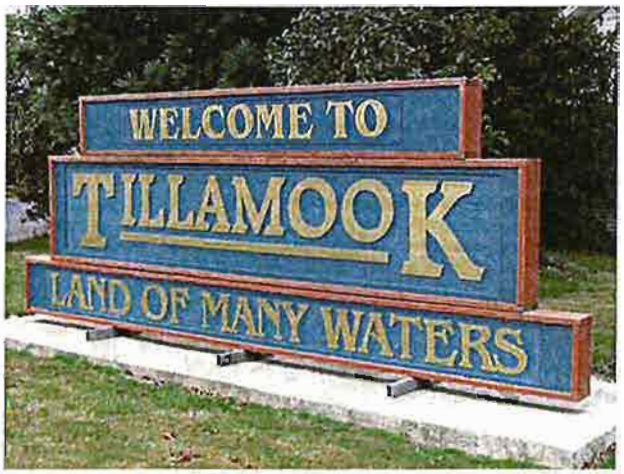
Table 1.3 – Estimated Costs of Programs and Improvements

Item	Description	Phase 1	Phase 2	Phase 3	Total
	Collection System				
I1	Smoke Testing and Video Program	$30,000	$30,000	$30,000	$180,000
I2	cMOM Program		$30,000	$30,000	$60,000
I3	1st Street Sewer Overflows	$3,000			$3,000
I5	Upgrade STEP systems in Sub Basin 5	$130,000	$130,000	$130,000	$390,000
I6	Construct a new Southside Interceptor		$790,000	$790,000	$1,580,000
I7	Construct a new 12th Street Interceptor			$1,000,000	$1,000,000
I8	Construct Interceptor to Fairview Area		$1,660,000		$1,660,000
I9	Upsize Evergreen Drive Interceptor and Replace Manhole at 1st St. and Stillwell Avenue	$240,000			$240,000
I10	Upgrade 12th Street Pump Station and Forcemain	$75,000	$165,000	$165,000	$405,000
	Wastewater Treatment Plant				
IPS2	Submersible Influent Pump Station	$1,341,000			$1,341,000
HW1	Headworks Improvements	$231,000			$231,000
T7	Secondary Treatment w/ Nitrifying Selector Activated Sludge	$3,045,000			$3,045,000
D1	Chlorine Disinfection Improvements	$1,000,000	$195,000	$195,000	$1,390,000
E1	Outfall Improvements	$55,000			$55,000
S4	Anaerobic Digestion with RAS Thickening and Converted Sludge Storage	$2,963,000			$2,963,000
SP1	Site Work/Yard Piping	$250,000			$250,000
SP2	Site Electrical Upgrades	$100,000			$100,000
SW1	Standby Power	$315,000			$315,000
LA1	Purchase of North Site and Related Costs	$245,000			
LA2	Biosolids Land Purchase		$600,000		
	Total Estimated Project Costs	**$10,023,000**	**$4,405,000**	**$1,145,000**	**$15,573,000**

Section 2: Introduction and Purpose

2.1 Introduction

The City is located on Tillamook Bay (Bay) along the Oregon coast. The Bay, one of the largest on the Oregon coast, is fed by five Coastal Mountain streams. The City is situated between two of the streams – the Trask and Wilson rivers. In addition, the Hoquarten, Dougherty, and Hall Sloughs run through town on their way to the Bay. It is truly a land of many waters.



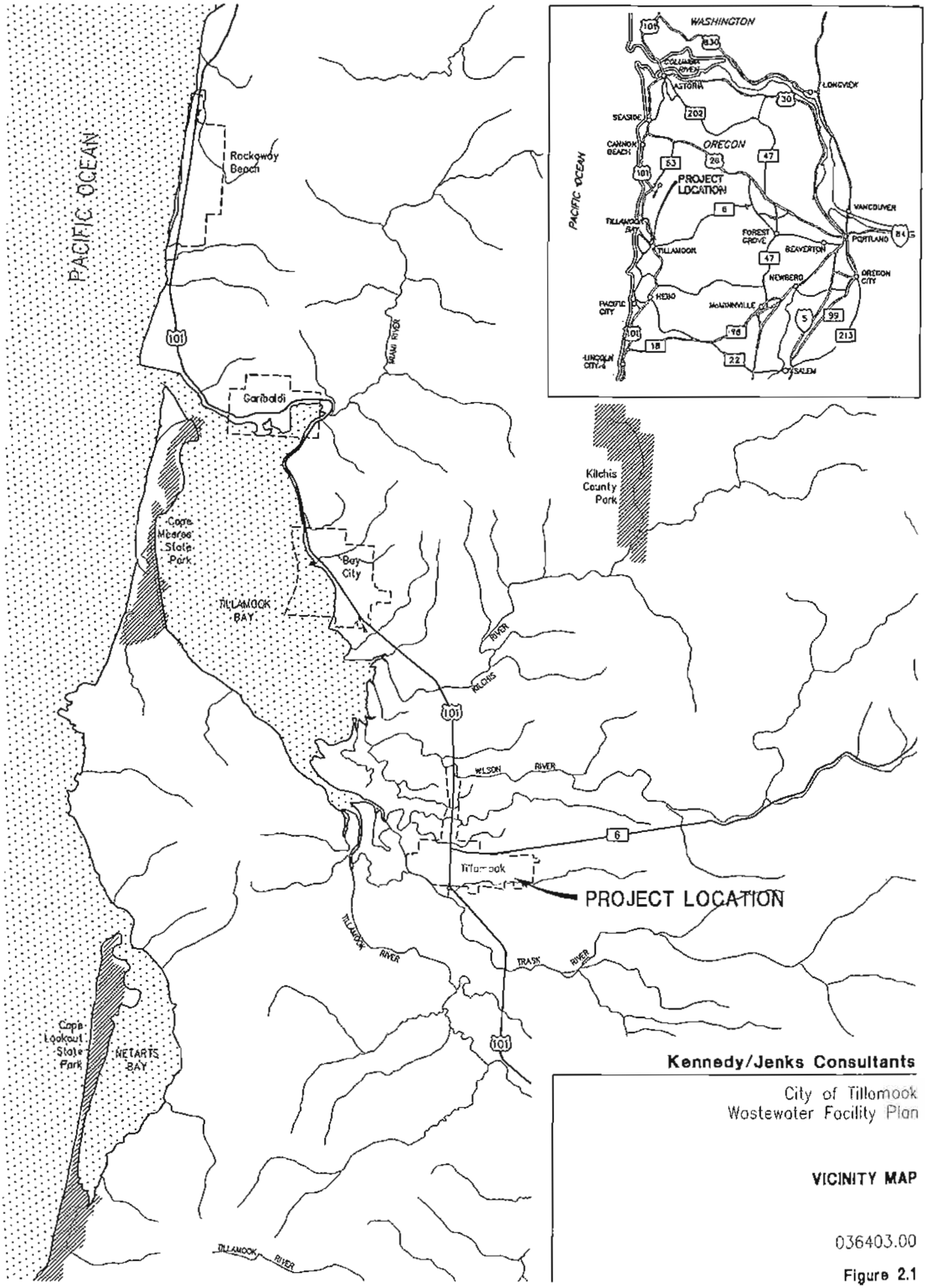
Nearby communities include Bay City, Garibaldi, and Rockaway Beach to the north and Oceanside to the southwest. The City, Bay, and vicinity are shown on Figure 2.1.

The City is well known for the local Tillamook Cheese Factory, an international manufacturer and distributor of locally made cheese. The City is surrounded by farm and pasture land, much of which supports the dairy industry. The City also has one working lumber mill and a number of businesses and shops that provide visitor services along the Pacific Coast Highway (Highway 101). The economics of the town are supported primarily by these industries along with fishing and oyster harvesting in the Bay. Oyster harvesting has been a growing industry in recent years.

2.2 Purpose

In October 2002, the City hired Kennedy/Jenks Consultants to prepare a Wastewater Facilities Plan as part of the City's Mutual Agreement and Order (MAO) with DEQ. The Wastewater Facilities Plan will address the current operational and maintenance deficiencies in the system, as well as evaluate the need for overall capacity improvements to accommodate future growth. It is intended, in format and content, to comply with the general requirements of DEQ for Wastewater Facilities Plans and with the State and DEQ requirements for Facilities Plans funded by the SRF.

This Facilities Plan will incorporate land use and population data consistent with the adopted City Comprehensive Plan. The planning interval is a 20-year period.



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City of Tillamook
Wastewater Facility Plan

VICINITY MAP

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Figure 2.1

Section 3: Study Area Characteristics

3.1 Planning Area

The City is located in Tillamook County about 10 miles east of the Pacific Ocean and two miles southeast of the Bay. The City is about 70 driving miles from Portland and is served by Highway 6 from the east and Highway 101 in the north-south directions. The planning area, characterized as coastal plain, is generally flat and gently slopes upward towards the east. It is surrounded by the forested foothills of the Coastal Range Mountains which rise abruptly at the edge of the plain and are characterized by rugged ridges and steep-sided canyons. An opening to the northwest provides access to the Bay. Dairy fields and forestland surround the City's Urban Growth Boundary (UGB). The regional area is shown on Figure 2.1. The planning area, which includes the UGB, is shown on Figure 3.1.

3.1.1 Climate and Rainfall

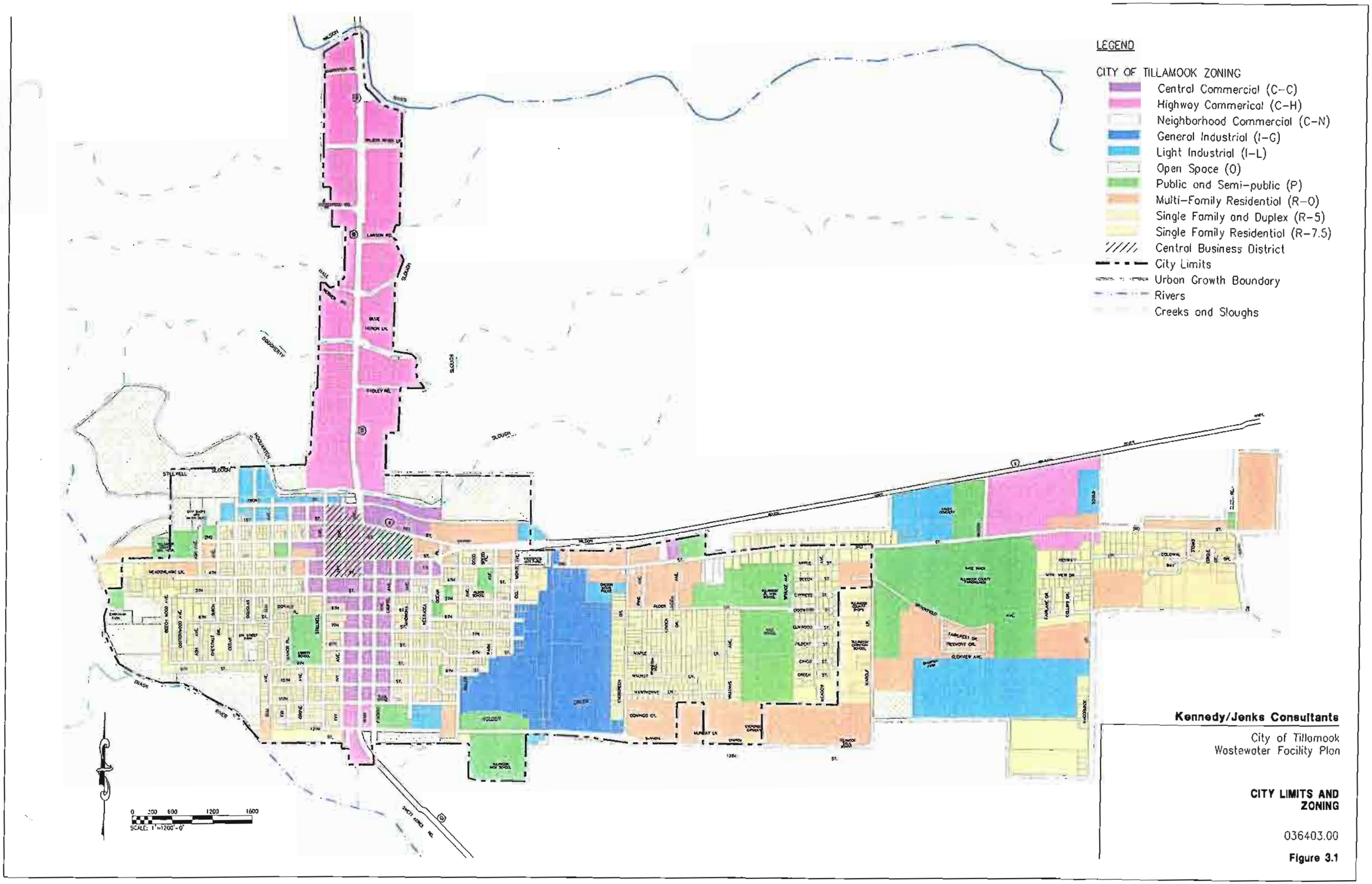
Predominately westerly winds moderate the marine climate of the Tillamook area resulting in generally cool summers and mild winters. Proximity to the Pacific Ocean results in relatively high humidity and an average frost-free season of 182 days. The warmest month is typically September while the coldest temperatures normally occur in January. Average annual precipitation varies from about 90 inches in the planning area with up to 150 inches in the higher areas to the east of the city. On average, more than 70 percent of the annual rainfall occurs from November to March. December is normally the wettest month while July is typically the driest month. Storm duration in winter months varies from less than a day to more than a week with precipitation totals frequently exceeding 10-inches.

3.1.2 Surface Waters

The Tillamook Bay drains an area of approximately 540 square miles. The five principal rivers draining into the Bay are the Miami, Kilchis, Wilson, Trask, and the Tillamook with the Wilson and the Trask rivers predominating in discharge volume. Within the area surrounding the study area are the floodplains and lower reaches of the Wilson, Trask, and Tillamook rivers. The Tillamook WWTP discharges directly to the Trask River, about two miles upstream from the Bay. Tidal action influences the lower reaches of the Trask River, and river flows will reverse directions with tidal changes at the point where the WWTP discharges. U.S. Geological Survey (USGS) Streamflow Statistics indicate that the average discharge rate for the Trask River is about 280 cubic feet per second (cfs) from May to October and roughly 1,680 cfs from November to April with the highest discharge occurring in January and the lowest in August. Floods in the study area historically occur from October through April with the period from December to February having the highest occurrence.

3.1.3 Soil, Geology, and Habitat

The City, surrounded by agricultural land primarily in soil classes II through IV, is committed to preserving as much agricultural land as possible. Current ordinances require a rigorous review process before incorporating new land into the urban growth boundary. The geology of the City is typical for a coastal community, with the coastal range to the east and significant estuary



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City of Tillamook
Wastewater Facility Plan

CITY LIMITS AND ZONING

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Figure 3.1

lands where the rivers meet the Pacific Ocean. Most of the city is no more than 20 feet above mean sea level. Given the high rainfall in the area, the City's low elevation and the types of soils present, it is not surprising that the soils often reach saturation, causing significant local flooding.

The Tillamook Bay area, particularly just uphill from the City, is home to a large population of wild elk. The nearby forests also support a wide variety of other animals and plants. Due to the large local dairy industry, cattle are the most common animal seen in the area. Native fauna for the areas immediately adjacent to the City includes primarily wetland grasses and shrubs.

3.1.4 Air Quality

In general, the air quality in the City is adequate. The odors from the local dairies can be significant on days with little to no wind.

3.1.5 Public Health Hazards

The area has few, if any, general health hazards. Recently however, Tillamook County (County) has done an evaluation of the septic tanks to the east of the City in the area commonly called Fairview. Their assessment indicated that the septic tanks in Fairview are not working adequately, particularly when the groundwater is high. The unofficial County response is that these septic tanks will need to be connected to the sewer system in the near future. If immediate connection to the sewer system becomes necessary, the County can mandate a Health Hazard Annexation, requiring the systems to be connected to the City sewer immediately.

3.2 Land Use

3.2.1 Zoning

Zoning designation, as specified in the City of Tillamook Comprehensive Plan (2001Comp Plan), regulates land use for the study area. A land use map, shown on Figure 3.1, delineates the city limits and the UGB and shows the zoning designations for the area. Table 3.1 summarizes the current land use categories within the city limits and Table 3.2 shows the land use summary for the area within the UGB lying outside the city limits [called the Urban Growth Area (UGA) for the remainder of this report]. The Comp Plan indicates that within the city limits, over 80 percent of the developable lands have been utilized since the early 1970s. The primary avenue to future growth within the current city limits would be to convert lots occupied by single family dwellings to multi-family dwellings and by development of vacant single family and duplex lots. The highway/commercial zone north of the Hoquarten Slough, which was incorporated by the City in 1982, has available land area for additional growth; but, flood plain designation by the Federal Emergency Management Agency (FEMA) and the recent determination of much of this area as jurisdictional wetland makes significant additional development unlikely.

The UGA lies almost entirely within the eastern portion of the UGB. The greatest potential for growth within the UGB is within the UGA as over 2,000 multi and single family lots are available for development. The current trend in development within the UGA indicates that growth in this area is expected to continue. According to the Comp Plan, expansion to the north and south of

the UGB is not expected, as these areas are productive farmlands that contribute to the economic health and stability of Tillamook County (County).

Most of the existing businesses and residences within the eastern portion of the UGA are served by the Fairview Water District (Fairview District). According to planning documents for the Fairview District [*Fairview Water District Master Plan* (LDC Design Group 2001)] approximately 75 percent of the anticipated buildout population served by this water district will reside within the current UGA.

3.2.2 Wastewater System Expansion

While the population within the city limits is expected to grow at a moderate rate, this growth would likely result from an increase in population density and would primarily be incorporated by the existing collection system. Any significant expansion of the service area over the planning period is anticipated to occur within the current UGA. Eastward expansion beyond the limits of the UGA would be possible; however, there are no areas currently anticipated for annexation within the planning period. The County has plans to undertake a sanitary survey to determine if untreated sewage from septic tanks located within the UGA poses a risk to human health and the environment. Pending the results of this survey, incorporation of nearly 400 existing houses and businesses could occur as the collection system is expanded and treatment capacity becomes available. Of the nearly 500 existing commercial and residential units within the UGA, approximately 100 units are currently served by the Tillamook wastewater system.

3.2.3 Buildout Density

While it is unlikely that buildout density will be approached within the UGB during the planning period, Table 3.1 and Table 3.2 present the potential commercial and residential distribution for the current city limits and the UGA, respectively.

Table 3.1: City of Tillamook Zoning Characteristics

Zone	Designation	Total Acres	Developed Lots	Potential Lots	% of Lots Developed
C-C	Central Commercial	47.9	220	9	96%
C-H	Highway Commercial	120.4	59	18	77%
C-N	Neighborhood Commercial	5.2	10	0	100%
I-G	General Industrial	85.6	30	1	97%
I-L	Light Industrial	10.6	26	4	87%
OS	Open Space	38.1	7	0	100%
P&S-P	Public and Semi-Public	87.3	27	0	100%
R-0	Multi-Family Residential	69.6	397	937	30%
R-5	Single Family and Duplex	78.1	547	114	83%

Zone	Designation	Total Acres	Developed Lots	Potential Lots	% of Lots Developed
R-7.5	Single Family Residential	146.4	732	125	85%
Total Acres		**689**	**Total Area Developed**		**83%**

Table 3.2: UGA Zoning Characteristics

Zone	Designation	Total Acres	Developed Lots	Potential Lots	% of Lots Developed
C-H	Highway Commercial	32.7	9	2	82%
I-L	Light Industrial	73.5	6	2	75%
OS	Open Space	65.6	1	0	100%
P&S-P	Public and Semi-Public	93.0	5	0	10%
R-O	Multi-Family Residential	71.8	221	1,489	13%
R-5	Single Family and Duplex	42.6	57	273	17%
R-7.5	Single Family Residential	90.0	110	344	24%
Total Acres		**467**	**Total Area Developed**		**59%**

3.3 Population

3.3.1 Existing Population

There are approximately 1,725 residential and commercial customers currently connected to the wastewater system serving an estimated population of 4,475. Approximately 1,625 connections are located within the city limits with 100 additional connections located within the UGA. Approximately 70 new residential connections were expected to connect within the UGA in late 2003 or early 2004. With the addition of the 70 new units, the estimated population served by the wastewater system would be 4,650 (based on 1990 Tillamook County Census value of 2.52 persons per typical dwelling unit). The historical population for the Tillamook city limits is shown on Table 3.3.

Table 3.3: City of Tillamook Historical Population

Year	1970	1980	1990	2003
Population	3,968	3,981	4,006	4,352

Note:
Source: U.S. Census 1980, 1990

3.3.2 Population Projections

Assuming an agreement is established between the County and UGA residents, 400 existing units in the UGA would be connected to the wastewater system as the collection system is expanded and treatment plant capacity is increased. Although it is not possible to predict the future, it is logical to assume that these customers will be brought online as concerns regarding fecal contamination of surface and ground water increase. Providing sewer service to these customers is also consistent with the *Comprehensive Conservation and Management Plan for Tillamook Bay* (1999). For planning purposes, it will be assumed that these customers are fully incorporated into the wastewater system by 2010. Applying a rate of 2.52 persons per household would result in a population addition of 1,008.

The Tillamook County Department of Community Development (TCDCD) has compiled population data generated by the Office of Economic Analysis, Department of Administration Services for 1980, 1990, and 2000 and projected this data forward through 2040. According to the projections, the population would increase to 5,760 within the current city limits by the year 2025. The TCDCD does not provide a population projection for the UGA so the corresponding annual growth rate applied to the city limits was applied to the UGA from the years 2010 to 2025.

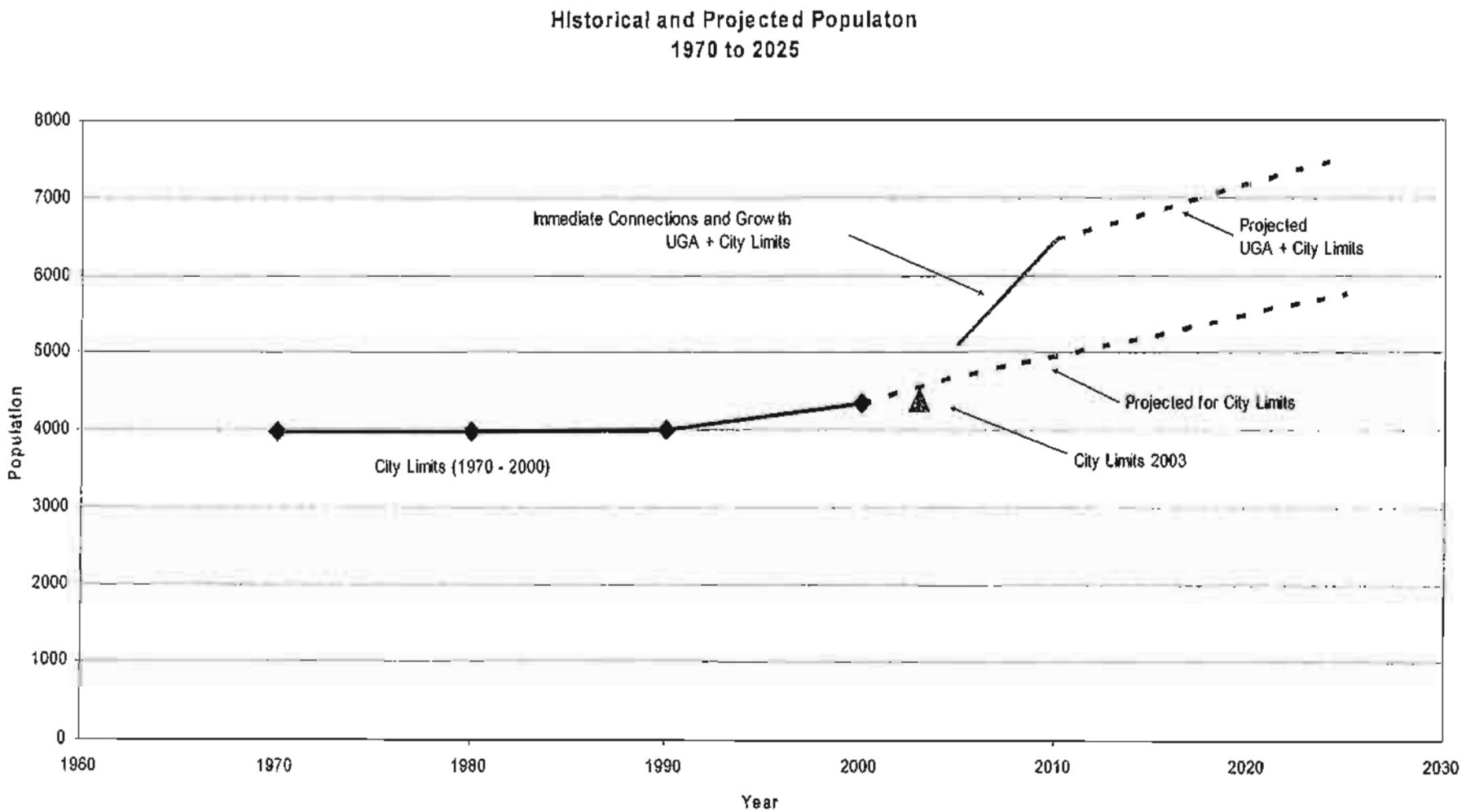
Table 3.4 summarizes the current and projected population for the current service area and anticipated expansion area. The projection corresponding to information presented in the table is shown graphically on Figure 3.2.

Table 3.4: Current and Projected Population (connected to wastewater system)

Year	City Limits	Growth Rate[a]	UGA	Growth Rate[b]	Total
2003[c]	4,375[d]	NA	250[e]	NA	4,625
2005	4,690[f]	NA	426[g]	NA	5,116
2010	4,940	1.05%	1,510[h]	1.05%	6,450
2015	5,205	1.06%	1,590	1.06%	6,795
2020	5,490	1.06%	1,675	1.06%	7,165
2025[i]	5,760	0.96%	1,755	0.96%	7,515

Notes:

(a) Growth rate projections from Office of Economic Analysis, Department of Administrative Services for years 2005 to 2010 (uses "Low Growth" estimate).
(b) Growth projections not available for UGA. Uses corresponding "City Limits" growth rates for years 2005 to 2025.
(c) Start of planning period.
(d) Population estimate from City of Tillamook Planning Department.
(e) Includes 100 current connections to the wastewater system.
(f) Projected population estimate from Office of Economic Analysis, Department of Administrative Services.
(g) Total reflects the existing 250 persons within the UGA, plus 70 new connections planned for 2003/2004 (70 new connections x 2.52 persons per new connection).
(h) Total reflects (existing 426 persons within the UGA, plus 400 existing connections previously using septic tanks x 2.52 persons per connection) x 1.05 percent growth rate.
(i) End of planning period.



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City of Tillamook Wastewater Facility Plan

FIGURE 3.2

Section 4: Existing Collection System

4.1 Service Area

The existing Tillamook wastewater service area is primarily contained within the incorporated City limits. The easternmost extension of the sewer system lies outside the City limits but within the Urban Growth Boundary (UGB) in the area referred to as the Urban Growth Area (UGA). The service area gently slopes downward from east to west and from north to south. Most of the existing sewer system was constructed at the minimum hydraulic grade due to the shallow grade.

The system serves approximately 4,600 residents and includes dense downtown residential/commercial development, a highway commercial area north of Hoquarten Slough, and mixed residential and commercial development east of downtown. There is a limited industrial wastewater contribution from Publisher's sawmill facility.

The most recent expansion of the service area has occurred toward the east and includes residential and commercial customers along 3rd Street and south of the Fairgrounds. Some of these customers are directly connected to the gravity line along 3rd Street. The remaining customers rely on the Brookfield Pump Station to lift sewage to the gravity line on 3rd Street. Failure of onsite septic systems in the UGA has prompted recent discussions towards extending wastewater service to about 400 existing customers in this area. Additional new single family, multi-family, and commercial units are anticipated for this area over the planning period. Table 4.1 shows the areas of the UGB, the incorporated city limits, and the UGA.

Table 4.1: UGB, UGA, and the Incorporated City Limits

Area	(Acres)	Notes
Urban Growth Boundary	1,156	(a)
Incorporated City Limits	689	(b)
Urban Growth Area	467	(c)

Notes:

(a) The Urban Growth Boundary (UGB) includes the incorporated city limits plus the Urban Growth Area (UGA).
(b) The entire incorporated area is currently served by the Tillamook wastewater system.
(c) 467 acres within the UGB lie outside the city limits in the UGA. Approximately 20 percent of existing residential and commercial units in this area are currently served by the Tillamook Wastewater system. This is the area of highest anticipated growth during the planning period.

4.2 System Description

For purposes of this report, the existing collection system was divided into five sub-basins based on the drainage areas served. Flow from the five sub-basins is ultimately conveyed to the treatment plant through one of two interceptor sewers referred to as the 4th Street Interceptor and the Southside Interceptor. A third interceptor, the Northside Interceptor, conveys flow from Sub Basin 5 to the 4th Street Interceptor. Two additional interceptors, the 12th Street Interceptor and the Eastside Interceptor, convey flow from the eastern portion of the

collection system to the 12th Street Pump Station. The 12th Street Pump Station discharges to the Southside Interceptor via a 12-inch forcemain. The sub-basins and major conveyances are shown on Figure 4.1 and discussed below. A more detailed sewer map, showing individual pipes, manholes, and other related features is presented as Figure 4.2. An enlarged reproduction of this figure is included at the back of this report.

4.3 Sub Basins

4.3.1 Sub Basin 1

Year Built:	1920 (approximate)
Pipe Material:	Clay
Linear Feet of Pipe:	37,500 (approximate)
Number of Manholes:	150 (approximate)
Conveyance:	4th Street Interceptor
Pump Stations:	Front Street Pump Station

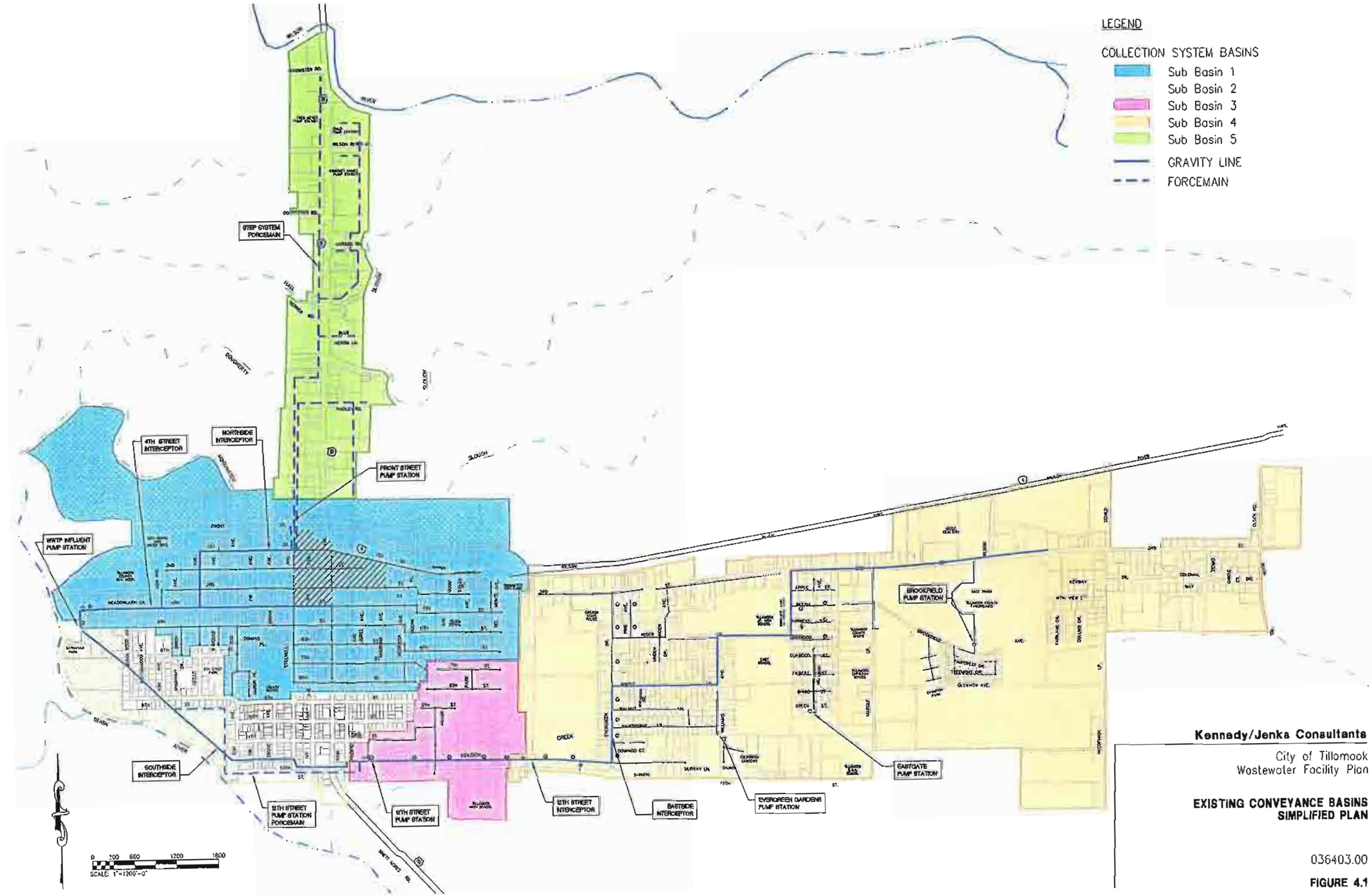
Sub Basin 1 serves roughly 215 acres in the central downtown area and consists primarily of 8-inch collection pipes. The majority of pipes in this sub basin were constructed around 1920 and constitute the oldest portion of the City's collection system. A run of pipe along the west end of 4th Street, which serves as the major conveyance interceptor for this sub basin, consists of 16-inch and 12-inch diameter pipes and is referred to as the 4th Street Interceptor. The run of pipe between the manholes at 1st Street/Stillwell Avenue and 4th Street/Cedar Avenue, which serves to convey the flow from Sub Basin 5 to the 4th Street Interceptor, is referred to as the Northside Interceptor. The Front Street Pump Station, which serves a small, low-laying area along Front Street, discharges to a nearby manhole at 1st Street/Stillwell Avenue. This manhole also receives effluent from septic tank effluent pump (STEP) systems originating from the Sub Basin 5 via pressure mains. The combined Sub Basin 1 and Sub Basin 5 flows are conveyed by gravity to the WWTP via the 4th Street Interceptor. The Tillamook County Hospital (Hospital) is connected to 4th Street Interceptor at 3rd Street/Ash Avenue. In general, Sub Basin 1 has adequate hydraulic capacity except for the Northside and 4th Street Interceptors which are surcharged under high flow conditions. However, even under surcharged conditions, manholes along these interceptors do not overflow due to the substantial burial depth of these pipes.

Problem Areas

- The Northside Interceptor, which serves as the gravity conveyance for flows from Sub Basin 5, surcharges during periods of high flow. When this occurs, sewage can back up into two businesses located north of the intersection at 1st Street and Stillwell Avenue.

- Grease accumulation from area restaurants result in clogged lines in the downtown commercial district requiring periodic cleaning by treatment plant staff.

- The manhole at 1st Street/Stillwell Avenue, which accepts effluent from Sub Basin 5, has ongoing decomposition and odor issues as flow from Sub Basin 5 tends to be septic and has elevated hydrogen sulfide levels. Field Investigations reveal that the crown of



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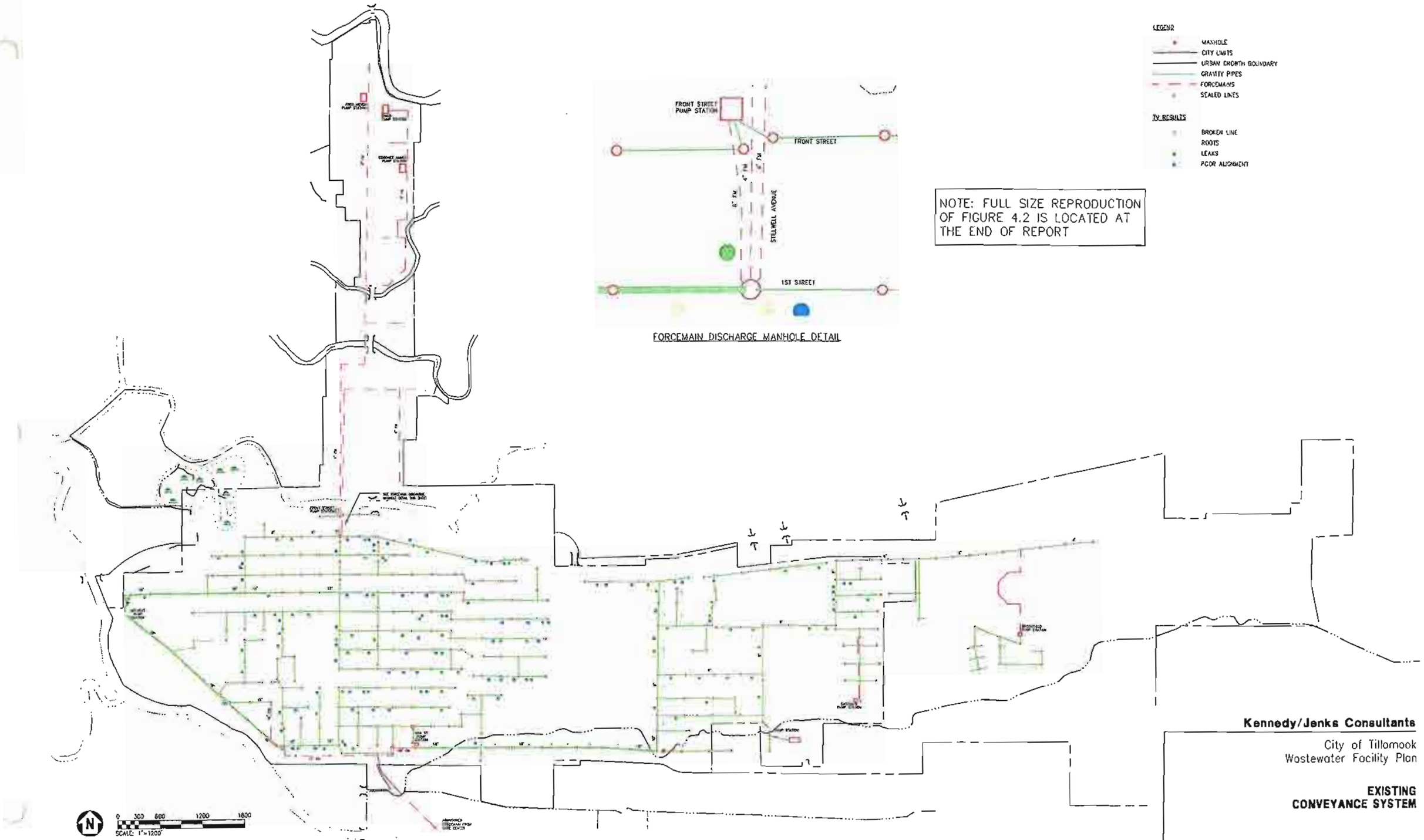
City of Tillamook
Wastewater Facility Plan

EXISTING CONVEYANCE BASINS SIMPLIFIED PLAN

036403.00

FIGURE 4.1



Kennedy/Jenks Consultants

City of Tillamook
Wastewater Facility Plan

EXISTING CONVEYANCE SYSTEM

036403.00

FIGURE 4.2

this manhole may be lacking structural integrity. The City has had plans and specifications prepared to replace this manhole in Fall 2005.

- Pipe alignment problems, leaky pipes/manholes, and root infestations of pipes are present throughout the sub basin. Figure 4.2 shows the results of television camera investigations documenting these problems. The closed-circuit television investigations were conducted from 1973 to 1996.

4.3.2 Sub Basin 2

Year Built:	1942 (approximate)
Pipe Material:	Clay
Linear Feet of Pipe:	13,000 (approximate)
Number of Manholes:	55 (approximate)
Conveyance:	South Interceptor
Pump Stations:	None

Sub Basin 2 was constructed around 1942 and serves approximately 70 acres in the southwest portion of the city. The system consists primarily of 8-inch collection sewer but this sub basin also incorporates the Southside Interceptor that consists of 12-inch, 15-inch, and 18-inch gravity pipe. Sub Basin 2 is connected to the Southside Interceptor at six places. Approximately 800 feet of 15-inch gravity pipe was added to Sub System 2 in 1975 to serve a residential area south of 9th Street and to accept flow from the 12th Street Pump Station via a force main extension constructed to alleviate flow restrictions along 12th Street. This 15-inch pipe discharges to the Southside Interceptor south of Chestnut Drive where the interceptor transitions from 15-inch to 18-inch diameter. In general, Sub Basin 2 has adequate hydraulic capacity with the exception of the Southside Interceptor which is discussed in greater detail in a subsequent section of this report. Most, if not all, of the hydraulic deficiencies in Sub System 2 result from hydraulic restrictions in the Southside Interceptor.

Problem Areas

- The 15-inch gravity pipe constructed in 1975 has overflowed at manholes during periods when the seasonal pond is high (see discussion on the Southside Interceptor) and the 12th Street pump station is discharging. This is due to restricted flow in the Southside Interceptor.

- Pipe alignment problems, leaky pipes/manholes, and root infestations of pipes are present throughout the sub basin. Figure 4.2 shows the results of television camera investigations documenting these problems. The closed-circuit television investigations were conducted from 1973 to 1996.

4.3.3 Sub Basin 3

Year Built:	1947 (approximate)
Pipe Material:	Clay
Linear Feet of Pipe:	6,000 (approximate)

Number of Manholes:	30 (approximate)
Conveyance:	12th Street Pump Station to South Interceptor
Pump Stations:	12th Street Pump Station

Sub Basin 3 consists primarily of 8-inch collection sewer constructed around 1947 and serves an area of approximately 40 acres. Sewage originating from this sub system flows by gravity to the 12th Street Pump Station located within the Tillamook County Public Utility District (PUD) motor pool yard. The current 12th Street Pump Station, built in 1957, replaced an earlier pump station and discharges to the Southside Interceptor via 2,900 lineal feet of 12-inch force main. The 12th Street Pump Station is discussed in greater detail later in this section. In general, this sub basin has adequate hydraulic capacity.

Problem Areas

- Sub Basin 3 is relatively small in the total area served and does not exhibit surcharging or overflowing of manholes. However, closed-circuit television camera investigations have documented poor pipe alignment, leaky pipes/manholes, and root infestation of pipes in this sub basin. Results of the television inspections made between 1975 and 1996 are shown on Figure 4.2.

4.3.4 Sub Basin 4

Year Built:	1958 through 1988
Pipe Material:	Concrete/PVC
Linear Feet of Pipe:	36,500 (approximate)
Number of Manholes:	110 (approximate)
Conveyance:	Eastside Interceptor to 12th Street Interceptor to 12th Street Pump Station
Pump Stations:	Eastgate, Brookfield, and Evergreen Gardens

Sub Basin 4 lies east of the railroad tracks bisecting the sawmill property and consists of about 500 acres of developed area. The pipes in this basin are primarily 8-inch collector pipes with about 1,000 feet of 10-inch gravity pipe along Evergreen Drive. This sub basin incorporates part of the 12th Street Interceptor as well as the Eastside Interceptor. The 12th Street Interceptor is an 18-inch reinforced concrete pipe running between the 12th Street Pump Station and the South end of Evergreen Drive. It lies under the saw mill sorting yard for about one third of its length. The Eastside Interceptor consists of 10-inch and 8-inch collector pipes that serve as the primary conveyance conduit through the sub basin. The original portion of this sub basin was constructed in 1958 with additional branches added to service expansion areas. The most recent addition was constructed in 1988 and runs east along 3rd Street just beyond Wilson River Loop. There are three small pump stations located within this sub basin serving low laying developments. These pump stations discharge directly to nearby gravity pipes at unknown locations (general location is known but exact location is not documented). Generally speaking, there is insufficient capacity in the Eastside Interceptor to convey existing peak flows. Expansion of the collection system in the UGA will require improved conveyance through Sub Basin 4.

Problem Areas

- Sub Basin 4 has several manholes that overflow during high flow conditions. These manholes are located along the south end of Evergreen Drive, at the east end of Maple Lane, midway along Meadow Avenue, and at the western most manhole on the 5-Rivers property. These overflows are related to insufficient capacity of the Eastside Interceptor. The overflowing manhole on Meadow Avenue occurs when the system is surcharged and the Eastgate Pump Station is called to run.

- Surcharging of manholes to varying degrees is exhibited under high flow conditions along the south side of Evergreen Drive. On occasion, this has resulted in sewage backing up into adjacent low laying homes along this route.

- Pipe alignment problems, leaky pipes/manholes, and root infestations of pipes are present throughout the sub basin. Figure 4.2 shows the results of television camera investigations documenting these problems. The closed-circuit television investigations were conducted from 1973 to 1996.

4.3.5 Sub Basin 5

Year Built:	1977 with later expansions
Pipe Material:	PVC (forcemain)
Linear Feet of Pipe:	12,000 (approximate)
Number of STEP Tanks:	60 (approximate)
Number of STEP Pumps:	60 (approximate)
Pump Stations:	4 Duplex Submersible (Fred Meyers, Coronet Plaza, and 2 at Shilo Inn)
Conveyance:	Force main to Sub Basin 1

Sub Basin 5 is comprised of individual pumping systems serving customers along Highway 101 between Hoquarten Slough and the Wilson River. This sub basin is low-lying and flat, making it unfavorable for gravity collection and prone to periodic flooding during heavy winter rainfall events.

Residences and commercial establishments in this sub basin were originally served by septic tanks prior to annexation by the City. Following annexation, these septic tanks were converted to STEP systems by installing submersible pumps and level controls. The individual STEP systems are connected to one of two 4-inch pressure mains allowing the liquid portion of the waste to be pumped to the treatment plant. Portions of the original STEP system underwent upgrades and expansion in 1998. Approximately 90% of the septic tanks are of fiberglass construction with the remaining being concrete. Most of the STEP tanks have the pumps mounted in the upper zone of the tanks but some of the pumps sit on the floor of the tanks allowing solids to be pumped off the bottom of the tanks. A recent inventory of the individual STEP system equipment is included in Appendix J.

In addition to the STEP systems, three duplex submersible pump stations serve three businesses in this area (Fred Meyer, Coronet Annex and Shilo Inn). These stations pump whole, raw sewage. The pumps at Shilo Inn are grinder pumps.

Both pressure mains serving Sub Basin 5 discharge to a manhole at 1st Street/Stillwell Avenue located in Sub Basin 1. Ultimately, this effluent/sewage is conveyed to the WWTP via the Northside and 4th Street Interceptors.

Problem Areas

- The forcemains serving this basin have no cleanouts or provision for cleaning.
- According to City staff, several of the septic tanks have broken bottoms resulting from pumping the tanks during periods of high groundwater.
- This sub basin has substantial problems with grease buildup in the STEP tanks and wet wells as many of the commercial establishments are involved in food preparation. Existing grease traps are generally ineffective or not operated properly.
- Hydrogen sulfide production in the septic tanks and force mains results in a corrosive environment and odor problems at the discharge point of the force main.
- Maintenance of individual STEP system pumps and controls is intensive, as the equipment is old and prone to frequent breakdown.
- A variety of STEP system pumps are currently in service which causes maintenance problems with regard to stocking spare pumps and parts. Power supply varies from location to location. Some areas are served by 3-phase power and others are served by single phase power.
- Flooding and high groundwater on a periodic basis can inundate individual STEP systems. Inflow and infiltration (I/I) from lateral connections to the STEP tanks is likely present during times of high groundwater although it has never been quantified.

4.4 Interceptors / Step System Forcemain

4.4.1 4th Street Interceptor

The 4th Street Interceptor was built in 1920 to convey sewage from Sub Basin 1 and terminates at the WWTP influent pump station. Primarily, it runs along 4th Street and receives flow from Sub Basin 1 and Sub Basin 5. This interceptor is 18 inches in diameter between the west end of 4th Street and the WWTP then reduces to 16 inches as it turns east along 4th Street to Cedar Avenue. Between Cedar Avenue and Stillwell Avenue, the pipe is 12 inches reducing to an 8-inch pipe at Stillwell Avenue. Except for the 18-inch diameter section, this pipe receives flow along its length from lateral connections. Record drawings for these pipes are not available. For purposes of analysis, it is assumed that the interceptor was constructed at a 0.4 percent slope, which is typical for most of the pipes within the city limits. Based on Manning's equation, the full-flow, non-surcharged capacity of this interceptor is about 4 MGD, 3 MGD, and 1.3 MGD for the 18-inch, 16-inch, and 12-inch pipe sections respectively. Additional capacity is possible under surcharged conditions. There are no known overflowing manholes or excessively surcharged manholes along this interceptor.

Problem Areas

- Pipe alignment problems, leaky pipes/manholes, and root infestations of pipes are present along this interceptor. Figure 4.2 shows the results of closed-circuit television camera investigations documenting these problems. The closed-circuit television investigations were conducted from 1973 to 1996.

4.4.2 Northside Interceptor

The Northside Interceptor is a run of 8-inch collector pipe in Sub Basin 1 but is identified in this report as an interceptor because it now conveys flow from Sub Basin 5. It runs from the manhole located at 1st Street/Stillwell Avenue to the manhole at 4th Street/Cedar Avenue. Assuming that this pipe was laid at a 0.4 percent slope, which is typical for pipes in the city limits, it would have a full flow, non-surcharged capacity of approximately 0.5 MGD. Higher capacities are possible under surcharged conditions. Of particular interest are two low-lying businesses along 1st Street which are served by this interceptor. These businesses have experienced overflows of interior drains and fixtures during high flow events. These businesses should have been connected to the collector pipe along Front Street as this pipe is several feet lower than the collector pipe on 1st Street.

Problem Areas

- An open cleanout along 1st Street currently allows the interceptor to discharge onto Front Street (via a drainpipe routed between 1st Street and Front Street) under surcharge conditions. This pipe was installed to alleviate backup of sewage into the businesses located between 1st Street and Front Street.

4.4.3 Southside Interceptor

The Southside Interceptor was constructed in 1947 to convey sewage from Sub Basin 2 to the influent pump station at the treatment plant. It currently conveys flow from Sub Basins 2, 3, and 4. As previously described, Sub Basin 4 includes land east of the existing City limits. This area will be the site of the majority of future development. Consequently a large portion of flow from new development will be routed through this interceptor. In 1975, a 15-inch gravity sewer extension was connected to the Southside Interceptor at a point close to the west end of 9th Street to serve residential lots along the south side of 9th Street and to serve as the 12th Street Pump Station discharge point. The Southside Interceptor is 15-inches in diameter at the west end of 12th Street increasing to 18 inches at the point where it receives additional flow from the 15-inch gravity extension. Record drawings indicate that the interceptor was slip-lined in 1975, which would reduce the internal diameter. A portion of this interceptor was constructed under a large, seasonal pond adjacent to the Trask River. Manholes located within the seasonal pond routinely overflow during wet weather. Record drawings show that the interceptor pipe was laid at a 0.15 percent slope indicating that the non-surcharged flow capacity (prior to slip lining) is roughly 1.6 MGD for the 15-inch section and 2.6 MGD for the 18-inch section. The actual non-surcharged capacity is likely less than this due to the decrease in diameter resulting from the slip-lining. Additional capacity is possible under surcharged conditions.

Problem Areas

- At the WWTP influent pump station, the invert of the Southside Interceptor is near the bottom of the pump station wetwell. The result is that the interceptor is in a permanently surcharged condition and the flow capacity of this pipe is reduced with an increasing water surface level in the influent pump station. During wet weather, a low laying manhole along the Southside Interceptor regularly overflows, in part due to the surcharged influent pump station.

- Calculations indicate that the interceptor is undersized even without the surcharge issues from the influent pump station.

4.4.4 12th Street Interceptor

The 12th Street Interceptor was built in 1957 to convey flow from the expansion area to the east of Evergreen Drive. It serves Sub 4. Because the largest portion of growth will occur in Sub Basin 4, this interceptor will be required to handle increasing flows in the future. This interceptor is 18-inches in diameter and runs east from the 12th Street Pump Station to the south end of Evergreen Drive. About one third of the pipe was constructed under the log storage and sorting area of the saw mill. At Evergreen Drive, the 12th Street Interceptor receives flow from a 10-inch collector pipe which constitutes the lower reach of the Eastside Interceptor. Record drawings indicate that the pipe was laid at a 0.17 percent slope. Manning's equation predicts the non-surcharged, full-pipe capacity to be approximately 2.8 MGD. Additional capacity is possible under surcharged conditions. The pipe was inspected, cleaned, and re-grouted at the joints in 1975.

Problem Areas

- Discussion with City staff indicate that past smoke testing indicated that this pipe may exhibit the potential for excessive I/I, depending on groundwater conditions. No written records of the smoke testing exist.

- It is possible that there has been settlement of the pipe through the lumber yard area, perhaps causing the leaky joints noted above.

4.4.5 Eastside Interceptor

The Eastside Interceptor is a series of collector pipes in Sub Basin 4 but is identified in this report as an interceptor because it serves as the primary conveyance path from the upper to lower reach of Sub Basin 4 as well as the conduit for flow originating within the UGA (future expansion area). For discussion purposes, the interceptor is defined as running from the manhole located at 3rd Street/Spruce Avenue to the manhole at 12th Street/Evergreen Avenue as it meanders through the Sub Basin 4. The pipes are part of the expansion of the collection system eastward and were primarily constructed in the late 1950s. Record drawings indicate that these pipes were laid at a 0.4 percent slope. Based on this slope, the capacity is about 0.9 MGD for the 10-inch stretch along Evergreen Drive and approximately 0.5 MGD for the remaining 8-inch sections. Higher capacities would be possible under surcharged conditions.

Problem Areas

- The Eastside Interceptor has several manholes that overflow during high flow conditions. These low laying manholes located along the south end of Evergreen Drive and at the intersections of Maple Street and Williams Avenue are the first to overflow. Other manholes in the same general vicinity have overflowed under extreme flow conditions.

- Surcharging of manholes to varying degrees is exhibited under high flow conditions along the Eastside Interceptor and connector pipes. On occasion, this has resulted in sewage backing up into low laying homes along this route, specifically on Evergreen Drive at Domingo Court..

4.4.6 Step System Forcemains

There are two forcemains serving the STEP systems and related conventional pump stations in Sub Basin 5. One force main serves both the east and west side of the highway north of Dougherty Slough to the Wilson River. A second forcemain serves the area on the east side of the highway south of Dougherty Slough. Both force mains are 4-inch PVC. There are no known cleanouts or other provisions provided for cleaning these forcemains. According to City staff, the force mains have never clogged or been serviced. The general location of the forcemain is shown on Figure 4.2 but the specific location of any section of pipe is not precisely known as record drawings do not exist. Both force mains cross Hoquarten Slough north of Stillwell Avenue and discharges to a manhole at 1st Street and Stillwell Avenue. This manhole shows signs of hydrogen sulfide damage and is planned to be replaced in 2005.

4.5 Pump Stations

There are five pump stations located within Sub Basins 1, 2, 3, and 4, which are described below. The Sub Basin 5 collection system, as described above, includes three small submersible stations and about 60, 1 and 2 hp STEP systems. The submersible stations are described in greater detail below. All of the pump stations in Sub Basin 5 serve private, commercial, and residential customers but are maintained by the City.

The pump stations serving Sub Basins 1 through 4 are listed below:

- 12th Street Pump Station
- Evergreen Gardens Pump Station
- Brookfield Pump Station
- Front Street Pump Station
- Eastgate Pump Station.

The 12th Street Pump Station is the major pump station serving the collection system and is the only station equipped with standby power. It serves Sub Basins 3 and 4. In the coming years, the 12th Street station will be required to handle increased pumping demands as any expansion of the service area eastward will require conveyance through this station. The remaining four stations serve local low laying areas and discharge to nearby gravity sewers (none discharge

directly to manholes). All five stations are equipped with run time meters; however, none of the stations are equipped with flow meters.

Run time data from three recent years is tabulated and is graphically presented on a month-to-month basis in Appendix A. Typically, the run time for each station is recorded every three to five days, but occasionally up to two weeks has passed between the recording periods. For this reason, it is not possible to accurately correlate pump run times with treatment plant flow or rainfall records. Notwithstanding, the records are valuable in that they indicate if the stations are operating near capacity or if they have significant reserve capacity.

4.5.1 12th Street Pump Station

Location:	Tillamook PUD Yard at Laurel and 12th Street
Constructed:	1957
Overhauled:	2001 (replaced impellers)
Design:	Wet Well/Dry Well
Pump Manufacturer & Model:	Krogh 8x8x14 1977C
Pump Type:	Vertical Non-Clog Centrifugal
Number of Pumps:	2 - one duty, one standby
Pump Duty Point:	2400 gpm @ 58 ft (one pump)(a) 3000 gpm @ 79 ft (two pumps)(a)
Pump HP:	20 hp (per pump)
Voltage:	208 v, 3-phase
Level Controller:	Bubbler System
Force Main:	277 ft of 12-inch asbestos-cement pipe (1957); plus 2,570 ft of 12-inch ductile iron pipe (force main extended 1975)
Discharge Manhole:	In field south of Elm Avenue, west of 10th Street. Manhole shows no indication of corrosion.
Ancillary Equipment:	60-kW, 208 volt, 3 phase, diesel powered Generator Set Automatic Transfer Switch Battery Charger Hour Meters Air Compressor (for bubbler controller, new 2004) Backup Air Compressor Pump Control Panels Ventilation Fan
Wet Well:	14ft diameter, 26ft deep: incorporates wet well/dry well, approx. 400 gal/ft depth in wet well
Alarms:	HWL, Loss of Power, Thermal Overload
SCADA:	Autodialer
Overflow:	No overflow pipe (b)
Spare Parts:	Packing, bearings
Standby Power:	Yes

Note:
(a) Duty points based on pump curve/system curve analysis (included in Appendix A).
(b) The rim elevations of the last 7 manholes upstream from the pump station are 5 ft to 6 ft lower than the top of the wetwell. Therefore, manholes (not the wetwell) will overflow under a pump station failure condition. At AWWF, the combined volume of the wetwell and the 12th Street Interceptor would provide approximately 3 hours of storage prior to overflow of the lowest manholes.

Condition/Discussion

- Based on pump run time records for wet weather months, the 12th Street Pump Station is currently running beyond its firm capacity. Reliability requirements mandate that the pump station should be able to handle peak flow events with one pump out of service. Records indicate that both pumps are required to operate up to 90 percent of the time for 2- to 3-day periods. This would indicate that both pumps are likely required to operate at 100 percent capacity during peak flow events.

- The discharge piping inside of the station shows surface corrosion in some areas and would benefit from recoating.

- The standby generator is aging and has maintenance issues from time to time. The City has a 150 kW generator installed at the treatment plant (new in 2003) that could be relocated to the 12th Street Pump Station following the upcoming WWTP upgrades.

- There are no louvers in the generator room, causing significant heat build-up unless the overhead door is opened.

- The existing 12-inch forcemain may not provide sufficient capacity to convey future peak flows.

4.5.2 Evergreen Gardens Pump Station

Location:	12th Street at the Evergreen Gardens Complex
Constructed:	Year unknown
Design:	Pre-Packaged Hydronix Station with FRP Wet Well
Overhauled:	
Pump Manufacturer & Model:	Hydromatic SPGF300M2-4
Pump Type:	Submersible grinder pumps
Number of Pumps:	2 - one duty, one standby
Pump Duty Point:	84 gpm @ 30' TDH
Pump HP:	1.5 hp (per pump)
Voltage:	230 v, 3 phase
Level Controller:	Mercury Level Switch
Force Main:	500 ft of 4-inch PVC (approximate)
Discharge Point:	South end of Williams Avenue. There is no manhole at discharge point. Manhole downstream of discharge point shows no corrosion damage.

Ancillary Equipment:	Hour Meters
	Air Compressor for air injection into discharge line (for H_2S control)
	Pump Control Panel
Wet Well:	5 ft diameter, 145 gal/ft depth
Alarms:	Float with light as alarm
SCADA:	No
Overflow:	No Overflow Pipe (no history of overflows)
Spare Parts:	No
Standby Power:	No

Condition/Discussion

- Fittings and steel components located inside of wet well are corroded.
- Air compressor building blocks access to the wet well, making it difficult to service the station with the Vactor Truck.
- Pumps and appurtenances generally in good running order.

4.5.3 Brookfield Pump Station

Location:	Brookfield Avenue
Constructed:	1998
Design:	Pre-Packaged Hydronix Station with FRP Wet Well and Above Grade Enclosure for Piping, Valves, and Appurtenances
Overhauled:	Never
Pump Manufacturer & Model:	Hydromatic S4NX500JC
Pump Type:	Submersible, 4-inch Discharge
Number of Pumps:	2 - one duty, one standby
Pump Duty Point:	170 gpm @ 38.5 ft
Pump HP:	5 hp (per pump)
Voltage:	Pump: 230 v, 3 phase, Controls 120 v
Level Controller:	Mercury Level Switch
Force Main:	1,500 ft of 6-inch PVC (approximate)
Discharge Point:	Along 3rd Street just west of Fairgrounds. No manhole at discharge point.
Ancillary Equipment:	Hour Meters
	Valve Vault with Solenoid Actuated Backflow Valve (for corrosion/odor control)
	1500 W Heater
	0.03 KVA Transformer

	0.25 hp Compressor for Solenoid Valve
Wet Well:	8 ft diameter, 19.5' deep, 210 gal/ft depth
Alarms:	HWL Float with light as alarm
SCADA:	No
Overflow:	No overflow pipe (no history of overflows)
Spare parts:	Floats
Standby Power:	No

Condition/Discussion

- Station is in very good condition.

4.5.4 Front Street Pump Station

Location:	Front Street
Constructed:	1964
Design:	Pre-Packaged Station, Wet Well / Drypit
Overhauled:	Unknown
Pump Manufacturer & Model:	Smith & Loveless
Pump Type:	Centrifugal Non-Clog
Number of Pumps:	2 - one duty, one standby
Pump Duty Point:	100 gpm @ 26 ft
Pump HP:	3 hp (per pump)
Voltage:	230 v, 3 phase
Level Controller:	Bubbler System
Force Main:	300 feet, 4-inch cast iron mechanical joint pipe
Discharge Manhole:	1st Street and Stillwell Avenue. Crown of manhole has been re-grouted to repair corrosion damage. Manhole Scheduled to be replaced in 2005.
Ancillary Equipment:	Hour Meters
Wet Well:	8ft diameter wet well / dry well, approx. 165 gal/ft depth in wet well
Alarms:	Float with light as alarm.
SCADA:	No
Overflow:	None
Spare Parts:	No
Standby Power:	No

Condition/Discussion

- Pump station is in good condition.

4.5.5 Eastgate Pump Station

Location:	South end of Meadow Avenue
Constructed:	Approx. 1965
Design:	Smith & Loveless Pre-Packaged Station, Wet Well / Drypit
Overhauled:	Unknown
Pump Manufacturer & Model:	Allis Chalmers 400 8x3x8
Pump Type:	Closed Coupled Centrifugal
Number of Pumps:	2 - one duty, one standby
Pump Duty Point:	150 gpm@25 ft
Pump HP:	3 hp (per pump)
Voltage:	230 v, 1 phase
Level Controller:	Bubbler System
Force Main:	Approximately 850 feet, 6-inch ductile iron
Discharge Point:	Elmwood St. and Dogwood Ave. No manhole at discharge point.
Ancillary Equipment:	Hour Meters
Wet Well:	8ft diameter wet well/dry well, approx. 165 gal/ft depth in wet well
Alarms:	Float with light as alarm.
SCADA:	No
Overflow:	No overflow pipe (no history of overflows)
Spare Parts:	No
Standby Power:	No

Condition/Discussion

- Pump station is in good condition.
- Manhole downstream of discharge point overflows when pipe is surcharged and pump station is called to run.

In addition to the approximately 60 STEP stations serving customers in Sub Basin 5 are three submersible pump stations listed below. These pump stations serve three of the largest wastewater contributors in the sub basin and differ from the STEP systems in that they pump whole, raw sewage.

- Fred Myer Pump Station
- Cornet Annex Pump Station
- Shilo Inn Pump Station

4.5.6 Fred Meyer Pump Station

Location:	Fred Meyer parking Lot
Constructed:	March, 1996
Overhaul:	Pumps were replaced in 1999
Design:	Rail-mounted duplex submersible pumps
Pump Manufacturer & Model:	Gorman Rupp, J Series, Model JSV3C60-X2.7
Pump Type:	Solids-handling centrifugals
Number of Pumps:	2
Pump Duty Point:	145gpm @ 25 ft TDH
Pump HP:	2.7 hp (each pump)
Voltage:	460v/3-phase
Level Controller:	5 Floats (low level-redundant off, stand-by-lead on, lag on, off, and high level alarm)
Force Main:	4-inch/PVC
Discharge Point:	1st Street and Stillwell Ave
Ancillary Equipment:	None
Wet Well:	8 ft diameter, 12 ft depth/FRP
Alarms:	High level, low level/light and Audible both inside building and out at control housing
SCADA:	NA
Overflow:	No overflow pipe (no history of overflows)
Spare Parts:	Spare Pump and floats
Standby Power:	No

Condition/Discussion

- Pump station is in good condition.

4.5.7 Cornet Annex Pump Station

Location:	Tillamook Coastal Plaza (a.k.a. Cornet). In back of the Plaza.
Constructed:	Approximately 1975 (exact date unknown)
Overhauled:	2004
Design:	Duplex Submersible (pumps sit in tank bottom)
Pump Manufacturer & Model:	Meyers 2 hp WHREE20-21C
Pump Type:	Solids handling centrifugal
Number of Pumps:	2 – one duty, one standby
Pump Duty Point:	145 gpm@25 ft
Pump HP:	2 hp
Voltage:	230 v/1-phase
Level Controller:	4 Floats (OFF, ON, lag ON and HWL Alarm)
Force Main:	4-Inch/PVC

Discharge Point:	1st Street and Stillwell Avenue
Ancillary Equipment:	None
Wet Well:	8 ft diameter, 12 ft depth/FRP
Alarms:	Audible and visual alarm at control panel
SCADA:	No
Overflow:	No (no history of overflows)
Spare Parts:	Spare Pump and Floats
Standby Power:	No

Condition/Discussion

- Pump station is in good condition.

4.5.8 Shilo Inn Pump Station

(serves Restaurant, K&B Deli, and the Pool Restroom)

Location:	Shilo Inn and deli parking lot.
Constructed:	Approximately 1978 (Exact date unknown)
Design:	Rail-Mounted Duplex Submersible
Overhauled:	2004 - one new pump, one pump rebuilt
Pump Manufacturer & Model:	Hydromatic SPGH300 grinder
Pump Type:	Submersible grinder pumps
Number of Pumps:	2
Pump Duty Point:	Unknown
Pump HP:	3 hp
Voltage:	208/3 phase
Level Controller:	5 Floats (low level-redundant off, stand-by-lead on, lag on, off, and high level alarm)
Force Main:	4-Inch/PVC
Discharge Point:	1st Street and Stillwell.
Ancillary Equipment:	Emergency backup control system.
Wet Well:	8 ft diameter, 12 ft Deep/FRP
Alarms:	High level, low level/light and Audible both inside building and out at control housing
SCADA:	None
Overflow:	No
Spare Parts:	Spare Pump and floats
Standby Power:	No

Condition/Discussion

- Pump station is in good condition.

Section 5: Existing Wastewater Treatment Plant

5.1 Background and Process Description

5.1.1 Background

The Wastewater Treatment Plant (WWTP) is located at the west end of the City of Tillamook, just east of the Trask River, as shown on Figure 5.1. The original WWTP was constructed at the existing site in 1948. At that time, the headworks, influent pump station, one primary clarifier, chlorine contact tank, primary digester, and sludge drying beds were constructed. In 1958, the plant was upgraded with new influent pumps and pump controls, primary clarifier modifications, a trickling filter, and a new primary digester. In 1969, construction added trickling filter recirculation, secondary clarification, and new disinfection facilities.

The trickling filter process was replaced with RBCs in 1978, along with new secondary clarifiers. At that time, the old secondary clarifier was converted to the chlorine contact tank, the trickling filter was converted to a sludge holding tank, and an Operations Building was built with new facilities for blowers and chlorine equipment.

In 1998, the City completed requirements of a Mutual Agreement and Order (MAO) for construction of new headworks and a new primary clarifier. In 2004, the City made various improvements to the plant that had been identified in a Corrective Action Plan (CAP). Improvements associated with the CAP included a third blower, RBC recirculation modifications, automated sludge valving for the primary clarifier, a new standby generator, and a supernatant pump station to transfer digester decant streams downstream of the influent sampler.

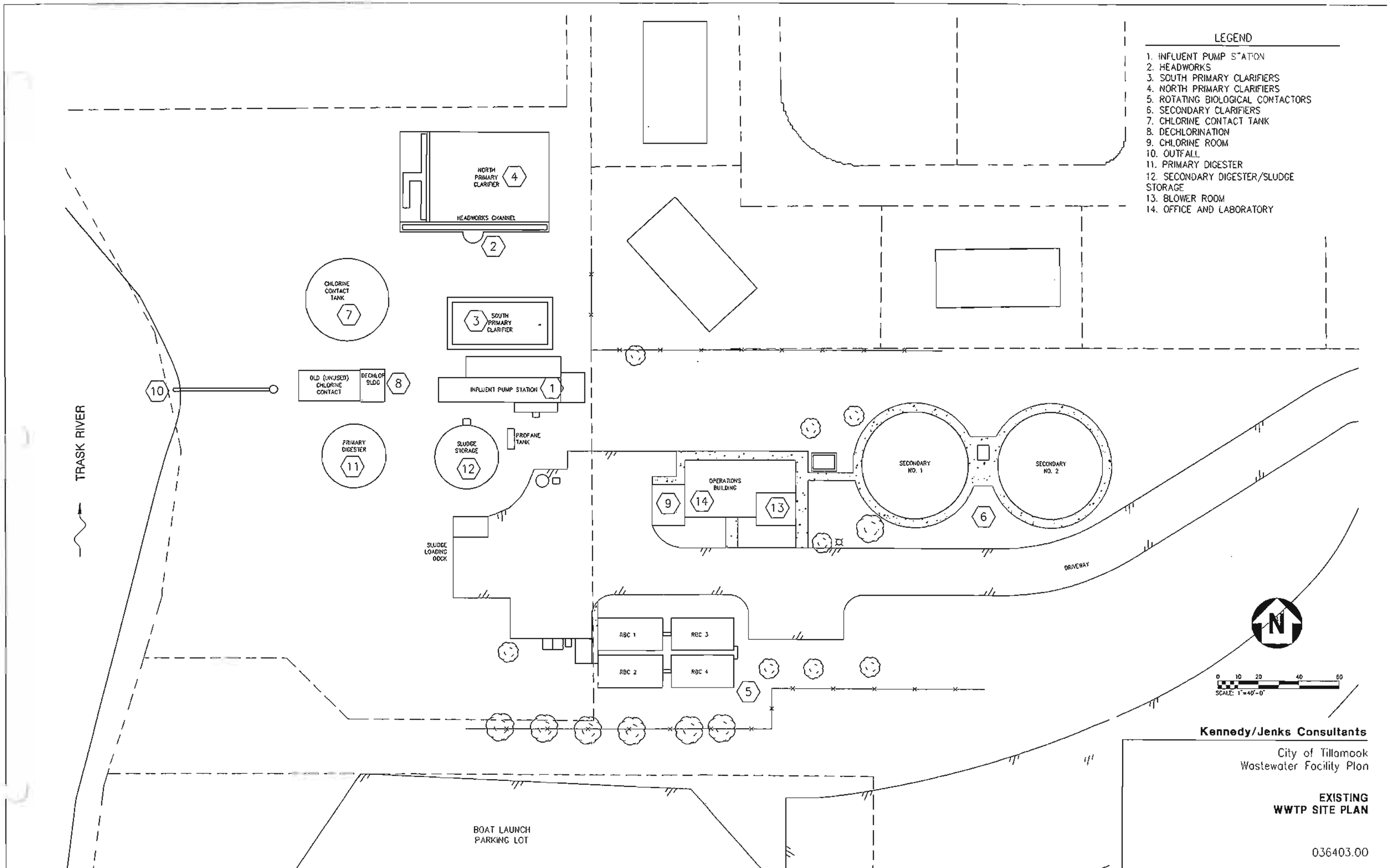
A site plan of the existing WWTP is shown on Figure 5.1.

5.1.2 Process Description

A simplified process schematic for the existing Tillamook WWTP as operated in 2004 is provided as Figure 5.2. A more detailed flow diagram along with design criteria for the existing plant is presented on Figure 5.3. A hydraulic profile of the Tillamook WWTP is presented on Figure 5.4.

As illustrated on Figure 5.2, raw sewage is pumped from the influent pump station to the head works channel located at the north primary clarifier. Raw wastewater is sampled at the upstream end of the headworks channel. A Parshall flume, a vortex grit chamber and a comminutor are located in the headworks channel.

Raw sewage flows to one of two north clarifier basins through the distribution channel for primary treatment. Settled sludge from the north primary clarifier is pumped to anaerobic digesters while primary effluent flows by gravity to the RBCs. The old (South) primary clarifier is currently used only for the storage of digested sludge due to its deteriorating condition.



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City of Tillamook
Wastewater Facility Plan

EXISTING WWTP SITE PLAN

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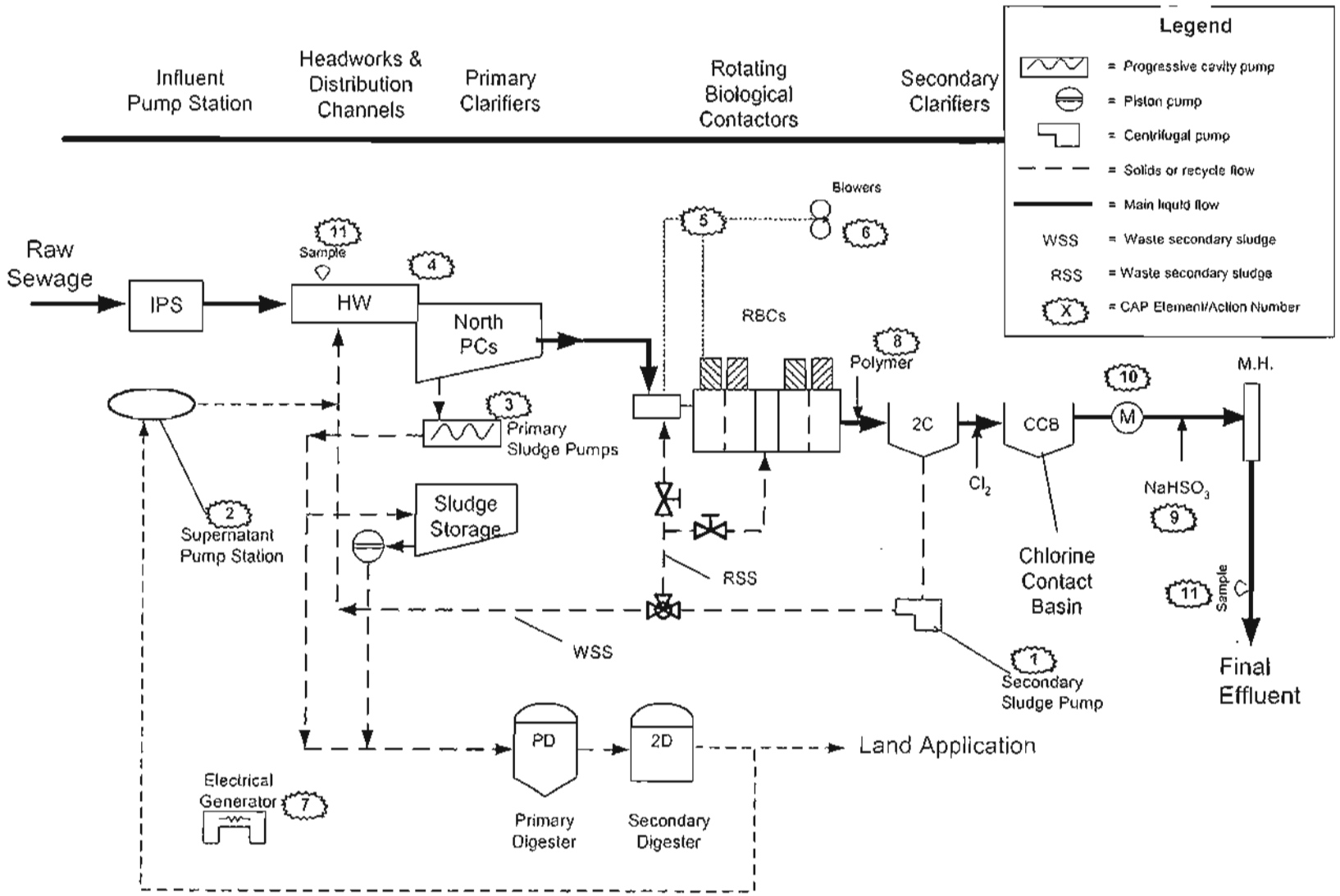
FIGURE 5.1



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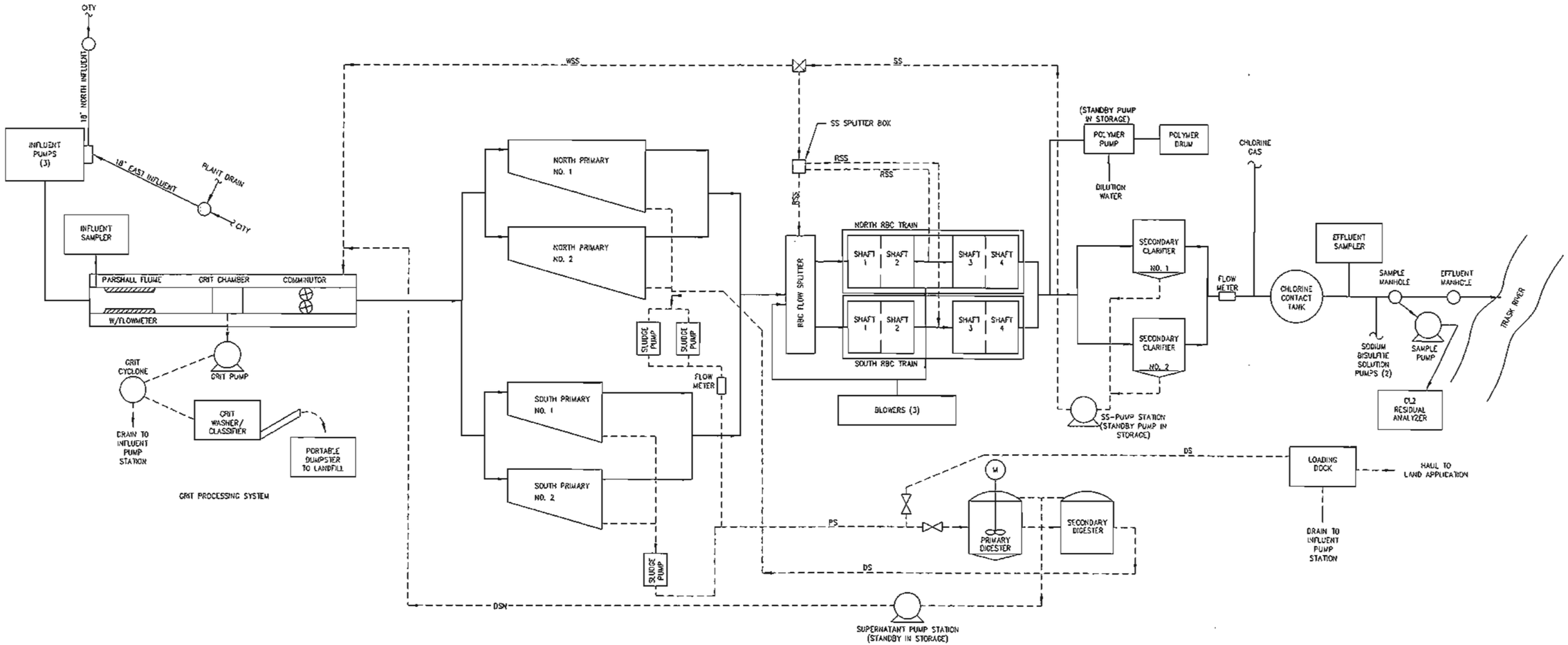
City of Tillamook Wastewater Facility Plan
Simplified Process Schematic
2004 Operation

FIGURE 5.2



DESIGN CRITERIA

Population 5400

Plant Flows (mgd)
Dry Weather Flow Rates
ADWF 1.07
MMDWF 2.4
Wet Weather Flow Rates
PWWF 6.1 AWWF 2.7
MMWWF 4.4 Instantaneous 8.5

Average Raw Wastewater Load (lb/d)
Raw Wastewater
BOD 1250
TSS 1250
Primary Effluent
BOD 930
Soluble BOD 530

Septage: Not Accepted

Effluent Limit Concentration in Draft Permit (mg/l)
May 1 - Oct 31 (monthly+weekly)
BOD 20*30
TSS 20*30
FC/100 200*400
November 1 - April 30 (monthly+weekly)
BOD 30*45
TSS 30*45
FC/100 49/200
Year-Around
pH 6.0 – 9.0
BOD and TSS removal efficiency 85%
Chlorine (daily maximum*monthly average) 0.02*0.01
Excess Thermal Load (BTU/day)
7– day moving average 141
Monthly average 62
Turbidity as NTU (not to increase background beyond) 10%

Preliminary Treatment
Parshall Flume
Throat Width (in) 12
Flow Capacity (mgd) 10
Grit Removal
Number and Type 1 - Pista Grit
Capacity (mgd) 10
Grit Handling Type cyclone separator
Grit Pump
Capacity (gpm) at 23 ft TDH 250
Horsepower 7.5
Comminutor
Capacity (mgd) 6
Type channel monster
Horsepower 3

North Primary Clarifiers
Unit Description
Number of Clarifiers 2
Dimensions ea. (W*L*SWD) in ft 12*50*12
Total Surface Area in sq ft 1200
Average Dry Weather Loading - 1 Unit in Service
Hydraulic Retention Time (hrs) 2.8
Overflow Rate in gpd/sq ft 890
Wet Weather Loading - 2 Units in Service
Maximum Month in gpd/sq ft 1840
Peak Day in gpd/sq ft 2500
Type of Sludge Pumps Progressive Cavity with VFDs
Number and capacity each 2 at 100gpm
Horsepower each 7.5
Magnetic Sludge Meter Size, in 3

South Primary Clarifiers
Number of Clarifiers 2
Dimensions ea. (W*L*SWD) in ft 12*50*12
Total Surface Area in sq ft 1200
Type of Sludge Pumps Piston
Number and capacity each 1 at 60 gpm

Rotating Biological Contactors (RBCs)
RBC Description
Number of Trains 2
Number of Shafts & Size (Surface Area in sq ft) 4 @ 95,300
RBC Type Air Drive with RSS Activation
1st Stage Surface Area in sq ft
as a 4 Stage Unit 95,300
as a 3 Stage Unit 190,600
Total RBC Surface Area in sq ft 381,200
RBC Centrifugal Blowers
Number 3
Capacity and Size each 700 scfm at 25hp
RBC Loading
SBOD load in lb SBOD/d/1000 sq ft
1st Stage Loading – 4 Stage Operation 5.6
1st Stage Loading 3 Stage Operation 2.8
Total Loading 1.4
HLR - gpd/sq ft
Average Dry Weather 2.8
Maximum Wet Weather 23.2
RSS Recycle
Capacity (gpm) 100
Return Location at RBC 1st or 3rd Stages

Secondary Clarifiers
Unit Description
Number 2
Dimensions each (dia*SWD) in ft 45*10
Total Surface Area in sq ft 3180
Unit Hydraulic Loading Rate in gpd/sq ft
Average Dry Months – 1 Unit in Service 670
Maximum Wet Weather Month – 2 Units in Service 1330
Peak Day – 2 Units in Service 1920
SS Pump Station
Number and Type (1 installed) 2 Submersibles
Capacity and Size (gpm*hp) each 100*5
Pumped Flow Destination SS, WSS and RSS
Chlorination
Unit Description
Number of Basins 1
Dimensions (dia*SWD) in ft 40*9
Volume in gallons 84,550
Unit Hydraulic Retention Time (min)
Average Dry Months 110
Maximum Wet Weather Month 28
Peak Day 20
Effluent Flow Meter
Type Open Channel – Electromagnetic
Capacity (mgd) 10
Polymer Feed
Type of Polymer Liquid Cationic
Number and Type (1 installed) 2 – Diaphram
Dose 0.5 to 8 mg/l

Digested Solids (DS)
TSS load in lb TSS/d 800
VSS load in lb VSS/d 500
Volume in gpd @ 2. 5% TSS 4000
Volume in gpd @ 3. 5% TSS 2900
Volume in gpd @ 5. 5% TSS 1800
Primary Digester
Unit Description
Volume in gallons 67,000
Mixer
Type Propeller – Draft Tube
Size, diameter x horsepower (in*hp) 15*10
Unit Loading
Hydraulic Retention Time in days at 3.5% TSS 20
Solids Loading in lb VSS/d/cu ft 0.06
Supernatant Pump Station
Number of Pumps and Type (1 installed) 2 – Submersible
Capacity and Size (gpm*hp) each 100*5
Sludge Storage
Unit Size in gallons
Secondary Digester 84,500
South Primary Clarifiers (includes 3 ft water cap) 80,700
Total Potential Sludge Storage 165,200
Sludge Storage Time in days at 3.5%
Secondary Digester 29
South Primary Clarifiers (includes 3 ft water cap) 28
Total Potential Sludge Storage 57
Outfall Line
Size as length from CL Tank x pipe dia. (ft*in) 80*18
Discharge Location Edge of Bank
Electrical Reliability
Standby Power Size (kw) and Type 150 kw Diesel
EPA Class one

Alarm Telemetry: Autodialer
Following alarms activate Autodialer:
CL2 Feed Room Leak Detector
CL2 Storage Room Leak Detector
CL2 Storage Low Pressure
Channel Monster Failure (North Primary
Channel Shredder Failure (South Primary)
Secondary Clarifier No. 1 High Torque
Secondary Clarifier No. 1 Shutdown
Secondary Clarifier No. 2 High Torque
Secondary Clarifier No. 2 Shutdown
Secondary Sludge Sump HWL
Raw Sewage P1 Failure
Raw Sewage P2 Failure
Raw Sewage P3 Failure
Raw Sewage Sump HWL
Raw Sewage Dry Well Flood
RBC Blower Low Air Pressure
North Primary Clarifier Drive Failure
South Primary Clarifier Drive Failure
Plant Power Outage

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City of Tillamook
Wastewater Facility Plan

EXISTING PLANT FLOW DIAGRAM AND DESIGN CRITERIA

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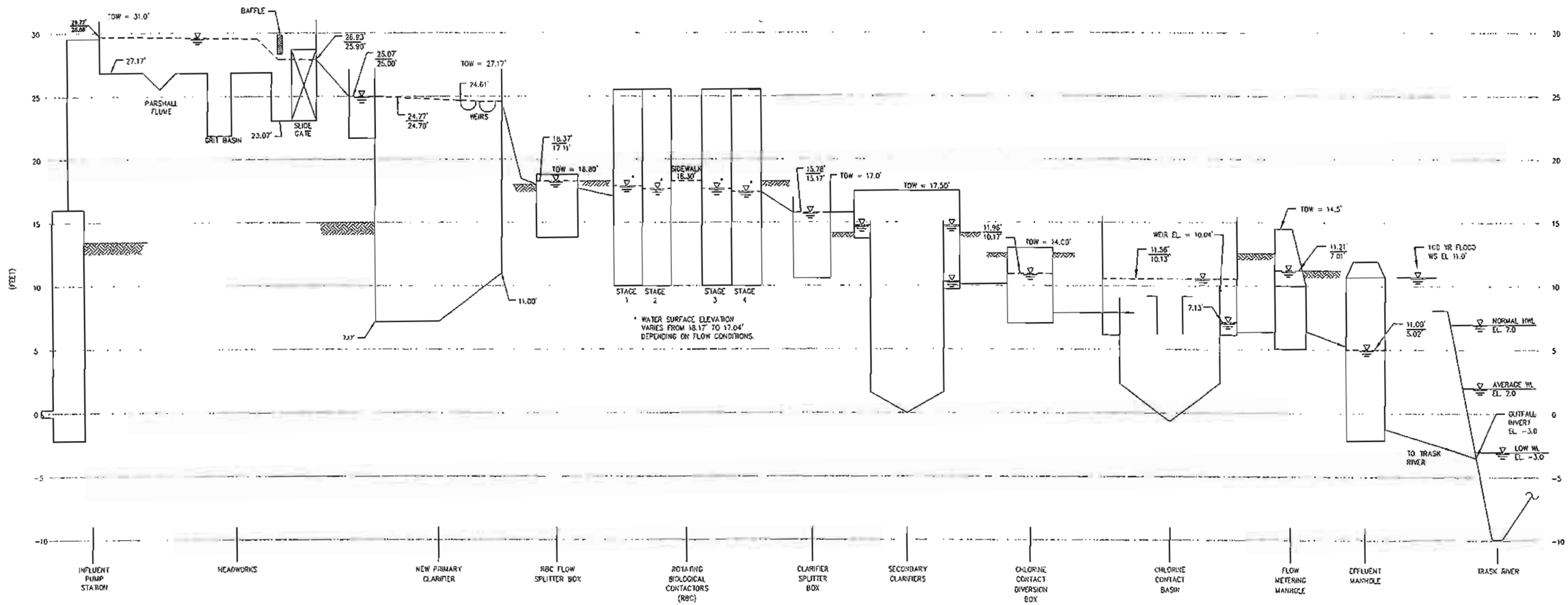
FIGURE 5.3



KEY:

WATER LEVEL

24.77' PEAK Q LEVEL
24.70' AVG. Q LEVEL

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City of Tillamook
Wastewater Facility Plan

EXISTING PLANT HYDRAULIC PROFILE

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FIGURE 5.4

Secondary treatment is achieved through the use of rotating biological contactors (RBCs). The RBCs use plastic media mounted on rotating shafts. Coarse bubble diffusers, supplied by three centrifugal blowers, provide the rotation energy to the RBC shafts.

Biological treatment in the RBCs is followed by two, circular center-feed secondary clarifiers units with rotating sludge rake mechanisms. The clarifiers have been modified with Stamford baffles to reduce solids carryover. Polymer addition is done on an as-needed basis to enhance settling and is generally not utilized during wet weather conditions.

Gas chlorine from150 lb cylinders is used to disinfect the final effluent. Chlorination is followed by de-chlorination using liquid sodium bisulfate ($NaHSO_3$).

Secondary sludge is pumped to the headworks channel and subsequently co-thickened in the primary clarifier. The combined sludge is then pumped to the primary digester for stabilization. Following treatment in the primary digester, solids are transferred to the secondary digester for final digestion and storage. The secondary digester can also serve as a thickener; however, because of the adverse impact on plant operation, decanting from the secondary digester is generally not done. Should overflow from the secondary digester occur, it would flow into the supernatant pump station for return to the headworks for treatment.

With the exception of the influent pump station, water flows by gravity though the WWTP as indicated on Figure 5.4. The new headworks and North Primary Clarifiers are located substantially above the surrounding ground surface, while the tops of down-stream units such as the secondary clarifiers and chlorine contact basin are located just above the surrounding ground. The outfall line is submerged during high or average tidal conditions, but the top of the outfall may be visible during low tidal conditions.

5.1.3 2002 CAP and Impact on Plant Performance

In 2002 the City was required to initiate a corrective action project (CAP). The CAP was required because of odor complaints and noncompliance with discharge standards. It was initially felt that minor modifications to the headworks along with enhancement to the RBCs could bring the plant into compliance. In studying problems at the WWTP, it was discovered that solids handling procedures and the impact of recycle streams obscured early estimates of plant loading. Repositioning of sampling equipment and contract hauling of solids improved record keeping and help to bring the WWTP back into compliance. In 2004, the following CAP improvements (also noted on Figure 5.2) were completed:

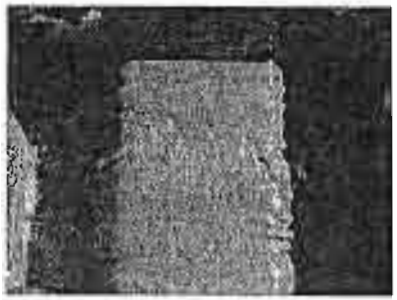
Item	CAP Element	Action
1	Secondary Solids Recirculation System	A secondary solids recirculation system was added to "activate" the RBCs by recirculating biologically active solids from the secondary clarifier to the RBCs.
2	Supernatant Pump Station	A digester supernatant pump station was constructed to return overflow from the digester to a location downstream of the influent sampler.

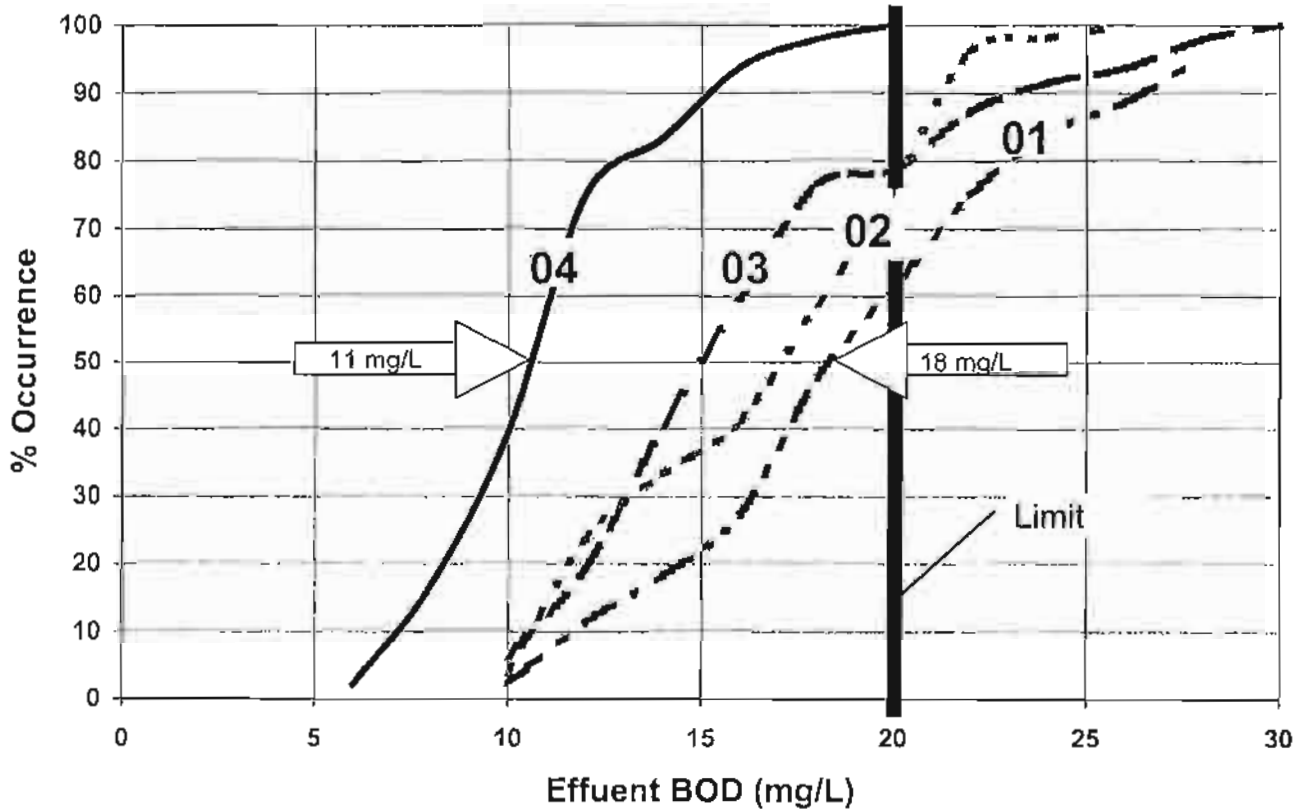
Item	CAP Element	Action
3	Primary Sludge Pumping Improvements	Variable frequency drives, a flow meter, electric actuators, and automatic controls were added at the primary sludge pump station to reduce manual operation and improve sludge thickening.
4	Headworks And Distribution Channel Improvements	Concrete work and other modifications were made to both the head works and distribution channels to eliminate dead zones and sludge accumulation.
5	Air Piping Improvements	Existing air piping between the blowers to the RBCs was inspected and repaired.
6	Air Blower Upgrade	A new 30-horsepower, variable frequency drive blower was installed to provide standby blower capacity (blower air powers the RBC shafts).
7	Standby Generator	A new 150 kilowatt (kW) standby generator was installed to replace an existing 75 kW unit.
8	Standby Polymer Feed Equipment	A new standby polymer feed system was supplied to match an existing unit.
9	Standby Dechlorination Pump	A new standby dechlorination pump was installed to provide added reliability.
10	Effluent Flow Meter	A new effluent flow meter was installed to provide accurate control of disinfection equipment.
11	Sampler Improvements	New wires/conduit were installed and existing raw and final effluent samplers were programmed so that samples could be collected proportional to the plant flow rate (versus time-based). The location of the influent sampler was moved upstream of the recycle discharge pipe at the headworks.

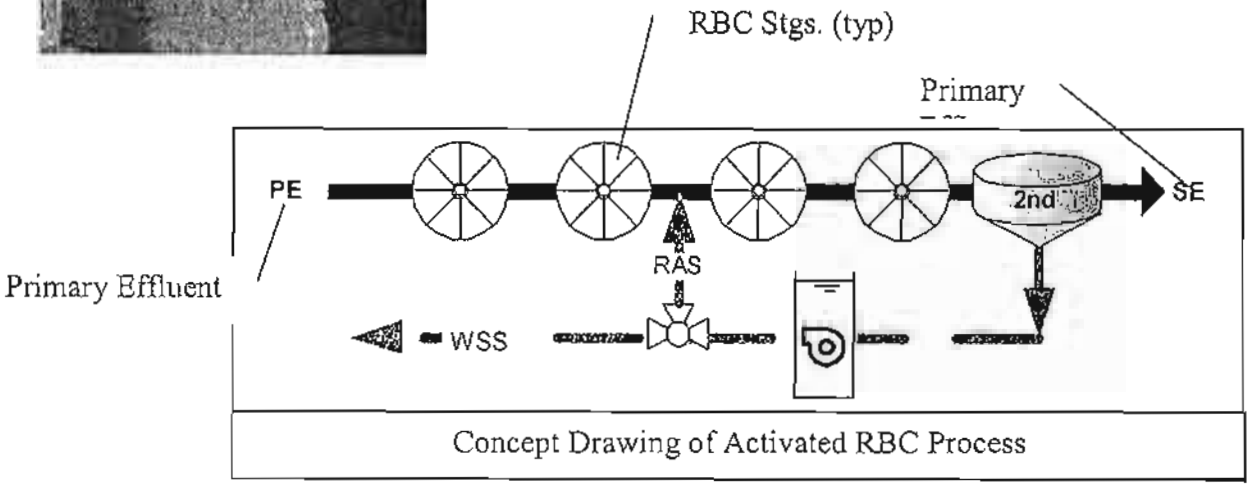
The outcome of the CAP improvements is evident when considering effluent quality as illustrated by the BOD comparison shown on Figure 5.5. In 2001, effluent BOD during dry weather months (May –Sept) averaged 18 mg/l, and values exceeded the 20 mg/l limit 40% of the time. During 2002 and 2003 effluent quality improved primarily attributed to improved solids handling operations. In 2004, the "activated" RBC process was initiated allowing return activated sludge to be recycled to the RBCs. During 2004, effluent BOD averaged 11 mg/l, and never exceeded 20 mg/l. Other CAP improvements have led to almost complete elimination of odor complaints and improved operation of the North Primary Clarifiers.

RAS splitter Box

RAS prior to RBCs







Concept Drawing of Activated RBC Process

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City of Tillamook Wastewater Facility Plan

EFFECT OF CAP IMPROVEMENTS

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Figure 5.5

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5.2 Facility Condition

This section of the report will describe and summarize the condition of each of the existing unit processes. They will be presented in the same order as they are numbered on Figure 5.1.

5.2.1 Influent Pump Station

All flow to the treatment plant passes through the influent pump station and flows by gravity through the rest of the treatment plant. The influent pump station is a wet well/dry well configuration with two 15 hp vertical non-clog centrifugal pumps (Pumps 1 and 2) and one, 20 hp submersible pump (Pump 3). The wet well and dry wells are rectangular, cast-in-place concrete. The dry well and part of the wet well are covered with a small building. The remaining portion of the wet well is outside the building and covered with grating.

Recent field testing indicated that with all pumps running, the capacity of the dry well pumps is approximately 950 gallons per minute (gpm) each, and the capacity of the submersible pump is approximately 1,080 gpm. At normal wetwell elevations, peak capacity of the pump station is approximately 4.3 MGD. Firm (redundant) capacity of the pump station is approximately 2.7 MGD. Higher flows are possible if the wetwell water surface is elevated.

Pump curves are included in Appendix A. Pump Station information is as follows:

Constructed:	1948
Overhauled:	1958, 1979 (replaced impellers, added submersible pump), 2003 (rebuilt pumps)
Design:	Wet Well/Dry Well
Pump Number, Type, Manufacturer & Model:	2 - Vertical Mount Centrifugal: Peerless 8x8x4M 1 - Submersible Centrifugal: Peabody Barnes SEH2002
Pump Duty Points & HP:	VM Centrifugal: 1,500 gpm @ 35 ft TDH, 15 hp Submersible: 1,140 gpm @ 34 ft TDH, 20 hp
Voltage:	230 v, 3-phase
Pump Controls:	Flomatcher Type S-2100 control system for 2 variable speed pumps and 1 constant speed pump

Pumps 1 and 2 are powered through load cells (which allow variable pump speeds), while Pump 3 has only a constant speed motor. Under normal conditions, Pumps 1 and 2 operate as lead/lag. If the level continues to rise, Pump 3 is activated. The pumps are controlled by a Flomatcher control panel which uses a bubbler type level sensor to measure the depth in the wet well. The Flomatcher system relies on compressed air. If the compressor fails, the system is completely offline and no alarm is activated.

The pump station was constructed in 1948 and was significantly modified in 1958 and again in 1979. Upgrades included the addition of a submersible pump, new impellers for the dry well pumps, and the Flomatcher control panel. Some of the manual plug valves are reportedly inoperable and the check valves require frequent maintenance due to their vertical orientation. The pumps are nearing the end of their service life and need replacement, as does the antiquated control system. While the subsurface structure is in adequate condition, the above grade structure is in relatively poor condition. In addition to the pump controls, the building houses the digester boiler, a small bathroom, and the old, but still used primary sludge piston pump.

Two 18-inch gravity lines feed the influent pump station. One line comes from the southeast; the other is from the north. The invert of the 18-inch pipe from the southeast is at approximately the bottom of the influent pump station. Because of this, the collection system to the southeast is in a permanently surcharged condition.

5.2.2 Headworks

The headworks structure was constructed in 1998. It consists of a single channel built on the south edge of the north primary clarifier. The channel is approximately 60 feet long by 2½ feet wide. A Parshall flume is installed for measurement of influent. Preliminary treatment includes grit removal, manual bar screening, and comminution.

The Parshall flume has a 12-inch throat width and a total capacity of approximately 10 MGD. The grit basin is a vortex-type, 8-foot diameter unit with a peak capacity of approximately 6 MGD. Grit is pumped from the bottom of the grit basin by a WEMCO™ grit pump into a grit classifier. The comminutor has a rated capacity of approximately 10 MGD. The grit system removes approximately 20 cubic feet (cf) of material per week (five, 30-gallon cans/wk) during high flows, and 4 cf of material per week during low flows. The material is landfilled.

Since startup, solids deposition upstream of the Parshall flume and elsewhere in the distribution channel during low flow periods has caused odors and ongoing maintenance. Modifications included as part of the 2003 CAP eliminated these problems. In general, the headworks structure is in good condition.

5.2.3 South Primary Clarifier

The south primary clarifier, constructed in 1948, was part of the original plant. It is currently used for sludge storage. The clarifier consists of two parallel basins, each of which is 12 feet wide by 50 feet long and 12 feet deep.

The sludge collection mechanism uses a wooden rake and drive chain assembly. A piston pump, located in the influent pump station, was used to pump sludge from the

clarifier. The concrete tankage is in fair condition. The mechanical appurtenances, piping, and valves are in poor condition. The basin is currently used to store digested solids.

5.2.4 North Primary Clarifier

The north primary clarifier was constructed in 1998. The structure includes two rectangular basins, 20 feet wide by 60 feet long. The side water depth is 14.0 feet. Solids are collected by fiberglass rake mechanisms. Sludge is pumped from four hoppers by two progressing cavity pumps to the primary digester, secondary digester, or to the sludge loading dock. Electrically actuated valves and a PLC were added under the 2004 CAP to reduce the required operator attention and increase the solids concentration of the wasted sludge. Overall, the clarifier is in good condition.

At a peak surface loading rate of 2,000 gpd/sq ft, the north primary has a peak hydraulic capacity of 4.8 MGD. At 4.8 MGD the hydraulic detention time is 75 minutes.

During summer months, the clarifier provides more capacity than optimal. During average summer flows of 0.5 MGD, the detention time is approximately six hours or about twice the normal average retention time.

5.2.5 Rotating Biological Contactors

The RBCs consists of multiple layers of plastic disks rotating on a horizontal shaft. About half of the disc is submerged in the wastewater while the remainder of the disc is exposed to atmosphere. The shafts and discs are continuously rotated by the upward force of air provided by the blowers. Small cups on the discs trap the rising air and force the discs to turn. Aerobic bacteria grow on the media and provide treatment of the wastewater. Nutrients are absorbed when the disc is submerged and oxygen is provided as the disc moves through the air.

The RBCs were constructed in 1978 along with two new secondary clarifiers. The system is composed of four concrete basins divided into two "trains" of discs. Each "train" consists of two shafts and each shaft has two sets of RBC discs (called a stage). Both trains were designed to be operated as 4-stage units by the installation of baffles between the discs and at the center of each shaft. Not long after startup (1978), the 1st-stage baffle was removed from one of the RBC trains, which now operates with only a 3-stage unit.

Each RBC shaft has 96,000 square feet (sq ft) of surface area, for a total media surface area of 384,000 sq ft. Loading of RBCs is usually defined in terms of soluble biological oxygen demand (SBOD) per thousand square feet per day. Although there may be differing opinions as to maximum reliable capacity of the existing RBCs, it appears the RBC system is experiencing signs of overloading, especially during the summer months when treatment efficiency is most important.

Two gravity fine screens are installed upstream of the RBCs. During the 1998 construction project, the piping was reconfigured such that the screens can only be fed from the old primary clarifier.

The RBC media, shaft and bearings are nearing the end of their useful life. None of the discs or shafts have been replaced since installed. Bearings on the RBCs have been replaced as needed; however, it is estimated that all of the media and shafts will need to be replaced within five years.

The two original blowers are located in the Operations Building. Although one blower is considered a standby unit, due to deteriorating gaskets in the buried air piping it has been necessary to operate both blowers to turn the RBCs at the recommended speed. As part of the 2004 CAP the air piping was repaired and third blower was installed to provide enhanced reliability.

The 2004 CAP improvements (described in Section 5.1.3) included modifications to recycle secondary solids to the RBCs.

5.2.6 Secondary Clarifiers

New secondary clarifiers were constructed along with the RBCs in 1978. The clarifiers are 45 feet diameter and have a10-foot side water depth providing for a peak flow surface overflow rate of 1,200 gpd/sq ft at a combined peak flow capacity of 3.8 MGD. They are a center influent well design with peripheral weirs and launders and have a rotating rake mechanism that directs settled solids toward a collection sump in the basin floor. A rotating skimmer arm and scum box collect floating material. A shared wetwell and pump is used to pump settled solids and scum to the headworks channel. The secondary solids are ultimately captured in the primary clarifier.

Stamford baffles, consisting of fiberglass reinforced polyester sheets, were retrofitted to the scum baffle to enhance the clarifier performance. These baffles form a horizontal baffle around the perimeter of the clarifier to prevent the escape of solids in the upward moving currents along the walls of the basin.

The sludge pump capacity is approximately 110 gpm. During the wet weather months, sludge is wasted to the primary clarifiers (conventional RBC treatment). During the dry weather months, secondary solids can be continuously recycled to the RBCs. Periodically, a set volume of waste sludge is wasted to the primary clarifier.

Although 25 years old, the secondary clarifiers are in fair condition. The clarifier mechanism drives, motors, and bearings have recently been rebuilt. The internal concrete surface is slightly pitted.

5.2.7 Chlorine Disinfection

The chlorine facility is located in the Operations Building. It is composed of a larger room designed for housing one ton chlorine cylinders and a smaller room for housing chlorine feed equipment. Both rooms have chlorine leak detectors but no scrubbers. The plant currently uses 150-lb cylinders in lieu of the larger one ton cylinders and a vacuum feed system. Chlorine consumption is monitored using a scale designed for two 150-lb cylinders.

Chlorine is metered and injected using two Wallace & Teirnan v-notch chlorinators. Chlorinator 1 is an automatic control unit with a capacity of 300 lbs/day. Chlorinator 2 is a manual unit with a capacity of 200 lbs/day. The manual unit is used as a backup to the automatic feeder. The plant typically uses about one 150-lb cylinder every two to four days.

The chlorine contact tank is a 40-foot diameter basin. It was constructed in 1969 as a secondary clarifier and converted to a chlorine contact tank in 1978. According to the 1978 design documents, total volume of the basin is 84,550 gallons. At a hydraulic retention time of 15 minutes, peak flow capacity of the basin is 8.1 MGD. At a hydraulic retention time of 60 minutes, maximum month flow capacity of the basin is 2.0 MGD.

The basin and its appurtenances are in fair condition.

5.2.8 Dechlorination

Liquid sodium bisulfite introduced at the outlet to the chlorine contact tank is used for dechlorination. Residual chlorine is measured at the effluent sample manhole. Sodium bisulfite is delivered using a Wallace & Teirnan Deox 2000 controller in conjunction with a Wallace & Teirnan metering pump. The Deox 2000 uses the signal from the effluent flow meter to pace the feed pump.

Sodium bisulfite is stored in two 550-gallon polyethylene containers. The liquid is delivered and transferred to the tanks via delivery truck three or four times per year. The two storage containers are not interconnected and are housed in a small outbuilding shown in the picture. The

shed is uninsulated. The building includes a small space heater to prevent the sodium bisulfite from freezing during winter months.

A manually controlled dechlorinator is installed as a backup to the automatic feed system.

The dechlorination system was installed in 1998 to meet the National Pollutant Discharge Elimination System (NPDES) permit requirements.

5.2.9 Outfall

The outfall is an 18-inch concrete pipe originating from the effluent sampling/flow metering manhole. The original outfall had a headwall and discharged at the edge of the river. As part of the 1978 improvements, the headwall was demolished and replaced with a 42-inch manhole and submerged discharge pipe. This outfall structure has been damaged over subsequent years. There are plans to improve the existing outfall by constructing a new outfall manhole and setting the outfall pipe invert to match the river bottom at the edge of the bank.

A summary of the 2003 Effluent Water Quality Study for the City of Tillamook is presented in Section 7 of this Facility Plan.

5.2.10 Primary Anaerobic Digester

The primary anaerobic digester was constructed in 1957. It is a 27-foot-diameter, fixed-cover concrete digester with a 16-inch propeller mixer enclosed in a center-mounted draft tube. The capacity of the digester is approximately 67,700 gallons. The digester is heated using hot water from a methane/ propane-fired boiler. The methane is recovered from the digester. Propane is used as a backup source of heating fuel. The mixer motor was originally 5 horsepower (hp), but was retrofitted to 10 hp in 1979.

The structure is concrete-lined CMU block. There are several places where liquid is be seeping through the tank wall.

Based on a detention time of 25 days, the capacity of the digester is 2,480 gal/day. At a solids concentration of 3.5 percent, this is approximately 725 lbs solids/day. Currently, the plant produces close to 1,100 lbs of solids per/day.

The minimum allowable detention time under 40 CFR 503 for Class B anaerobic digestion is 15 days. At a detention time of 15 days, the capacity of the digester is about 1,200 lbs solids/day. These numbers indicate that the primary digester is currently operating at or beyond the limit of its capacity.

Heating of the digester is accomplished with a U.S. Radiator Company cast iron boiler with a rated output of 162,000 BTUs per hour. It can be fired with either methane gas or commercial propane. Temperature control of the digester is controlled by manually varying the amount of

hot water circulated to the heat exchanger to maintain a temperature between 90 and 95 degrees Fahrenheit (°F). This temperature range promotes an efficient digestion process. At this temperature, total holding time required for Class B sludge is 15 days.

Hot water supply and return lines are buried between the boiler and the digester. During the 1980 modifications of the plant, these lines were insulated to help prevent heat loss between the boiler and the digester. Excess gas can be flared through a burner at the old control building. The flaring equipment is also in poor condition, so it is not safe for use. Currently, excess gas is released to the atmosphere without flaring.

5.2.11 Secondary Digester / Sludge Holding

The secondary digester, constructed in 1948, was originally the plant's primary digester. The secondary digester is unheated and is used to store digested sludge pending disposal. It is a 30-foot-diameter concrete fixed-cover structure with no mixing or heating capability. It has a liquid volume capacity of 84,500 gallons, which will provide about 22 days storage based on current production rates. The tank is partially buried. From a structural standpoint, it appears to be in good condition.

5.2.12 Operations Building

The Operations Building, constructed in 1978, includes the laboratory with office space, a lunch/meeting room, a locker room with a shower and toilet, the blower room, a workshop, and the chlorine storage and feed equipment rooms. The lab is approximately 150 square feet. Plant staff currently performs most of their own lab work, including BOD and TSS testing. The lab, although somewhat small, is adequate for current and future needs. The Operations building should be adequate to support the plant through the next phase of upgrades.

5.2.13 Electrical, Controls, and Standby Power

With the exception of the most recent CAP improvements, the majority of the electrical equipment and wiring does not meet current code requirements. In particular, the electrical panels and controls associated with the influent pump station and the anaerobic digester are old and problematic.

All control functions related to treatment plant operation must be made at the local panels. There is no centralized control, or SCADA system in place. Alarms are annunciated at the panel in the laboratory and transmitted via autodialer during hours when the plant is unmanned.

A new 150 kW diesel generator was installed as part of the CAP improvements. It is sized to support two influent pumps, one blower, the communitor, disinfection and dechlorination equipment, and miscellaneous lighting.

5.2.14 Sampling and Metering

Two automatic samplers are installed at the WWTP. Influent samples are collected at the upstream end of the headworks channel. Effluent samples are collected in the effluent sample manhole, downstream of the chlorine contact tank. Both samplers receive a signal from the effluent flow meter and can be programmed to collect a 24-hour, flow-paced sample. Both samplers have a refrigerated compartment to keep the sample fresh.

Influent flow is measured at the Parshall flume installed in the headworks channel and recorded on a chart recorder in the Operations Building. Effluent flow is measured by a velocity x area magnetic flow meter installed in the gravity pipe immediately upstream of the effluent sample manhole and recorded in the bisulfate feed building. The effluent flow meter signal is used to pace chlorine and sodium bisulfite feed rates.

5.2.15 Odor Control

In the past, the plant has periodically generated foul odors prompting complaints from neighbors. Primarily, the odors originate from the headworks channel and primary clarifier. The 2003 CAP addressed the source of these odors by eliminating areas where solids tended to accumulate and putrefy. Another source of odor generation resulted from the practice of storing solids in the primary clarifier. Minimizing the solids inventory at the treatment plant should eliminate/control this source of odor.

There is no specially designed equipment for odor control other then good housekeeping and maintaining regular wasting of solids.

Section 6: Wastewater Flows and Loads

6.1 Historical Wastewater Flows

Wastewater flows for cities in western Oregon are strongly influenced by the seasonal variations in rainfall. Analysis of historical flow and rainfall data illustrates the influence of precipitation on flow and can be used to develop existing and future flow estimates. The analysis of historical data presented in this section is based on daily monitoring reports (DMRs) for the Tillamook WWTP from January 1999 to February 2003. A summary of DMR data is presented in Appendix C.

6.1.1 Definitions

The flow rates used by the Department of Environmental Quality (DEQ) to describe seasonal flow rates and other parameters used in this section are defined below.

<u>Dry Weather (DW):</u>

- **Average Dry Weather Flow (ADWF):** The average flow during the dry weather season months of May through October. This is the flow rate on which dry-weather mass loading is based.

- **Maximum Month Dry Weather Flow ($MMDWF_{10}$):** The maximum monthly average dry-weather flow with a 10 percent probability of occurrence defines the monthly average design flow for the wettest summer month of high groundwater. In western Oregon, this month almost invariably occurs in May. This flow rate results from a combination of sewage baseline flow and I/I contribution. The value of 10 indicates that this flow rate, on average, is likely to occur once every 10 years. Stated another way, there would exist a 10 percent probability that this flow rate is likely to occur in any given year.

<u>Wet Weather (WW):</u>

- **Average Wet Weather Flow (AWWF):** The average of flows for the months of November through April. This is the flow rate on which wet-weather mass loading is based.

- **Maximum Month Wet Weather Flow ($MMWWF_5$):** The maximum monthly average wet-weather flow with a 20 percent probability of occurrence defines the monthly average design flow for the wettest winter month of high groundwater. In Oregon, this month is typically January as ground water levels are high and rainfall is normally greatest. This flow rate results from a combination of sewage baseline flow and I/I contribution. The value of five indicates that this flow rate, on average, is likely to occur once every five years. Stated another way, there would exist a 20 percent probability that this flow rate is likely to occur in any given year.

Peak Flow (PF):

- **Peak Day Flow (PDF_5):** The peak day average flow associated with a 5-year storm, normally of 24-hour duration occurring during a period of high groundwater. This storm would have a statistical chance of occurring, on average, once every five years, resulting in a 20 percent probability that this flow rate is likely to occur in any given year.

- **Peak Instantaneous Flow (PIF_5):** The peak instantaneous flow attained during the PDF_5. This flow is often defined as the sustained one-hour peak flow.

Other Flow Related Terms:

- **Inflow:** This term refers to stormwater runoff that enters the sanitary sewer from seasonally flooded manhole covers or from improper connections between the sanitary and storm sewer systems. Sources of stormwater runoff can be streets, parking lots and service yards, roof tops and other impervious areas.

- **Infiltration:** This term refers to groundwater that enters sanitary sewer lines and is normally due to deterioration of pipes and manholes, leaky connections between pipes and lateral connections or laterals that are in poor condition.

- **Gallons per capita day (gpcd):** This term refers to the average daily flow in gallons per day divided by the population served.

6.1.2 Flow Records

When analyzing flow records, it is important to identify any limitations or inconsistencies in the data. For the Tillamook wastewater system, the following factors must be considered when reviewing historical flow data:

- Rainfall has a strong influence on treatment plant flows. Wastewater flow typically peaks in January when groundwater and rainfall is high and is lowest in September when groundwater has subsided and rainfall is low. Flow during summer months is primarily due to wastewater contribution and serves as the baseline for estimating peaking factors and I/I contribution during winter months.

- The influent and effluent flow meters currently installed at the WWTP plant typically indicate a 15 percent to 30 percent differential in readings depending on flow rate. The effluent meter is an 18-year old flow velocity meter while the influent meter, installed in April 1999, utilizes an ultrasonic level sensor and a Parshall flume. The influent meter is the more reliable of the two meters and indicates lower values. Historical data taken from records prior to April 1999 has been adjusted to account for the exaggerated readings.

- Flow is typically recorded once per day by treatment plant staff and may not accurately reflect the effect of rainfall recorded for the same period. Significant rainfall occurring late in the recording period may not be reflected until the following day's flow measurement.

Data for high flow events are underestimated for the following reasons:

- There are several hydraulic bottlenecks in the collection system attenuating high flows by surcharging pipes and overflowing at manholes. The overflows are not metered.
- Run time records for the 12th Street Pump Station reflect that both pumps have, on occasion, run 24-hours per day indicating that this pump station may restrict flow during peak flow events.
- The influent pump station at the WWTP has a normal maximum capacity of approximately 4.3-MGD according to the influent flow meter (flows up to 5.0 MGD are possible with extreme wet well levels). This limits the flow to the plant.
- As the influent pump station level rises, the collection system will surcharge the South Interceptor which conveys flows from Sub Basins 2, 3 and 4. This results in overflows at low laying manholes and possible exfiltration of sewage. The overflows are not metered.

The results of hydraulic restrictions listed above are that peak monthly, peak daily and peak hourly flow records recorded at the WWTP on the DMRs are lower than what would occur if the restrictions and overflows in the collection system were removed. It is important to note that because the recorded flows are low, and because these flows will be used to project future peak flows, the 20-year projected flows could very well be lower than what the plant experiences in the next 20 years.

6.1.3 Rainfall and Wet Weather Flow

Average monthly plant flow (based on the influent flow meter) and monthly rainfall totals for the same period are shown on Figure 6.1 for a 4-year period. This figure clearly illustrates the strong influence of rainfall on plant flow. For the months when rainfall is low, groundwater and surface water runoff have a minimal impact on plant flow. As rainfall increases in winter months, a corresponding increase in plant flow is evident indicating that I/I is significant.

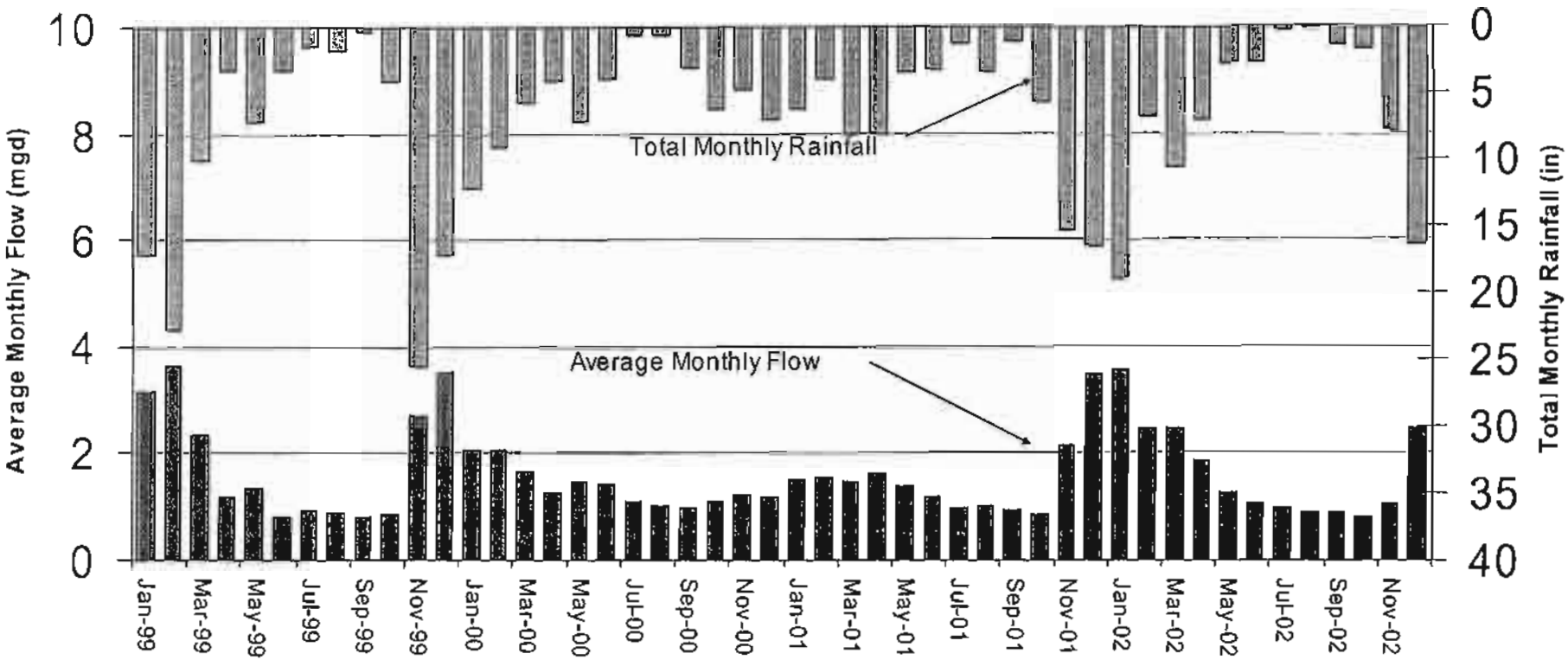
No detailed I/I analysis will be completed for the Tillamook collection system due in part to the lack of a long record of reliable flow data. When evaluating improvements to the system, some estimate will be made of the cost to reduce I/I in the entire system.

6.1.4 Flow Analysis

The flow analysis presented in this section is based on the DEQ document *Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon: MMDWF, MMWWF, PDAF, and PIF, Rev. IV.96.*

6.1.4.1 Maximum Month Flows (MMDWF10 and MMWWF5)

The following analysis uses information taken from *Climatology of the United States No. 20, Climatic Summaries for Selected Sites, 1951 to 1980, National Oceanic and Atmospheric Administration (NOAA).* This reference lists statistical rainfall information for selected meteorological stations in the United States. Records for the Tillamook Weather Station 1W



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City of Tillamook Wastewater Facility Plan
Monthly Flow and Rainfall Summary
January 1999 to December 2002

FIGURE 6.1

were used as the basis of statistical rainfall information for the study area and are presented in Appendix D. Based on this data, the monthly rainfall total associated with the $MMWWF_5$ (normally January) is equal to 20.53-inches and the monthly rainfall associated with the $MMDWF_{10}$ (normally May) is equal to 6.54-inches. The rainfall events associated with these flow rates would have a 20 percent and 10 percent chance, respectively, of occurrence in any given year.

The first step in making a maximum monthly flow rate estimate is to identify the relationship between average daily plant flow and monthly rainfall totals by constructing a graph. The graph uses only the months of January through May, as this is the period when groundwater is normally elevated. To accurately reflect the relationship between rainfall and flow, data was taken from a recent year when rainfall patterns are close to normal. Examination of Figure 6.1 shows that 1999, 2000 and 2002 all show a similar pattern of rainfall for the months of January through May in both quantity and pattern of occurrence. Year 2001 experienced an unusually dry winter and since the DEQ guidelines recommend using the most current year reflecting a normal rainfall pattern, 2002 was used as the basis for the analysis. Figure 6.2 is a graph showing the relationship between monthly average plant flow and rainfall for the period of January 2002 to May 2002. The best-fit line was drawn through the data points. Rainfall quantities corresponding to the monthly precipitation totals with a 10 percent and 20 percent probability of occurrence for the months of May and January respectively were extended to the best-fit line and then projected to the y-axis to arrive at the $MMDWF_{10}$ and the $MMWWF_5$ estimates. The analysis indicates a value of 3.17 MGD corresponding to the $MMWWF_5$ and a value of 1.5 MGD corresponding to the $MMDWF_{10}$. These values correspond favorably to actual plant flow and rainfall data for the months of January 2002 and April 2002 (see note b) as shown on Table 6.1.

Table 6.1: Estimated $MMWWF_5$ and $MMDWF_{10}$ Compared to Actual Rainfall and Flow Records

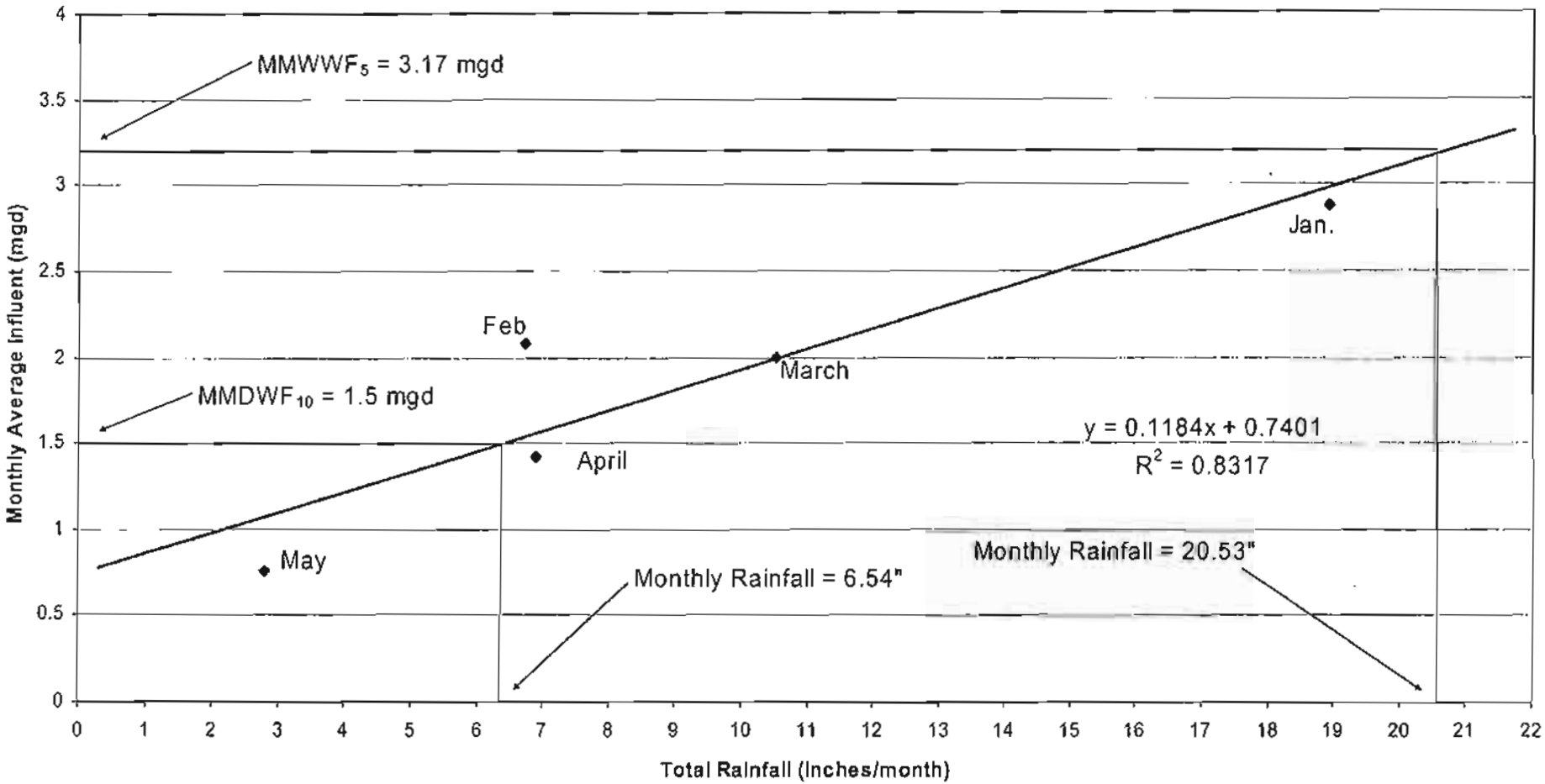
	Rainfall (inches)	Flow (MGD)
$MMWWF_5$ [(a)]	20.53	3.17
January 2002 ADF[(b)]	19.05	2.88
$MMDWF_5$[(a)]	6.54	1.5
April 2002 ADF[(b)]	6.9	1.42

Notes:

(a) Estimate based on DEQ Wet-Weather Analysis Guidelines
(b) From actual recorded rainfall and flows. Rainfall/Flow records from April 2002 were used for comparison as the rainfall totals were close to the historical 10 percent probability value for May. May 2002 had below-average rainfall.

6.1.4.2 Peak Day Flow (PDF5)

In western Oregon, the PDF_5 corresponds to the 5-year storm by definition. It is the plant flow that will result from a 5-year storm during a period of high groundwater. Typically, the 24-hour duration storm is used since data is normally collected at the treatment plant once per day. As



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City of Tillamook Wastewater Facility Plan
MMDWF and MMWWF Estimates
Flow vs. Rainfall January to May 2002

FIGURE 6.2

previously stated in this section, recorded peak flow values are underestimated due to surcharged pipes and manholes, collection system overflows and pump station limitations. For this reason, a statistical approach was used to derive the PDF_5 by constructing a graph that shows the relationship between average daily plant flow and daily rainfall under high groundwater conditions. This methodology assumes that there is an approximately linear relationship between plant flow and rainfall, where plant flow steadily increases with increasing rainfall. DMR records were examined for the wet-weather periods between January 2001 and February 2003. 24-hour rainfall accumulations greater than 0.5-inches and the corresponding plant flow are shown in Figure 6.3. As indicated by this graph, there is a poor correlation between daily rainfall and plant flow. The reasons for this are as follows:

- Consecutive daily rainfall events with increasing totals have a better correlation to plant flow than consecutive daily rainfall events showing decreasing totals. For instance, a 0.5-inch rainfall event following two days of 1-inch rainfall will indicate a much greater plant flow than would normally be associated with 0.5-inches of rainfall. Conversely, 1-inch of rainfall following two days of 0.5-inch rainfall will show a strong correlation between the rainfall event and the plant flow.

- Rainfall events resulting in plant flows approaching 3.75 MGD (the approximate un-surcharged capacity of the influent pump station) do not indicate an accurate relationship between rainfall and plant flow as the diurnal peaks are attenuated by surcharges, overflow and pump station limitations.

- Plant flows corresponding to significant rainfall events occurring after several days of no rain are reduced as groundwater has subsided to a degree during the period of no rainfall.

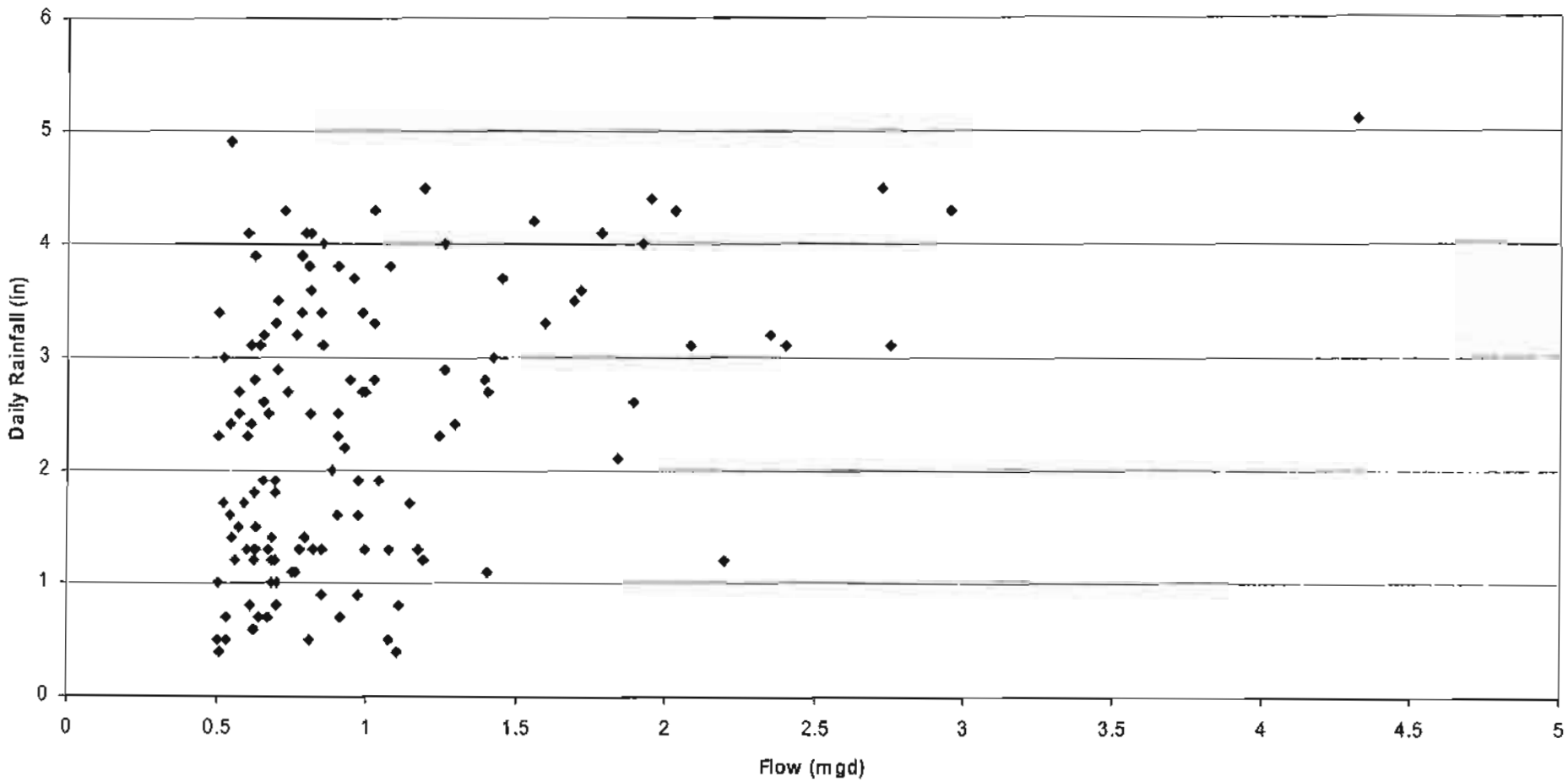
For these reasons, it is necessary to distinguish the rainfall events that exhibit a better correlation to recorded plant flow from those with a poor correlation. Figure 6.4 shows rainfall and flow data for the same period shown in Figure 6.3 with selected data points omitted for the following reasons:

- The plant flow exceeded 3.75 MGD minimizing the effect of extreme rainfall totals on plant flow.

- Consecutive daily rainfall totals were decreasing exaggerating the effect of rainfall on plant flow.

- Antecedent groundwater levels were low minimizing the effect of significant rainfall totals on plant flow.

The best-fit line was drawn through the data points presented in Figure 6.4. Rainfall quantity corresponding to the 24-hour, 5-year rainfall total was extended to the best-fit line and then projected to the y-axis to arrive at the PDF_5 estimate of 5.97 MGD. According to the isopluvial rainfall map shown in Appendix D, the 5-year, 24-hour storm for the Tillamook area is equal to 4.75-inches.



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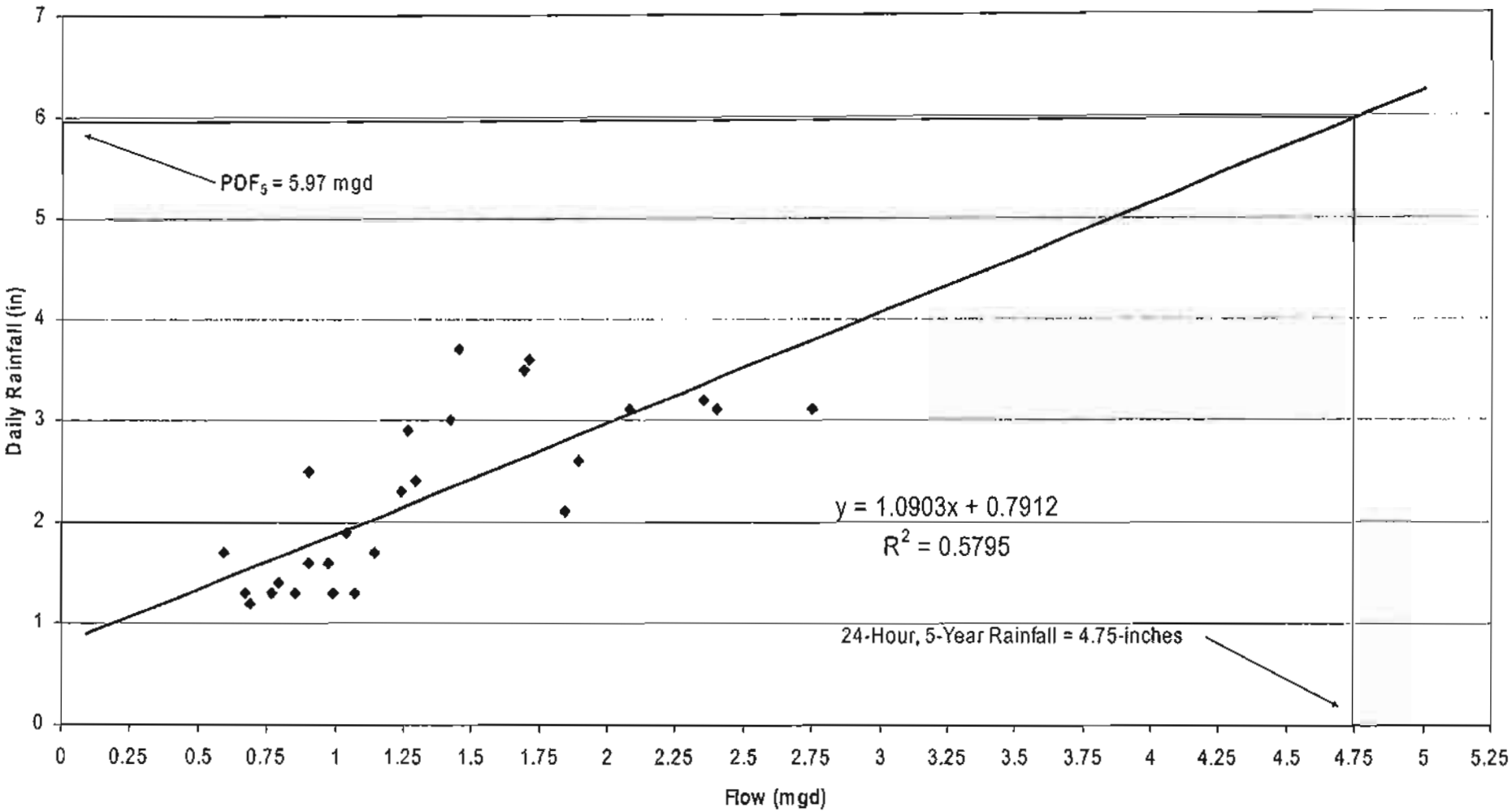
City of Tillamook Wastewater Facility Plan
24-Hour Rainfall and Flow for Storms Greater than 0.5-inch
January 2001 to February 2003

FIGURE 6.3



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PDF Estimate

24-Hour Rainfall and Flow for Selected Storms

FIGURE 6.4

The data points used in Figure 6.4 are listed on Table 6.2.

Table 6.2: Selected Rainfall and Plant Flow for Estimating PDF_5

Date	Rainfall (inches)	Plant Flow (MGD)
4-Jan-01	0.99	1.3
13-Jan-01	0.79	1.4
30-Jan-01	1.07	1.3
16-Feb-01	0.85	1.3
1-Mar-01	0.69	1.2
18-Mar-01	0.67	1.3
27-mar-01	0.79	1.4
31-Mar-01	0.90	1.6
30-Apr-01	1.14	1.7
14-Nov-01	1.89	2.6
22-Nov-01	1.42	3.0
28-Nov-01	1.69	3.5
7-Jan-02	2.35	3.2
20-Jan-02	1.71	3.6
21-Jan-02	0.59	1.7
11-Mar-02	2.08	3.1
19-Mar-02	2.75	3.1
10-Apr-02	0.97	1.6
14-Apr-02	1.24	2.3
11-Dec-02	1.84	2.1
18-Dec-02	1.45	3.7
27-Dec-02	0.90	2.5
30-Dec-02	1.26	2.9
13-Jan-02	1.04	1.9
22-Jan-03	0.77	1.3
26-Jan-03	2.4	3.1
17-Feb-03	1.29	2.4

6.1.4.3 Peak Instantaneous Flow (PIF5)

Since the available DMR data does not provide a basis for determining the actual PIF_5, it is necessary to derive an estimate. According to the DEQ guidelines, the PIF_5 may be estimated by means of extrapolation using a log-log probability graph and the following assumptions:

- The year of interest will include the MMWWF5, and within it a PDF5 and the PIF5. The average wet weather flow rate (AWWF) will be the average of the flows over the months of May through October. All bottlenecks are removed from the system to eliminate peak-shaving effects. Wet weather prevails during the "wet weather" months. These assumptions yield the following probabilities of occurrence:

- The average wet weather flow occurs 50 percent of the time.- The $MMWWF_5$ occurs 1/12 of the time or has a 8.3 percent probability of occurrence

- The PDF_5 occurs once in 365 days or has a 0.27 percent probability of occurrence

- The PIF_5 occurs once in 8,760 hours or has a 0.011 percent probability of occurrence

Figure 6.5 shows a plot of the calculated AWWF for 2002 and the derived values for $MMWWF_5$ and PDF_5 and then extends the resulting best-fit line to arrive at an estimate for the PIF_5. The graph indicates a PIF_5 equal to 8.47 MGD.

Table 6.3 summarizes the calculated and derived flows for the existing collection system.

Table 6.3: Existing Seasonal Flows and Peaking Factors

Flow	Flow (MGD)	Peaking Factor[c]	Per Capita contribution (gpcd)[d]
ADWF[a]	0.55	1.0	119
$MMDWF_{10}$[b]	1.50	2.7	324
AWWF[a]	1.86	3.4	402
$MMWWF_5$[b]	3.17	5.8	685
PDF_5[b]	5.97	10.9	1,291
PIF_5[b]	8.47	15.4	1,831

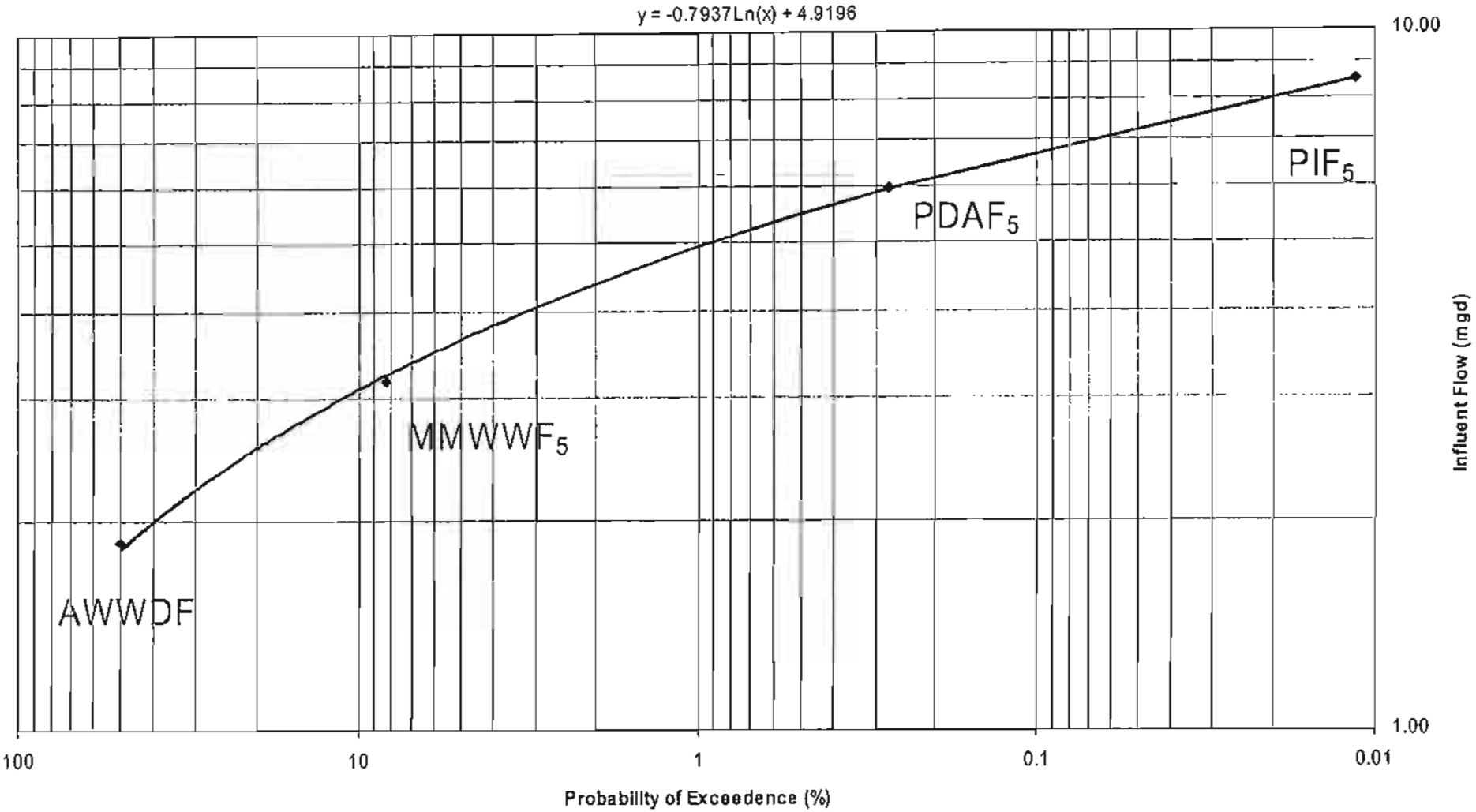
Notes:

(a) Based on year 2002.
(b) Estimated using DEQ Guidelines.
(c) Peaking Factor based on Average Dry Weather Flow (ADWF).
(d) Based on a 2003 population of 4,625.

6.2 Historical Wastewater Loads

Historical BOD_5 loads in pounds per day and mg/l are shown on Figure 6.6 for the years from 1999 to 2002. Historical TSS loads in pounds per day and mg/l are shown on Figure 6.7 for the years from 1999 to 2002.

Inspection of the BOD_5 graphs show a substantial reduction in both the total pounds and concentration for 2002 when compared with the other years shown. Inspections of the TSS graphs show a similar decrease in both total pounds and concentration for 2002. The reason for these reductions is that prior to 2002, the automatic sampler for influent TSS and BOD_5 was located downstream of the secondary solids return point (secondary solids are returned to the primary clarifier and co-thickened). In the latter part of 2000, the sampler was moved to a location upstream of the secondary solids return point resulting in a corresponding reduction in



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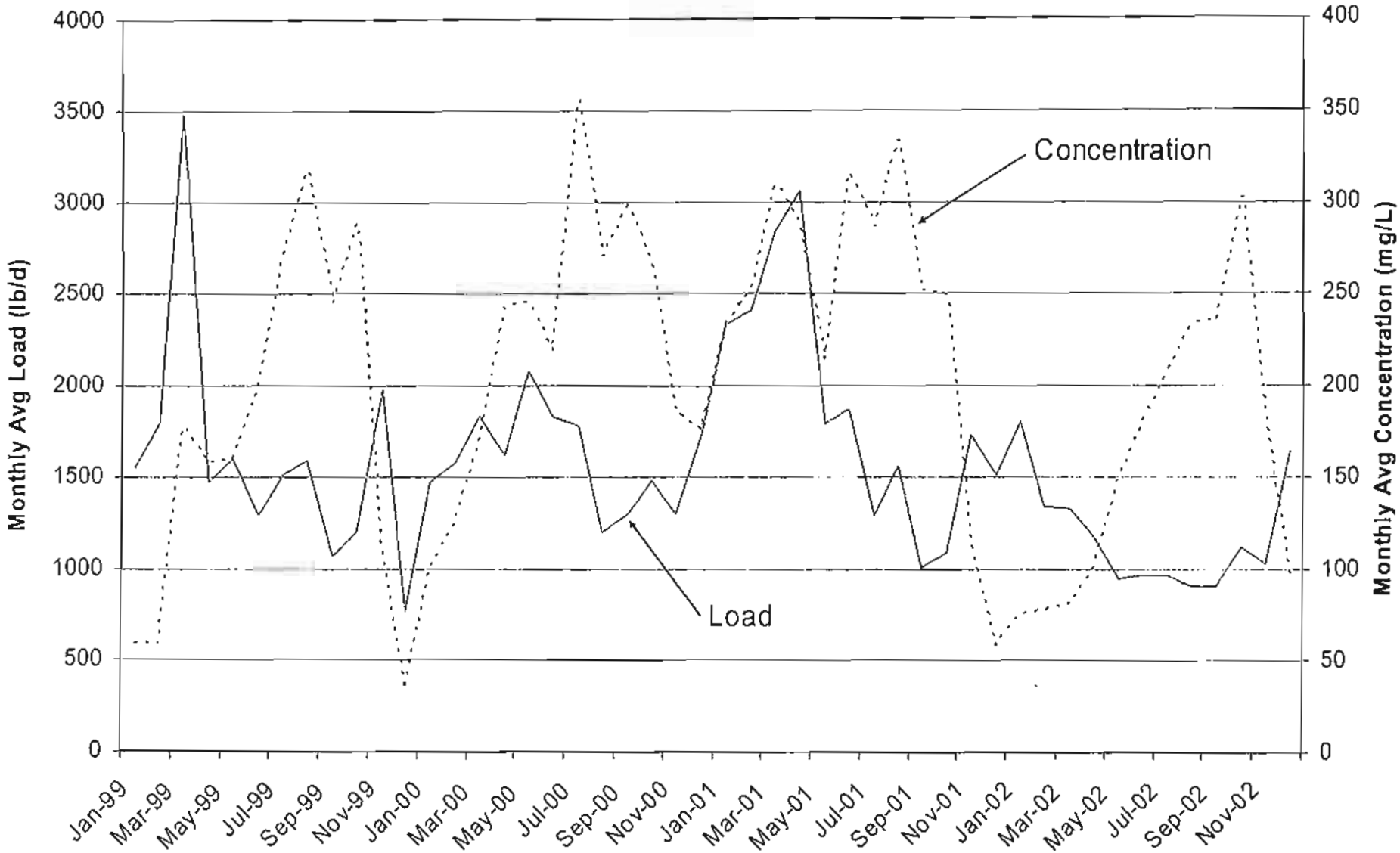
PIF Estimate
Flow Probability Analysis

FIGURE 6.5



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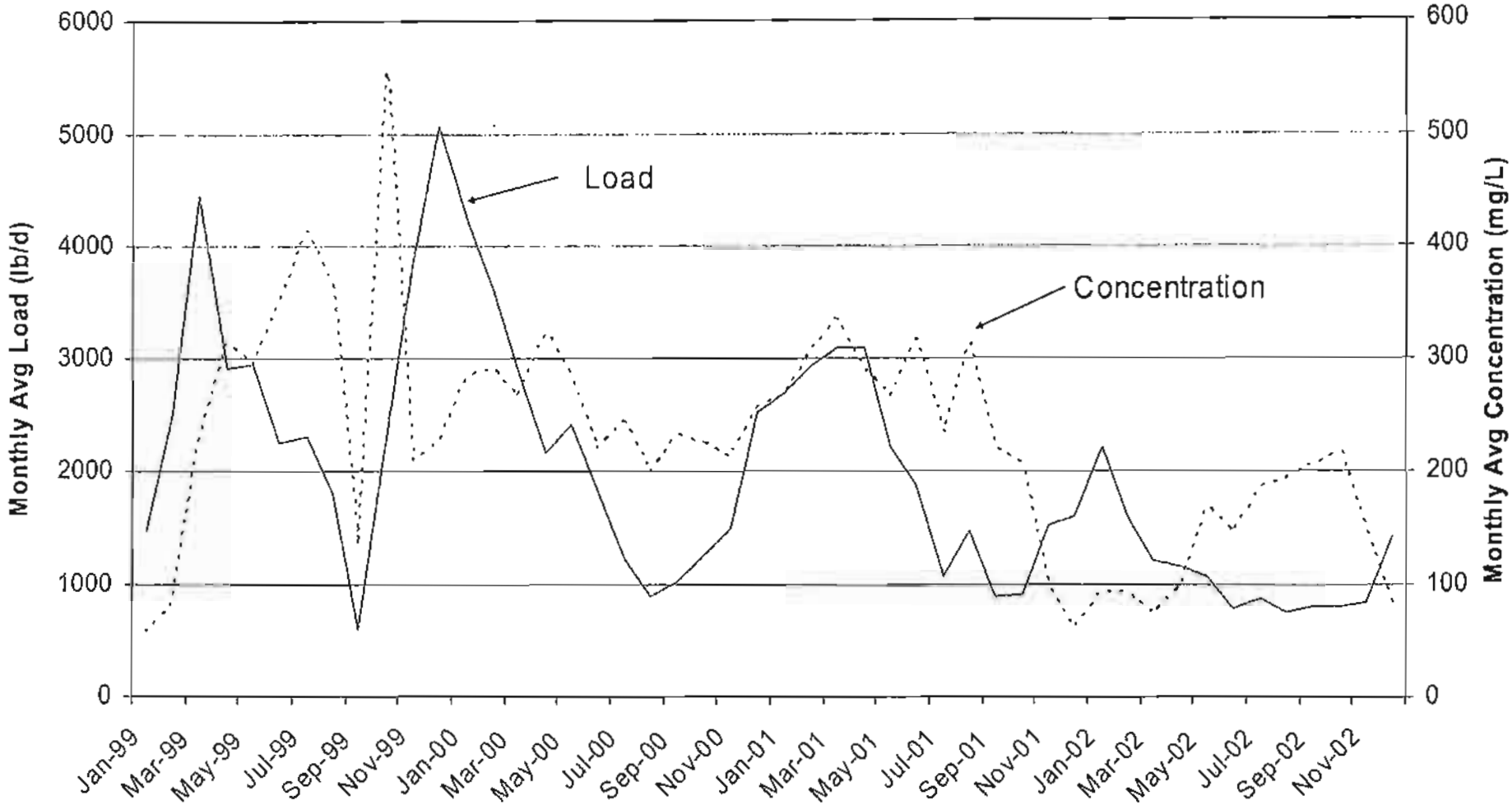
Influent BOD Loads

FIGURE 6.6



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Influent TSS Load

FIGURE 6.7

both TSS and BOD values. The average wet weather and dry weather values for BOD_5 and TSS are summarized on Table 6.4.

The average dry weather values (ADWF) indicated on this table are within normal ranges for communities similar to Tillamook while the average wet weather values (AWWF) tend towards the upper range of normal.

Table 6.4: 2002 BOD_5 and TSS Values

	Load (ppd)	Concentration (mg/l)	Per Capita Contribution (ppcd)*
AWWF BOD_5	1,384	101	0.30
ADWF BOD_5	966	218	0.21
Annual Average BOD_5	**1,175**	**159**	**0.25**
AWWF TSS	1,404	97	0.30
ADWF TSS	841	186	0.18
Annual Average TSS	**1,122**	**142**	**0.24**

Note:

* = Per capita values are based on an estimated 2003 population of 4,625 persons.

One possible explanation for the elevated BOD and TSS levels (in ppcd) during wet weather months is that sludge was normally wasted from the primary clarifier to the primary digester over a 12-hour period during one of two weekly sampling periods. When this occurs, digester supernatant, high in BOD and TSS, is recycled to the influent pump station. As a result, influent samples taken during this period can artificially elevate the BOD and TSS concentration. When used as the basis for calculating the monthly loading, these values can skew averages.

6.3 Future Wastewater Flows

Based on the information presented in the preceding sections, it is obvious that surface runoff and groundwater resulting from rainfall have a large impact on plant flow during wet weather months. It is important to predict future flows with some degree of accuracy as sizing of future conveyance structures and treatment components are based on projected seasonal averages, peak daily, and peak hourly flow rates. A good estimate of future flows and loading characteristics are also required to determine treatment requirements to meet discharge limits. Examples of conveyance system and treatment plant components and their corresponding design flow rate include the following:

- **Average Daily Dry Weather Flow (ADWF):** Since this flow is normally the lowest of all categories of interest, it normally does not impact the peak capacity design consideration of any collection system or treatment components. However, low flows must be considered in order to insure that adequate scouring velocities are maintained in channels, retention time in treatment basins is not so excessive as to cause anoxic conditions and that pump station wet wells do not retain sewage for so long that hydrogen sulfide production occurs. For plants using trickling filters, this flow is important to determine the required capacity of recirculation systems. Also, discharge

limits are often more stringent during low flow seasons requiring more efficient removal of TSS and BOD. Average flows are also used in O&M calculations to estimate chemical costs, sludge production, and energy costs.

- **Average Daily Wet Weather Flow (AWWF):** As this is not a peak flow, it does not impact maximum capacity design considerations. The primary considerations related to this flow would be solids digestion capacity and O&M planning. This flow serves as the basis for wet weather discharge limits.

- **Peak Monthly Flows (MMDWF and MMWWF):** These flows are important when sizing biological treatment capacity, certain chemical feed systems, equalization basins, and chemical storage facilities.

- **Peak Daily Flow (PDF):** Depending on the permitting agency and the treatment requirements, it may not be required to provide biological treatment capacity for this flow; however, conveyance components including interceptor capacity, collection system and influent pump station capacity, and headworks facilities are normally designed to handle this flow with out surcharging. Primary and secondary clarifiers are normally designed to provide removal at this flow rate. In addition, disinfection facilities will often be required to adequately treat this flow.

- **Peak Hourly Flow (PHF):** While this flow normally occurs once per year by definition, it is possible to receive flows of similar magnitude on several occasions in any given wet weather season. Normally, permitting agencies do not require full treatment of this flow. However, with the introduction of federal rules on CMOM regulations over the next few years, conveyance systems must be designed to handle this flow rate without overflows. Components requiring adequate hydraulic capacity to handle this flow would be interceptors, collection system and influent pump stations, headworks, and conveyance pipes and channels within the treatment plant.

6.3.1 Flow Projections

Flow projections for the Tillamook wastewater system consists of two distinct components: 1) base flow contribution from domestic and commercial discharges, and 2) an I/I contribution resulting from surface runoff and groundwater. Different approaches will be taken to arrive at the various flow rates from the existing collection system and the anticipated expansion area as a newly constructed sewer system will have much different I/I characteristics than one that is, on average, over 50-years old. The one flow of interest that should exhibit similar characteristics for both systems is the ADWF. This is because I/I has minimal effect on this flow rate and both systems should receive a similar per capita contribution. For Tillamook, the existing ADWF is approximately 119 gpcd. Projecting other flow rates for the existing and future collection systems requires different approaches.

6.3.2 Existing System Flow Projections

The existing collection system was analyzed in preceding sections and flows are summarized in Table 6.4. As it is unlikely that significant additional sewer construction will occur in the existing service area over the planning period, it will be assumed that the current I/I rate will not

increase. This is a valid assumption since Facility Plan recommendations will include rehabilitation of the existing system over the planning period to reduce the overall I/I rate and any new sewer lines added will exhibit far better resistance to I/I than existing pipes. In addition, the existing service area is close to build-out density and will require few if any new sewer connections.

Flow projections for future connections will be based on a base rate of 120 gpcd plus a small peaking factor. The proposed peaking factors, presented in Table 6.5, account for non-I/I peaks. Table 6.6 summarizes the projected flows for the existing service area based on these peaking factors.

Table 6.5: Future Connections Flows and Peaking Factors

Flow	Assumed Peaking Factor	Per Capita Contribution (gpcd)
ADWF	1.00	120
MMDWF	1.25	150
AWWF	1.50	180
MMWWF	2.00	240
PDF	3.00	360
PIF	4.00	480

Table 6.6: Flow Projections for the Existing Service Area

Year	Population	ADWF[a] (MGD)	MMDWF[b] (MGD)	AWWF[b] (MGD)	MMWWF[b] (MGD)	PDF_5[b] (MGD)	PIF_5[b] (MGD)
2003	4,625	0.55	1.50	1.86	3.17	5.97	8.47
2005	4,940	0.59	1.55	1.92	3.25	6.08	8.62
2010	5,190	0.62	1.58	1.96	3.31	6.17	8.74
2015	5,455	0.65	1.62	2.01	3.37	6.27	8.87
2020	5,470	0.69	1.67	2.06	3.44	6.37	9.01
2025	6,010	0.72	1.71	2.11	3.50	6.47	9.13

Notes:

(a) ADWF is assumed to equal the "base flow" contribution from domestic and commercial customers. Based on per capita contribution of 120 gallons per day.

(b) 2003 flow estimate plus increase corresponding to projected population increase.

6.3.3 Expansion System Flow Projections

As expansion of the service area will require construction of a new collection system, it is assumed that I/I contribution will be much lower than for the existing system. This is a valid assumption in that newer systems are more water tight than older systems as pipes, gaskets and sealing systems have improved over time. In addition, progressive settlement of pipes, manholes and lateral lines, which cause pipes to leak or fail, is not present in a newly constructed system and can be minimized with proper construction techniques.

There are several approaches that can be taken to arrive at an I/I estimate for a proposed collection system. Among those commonly used are an I/I rate expressed in gallons/inch-diameter/ft of pipe, gallons per capita served, and gallons per acre served. Based on a recommendation made by DEQ for this particular project, 1,000 gallons per acre served will be used as the basis for estimating I/I for the expansion area. It is assumed that each acre of the newly developed area would house 10 people. This is based on a UGB minimum lot size of 7,500 square feet per single family house or about 10 persons (4 houses x 2.5 person per house) per acre including streets and open space. This I/I rate was applied to all the flow rates of interest excluding ADWF and the PIF. This approach assumes that the I/I rate would be constant across all wet weather months. It is simplistic in that it assumes that I/I does not increase with increasing rainfall but it is also conservative in that I/I for all wet weather months is assumed to equal I/I for the MMWWF.

A peaking factor was applied to the ADWF to arrive at the PIF. For the expansion area, a peaking factor was referenced from *Metcalf and Eddy, Water Engineering, 3rd Ed.* For a system with an average flow rate less than 0.50 MGD, it is common to apply a peak hour/average flow factor equal to 4. The flows of interest for the expansion area are presented in Table 6.7.

Table 6.7: Flow Projections for the Expansion Service Area

Year	Population[a]	ADWF[b] (MGD)	MMDWF[c] (MGD)	AWWF (MGD)	MMWWF (MGD)	PDF (MGD)	PIF (MGD)
2003	NA	NA	NA	NA	NA	NA	NA
2005	176	0.02	0.04	0.05	0.06	0.08	0.10
2010	1,260	0.15	0.32	0.35	0.43	0.58	0.73
2015	1,340	0.16	0.34	0.38	0.46	0.62	0.78
2020	1,425	0.17	0.36	0.40	0.48	0.66	0.83
2025	1,505	0.18	0.38	0.42	0.51	0.69	0.87

Notes:

(a) Projected population of UGA minus the 250 persons existing in 2003 (existing 250 persons accounted for in "Existing Service Area", Table 6.6).

(b) ADWF is assumed to equal the "base flow" contribution from domestic and commercial customers. Based on per capita contribution of 120 gallons per day.

(c) I/I applied is based on 1,000 gal/acre/day and 10 persons per acre.

6.3.4 Combined Existing and Anticipated Expansion Area Flow Projections

Projected flows for the combined service area are presented on Table 6.8.

Table 6.8: Flow Projections for the Combined Service Area

Year	Population	ADWF (MGD)	MMDWF (MGD)	AWWF (MGD)	MMWWF (MGD)	PDF_5 (MGD)	PIF_5 (MGD)
2003	4,625	0.55	1.50	1.86	3.17	5.97	8.47
2005	5,116	0.61	1.59	1.97	3.31	6.16	8.72
2010	6,450	0.77	1.90	2.31	3.73	6.75	9.47
2015	6,795	0.82	1.96	2.38	3.82	6.89	9.65
2020	7,165	0.86	2.02	2.46	3.92	7.03	9.83
2025	7,515	0.90	2.08	2.53	4.01	7.16	10.01

6.4 Future Plant Loads

Because the historical wastewater loading data is very limited (only one year of data), future plant loading will not be based exactly on the historical values. Future plant loading will be based on values of 0.2 ppcd for both BOD and TSS. These are average per capita load values typically used for design purposes for small communities without a significant industrial wastewater contribution, and are close to the historical dry weather loading values for the City presented in Section 6.2. Projected wastewater loads, based on the projected population, are presented in Table 6.9.

Table 6.9: Future Plant Loads

	Population			Plant Load[a]	
Year	City Limits	UGA	Total	TSS (ppd)	BOD (ppd)
2003	4,375	250	4,625	925	925
2005	4,690	426	5,116	1,025	1,023
2010	4,940	1,510	6,450	1,290	1,290
2015	5,205	1,590	6,795	1,359	1,359
2020	5,490	1,675	7,165	1,433	1,433
2025	5,760	1,755	7,515	1,503	1,503

Note:

(a) Plant load is based on combined city Limits plus UGA population x 0.2 ppd.

6.5 Summary of Design Flows and Loads

The flow and load values used for the sizing and design of the proposed improvements are summarized in Table 6.10 below. These include the projected flows from both the City Limits and the Urban Growth Area.

Table 6.10: Summary of Design Flows and Loads

Year	2003	2005	2010	2015	2020	2025
ADWF (MGD)	0.55	0.61	0.77	0.82	0.86	0.90
MMDWF (MGD)	1.50	1.59	1.90	1.96	2.02	2.08
AWWF (MGD)	1.86	1.97	2.31	2.38	2.46	2.53
MMWWF (MGD)	3.17	3.31	3.73	3.82	3.92	4.01
PDF (MGD)	5.97	6.16	6.75	6.89	7.03	7.16
PIF (MGD)	8.47	8.72	9.47	9.65	9.83	10.01
TSS (ppd)	925	1,025	1,290	1,359	1,433	1,503
BOD (ppd)	925	1,023	1,290	1,359	1,433	1,503

Section 7: Basis of Planning

7.1 Regulatory Requirements

The treatment, disposal, and reuse of wastewater effluent and sludge is regulated by the U.S. Environmental Protection Agency (EPA) and administered in the State of Oregon by DEQ. Wastewater discharge to surface water and sludge disposal are regulated through the National Pollutant Discharge Elimination System (NPDES) permitting process. The EPA (through DEQ) is required to coordinate with other federal and state agencies, including the NOAA, the U.S. Army Corps of Engineers, the U.S. Fish and Wildlife Service, the Oregon Department of Human Services (DHS), and the Oregon Department of Fish and Wildlife (ODFW). This chapter is a summary of the pertinent regulations from these agencies as incorporated into the City of Tillamook NPDES Permit.

7.1.1 Effluent Discharge Regulations

7.1.1.1 Surface Water Discharge of Effluent

Discharging treatment plant effluent to surface water requires an NPDES Permit from DEQ. This discharge method is governed by OAR 340-41. The City has an existing NPDES permit but may be requesting changes to the permit.

7.1.1.2 NPDES Permit

The current NPDES Permit for the City was issued on 6 January 2003. The permit incorporates compliance with current water quality standards including Total Maximum Daily Load (TMDL) limitations developed for the Tillamook Bay Watershed. On 1 November 2004, the Oregon DEQ modified the NPDES permit in accordance with OAR 340-041-0061(10)(D). The NPDES Permit including the November 2004 modification is provided in Appendix E. The permit limits are outlined on Table 7.1.

Table 7.1: NPDES Permit - Effluent Discharge Limitations

Parameter	Average Effluent Monthly (mg/l)	Concentrations Weekly (mg/l)	Monthly Average (lb/day)[a]	Weekly Average (lb/day)[a]	Daily Maximum (lbs)[a]
May 1 – October 31					
BOD_5	20	30	177	265	354
TSS	20	30	177	255	354

Fecal Coliform Bacteria	Shall not exceed a monthly geometric mean of 200 organisms per 100 ml. No single sample shall exceed 400 organisms per 100 ml.
BOD_5 and TSS Removal Efficiency	Shall not be less than 85% based on a monthly average for BOD5 and shall not be less than 85% based on a monthly average for TSS.

Parameter	Average Effluent Concentrations Monthly (mg/l)	Weekly (mg/l)	Monthly Average (lb/day)[a]	Weekly Average (lb/day)[a]	Daily Maximum (lbs)[a]
November 1 – April 30					
BOD_5	30	45	530	795	1060
TSS	30	45	530	795	1060
Fecal Coliform Bacteria	Shall not exceed a monthly geometric mean of 49 organisms per 100 ml. No single sample shall exceed 151 organisms per 100 ml.				
BOD_5 and TSS Removal Efficiency	Shall not be less than 85%, except that when the monthly average for BODs or TSS is less than 167 mg/L the percent removal shall not apply, provided that the treatment plant shall be operated as effectively as possible.				
Year-Round					
pH	Shall be within the range of 6.0 to 9.0.				
Chlorine Residual	Shall not exceed a Maximum Daily Limit of 0.02 mg/l. Shall not exceed a Monthly Average limit of 0.01 mg/l.				
Excess Thermal Load	Shall not exceed a 7-day moving average of the daily maximums of 141 million BTU/day. Shall not exceed a monthly average of 62 million BTU/day.				
Notes	[a] Average dry weather design flow to the facility equals 1.06 MGD. Per OAR 340-0061(10)(A) the summer mass load limits are based upon average dry weather design flow to the facility. Per OAR 340-041- 0061(10)(B), the winter mass load limits are based upon average wet weather design flow to the facility equaling 2.12 MGD. Per OAR 340-041-0061(10)(C), the daily mass load limit is suspended on any day in which the flow to the treatment facility exceeds 2.12 MGD (twice the design average dry weather flow).				

Dry Weather Conditions: Existing MMDWF is estimated at 1.50 MGD. With the existing BOD and TSS load limitation of 177 lb/d, the required BOD/TSS effluent concentration from the WWTP during the dry weather maximum month must be 14 mg/l, or less. Future design MMDWF is estimated at 2.08 MGD. With the existing BOD and TSS load limitation of 177 lb/d,

the required BOD/TSS effluent concentration from the WWTP during the future design dry weather maximum month must 10 mg/l, or less.

Wet Weather Conditions: The current Maximum Monthly Wet Weather Flow (MMWWF) is estimated at 3.17 MGD. At this flow rate, the BOD and TSS concentration must be 20 mg/l or less to meet the effluent load limit of 530 lb/d. The future MMWWF design flow is 4.01 MGD. At this flow rate, the BOD and TSS concentration must be 15 mg/l or less to meet the effluent load limit of 530 lb/d.

7.1.2 Effluent Water Quality Study

7.1.2.1 Water Quality Study

Kennedy/Jenks completed a mixing zone analysis for the existing WWTP and discharge structure. The *Effluent Water Quality Study* was completed in February 2003 and finalized in October 2003. Findings of the study were used to assess potential impacts from effluent constituents. A reasonable potential analysis was done to evaluate the possibility of exceeding the water quality standards in the Trask River. A summary of the study is presented herein. Significant excerpts from the study are provided in Appendix B.

The 1-hour, average minimum flow used to calculate the zone of initial dilution (ZID) in the river is 6.72 CFS and that the worse-case, 4-day average flow used to calculate dilution at the mixing zone (MZ) is 269 CFS.

For the future average dry weather flow (ADWF), the predicted dilution factor is 1.4 at the ZID. With the outfall in its present configuration, a dilution factor of 6.5 is predicted at the regulatory MZ. If the outfall is reconfigured by lowering it 5-feet (to the river bed) and extending it approximately 10-feet, the predicted dilution factor would be 19. Both predictions are based on the future Average Dry Weather Flow (ADWF).

The results of the Reasonable Potential Analysis show that copper, silver and total cyanide have the potential to exceed the acute 1-hour criteria maximum concentration (CMC). Aluminum, silver and total cyanide have the potential to exceed the chronic 4-day criteria continuous concentration (CCC). However, it is important to note that background concentrations in the Trask River for both aluminum and silver are found to be higher than the chronic 4-day CCC levels.

Findings from the Effluent Water Quality Study indicate the following:

1. Outfall location and Type – Steps should be taken to improve the existing outfall. A preliminary estimate indicates that a dilution factor of 19 is possible by relocating the outfall to the bottom of the river near the bank.

2. Ammonia - With an increased dilution factor, concerns over chronic toxicity of ammonia will be reduced although monitoring or limits on ammonia could still be imposed.

3. Temperature – Based on existing data and temperature standards, there does not appear to be a temperature issue.

BOD Dilution – With the outfall at its present configuration, minimum dilution requirements stipulated by Oregon Administrative Rules could require a treated effluent BOD of 6.5 mg/l or less during worse case conditions. With an improved outfall, it is possible to increase the dilution by 3-fold thereby increasing the effluent BOD limit to 19 mg/l under worse case conditions.

7.1.2.2 Land Application of Effluent

Land application of treatment plant effluent requires a Water Pollution Control Facility Permit from DEQ, with requirements based on state water quality regulations. Reuse of effluent by land application is governed by OAR 340-55 (reuse) and OAR 340-40 (groundwater quality). Requirements for reuse of effluent depend on the rainfall patterns, use of the land, the site's accessibility to the public and the available buffer areas around the site.

The City of Tillamook currently does not have a WPCF Permit or reuse permit. The subsurface conditions and precipitation patterns in Tillamook are typical of coastal communities and are not at all favorable for land application. The area is typified by high groundwater and high rainfall. Based on these factors, land application of effluent is not a viable alternative in Tillamook

7.1.2.3 On-Site Sewage Disposal

On-site sewage disposal (infiltration basin) is regulated by the DEQ through OAR 340-71 and 340-72. These regulations mandate minimum separation distances, effective soil depths, and minimum drain field area.

The use of on-site disposal for the treatment plant effluent is not viable due to the subsurface conditions and rainfall patterns mentioned above, as well as the large areas of land that would be required.

7.1.3 Sludge Management Regulations

Sludge is a mixture of solids and liquid that is an end-product of the wastewater treatment process. The material is commonly referred to as biosolids and it is disposed of by applying it to agricultural or forest land. The Code of Federal Regulations (40 CFR 503) defines standards for three measures of biosolids quality:

- Pathogens
- Vector attraction (the tendency of the sludge to attract rodents, insects and other organisms that can spread disease)
- Trace elements.

7.1.3.1 Pathogen Requirements

Pathogen requirements define two classes of biosolids, Class A and Class B. Class A is the higher standard and requires destruction of pathogens to below detectable levels before disposal. Class B requirements call for reducing pathogens to levels that do not pose a threat to

the public before disposal and applying the biosolids to land in such a way that pathogens are further reduced.

To be classified as Class A, biosolids must be treated using one of the EPA's *Processes to Further Reduce Pathogens* (PFRP), or an equivalent process. These processes include composting, heat drying, heat treatment, thermophilic aerobic digestion, beta ray irradiation, gamma ray irradiation and pasteurization. Regardless of the process used, Class A biosolids must not exceed maximum allowable fecal coliform density or salmonella bacteria density.

Class B biosolids must be treated using one of the EPA's *Processes to Significantly Reduce Pathogens* (PSRP), or an equivalent process. These processes include aerobic digestion, air drying, anaerobic digestion, composting, and lime stabilization.

7.1.3.2 Vector Attraction Requirements

Biosolids must meet one of the following requirements for reducing vector attraction if they are to be applied to land without restrictions:

- Volatile solids in the sludge reduced by a minimum of 38 percent during digestion.
- Laboratory testing to demonstrate sufficient volatile solids destruction during digestion.
- The specific oxygen uptake rate for sludge treated by aerobic digestion less than or equal to 1.5 mg oxygen per hour per gram of total solids at a temperature of 20° C.
- Aerobic processing of sludge for a minimum of 14 days with an average temperature of at least 45° C and a minimum temperature of 40° C.
- Alkali addition shall raise the pH of the sludge to a minimum of 12 for two hours and maintain the pH at a minimum of 11.5 for an additional 22 hours without additional alkali.
- Moisture reduction (75% to 90% depending on level of treatment).
- Biosolids injection below the ground surface.
- Incorporation of biosolids into the soil.
- Covering biosolids at the end of each operating day.

The use of the land where the biosolids are applied is restricted if vector attraction reduction is achieved by measures such as injecting the biosolids below the surface of the land, or disposing of them on the surface and incorporating them into the soil within six hours.

7.1.3.3 Trace Elements

Eight elements typically found in biosolids have been identified as critical. Four limits have been set for each of these trace elements:

1. Ceiling Concentrations – maximum pollutant concentration for sludge sold or given to the public.
2. Cumulative Pollutant Loading Rates – maximum loading per hectacre for sludge applied to agricultural land or forest land.
3. Pollutant Concentrations – maximum pollutant concentration for sludge applied to a lawn or home garden.
4. Annual Pollutant Loading Rates – maximum load per hectacre per year for sludge sold or given away

7.1.3.4 Biosolids Management Plan

A Biosolids Management Plan serves as a central administrative tool to guide the production, storage, transportation, and land application of biosolids for beneficial use in Oregon. All wastewater treatment facilities that apply biosolids to land must have a Biosolids Management Plan approved by the DEQ.

The City of Tillamook has a DEQ approved Biosolids Management Plan that was written in October 2001 and amended in July 2002. The plan meets requirements for Class B pathogen reduction via anaerobic digestion and vector attraction reduction through incorporation of the solids in the soil. Through 2001, biosolids were land applied on various sites within the County. The City also takes biosolids to the Port of Tillamook receiving station for disposal during those times when land application is not practical. A copy of the current Biosolids Management Plan is included in Appendix F.

The City will likely continue to treat and dispose of biosolids in accordance with their current Biosolids Management Plan during the next 20-year planning period.

7.1.4 Septage Management Regulations

If septic tanks are used to remove solids from wastewater flows, the septage must be disposed of at a facility permitted to treat septage. The 40 CFR 257 regulations outline the rules pertaining to the treatment and disposal of industrial/commercial/domestic septage.

Treatment and disposal of septage residual solids must meet the same pathogen reduction and vector attraction standards as sewage sludge.

The City of Tillamook has approximately 63 customers using STEP systems. Septage from these tanks is periodically pumped to the City's Vactor truck and introducing at the headworks of the treatment plant. Planned improvements will allow septage to be introduced directly to the anaerobic digester following screening.

7.1.5 Capacity Management Operation and Maintenance

CMOM is a program currently proposed (but not yet required) by the EPA. The CMOM program is intended to prevent sanitary sewer overflows (SSOs) and WWTP overloading through proactive management of the collection system.

The rule is currently in pre-draft form and is scheduled to be published in draft form for public comment in the near future. This rule has been "imminent" for several years and is likely to be adopted within the next 5 to 10 years. When adopted, the CMOM program would be administered by DEQ and incorporated into NPDES permits. In a few cases, DEQ has already incorporated similar requirements.

It is most probable that the CMOM rule will not be accompanied by any funding mechanisms. The cost of the program will be paid by local municipalities and ratepayers.

7.1.5.1 Purpose of Program

The primary purpose of the program is to ensure that owners to take a proactive approach to provide adequate sewer maintenance/capacity to eliminate and prevent sewer overflows. The plan prohibits SSOs, but acknowledging that they may still occur, has mechanisms for reporting SSOs to both DEQ and the public. In an effort to eliminate overflows the program will require owners to undertake a thorough inventory, evaluation, and upgrades of the system where necessary. It will also require a maintenance plan to prevent future overflows.

7.1.5.2 Key Elements

The key elements of the program are:

- General Standards – Outlines general requirements of the permittee.
- Summary of Management Program – Written CMOM program with goals, organization chart, legal authority documentation, measures, and activities that are in place to maintain the system, design and performance standards, and program monitoring records.
- Overflow Response Plan – Emergency response plan for SSOs.
- System Evaluation & Capacity Assurance Plan – Documentation of hydraulic deficiencies and prioritization of scheduled improvements.
- Program Audit – Evaluation of program compliance with NPDES permit.
- Certification
- Communication – System for communicating between Owner, customers, regulators, and other interested parties.

A large portion of the effort needed to address the Key Elements will be required by Elements 2 and 4 of the Collection System Improvements. The most cost effective method of addressing the requirements for documenting and then updating and managing all of the data (pipe type, location, slope, condition, inspection schedules, pump station maintenance, etc) would be a GIS based or similar model of the collection system.

7.1.6 Regulation References

- Oregon Administrative Rules (OAR) can be found at: http://arcweb.sos.state.or.us/banners/rules.htm
- Oregon Revised Statues (ORS) can be found at: http://www.leg.state.or.us/ors/
- Code of Federal Regulations (CFR) can be found at: http://www.access.gpo.gov/nara/cfr/cfr-table-search.html
- US EPA Wastewater Home Page can be found at: http://www.epa.gov/owm/index.htm
- DEQ Water Quality Home Page can be found at: http://www.deq.state.or.us/wq/wqpermit/wqpermit.htm
- DEQ Guide Documents can be found at: http://www.deq.state.or.us/wq/wqrules/Guidance.htm

7.2 Plant Reliability Criteria

The Tillamook WWTP expansion will be equipped to meet Class I Reliability Requirements, as defined in EPA's Technical Bulletin, "Design Criteria for Mechanical, Electrical, and Fluid System Component Reliability," EPA 430-99-74-001. A summary of those requirements is as follows:

- Mechanically Cleaned Bar Screens. A backup bar screen, designed for mechanical or manual cleaning, shall be provided. Facilities with only two bar screens shall have at least one bar screen designed to permit manual cleaning.
- Pumps. A backup pump shall be provided for each set of pumps performing the same function. The capacity of the pumps shall be such that, with any one pump out of service, the remaining pumps will have the capacity to handle the peak flow.
- Primary Sedimentation Basins. The units should be sufficient in number and size so that, with the largest-flow-capacity unit out of service, the remaining units should have a design flow capacity of at least 50 percent of the total design flow.
- Final Sedimentation Basins and Trickling Filters. The units shall be sufficient in number and size so that, with the largest-flow-capacity unit out of service, the remaining units shall have a design flow capacity of at least 75 percent of the total design flow.
- Activated Sludge Process Components.
  - Aeration Basin. A backup basin will not be required; however, at least two equal-volume basins shall be provided.
  - Aeration Blowers or Mechanical Aerators. There shall be a sufficient number of blowers or mechanical aerators to enable the design oxygen transfer to be maintained with the largest-capacity-unit out of service. It is permissible for the

backup unit to be an uninstalled unit, provided that the installed units can be easily removed and replaced. However, at least two units shall be installed.

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

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

- Two separate and independent power sources, either from two separate utility substations or from a single substation and an on-site generator. The backup power supply shall be sufficient to operate all vital components during peak wastewater flow conditions, including critical lighting and ventilation.

7.3 Basis for Cost Estimate

Installed construction cost of treatment units is determined by calculating material take-offs or by procuring quotations from equipment suppliers and multiplying these times appropriate unit costs. The estimated total capital cost of treatment units is determined by multiplying the installed construction cost x non-construction cost factor (NCCF). The basis for the NCCF used in estimating is 1.9047 (1.9047 = 1.15 x 1.0617 x 1.20 x 1.30) and consist of items listed in Table 7.2 below.

Table 7.2: Basis for Cost Estimates

Item		Cost as Percent of Construction Cost
Taxes		0%
Contractor Overhead and Profit		15%
Escalation to Midpoint of Construction		6.17%
Contingency		20%
Engineering/Legal/Administrative		30%
Design Engineering	13%	
Construction and Startup Services	10%	
Administrative and Legal Expenses	3%	
Other Professional Services (Survey, Geotechnical)	4%	

7.4 Basis for Flow and Load Projections

7.4.1 Conveyance System

Flow projections for conveyance system components include both 20-year projections for mechanical equipment (pump stations) and full build-out projections for pipe sizing. Conveyance system flows are based on land use information provided by the county and on the following assumptions:

- 2.52 capita/residence, zones R-5 and R-7.5
- 1.764 capita/residence, zone R-0 (70% of R-5 and R-7.5 density)
- 120 gpd/capita, average dry weather flow
- 15.4 peak hour to average dry weather flow, existing system
- 11 peak hour to average dry weather flow, future
- 4.0 peak flow peaking factor, future development
- 2.0 peak flow peaking factor, commercial and industrial
- 2,000 gal/acre/day commercial and industrial flows.

7.4.2 Wastewater Treatment Facilities

Improvements for the conveyance and treatment facilities are based on 20-year projected flows and loads. Future flows are based on population projections (see Section 3), on the *DEQ Guidelines,* and on the following assumptions:

- 120 gpd/capita, average dry weather flow
- 1,000 gal/acre/day I/I flows for future development
- Future development peaking factors as shown in Table 7.3.

7.5 Basis for Influent Pumping and Flow Conveyance

The flows and loads developed in Section 6 will serve as the basis for sizing the influent pumping, conveyance structures, and treatment capacity of the WWTP. The proposed criteria for these unit processes are presented in Table 7.3.

Table 7.3 – Design Criteria for Influent Pumping and Flow Conveyance

Process	No. Units	Design Basis	Capacity
Influent Pump Station	1	Peak Hour Flow	10 MGD
Headworks	1	Peak Hour Flow	10 MGD
Primary Clarifier	2	Peak Hour Flow	10 MGD
Aeration Basin Hydraulic Capacity	2	Peak Dry Weather Hour Flow	5.0 MGD
Secondary Clarifiers Hydraulic Capacity	2	Peak Dry Weather Hour Flow	5.0 MGD
Chlorine Contact Tank	2	15-minutes at Peak Hour Flow	10 MGD

The criteria presented in Table 7.3 were developed to provide the level of treatment necessary to meet the anticipated effluent limits for BOD, TSS, and ammonia, and to eliminate overflows of the collection system. By providing sufficient influent pumping capacity to meet the predicted peak-hour flow event, overflows of the collection system can be avoided. Flow blending, or bypassing a portion of the primary effluent around the secondary process, is proposed to protect the secondary treatment system from hydraulic overload.

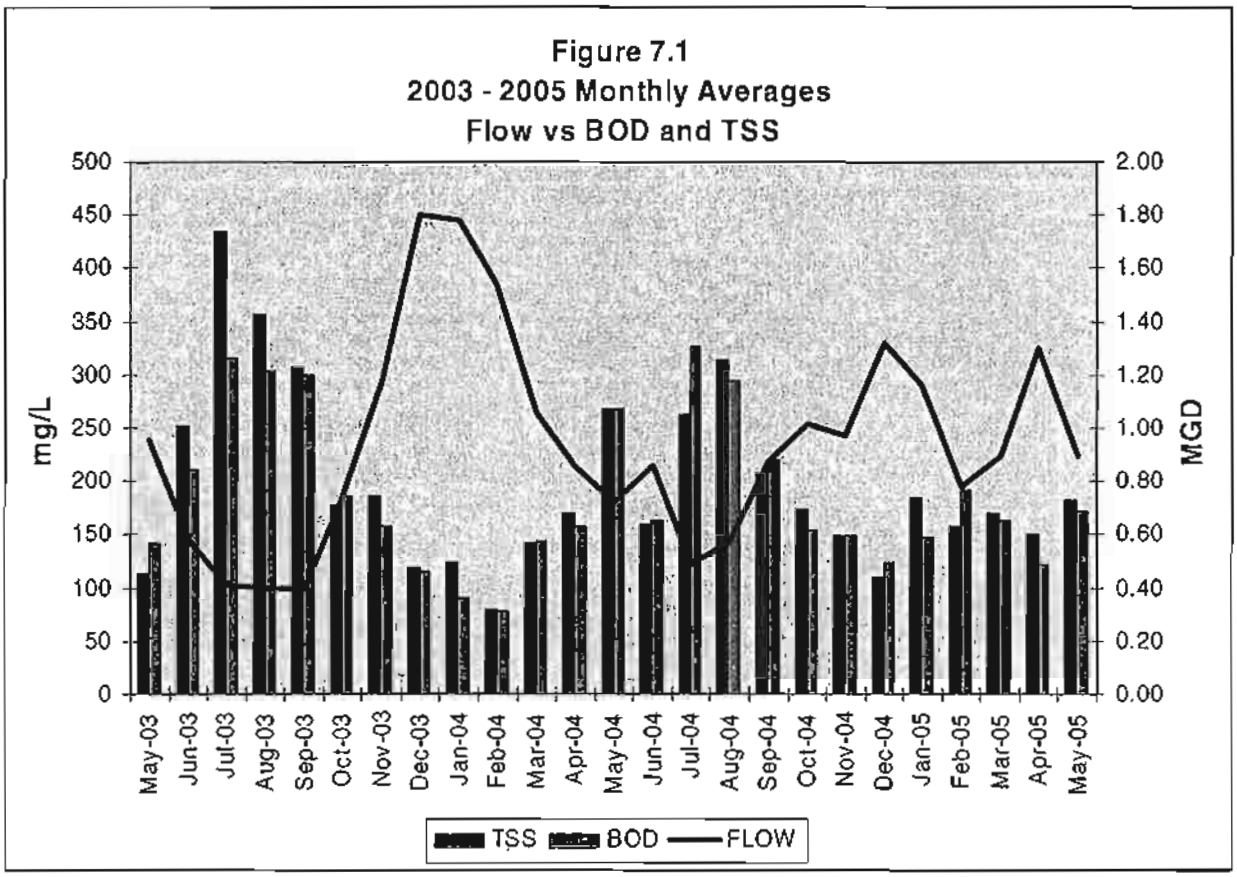
7.5.1 Flow Blending

Flow blending is generally recognized to be the practice of diverting a part of the peak wet-weather flow around the biological treatment process then combining effluent from all processes prior to disinfection and dechlorination. The goal is to protect the biological treatment process from excessive flows that could washout biological solids resulting in increased effluent TSS loading and decreased treatment efficiency. This is especially true of the Tillamook WWTP due to the extreme ratio of average to peak flow.

The following six criteria have been developed to ensure that flow blending will result in outfall permit compliance:

1. The final discharge effluent will meet NPDES permit discharge requirements.
2. The treatment scenario for peak-flow management is consistent with generally accepted practices and long-term design criteria.
3. All incoming flow must meet the equivalent of primary clarification.
4. The peak-flow treatment scenario is used only during peak wet weather when capacity of the biological treatment process is fully utilized.
5. Sufficient monitoring must be included in the NPDES permit to ensure compliance with applicable water quality based effluent limitations and the Clean Water Act.
6. The collection system must be properly maintained and operated.

Historical operating data from the Tillamook WWTP show, in general, that as flow increases during wet weather events, the influent BOD and TSS concentration decrease proportionally (see Figure 7.1). Therefore, during the extreme flows predicted for the peak hour, the influent loading will be dilute. When combined with the secondary effluent, the blended effluent should reliably meet the limits imposed by the NPDES permit.



All incoming flow will pass through the primary clarifier and subsequently flow through the bypass structure. The bypass structure will allow flow in excess of the peak dry weather flow to bypass the secondary treatment process. This will ensure that flow will not be bypassed except during peak events and that the capacity of the secondary process is fully utilized prior to the initiation of blending. The bypass flow and secondary effluent streams will be combined upstream of the chlorine contact tank ensuring that all flow through the plant is disinfected prior to discharge. Flow proportional effluent sampling downstream of the disinfection process will ensure that permit limits are being met.

The dry weather months (May through October) are considered critical with respect to effluent discharge to the Trask River. High stream flow in the Trask River during the other months provide for much greater dilution. For this reason, designing the secondary treatment units to meet the Dry Weather Peak Hour Flow (DWPHF) is proposed. It is worthy to reiterate that on days when extreme flows occur, the influent BOD and TSS concentration is proportionally low.

A value of 5 MGD was derived as the DWPHF. This value is based on a peaking factor of 2.5 times the 99.5% exceedance value for the dry weather max day flow. As shown on Figure 7.2 (*Monthly Dry Weather Flow Data 1999 – 2005*), the maximum daily flow exceeded 2.0 MGD on 3 occasions. Examination of the Daily Monitoring Reports for the days with flow exceeding 2.0 MGD shows that rainfall totals exceeded 2-inches and were preceded by rainfall of over 1-inch the previous day. The National Weather Service and Oregon State Climatologist do not compile rainfall exceedance statistics for the "dry weather months" so it is not possible to accurately quantify the 24-hour rainfall event that would trigger a WWTP flow in excess of 2 MGD. However, the flow data indicate that on average, WWTP flows have exceeded 2 MGD about once every 340 dry weather days, or roughly once every 2 years.

Components of the Facility Plan address collection system problems and improvements over the 20-year planning period to reduce wet weather flows to the WWTP.

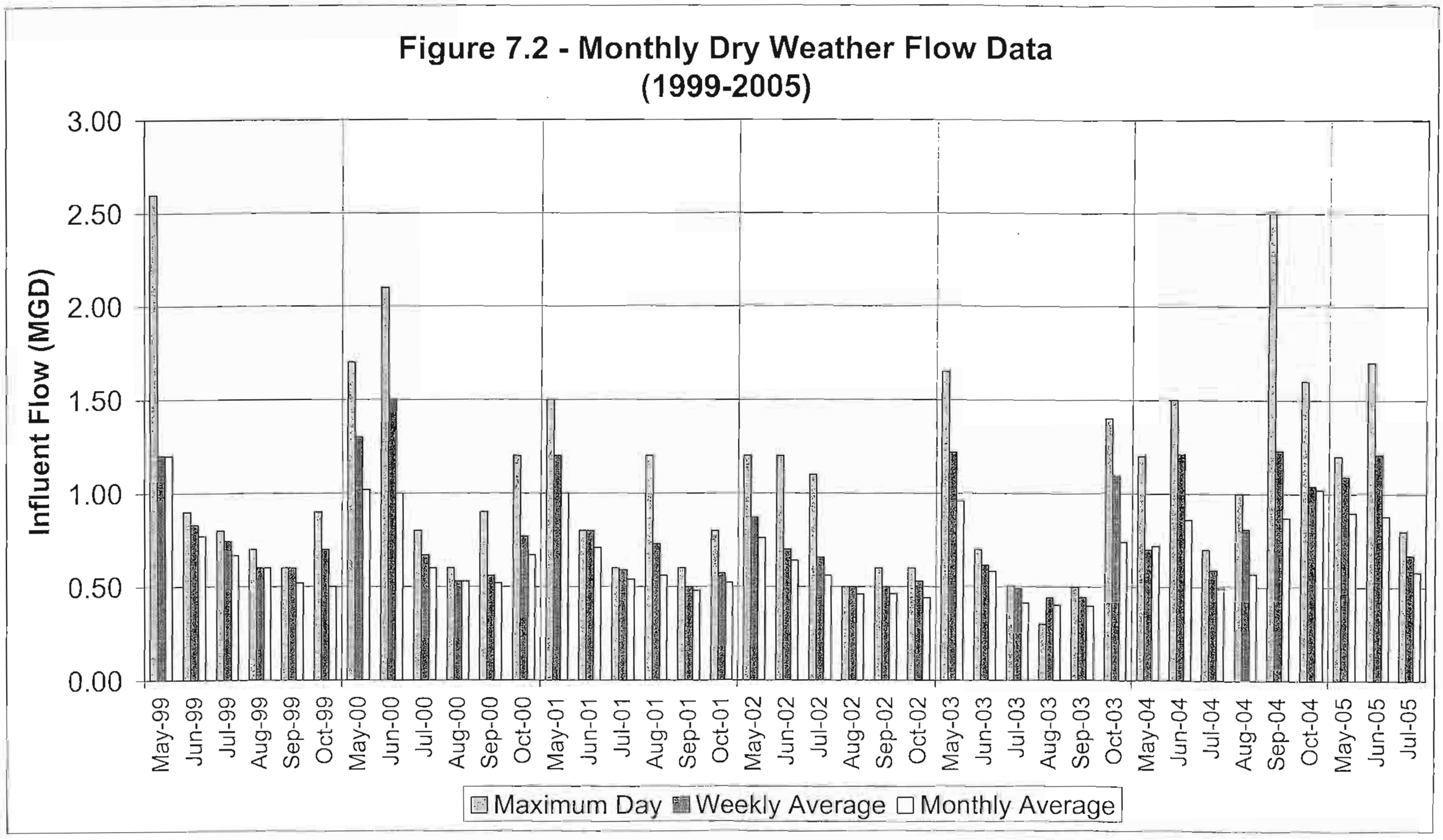


Figure 7.2 - Monthly Dry Weather Flow Data
(1999-2005)
Influent Flow (MGD)
3.00
2.50
2.00
1.50
1.00
0.50
0.00
May-99
Jun-99
Jul-99
Aug-99
Sep-99
Oct-99
May-00
Jun-00
Jul-00
Aug-00
Sep-00
Oct-00
May-01
Jun-01
Jul-01
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Oct-02
May-03
Jun-03
Jul-03
Aug-03
Sep-03
Oct-03
May-04
Jun-04
Jul-04
Aug-04
Sep-04
Oct-04
May-05
Jun-05
Jul-05
Maximum Day
Weekly Average
Monthly Average

Section 8: Collection System Alternatives

8.1 System-Wide Alternatives

Based on the collection system discussion in Section 4, the O&M programs and system-wide improvements discussed in this section are recommended.

8.1.1 I1: Smoke Testing, TV, and Maintenance Program

A systematic and planned O&M program is critical to effective operation of the conveyance system. As part of this program, smoke testing of all pipes over a 10-year period is recommended to check for illicit storm water connections and to discover possible areas of excessive I/I. A schedule to video all pipes over a 10-year period (approximately 10,000 feet per year) is also recommended. These activities would be incorporated under a CMOM program.

Funds should be allocated on an annual basis to address the deficiencies discovered during smoke testing and video monitoring. The money would be allocated on items such as pipe cleaning, removing root intrusion, lining pipes and laterals, and replacing manhole lids. Total estimated cost for this alternative, including a $40,000 allowance for maintenance, is approximately $80,000 per year.

If the smoke testing and maintenance (but not the video work) are done by city staff, it is estimated that this work will require two employees during four summer months, or 0.6 Full Time Equivalents (FTE).

8.1.2 I2: Development of CMOM Program

The items listed in Alternative I1 above are elements of the CMOM program, but would not be considered a full program. Other key elements that will be required include writing and documenting the program and development of software to manage the program and the system. Typically, this includes a GIS-type asset management program. The estimated cost for the CMOM program (in addition to the annual cost of Alternative I1) is approximately $60,000.

8.2 Sub Basin Alternatives

8.2.1 I3: Eliminate 1st Street Overflows and Manhole Replacement/Protection

Eliminate Overflows on 1st Street, Replace Manhole at 1st Street and Stillwell Ave., and install a Sodium Hypochlorite Dosing System to the STEP System Forcemain (Sub Basin 1).

- To eliminate the overflows along 1st Street, connect businesses on the north side of the intersection at 1st Street at Stillwell Avenue to the sewer that runs along Front Street and eliminate the high-flow overflow. The sewer on Front Street is several feet lower than the sewer on 1st Street. Redirection of these lateral lines would alleviate sewage backup in these buildings and eliminate the need to provide a high-flow overflow for the sewer on 1st Street.
- Replace the manhole at 1st Street and Stillwell Avenue. This manhole has been structurally damaged over the years by hydrogen sulfide released as STEP system effluent is discharged. The new manhole at 1st and Stillwell will be epoxy coated to protect it from hydrogen sulfide damage. The connecting pipe is clay which is not susceptible to hydrogen sulfide damage.
- Install a sodium hypochlorite dosing station in conjunction with the WWTP Expansion Project to neutralize hydrogen sulfide in the STEP system force. The station would be installed at the Front Street Pump Station site and is intended to eliminate odor and corrosion damage to concrete manholes and pipes. A technical memorandum will be included in the Predesign Report for the WWTP.

With the exception of replacing the manhole at 1st Street and Stillwell Avenue, additional improvements to the Northside Interceptor are not recommended until the STEP systems in Sub Basin 5 (which discharge to Northside Interceptor) have been updated. The existing STEP systems may contribute significant I/I, which should be reduced when they are replaced. The Northside Interceptor may be sufficient to handle peak flows under surcharged conditions. Due to the substantial depth of this pipe, manholes do not overflow.

For the two lateral reconnections at Front Street, the total estimated capital cost for Alternative I3 is budgeted at $30,000. For replacement of the manhole at 1st Street and Stillwell, $45,000 has been budgeted. The hypochlorite dosing system is budgeted at $120,000.

8.2.2 I4: Grease Management (Sub Basins 1 and 5)

It is recommended that the City enforce the grease ordinance for restaurants, including a requirement for installation and maintenance of effective grease traps for all food service establishments. This item should be a no-cost capital item for the City, but will require staff time to check on the systems and enforce the ordinance. It is estimated that this will require four hours per week, or 0.1 FTE.

8.2.3 I5: Replace STEP Systems in Sub Basin 5

Sub basin 5 has about 60 STEP systems that are not standardized and, in general, are equipped with old pumps and controls resulting in significant maintenance expenditures. In addition, some of the STEP system pumps sit on the bottom of the tanks requiring them to pump the solid portion of the STEP tank influent. According to staff, some of the tanks may be structurally compromised as a result of pumping them under high ground water conditions. All but seven of the existing STEP tanks are of FRP construction. The FRP tanks are 1,000-gallon and 1,500-gallon capacity. Grease buildup in the STEP tanks from food preparation is another ongoing problem.

To reduce maintenance requirements and increase reliability, it is recommended that the City upgrade the existing STEP system as follows:

- Implement and enforce a grease reduction ordinance for the customers connected to STEP systems.
- Catalogue the existing system. This should include the type of tank installed, the volume of the tank, the age of the tank, and the material of construction. Describe how the tank is configured. The type of pump, its configuration in the tank, and control system as well as its condition and age should be recorded. Compile any other pertinent information on the existing system.
- Prioritize the replacement program replacing the equipment from poorest to best condition.
- Develop a standard for equipment.
- Replace the oldest and most problematic STEP systems first and the newest and most reliable systems last.

The CIP budget for upgrading the existing STEP systems in Sub Basin 5 is $390,000. Once the STEP systems are fully upgraded, four hours per week, or 0.1 FTE is estimated to be required for maintenance of the systems.

8.3 Interceptor/Forcemain Alternatives

Using land use data provided by Tillamook County for long-term growth within the UGB, calculations were made to estimate the required capacity of the existing interceptors. Detailed calculations are presented in Appendix G. The results are presented in Table 8.1. For interceptor location, see Figure 4.1.

Table 8.1: Required Interceptor Capacity

Interceptor	Existing Pipe Size (In)	Existing Capacity (MGD)	Estimated Existing Peak Flow (MGD)	Estimated Buildout Peak Flow (MGD)
Southside Interceptor	18 and 15	2.60[(a)]	3.20	13.10
12th Street Interceptor	18	2.80	2.30	10.10
Eastside Interceptor	10	0.90	2.20	9.80
Northside Interceptor	8	0.50	0.50	0.50[(b)]
4th Street Interceptor	16	3.10	1.90	7.00

Notes:

(a) Assumes the Influent Pump Station has been replaced, allowing un-surcharged gravity flow.
(b) Sub Basin 5 flows only.

It is clear that the Southside, 12th Street, and Eastside Interceptors are significantly undersized to meet the potential buildout flows. Evaluations and recommended improvements to these three interceptors are presented below.

The Northside Interceptor capacity appears to have adequate long-term capacity. The 4th Street Interceptor, which appears to be undersized for build-out flows, is sufficient for current flows and may also be sufficient for future flows.

The County data indicates almost 400 available residential lots (including apartments) in Sub Basin 1. While this may be true, it is unlikely that these lots will be developed to their maximum density. In addition, the 4th Street Interceptor shows no current signs of overflowing. Based on these factors, no improvements to the 4th Street Interceptor is recommended at this time.

8.3.1 I6: Construct a New Southside Interceptor

The Southside Interceptor has capacity problems which currently cause overflows. This is due to the permanently surcharged condition of this pipe at its downstream end as the normal influent pump station wetwell level is above the crown of the pipe (wetwell level is elevated further under high flow conditions). Because of this, and the limited capacity of the influent pump station, the Southside Interceptor has progressively less capacity as flows to the WWTP increase and levels in the influent pump station rise.

Planning documents indicate that significantly more flow may be conveyed through this interceptor due to the land available for development to the east. Flow calculations show that even if the influent pump station is lowered, this interceptor is not sufficiently sized to convey the current or peak buildout flows. Given that calculations for existing peak flows are only estimates – no actual flow data is available – it is possible that the interceptor can convey the existing peak flows. The recommended course of action for the Southside Interceptor is to wait until a new influent pump station is constructed to determine if the capacity is sufficient to eliminate overflows. If overflows continue to occur, replacing the interceptor should be considered.

When the interceptor requires additional capacity, two general options for conveying these peak flows are available: increase the size of the existing interceptor by constructing a larger pipe, or construct a dual forcemain to convey the flow from the 12th Street Pump Station directly to the treatment plant.

The primary benefit of increasing the size of the existing interceptor is the elimination of the power cost required to pump through an extended forcemain. Total estimated capital cost to reconstruct the interceptor with 30-inch concrete sewer pipe, allowing for some realignment around the seasonal pond (south end of Cedar Avenue), is approximately $1,580,000.

The primary benefit of constructing new forcemain directly to the WWTP is that the existing interceptor would be adequate to convey the Sub Basin 2 flows with little to no work. The pipes were slip-lined in 1975 and are likely to be in good to adequate condition. Some work might still be necessary on the manholes located in the seasonal pond. The drawback is that dual forcemains would be required due to the large variation between average and peak flows. Total estimated present worth cost, including capital cost to construct an 8-inch/18-inch parallel

forcemain in the same general location as the existing gravity pipe plus 20 years of additional power costs, plus pump station upgrades is approximately $2,000,000.

Based on this analysis, it is recommended that a new gravity line be constructed when the capacity of the existing interceptor is no longer sufficient. Detailed sizing of the gravity main will occur after the WWTP influent pump station is on-line and more information regarding required capacity is available. Total estimated capital cost for the interceptor is approximately $1,580,000.

8.3.2 I7: Replace 12th Street Interceptor

The 12th Street Interceptor is also not sufficiently sized for build-out flows. While it is adequately sized for current flows, it will likely reach capacity within 5 to 10 years. In addition to capacity problems, the integrity of the pipe may be questionable where it crosses under the saw mill sorting yard based on video reports done in the 1970's.

When the capacity of the existing 12th Street Interceptor is no longer sufficient, it is recommended that a new 30-inch concrete gravity sewer be constructed in the 12th Street right-of-way. Total estimated capital cost for the gravity line is approximately $740,000. It is estimated that the interceptor will need replacement in about 2015.

8.3.3 I8: Construct Interceptor to Fairview Area

Calculations indicate that the Eastside Interceptor is undersized for existing flows. The lower portions of the interceptor currently overflow during storm events, confirming the calculations. Future eastward expansion into the UGA and Fairview cannot be accommodated by this interceptor due to the shallow cover on the upstream end of this pipe and the existing topography. Adding the expansion flows to the Eastside Interceptor will also compound the existing capacity problems.

Two general options were evaluated to address these issues: 1) upsize the Eastside Interceptor along its existing alignment to provide enough capacity to serve the future expansion into the UGA, or 2) construct a new gravity interceptor to serve the expansion into the UGA and upgrade the existing Eastside Interceptor as necessary to accommodate existing flows.

While there are benefits to utilizing the existing interceptor alignment, it may not be the best option because a significant portion of future development to the east may not be able to gravity flow to this pipe. As the ability to serve new development using gravity is preferable to constructing and operating new pump stations, a new interceptor along the southeast edge of the UGB is considered a better alternative.

The benefits to constructing a new interceptor include:

- The new interceptor could be designed to convey the flow from the UGA and Fairview area by gravity.

- Three existing pump stations could potentially be eliminated by gravity connections to the new interceptor. Removing the pumped flow from the existing Eastside Interceptor will help relieve existing capacity issues.

- New connections along the east end of 12th Street could be served without pump stations.

A new interceptor extending to the Fairview area is recommended. The interceptor would extend from the corner of 12th Street and Evergreen Drive to the Brookfield subdivision to serve the customers currently served by the Brookfield Pump Station. New gravity connections from properties along 12th Street and along Marolf Loop would be possible. Flow from new developments would be conveyed by the new interceptor. The proposed interceptor is referred to as the Marolf Interceptor.

Two proposed alignments for the Marolf Interceptor are shown on Figure 8.1. The alignment shown along 12th Street appears viable, although it may require fairly deep excavation (greater than 10 feet). The alignment shown next to Holden Creek would require easement acquisition and construction in wetland areas but would allow for shallower excavation. The choice between the two alignments will require more accurate topography information, a cost comparison for construction, the practically of obtaining easements to construct the pipe on private land, and the effort required to secure permits to allow construction within jurisdictional wetlands.

Also shown on Figure 8.1 is the Brookfield Interceptor, which would extend from the Marolf Interceptor to the developments along Brookfield. Total estimated cost for construction of the Marolf and Brookfield interceptors from 12th Street and Evergreen Drive to Brookfield Avenue at the Fairgrounds is approximately $1,660,000.

When property owners east of the Fairgrounds are required to connect to the City's sewer system, it is anticipated that they will form a Local Improvement District (LID) to finance the extension of the interceptor. The extension, called the Trask Road Interceptor in this report, is expected to be 18-inch and 12-inch lines, starting at the Fairgrounds and ending on Trask Road at about Circle Drive. Total estimated cost to the homeowners for construction of the extension of the Trask Road Interceptor is approximately $650,000.

8.3.4 I9: Upsize South End of East Interceptor (Evergreen Drive) and Replace Manhole at 1st Street and Stillwell Avenue

The lower sections of the existing Eastside Interceptor along Evergreen Drive from 12th Street to Maple Lane will be replaced in the fall of 2005. Total estimated cost to upsize the interceptor along is approximately $195,000.

<u>Alternative I9:</u>

In an effort to eliminate the current overflows along Evergreen Drive, it is recommended that this improvement be constructed as soon as possible:

- Upsize the existing Eastside Interceptor along Evergreen Drive.

Total estimated capital cost for these items is approximately $195,000.

8.4 Pump Station Alternatives

8.4.1 I10: Upgrade 12th Street Pump Station

The 12th Street Pump Station is a critical element in the conveyance of flows from sub basins 3 and 4. Currently the station discharges to the Southside Interceptor, which is in a permanently surcharged condition.

Upgrade of the 12th Street Pump Station should be undertaken in two phases. The first phased improvements should ensure that the pup station has ample capacity and the required reliability to meet the needs over the next 5 to 10 years.

Phase One Improvements would include:

- Installation of a flow meter and run time meters.
- Perform a detailed engineering analysis to determine the station capacity and pumping requirements.
- Upgrade the station as recommended by the engineering analysis to include new or upgraded pumps, upgraded controls including VFDs to optimize pump flow rates, redundant level alarms, and an autodialer to notify personnel of alarm conditions.
- Install a larger emergency generator. There is a 150 kW generator at the WWTP installed as part of the 2003 CAP project. This generator should be relocated to the 12th Street Pump Station when it is replaced by a larger unit at the treatment plant. This is expected to occur in 2007.

Total estimated capital cost for improvements to the 12th Street Pump station is approximately $400,000. It is recommended that the installation of flow meter and run time meters should be done as early as possible. The timing of the remaining work should be based on the determination of the Engineering Analysis.

8.5 Prioritized Alternatives

The conveyance system alternatives were assigned a high, medium, or low priority status. The priority levels are summarized below:

- High Priority - Replace, repair, or improve those components of the collection system that directly contribute to manhole overflows during peak flow events.
- Medium Priority - Repair or replace those components that require substantial and ongoing maintenance expense with a more reliable system. Repair, replace, or construct new interceptor lines to eliminate any remaining surcharging.

- Low Priority - Repair, replace, or construct new interceptor lines to accommodate future flows. Develop programs and ordinances necessary to conform to new regulations and new systems.

The alternatives outlined above are listed in Table 8.2 below along with their prioritization and their estimated capital cost.

Table 8.2: Conveyance System Prioritized Alternatives

	Alternative	Priority	Estimated Capital Cost	Proposed Timing
	Construct New Influent Pump Station at WWTP	High	See Liquid Stream Treatment Alternatives	WWTP Phase 1
I3:Ph1	Eliminate 1st Street Overflows (Sub Basin 1)	High	$30,000	ASAP
I9	Upsize Eastside Interceptor at Evergreen Drive	High	$195,000	ASAP
I8:Ph1	Construct Marolf Interceptor	High	$825,000	1 to 3 yrs
I3:Ph2	New Manhole at 1st Street and Stillwell Ave.	High	$45,000	
I3:Ph3	Sodium Hypochlorite Dosing Station	Medium	$120,000	1 to 3 yrs
I5	Upgrade Step Systems in Sub Basin 5	Medium	$390,000	1 to 5 yrs
I1	Smoke Testing, TV, and Maintenance Program	Medium	$80,000 per year [a]	ASAP
I6:Ph1	Construct New Southside Interceptor	Medium	$1,580,000	5 years[b]
I10	Upgrade 12th Street Pump Station	High/Medium	$400,000	1 to 3 yrs[c]
I8:Ph2	Construct Brookfield Interceptor	Low	$825,000	5 years[d]
I7	Replace 12th Street Interceptor	Low	$740,000	10 years
I2	Development of CMOM Program	Low	$60,000	5 years
I4	Grease Management (Sub Basins 1 and 5)	Low	No Capital Cost	Ongoing, Start Now
	Total[a]		$5,210,000	

Notes:

(a) Yearly costs not included in Total Cost at end of table.
(b) After new influent pump station is constructed and flows are re-evaluated.
(c) Install flow meter and run time meters and undertake engineering analysis. Make improvements as appropriate.
(d) When Fairview septic tank connections are imminent.

8.6 Infiltration/Inflow Alternatives

Given the high peaking factor (ADWF/PIF = 15) in Tillamook, it is clear that I/I is significant. Two alternate approaches are possible for dealing with excessive I/I. The first is to determine the sources and implement rehabilitation of sewer pipes, manholes and laterals. This approach can be cost-effective if isolated sources of excessive infiltration can be identified and corrected while the remainder of the collection system is known to be of high integrity.

An alternate approach assumes that it would not be cost effective to remedy I/I sources but rather to build sufficient conveyance and treatment plant capacity to handle the additional hydraulic load imposed by these sources. In order to determine the most logical solution for the Tillamook system, both approaches were considered in greater detail.

8.6.1 Rehabilitation of the Collection System

Based on closed circuit television inspections conducted several years ago, leaky pipes, manholes and lateral connections exist to varying degrees over the entire extent of the collection system (see Figure 4.2). Experience with other communities has shown that rehabilitation of the collection system pipes without rehabilitation/replacement of lateral connections will not significantly reduce infiltration. Therefore, for purposes of this analysis, it will be assumed that all of the existing sewers in Sub Basins 1 through 4 would require rehabilitation and that every existing lateral would require replacement. In Sub Basin 5, all laterals and STEP system tanks would be replaced. These assumptions are based on the closed circuit TV inspections and the age of existing laterals.

Removing the aforementioned sources of infiltration would require a substantial expenditure of capital funds as summarized in Table 8.3. The rehabilitation costs are based on the following assumptions:

- The cost of pipe rehabilitation is $4 per inch diameter per lineal foot of pipe. It was assumed that rehabilitation could be accomplished using in-situ coating techniques which would likely be the least-cost alternative for the majority of the existing pipes in the system.

- The cost of rehabilitation of manholes is $2,000 per manhole. The cost is based on an 8 to 10-foot deep, 4-foot diameter manhole, which is considered an average depth manhole for the sewer system. It is assumed that the manholes could be coated from the inside using a polyethylene lining system.

- The cost of replacing lateral connections is estimated to be $ 4,000 per connection.

- The cost of replacing a STEP system tank is estimated at $5,000 per tank.

- An allowance was added to the estimate to account for sections of pipes requiring excavation that could not be rehabilitated using in-situ techniques and for additional expenses required for lateral replacement in dense downtown commercial areas.

Table 8.3: I/I Alternative Cost: System-Wide Rehabilitation

Sub Basin	Rehabilitation Cost
1	$7,400,000
2	$3,100,000
3	$2,000,000
4	$5,900,000
5	$850,000
Open Cut (Downtown) Allowance	$4,800,000
Total	**$24,000,000**

One important consideration when undertaking infiltration removal rehabilitation is that complete success is not assured or even anticipated. Experience has shown that at best only 40% to 60% of I/I can be removed following extensive rehabilitation of sewer lines and laterals. Table 8.4 shows the current and projected flow rates for the combined service area with approximately 40% of the estimated I/I removed. See Table 6.8 for projected flows without I/I removal.

Table 8.4: Estimated Flows with 40% of I/I Removed[a]

Year	Population	ADWF (MGD)	AWWF (MGD)	MMDWF (MGD)	MMWWF (MGD)	PDF_5 (MGD)	PIF_5 (MGD)
2003	4,625	0.55	1.12	1.34	2.12	3.80	5.30
2005	5,116	0.58	1.21	1.43	2.22	3.93	5.49
2010	6,450	0.74	1.47	1.70	2.50	4.22	6.09
2015	6,795	0.78	1.53	1.76	2.57	4.31	6.22
2020	7,165	0.82	1.59	1.82	2.65	4.41	6.36
2025	7,515	0.87	1.64	1.88	2.72	4.50	6.49

Note:

(a) It is assumed that only excessive I/I from the existing service area would be removed. I/I rates for expansion areas would remain unchanged as a "tight" sewer system (1,000 gpd) was assumed for this area.

8.6.1.1 Convey and Treat I/I at the Treatment Plant

The alternative to a comprehensive rehabilitation effort of the collection system to reduce I/I is to provide adequate conveyance and treatment plant capacity to hydraulically handle the non-attenuated flows. The primary components requiring increased hydraulic capacity to handle peak day and peak hourly flows are the interceptors, collection system pump stations, treatment plant influent pump station, headworks, clarifiers, and conveyance channels and pipes. With respect to the existing conveyance system, the following should be considered:

- Records indicate that the 12th Street Pump Station is currently under capacity. This pump station was constructed in 1957 and requires upgrading. Providing adequate

capacity to handle the non-attenuated flows would likely cost approximately $200,000 more than just the necessary improvements to the pump station.

- The proposed improvements to the conveyance piping outlined previously in this section assume that no I/I removal has taken place. The Southside and Eastside Interceptors would require replacement with or without I/I reductions, although they might be smaller without the I/I flows. Without I/I flows, the 9th Street forcemain could be 20" instead of 24", the proposed interceptor to Fairview could be 21" instead of 24", and the 12th Street Interceptor could be 24" instead of 30". Total additional conveyance system cost if I/I is not removed is approximately $280,000.

With respect to the treatment plant, the following components should be considered:

- The existing influent pump station is over 45-years old, is under capacity, and is too shallow. Replacement of this pump station is key to providing adequate collection system capacity with or without I/I reduction. Providing adequate additional capacity to handle the non-attenuated flows would likely cost approximately $200,000.

- The current headworks is designed to handle a peak flow of 6.5 MGD. Reducing I/I in the collection system would allow for continued use of the existing headworks. Providing for the additional capacity required to handle non-attenuated peak flow would likely cost approximately $300,000.

- Additional secondary clarification will be required to adequately treat non-attenuated flows. Based on the difference between the attenuated and non-attenuated projected PDF for 2025 at a cost of $150 per 1,000 gallons treated per day, the cost to provide the additional secondary clarification would be approximately $500,000.

Total estimated cost, including engineering and contingency, to construct facilities with enough capacity to convey and treat the I/I flows, is approximately $1,500,000.

8.6.2 Discussion

Based on the rough costs estimated above, it is clear that removing the I/I from the collection system is significantly more expensive (about 10 times) than the cost to convey and treat the higher flows. The best alternative is to provide adequate capacity to convey and treat the current I/I flow.

After the construction of a new influent pump station, it will be possible to better identify those areas in Sub Basins 2, 3, and 4 with the worst I/I problems. At that time, it may prove cost effective to specifically address those areas. I/I problems associated with Sub Basin 5 will be addressed with the replacement of the STEP pumps as recommended above.

Section 9: Wastewater Treatment Plant Alternatives

9.1 Introduction

This section of the Facility Plan describes the following alternatives for upgrading the Tillamook WWTP.

IPS1: Wet Well / Dry Well Influent Pump Station

IPS2: Submersible Influent Pump Station

HW1: Headworks Improvements

T1: Parallel Selector Activated Sludge/RBC (SAS/RBC)

T2: Series Biotower/RBCs (BT/RBC)

T3: Oxidation Ditch No Primary Clarifiers (OXDnoPC)

T4: Additional RBCs (RBC/RBC)

T5: Oxidation Ditch with Primary Clarification (OXDwPC)

T6: Sequencing Batch Reactors with Primary Clarification (SBRs)

T7: Nitrifying Selector Activated Sludge with Primary Clarification (NSAS)

F1: Effluent Filter with Pumping

F2: Effluent Filter (Gravity)

D1: Disinfection with Gas Chlorine and Dechlorination

D2: Disinfection with Liquid Hypochlorite and Dechlorination

D3: Disinfection with Ultraviolet Radiation

E1: New Outfall for Treated Effluent

S1: Aerobic Digestion of Sludge with Mechanical Dewatering

S2: Sludge Thickening plus Anaerobic Digestion and New Sludge Storage

S3: Sludge Thickening plus Anaerobic Digestion and Mechanical Dewatering

S4: Sludge Thickening plus Anaerobic Digestion with Converted Sludge Storage

The WWTP will require improvements to address existing deficiencies (see Section 5) as well as to meet the demands of expected growth over the next 20 years (see Section 6). Detailed design data for each alternative is presented in Appendix H.

Table 9.1 summarizes the WWTP design criteria and effluent requirements presented in previous chapters. These criteria will be used in sizing the alternatives presented in this section.

Table 9.1: WWTP Design Criteria

Criteria	Value	
Influent Flows and Loads		
Average Dry Weather Flow (ADWF)	0.90	MGD
Max Month Dry Weather Flow (MMDWF)	2.08	MGD
Average Wet Weather Flow (AWWF)	2.53	MGD
Max Month Wet Weather Flow (MMWWF)	4.01	MGD
Peak Day Flow (PDF)	7.16	MGD
Peak Instantaneous Flow (PIF)	10.01	MGD
Total Suspended Solids (TSS)	1,503	ppd
5-Day Biochemical Oxygen Demand (BOD)	1,503	ppd
Effluent Limits [(a)]		
Winter		
Monthly Average TSS	15[(b)] to 30	mg/l
Monthly Average BOD	15[(b)] to 30	mg/l
Monthly Average Ammonia	None	
Monthly Geometric Mean Fecal Coliform	46	MPN/100 ml
Summer		
Monthly Average TSS	10[(b)] to 20	mg/l
Monthly Average BOD	10[(b)] to 20	mg/l
Monthly Average Ammonia	Monitoring Required	or 10 mg/l[(c)]
Monthly Geometric Mean Fecal Coliform	200	MPN/100 ml
All Year		
Monthly Average Chlorine Residual	0.01	mg/l
pH	6.0 to 9.0	
BOD and TSS Removal Efficiency	85%	
Monthly Average Excess Thermal Load	62	million BTU/day

Note:

(a) Not a comprehensive summary. See Chapter 6 for complete limits.
(b) Probable concentration requirement to meet mass load limits during maximum month flow conditions.
(c) Limit or monitoring requirements not established at this time.

The description of alternatives includes structures and equipment that are sized for the 20-year flow and loading projections. There were seven liquid treatment alternatives developed for upgrading the WWTP. These are:

1. Parallel Selector Activated Sludge/RBC (SAS/RBC)
2. Series Biotower/RBCs (BT/RBC)
3. Oxidation Ditch No Primary Clarifiers (OXDnoPC)
4. Additional RBCs (RBC/RBC)

5. Oxidation Ditch with Primary Clarification (OXDwPC)
6. Sequencing Batch Reactors (SBRs)
7. Nitrifying Selector Activated Sludge (NSAS)

Table 9.2 presents a present worth analysis of the 7 WWTP alternatives. The estimated capital cost plus the annual power cost for each alternative is used to make the 20-year present worth calculation. A description of the alternatives, their capital cost, and a discussion on operational complexity is presented in this section. A detailed cost estimate for each alternative is included as Appendix I.

Table 9.2: Comparative Cost for WWTP Upgrade ($1000)

Item	Alternative 1	2	3	4	5	6	7
	SAS/RBC	BT/RBC	OXDnoPC	RBC/RBC	OXDwPC	SBR	NSAS
Influent Pump Station	$ 1,340	$ 1,340	$ 1,340	$ 1,340	$ 1,340	$ 1,340	$ 1,340
Headworks	$ 230	$ 230	$ 230	$ 230	$ 230	$ 230	$ 230
Secondary Treatment	$ 3,172	$ 3,442	$ 3,844	$ 3,435	$ 2,784	$ 3,469	$ 3,045
Effluent Filtration	$ 800	$ 800		$ 800			
Disinfection	$ 396	$ 396	$ 396	$ 396	$ 396	$ 396	$ 396
Outfall	$ 35	$ 35	$ 35	$ 35	$ 35	$ 35	$ 35
Solids Processing	$ 2,238	$ 2,238	$ 2,161	$ 2,238	$ 2,583	$ 2,583	$ 2,583
Site Work	$ 469	$ 469	$ 469	$ 469	$ 469	$ 469	$ 469
Standby Power	$ 314	$ 314	$ 314	$ 314	$ 314	$ 314	$ 314
Wastewater Treatment (WWTP) Plant Cost	$ 9,000	$ 9,270	$ 8,590	$ 9,260	$ 8,160	$ 8,860	$ 8,420
Annual Comprative Operating Cost (COC)	111	102	121	96	92	89	90
Present Worth of Annual COC	$1,360	$1,250	$1,500	$1,180	$1,130	$1,100	$1,110
Total Comparative PW Cost of WWTP Upgrade	$ 10,360	$ 10,520	$ 10,090	$ 10,440	$ 9,290	$ 9,960	$ 9,530

Notes:

(a) Comparative operating costs Include electrical power usage. Labor and costs that do not differ between alternatives are not included.

(b) Present worth is based on 20-years of operation and an interest rate of 5.125%

9.2 Influent Pumping and Headworks Improvements

Given the condition of the existing influent pump station and its hydraulic limitations, it is recommended that the station be replaced as soon as possible. Two configurations are considered as the best alternatives: a dry well/wet well configuration, or a submersible configuration. The new pump station will incorporate a deeper wetwell that would allow the Southside Interceptor to flow un-surcharged. This section will also discuss improvements for the existing headworks.

9.2.1 IPS1: Wet Well / Dry Well Influent Pump Station

A new wet well/dry well pump station would be a cast-in-place design, approximately 16-foot in diameter with room to install five (four duty, one standby) influent pumps. Firm capacity of the pump station would be 10 MGD. The wet well portion of the station, with approximately 1,350

gallons per foot of depth, would have a 3-foot operating depth. The bottom of the station would be approximately 25 feet below existing grade.

The estimated comparison cost for construction of the wet well/dry well pump station, including pumps, controls, and appurtenances is approximately $1,701,000.

9.2.2 IPS2: Submersible Influent Pump Station

This alternative includes five pumps (4 duty and 1 standby). The submersible station would be a cast-in-place rectangular station. The operating depth and overall depth of the station are the same as the wet well / dry well station.

The estimated comparison cost for construction of the submersible pump station, including pumps, controls, and appurtenances is approximately $1,340,000.

Since the reliability, performance, and operations and maintenance cost for the two types of pump stations is similar, it is recommended that a submersible station be selected due to the significant advantage in capital cost.

9.2.3 HW1: Headworks Improvements

The plant has historically received a substantial amount of plastic products and other debris in the raw sewage that eventually get consolidated in the digested sludge making land application of biosolids unattractive to local farmers. Treatment plant staff spends a significant amount of effort removing this material using a manual screening process. The installation of a fine screen at the headworks would solve this problem.

The proposed headworks screen would be installed in the headworks channel in the location of the existing Parshall flume. A bypass channel with a manual bar screen will be constructed in case the fine screen blinds. The estimated comparison cost for installation of a headworks screen, bypass channel, and controls, is estimated at $230,000.

9.3 Secondary Treatment Alternatives

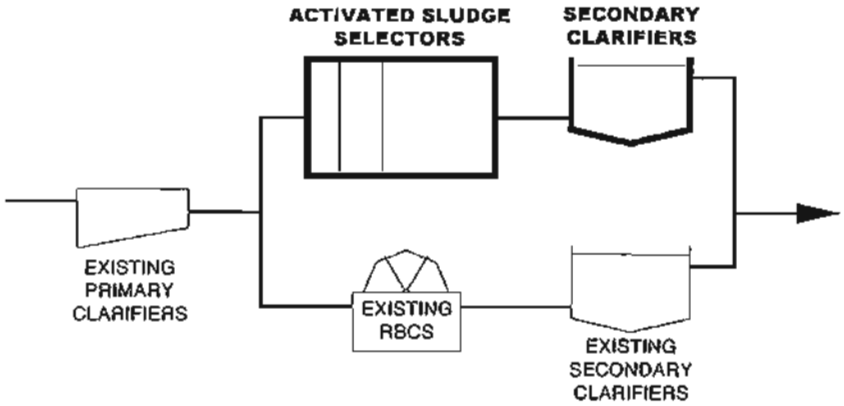
This section will describe and evaluate seven alternatives for secondary treatment. Effluent filtration will be required for all systems using fixed film secondary treatment.

9.3.1 T1: Parallel Selector Activated Sludge and RBC (SAS/RBC)

In an effort to make use of existing infrastructure, this alternative includes continued use of the existing RBC units for treatment of a portion of the flow and Selector Activated Sludge (SAS) for treatment for the remainder of the flow. Based on a 1st-stage loading rate of 2.5 lb SBOD per 1,000 sf of media, and operating the existing RBCs as a 4-stage unit, the RBC system could treat approximately 1/3 of the design loading or about 0.26 MGD. With the SAS/RBC alternative, average SBOD loading to the RBCs would be approximately 0.6 lb per 1,000 sf. Effluent from the RBCs would be settled in the existing secondary clarifiers.

The remaining 0.64 MGD average dry weather flow would be treated in SAS basins. Each selector would be divided into multiple zones with fine bubble aeration available to provide the necessary oxygen. The different zones would provide both anoxic and oxic (aerated) treatment. At a 6-day MCRT and 3,000 mg/l mixed liquor, two basins would be necessary, each being 70,000 gallons. Selector activated sludge basins are a very common treatment process and provide more flexibility than most other activated sludge process modes (including oxidation ditches).

Mixed liquor from the SAS basins would be settled in new clarifiers. Two 58-foot diameter clarifiers would eventually be constructed. The new clarifiers would be 14 feet deep to better settle the suspended growth sludge and would have peak-day overflow rates of 1,200 gpd/sq ft. Waste activated sludge (WAS) and Return Activated Sludge (RAS) pumping would be required. A new effluent filtration system would be constructed with the SAS/RBC alternative to assure that effluent can meet stringent BOD/TSS concentration limits that may be required in the future. The filter is described in Section 9.3.



Schematic of Parallel SAS/RBCs

With this alternative, the existing primary clarifiers would continue to be used. Operationally, this system is more complex than the other options due primarily to the requirement to operate two entirely independent systems.

The estimated comparison cost for this option is $3,172,000. Assuming that only seasonal partial nitrification is required; The estimated annual comparative operation cost is $37,000 per year.

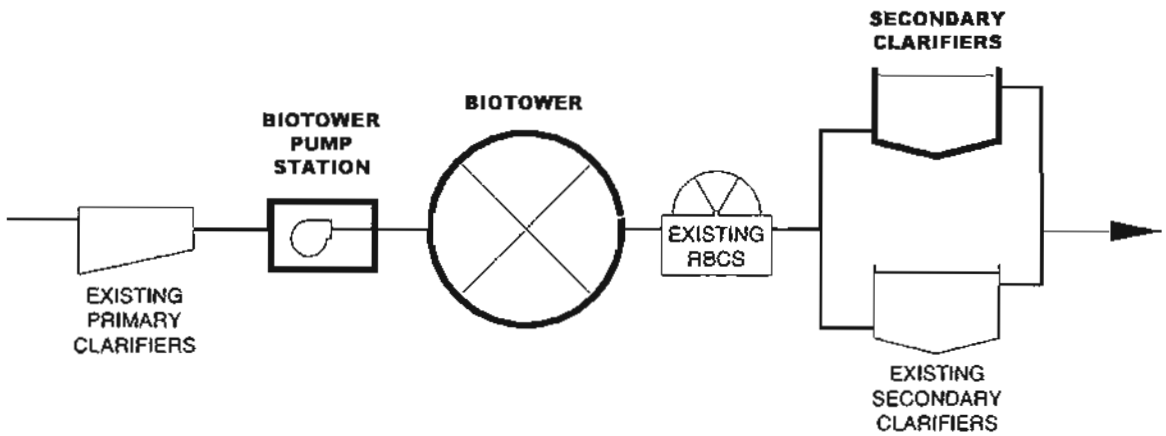
9.3.2 T2: Series Biotower/RBC (BT/RBC)

This alternative makes use of the existing RBCs by using them to polish effluent from a high-rate biotower. Use of combined or dual secondary processes can produce a high quality effluent. The biotower, filled with plastic media would provide the majority of treatment. The RBCs, located downstream of the biotower would provide additional treatment, or polishing.

The biotower would be a "tilt-up" concrete structure filled with stacked plastic media. Assuming a loading rate of 40-lb BOD per day per 1,000 cf, the required volume of the media would be approximately 25,500 cf. With a media depth of 16 feet, the diameter would be 45 ft.

The biotower pump station would be capable of pumping the maximum month flow and would utilize vertical turbine pumps. The same pump station would be used to recycle flow over the biotower, keeping the media wet at all times and keeping the biomass alive even during very low flows. Flows in excess of the maximum month flow would go directly to the RBCs. Soluble BOD in the biotower effluent would be expected to average less then 15 mg/l and will be further treated in the second stage RBC system.

Two 58-foot diameter clarifiers with peak-day overflow rates of approximately 1,200 gpd/sq ft would be required for this alternative. The existing primary clarifiers would continue to be used for primary clarification and co-thickening. A new effluent filtration system would be required with this alternative to ensure that effluent can meet stringent BOD/TSS concentration limits that may be required in the future. The filter is described in Section 9.3.



Schematic of Series Biotower/RBC

This system would be comparable in complexity to the parallel RBC/SAS system. However, with the series biotower/RBC arrangement the processes are both fixed film, which provides a somewhat simpler system than the parallel RBC/SAS.

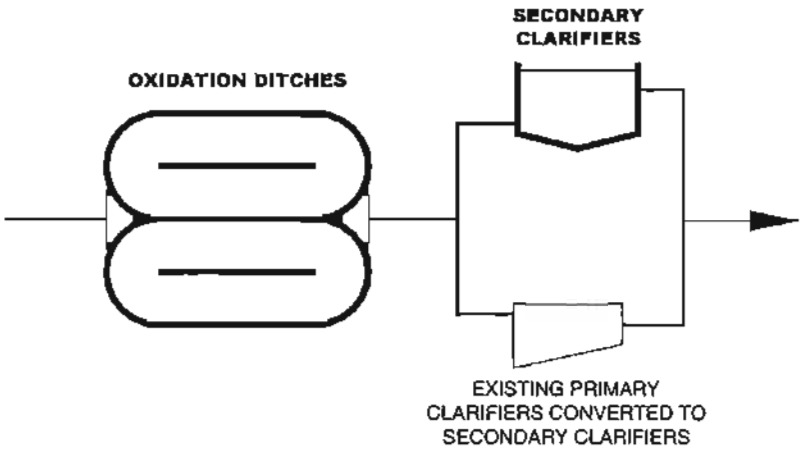
The estimated comparison cost for this alternative is $3,442,000. Assuming that seasonal partial nitrification is required, the estimated annual comparative operation cost is $28,000 per year.

9.3.3 T3: Oxidation Ditch No Primary Clarifiers (OXDnoPC)

For this alternative, the existing north primary clarifiers are converted to secondary clarifiers which is consistent with the operation of the great majority of oxidation ditches where no primary clarification is used. The elimination of primary solids would allow the use of aerobic digestion which would have lower capital costs than an anaerobic system.

In general, an oxidation ditch is a round or elongated structure forming a continuous channel or series of concentric channels. The wastewater is continuously circulated around the basin(s) using surface aerators and submerged mixers. This configuration can provide alternating aerobic and anoxic zones facilitating nutrient removal if desired.

The oxidation ditch would be designed for a 10 day MCRT. The total volume of two basins would be about 600,000 gallons. The system would be designed to provide operational flexibility, including the ability to remove one ditch from service, and the ability to operate the basins in a contact stabilization mode during high flow events.



Schematic of Oxidation Ditch with no Primary Clarifiers

In the OXDnoPC alternative,the existing north primary clarifier would be converted to a rectangular secondary clarifier. While uncommon in the northwest, rectangular secondary clarifiers can be more efficient, and are very common in California and other parts of the country. When equipped and operated properly, they can be operated at a loading rate of 1,000 to 1,200 gpd/sq ft of surface area. Two additional 65-foot diameter clarifiers would need to be constructed to meet the 20-year flow requirements.

It is likely that the existing headworks/influent channel could continue to be used with minor modification. The oxidation ditch could be built at grade and the mixed liquor could gravity flow to the secondary clarifiers.

This alternative would require a large change in day-to-day operations of the plant, primarily due to the elimination of the primary clarifiers (and, therefore, the co-thickening element of sludge handling), and the elimination of the fixed film process. Operationally, suspended growth treatment processes can require more testing and operator attention then fixed film systems.

The estimated comparison cost for construction of 20-year secondary treatment facilities, including construction of the new oxidation ditch, conversion of the existing primary clarifiers to secondary clarifiers, and construction of new secondary clarifiers is approximately $3,644,000. Assuming seasonal partial nitrification, the estimated annual comparative operation cost is $30,000 per year.

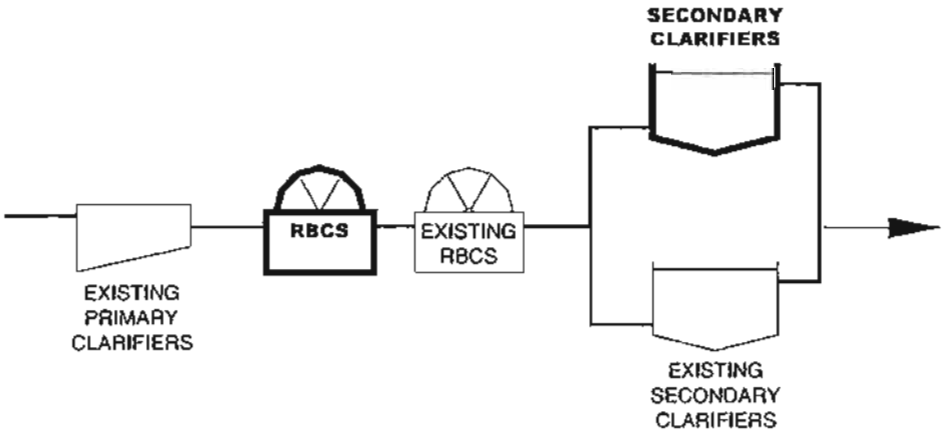
9.3.4 T4: Additional RBCs (RBC/RBC)

This alternative was developed as the "least change" alternative in terms of O&M activities. However, it is also the option that offers the least opportunity to convert to suspended growth (activated sludge) in the future. Although the series Biotower/RBCs is also a completely fixed-film process, biotowers have been used in conjunction with suspended growth processes. Additional RBCs would be constructed to provide 1st-stage treatment. Two design criteria are used in sizing the RBCs; 1st-stage loading and average system loading. Using a 1st-stage loading criteria of 2.5 lb SBOD per day per 1,000 sq ft of surface area, the total 1st-stage surface area required is approximately 260,000 sq ft. For purposes of estimating cost, this alternative assumes the construction of four additional RBCs in a configuration identical to the existing four units. The new units would be configured to allow parallel flow through all units. It is assumed that the new units would be motor-driven.

At an average system loading of 1.0 lb SBOD per day per 1,000 sq ft of surface area, the total surface area required to meet the design criteria is 654,000 sq ft of RBC media. The four new 1st-stage RBCs would be used in series with the existing four RBCs to provide a total media surface area of 800,000 sq ft.

Secondary solids would continue to be settled in the existing clarifiers. Like the series biotower/RBCs, this alternative will require two new 45-foot diameter clarifiers. The clarifiers would be 14 feet deep. The primary clarifier would remain in service and would continue to be used to co-thicken primary and secondary solids. The filter is described in Section 9.3.

From an operational perspective, this may be the simplest of all the alternatives as there is only one treatment process to monitor.



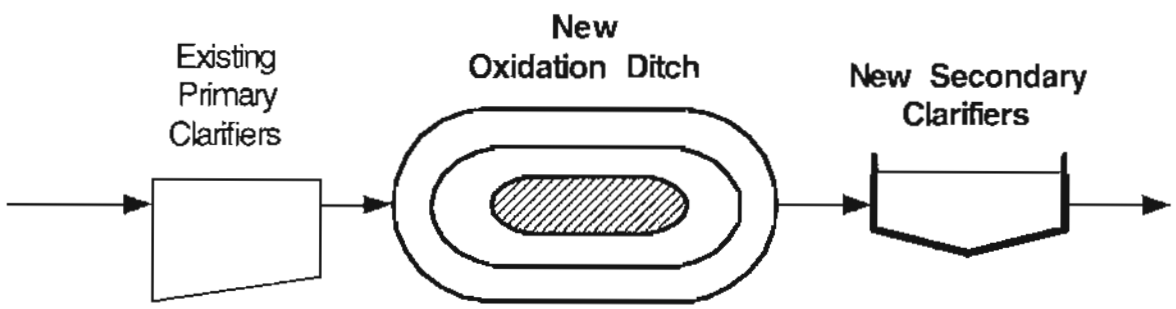
Schematic of Additional RBCs

The estimated comparison cost for construction of 20-year secondary treatment facilities, including construction of the new RBCs, and construction of new secondary clarifiers is approximately $3,435,000. Assuming seasonal partial nitrification, the estimated annual comparative operation cost is $22,000 per year.

9.3.5 T5: Oxidation Ditch with Primary Clarification (OXDwPC)

In this alternative, the existing primary clarifier is retained and an oxidation ditch is added. This alternative is referred to as the Oxidation Ditch with Primary Clarification (OXDwPC) alternative.

The oxidation ditch would be designed to provide a 10-day MCRT. Total volume of the ditch would be about 400,000 gallons. The oxidation ditch channels will use common wall construction; however, each channel will be hydraulically independent so that either channel may be removed from service. The channels would be configured to operate in a contact stabilization mode for high flow events.



Schematic of Oxidation Ditch with Primary Clarification

Two new 65-foot diameter circular secondary clarifiers would be constructed with a peak loading rate of 1,200 gpd/sq ft.

The existing headworks/influent channel and existing primary clarifiers would continue to be used. The existing round 45 foot diameter secondary clarifiers would be converted to sludge storage.

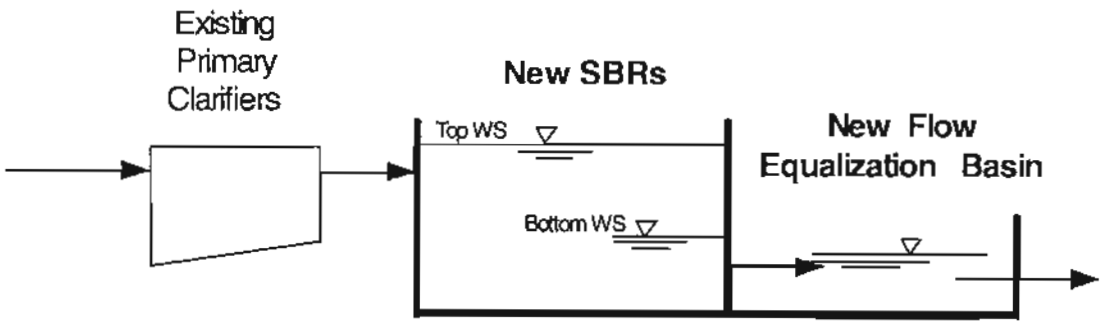
This alternative would require a significant change in the operations of the plant. The existing primary clarifiers would remain in service. Co-thickening of secondary sludge would be discontinued. Operationally, suspended growth treatment processes will require more testing and operator attention than the existing fixed film system.

The estimated comparison cost for construction of 20-year secondary treatment facilities, including construction of the new oxidation ditch, secondary clarifiers and sludge pumping is approximately $2,784,000. Assuming seasonal partial nitrification, the estimated annual comparative operation cost is $21,000 per year.

9.3.6 T6: Sequencing Batch Reactors (SBRs)

This alternative involves the construction of sequencing batch reactors (SBRs) for secondary treatment. The existing primary clarifier would continue to be used. Stand alone secondary clarifiers are not required with this process as this function is incorporated into the SBR basin. The existing secondary clarifiers would be converted to sludge storage. The existing RBCs would be demolished.

The SBRs used in this evaluation are sized to handle the peak day flow of 7.16 MGD in 10 cycles or a total decant volume requirement of 0.76 MG. In addition to the decant volume is the reactor volume required to maintain an oxic MCRT of 10 days. Since the SBR is aerated only 50% of the time, the equivalent total MCRT 20-days (design oxic MCRT x total time/oxic time = 10-days x 72 min/36 min). Therefore, a total basin volume of 1.6 MG would be required. This equates to two basins, each 68 ft x 89 ft x 18 ft deep. In addition, a flow equalization basin would follow the SBRs to even out surging during the decanting of the reactors.



Schematic of Sequencing Batch Reactors

The SBR alternative would require a significant change in the operations of the plant. The existing primary clarifiers would remain in service. Operationally, suspended growth treatment processes can require more testing and operator attention than the existing fixed film system.

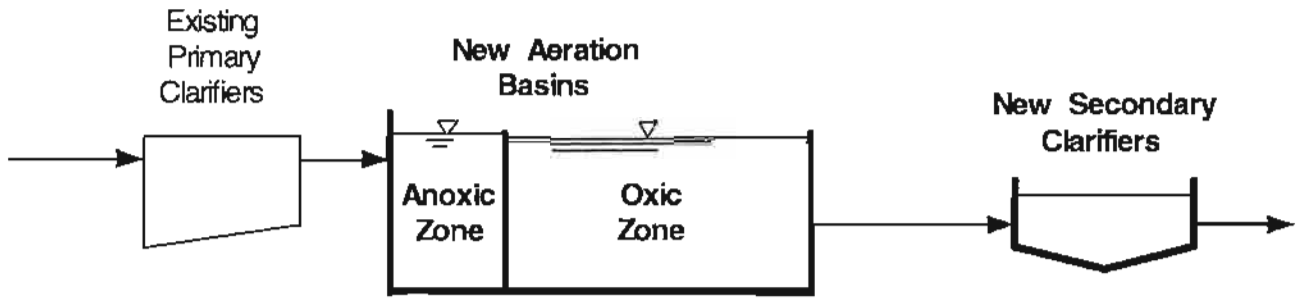
The estimated comparison cost for construction including construction of the two SBR reactors, an equalization basin, and waste sludge pumping (return sludge pumping is not required) is approximately $3,489,000. Assuming seasonal partial nitrification, the estimated annual comparative operation cost is $18,000 per year.

9.3.7 T7: Nitrifying Selector Activated Sludge (NSAS)

This option is referred to as the Nitrifying Selector Activated Sludge (NSAS) alternative. The NSAS system uses rectangular basins, a diffused aeration system, and anoxic selector zones. However, the existing RBCs would be decommissioned. For this option, the aeration basin will be sized to provide an aerobic MCRT of 12 days to ensure sufficient nitrification. The overall organic loading will be low which will make the NSAS alternative forgiving from an operations standpoint.

With the NSAS alternative the existing primary clarifiers would continue to be used. Two new 65-foot diameter circular secondary clarifiers would be constructed in the initial phase.

The selector aeration basin will consist of two parallel rectangular basins sized for 12 days mean cell residence time (MCRT) at a mixed liquor concentration of 2,500 milligrams per liter (mg/l). Total volume of the two basins would be about 444,000 gallons, which will result in a hydraulic residence time (HRT) of 12 hours at Average Dry Weather Flow (ADWF). The aeration basins will use common wall construction. Each basin will be hydraulically independent so that either basin may be removed from service.



Schematic of Nitrifying Selector Activated Sludge

Two 65-foot diameter circular secondary clarifiers would be constructed with a peak day loading of 1,060 gpd/sq ft.

The existing headworks and existing primary clarifier would continue to be used. The existing secondary clarifiers would be converted to sludge storage. The existing RBCs would be demolished.

The NSAS alternative would require a significant change in the operations of the plant. The existing primary clarifiers would remain in service. Operationally, suspended growth treatment processes can require more testing and operator attention than the existing fixed film system.

The estimated comparison cost for construction of this alternative, including construction of the new selector basin, new secondary clarifiers, and a sludge pumping facility is approximately $3,044,000. Assuming seasonal partial nitrification, the estimated annual comparative operation cost is $19,000 per year.

9.3.8 Summary of Liquid Stream Alternatives

Table 9.3 compares the costs for the seven treatment alternatives. It may advantageous to phase the construction of some components of total treatment system based on the liquid stream alternatives. Possible phasing is discussed later in this chapter along with presentation of the total system developed around each of the treatment alternatives.

Table 9.3: Summary of Estimated Liquid Stream Treatment Alternative Costs

Alternative	Capital Cost (2004 dollars)	Comparative Annual Operation Cost
Alternative T1: Parallel SAS/RBCs	$3,172,000	$37,000
Alternative T2: Series Biotower/RBCs	$3,442,000	$28,000
Alternative T3: Oxidation Ditch no PC	$3,644,000	$30,000
Alternative T4: Additional RBCs	$3,435,000	$22,000
Alternative T5: Oxidation Ditch with PC	$2,784,000	$21,000

Alternative	Capital Cost (2004 dollars)	Comparative Annual Operation Cost
Alternative T6: SBRs	$3,489,000	$18,000
Alternative T7: NSAS	$3,044,000	$19,000

9.4 Effluent Filtration

To achieve the anticipated dry weather design criteria of 10 mg/l for effluent BOD and TSS, effluent filtration and pumping is assumed to be required for Alternatives T1, T2, and T4. Alternatives which include conservatively designed suspended growth systems such as T3, T5, T6, and T7, do not include provisions for effluent filtration within the facility planning period.

While several types of effluent filters are available, the most cost effective type currently available for this size and type of treatment plant are the cloth media disk filters. This type of filter is the basis for the following discussion.

9.4.1 F1: Effluent Filter with Pumping

Cloth media disk filters typically require about two feet of head loss across the filter. Since all the alternatives requiring effluent filtration utilize the existing secondary clarifiers, a pump station will be required to provide sufficient head. The filter would be sized for the maximum month dry weather flow of 2.08 MGD. The filter and associate pump station would be cast-in-place concrete. The cloth media filters are sold as package units by several manufacturers.

The estimated cost for the effluent filter and pump station is $800,000. The estimated comparative operation cost for the 20-year planning period is $2,800 per year.

9.4.2 F2: Effluent Filter (Gravity)

The gravity effluent filter system includes the concrete basin and filter equipment, sized for 2.08 MGD. Provision could be added to allow some of the winter flow to pass through the filter.

The estimated comparison cost for the gravity effluent filter is $663,000. The estimated comparative operation cost for the 20-year planning period is $1,000 per year.

9.4.3 Summary of Effluent Filtration Alternatives

Table 9.4 below shows the costs for filtration with and without pumping.

Table 9.4: Summary of Effluent Filtration Costs

Alternative	Estimated Capital Cost (2004 dollars)	Estimated Average O&M Cost
Alternative F1: Effluent Filter with Pumping	$800,000	$2,800
Alternative F2: Effluent Filter (Gravity)	$663,000	$1000

9.5 Disinfection Alternatives

To protect the public health, effluent must be disinfected before it can be discharged from the treatment plant. In general, there are three main options for effluent disinfection: gaseous chlorine (currently used at the WWTP), sodium hypochlorite, and ultraviolet light. To protect aquatic species, it is necessary to dechlorinate disinfected effluent that is discharged to surface waters. All alternatives using chlorine disinfection assume that the existing chlorine contact tank is reused. The inability of the existing chlorine contact tank to satisfy Class 1 reliability criteria and to sufficiently meet peak flow demands may require a new chlorine contact tank.

9.5.1 Alternative D1: Disinfection with Gas Chlorine and Dechlorination

Gaseous chlorine disinfection is currently in use at the Tillamook WWTP. It is a common process for disinfecting wastewater effluent and uses a vacuum based chlorinator to control the flow rate of gas. Chlorine gas and water are combined in the gas eductor to form a solution that is diffused into the effluent. The chlorinated effluent flows into a chlorine contact tank (CCT) where disinfection occurs. Generally, recommended contact times are 60 minutes at average flow and 15 minutes at peak flow.

At the Tillamook WWTP, the existing chlorine contact tank is a retrofitted (round) secondary clarifier. Contact times for the design flows are 140 minutes at ADWF and 16 minutes at PDF. There are problems associated with the existing CCT as follows:

- The CCT has no dual basin component as required to meet Class I reliability standards.
- Short circuiting of flows is probable in this type of tank resulting in low efficiency. Short circuiting results in insufficient retention for some portion of the flow resulting in incomplete disinfection.
- Very little mixing is provided in a converted secondary clarifier. By its design, this type of tank is intended to provide quiescent conditions. Good mixing is required for proper disinfection.
- The water surface in the existing CCT is at a low elevation with respect to the Trask River water surface during flood stage. As the river rises, the flow capacity through the

CCT is proportionately reduced. The new influent pump station with a peak capacity of 10 MGD will overwhelm the existing CCT under high river conditions.

- The piping connections to and from the existing CCT are too small to accommodate the design peak hour flow.
- The current tank does not have an adequate location for adding the dechlorination agent.

For the reasons stated, it is recommended that the existing CCT be replaced with a new CCT that meets the current requirements for reliability, efficiency, retention time, and hydraulic capacity. The proposed CCT would be a dual-channel design so that either side could be isolated for maintenance. The channels would have a high length to width ratio to prevent short-circuiting and to facilitate mixing along the entire length of the channel. The water surface elevation of the tank would be such that the anticipated peak hour flow could be routed through the tank when the Trask River is elevated to the 100-year flood stage.

A rectangular, cast-in-place CCT with appurtenances meeting these requirements would have equal size channels, each 100 feet long, 5 feet wide, and 10 feet deep. The tank would be configured so that the basins fold back on themselves, reducing the overall length of the tank. The total volume would be 75,000 gallons or 37,500 gallons each basin. An influent and effluent mixing zone would be incorporated to facilitate efficient mixing of disinfection chemicals.

The estimated comparison cost for the CCT and appurtenances is $525,000.

Assuming a dosage of 10 mg/l of chlorine, the MMWWF demand for chlorine is approximately 380 pounds per day (ppd). This exceeds the 300 ppd capacity of the existing automatic chlorinator, but does not exceed the plant's total chlorine feed capacity (automatic and manual) of 550 ppd.

Chlorine gas is classified as a hazardous material. Changes in safety regulations and insurance costs have significantly increased the cost of gaseous systems due to their hazardous nature. The current Uniform Fire Code (UFC) requires the addition of facilities to contain and neutralize gas leaks and prevent their release to the atmosphere. This safety equipment, called a chlorine scrubber, is expensive and uses another hazardous material (liquid caustic) which also must be contained if spilled.

The requirement for a gas scrubber is ultimately the decision of the local Fire Marshall. Occasionally, the requirement for scrubbing is not enforced for systems with 150-lb cylinders. The requirement for scrubbing is always required for 1-ton cylinder systems.

Based on the consumption rate of chlorine outlined above, it appears that by the end of the planning period, the plant could be using more than two 150-lb cylinders per day. This would be an excessive requirement of the operators. If gas chlorine use is continued, it will be necessary to go back to ton cylinder usage (the system was originally designed for ton cylinders) and provide chlorine scrubbing.

Chlorine and chlorine residuals are toxic to aquatic life and the concentration of chlorine in the effluent are regulated in the NPDES permit. Dechlorination is required before discharging to the

surface waters to protect aquatic life and to meet the permit limits. Dechlorination is a process to neutralize residual chlorine in the effluent. Dechlorination can be accomplished using sulfur dioxide gas or sodium bisulfite liquid.

The Tillamook WWTP currently uses liquid sodium bisulfite for dechlorination. Assuming a 25 percent trade solution of sodium bisulfite, MMWWF demand at the design flow will be approximately 130 gal/day, or almost 4,000 gallons per month. The existing storage container volume is 1,100 gallons. If this system is to be used throughout the planning period, additional dosage and storage capacity will be required.

The estimated comparison cost for a new CCT, a new automatic chlorinator, conversion to one-ton cylinders, installation of gas chlorine safety equipment (chlorine scrubbing), and additional dechlorination chemical storage and dosage is $1,396,000. The Phase One cost for the new CCT and miscellaneous upgrades to the chlorine feed system is estimated at $1,000,000. The estimated comparative operation cost for the 20-year planning period is $48,000 per year.

9.5.2 D2: Disinfection with Liquid Hypochlorite and Dechlorination

Sodium hypochlorite (liquid beach) can be introduced to wastewater using gravity or by metering it through a pump. As with gas chlorine disinfection, the chlorinated effluent must flow through a chlorine contact tank. A new CCT would be required for this option. To remove any residual chlorine, chemical dechlorination is necessary before discharge to the Trask River.

For safety considerations, disinfection with liquid sodium hypochlorite is often being adopted by utilities that previously used gaseous chlorine. Although the topic of which type of disinfection is safer is debated, it is generally accepted that gas chlorine is potentially fatal if accidentally released whereas sodium hypochlorite, while very corrosive, is less likely to cause death. One exception to this is the formation of chlorine gas by accidentally mixing sodium hypochlorite with sodium bisulfate (disinfection chemical). Human exposure to sodium hypochlorite can cause skin burns and severe eye damage or blindness.

Delivery, storage, and feeding of liquid sodium hypochlorite is maintenance intensive as the chemical tends to leak at joints, cause gas binding problems in feed systems, precipitates solids in tanks and pipes, and is severely corrosive. A hypochlorite storage and feeding system lifespan is estimated at 10-years. Delivery and storage of the chemical requires special design features to meet safety requirements. Typically, at least two storage tanks are required for reliability. This alternative, as with the gaseous alternative, would require a new CCT.

A liquid hypochlorite system for Tillamook would require about 7,000 gallons of storage, three peristaltic pumps (two duty pumps), and associated piping, appurtenances, and electrical improvements. The storage tanks require secondary containment and protection from the sun.

Dechlorination improvements would be the same as for Alternative D1 above.

The estimated comparison cost for a new chlorine contact tank, hypochlorite storage, dosing pumps, and associated piping, and for the recommended dechlorination improvements is $1,400,000.

The estimated comparative operation cost for the 20-year planning period is $55,000 per year.

9.5.3 D3: Disinfection with Ultraviolet Radiation

Ultraviolet (UV) disinfection uses ultraviolet light at a wavelength of 254 µm to sterilize microbial pathogens resulting in microbial inactivation. The light is produced by passing electric current through gas filled lamps. The lamps are typically protected from direct contact with the effluent by quartz sleeves which allow for maximum transmission of the light while being cleanable and durable.

Three general types of UV bulbs are used for wastewater disinfection systems: low pressure, low intensity bulbs (low/low); low pressure, high intensity bulbs (low/high); and medium pressure bulbs (medium). The low/low systems and the medium systems have been around the longest; the low/high systems are relatively new. The total UV dose per bulb increases in this order: low/low, low/high and medium. As UV dose increases (and the number of required bulbs decreases), so does the potential for fouling of the quartz sleeves. In response, the UV system manufacturers now provide automatic cleaning systems for the low/high and the medium systems.

The low/high UV systems are typically the most cost effective and operator-friendly type of systems for systems the size of Tillamook. UV system sizing is dependent on the PIF, the effluent solids concentration, and on the transmittance of the water. Some manufacturers are hesitant to propose UV systems for fixed-film systems because the fixed-film effluent can have unusually low transmittance and often has higher solids concentrations than activated sludge effluent. A low/high system at the Tillamook WWTP would likely have on the order of 215 bulbs and would draw approximately 54 kW at peak flow.

The proposed system would need a new contact tank for installation of the UV equipment. Ballasts and control panels would need to be located in a new building constructed for this purpose.

The estimated comparison cost for a new UV system including a new contact tank and building is $1,530,000. The estimated comparative operation cost for the 20-year planning period is $35,000 per year.

9.5.4 Summary of Disinfection Alternatives

Table 9.5 below compares the costs for each of the alternatives along with a present worth analysis giving the capital cost plus the present worth of the annual comparative operating cost (COC) over the 20-year planning period. Costs are presented here for comparison of alternatives.

Table 9.5: Summary of Disinfection Costs (2004 dollars)

Alternative	Estimated Capital Cost	Annual COC	Comparative Present Worth Cost [a]
D1: Gas Chlorine and Dechlorination	$1,396,000	$48,000	$2,102,000
D2: Liquid Hypochlorite and Dechlorination	$1,400,000	$55,000	$2,209,000
D3: Ultraviolet Radiation (b)	$1,530,000	$45,000	$2,192,000

Note:

(a) Assumes an annual interest rate of 5.125% and an annual inflation rate of 2%.
(b) Not recommended for use with fixed film liquid stream processes.

9.6 Effluent Disposal Alternatives

Effluent disposal alternatives include land application and surface water discharge.

9.6.1 Land Application

Land application of wastewater effluent includes ground infiltration and water reuse (irrigation). Both methods generally require large areas. Difficulties associated with land application of effluent by either method include:

- Large amounts of land are typically required.
- It is important that the land be in relatively close proximity to the treatment plant to minimize pumping or transmission costs.
- Typically a large pump station is required to pump effluent to the site.
- Large storage basins are typically required to store effluent during rainy periods when it cannot be land applied.
- The water quality required for land application varies, but can be quite high if any human contact with the effluent will take place.

Due to the significant rainfall in the area, the high cost of land, and the lack of available land, land application is not a viable option for Tillamook and will not be considered further.

9.6.2 Surface Water Discharge

The current method of effluent disposal is surface water discharge. A mixing zone evaluation for the discharge to the Trask was completed by Kennedy/Jenks Consultants in 2003. The evaluation recommended that the outfall be improved by lowering the discharge pipe and extending it slightly to provide better mixing and dilution.

A mixing zone analysis was also completed by DEQ with the 2003 update to the City's MAO. It also indicated that improvements to the existing outfall would be necessary.

Given that the City's current and anticipated future NPDES permit limits are achievable through "standard" secondary treatment processes, it is assumed that discharge to the Trask River is the most reasonable and cost effective alternative.

9.6.2.1 E1: Outfall Modifications

It is recommended that the existing outfall be modified to provide better mixing. The DEQ has indicated that a new outfall pipe should be extended to the river bottom, a few feet from the edge of the bank. A more extensive outfall modification is not recommended since large trees and other debris in the river during floods would damage or destroy an outfall configuration extending into the river.

The estimated comparison cost for a new outfall, including a new effluent manhole, temporary partial cofferdam of the Trask, and work on the bottom of the Trask is $35,000. The estimated cost for the updated mixing zone analysis, assuming the river data in the existing model can be reused, is approximately $10,000.

No operating costs are associated with the outfall.

9.7 Biosolids Management

Management of biosolids from the treatment plant encompasses several steps: treatment (typically volatile solids reduction), volume reduction (thickening or dewatering), and disposal. All components must work together and adhere to the standards set forth in the Code of Federal Regulations 40 CFR Part 503 and 40 CFR Part 257 (See Section 7).

For purposes of clarity, this subsection will briefly describe each of the three steps and then several alternatives will be developed and discussed.

9.7.1 Treatment

Sludge is treated to reduce pathogens and vector attraction. Two sludge treatment options were evaluated: aerobic digestion and anaerobic digestion. Both of these processes can produce Class B biosolids as defined in 40 CFR 503.

9.7.1.1 Aerobic Digestion

In the aerobic digestion process, volatile solids in sludge are reduced in a tank that is supplied with air, normally by means of blowers and diffusers or floating aerators. Federal pathogen reduction regulations for municipal sludge require a minimum solids retention time (SRT) of 40 days at 20 degrees Celsius (°C) [68 degrees Fahrenheit (°F)] or 60 days at 15 °C (59 °F). In the moderate climates of the Pacific Northwest, aerobic digesters are typically designed for a 60-day SRT.

Typically, aerobic digestion is not recommended for primary or co-thickened sludge. This is due to the extremely high air requirements of primary sludge. Aerobic digestion is only recommended for liquid treatment processes that do not produce primary sludge. Because fixed-film treatment processes (all options except the oxidation ditch) generally require primary clarification, their use precludes the use of aerobic digestion.

Further stabilization can be achieved and the size of the aerated tank can be reduced if a facultative sludge lagoon is used after aerobic digestion. Additional VSS reduction and thickening takes place and reduction in volatile solids of up to 50 percent are possible. The lagoons are periodically dredged and the sludge disposed of, typically on a yearly basis. Facultative sludge lagoons are open-air retention ponds, typically 10 to 15 feet deep, and can require significant amounts of land. While a facultative lagoon is often a very cost effective solids treatment, thickening, and storage process, it appears inappropriate for Tillamook due to the land requirements and the potential for odors.

The primary disadvantages of aerobic digestion are the potential for odors and relatively high operating costs for blowers or surface aerators. The primary advantages of aerobic digestion are the low capital cost and relatively simple operation.

9.7.1.2 Anaerobic Digestion

In the anaerobic digestion process, sludge is digested in a closed oxygen-free tank where sludge gases (chiefly methane and carbon dioxide) and water are produced. The gas can be flared (burned off) or used to heat the digester contents. The process has been in use for over a century and is extremely reliable. The Tillamook WWTP currently uses anaerobic digestion for sludge treatment.

The anaerobic digester for a plant this size would likely be a single stage digester. The circular concrete tank could have a fixed or floating cover. If used in conjunction with a covered sludge storage tank, the digester cover could be fixed. Mixing can be provided with a draft tube (sludge is withdrawn from the tank and returned) or a slow-speed turbine mixer (similar to the existing anaerobic digester), or by an externally-mounted pump. Heat is provided by a natural gas or propane boiler in conjunction with a heat exchanger. Cost savings can be realized if the boiler is also configured to burn the methane byproduct of anaerobic digestion. Digested sludge is drawn from the tank for disposal or for transfer to the sludge storage tank.

The primary advantages associated with anaerobic digestion are low energy costs and the potential for slightly thicker (up to 3 or 4%) solids production. The primary disadvantages are high capital costs, operational complexity, and production of a strong supernatant that must be recycled and treated.

9.7.2 Volume Reduction

Sludge volume is reduced to minimize transportation and disposal costs. Ultimately, the disposal method will dictate the amount and type of volume reduction necessary and will indicate which type of volume reduction is required.

9.7.2.1 Thickening

Thickening is generally defined as the increase in solids content from 1 or 2 percent solids up to 4 to 6 percent solids. Common thickening methods include gravity thickening, gravity belt thickening, rotary drum thickening, or screw press thickening. As a precursor to thickening, sludge is often combined with polymer to aid in flocculation and to facilitate the release free water. If solids are thickened before stabilization, significant energy savings can be realized as the need to heat excess water is eliminated. A mechanical thickener is typically operated during staff working hours and requires periodic operator attention.

9.7.2.2 Dewatering

Dewatering is generally defined as the increase in solids content from 1 to 3 percent up to 12 to 25 percent solids. For this size of plant, typical dewatering equipment includes belt filter presses and centrifuges. Dewatering equipment is typically more expensive to install and more difficult to operate and maintain than thickening equipment.

Typically, a belt filter press is less expensive than a centrifuge but requires a larger building. Equipment required in addition to the thickening device would include polymer feed equipment, sludge pumps, flow meters, and dewatered sludge conveyance/storage. A dewatering unit is typically operated during staff working hours and requires periodic operator attention for proper operation.

9.7.3 Disposal

Currently, biosolids from the Tillamook WWTP are land-applied at about 4 to 6 percent solids. The disposal site is owned and managed by others. Biosolids are hauled approximately five miles to the site by City staff in a 1,000-gallon Vactor truck. The City is considering other disposal options.

Sludge disposal options depend on the type of sludge treatment provided. Class A sludge has very few restrictions on its disposal because it is sufficiently treated for distribution to the public for use on lawns or home gardens. Class B sludge must be applied to sites that have restricted access and use. It is assumed that the Tillamook WWTP will continue to produce Class B sludge due to the high cost to produce Class A sludge.

9.7.3.1 Liquid and Cake Land Application

The regulations pertaining to application sites from the Oregon Administrative Rules Chapter 340, Division 50 include the following:

- Slope: In general, liquid biosolids should not be surface applied on bare solids where the ground slope exceeds 12 percent. Well vegetated sites with slopes up to 30 percent may be used for cake biosolids.
- Soil Depth: Soil should have a minimum rooting depth of 24 inches.
- Soil Type: Sites with saline and/or sodic soils should be avoided.

- Depth to Groundwater: For application of liquid biosolids, the minimum depth to permanent groundwater should be 4 feet and the minimum depth to temporary groundwater should be one foot.

- Buffers: Recommended buffers are as follows:

  - Surface Water or Ditch: 50 feet

  - Domestic Water Source or Well: 200 feet

  - Road: Varies depending on type and amount of traffic

  - Residential Area: Buffer strip large enough to prevent nuisance odors or, for application of liquid biosolids, wind drift.

Biosolids are applied by spraying liquid or thickened sludge from a fixed or truck-mounted cannon or by flinging and tilling in dewatered sludge. If a truck or trailer-mounted system is used, access trails are needed at intervals of 250 feet or less.

The amount of land required for disposal depends on the quantity (dry tons) of biosolids to be applied, the nitrogen content of the biosolids, and the nitrogen uptake capacity of the land. Preliminary information provided by DEQ indicates that for the Tillamook WWTP, the application rate for aerobically digested biosolids would be 3.1 dry tons per acre per year and the application rate for anaerobically digested biosolids would be 3.6 dry tons per acre per year. Table 9.6 below shows the estimated land required for the two types of biosolids.

Table 9.6: Area Requirements for Land Application

Liquid Stream Type	Area (acres)
Anaerobic Digestion	50
Aerobic Digestion	60

9.7.3.2 Landfill

Removal of biosolids to a landfill is also an alternative and can be cost effective depending on the hauling distance and the charge from the landfill. Landfilled biosolids are not required to meet the 40 CFR 503 regulations. The only requirement is that the biosolids must pass the paint filter test, which tests for liquid that will migrate through a specific filter in a set amount of time.

The primary benefit of landfill disposal is the reduced testing and reporting requirements.

The closest landfill to the Tillamook WWTP is just outside Corvallis, Oregon at Coffin Butte. Given the significant distance to this landfill (approximately 80 miles), it would not be cost effective to haul to the landfill.

9.7.4 Biosolids Alternatives

9.7.4.1 S1: Aerobic Digestion of Sludge with Mechanical Dewatering

This alternative includes construction of a 0.24 MG primary aerobic digester, conversion of the existing secondary clarifiers to secondary aerobic digesters, covers for all three digesters, an odor control system, and blowers and diffusers for the digester tanks, a mechanical dewatering device (centrifuge or belt filter press), construction of a building to house the dewatering unit and its appurtenances, and construction of an area for biosolids cake storage sized for up to four months. The combined volume of the primary and secondary aerobic digesters equals 60 days SRT, providing treatment to Class B standards.

This alternative produces biosolids at 14 to 16 percent solids. At the 20-year design conditions, and assuming land application for eight months of the year, hauling of approximately 15 loads per month at 2,500 gallons per load will be required. It will also require operator attention to the dewatering unit approximately one day per week. Assuming an additional four hours per load, this would require 0.55 FTE. The estimated land required for the 20-year design conditions is approximately 60 acres.

The estimated capital cost for aerobic digestion with mechanical dewatering is $2,161,000.

The estimated comparative operation cost for the 20-year planning period is $23,600 per year.

9.7.4.2 S2: Sludge Thickening plus Anaerobic Digestion and New Sludge Storage

This alternative includes construction of a new anaerobic digester with fixed cover and external draft tube for mixing, a propane/methane boiler for heating of the digester contents, and construction of a building to house the boiler and appurtenances, This alternative also includes construction of a sludge storage tank sized for up to four months storage, and floating cover with gas collection for the sludge storage tank. This assumes that a propane tank will be leased. Preliminary estimates indicate that the digester will produce sufficient methane to fire the boiler under normal operating conditions. The volume of the anaerobic digester equals 20 days HRT, allowing treatment to Class B standards.

This alternative produces biosolids at 3 to 4 percent solids. At the 20-year design conditions and assuming application for only 2/3 of the year, this will required hauling of approximately 60 loads per month at 2,500 gallons per load. Assuming four hours per load, this would require 1.1 FTE. The estimated land required for the 20-year design conditions is approximately 50 acres.

Digestion and storage tankage volume as well as hauling requirements could be reduced with the use of a gravity thickener. The thickener would be a small (12-foot diameter) concrete tank and its construction cost would offset by the smaller digestion and storage tankage. The primary disadvantages of a gravity thickener would be the added operational complexity and the additional space required. For purposes of this estimate, it is assumed that a gravity thickener is not installed.

The estimated comparison cost for anaerobic digestion is $2,238,000.

9.7.4.3 S3: Sludge Thickening plus Anaerobic Digestion with Mechanical Dewatering

This alternative includes construction of all items found in Alternative S2 above minus the sludge storage tank and including a mechanical dewatering device (centrifuge or belt filter press), construction of a building to house the dewatering unit and its appurtenances, and construction of an area for biosolids cake storage sized for up to four months. The volume of the anaerobic digester equals 20 days HRT, allowing treatment to Class B standards.

This alternative produces biosolids at 14 to 16 percent solids. At the 20-year design condition, and assuming application for only eight months of the year, this will require hauling of approximately 15 loads per month at 2,500 gallons per load. It will also require operator attention to the dewatering unit approximately one day per week. Assuming an additional four hours per load for hauling, this would require 0.55 FTE. The estimated land required for the 20-year design conditions is approximately 50 acres.

The estimated capital cost for anaerobic digestion with mechanical dewatering is $1,889,000.

The estimated comparative operation cost for the 20-year planning period is $3,100 per year.

9.7.4.4 S4: Sludge Thickening plus Anaerobic Digester with Converted Sludge Storage

This alternative includes construction of all items found in Alternative S2 however, instead of building new sludge storage, the existing 45-foot diameter secondary clarifiers will be covered and converted to sludge storage tanks. The volume of the anaerobic digester would provide 20 days HRT, meeting requirements for treatment to Class B standards.

This alternative produces biosolids at 3 to 4 percent solids. At the 20-year design conditions and assuming application for only eight months of the year, this will require hauling of approximately 60 loads per month at 2,500 gallons per load. Assuming four hours per load, this would require 1.1 FTE. The estimated land required for the 20-year design conditions is approximately 50 acres.

Digestion and storage tankage volume as well as hauling requirements could be reduced with the use of a gravity thickener. The thickener would be a small (12-foot diameter) concrete tank and its construction cost would offset by the smaller digestion and storage tankage. The primary disadvantages of a gravity thickener would be the added operational complexity and potential for odor generation. For purposes of this estimate, it is assumed that a gravity thickener is not installed.

The estimated land required for the 20-year design conditions is approximately 50 acres.

The estimated capital cost for anaerobic digestion with mechanical dewatering is $2,583,000.

The estimated comparative operation cost for the 20-year planning period is $3,100 per year.

9.7.5 Summary of Biosolids Alternatives

Table 9.7 below compares the costs for each of the alternatives outlined above. Costs are presented here for comparison of alternatives.

Table 9.7: Summary of Biosolids Alternatives

Alternative	Estimated Capital Cost (2004 dollars)	Estimated Average O&M Cost
S1: Aerobic Digestion with Mechanical Dewatering	$2,161,000	$23,600
S2: Sludge Thickening, Anaerobic Digestion, and New Sludge Storage	$2,238,000	$3,100
S3: Sludge Thickening, Anaerobic Digestion and Mechanical Dewatering	$1,889,000	$3,100
S4: Sludge Thickening, Anaerobic Digestion with Converted Sludge Storage	$2,583,000	$3,100

9.8 Development and Evaluation of Complete Upgrade Alternatives

Seven complete alternatives for upgrading the Tillamook WWTP were developed for evaluation. This section will present a summary of each complete alternative along with a preliminary site plan and cost estimate of the capital costs by phases.

9.8.1 Alternative 1: SAS/RBC

The existing primary clarifier and RBCs would continue to be used.

Alternative 1 includes the following unit processes:

- IPS2: Submersible Influent Pump Station
- HW1: Headworks Improvements
- T1: Parallel Selector Activated Sludge / RBC
- F1: Effluent Filter with Pumping
- D1: Gas Chlorine with Dechlorination
- E1: New Outfall
- S2: Anaerobic Digestion and New Sludge Storage
- Assumed Sludge Disposal: Land Application of Liquid, Class B Sludge.

Operator requirements for this system, based on the criteria and points system in OAR 340-049, requires a Class IV operator for a total of 80 points.

The average yearly operational cost for this complete alternative, including power and chemicals but not including staff, is approximately $111,000 per year.

A site plan of Alternative 1 is presented on Figure 9.1. The estimated comparison cost for Alternative 1 is estimated to be $9,000,000. Potential phasing for Alternative 1 is presented in Table 9.8.

Table 9.8: Alternative 1 Phasing

Phase	Capital Cost (2004 Dollars)
Phase 1 (Now): Influent Pump Station, Headworks Improvements, SAS Basins, Replace ½ RBC Shafts and Discs, 3rd Secondary Clarifier, Gravity Thickener, Anaerobic Digester, ½ Sludge Storage, Sludge Truck, Outfall, Standby Power, and Site Work	$6,140,000
Phase 2 (5 Years): 4th Secondary Clarifier, ½ RBC Shafts and Discs, Remainder Sludge Storage	$1,660,000
Phase 3 (10 Years): Effluent Filter, Disinfection Improvements	$1,200,000
Total	**$9,000,000**

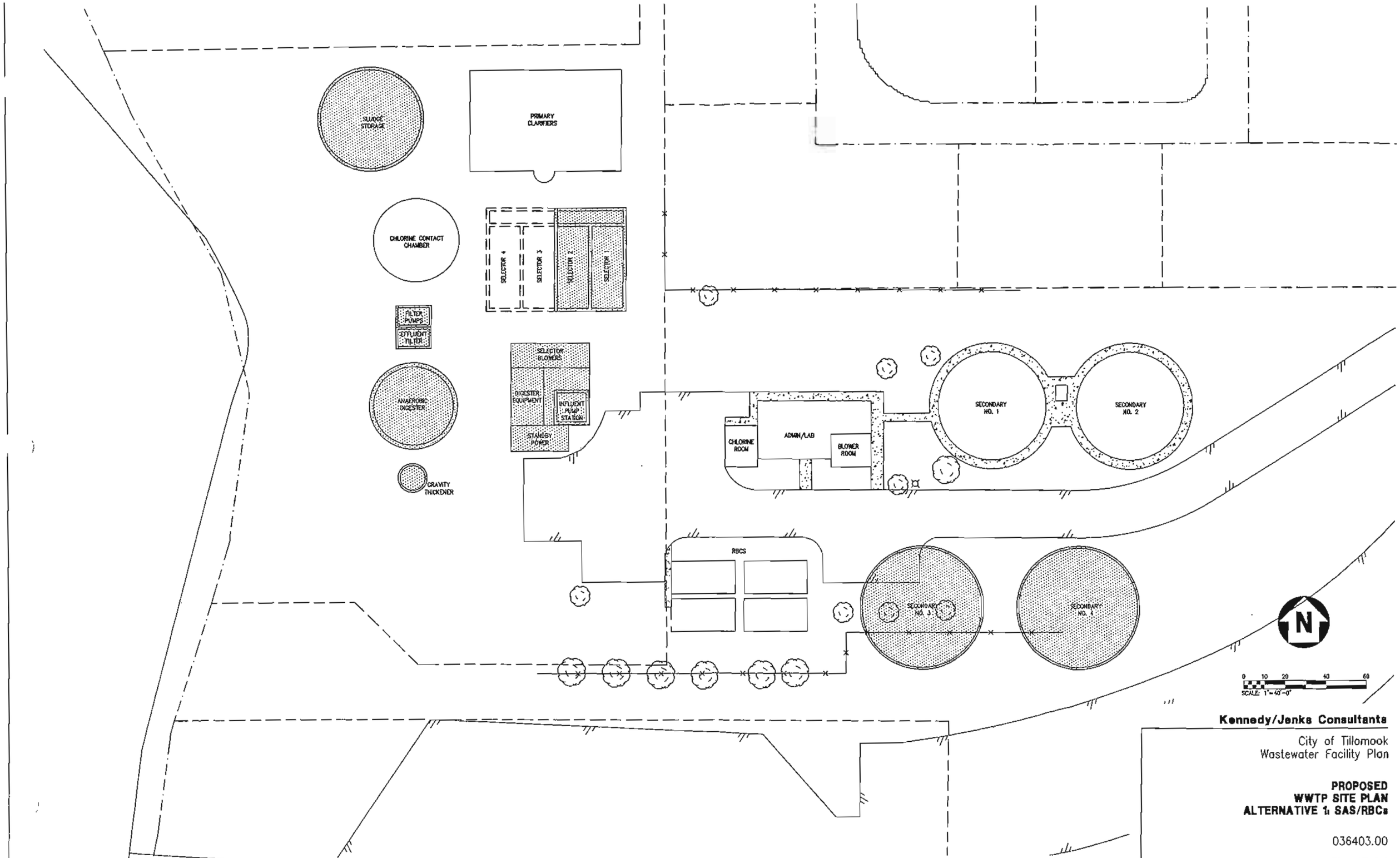
9.8.2 Alternative 2: Biotower/RBCs

The existing primary clarifier and RBCs would continue to be used.

Alternative 2 includes the following unit processes:

- IPS2: Submersible Influent Pump Station
- HW1: Headworks Improvements
- T2: Series Biotower / RBC
- F1: Effluent Filter with Pumping
- D1: Gas Chlorine with Dechlorination
- E1: New Outfall
- S2: Sludge Thickening, Anaerobic Digestion, and Storage at 3.5% TSS
- Assumed Sludge Disposal: Land Application of Liquid, Class B Sludge.

Operator requirements for this system, based on the criteria and points system in OAR 340-049, requires a Class IV operator for a total of 77 points.



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Wastewater Facility Plan

PROPOSED
WWTP SITE PLAN
ALTERNATIVE 1: SAS/RBCs

036403.00

FIGURE 9.1

The average yearly operational cost for this complete alternative, including power and chemicals but not including staff, is approximately $102,000 per year.

A site plan of Alternative 2 is presented on Figure 9.2. The estimated comparison cost for Alternative 2 is estimated to be $9,270,000. Potential phasing for Alternative 2 is presented in Table 9.9.

Table 9.9: Alternative 2 Phasing

Phase	Capital Cost (2004 Dollars)
Phase 1 (Now): Influent Pump Station, Headworks Improvements, Biotower and Biotower Pump Station, Replace ½ RBC Shafts and Discs, 3rd Secondary Clarifier, Gravity Thickener, Anaerobic Digester, ½ Sludge Storage, Sludge Truck, Outfall, Standby Power, and Site Work	$6,480,000
Phase 2 (5 Years): 4th Secondary Clarifier, ½ RBC Shafts and Discs, Remainder Sludge Storage	$1,590,000
Phase 3 (10 Years): Effluent Filter, Disinfection Improvements	$1,200,000
Total	**$9,270,000**

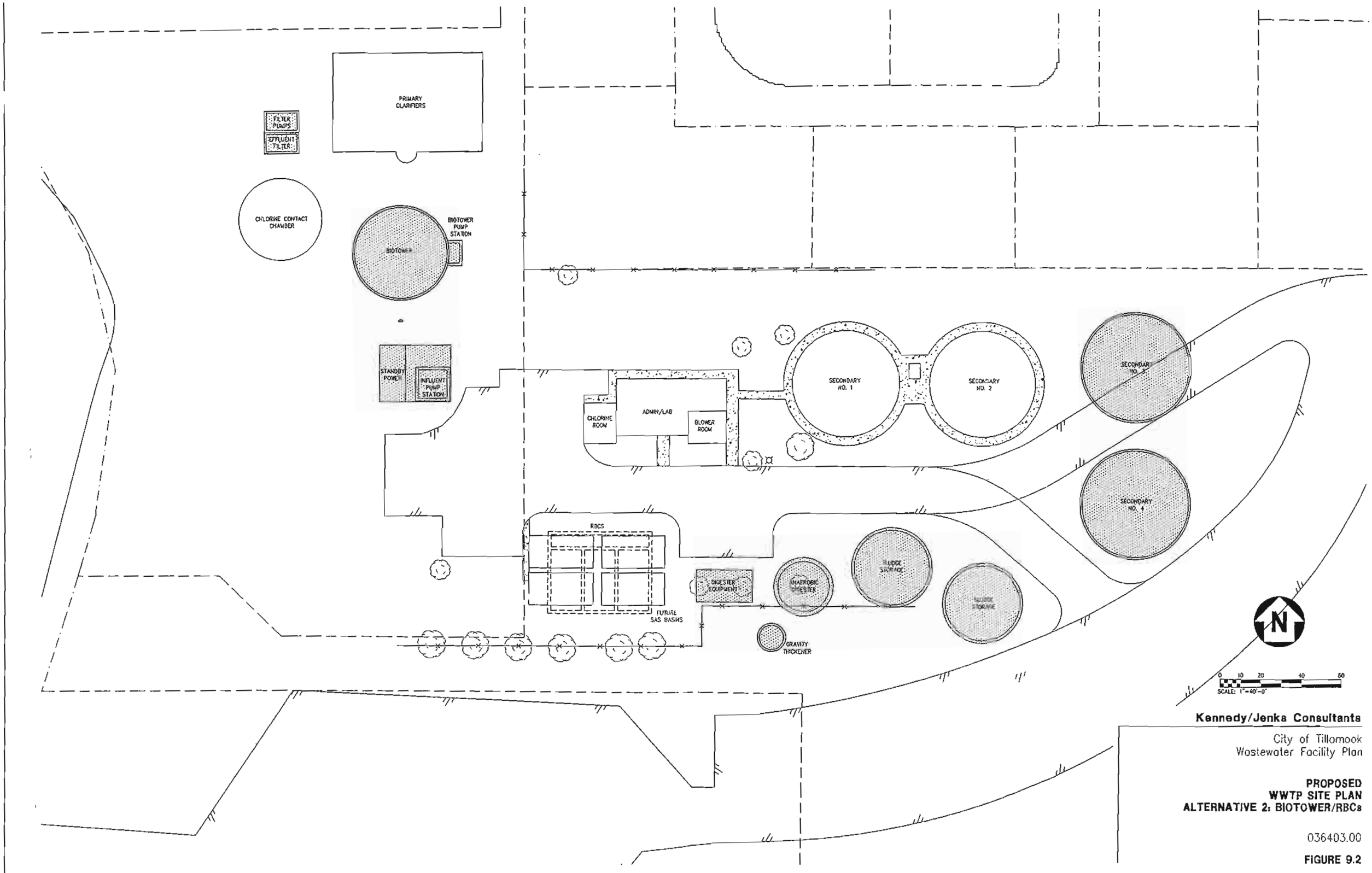
9.8.3 Alternative 3: Oxidation Ditch with no Primary Clarifiers (OXDnoPC)

The existing rectangular primary clarifier would be converted to secondary clarifiers.

Alternative 3 includes the following unit processes:

- IPS2: Submersible Influent Pump Station
- HW1: Headworks Improvements
- T3: Oxidation Ditch
- D1: Gas Chlorine with Dechlorination
- E1: New Outfall
- S1: Aerobic Digestion, Dewatering, and Cake Storage
- Assumed Sludge Disposal: Land Application of Cake, Class B Sludge.

Operator requirements for this system, based on the criteria and points system in OAR 340-049, requires a Class III operator for a total of 65 points.



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City of Tillamook
Wastewater Facility Plan

PROPOSED
WWTP SITE PLAN
ALTERNATIVE 2: BIOTOWER/RBCs

036403.00

FIGURE 9.2

The average yearly operational cost for this complete alternative, including power and chemicals but not including staff, is approximately $121,000 per year.

A site plan of Alternative 3 is presented on Figure 9.3. The estimated comparison cost for Alternative 3 is estimated to be $8,590,000. Potential phasing for Alternative 3 is presented in Table 9.10.

Table 9.10: Alternative 3 Phasing

Phase	Capital Cost (2004 Dollars)
Phase 1 (Now): Influent Pump Station, Headworks Improvements, Oxidation Ditch, Convert Primary Clarifiers to Secondaries, 3rd Secondary Clarifier, Aerobic Digester, Outfall, Standby Power, and Site Work	$6,637,000
Phase 2 (5 Years): 4th Secondary Clarifier, Dewatering Equipment and Building, Cake Sludge Truck	$1,557,000
Phase 3 (10 Years): Disinfection Improvements and	$396,000
Total	**$8,590,000**

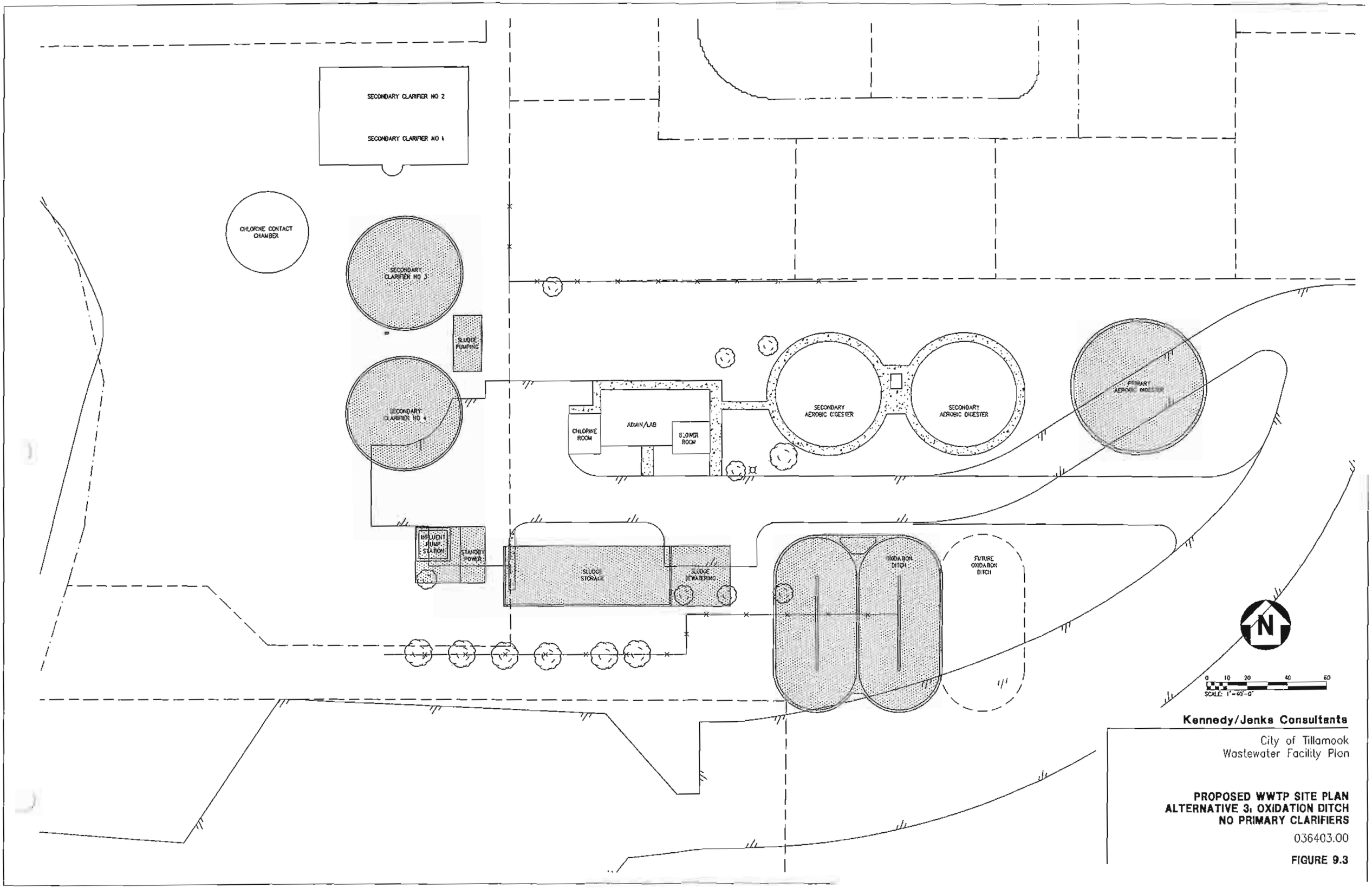
9.8.4 Alternative 4: Additional RBCs (RBC/RBC)

The existing primary clarifier would continue to be used.

Alternative 4 includes the following unit processes:

- IPS2: Submersible Influent Pump Station
- HW1: Headworks Improvements
- T4: Additional RBCs
- F1: Effluent Filter with Pumping
- D1: Gas Chlorine with Dechlorination
- E1: New Outfall
- S2: Sludge Thickening, Anaerobic Digestion, and Storage at 3.5% TSS
- Assumed Sludge Disposal: Land Application of Liquid, Class B Sludge.

Operator requirements for this system, based on the criteria and points system in OAR 340-049, requires a Class III operator for a total of 70 points.


SECONDARY CLARIFIER NO 2
SECONDARY CLARIFIER NO 1
CHLORINE CONTACT CHAMBER
SECONDARY CLARIFIER NO 3
SLUDGE PUMPING
SECONDARY CLARIFIER NO 4
CHLORINE ROOM
ADMIN/LAB
BLOWER ROOM
SECONDARY AEROBIC DIGESTER
SECONDARY AEROBIC DIGESTER
PRIMARY AEROBIC DIGESTER
INFLUENT PUMP STATION
STANDBY POWER
SLUDGE STORAGE
SLUDGE DEWATERING
OXIDATION DITCH
FUTURE OXIDATION DITCH
N
0 10 20 40 60
SCALE: 1"=40'-0"
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Wastewater Facility Plan
PROPOSED WWTP SITE PLAN
ALTERNATIVE 3: OXIDATION DITCH
NO PRIMARY CLARIFIERS
036403.00
FIGURE 9.3

The average yearly operational cost for this complete alternative, including power and chemicals but not including staff, is approximately $96,000 per year.

A site plan of Alternative 4 is presented on Figure 9.4. The estimated comparison cost for Alternative 4 is estimated to be $9,260,000. Potential phasing for Alternative 4 is presented in Table 9.11.

Table 9.11: Alternative 4 Phasing

Phase	Capital Cost (2004 Dollars)
Phase 1 (Now): Influent Pump Station, Headworks Improvements, New RBCs, Replace ½ RBC Shafts and Discs, 3rd Secondary Clarifier, Gravity Thickener, Anaerobic Digester, ½ Sludge Storage, Sludge Truck, Outfall, Standby Power, and Site Work	$6,470,000
Phase 2 (5 Years): 4th Secondary Clarifier, ½ RBC Shafts and Discs, Remainder Sludge Storage	$1,590,000
Phase 3 (10 Years): Effluent Filter, Disinfection Improvements	$1,200,000
Total	**$9,260,000**

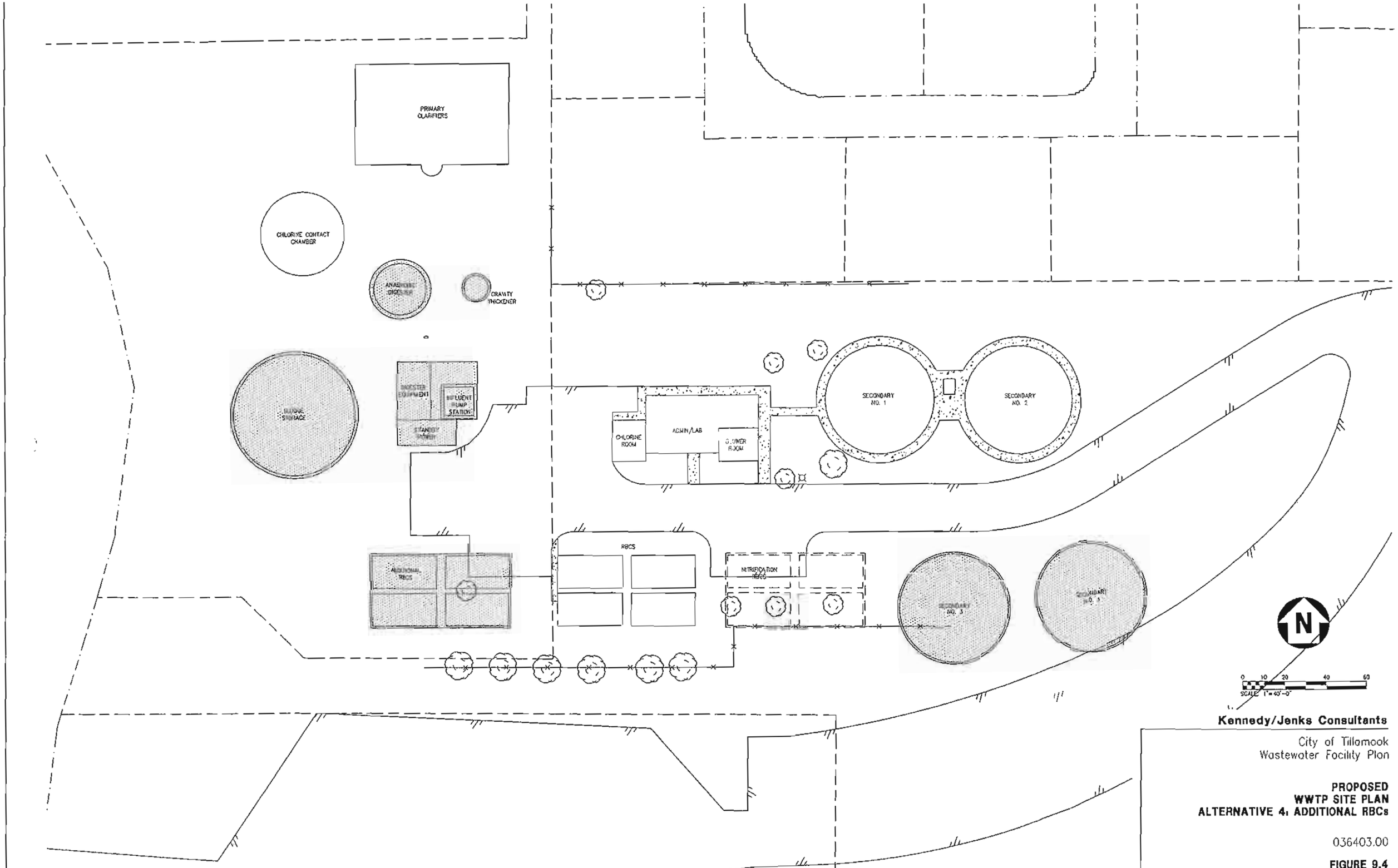
9.8.5 Alternative 5: Oxidation Ditch with Primary Clarification (OXDwPC)

The existing primary clarifier would continue to be used.

Alternative 7 includes the following unit processes:

- IPS2: Submersible Influent Pump Station
- HW1: Headworks Improvements
- T5: Oxidation Ditch preceded by Primary Clarification
- D1: Gas Chlorine with Dechlorination
- E1: New Outfall
- S2: Sludge Thickening, Anaerobic Digestion, and Storage at 3.5% TSS
- Assumed Sludge Disposal: Land Application of Liquid, Class B Biosolids.

Operator requirements for this system, based on the criteria and points system in OAR 340-049, requires a Class III operator for a total of 70 points.



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Wastewater Facility Plan

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ALTERNATIVE 4: ADDITIONAL RBCs

036403.00

FIGURE 9.4

The average yearly operational cost for this complete alternative, including power and chemicals but not including staff, is approximately $92,000 per year.

A site plan of Alternative 5 is presented on Figure 9.5. The estimated comparison cost for Alternative 5 is estimated to be $8,160,000. Potential phasing for Alternative 5 is presented in Table 9.12.

Table 9.12: Alternative 5 Phasing

Phase	Capital Cost (2004 Dollars)
Phase 1 (Now): Influent Pump Station, Headworks Improvements, New Oxidation Ditch, 2- new Secondary Clarifiers, Sludge Thickening, Anaerobic Digester, Convert Existing Tankage to Sludge Storage, Sludge Truck, Outfall, Standby Power, and Site Work	$7,300,000
Phase 2 (5 Years)	No Plant Improvements
Phase 3 (10 Years): Disinfection Improvements and Sludge Storage	$860,000
Total	**$8,160,000**

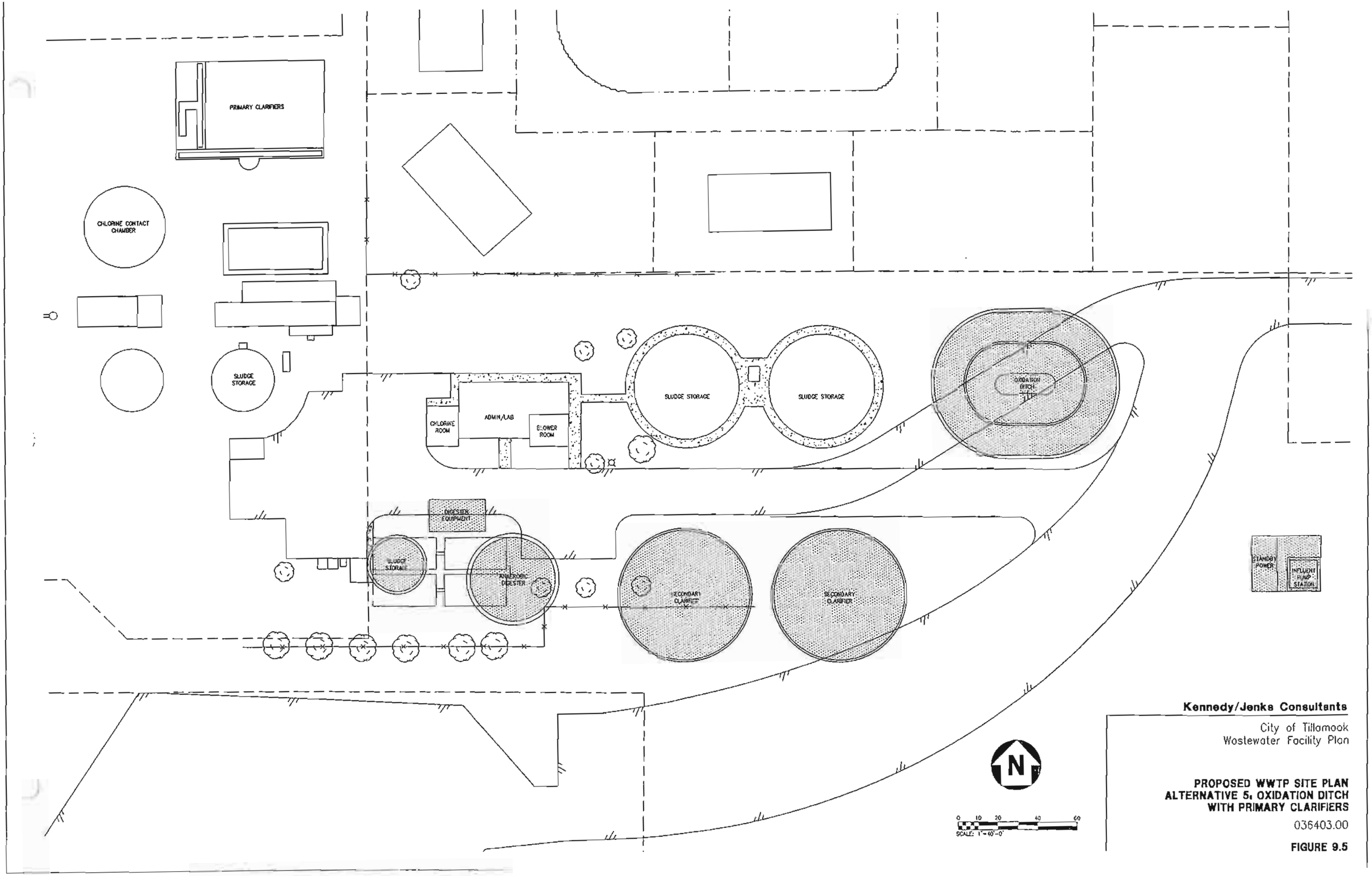
9.8.6 Alternative 6: Sequencing Batch Reactors (SBRs)

The existing primary clarifiers would continue to be used.

Alternative 7 includes the following unit processes:

- IPS2: Submersible Influent Pump Station
- HW1: Headworks Improvements
- T6: Sequencing Batch Reactors
- D1: Gas Chlorine with Dechlorination
- E1: New Outfall
- S2: Sludge Thickening, Anaerobic Digestion, and Storage at 3.5% TSS
- Assumed Sludge Disposal: Land Application of Liquid, Class B Sludge.

Operator requirements for this system, based on the criteria and points system in OAR 340-049, requires a Class III operator for a total of 70 points.



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City of Tillamook
Wastewater Facility Plan

PROPOSED WWTP SITE PLAN
ALTERNATIVE 5, OXIDATION DITCH
WITH PRIMARY CLARIFIERS

036403.00

FIGURE 9.5

The average yearly operational cost for this complete alternative, including power and chemicals but not including staff, is approximately $89,000 per year.

A site plan of Alternative 6 is presented on Figure 9.6. The estimated comparison cost for Alternative 6 is estimated to be $8,860,000. Potential phasing for Alternative 6 is presented in Table 9.13.

Table 9.13: Alternative 6 Phasing

Phase	Capital Cost (2004 Dollars)
Phase 1 (Now): Influent Pump Station, Headworks Improvements, New SBRs and Flow Equalization, Sludge Thickening, Anaerobic Digester, Convert Existing Tankage to Sludge Storage, Sludge Truck, Outfall, Standby Power, and Site Work	$8,000,000
Phase 2 (5 Years):	No Plant Improvements
Phase 3 (10 Years): Disinfection Improvements and Sludge Storage	$860,000
Total	**$8,860,000**

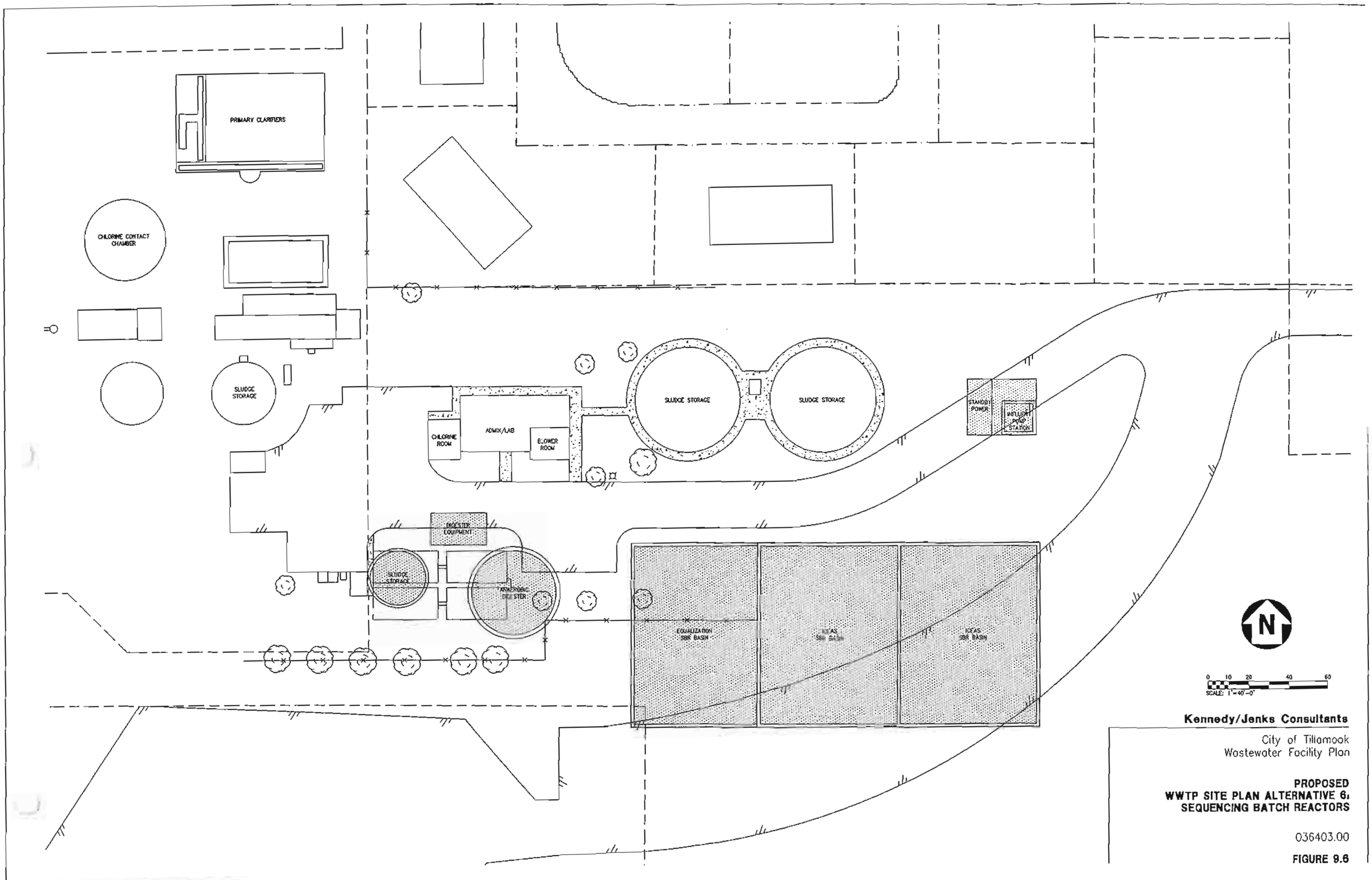
9.8.7 Alternative 7: Nitrifying Selector Activated Sludge (NSAS)

The existing primary clarifier would continue to be used.

Alternative 7 includes the following unit processes:

- IPS2: Submersible Influent Pump Station
- HW1: Headworks Improvements
- T5: Nitrifying Selector Aeration Basins preceded by Primary Clarification
- D1: Gas Chlorine with Dechlorination
- E1: New Outfall
- S2: Sludge Thickening, Anaerobic Digestion, and Storage at 3.5% TSS
- Assumed Sludge Disposal: Land Application of Liquid, Class B Biosolids.

Operator requirements for this system, based on the criteria and points system in OAR 340-049, requires a Class III operator for a total of 70 points.



Kennedy/Jenks Consultants

City of Tillamook
Wastewater Facility Plan

PROPOSED
WWTP SITE PLAN ALTERNATIVE 6:
SEQUENCING BATCH REACTORS

036403.00

FIGURE 9.6

The average yearly operational cost for this complete alternative, including power and chemicals but not including staff, is approximately $90,000 per year.

A site plan of Alternative 7 is presented on Figure 9.7. The estimated comparison cost for Alternative 7 is estimated to be $8,420,000. Potential phasing for Alternative 7 is presented in Table 9.14.

Table 9.14: Alternative 7 Phasing

Phase	Capital Cost (2004 Dollars)
Phase 1 (Now): Influent Pump Station, Headworks Improvements, New Selector Aeration Basins, 2- new Secondary Clarifiers, Sludge Thickening, Anaerobic Digester, Convert Existing Tankage to Sludge Storage, Sludge Truck, Outfall, Standby Power, and Site Work	$7,560,000
Phase 2 (5 Years)	No Plant Improvements
Phase 3 (10 Years): Disinfection Improvements and Sludge Storage	$860,000
Total	**$8,420,000**

9.8.8 Comparison and Evaluation

A comparison of these alternatives is presented in Table 9.15 below. It summarizes costs and qualities of each of the four alternatives.

Table 9.15: Alternative Comparison

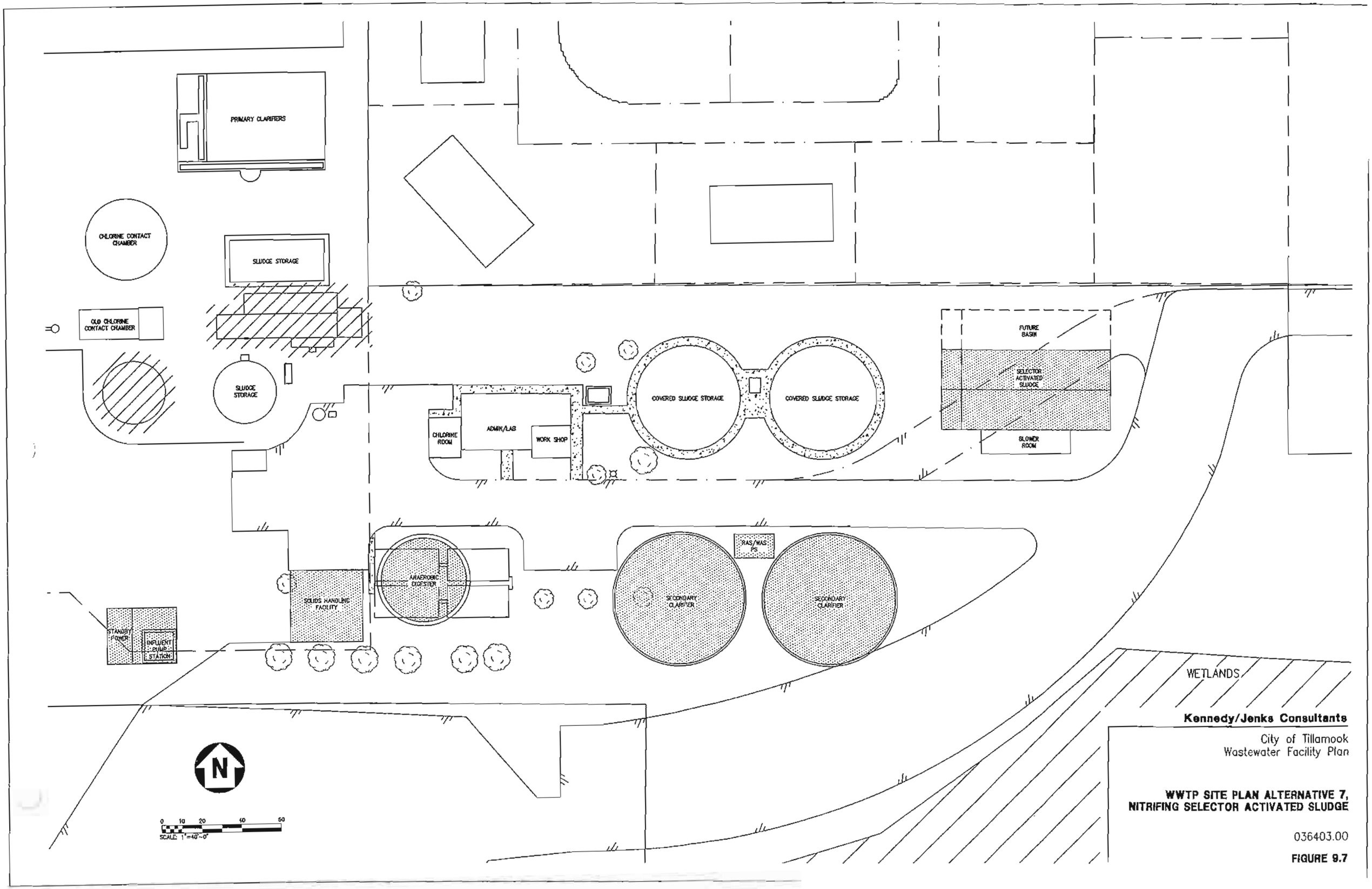
Criteria	1 SAS/RBC	2 BT/RBC	3 OXDnoPC	4 RBC/RBC	5 OXDwPC	6 SBR	7 NSAS
Capital Cost (M $)	$9.00	$9.27	$8.59	$9.26	$8.16	$8.86	$8.42
Comparative Yearly Operating Cost (1000 $)	$111	$102	$121	$96	$92	$89	$90
Comparative Present Worth (M $) [(a) (b)]	$10.4	$10.5	$10.1	$10.4	$9.3	$10.0	$9.5
Mechanical Complexity	–	0	+	0	0	0	0
Process Control	–	+	+	+	+	0	+
Handling of High Flows	–	++	0	+	0	-	0
Odors	0	-	0	+	0	0	0
Effluent Quality[(c)]	+	+	++	–	++	++	++
Land Requirements	0	+	-	0	-	-	0
Sludge Handling	0	+	-	+	0	0	0

Notes:

(a) Assumes an interest rate of 5.125% per year over 20 years
(b) Includes Capital Plus Capital Cost plus PW of Operating Cost
(c) All processes are expected to meet discharge limits.

Legend:

++ = Very Good
+ = Good
0 = Average
− = Poor


PRIMARY CLARIFIERS
CHLORINE CONTACT CHAMBER
SLUDGE STORAGE
OLD CHLORINE CONTACT CHAMBER
SLUDGE STORAGE
CHLORINE ROOM
ADMIN/LAB
WORK SHOP
COVERED SLUDGE STORAGE
COVERED SLUDGE STORAGE
FUTURE BASIN
SELECTOR ACTIVATED SLUDGE
BLOWER ROOM
SOLIDS HANDLING FACILITY
ANAEROBIC DIGESTER
RAS/WAS PS
SECONDARY CLARIFIER
SECONDARY CLARIFIER
STANDBY POWER
INFLUENT PUMP STATION
WETLANDS
N
0 10 20 40 60
SCALE: 1"=40'-0"
Kennedy/Jenks Consultants
City of Tillamook
Wastewater Facility Plan
WWTP SITE PLAN ALTERNATIVE 7,
NITRIFING SELECTOR ACTIVATED SLUDGE
036403.00
FIGURE 9.7

Section 10: Apparent Best Alternative

10.1 Selection Process

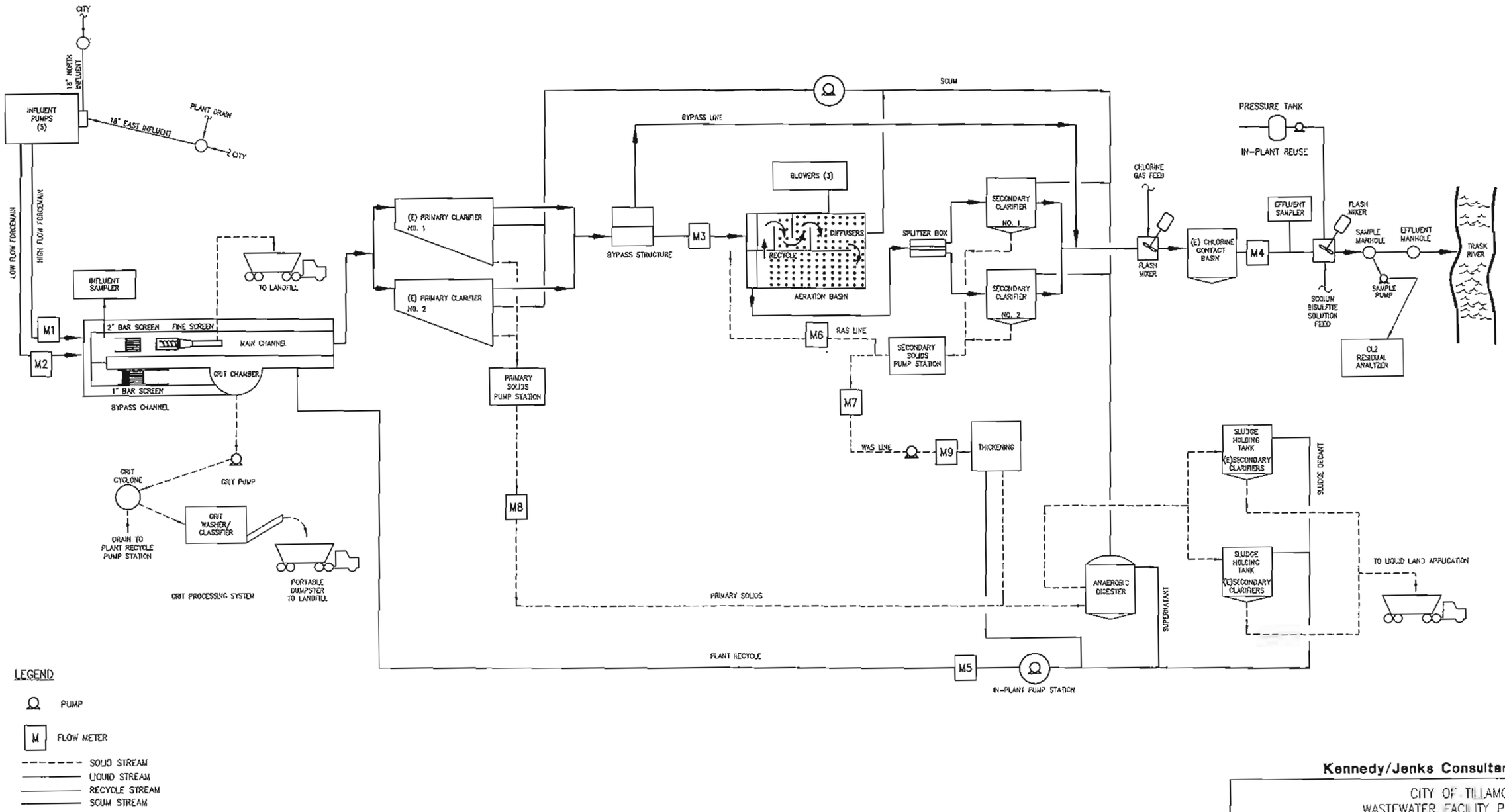
The Apparent Best Process is based on the Nitrifying Selector Activated Sludge (NSAS) treatment alternative. The recommendations for expansion of the WWTP include the following elements:

- New Influent Pump Station
- Headworks Improvements including a new influent screen and bypass channel
- New Activated Sludge Process – including:
  - Flexible Aeration Tank with dual, parallel basins
  - Selector zones for control of filamentous sludge
  - Diffused aeration equipment and blowers
  - Maximum side water depth to minimize land area requirements
- Two new Secondary Clarifiers with a new RAS and WAS pumping facility
- New chlorine contact tank with dual basins and chemical flash mixers
- Reconstructed submerged effluent outfall
- New Solids Handling Facility including a mechanical thickening device, a new anaerobic digester, a septage receiving station, and conversion of existing basins for sludge storage
- New Sludge Hauling Truck
- Consideration of a City-owned site for land application of Class B biosolids

A system flow schematic, site plan, and hydraulic profile for the Apparent Best Alternative are shown in Figures 10.1, 10.2, and 10.3, respectively. Table 10.1 includes a description of unit processes and design criteria for the Apparent Best Alternative.

10.2 Site Location Discussion

At the time the final version of this Facility Plan was being produced, the adjacent property to the north of the existing WWTP site was offered for sale. This property, referred to as the North Site, would provide several advantages over the existing site for construction of the new treatment facility. In August of 2005, the City and the property owner agreed on a purchase price for the property contingent upon available funding, a favorable geotechnical investigation,



LEGEND

- PUMP
- M FLOW METER
- SOLID STREAM
- LIQUID STREAM
- RECYCLE STREAM
- SCUM STREAM

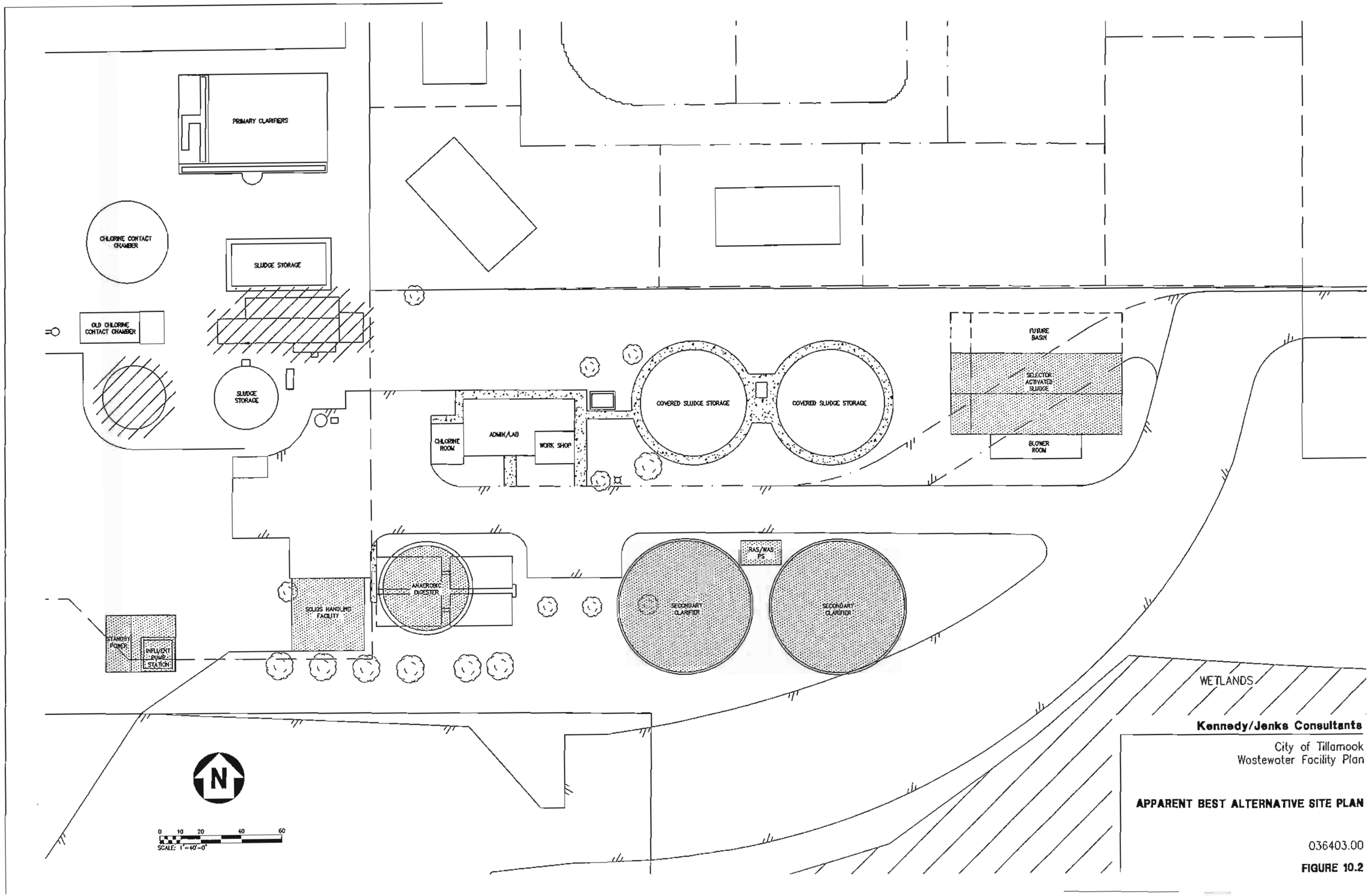
Kennedy/Jenks Consultants

CITY OF TILLAMOOK
WASTEWATER FACILITY PLAN

APPARENT BEST ALTERNATIVE,
SIMPLIFIED FLOW SCHEMATIC
(NITRIFING SELECTOR ACTIVATED SLUDGE)

036403.00

FIGURE 10.1



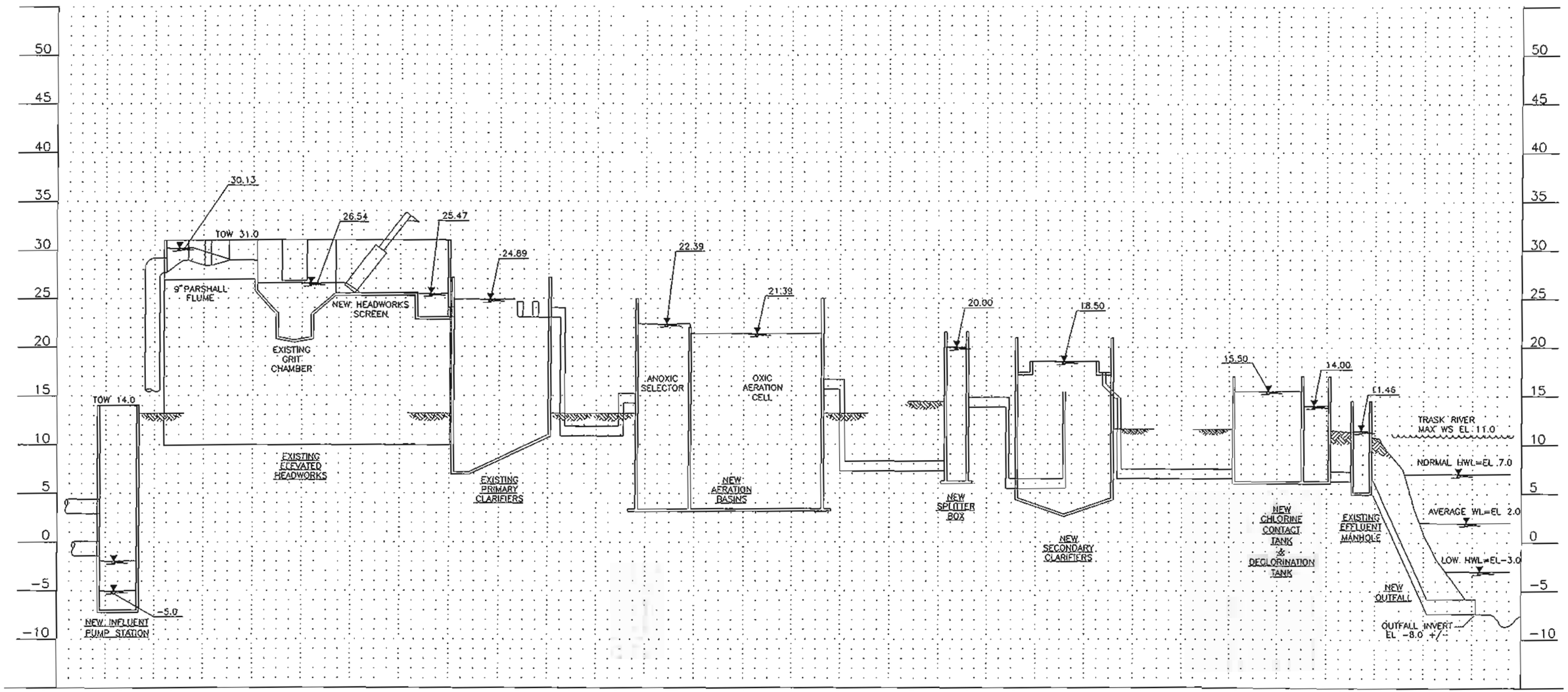
Kennedy/Jenks Consultants

City of Tillamook
Wastewater Facility Plan

APPARENT BEST ALTERNATIVE SITE PLAN

036403.00

FIGURE 10.2



VERTICAL SCALE: 1"=10'

Kennedy/Jenks Consultants

City of Tillamook
Wastewater Facility Plan

APPARENT BEST ALTERNATIVE HYDRAULIC PROFILE

036403.00

FIGURE 10.3

Table 10.1: Apparent Best Upgrade Alternative - Units Sizes and Descriptions

Unit Description and Loading			
Item	Value		
	Year 2003	Design	Comments
Population	4625	7515	
PLANT LOADING			
Dry Weather Flow (mgd)			
Average (ADWF)	0.6	0.9	
Maximum Month (MMDWF)	1.5	2.1	
Wet Weather Flow (mgd)			
Average (AWWF)	1.9	2.5	
Maximum Month (MMWWF)	3.2	4.0	
Peak Day (PDF)	6.0	7.2	
Peak Instantaneous (PIF)	8.5	10.0	
Average Raw Wastewater Load (lb/d)			
BOD	925	1500	
TSS	925	1500	
Primary Effluent Load (lb/d)			
BOD (30% Removal)	650	1050	140 mg/l
TSS (60% Removal)	370	600	80 mg/l
Waste Primary Solids (lb/d or gal/d)			
TSS load in lb TSS/d	560	900	
VSS load in lb VSS/d	390	630	
Volume in gal @ 3. 5% TSS	1920	3080	
Waste Secondary Solids (lb/d or gal/d)			
TSS load in lb TSS/d	460	740	
VSS load in lb VSS/d	370	590	
Volume in gal @ 0.8% TSS	6900	11100	
Volume in gal @ 4.0% TSS	1400	2200	
Waste Combined Solids (lb/d or gal/d)			
TSS load in lb TSS/d	1000	1600	
VSS load in lb VSS/d	800	1200	
Inert Solids load in lb/d	200	400	
Volume in gal per day	3300	5300	
Percent Solids prior to Digestion	3.6	3.6	
Percent Solids after Digestion (50% VSS reduction)	2.2	2.3	
VSS load in lb VSS/d after Digestion	400	600	
TSS load in lb TSS/d after Digestion	600	1000	
UNIT LOADING AND DESCRIPTION			
Existing in 2005 to be Demolished or Converted to Other Uses			
Comminutor	To be Removed		
Secondary Clarifiers	Converted to Sludge Storage		236,000 gal
South Primary Clarifiers	Abandond in-place		107000.0 gal
Waste Solids Pumping Station	Equipment Salvaged		
Rotating Biological Contactors	To be Demolished		
RBC Air Blowers	Equipment Salvaged		
Outfall	To be Replaced		
Influent Pump Station	To be Demolished		
Primary Digester	Abandond in-place		67,000 gal
Secondary Digester	Converted to Sludge Storage		84,500 gal

Table 10.1: Apparent Best Upgrade Alternative - Units Sizes and Descriptions

Unit Description and Loading			
Item	Value		
	Year 2003	Design	Comments
Population	4625	7515	
Influent Pump Station			
New Submersible Station			
Depth	20 ft		
Type	Rectangular, Cast in Place		
Width	12 ft		
Length	16 ft		
New Influent Pumps			
Type	Centrifugal solids-handling, submerisble		
Number and Capacity in MGD	2 @ 5.01		
Number and Capacity in MGD	2@ 2.50		
Head capacity each	35 ft TDH		
New Level Control	Ultrasonic level sensor		
Headworks			
Existing Channel Width	30 Inches		
Existing Flow Measurement			
Type	Parshall Flume		
Opening Size	12-inch		
Peak Flow Capacity	10.4 MGD		
Existing Grit Removal			
Type	Vortex		
Size	8 ft diameter		
Peak Flow Capacity	10 MGD		
Manufacturer	Smith&Loveless Pista Model 4A		
Existing Comminutor	To be Removed		
New Fine Screen			
Type	In-channel basket perforated plate		
Opening Size	1/4 inch		
Peak Flow Capacity	10 MGD		
Primary Settling			
Existing Primary Clarifiers			
Number	2		
Width, each	20 ft		
Length	60 ft		
Depth	13.7 ft		
Loading criteria (at PDF)	3000 gpd/sqft		
Biological Treatment			
New Nitrifying Activated Sludge Basins			
Aeration Basins			
Number Basins	2		
Total MCRT	12.2 days		
Oxic MCRT	9.1 days		
Organic Loading Rate	18 lb BOD/d/1000 cu ft		
MLSS	2500 mg/l		
Total mass of sludge	9000 lb TSS		
Basin Volume, each	0.22 mgal		
Basin depth, each	18 ft		
Basin width, each	20 ft		
Basin length	82 ft		
HRT at ADWF (not incl RAS)	11.7 hrs		
HRT at MMWWF (not incl RAS)	2.6 hrs		
Oxygen demand criteria			
Total Required	2000 lb O_2/d		
Normal Use	1200 lb O_2/d		
New Aeration Diffusers			
Type	Fine Bubble - Cyclic		
New Air Blowers			
Total air flow required	720 scfm		
Type	Centrifugal or Positive Dispalcement		
Discharge pressue	9.5 psi		
Number (includes 1- standby)	3		
Capacity Each	360 scfm		
Size Each	30 bhp		
New Anoxic Selector Mixing			
Basin mixing each, total	8 hp		

Table 10.1: Apparent Best Upgrade Alternative - Units Sizes and Descriptions

Unit Description and Loading			
Item	Value		
	Year 2003	Design	Comments
Population	4625	7515	
Secondary Settling			
New Secondary Clarifiers			
Number	2		
Diameter	65 ft		
Surface Area, each	3317 sq ft		
Depth	14 ft		
One clarifier in Service			
Overflow Rate - ADWF	270 gpd/sq ft		
Overflow Rate - MMDWF	630 gpd/sq ft		
Hydraulic Retention Time - ADWF	9 hrs		
Hydraulic Retention Time - MMDWF	4 hrs		
Solids Loading Rate - ADWF (RAS = $0.5xQ_{ADWF}$)	8 lb TSS/d-sq ft		
Solids Loading Rate - MMDWF (RAS = $1.0xQ_{ADWF}$)	19 lb TSS/d-sq ft		
Two clarifiers in Service			
Overflow Rate - AWWF	380 gpd/sq ft		
Overflow Rate - MMWWF	600 gpd/sq ft		
Hydraulic Retention Time - AWWF	7 hrs		
Hydraulic Retention Time - MMWWF	4 hrs		
Solids Loading Rate - AWWF (RAS = $0.5xQ_{ADWF}$)	9 lb TSS/d-sq ft		
Solids Loading Rate - MMWWF (RAS = $1.0xQ_{ADWF}$)	15 lb TSS/d-sq ft		
New Sludge Pumping			
Return Activated Sludge Pumps			
Number	3		
Type	Solids Handling Centrifugal		
Capacity, each	325 gpm		
Waste Activated Sludge Pumps			
Number	2		
Type	Solids Handling Centrifugal		
Capacity, each	50 gpm		
Disinfection			
Existing Contact Chamber			
Diameter	40 ft		
Depth	9 ft		
Volume	84600 gal		
Contact Time at ADWF	135 min		
Contact Time at MMWWF	30 min		
Existing Chlorination System			
Type	Automatic Gas, 150 lb cylinders		
Assumed chlorine dosage	8 mg/L		
Chlorine demand at MMWWF	268 ppd		
Capacity of Automatic Chlorinator	300 ppd		
Existing Dechlorination System			
Type	Automatic, liquid Sodium Bisulfite		
Sodium Bisulfite Design Dosage	5 mg/L		
Sodium Bisulfite Demand at MMWWF	167 lb/day		
Volume	2312 gal/mo		
Sodium Bisulfite Storage Volume	1100 gal		
Delivery frequency	14 days		

Table 10.1: Apparent Best Upgrade Alternative - Units Sizes and Descriptions

Unit Description and Loading			
Item	Value		
	Year 2003	Design	Comments
Population	4625	7515	
Effluent Flow Measurement			
Existing Meter Type	Velocity x Area Meter		
New Mechanical WAS Thickening			
Type	Gravity Belt Thickener		
Frequency of Use			
Days per Week	4 days		
Hours per Day	6 hours		
Feed rate	60 gpm		
Size	1 meter		
Final Solids Concentration	4.0 % Solids		
New Anaerobic Digester			
Number	1		
Diameter	33 ft		
Affective Sludge Depth	20 ft		
Volume	128,000 gal		
Hydraulic Retention Time (HRT)	24 days		
Solids Loading	0.070 lb VSS/cuft-day		
Assumed volatile solids reduction	50 %		
Mixing			
Mixing Type	Draft Tube		
Mixing Demand Criteria	0.05 hp/1000 gal		
Mixing horsepower req'd	10 hp		
Heating			
Type	Methane / Propane Boiler		
Sludge Storage			
Stored Solids Characteristics			
TSS Load	1000 lb/d		
Description			
Existing Clarifiers			
Number	2		
Depth	10 ft		
Diameter	45 ft		
Volume Each	118,000 gal		
Existing Seconday Digester			
Number	1		
Volume Each	84,500 gal		
Total Dedicated Storage Volume	320,500 gal		
Dedicated Sludge Storage Time (days)			
at 2.5% Solids	67 days		
at 3.5% Solids	94 days		

an environmental review, and clear title. If funding is available and the contingencies are resolved favorably, it is likely that the new treatment facility will be constructed on the North Site.

10.3 Existing Site

The existing WWTP site has a high concentration of buried pipes, which are especially concentrated in two areas:

- Between the existing pump station and the property line to the east.
- Between the south primary clarifier and existing chlorine contact tank.

These pipes were constructed during the periodic modifications that were made to the plant over the past 55 years. Due to the location of the existing primary clarifier and the need to connect the new influent pump station, the new secondary treatment process, and the new sludge processing facility to this structure, a vast number of new pressure and gravity lines will need to be laid in these congested areas. Installation of pipes in these areas will result in significantly higher costs than would the same pipes constructed in an open area. Additionally, a lack of detailed location drawings for the existing pipes could result in schedule delays and change orders during construction.

The existing site will have limited space for efficient expansion. If the City needs to expand the plant beyond the next planning horizon, it could not be done cost-effectively on the current site.

The need to demolish existing treatment units to make room for new solids processing facilities will require that construction of the new facility be done in two distinct phases in order to maintain treatment during construction.

10.4 North Site

The North Site is essentially a "green field" site as it has no existing structures or buried utilities. From a construction standpoint, it offers many potential advantages as follows:

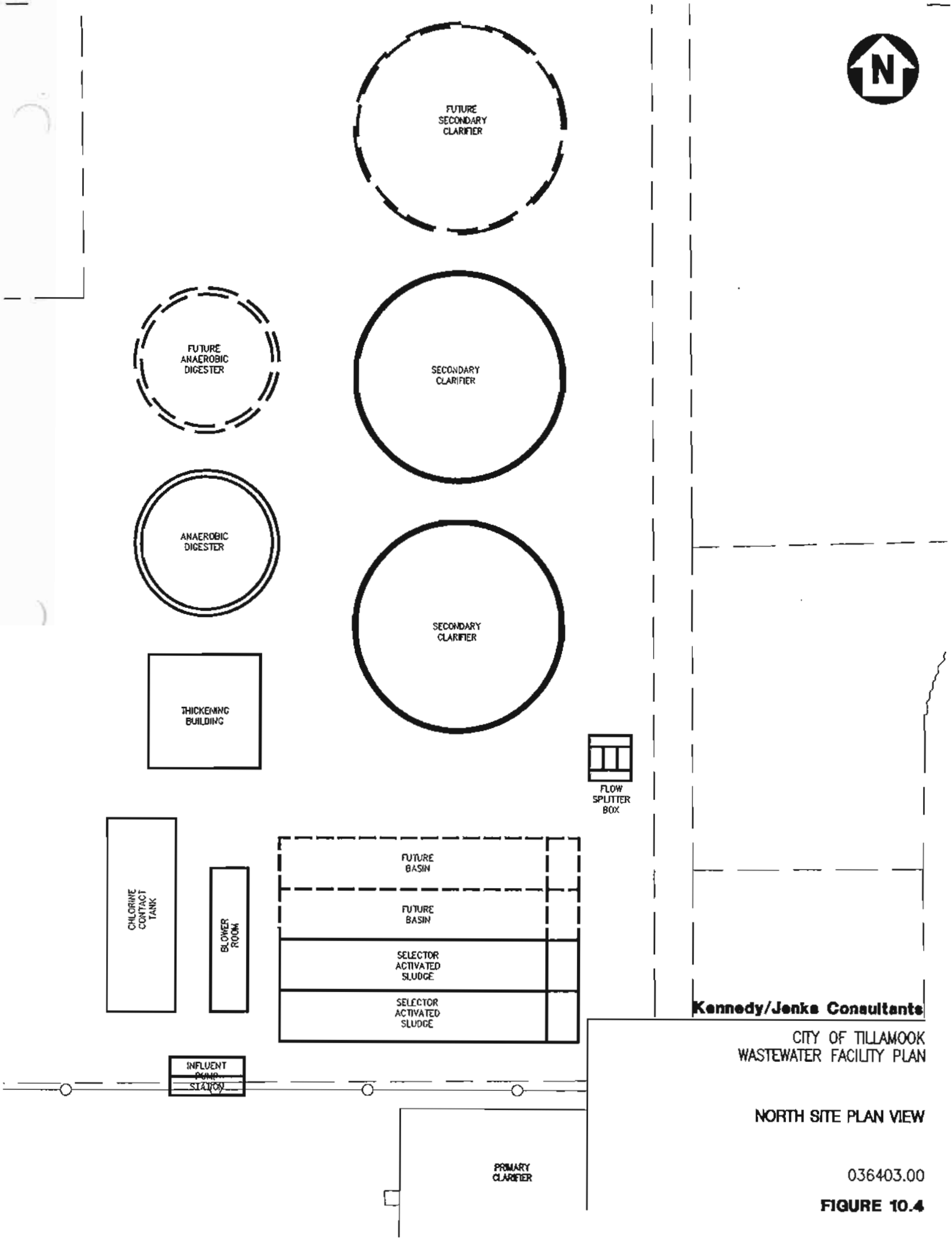
- unobstructed alignment of new piping
- room for future expansion
- more efficient layout of basins and processes
- reduced pipe length between basins
- better access to unit processes
- removal of uncertainty with regards to existing pipe and structures
- increased construction efficiency

Perhaps the most important advantage of building on the North Site from a construction cost standpoint would be in the elimination of the need to phase the construction in order to keep the existing plant in operation during construction. Remobilization of the various trades to build a

two-phase project could potentially add 10% to the total project cost. Another major advantage to building on the North Site would be in the elimination of construction interference with the daily operation of the treatment plant.

If the North site is utilized for the WWTP expansion, several of the existing structures on the existing site will be retained including the Administration/Control Building, the secondary clarifiers (converted to sludge storage), the supernatant pump station (converted to the septage receiving pump station), and the primary and secondary digester (used for sludge storage).

A possible site layout for the North Site is shown on Figure 10.4. The spatial relationship between the existing site and the North Site is shown on Figure 10.5. The flow schematic and hydraulic profile for this site would essentially be the same as shown on Figures 10.1 and 10.3.



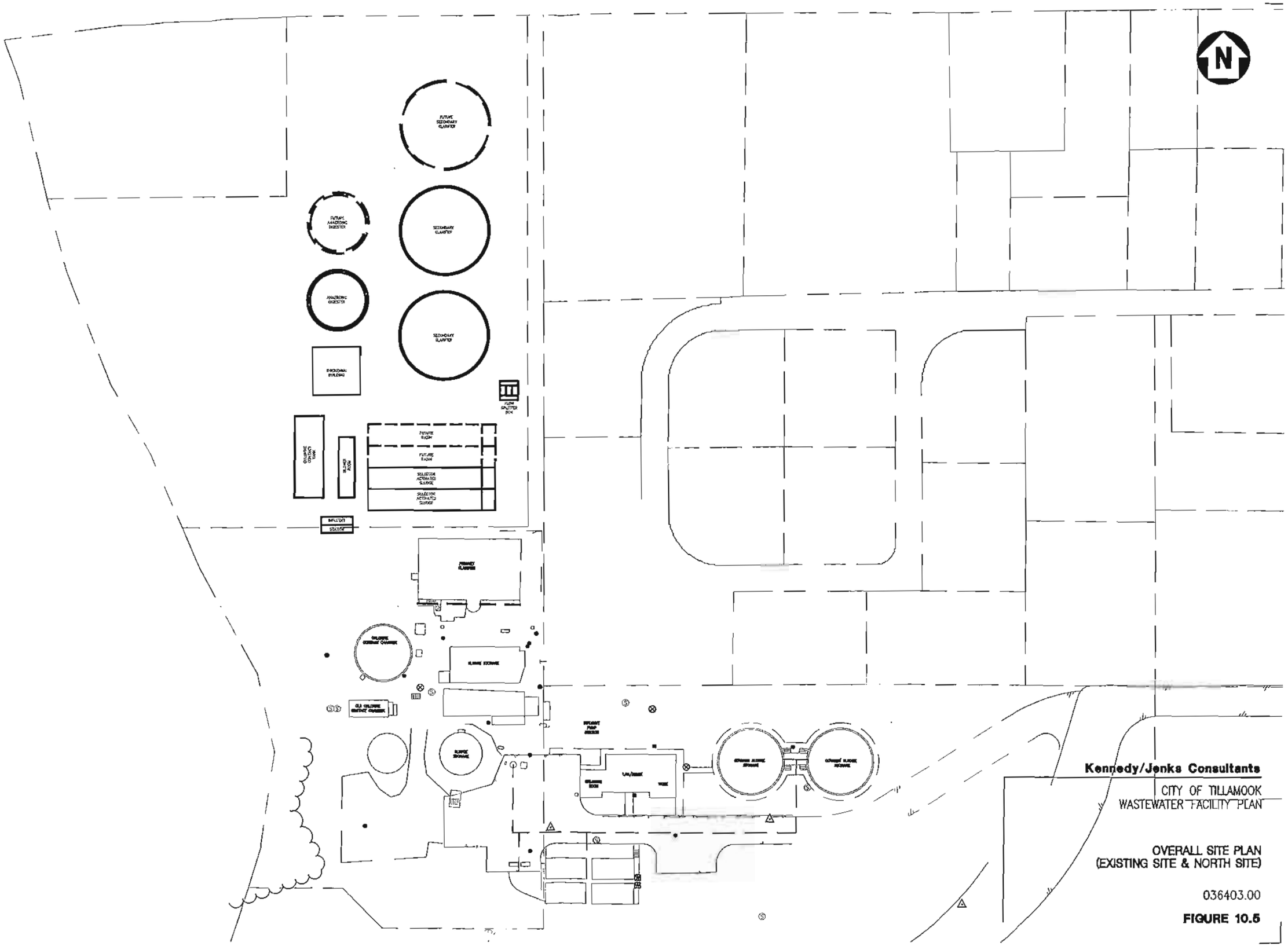
Kennedy/Jenks Consultants

CITY OF TILLAMOOK
WASTEWATER FACILITY PLAN

NORTH SITE PLAN VIEW

036403.00

FIGURE 10.4



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CITY OF TILLAMOOK
WASTEWATER FACILITY PLAN

OVERALL SITE PLAN
(EXISTING SITE & NORTH SITE)

036403.00

FIGURE 10.5

Section 11: Funding Alternatives

There are multiple state and federal loan and grant funding programs that can fund public works projects. Each has its own requirements, terms, advantages, and limitations. The typical process to obtain a loan or grant is as follows:

1. Define the proposed project and its anticipated cost.
2. Identify likely funding sources with programs having eligibility requirements that the project could satisfy.
3. Schedule a "Public Works Financial Assistance One Stop Meeting" that brings the stakeholders and the potential funding agencies together for a group discussion.
4. Determine which funding programs best meet the needs of the project.
5. Contact the funding agencies and work toward getting qualified and making an application for the loan or grant program.

Timing is the key to securing the preferred funding instrument as some programs require substantial lead time and may disburse funds only at prescribed times during the year.

11.1 Local Funding Options

In addition to state and federal grant and loan programs local financing alternatives exist in the form of tax-free bonds. Bonds are essentially loans made to the City by an underwriter. The underwriter sells the bonds to investors. The City agrees to repay the principal and interest at a predetermined interest rate either through utility revenue or through tax revenue. Common types of bonds are General Obligation Bonds and Revenue Bonds. The Bancroft Bond is another, less common type of bond.

11.2 General Obligation Bonds

General Obligation Bonds (GO Bond) are a type of Municipal Bond where principal and interest are secured by the full faith and credit of the issuer and usually supported by either the issuer's unlimited or limited taxing power. General Obligation Bonds must be voter-approved. General Obligation Bonds have the following features that make them attractive to investors:

- Income from GO Bonds is free from federal taxes, and in some cases, it is also free from state and local taxes.
- Income from interest and repayment of principal is generally regarded with a high degree of safety.
- If circumstances change, GO Bonds are actively traded in the secondary market. The bearer is not locked in until the bond's maturity.

The bearer can receive semi-annual interest payments and get their principal payment back at maturity

11.3 Revenue Bonds

A revenue bond is one that is repaid from system user fees. Unlike a GO bond, funds levied from taxes cannot be used to make annual payments of principal and interest. Revenue bonds lack the security associated with the authority to tax to make payments to retire the debt. For this reason, the stability and financial performance history of the community is key to providing an assurance of repayment. Since revenue bonds do not carry the guarantee of taxation, the interest rates charged by the underwriter for these bonds are typically higher than those for a GO bond.

11.4 Bancroft (Improvement) Bonding

Bancroft Bonds for wastewater system improvements are allowed under Oregon law. Cities are limited by the amount of bonding capacity available under this method of finance. Oregon law requires that Bancroft Bonds not exceed three percent of a community's true cash value. Developed and undeveloped property share equally in the burden created by a wastewater system improvement. An assessment for improvements is developed and a lien against the property is perfected. Owners have the option of paying for their share of the improvements in cash or over time. When Bancroft Bonds are sold to finance construction of wastewater system improvements, the resulting assessment can be paid off over a 20-year period in semi-annual installments.

11.5 State and Federal Programs

There are three main state and federal agencies that administer a total of five funding programs to finance wastewater improvements. Other region-specific funding programs and financing options may be available.

11.6 USDA Rural Utilities Services (RUS)

The U.S. Department of Agriculture Rural Development Administration (RD) administers two funding programs. RD can be contacted at:

http://www.usda.gov/rus/water/programs.htm

Oregon Rural Development
Water and Environmental Programs
101 SW Main, Suite 1410
Portland, OR 97204-3222
State Contact: Wayne Dunlap
Phone: (503) 414-3362

Regional Contact: David Force
Phone: (503) 414-3327

11.6.1 Water and Waste Disposal Loans and Grants

The funding program provides federal loans and grants to rural municipalities, counties, special districts, Indian tribes, and non-profit organizations to construct, enlarge, or modify water treatment and distribution systems and wastewater collection and treatment systems. Preference is given to projects in low-income communities with populations below 10,000. Grant and loan assistance is based on a tiered schedule, with the loan rate based on percent of Median Household Income (MHI). To qualify for the lowest loan rates the MHI to be less than 80% of Oregon MHI.

11.7 Oregon Department of Environmental Quality (DEQ)

Information regarding financing options through DEQ can be found at:

http://www.deq.state.or.us/wq/wqgrant/wqgrant.htm

Oregon Department of Environmental Quality
811 SW Sixth Avenue
Portland, OR 97204-1390
Contact: Richard Santner
Phone: (503) 229-5219

11.7.1 Clean Water State Revolving Fund (CWSRF)

The CWSRF provide low interest rate loans for use in the planning, design, and construction of water pollution control facilities. The program is focused on achieving and maintaining water quality standards within Oregon communities. Any publicly-owned utility within the state is eligible for a CWSRF loan. Applicants are prioritized in terms of relative project need during a periodic pre-application process.

11.8 Oregon Economic and Community Development Department (OECDD)

OECDD offers a number of funding programs including the Special Public Works Fund (SPWF) and the Water/Wastewater Financing Program which are applicable to wastewater related projects. Information on OECDD programs can be found at:

http://www.econ.state.or.us/index.htm

Headquarters (Salem)
775 Summer Street, NE
Suite 200
Salem, OR 97301-1280
Phone: (503) 986-0123

Local Contact: Vicki Goodman
Phone: (503) 842-4045

11.8.1 Special Public Works Fund (SPWF)

The program was created in 1985 by the Oregon State Legislature. It is capitalized through the issuance of state revenue bonds and though Oregon State lottery proceeds. The SPWF is intended to promote the creation of jobs for Oregonians. Loans and grants are issued through this program to facilitate the construction of public infrastructure to support industrial/manufacturing and eligible commercial development. Eligible commercial development is defined as activity that is marketed nationally or internationally and attracts business from outside of Oregon.

The program is open to municipalities as described in the SPWF Applicant's Handbook and generally includes cities, counties, water supply districts, water and wastewater authorities, sanitary districts, port authorities, water control districts, county service districts, and tribal councils.

11.8.2 Water/Wastewater Financing Program (W/W)

This program was created by the Oregon State Legislature in 1993. It is capitalized via the sale of state revenue bonds as well as with a portion of Oregon State's lottery proceeds. The primary purpose of the program is to provide financing for the construction of public infrastructure required to ensure compliance with the Safe Drinking Water Act or the Clean Water Act. Specifically, it is intended to assist local governments facing state and federal mandates relating to public drinking water systems and wastewater systems.

The program is available to cities, counties, water supply districts, water and wastewater authorities, sanitary districts, port authorities, water control districts, county service districts and tribal councils of Indian tribes. Funding levels awarded to qualified applicants are determined by a financial analysis based on demonstrated need and an inability to afford additional loans. Communities exhibiting low and moderate income receive priority. The maximum grant from this program is approximately $500,000; the maximum available loan amount is $10 million.

11.8.3 Community Development Block Grant Program (CDBG)

This federal program is administered at the state level by the OECDD. Eligibility standards are the same as those for other OECDD programs. The program is intended to assist low and moderate income communities. To meet the national policy objective for low and moderate income, 51% of the people benefiting from the project must fall in this income range.

CDBG grants of up to $750,000 are available for planning, design, and construction of wastewater system improvements. An eligible project must demonstrate compliance with the Safe Drinking Water Act, the Clean Water Act, and/or compliance requirements established by the Oregon Health Department or the Department of Environmental Quality.

11.9 City of Tillamook Preliminary Eligibility Summary

Table 11.1 provides a summary of the feedback from state and federal funding agencies with respect to the planned wastewater facility improvements.

Table 11.1: Preliminary Funding Program Eligibility Summary

Program	Eligibility
USDA Rural Utilities Services (RUS)	
Water and Waste Disposal Loans and Grants:	**Loans - Yes; Grants - Maybe**. Based on Tillamook's MHI from the 2000 Census, the City is eligible for low interest loans and up to 45% of a project may be grant eligible if user rates are on the order of $40/mo.
Oregon Department of Environmental Quality (DEQ)	
Clean Water State Revolving Fund (SRF)	**Yes - Loans Only.**
Oregon Economic and Community Development Department (OECDD)	
Special Public Works Fund (SPWF)	**Probably Not**. Funding of projects is linked to creation of jobs in the private sector. Wastewater improvements are not typically eligible for this type of funding unless they provide for private sector business growth.
Water/Wastewater Financing Program (W/W)	**Loans - Yes; Grants - Maybe**. User rates on the order of $33.60/mo are required before the City would be eligible for grant funding. There is currently the possibility of $0.5 M in grant funding available through this source.
Community Development Block Grant Program (CDBG)	**Possible**. The 2000 Census data indicates the City has only 48.1% low and moderate income households and is, therefore, ineligible (51% is required) for the CDBG grant monies. It is possible that the percentage of low and moderate income eligibility could be re-calculated through an independent survey.

11.10 Earmark Funding

Earmark funds are appropriated in federal budgets and have been "earmarked" for specific communities. They are acquired through lobbying efforts and are not constrained by population, income, or need. A lobbyist is normally retained to help obtain funding. There is no guarantee that funds would be provided and, if provided, the money would likely not be provided in one lump sum. The total amount may be spread over several years.

If the City is interested in pursuing earmark funding, a feasibility study that investigates the possibility of receiving funds would be recommended. The estimated cost to both evaluate feasibility and to hire a lobbyist is approximately $40,000.

Section 12: Capital Improvement Plan

The capital improvement plan (CIP) is a listing of the estimated costs for the recommended maintenance and capital improvement projects over the 20-year planning period. The CIP projects are intended to improve the operation and maintenance of the collection system, correct deficiencies in the collection and treatment systems, and provide the capacity to accommodate anticipated growth and development. The need for the CIP projects is discussed in Sections 4 and 5.

12.1 Basis of the Cost Estimates

Construction costs were estimated either on a unit cost or lump sum basis. For pipelines, concrete, and architectural structures, estimates are unit cost based. The base cost of equipment or mechanical systems for treatment plant or pump station improvements is based on quotations from equipment suppliers plus the estimated cost for installation. For all CIP components, the total cost was calculated by multiplying the base cost times a non-construction cost factor (NCCF). The basis for the NCCF is 1.9047 and consists of items listed in Table 12.1.

Table 12.1: Non-Construction Cost Factor Calculation

Item		Cost as Percent of Construction Cost
Taxes		0%
Contractor Overhead and Profit		15%
Escalation to Midpoint of Construction		6.17%
Contingency		20%
Engineering/Legal/Administrative		30%
Design Engineering	13%	
Construction and Startup Services	10%	
Administrative and Legal Expenses	3%	
Other Professional Services (Survey, Geotechnical)	4%	
Total Non-Construction Cost Factor = 1.9047 (1 x 1.15 x 1.0617 x 1.20 x 1.3)		

12.2 Recommended Alternatives and CIP Costs

The recommended collection system alternatives are discussed in Section 8. The recommended treatment plant alternatives are discussed in Section 9. Table 12.2 lists the recommended improvements over the 20-year planning period and their associated costs. The timing of the improvements is roughly based on a 5-year increment between phases but the actual implementation date for any of the improvements is primarily dependent upon need.

Table 12.2 – Capital Improvement Plan

Item	Description	Phase 1	Phase 2	Phase 3	Total
	Collection System				
I1	Smoke Testing and Video Program	$30,000	$30,000	$30,000	$120,000
I2	cMOM Program		$30,000	$30,000	$60,000
I3	1st Street Sewer Overflows, Manhole at 1st St. and Stillwell Avenue, NaOCL Dosing Station	$195,000			$195,000
I5	Upgrade STEP systems in Sub Basin 5	$130,000	$130,000	$130,000	$390,000
I6	Construct a new Southside Interceptor		$790,000	$790,000	$1,580,000
I7	Construct a new 12th Street Interceptor			$1,000,000	$1,000,000
I8	Construct Interceptor to Fairview Area		$1,660,000		$1,660,000
I9	Upsize Evergreen Drive Interceptor	$195,000			$195,000
I10	Upgrade 12th Street Pump Station and Forcemain	$75,000	$165,000	$165,000	$405,000
	Wastewater Treatment Plant				
IPS2	Submersible Influent Pump Station	$1,341,000			$1,341,000
HW1	Headworks Improvements	$231,000			$231,000
T7	Secondary Treatment w/ Nitrifying Selector Activated Sludge	$3,045,000			$3,045,000
D1	Chlorine Disinfection Improvements	$1,000,000	$195,000	$195,000	$1,390,000
E1	Outfall Improvements	$55,000			$55,000
S4	Anaerobic Digestion with RAS Thickening and Converted Sludge Storage	$2,963,000			$2,963,000
SP1	Site Work/Yard Piping	$250,000			$250,000
SP2	Site Electrical Upgrades	$100,000			$100,000
SW1	Standby Power	$315,000			$315,000
LA1	Purchase of North Site and Related Costs	$245,000			245,000
LA2	Biosolids Land Purchase		$600,000		600,000
	Total Estimated Project Costs	**$10,170,000**	**$3,600,000**	**$2340,000**	**$16,110,000**

12.3 Impact to User Rates

The impact of the $16.1M project to future user rates depends primarily on the growth rate of the customer base, the interest rates applied to loans, and the availability of grants.

The annual customer base growth rate (approximately1%) used for the Facility Plan is the same annual growth rate adopted by the City's planning department. The Facility Plan growth projection assumes that 500 existing homes in the UGA (currently using onsite septic systems) are incorporated into the service area within the first few years of the planning period. In reality, the actual customer growth rate may be less and the existing homes within the UGA may not be required to connect to the sewer system. For capacity planning, the annual growth rate does not make a significant difference as the overriding criteria driving the size of the treatment and conveyance facilities are peak flows experienced during the wet weather months. However, for financial planning purposes, it would be prudent to plan for lower customer base growth rates and to assume that the 500 existing homes are not incorporated into the service area.

When a 0.5% annual growth rate is used to make the population projection, and the 500 homes in the UGA are not included, an initial residential user rate of approximately $48 per month is required to fund the City's CIP and pay the anticipated annual O&M expenses. Based on the same assumptions, it is anticipated that a $4 to $6 per month residential rate increase will be required every 5-years to continue to fund the CIP and provide for O&M expenses. A more detailed user rate and system development charge (SDC) study is recommended to refine financial information.

Section 13: Public Involvement and Process Selection

13.1 Public Involvement

Public involvement in the Facility Planning effort involved City staff, City Council members, citizens, outside professionals, and local media outlets.

A Technical Advisory Committee (TAC) made up of City employees and the Public Advisory committee (PAC) consisting of council members, the Mayor, and other citizens helped lead the decision making process regarding process selection. The DEQ's representative and the City's engineer were an integral part of the process as well.

Several monthly City Council meetings during this period were attended by the City's Engineer. Presentations were made to Council during these meetings to inform the City's decision makers of the direction of the Facility Planning process and to discuss the recommendations made in the Facility Plan. The general public and local media are always invited to these meetings.

Two evening workshops were held at City Hall to exclusively discuss the challenges faced by the City regarding upgrades of the collection system and the treatment plant and the anticipated rate increases required to finance the improvements. The City's Public Work's Director and the City's engineer presented material to the City Council members, the public, and members of the media at these workshops.

An Open House was held on 28 October 2005 at Tillamook High School to discuss the planned upgrades for the WWTP Facilities and the anticipated sewer rate increases. The City sent a Fact Sheet entitled "Upgrading the Tillamook Wastewater Facilities" to all water/sewer customers along with an invitation to the Workshop. The City also provided a more detailed version of the Fact Sheet at the Workshop. Both the Mailer and the Fact Sheet are included at the end of this section.

A sewer rate increase public notice meeting was held at City Hall on 16 May 2005. The City announced the public hearing on the sewer rate increase by including notice of the meeting in the monthly water/sewer bills mailed on 1 May 2005. A copy of May's sewer bill is included at the end of this section.

The local newspaper, The Headlight Herald, has published articles on the process selection discussions, DEQ's involvement, and the anticipated affect of the improvements on sewer rates.

13.2 WWTP Process Selection

The selection process, initiated in April 2003, culminated in the decision to recommend the NSAS based treatment system. The TAC, along with the City's Engineer led the effort. The TAC was selected by the Public Works Director and included the following City employees:

- Public Works Director, Arley Sullivan
- WWTP Superintendent, Donita Parks

- Water System Supervisor, Tim Lyda
- Street and Stormwater System Field Supervisor, Jon Williams.

The TAC originally met on 30 April and 12 June 2003 to discuss and evaluate the treatment alternatives under consideration. On 23 and 24 June 2003, the TAC accompanied the Engineer on a 2-day field trip to various treatment plants to view process equipment and interview operations staff.

Four alternatives were considered at that time, these alternatives were:

T1: Parallel Selector Activated Sludge/RBC (SAS/RBC)

T2: Series Biotower/RBCs (BT/RBC)

T3: Oxidation Ditch No Primary Clarifiers (OXDnoPC)

T4: Additional RBCs (RBC/RBC)

Based on the information presented and the field trip experience, the two alternatives viewed as the best alternatives were T2: Series Biotower/RBCs and T3: Oxidation Ditch No Primary Clarification. These preferences were incorporated into the Draft Facility Plan. Based on recommendations made following DEQ review of the Draft Facility Plan, the TAC met again on 27 May 2004 to consider three additional alternatives as follows:

T5: Oxidation Ditch with Primary Clarification (OXDwPC)

T6: Sequencing Batch Reactors (SBRs)

T7: Nitrifying Selector Activated Sludge (NSAS)

The NSAS alternative was presented to the public in an Open House in Tillamook on 28 October 2004. Based on the advantages offered by this alternative (Alternative T7), especially with regard to the reduced basin footprint, excellent effluent quality, and flexibility regarding operation, it was selected as the best option for providing secondary treatment at the upgraded treatment plant.

Working closely with the TAC in the selection process was the Public Advisory Committee meeting (PAC) that consisted of eight city residents including various council members and planning committee members. PAC meetings were also attended by City Manager, Mark Gervasi and City Planner, David Mattison. The PAC was invited on a field trip of treatment plants which occurred on 17 September 2003.

In summary, the Facility Planning and Selection Process was an inter-active program that involved nearly two years of research and input from the City's Public Works staff, members of the Public Advisory Committee, the Oregon DEQ, and the City's engineering representative.

Eastgate Lift Station

Period Days	date	Pump 2 Hours	Pump 1 Hours	Total Pump Hours	Total Period Hours	% Run Time	Approx. Month of Occurrence	Monthly Time (%)
7	12/6-12/13	15.4	0	15.4	168	9%	December	10%
6	12/13-12/20	20.5	0	20.5	144	14%		
7	12/20-12/27	12.3	0	12.3	168	7%		
1	12/27-12/28	1.8	0	1.8	24	8%		
2	12/28-12/30	1.5	1.2	2.7	48	6%		
4	12/30/99 -1/3/0(	3	2.3	5.3	96	6%	January	7%
3	1/3/00-1/6	3.3	2.5	5.8	72	8%		
4	1/6-1/10	4.3	3.2	7.5	96	8%		
2	1/10-1/12	2.1	1.5	3.6	48	8%		
12	1/12-1/24	10.7	8.2	18.9	288	7%		
3	1/24-1/27	2.6	2	4.6	72	6%		
4	1/27-1/31	3.3	2.6	5.9	96	6%		
7	1/31/00-2/7/00	6.3	4.8	11.1	168	7%	February	7%
4	2/7-2/11	3.4	2.7	6.1	96	6%		
3	2/11-2/14	2.5	1.9	4.4	72	6%		
4	2/14-2/18	3.5	2.8	6.3	96	7%		
4	2/18-2/22	3.1	2.5	5.6	96	6%		
3	2/22-2/25	2.4	2.2	4.6	72	6%		
3	2/25-2/28	3.2	2.4	5.6	72	8%		
3	2/28-3/3/00	3.2	2.5	5.7	72	8%	March	6%
3	3/3/- 3/6	2.8	2.2	5	72	7%		
4	3/6/00-3/10/00	3	2.3	5.3	96	6%		
3	3/10-3/13	2.7	2.1	4.8	72	7%		
4	3/13-3/17	3.1	2.4	5.5	96	6%		
3	3/17-3/20	2.6	2	4.6	72	6%		
4	3/20-3/24	3.1	2.3	5.4	96	6%		
3	3/24-3/27	2.6	1.9	4.5	72	6%		
4	3/27-3/31	2.9	2.3	5.2	96	5%		
3	3/31-4/3/00	2.2	1.6	3.8	72	5%	April	5%
4	4/3-4/7	2.9	2.2	5.1	96	5%		
3	4/7-4/10	2.1	1.6	3.7	72	5%		
7	4/10-4/17	5.1	4.1	9.2	168	5%		
4	4/17-4/21	3	2.3	5.3	96	6%		
3	4/21-4/24	2.6	2	4.6	72	6%		
4	4/24-4/28	2.8	2.1	4.9	96	5%		
7	4/28-5/5/00	2.8	2.3	5.1	168	3%	May	5%
3	5/5-5/8	2.4	1.8	4.2	72	6%		
4	5.8-5/12	3.5	2.7	6.2	96	6%		
3	5/12-5/15	2.9	2.2	5.1	72	7%		
4	5/15-5/19	3.2	2.5	5.7	96	6%		
3	5/19-5/22	2.4	1.9	4.3	72	6%		
3	5/22-5/25	2.1	1.9	4	72	6%		
5	5/25-5/30	4.1	3	7.1	120	6%		
2	5/30-6/1/00	1.5	1.2	2.7	48	6%		
4	6/1-6/5	3.1	2.5	5.6	96	6%	June	6%
4	6/5-6/9	2.8	2.3	5.1	96	5%		
3	6/9-6/12	3	2.3	5.3	72	7%		
4	6/12-6/16	3	2.4	5.4	96	6%		
3	6/16-6/19	2.5	2.5	5	72	7%		
4	6/19-6/23	5.3	4.3	9.6	96	10%		
3	6/23-6/26	2.7	2.2	4.9	72	7%		

Eastgate Lift Station

Period Days	date	Pump 2 Hours	Pump 1 Hours	Total Pump Hours	Total Period Hours	% Run Time	Approx. Month of Occurrence	Monthly Time (%)
10	6/26/7/6/00	6.9	5.6	12.5	240	5%		
4	7/6-7/10	2.5	2	4.5	96	5%	July	5%
4	7/10-7/14	2	1.6	3.6	96	4%		
3	7/14-7/17	2.3	1.8	4.1	72	6%		
7	7/17-7/24	4.3	3.4	7.7	168	5%		
3	7/24-7/27	2.1	1.6	3.7	72	5%		
4	7/27-7/31	2.4	1.9	4.3	96	4%		
4	7/31-8/4/00	2.5	1.9	4.4	96	5%	August	5%
3	8/4-8/7	1.8	1.4	3.2	72	4%		
3	8/7-8/10	1.7	1.3	3	72	4%		
7	8/10-8/17	4.1	4.6	8.7	168	5%		
4	8/17-8/21	2.3	1.9	4.2	96	4%		
4	8/21-8/25	2.3	1.9	4.2	96	4%		
3	8/25-8/28	1.9	1.7	3.6	72	5%		
3	8/28-8/31	0	2.3	2.3	72	3%		
5	8/31/-9/5/00	0	5.4	5.4	120	5%	September	5%
3	9/5-9/8	0	2.4	2.4	72	3%		
3	9/8-9/11	0	4.4	4.4	72	6%		
7	9/11-9/18	0	7	7	168	4%		
4	9/18-9/22	0	4.4	4.4	96	5%		
3	9/22-9/25	0	3.9	3.9	72	5%		
7	9/25-10/2/00	0	8.2	8.2	168	5%		
4	10/2-10/6	0	4.6	4.6	96	5%	October	5%
3	10/6-10/9	0	3.6	3.6	72	5%		
4	10/9-10/13	0	5.2	5.2	96	5%		
3	10/13-10/16	0	4.3	4.3	72	6%		
4	10/16-10/20	0	5.1	5.1	96	5%		
3	10/20-10/23	0	4.2	4.2	72	6%		
4	10/23-10/27	0	5.2	5.2	96	5%		
3	10/27-10/30	0	4.7	4.7	72	7%		
7	10/30-11/6/00	3.9	3	6.9	168	4%	November	5%
3	11/6-11/9	2.5	1.8	4.3	72	6%		
4	11/9-11/13	2.4	2.9	5.3	96	6%		
3	11/13-11/16	2.2	1/1/1900	3.9	72	5%		
4	11/16-11/20	2.8	2	4.8	96	5%		
7	11/20-11/27	2.9	5.9	8.8	168	5%		
4	11/27-12/1/00	0.1	3.8	3.9	96	4%		
3	12/1-12/4	2	1.8	3.8	72	5%	December	5%
4	12/4-12/8	2.2	2	4.2	96	4%		
3	12/8-12/11	1.7	1.8	3.5	72	5%		
3	12/11-12/14	1.8	1.4	3.2	72	4%		
4	12/14-12/18	2.8	2.2	5	96	5%		
3	12/18-12/21	2.2	2	4.2	72	6%		
5	12/21-12/26	4	3.4	7.4	120	6%		
2	12/26-12/28	1.3	1.2	2.5	48	5%		
5	12/28-1/2/01	4.6	3.3	7.9	120	7%		
2	1/2-1/4	1.4	0.9	2.3	48	5%	January	6%
4	1/4-1/8	3.9	2.9	6.8	96	7%		
4	1/8-1/12	3.6	2.5	6.1	96	6%		
4	1/12-1/16	3.4	2.5	5.9	96	6%		
3	1/16-1/19	2.5	1.7	4.2	72	6%		

Eastgate Lift Station

Period Days	date	Pump 2 Hours	Pump 1 Hours	Total Pump Hours	Total Period Hours	% Run Time	Approx. Month of Occurrence	Monthly Time (%)
3	1/19-1/22	2.4	1.8	4.2	72	6%		
3	1/22-1/25	2.4	1.8	4.2	72	6%		
4	1/25-1/29	3	2.2	5.2	96	5%		
7	1/29-2/5/2001	3.5	2.6	6.1	168	4%	February	5%
4	2/5-2/9	3	2.3	5.3	96	6%		
3	2/9-2/12	2.6	1.9	4.5	72	6%		
4	2/12-2/16	2.8	2.1	4.9	96	5%		
7	2/16-2/23	5	3.8	8.8	168	5%		
3	2/23-2/26	2.8	2.1	4.9	72	7%		
4	2/26-3/2/01	2.8	2.1	4.9	96	5%		
3	3/2-3/5	2.3	1.7	4	72	6%	March	5%
4	3/5-3/9	2.7	2.1	4.8	96	5%		
3	3/9-3/12	2.1	1.6	3.7	72	5%		
4	3/12-3/16	2.4	1.8	4.2	96	4%		
3	3/16-3/19	2.4	1.9	4.3	72	6%		
4	3/19-3/23	3.1	2.4	5.5	96	6%		
10	3/23-4/2/01	7.6	5.7	13.3	240	6%		
5	4/2-4/7	3.3	2.5	5.8	120	5%	April	5%
6	4/7-4/13	5.9	4.5	10.4	144	7%		
6	4/13-4/16	2.6	1.9	4.5	144	3%		
7	4/16-4/23	5.3	4	9.3	168	6%		
3	4/23-4/26	2	1.5	3.5	72	5%		
4	4/26-4/30	3.2	2.4	5.6	96	6%		
4	4/30-5/4/01	3.1	2.3	5.4	96	6%		
3	5/4-5/7	2.3	1.7	4	72	6%	May	5%
3	5/7-5/10	2.1	1.5	3.6	72	5%		
4	5/10-5/14	2.9	2.2	5.1	96	5%		
2	5/14-5/16	1.6	1.1	2.7	48	6%		
5	5/16-5/21	3.9	2.9	6.8	120	6%		
3	5/21-5/24	2.2	1.6	3.8	72	5%		
5	5/24-5/29	3.6	2.7	6.3	120	5%		
3	5/29-6/1/01	1.9	1.3	3.2	72	4%		
3	6/1-6/4	2.3	1.6	3.9	72	5%	June	5%
3	6/4-6/7	2.1	1.7	3.8	72	5%		
4	6/7-6/11	2.8	2.1	4.9	96	5%		
3	6/11-6/14	1.9	1.4	3.3	72	5%		
4	6/14-6/18	2.6	2	4.6	96	5%		
3	6/18-6/21	2	1.5	3.5	72	5%		
4	6/21-6/25	2.4	1.8	4.2	96	4%		
4	6/25-6/29	2.7	2	4.7	96	5%		
7	6/29/7/6/01	4.5	3.5	8	168	5%	July	5%
3	7/6-7/9	1.8	1.3	3.1	72	4%		
4	7/9-7/13	2.4	1.8	4.2	96	4%		
3	7/13-7/16	2	1.6	3.6	72	5%		
4	7/16-7/20	2.6	1.6	4.2	96	4%		
3	7/20-7/23	2.1	1.6	3.7	72	5%		
4	7/23-7/27	2.5	2	4.5	96	5%		
7	7/27-8/3/01	4.7	3.6	8.3	168	5%		
3	8/3-8/6	2	1.5	3.5	72	5%	August	5%
4	8/6-8/10	2.2	1.8	4	96	4%		
5	8-8/10-8/13	2.1	2.5	4.6	120	4%		

Eastgate Lift Station

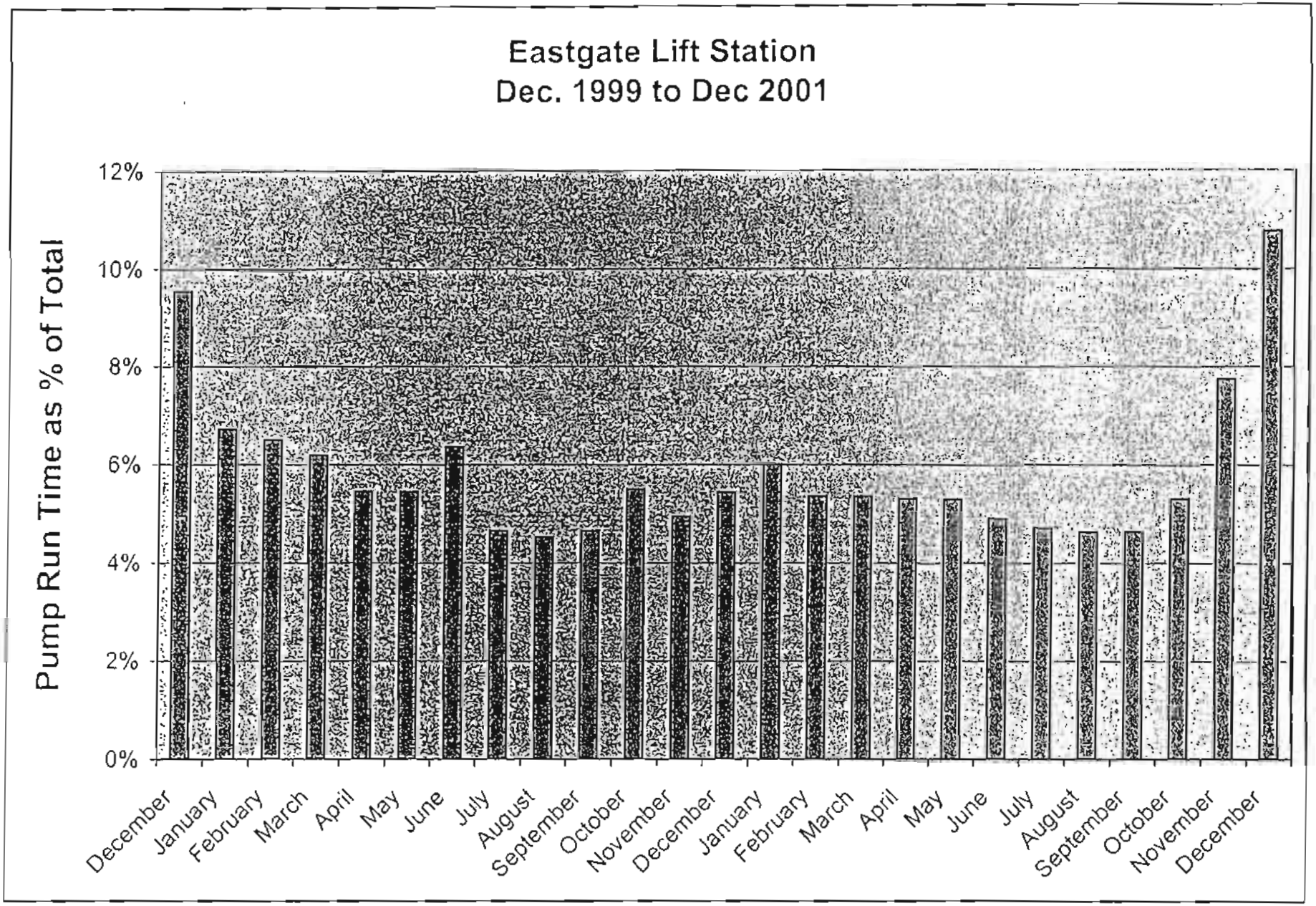
Period Days	date	Pump 2 Hours	Pump 1 Hours	Total Pump Hours	Total Period Hours	% Run Time	Approx. Month of Occurrence	Monthly Time (%)
4	8/13-8/17	2.3	1.8	4.1	96	4%		
3	8/17-8/20	2.1	1.5	3.6	72	5%		
4	8/20-8/24	2.6	2.1	4.7	96	5%		
3	8/24-8/27	1.9	2.4	4.3	72	6%		
4	8/27-8/31	2.6	1.9	4.5	96	5%		
4	8/31-9/4/01	2.6	2.1	4.7	96	5%	September	5%
3	9/4-9/7	1.7	1.3	3	72	4%		
3	9/7-9/10	1.9	1.4	3.3	72	5%		
3	9/10-9/13	1.6	1.3	2.9	72	4%		
5	9/13-9/18	3.3	2.3	5.6	120	5%		
3	9/18-9/21	2	1.5	3.5	72	5%		
7	9/21-9/28	4.6	3.5	8.1	168	5%		
6	9/28-10/4/01	3.8	3.9	7.7	144	5%	October	5%
6	10/4-10/10	4.6	3.5	8.1	144	6%		
5	10/10-10/15	3.6	2	5.6	120	5%		
4	10/15-10/19	2.5	2	4.5	96	5%		
7	10/19-10/26	4.9	3.7	8.6	168	5%		
7	10/26-11/2/01	5.3	4.7	10	168	6%		
3	11/2-11/5	2.4	1.4	3.8	72	5%	November	8%
4	11/5-11/9	2.9	2.4	5.3	96	6%		
4	11/9-11/13	4.2	3.3	7.5	96	8%		
3	11/13-11/16	2.3	1.7	4	72	6%		
3	11/16-11/19	2.7	2.1	4.8	72	7%		
7	11/19-11/26	7.1	6.1	13.2	168	8%		
4	11/26-11/30	4.3	4.1	8.4	96	9%		
4	11/30-12/4/01	6.6	5.9	12.5	96	13%		
5	12/6-12/11	2.3	1.9	4.2	120	4%	December	11%
6	12/11-12/17	9.1	7.5	16.6	144	12%		
7	12/17-12/24	7.3	6	13.3	168	8%		
2	12/24-12/26	9.3	8.3	17.6	48	37%		
9	12/26-1/4/02	2.4	2	4.4	216	2%	January	8%
3	1/4-1/7	4	3.1	7.1	72	10%		
7	1/7-1/14	9.1	7.1	16.2	168	10%		
4	1/14-1/18	3.7	3.4	7.1	96	7%		
4	1/18-1/22	4.5	3.6	8.1	96	8%		
3	1/22-1/25	5	3.9	8.9	72	12%		
3	1/25-1/28	4.1	3.1	7.2	72	10%		
4	1/28-2/1/02	4.5	3.5	8	96	8%		
6	2/1-2/7	6.3	5	11.3	144	8%	February	7%
4	2/7-2/11	4.5	3.6	8.1	96	8%		
8	2/11-2/19	7.2	5.6	12.8	192	7%		
3	2/19-2/22	2.5	2	4.5	72	6%		
3	2/22-2/25	2.7	2.3	5	72	7%		
3	2/25-2/28	2.5	2.1	4.6	72	6%		
4	2/28-3/4/02	3	2.6	5.6	96	6%	March	7%
4	3/4-3/8	2.9	2.6	5.5	96	6%		
3	3/8-3/11	1.8	2.5	4.3	72	6%		
3	3/11-3/14	4.2	2.6	6.8	72	9%		
4	3/14-3/18	4	3.2	7.2	96	8%		
4	3/18-3/22	3.3	2.7	6	96	6%		
3	3/22-3/25	3.4	2.8	6.2	72	9%		

Eastgate Lift Station

Period Days	date	Pump 2 Hours	Pump 1 Hours	Total Pump Hours	Total Period Hours	% Run Time	Approx. Month of Occurrence	Monthly Time (%)
11	3/25-4/5/02	8.4	6.4	14.8	264	6%		
4	4/5-4/9	3	3.1	6.1	96	6%	April	6%
9	4/9-4/18	7.6	6.6	14.2	216	7%		
5	4/18-4/23	4.2	3.4	7.6	120	6%		
6	4/23-4/29	4.6	3.7	8.3	144	6%		
3	4/29-5/2/02	2.2	1.8	4	72	6%		
4	5/2-5/6	2.8	2.4	5.2	96	5%	May	7%
3	5/6-5/9	2.1	1.7	3.8	72	5%		
4	5/9-5/13	2.9	2.4	5.3	96	6%		
3	5/13-5/16	2	1.8	3.8	72	5%		
4	5/16-5/20	8.7	4.7	13.4	96	14%		
4	5/20-5/24	2.6	2.2	4.8	96	5%		
6	5/24-5/30	4.4	4	8.4	144	6%		
11	5/30-6/10/02	7.2	6.8	14	264	5%	June	5%
3	6/10-6/13	2.2	1.7	3.9	72	5%		
8	6/13-6/21	5.3	4.6	9.9	192	5%		
6	6/21-6/27	3.6	3.4	7	144	5%		
5	6/27-7/2/02	3.6	3.1	6.7	120	6%		
13	7/2-7/15	8.3	7.3	15.6	312	5%	July	5%
3	7/15-7/18	1.8	1.8	3.6	72	5%		
4	7/18-7/22	2.4	2.4	4.8	96	5%		
7	7/22-7/29	4.3	3.7	8	168	5%		
7	7/29-8/5/02	4.2	3.8	8	168	5%	August	5%
7	8/5-8/12	4.3	3.7	8	168	5%		
4	8/12-8/16	2.4	2	4.4	96	5%		
3	8/16-8/19	2	1.8	3.8	72	5%		
4	8/19-8/23	2.5	2.2	4.7	96	5%		
6	8/23-8/29	3.8	3.3	7.1	144	5%		
8	8/29-9/6/02	4.9	4.7	9.6	192	5%	September	5%
7	9/6-9/13	4.7	4.3	9	168	5%		
4	9/13-9/17	2.5	2.2	4.7	96	5%		
3	9/17-9/20	1.8	1.4	3.2	72	4%		
3	9/20-9/23	2	1.7	3.7	72	5%		
4	9/23-9/27	2.5	2	4.5	96	5%		
4	9/27-10/1/02	2.6	3.8	6.4	96	7%		
3	10/1-10/4	1.9	1.6	3.5	72	5%	October	5%
7	10/4-10/11	1.9	1.6	3.5	168	2%		
3	10/11-10/14	4.8	3.8	8.6	72	12%		
4	10/14-10/18	2	1.6	3.6	96	4%		
3	10/18-10/21	2.3	1.8	4.1	72	6%		
3	10/21-10/24	1.7	1.8	3.5	72	5%		
4	10/24-10/28	1.8	1.5	3.3	96	3%		
4	10/28-11/1/02	2.6	2.1	4.7	96	5%		
3	11/1-11/4	2	1.7	3.7	72	5%	November	5%
4	11/4-11/8	2.2	1.7	3.9	96	4%		
3	11/8-11/11	2.1	1.7	3.8	72	5%		
3	11/11-11/14	3.1	2.4	5.5	72	8%		
4	11/14-11/18	1.3	1.1	2.4	96	3%		
4	11/18-11/22	3.2	2.5	5.7	96	6%		
5	11/22-11/27	2.8	2.2	5	120	4%		

Eastgate Lift Station

Period Days	date	Pump 2 Hours	Pump 1 Hours	Total Pump Hours	Total Period Hours	% Run Time	Approx. Month of Occurrence	Monthly Time (%)
8	11/27-12/5/02	3.5	2.8	6.3	192	3%	December	6%
4	12/5-12/9	5.2	4.4	9.6	96	10%		
3	12/9-12/12	2.6	2.1	4.7	72	7%		
4	12/12-12/16	2.2	1.8	4	96	4%		
4	12/16-12/20	5	4.3	9.3	96	10%		
3	12/20-12/23	4.3	3.4	7.7	72	11%		
4	12/23-12/27	2.3	2	4.3	96	4%		
3	12/27-12/30	3.2	2	5.2	72	7%		


Eastgate Lift Station
Dec. 1999 to Dec 2001
Pump Run Time as % of Total
12%
10%
8%
6%
4%
2%
0%
December
January
February
March
April
May
June
July
August
September
October
November
December
January
February
March
April
May
June
July
August
September
October
November
December

Front Street Lift Station

Front Street Lift Station

Period Days	date	Pump 1 Hours	Pump 2 Hours	Total Pump Hours	Total Period Hours	% Run Time		
10	12/3-12/13/1999	12.5	11.9	24.4	240	10%	December	12%
7	12/13-12/20	17.8	16.5	34.3	168	20%		
7	12/20-12/27	7.4	6.9	14.3	168	9%		
3	12/27-12/30	2.5	2.5	5	72	7%		
4	12/30-1/3/00	2.4	2.1	4.5	96	5%	January	7%
3	1/3-1/6	2.4	2.4	4.8	72	7%		
4	1/6-1/10	3.7	3.6	7.3	96	8%		
14	1/10-1/24	13.3	12.4	25.7	336	8%		
3	1/24-1/27	2.6	2.5	5.1	72	7%		
4	1/27-1/31	3.2	3.1	6.3	96	7%		
7	1/31-2/7/00	5.6	5.3	10.9	168	6%	February	6%
4	2/7-2/11	3.5	3.2	6.7	96	7%		
3	2/11-2/14	2.1	2	4.1	72	6%		
4	2/14-2/18	3	3.1	6.1	96	6%		
4	2/18-2/22	2.3	2.1	4.4	96	5%		
3	2/22-2/25	1.9	1.9	3.8	72	5%		
3	2/25-2/28	1.8	1.7	3.5	72	5%		
3	2/28-3/3/00	2.8	2.6	5.4	72	8%	March	12%
3	3/3-3/6	2.4	2.4	4.8	72	7%		
4	3/6-3/10	3.6	2.6	6.2	96	6%		
3	3/10-3/13	1.6	2.3	3.9	72	5%		
4	3/13-3/17	2.5	2.4	4.9	96	5%		
3	3/17-3/20	1.8	1.8	3.6	72	5%		
4	3/20-3/24	2.7	2.5	5.2	96	5%		
3	3/24-3/27	12.2	16	28.2	72	39%		
4	3/27-3/31	8.7	17	25.7	96	27%		
3	3/31-4/3/00	1.5	1.2	2.7	72	4%	April	34%
3	4/3-4/7	1.7	1.4	3.1	72	4%		
3	4/7-4/10	1.3	1.2	2.5	72	3%		
7	4/10-4/17	33	3	36	168	21%		
4	4/17-4/21	51.6	2.8	54.4	96	57%		
3	4/21-4/24	45.6	46.8	92.4	72	128%		
1	4/24-4/25	28.6	1.2	29.8	24	124%		
3	4/25-4/28	1.4	1.2	2.6	72	4%		
1	4/28-5/1/00	1.7	1.5	3.2	24	13%		
4	5/1-5/5	2.4	2.2	4.6	96	5%	May	4%
3	5/5-5/8	1.5	1.4	2.9	72	4%		
4	5/8-5/12	1.4	1.4	2.8	96	3%		
3	5/12-5/15	1.3	1.1	2.4	72	3%		
4	5/15-5/19	1.6	1.6	3.2	96	3%		
3	5/19-5/22	1.5	1.4	2.9	72	4%		
6	5/22-5/30	3.2	3	6.2	144	4%		
2	5/30-6/1/00	0.7	0.7	1.4	48	3%		
4	6/1-6/5	1.2	1.1	2.3	96	2%	June	35%
4	6/5-6/10	1.2	1.2	2.4	96	3%		
2	6/10-6/12	1.2	1.3	2.5	48	5%		
3	6/12-6/15	1.8	1.4	3.2	72	4%		
4	6/15-6/19	2.6	2.5	5.1	96	5%		
3	6/19-6/22	2.1	1.9	4	72	6%		
4	6/22-6/26	2.4	2.3	4.7	96	5%		
4	6/26-6/30	199.8	8.5	208.3	96	217%		
10	6/30-7/10	1.4	1.4	2.8	240	1%	July	4%
3	7/10-7/13	0.5	1.3	1.8	72	3%		
4	7/13-7/17	2.8	2.1	4.9	96	5%		

Front Street Lift Station

Front Street Lift Station

Period Days	date	Pump 1 Hours	Pump 2 Hours	Total Pump Hours	Total Period Hours	% Run Time		
7	7/17-7/24	8.1	2.5	10.6	168	6%		
3	7/24-7/27	1.9	1.8	3.7	72	5%		
4	7/27-7/31	2.3	2.2	4.5	96	5%		
4	7/31-8/4/00	4.3	4.1	8.4	96	9%	August	5%
3	8/4-8/7	1.5	1.65	3.15	72	4%		
3	8/7-8/10	1.7	1.6	3.3	72	5%		
6	8/10-8/16	3.7	3.4	7.1	144	5%		
5	8/16-8/21	2.3	2.3	4.6	120	4%		
4	8/21-8/25	2.6	2.4	5	96	5%		
3	8/25-8/28	1.7	1.5	3.2	72	4%		
3	8/28-8/31	1.1	1.1	2.2	72	3%		
5	8/31-9/5/00	1.9	1.7	3.6	120	3%	September	3%
2	9/5-9/7	0.8	0.7	1.5	48	3%		
4	9/7-9/11	1.5	1.4	2.9	96	3%		
8	9/11-9/18	2.2	2.3	4.5	192	2%		
4	9/18-9/22	1.6	1.4	3	96	3%		
3	9/22-9/25	1.5	1.4	2.9	72	4%		
4	9/25-9/29	2.6	2.8	5.4	96	6%		
7	9/29-10/6/00	1.6	1.3	2.9	168	2%	October	4%
3	10/6-10/9	0.7	0.8	1.5	72	2%		
4	10/9-10/13	1.6	1.4	3	96	3%		
3	10/13-10/16	0.9	0.9	1.8	72	3%		
4	10/16-10/20	1.6	1.3	2.9	96	3%		
3	10/20-10/23	1.1	1	2.1	72	3%		
4	10/23-10/27	1.6	1.4	3	96	3%		
3	10/27-10/30	9	6	15	72	21%		
7	10/30-11/6	2.7	2.8	5.5	168	3%	November	3%
3	11/6-11/9	1.7	1.3	3	72	4%		
4	11/9-11/13	1.6	1.8	3.4	96	4%		
3	11/13-11/16	0.5	0.5	1	72	1%		
4	11/16-11/20	0.9	0.7	1.6	96	2%		
7	11/20-11/27	2.2	1.9	4.1	168	2%		
4	11/27-12/1	1.3	1.4	2.7	96	3%		
3	12/1-12/4	1.7	2	3.7	72	5%	December	2%
4	12/4-12/8	1.5	1.2	2.7	96	3%		
3	12/8-12/11	1.5	0.3	1.8	72	3%		
3	12/11-12/14	0.2	0.2	0.4	72	1%		
4	12/14-12/18	0.7	0.3	1	96	1%		
3	12/18-12/21	0.2	0.2	0.4	72	1%		
5	12/21-12/26	0.9	0.9	1.8	120	2%		
2	12/26-12/28	0.3	0.3	0.6	48	1%		
2	12/28-1/2/01	0.9	0.9	1.8	48	4%		
2	1/2-1/4	1.1	1.2	2.3	48	5%	January	3%
4	1/4-1/8	1.6	1.8	3.4	96	4%		
4	1/8-1/12	1.7	2	3.7	96	4%		
4	1/12-1/16	1.1	1.2	2.3	96	2%		
3	1/16-1/19	0.9	1.2	2.1	72	3%		
3	19-1/22	0.8	0.8	1.6	72	2%		
3	1/22-1/25	0.9	1.1	2	72	3%		
4	1/25-1/29	0.8	0.9	1.7	96	2%		
3	1/29-2/1/01	0.9	1.1	2	72	3%		
4	2/1-2/5	1.2	1.3	2.5	96	3%	February	4%
4	2/5-2/9	1.6	1.9	3.5	96	4%		
3	2/9-2/12	1	1.2	2.2	72	3%		

Front Street Lift Station

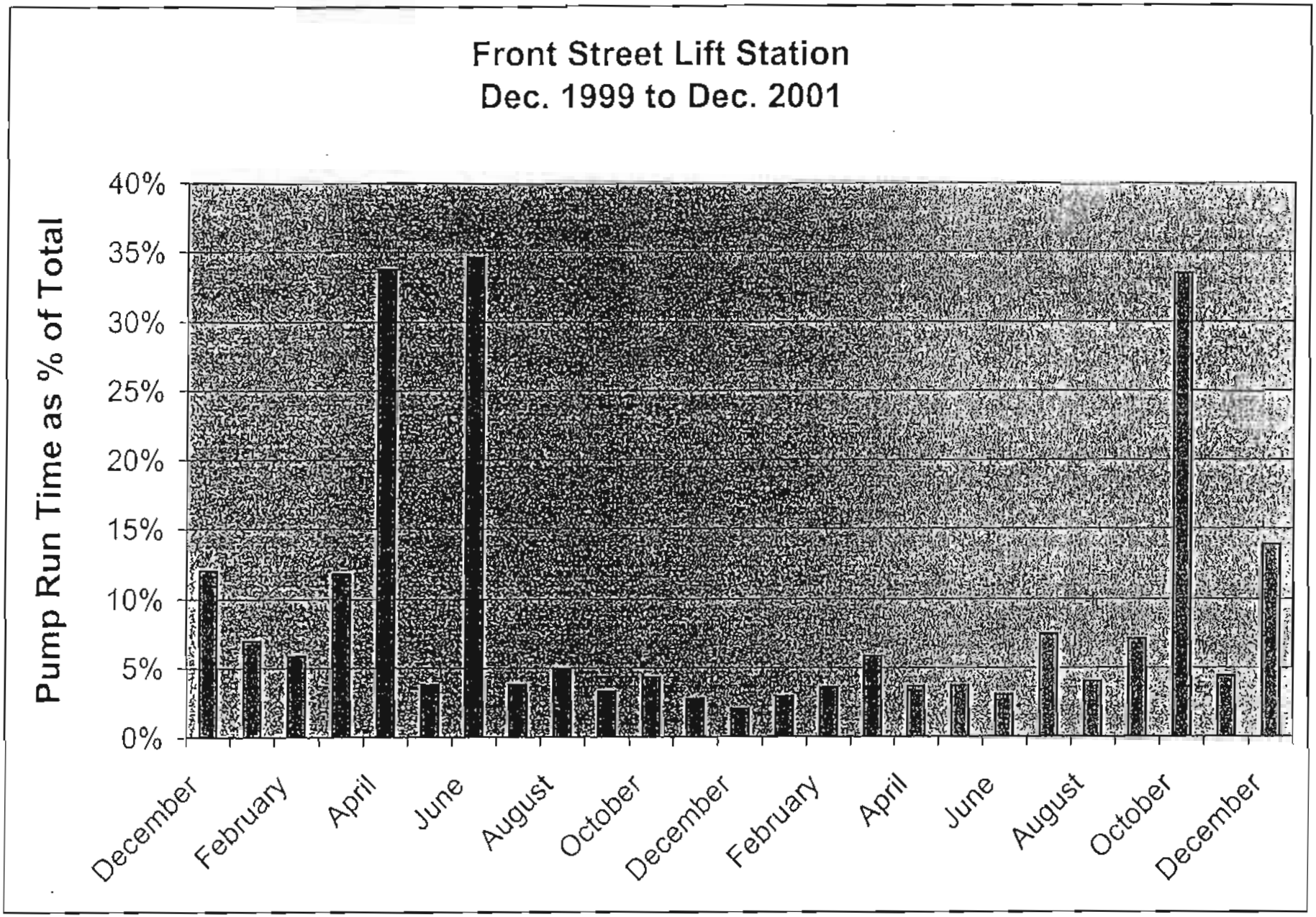
Front Street Lift Station

Period Days	date	Pump 1 Hours	Pump 2 Hours	Total Pump Hours	Total Period Hours	% Run Time		
4	2/12-2/16	1.6	1.7	3.3	96	3%		
6	2/16-2/22	2.9	2.8	5.7	144	4%		
8	2/22-3/2/01	2.1	5.5	7.6	192	4%		
3	3/2-3/5	9.4	14.4	23.8	72	33%	March	6%
4	3/5-3/9	0.9	1.3	2.2	96	2%		
7	3/9-3/16	2.3	0.1	2.4	168	1%		
3	3/16-3/19	0.6	3.5	4.1	72	6%		
4	3/19-3/23	1.7	1.9	3.6	96	4%		
10	3/23-4/2/01	3.4	4.1	7.5	240	3%		
4	4/2-4/6	1.4	1.6	3	96	3%	April	4%
7	4/6-4/13	2.9	3.7	6.6	168	4%		
3	4/13-4/16	1	1.2	2.2	72	3%		
7	4/16-4/23	2.9	3.4	6.3	168	4%		
3	4/23-4/26	1.5	1.8	3.3	72	5%		
4	4/26-4/30	1.5	1.6	3.1	96	3%		
4	4/30-5/4/01	1.3	1.8	3.1	96	3%	May	4%
3	5/4-5/7	1.2	1.3	2.5	72	3%		
3	5/7-5/10	1.4	1.9	3.3	72	5%		
4	5/10-5/14	1.1	1.3	2.4	96	3%		
2	5/14-5/16	0.9	1	1.9	48	4%		
5	5/16-5/21	2	2.6	4.6	120	4%		
3	5/21-5/24	1.3	1.4	2.7	72	4%		
5	5/24-5/29	2.4	3	5.4	120	5%		
2	5/29-6/1/01	1.3	0.3	1.6	48	3%		
3	6/1-6/4	1.4	0	1.4	72	2%	June	3%
3	6/4-6/7	1.6	0.2	1.8	72	3%		
4	6/7-6/11	2	2.6	4.6	96	5%		
7	6/11-6/18	1.9	2.3	4.2	168	3%		
3	6/18-6/21	1	1.1	2.1	72	3%		
4	6/21-6/25	1.4	1.6	3	96	3%		
4	6/25-6/29	1.5	1.9	3.4	96	4%		
7	6/29-7/6/01	2.2	2.5	4.7	168	3%	July	7%
3	7/6-7/9	2.5	1.8	4.3	72	6%		
4	7/9-7/13	1.1	1.4	2.5	96	3%		
3	7/13-7/16	32	2.6	34.6	72	48%		
4	7/16-7/20	1.6	1.4	3	96	3%		
3	7/20-7/23	1.6	1.5	3.1	72	4%		
7	7/23-7/30	1.6	1.8	3.4	168	2%		
4	7/30-8/3/01	1.6	1.5	3.1	96	3%	August	4%
3	8/3-8/6	1.4	1.4	2.8	72	4%		
4	8/68/10	2.1	2	4.1	96	4%		
3	8/10-8/13	1.5	1.5	3	72	4%		
4	8/13-8/17	1.7	1.8	3.5	96	4%		
3	8/17-8/20	2.2	1.9	4.1	72	6%		
4	8/20-8/24	2.2	2	4.2	96	4%		
3	8/24-8/27	1	1.3	2.3	72	3%		
4	8/27-8/31	2	1.7	3.7	96	4%		
4	8/31-9/4/01	1.9	1.6	3.5	96	4%	September	7%
3	9/4-9/7	26.5	2.8	29.3	72	41%		
3	9/7-9/10	0	2	2	72	3%		
3	9/10-9/13	1.2	1.2	2.4	72	3%		
5	9/13-9/18	1.5	1.5	3	120	3%		
3	9/18-9/21	1.1	1.1	2.2	72	3%		
7	9/21-9/28	2.8	2.7	5.5	168	3%		

Front Street Lift Station

Front Street Lift Station

Period Days	date	Pump 1 Hours	Pump 2 Hours	Total Pump Hours	Total Period Hours	% Run Time		
6	9/28-10/4/01	1.5	1.5	3	144	2%	October	33%
7	10/4-10/11	9	3.4	12.4	168	7%		
4	10/11-10/15	1.2	5.3	6.5	96	7%		
4	10/15-10/19	1.4	1.3	2.7	96	3%		
6	10/19-10/25	124.6	124.7	249.3	144	173%		
1	10/25-10/26	0.4	0.5	0.9	24	4%		
7	10/26-11/2/01	3.2	3	6.2	168	4%		
3	11/2-11/5	1	1	2	72	3%	November	4%
4	11/5-11/9	1.3	1.2	2.5	96	3%		
5	11/9-11/14	1.7	1.6	3.3	120	3%		
2	11/14-11/16	0.9	0.9	1.8	48	4%		
3	11/16-11/19	1.2	1.2	2.4	72	3%		
7	11/19-11/26	5.5	4.1	9.6	168	6%		
4	11/26-11/30	4.2	3.8	8	96	8%		
4	11/30-12/4/01	10.4	9.6	20	96	21%	December	14%
2	12/4-12/6	3	2.7	5.7	48	12%		
7	12/6-12/13	10.2	9.4	19.6	168	12%		
4	12/13-12/17	7.5	6.8	14.3	96	15%		
7	12/17-12/24	12-Jan	11.5	23.6	168	14%		
2	12/24-12/26/01	1.7	1.6	3.3	48	7%		

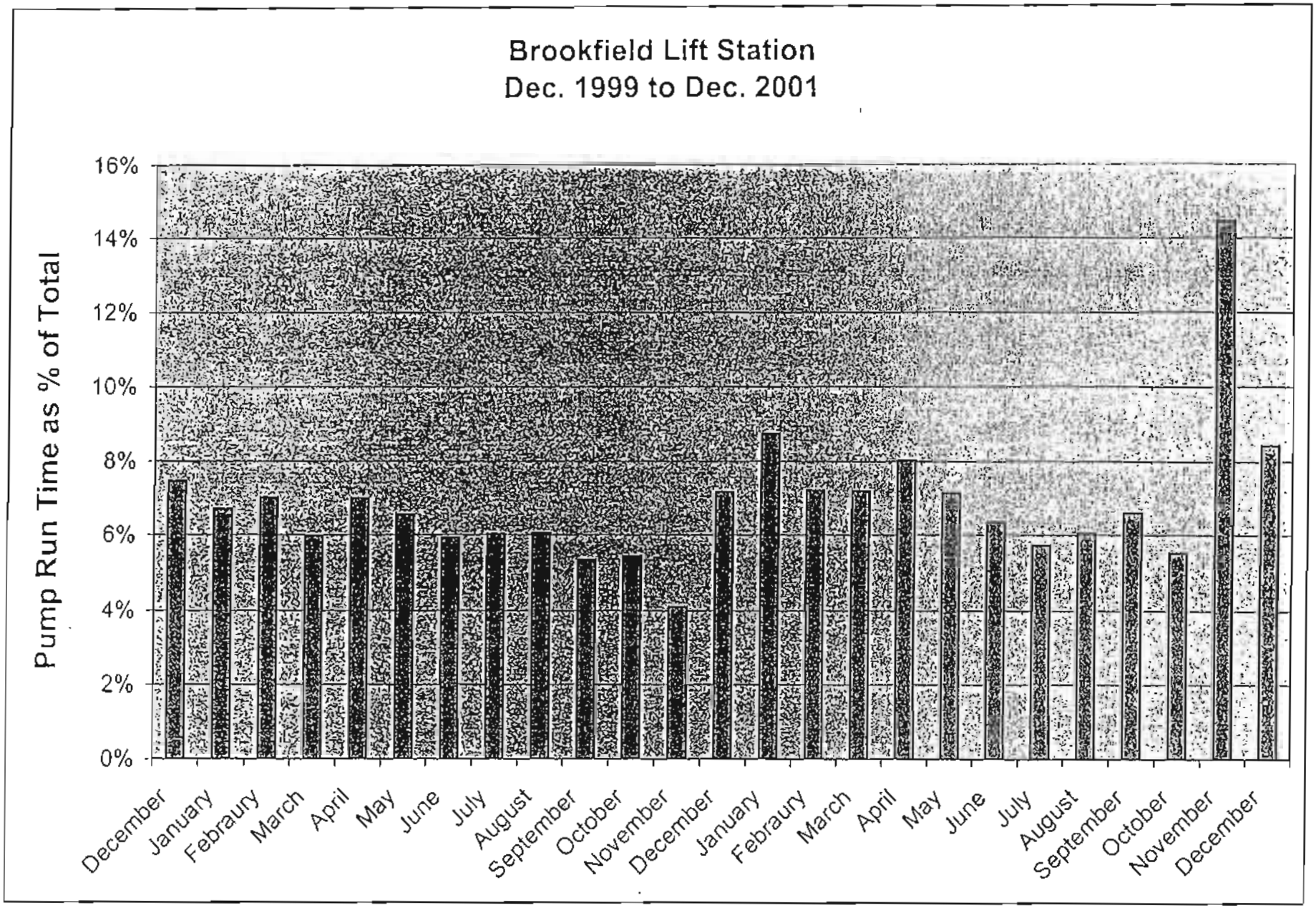

Front Street Lift Station
Dec. 1999 to Dec. 2001
Pump Run Time as % of Total
40%
35%
30%
25%
20%
15%
10%
5%
0%
December
February
April
June
August
October
December
February
April
June
August
October
December

Brookfield Lift Station

Period Days	date	pump#1 hrs.	pump#2 hrs.	Total Pump Hours	Total Period Hours	% Run Time			
17	12/3-12/20/1999	15.65	15.07	30.72	408	8%	December	31	7%
3	12/20-12/23	1.99	2.14	4.13	72	6%		4	
15	12/23-1/7/2000	13.8	14.02	27.82	360	8%		28	
2	1/25-1/27	1.49	1.34	2.83	48	6%	January	3	7%
15	1/27-2/11/00	12.6	11.9	24.5	360	7%		25	
7	2/11-2/18	8.2	6.46	14.66	168	9%	February	15	7%
7	2/18-2/25	3.26	4.99	8.25	168	5%		8	
6	2/25-3/3/00	5.42	5.29	10.71	144	7%		11	
7	3/3-3/10	5.62	5.44	11.06	168	7%	March	11	6%
7	3/10-3/17	5.16	5.12	10.28	168	6%		10	
7	3/17-3/24	4.91	4.94	9.85	168	6%		10	
7	3/24-3/31/	8.9	0	8.9	168	5%		9	
7	3/31-4/7/00	5.7	5.97	11.67	168	7%	April	12	7%
7	4/7-4/14/	5.91	5.97	11.88	168	7%		12	
7	4/14-4/21	6.03	6.06	12.09	168	7%		12	
7	4/21-4/28	5.67	5.64	11.31	168	7%		11	
7	4/28-5/5/00	5.65	5.61	11.26	168	7%	May	11	7%
7	5/5/-5/12	5.3	5.28	10.58	168	6%		11	
7	5/12-5/19	4.8	4.85	9.65	168	6%		10	
4	5/19-5/23	5.17	5.1	10.27	96	11%		10	
8	5/25-6/2/00	5.06	5.04	10.1	192	5%		10	
7	6/2-6/9	4.83	4.87	9.7	168	6%	June	10	6%
6	6/9-6/15	5.5	3.81	9.31	144	6%		9	
7	6/15-6/22	4.68	4.77	9.45	168	6%		9	
14	6/22-7/6/00	9.9	9.95	19.85	336	6%		20	
7	7/6-7/13	5.47	5.5	10.97	168	7%	July	11	6%
8	7/13-7/21	5.69	5.52	11.21	192	6%		11	
6	7/21-7/27	4.3	4.42	8.72	144	6%		9	
8	7/27-8/4/00	5.58	5.48	11.06	192	6%		11	
5	8/4-8/9	4.13	4.06	8.19	120	7%	August	8	6%
8	8/9-8/17	5.08	5.14	10.22	192	5%		10	
8	8/178/25-	5.9	5.92	11.82	192	6%		12	
6	8/25-8/31	4.57	4.48	9.05	144	6%		9	
8	8/31-9/8/00	4.9	5.03	9.93	192	5%	September	10	5%
14	9/8-9/22	9.25	9.37	18.62	336	6%		19	
14	9/22-10/6/00	8.95	8.67	17.62	336	5%		18	
7	10/6-10/13	4.49	4.45	8.94	168	5%	October	9	5%
7	10/13-10/20	4.47	4.32	8.79	168	5%		9	
7	10/20-10/27	4.58	4.57	9.15	168	5%		9	
7	10/27-11/3/00	4.74	4.89	9.63	168	6%		10	
7	11/3-11/10	4.31	4.33	8.64	168	5%	November	9	4%
6	11/10-11/16	5.19	5.2	10.39	144	7%		10	
15	11/16-12/1/00	4.3	4.24	8.54	360	2%		9	
7	12/1/-12/8	6.26	6.78	13.04	168	8%	December	13	7%
6	12/8-12/14-	4.69	4.69	9.38	144	7%		9	
7	12/14-12/21	5.62	5.73	11.35	168	7%		11	
5	12/21-12/26	4.49	4.45	8.94	120	7%		9	
2	12/26-12/28	1.83	1.77	3.6	48	8%		4	
5	12/28-1/2/01	4.35	4.34	8.69	120	7%		9	
2	1/2-1/4	5.96	6.12	12.08	48	25%	January	12	9%
6	1/4-1/8	3.57	3.51	7.08	144	5%		7	
4	1/8-1/12	3.48	13.39	16.87	96	18%		17	
12	1/12-1/25	12.09	2.18	14.27	288	5%		14	
11	1/25-2/5/01	9.26	9.18	18.44	264	7%	February	18	7%

Brookfield Lift Station

Period Days	date	pump#1 hrs.	pump#2 hrs.	Total Pump Hours	Total Period Hours	% Run Time			
4	2/5-2/9	4.21	4.19	8.4	96	9%		8	
13	2/9-2/2	11.39	11.33	22.72	312	7%		23	
8	2/22-3/2/01	6.39	6.32	12.71	192	7%		13	
6	3/2-3/9	6.24	6.09	12.33	144	9%	March	12	7%
7	3/9-3/16	5.29	5.34	10.63	168	6%		11	
7	3/16-3/23	6.16	5.93	12.09	168	7%		12	
14	3/23-4/6	11.69	11.88	23.57	336	7%		24	
7	4/6-4/13	6.98	6.83	13.81	168	8%	April	14	8%
6	4/13-4/20	6.6	6.77	13.37	144	9%		13	
6	4/20-4/26	5.27	5.36	10.63	144	7%		11	
8	4/26-5/4/01	6.95	7.21	14.16	192	7%		14	
5	5/4-5/11	5.38	5.23	10.61	120	9%	May	11	7%
5	5/11-5/16	4.95	5	9.95	120	8%		10	
8	5/16-5/24	6.2	6.2	12.4	192	6%		12	
5	5/24-5/29	3.73	3.71	7.44	120	6%		7	
3	5/29-6/1/01	2.12	2.01	4.13	72	6%		4	
7	6/1-6/8	5.43	5.48	10.91	168	6%	June	11	6%
7	6/8-6/15	4.64	4.78	9.42	168	6%		9	
6	6/15-6/21	5.02	5	10.02	144	7%		10	
7	6/21-6/29	5.48	5.28	10.76	168	6%		11	
7	6/29-7/6/01	4.78	4.83	9.61	168	6%	July	10	6%
7	7/6-7/13	5.1	4.86	9.96	168	6%		10	
7	7/13-7/20	4.75	4.78	9.53	168	6%		10	
7	7/20-7/27	4.92	4.86	9.78	168	6%		10	
7	7/27-8/3/01	4.65	4.63	9.28	168	6%		9	
7	8/3-8/10	5.12	4.96	10.08	168	6%	August	10	6%
7	8/10-8/17	5.08	5.13	10.21	168	6%		10	
7	8/17-8/24	5.18	5	10.18	168	6%		10	
7	8/24-8/31	5.15	5.03	10.18	168	6%		10	
7	8/31-9/7/01	5.07	5.05	10.12	168	6%	September	10	7%
6	9/7-9/13	3.38	4.38	7.76	144	5%		8	
15	9/13-9/28	11.29	11.31	22.6	360	6%		23	
4	9/28-10/2/01	5.15	5.04	10.19	96	11%		10	
10	10/2-10/12	5.04	5.07	10.11	240	4%	October	10	6%
6	10/12-10/18	4.5	4.29	8.79	144	6%		9	
8	10/18-10/26	5.87	5.79	11.66	192	6%		12	
7	10/26-11/2/01	5.43	5.19	10.62	168	6%		11	
7	11/2-11/9	35.43	8.4	43.83	168	26%	November	44	14%
4	11/9-11/13	8.35	8.38	16.73	96	17%		17	
3	11/13-11/16	2.79	2.59	5.38	72	7%		5	
4	11/16-11/20	4.32	4.35	8.67	96	9%		9	
10	11/20-11/30/01	10.52	10.32	20.84	240	9%		21	
3	11/30-12/3	5.96	6	11.96	72	17%		12	
9	12/312/12	6.25	5.85	12.1	216	6%	December	12	8%
12	12/12/12/24	11.36	11.52	22.88	288	8%		23	
2	12/24-12/26	5.88	5.65	11.53	48	24%		12	


Brookfield Lift Station
Dec. 1999 to Dec. 2001
Pump Run Time as % of Total
16%
14%
12%
10%
8%
6%
4%
2%
0%
December
January
Febraury
March
April
May
June
July
August
September
October
November
December
January
Febraury
March
April
May
June
July
August
September
October
November
December

12th Street Pump Station

12th Street Lift Station

Period Days	Date	Pump #1 hrs.	Pump #2 hrs.	Total Pump Hours	Total Period Hours	% Run Time		
3	12/3-12/6/1999	37.3	32.9	70.2	72	97.5%	December	103%
7	12/6-12/13	157.8	79.2	237	168	141.1%		
7	12/13-12/20	161.6	79.2	240.8	168	143.3%		
3	12/20-12/23	23.8	27.9	51.7	72	71.8%		
4	12/23-12/27	21.6	22	43.6	96	45.4%		
3	12/27-12/30	11.4	11.7	23.1	72	32.1%		
4	12/30-1/3/2000	16.3	16.6	32.9	96	34.3%	January	60%
4	1/3-1/7	29.6	34	63.6	96	66.3%		
3	1/7-1/10	25.4	28.9	54.3	72	75.4%		
8	1/10-1/18	67.7	83.7	151.4	192	78.9%		
9	1/18-1/27	60.4	61.1	121.5	216	56.3%		
4	1/27-1/31	19.5	19.2	38.7	96	40.3%		
7	1/31-2/7/00	46.2	58.9	105.1	168	62.6%	February	50%
4	2/7-2/11	28.1	30.7	58.8	96	61.3%		
3	2/11-2/14	15	16	31	72	43.1%		
4	2/14-2/18	22.7	22.2	44.9	96	46.8%		
4	2/18-2/22	12.9	13.5	26.4	96	27.5%		
3	2/22-2/25	14.4	15.7	30.1	72	41.8%		
3	2/25-2/28	20.5	22.1	42.6	72	59.2%		
6	2/28-3/6/00	46.1	48.9	95	144	66.0%	March	39%
4	3/6-3/10	17.7	18.9	36.6	96	38.1%		
7	3/10-3/17	19.6	20.7	40.3	168	24.0%		
3	3/17-3/20	15.8	17.3	33.1	72	46.0%		
4	3/20-3/24	18.5	20	38.5	96	40.1%		
3	3/24-3/27	11.6	13.6	25.2	72	35.0%		
4	3/27-3/31	14.2	14.6	28.8	96	30.0%		
3	3/31-4/3/00	8.5	8.8	17.3	72	24.0%		
3	4/3-4/7	9.1	9.8	18.9	72	26.3%	April	24%
3	4/7-4/10	6	6.1	12.1	72	16.8%		
4	4/10-4/14	8	9.1	17.1	96	17.8%		
3	4/14-4/17	11.8	13	24.8	72	34.4%		
4	4/17-4/21	11.6	12	23.6	96	24.6%		
3	4/21-4/24	7.5	8.1	15.6	72	21.7%		
4	4/24-4/28	11	11.8	22.8	96	23.8%		
3	4/28-5/1/00	10.1	11.1	21.2	72	29.4%		
4	5/1-5/5	11.3	11.9	23.2	96	24.2%	May	33%
3	5/5-5/8	9.3	10.3	19.6	72	27.2%		
4	5/8-5/12	26.4	31.1	57.5	96	59.9%		
3	5/12-5/15	17.3	19.4	36.7	72	51.0%		
4	5/15-5/19	14.9	15.9	30.8	96	32.1%		
3	5/19-5/22	9.7	10.1	19.8	72	27.5%		
3	5/22-5/25	9.2	9.4	18.6	72	25.8%		
5	5/25-5/30	11.8	12	23.8	120	19.8%		
2	5/30-6/1/00	5.4	5.7	11.1	48	23.1%	June	25%
4	6/1-6/5	8.1	8.5	16.6	96	17.3%		
4	6/5-6/9	8.4	9.2	17.6	96	18.3%		
3	6/9-6/12	12.2	13.6	25.8	72	35.8%		
4	6/12-6/16	20.7	25.5	46.2	96	48.1%		
3	6/16-6/19	14.3	16.1	30.4	72	42.2%		
3	6/19-6/22	8.9	9.2	18.1	72	25.1%		
4	6/22-6/26	9.7	10	19.7	96	20.5%		
10	6/26-7/6/00	18.4	19.6	38	240	15.8%		

12th Street Pump Station

12th Street Lift Station

Period Days	Date	Pump #1 hrs.	Pump #2 hrs.	Total Pump Hours	Total Period Hours	% Run Time		
4	7/6-7/10	6	6.4	12.4	96	12.9%	July	11%
3	7/10-7/13	4.7	5.1	9.8	72	13.6%		
4	7/13-7/17	4.9	5.2	10.1	96	10.5%		
4	7/17-7/21	5.1	5.5	10.6	96	11.0%		
3	21-7/24	3.4	3.6	7	72	9.7%		
3	7/24-7/27	3.6	3.9	7.5	72	10.4%		
4	7/27-7/31	4	4.2	8.2	96	8.5%		
4	7/31-8/4/00	4.2	4.6	8.8	96	9.2%	August	9%
3	8/4-8/7	2.7	3	5.7	72	7.9%		
3	8/7-8/10	3.3	3.3	6.6	72	9.2%		
7	8/10-8/17	7.2	7.6	14.8	168	8.8%		
4	8/17-8/21	3.8	4	7.8	96	8.1%		
4	8/21-8/25	3.7	4	7.7	96	8.0%		
3	8/25-8/28	2.8	2.9	5.7	72	7.9%		
3	8/28-8/31	2.9	3.9	6.8	72	9.4%		
5	8/31-9/5/00	4.8	4.9	9.7	120	8.1%	September	8%
2	9/5-9/7	2.1	2.3	4.4	48	9.2%		
4	9/7-9/11	3.8	4	7.8	96	8.1%		
4	9/11-9/15	4.1	4.4	8.5	96	8.9%		
3	9/15-9/18	2.8	3	5.8	72	8.1%		
4	9/18-9/22	3.6	3.7	7.3	96	7.6%		
3	9/22-9/25	2.8	2.8	5.6	72	7.8%		
7	9/25-10/2/00	7.5	8.1	15.6	168	9.3%		
7	10/2-10/9	7.4	7.8	15.2	168	9.0%	October	13%
4	10/9-10/13	4.9	5.2	10.1	96	10.5%		
3	10/13-10/16	4.2	4.3	8.5	72	11.8%		
4	10/16-10/20	5.8	6.1	11.9	96	12.4%		
3	10/20-10/23	7.1	7.4	14.5	72	20.1%		
4	10/23-10/27	5.9	6.3	12.2	96	12.7%		
3	10/27-10/30	7.5	7.9	15.4	72	21.4%		
7	10/30-11/6/00	14.1	14.4	28.5	168	17.0%	November	19%
3	11/6-11/9	8.6	8.4	17	72	23.6%		
4	11/9-11/13	10	10.2	20.2	96	21.0%		
3	11/13-11/16	6	7	13	72	18.1%		
4	11/16-11/20	7	7.1	14.1	96	14.7%		
2	11/20-11/22	3.3	3.5	6.8	48	14.2%		
5	11/22-11/27	10.8	10.9	21.7	120	18.1%		
4	11/27-12/1/00	12.3	12.3	24.6	96	25.6%		
3	12/1-12/4	8.5	8.5	17	72	23.6%	December	33%
4	12/4-12/8	9.3	9.9	19.2	96	20.0%		
3	12/8-12/11	6.2	6.5	12.7	72	17.6%		
3	12/11-12/14	6.8	7.1	13.9	72	19.3%		
4	12/14-12/18	16.2	16.5	32.7	96	34.1%		
3	12/18-12/21	11.7	11.9	23.6	72	32.8%		
5	12/21-12/26	33.7	38.8	72.5	120	60.4%		
2	12/26-12/28	7.3	8.4	15.7	48	32.7%		
4	12/28-1/1/2001	17	18	35	96	36.5%		
3	1/-1/4	8	8	16	72	22.2%	January	32%
4	1/4-1/8	17	19	36	96	37.5%		
4	1/8-1/12	16.1	17	33.1	96	34.5%		
4	1/12-1/16	19	19.9	38.9	96	40.5%		
6	1/16-1/22	22.1	22.7	44.8	144	31.1%		
3	1/22-1.25	11	11.4	22.4	72	31.1%		

12th Street Pump Station

12th Street Lift Station

Period Days	Date	Pump #1 hrs.	Pump #2 hrs.	Total Pump Hours	Total Period Hours	% Run Time		
4	1/25-1/29	10.6	11	21.6	96	22.5%		
7	1/29-2/5/01	30.1	29.8	59.9	168	35.7%	February	31%
4	2/5-2/9	17.7	18.5	36.2	96	37.7%		
3	2/9-2/12	10.8	11.1	21.9	72	30.4%		
4	2/12-2/16	12.8	13	25.8	96	26.9%		
7	2/16-2/23	24.1	24.6	48.7	168	29.0%		
3	2/23-2/26	12.7	13.3	26	72	36.1%		
4	2/26-3/2/01	11.3	11.8	23.1	96	24.1%		
3	3/2-3/5	12.5	12.6	25.1	72	34.9%	March	31%
4	3/5-3/9	12.2	12.3	24.5	96	25.5%		
7	3/9-3/16	17.2	17.1	34.3	168	20.4%		
3	3/16-3/19	10.6	10.6	21.2	72	29.4%		
4	3/19-3/23	15.8	15.5	31.3	96	32.6%		
10	3/23-4/2/01	46.3	44.9	91.2	240	38.0%		
4	4/2-4/6	18.6	17.7	36.3	96	37.8%	April	37%
7	4/6-4/13	40	39.8	79.8	168	47.5%		
3	4/13-4/16	14.7	15.3	30	72	41.7%		
4	4/16-4/20	16.3	16.4	32.7	96	34.1%		
3	4/20-4/23	11	11.1	22.1	72	30.7%		
3	4/23-4/26	10.1	10.1	20.2	72	28.1%		
4	4/26-4/30	15.2	15.2	30.4	96	31.7%		
4	4/30-5/4/01	23.5	23	46.5	96	48.4%	May	29%
3	5/4-5/7	11.8	12.3	24.1	72	33.5%		
3	5/7-5/10	9.5	9.5	19	72	26.4%		
4	5/10-5/14	10.1	9.8	19.9	96	20.7%		
2	5/14-5/16	11	11.2	22.2	48	46.3%		
5	5/16-5/21	18.3	18.6	36.9	120	30.8%		
3	5/21-5/24	8.2	8.5	16.7	72	23.2%		
5	5/24-5/29	11.3	11.8	23.1	120	19.3%		
3	5/29-6/1/01	6.2	6.4	12.6	72	17.5%		
3	6/1-6/4	7	7.2	14.2	72	19.7%	June	17%
4	6/4-6/8	9	9.1	18.1	96	18.9%		
3	6/8-6/11	7	7.5	14.5	72	20.1%		
4	6/11-6/15	6.7	7	13.7	96	14.3%		
3	6/15-6/18	7.5	8.1	15.6	72	21.7%		
3	6/18-6/21	5.3	5.5	10.8	72	15.0%		
4	6/21-6/25	6.4	7.2	13.6	96	14.2%		
4	6/25-6/29	6.8	6.8	13.6	96	14.2%		
7	6/29-7/6/01	10	10.6	20.6	168	12.3%	July	10%
3	7/6-7/9	5.1	4	9.1	72	12.6%		
4	7/9-7/13	4.5	5	9.5	96	9.9%		
3	7/13-7/16	3.3	3.6	6.9	72	9.6%		
4	7/16-7/20	4.6	4.5	9.1	96	9.5%		
7	7/20-7/27	7.1	7.5	14.6	168	8.7%		
3	7/27-7/30	2.7	2.7	5.4	72	7.5%		
4	7/30-8/3/01	3.7	3.8	7.5	96	7.8%	August	10%
3	8/3-8/6	2.5	2.8	5.3	72	7.4%		
4	8/6-8/10	3.9	4	7.9	96	8.2%		
3	8/10-8/13	3	3.1	6.1	72	8.5%		
4	8/13-8/17	3.5	3.6	7.1	96	7.4%		
3	8/17-8/20	2.7	2.9	5.6	72	7.8%		
4	8/20-8/24	8.2	8.5	16.7	96	17.4%		

12th Street Pump Station

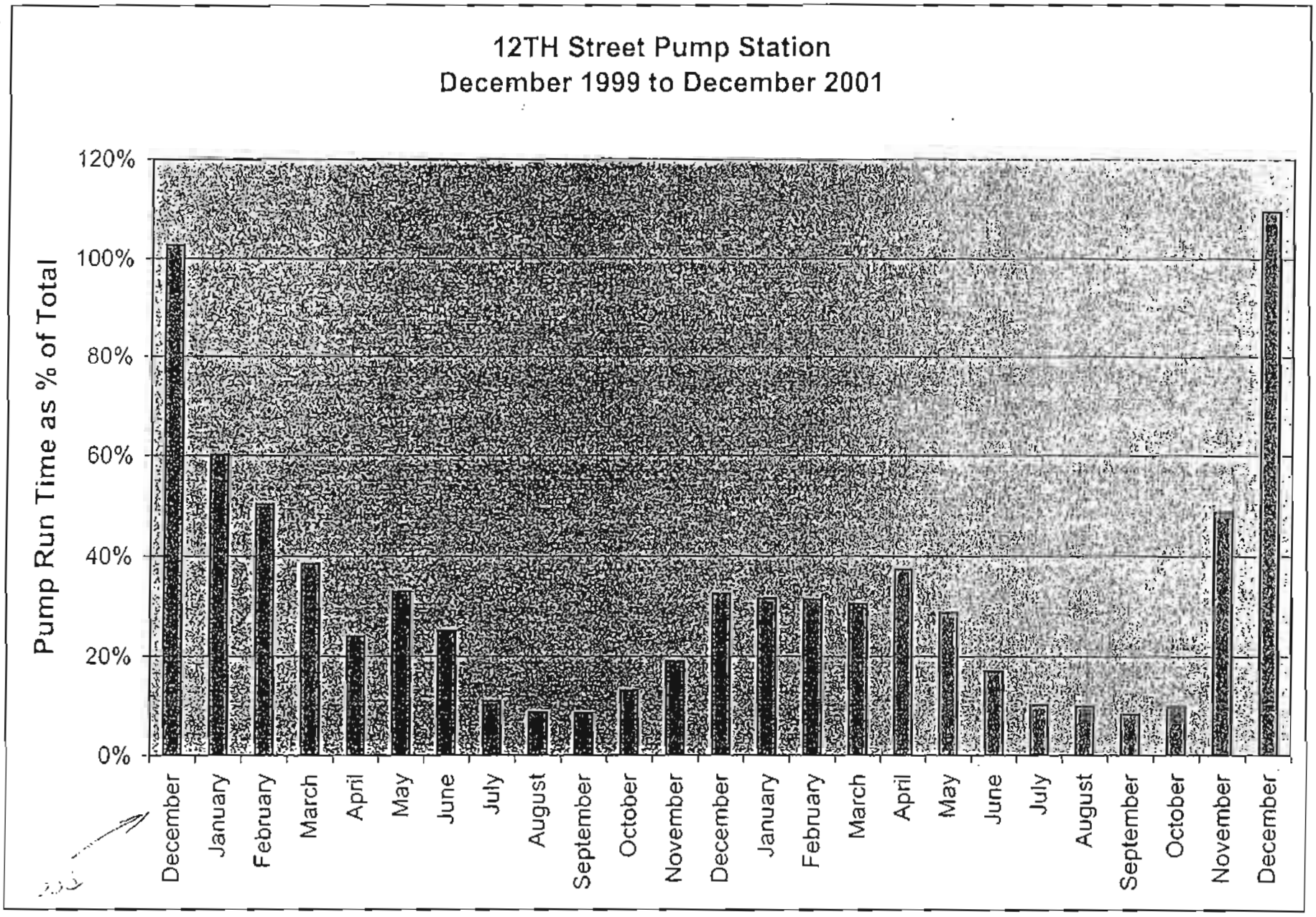
12th Street Lift Station

Period Days	Date	Pump #1 hrs.	Pump #2 hrs.	Total Pump Hours	Total Period Hours	% Run Time		
3	8/24-8/27	4.7	4.8	9.5	72	13.2%		
4	8/27-8/31	4.8	4.9	9.7	96	10.1%		
4	8/31-9/4/01	4.4	4.7	9.1	96	9.5%	September	8%
3	9/4-9/7	3.7	3.1	6.8	72	9.4%		
3	9/7-9/10	3	3.2	6.2	72	8.6%		
3	9/10-9/13	3.2	3.2	6.4	72	8.9%		
4	9/13-9/17	3.4	3.2	6.6	96	6.9%		
4	9/17-9/21	3.8	4	7.8	96	8.1%		
7	9/21-9/28	6.6	5.7	12.3	168	7.3%		
7	9/28-10/5/01	5.8	6.5	12.3	168	7.3%	October	10%
7	10/5-10/12	7.1	6.5	13.6	168	8.1%		
3	10/12-10/15	3.1	3.2	6.3	72	8.8%		
3	10/15-10/18	3.1	3.1	6.2	72	8.6%		
8	10/18-10/26	10	10	20	192	10.4%		
7	10/26-11/201	11.6	11.6	23.2	168	13.8%		
3	11/2-11/5	6.3	6.5	12.8	72	17.8%	November	49%
4	11/5-11/9	7.7	8	15.7	96	16.4%		
4	11/9-11/13	6.6	6.7	13.3	96	13.9%		
1	11/13-11/14	7.6	7.6	15.2	24	63.3%		
2	11/14-11/16	16.8	17.4	34.2	48	71.3%		
3	11/16-11/19	16	16.8	32.8	72	45.6%		
7	11/19-11/26	53.7	59.2	112.9	168	67.2%		
4	11/26-11/30	47.9	43.2	91.1	96	94.9%		
6	11/30-12/6/01	91.2	97.4	188.6	144	131.0%	December	110%
7	12/6-12/13	109.2	87.5	196.7	168	117.1%		
4	12/13-12/17	88.4	88.4	176.8	96	184.2%		
7	12/17-12/24	97.2	38.9	136.1	168	81.0%		
3	12/24-12/27	12.4	0	12.4	72	17.2%		
8	12/27-1/4/02	44.5	0	44.5	192	23.2%	January	99%
3	1/4-1/7	26.3	0	26.3	72	36.5%		
7	1/7-1/14	118	15.5	133.5	168	79.5%		
4	1/14-1/18	45.4	26.4	71.8	96	74.8%		
4	1/18-1/22	78.6	58	136.6	96	142.3%		
3	1/22-1/25	74.6	49.4	124	72	172.2%		
3	1/25-1/28	66.3	66.3	132.6	72	184.2%		
3	1/28-1/31	93	73.3	166.3	72	231.0%		
1	1/31-2/1/02	118.5	71.5	190	24	791.7%	February	106%
6	2/1-2/7	84.4	68.2	152.6	144	106.0%		
4	2/7-2/11	52	32	84	96	87.5%		
3	2/11-2/14	47.9	26.8	74.7	72	103.8%		
5	2/14-2/19	52	31	83	120	69.2%		
3	2/19-2/22	31.4	18.5	49.9	72	69.3%		
3	2/22-2/25	31.5	19.6	51.1	72	71.0%		
4	2/25-3/1/02	32.1	23.1	55.2	96	57.5%		
3	3/1/-3/4	17.9	15.3	33.2	72	46.1%	March	82%
4	3/4-3/8	24.4	18.7	43.1	96	44.9%		
3	3/8-3/11	33.1	19.4	52.5	72	72.9%		
3	3/11-3/14	65.9	44.7	110.6	72	153.6%		
4	3/14-3/18	43.4	41.2	84.6	96	88.1%		
3	3/18-3/21	68.6	59.6	128.2	72	178.1%		
4	3/21-3/25	56.5	31.9	88.4	96	92.1%		
11	3/25-4/5/02	90.8	59.6	150.4	264	57.0%		

12th Street Pump Station

12th Street Lift Station

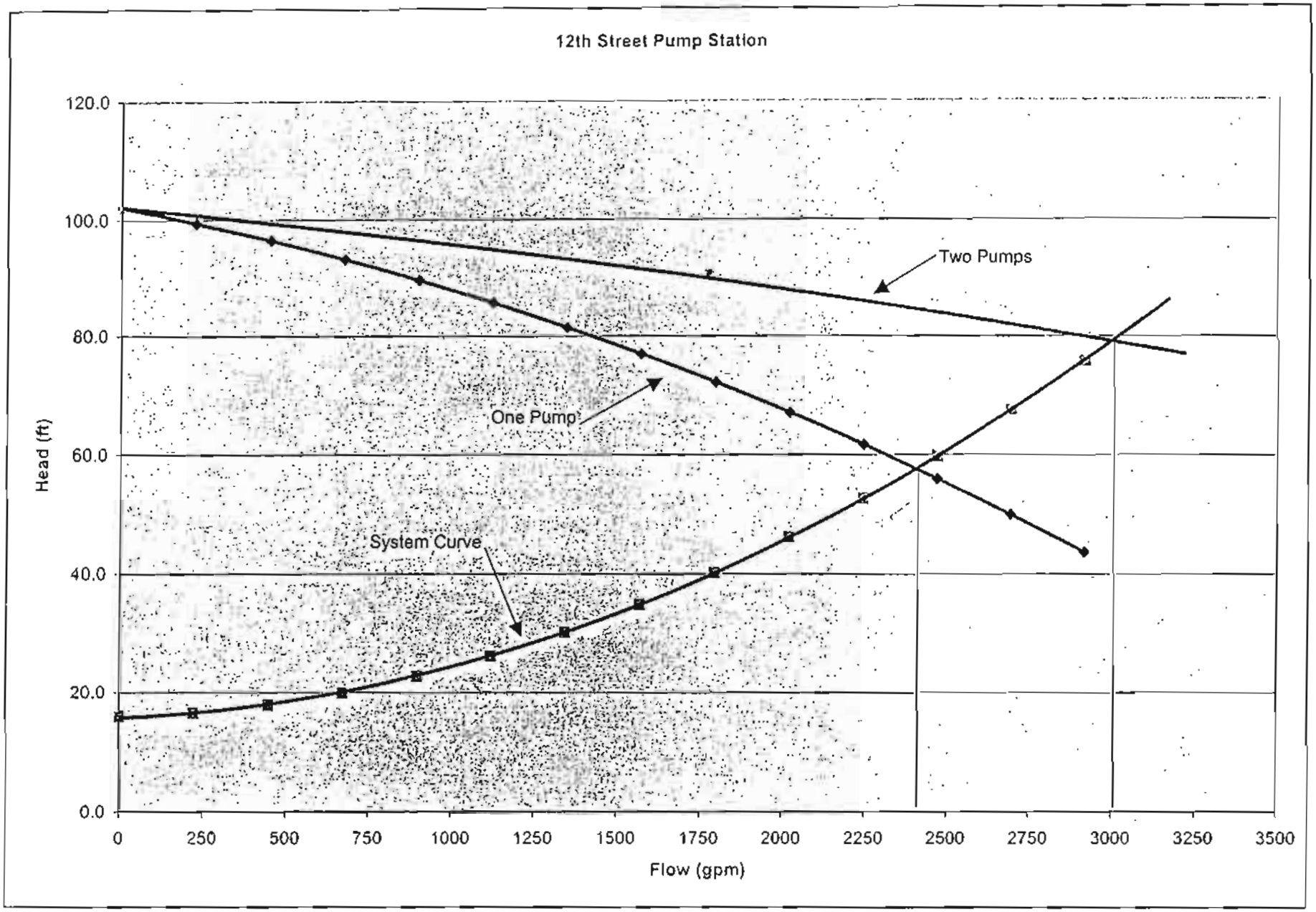
Period Days	Date	Pump #1 hrs.	Pump #2 hrs.	Total Pump Hours	Total Period Hours	% Run Time		
4	4/5-4/9	22.2	17.2	39.4	96	41.0%	April	62%
9	4/9-4/18	122.6	69.2	191.8	216	88.8%		
5	4/18-4/23	49	31.3	80.3	120	66.9%		
6	4/23-4/29	41.4	30.6	72	144	50.0%		
3	4/29-5/2	2.1	14.1	16.2	72	22.5%		
4	5/2-5/6	40.1	17.4	57.5	96	59.9%	May	37%
3	5/6-5/9	14.5	13.2	27.7	72	38.5%		
4	5/9-5/13	15.4	15.5	30.9	96	32.2%		
3	5/13-5/16	11.9	12	23.9	72	33.2%		
8	5/16-5/24	28.6	30.8	59.4	192	30.9%		
6	5/24-5/30	25.5	25.2	50.7	144	35.2%		
7	5/30-6/6/02	26.2	28.8	55	168	32.7%	June	43%
7	6/6-6/13	22.7	28.2	50.9	168	30.3%		
8	6/13-6/21	22	71.7	93.7	192	48.8%		
6	6/21-6/27	14.8	55.7	70.5	144	49.0%		
5	6/27-7/2/02	19.9	48	67.9	120	56.6%		
9	7/2-7/11	16.2	60.5	76.7	216	35.5%	July	38%
4	7/11-7/15	9.1	36.5	45.6	96	47.5%		
3	7/15-7/18	7.2	28.8	36	72	50.0%		
5	7/18-7/23	2.4	38.5	40.9	120	34.1%		
6	7/23-7/29	14	65.3	79.3	144	55.1%		
7	7/29-8/5/02	5.7	27.9	33.6	168	20.0%		
7	8/5-8/12	14.1	67	81.1	168	48.3%	August	53%
4	8/12-8/16	6.6	38.9	45.5	96	47.4%		
3	8/16-8/19	4.8	26.3	31.1	72	43.2%		
4	8/19-8/23	7	38.5	45.5	96	47.4%		
4	8/23-8/27	13.7	77.7	91.4	96	95.2%		
10	8/27-9/6/02	17.5	98.5	116	240	48.3%		
3	9/6-9/9	5.5	31.2	36.7	72	51.0%	September	48%
4	9/9-9/13	6.9	39	45.9	96	47.8%		
4	9/13-9/17	7.2	39	46.2	96	48.1%		
3	9/17-9/20	4.4	28.1	32.5	72	45.1%		
3	9/20-9/23	5.1	30	35.1	72	48.8%		
4	9/23-9/27	6.7	38.1	44.8	96	46.7%		
4	9/27-10/1/02	7.1	37.6	44.7	96	46.6%		
3	10/1-10/4	6.1	32	38.1	72	52.9%	October	29%
4	10/4-10/8	8	37.7	45.7	96	47.6%		
3	10/8-10/11	4.6	28.8	33.4	72	46.4%		
3	10/11-10/14	4.6	29.6	34.2	72	47.5%		
4	10/14-10/18	5.9	37.9	43.8	96	45.6%		
3	10/18-10/21	3	4.3	7.3	72	10.1%		
3	10/21-10/24	2.1	2.3	4.4	72	6.1%		
4	10/24-10/28	3.1	3.3	6.4	96	6.7%		
4	10/28-11/1/02	3	3.1	6.1	96	6.4%		
3	11/1-11/4	2.5	2.6	5.1	72	7.1%	November	15%
4	11/4-11/8	4.2	4.6	8.8	96	9.2%		
4	11/8-11/12	7	3.5	10.5	96	10.9%		
2	11/12-11/14	4.7	5	9.7	48	20.2%		
4	11/14-11/118	8.3	9	17.3	96	18.0%		
4	11/18-1/22	12	12.7	24.7	96	25.7%		
3	11/22-11/27	8.8	9.5	18.3	72	25.4%		
8	11/27-12/5	11	11.5	22.5	192	11.7%		


12TH Street Pump Station
December 1999 to December 2001
Pump Run Time as % of Total
120%
100%
80%
60%
40%
20%
0%
December
January
February
March
April
May
June
July
August
September
October
November
December
January
February
March
April
May
June
July
August
September
October
November
December

12th Street Lift Station

12th Street Pump Station

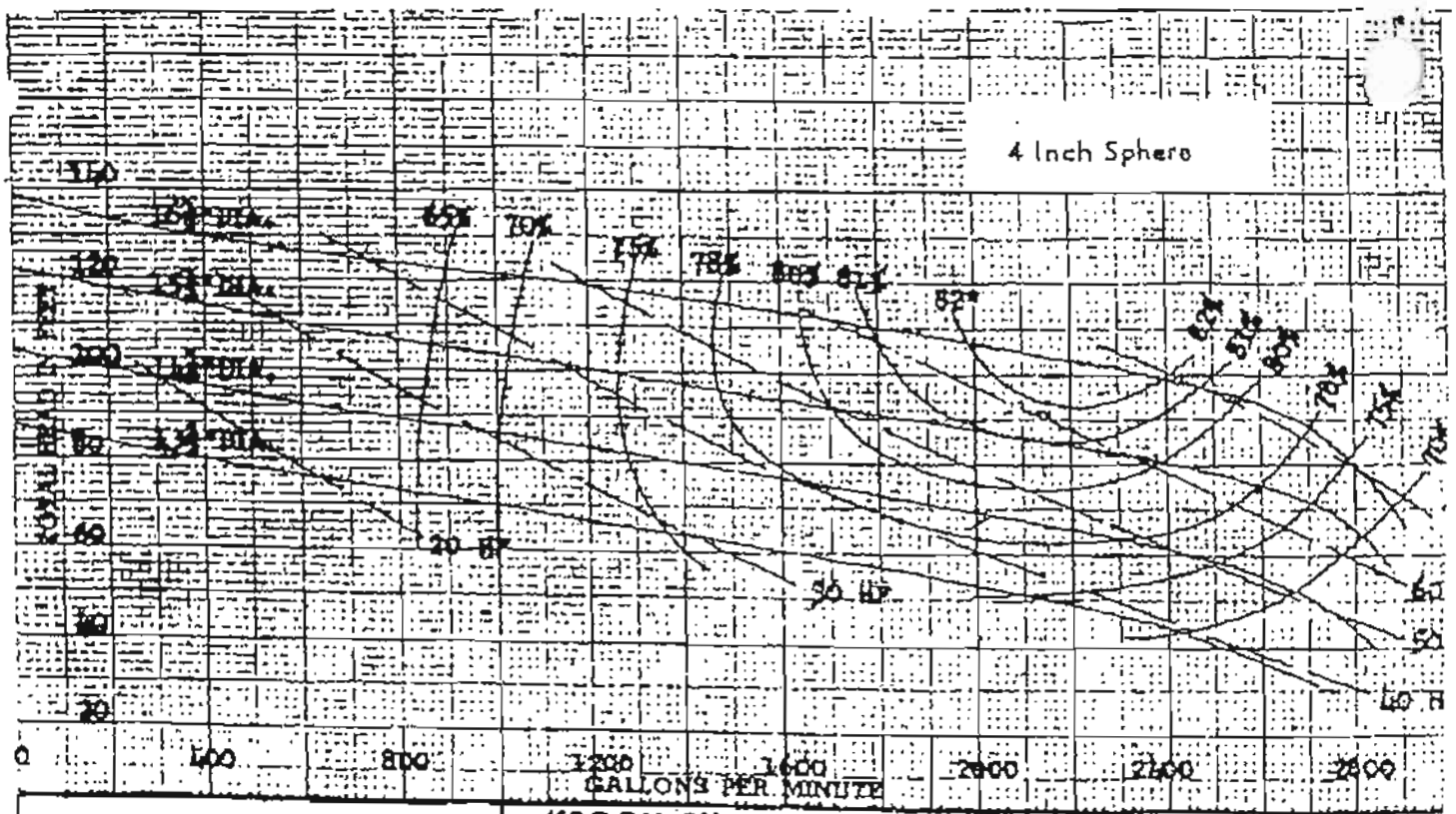
Period Days	Date	Pump #1 hrs.	Pump #2 hrs.	Total Pump Hours	Total Period Hours	% Run Time		
4	12/5-12/9	4.6	4.7	9.3	96	9.7%	December	57%
3	12/9-12/12	12.4	13.1	25.5	72	35.4%		
4	12/12-12/16	57.2	37.1	94.3	96	98.2%		
4	12/16-12/20	36.8	46.2	83	96	86.5%		
3	12/20-12/23	18.1	19.9	38	72	52.8%		
3	12/23-12/26	23.9	25.9	49.8	72	69.2%		
4	12/26-12/30/02	19.2	20.8	40	96	41.7%		


12th Street Pump Station
120.0
100.0
80.0
60.0
40.0
20.0
0.0
Head (ft)
0
250
500
750
1000
1250
1500
1750
2000
2250
2500
2750
3000
3250
3500
Flow (gpm)
Two Pumps
One Pump
System Curve

Allen Harvey
503 295 4901

Ruben
707.747.7599

Chuck



Peerless Pump Curve
bought by Krough

State of Oregon
Department of Environmental Quality — Memorandum

To: Beth Moore (Water Quality/NWR) **Date:** 8/27/04

From: Rob Burkhart (Water Quality/NWR)

Subject: Tillamook WWTP MZ and ZID Dilutions

Summary

Consultants for the City of Tillamook provided DEQ with an analysis of the Tillamook WWTP's mixing zone (MZ) and zone of initial dilution (ZID) in October 2003. Among other things, this analysis provided estimated dilutions for variously defined MZs and ZIDs.

Using the receiving water and discharge characterization data provided by Kennedy/Jenks, as well as other discharge data, I've performed an analysis (see below) to estimate the critical dilutions at the edges of the permit-defined MZ and ZID. This analysis resulted in an estimated critical dilution at the edge of the ZID of 1.4 and an estimated critical dilution at the edge of the MZ of 6.5 with the current outfall, and 19.1 with a single port diffuser set at the bottom of the river near the bank.[1,2] (Note: A new diffuser design will not impact the critical dilution at the edge of the ZID.)

Background

The City of Tillamook hired Kennedy/Jenks Consultants to conduct a study on effluent water quality for the Tillamook WWTP. The resulting report, *Wastewater Treatment Plant Effluent Water Quality Study, City of Tillamook*, was completed in October 2003. Within the report was an analysis of the available dilutions at the edge of the ZID and the MZ. Kennedy/Jenks provided varying dilutions dependent on the size of the ZID/MZ and the flow/velocity of the receiving stream (the Trask River).

The goal of the analysis presented here is to estimate the critical dilutions at the edge of the MZ and ZID as they are defined in the current Tillamook WWTP NPDES permit and using the appropriate data related to discharge and receiving water characteristics. Often, as noted below, the only data available are from the consultant's report.

ZID Dilution

The "zone of initial dilution" (ZID) is the area immediately surrounding the outfall. Outside of this defined area the acute water quality criteria must be met. The Tillamook WWTP NPDES Permit (#101239) defines the ZID as "that portion of the allowable mixing zone that is 10 feet downstream and 5 feet upstream of the point of discharge" (Permit Schedule A, 1.a.(4)). Since acute criteria are generally based on 1-hour average conditions, data from the worst-case 1-hour time period were used to determine the critical dilution. These data include the ambient (receiving stream) flow (and associated velocity) and the discharge flow. The critical ambient flow and velocity are usually the lowest 1-hour average. This assumption has been verified

[1] The term "dilution" is defined here as the ratio of the initial concentration (at the discharge point) to the concentration at the given location. The maximum available dilution may be expressed as $(Q_{ambient} + Q_{effluent})/Q_{effluent}$.

[2] The single port diffuser modeled had specific attributes (diameter, location, etc.) that may be modified to enhance the dilution.

by modeling to estimate the dilutions at higher flows and velocities (which, for the ZID, resulted in higher dilutions).

The lowest 1-hour average ambient velocity measured by Kennedy/Jenks at the discharge point was 0.004 m/s (from monitoring performed between 9/6/02 and 10/4/02). Based on the river's estimated cross-sectional area, this velocity was equated to a flow of 0.19 m^3/s. The critical effluent flow used in calculating ZIDs and MZs is usually the design average dry weather flow (ADWF). The design ADWF given in Kennedy/Jenks' Wastewater Treatment Facilities Plan (12/03) is 0.9 MGD. To add a margin of safety, the value of 1 MGD, or 0.044 m^3/s, has been used in this current analysis to estimate the instream dilutions.[3]

Due to the low ambient river flow (0.19 m^3/s), the maximum dilution, utilizing the full river flow, will be limited based on the following equation:

$$\text{Max Dilution} = (Q_e + Q_a)/ Q_e$$

Where: Q_e is the critical effluent flow, and
Q_a is the critical ambient river flow

Standard DEQ procedure is to use a Q_a value substantially less than the full stream flow at the edge of the ZID (usually on the order of 10%) in calculating the dilution. This allows for fish passage around the ZID. At 10% of the ambient river flow (0.019 m^3/s), the above equation gives a maximum dilution of 1.4. This is considered to be the critical dilution at the edge of the ZID.

Modeling of these conditions with Cormix results in an estimation that this maximum allowable dilution will be met almost immediately downstream of the discharge point. This is the case for both the existing outfall design and for the scenario with a single port diffuser located at the bottom of the river. (Note: Due to a bug in the software, the model continues to give larger dilutions further from the point of discharge – this situation was confirmed in a discussion with R. Doneker, one of the authors of Cormix).

MZ Dilution

The Tillamook WWTP NPDES Permit (#101239) defines the mixing zone as "that portion of the Trask River, which extends 100 feet downstream and 50 feet upstream of the point of discharge" (Permit Schedule A, 1.a.(4)). Chronic water quality criteria must be met outside of this defined area. Since chronic criteria are generally based on 4-day average conditions, data from the worst-case 4-day time period were used to determine the critical dilution. The critical ambient flow and velocity is usually the lowest 4-day average. This assumption has been checked by also modeling the dilutions at both the average 4-day flow and the maximum 4-day flow (based on monitoring performed between 9/6/02 and 10/4/02).

For the mixing zone analysis, Cormix modeling was performed to estimate the dilution with the current outfall and also for a scenario including a single-port diffuser located at the bottom of the Trask River.

For the scenario including the current outfall, the modeling results indicated that the critical ambient velocity was the minimum of the measured 4-day average velocities – 0.127 m/s. Based on the river's estimated cross-sectional area at this velocity, Kennedy/Jenks calculated that this equated to a flow of 7.63 m^3/s. Using this ambient river flow and an effluent flow of 1MGD (the ADWF plus a margin of safety), the estimated dilution at the edge of the mixing zone is 6.5.

[3] Kennedy/Jenks used a value for the critical effluent flow of 0.105 m^3/s, based on the maximum effluent flow during dry weather conditions over the four years prior to 2002.

For the scenario with a new outfall (a single port diffuser located at the bottom of the river), the modeling results indicated that the critical ambient velocity was the average of the measured 4-day average velocities – 0.167 m/s. Based on the river's estimated cross-sectional area at this velocity, the flow at this velocity was equal to 10.04 m^3/s. The modeling results predict a critical dilution of 19.1 at the edge of the MZ for this scenario.

Discussion/Conclusion

As stated above, the estimated critical dilution at the edge of the ZID is 1.4, and the estimated critical dilution at the edge of the MZ is 6.5 (with the current outfall). The low dilution for the ZID is primarily due to the lack of flow (and associated velocity) occurring in the receiving water during slack tide. (It may be prudent for the discharger to verify these low flows).

While the outfall design is not a critical factor in the dilution at the edge of the ZID, it is important for the dilution at the edge of the MZ. Due to the current outfall location on the bank, the effluent plume is predicted to be bank attached (which is not desirable) during most conditions. Modeling of a scenario with a new single port diffuser located at the bottom of the river gives an estimated critical dilution of 19.1 at the edge of the MZ.

Tillamook WWTP MZ and ZID Dilutions		
	Dilution at Edge of ZID	Dilution at Edge of MZ
Current Outfall	1.4	6.5
New Outfall w/Diffuser	1.4	19.1

Wastewater Treatment Plant
Effluent Water Quality Study
City of Tillamook
Tillamook, Oregon

9 October 2003

K/J 026076.00

Prepared for:

City of Tillamook
210 Laurel Avenue
Tillamook, Oregon 97141

Prepared by:

Kennedy/Jenks Consultants
2828 SW Naito Parkway, Suite 250
Portland, Oregon 97201

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Section 1: Summary

This report provides an assessment of potential water quality impacts from the City of Tillamook Wastewater Treatment Plant (WWTP) as required by the Oregon Department of Environmental Quality (DEQ).

1.1 Background

DEQ has listed the Trask River as being water quality limited for bacteria and temperature. On 31 July 2001, the U.S. Environmental Protection Agency (EPA) approved a total maximum daily load (TMDL) for the Tillamook Bay Watershed, which includes the Trask River. The TMDL assigned specific thermal load and waste load allocations, including a temperature waste load allocation for the Tillamook WWTP. These allocations are designed to assure there is no measurable temperature increase outside of the assigned mixing zone during critical periods of low dilution.

In a 13 September 2001 letter to the City of Tillamook, DEQ required that temperature monitoring, water quality sampling, and a mixing zone study be done in connection with the Tillamook WWTP National Pollutant Discharge Elimination System (NPDES) permit renewal. The requirements were subsequently incorporated into the Tillamook WWTP NPDES permit. These requirements have been completed and are documented in this WWTP Effluent Water Quality Study, which assesses potential water quality impacts to the Trask River from the Tillamook WWTP. This report describes when and how temperature was monitored, and when and how in-stream flows measurements were taken. A description of how effluent and receiving water quality will be assessed for chemical parameters of concern is also given. This report includes a study estimating the mixing zone in the receiving water. The mixing zone results were used to assess potential impacts from temperature and effluent constituents, including reasonable potential analysis to exceed water quality standards.

1.2 Mixing Zone

In-stream velocity and temperature were collected and used to conduct a study to estimate the mixing zone in the receiving water. The Cornell Mixing Zone Expert System (CORMIX) was used to model the effluent mixing within the receiving water. As required by DEQ, the model parameters used represent worst-case low flow and dry weather conditions.

The mixing zone results were then used to assess potential impacts from effluent constituents. The reasonable potential analysis was performed to determine if it was likely that effluent discharge would result in exceeding water quality standards in the receiving water.

1.2.1 Results

The acute toxicity was evaluated at the zone of initial dilution (ZID). The worst-case mixing occurs when the minimum 4-day ambient flow rate of 7.63 cubic meters per second (m^3/s) is combined with a 4-year maximum dry weather effluent flow rate of 0.105 m^3/s. Under these conditions, the centerline dilution is 10 (1:9) fold in the near field region or ZID. The ZID was

predicted to have a length of 86 meters and a width of 18 meters from the effluent discharge pipe. The regulatory mixing zone is set at 5 feet upstream and 10 feet downstream. Using the regulatory ZID length of 1.5 meters (5 feet), the ZID was predicted to have a width of 4.8 meters and a centerline dilution factor of 3.2 (1:2.2). With a regulatory ZID length of 3.0 meters (10 feet), the ZID was predicted to have a width of 4.5 meters and a centerline dilution factor of 4.5 (1:3.5).

Kennedy/Jenks Consultants used 7-day average flows to model conditions of chronic exposure under tidal influence. We feel that the 7-day average flow more accurately represents conditions of exposure over several changing tidal cycles.

The regulatory mixing zone was set at 15 meters (50 feet) upstream and 30 meters (100 feet) downstream. The 7-day average minimum ambient flow rate used was 8.59 m^3/s and the 4-year maximum dry weather effluent flow of 0.015 was used to model these conditions. Using a 7-day average low flow and a mixing zone boundary length of 15 meters, the mixing zone was predicted to be 7 meters wide and have a centerline dilution factor of 9 (1:8). By changing the mixing zone boundary to 30 meters in length, the mixing zone was predicted to be 7 meters wide and have a centerline dilution factor of 9 (1:8). For the dilution at the edge of the mixing zone to have the least chronic effects, the length of the mixing zone was predicted to be 370 meters long and 33 meters wide. The centerline dilution factor at the edge of the mixing zone would be 20 (1:19) and have a bulk dilution of 34.

1.3 Reasonable Potential Analysis

Three rounds of sampling of the effluent and receiving water upstream of the WWTP discharge pipe in the Trask River were analyzed for metals and cyanide. The results were compared against acute 1-hour Criteria Maximum Concentration (CMC) and chronic 4-day criteria continuous concentration (CCC) water quality criteria after taking account for the dilution in the ZID and the edge of the mixing zone, which was determined in the mixing zone study.

The initial modeling approach indicated that a potential for exceeding water quality criteria existed for several effluent constituents. Kennedy/Jenks Consultants had phone conversations and met with DEQ on 28 January 2003 concerning the reasonable potential to exceed water quality criteria. Kennedy/Jenks Consultants proposed using a 7-day minimum receiving water flow instead of the 4-day minimum flow (initially used) to model the size and dilution within the mixing zone. Because of unique tidal conditions on the Trask River, DEQ indicated that a 7-day average receiving water flow may be justifiable. This 7-day dilution factor was then applied to the 4-day CCC.

1.3.1 Results

The reasonable potential analysis (RPA) for acute CMC was conducted using four different scenarios: 1) 1-hour ZID length of 0.9 meters, 2) 4-day ZID lengths of 1.5 meters, 3) 3.0 meters, and 4) 86 meters. There was reasonable potential to exceed copper and silver in all four of the acute RPA scenarios. There was also reasonable potential to exceed total cyanide in all but the ZID of 86 meters long. The ZID of 1.5 meters long had the reasonable potential to exceed zinc water quality standards.

The RPA for the 7-day CCC used mixing zone lengths of 15 meters, 30 meters, and 370 meters and their corresponding centerline dilution factors of 9, 9, and 20, respectively. There was reasonable potential to exceed water quality standards for aluminum, silver, and total cyanide for the mixing zone length of 370 meters. The 15 meter and 30 meter mixing zone lengths had a reasonable potential to exceed aluminum, copper, silver, mercury, and total cyanide. However, it is important to note that both aluminum and silver background concentrations in the Trask River were found to be higher than the chronic 4-day CCC levels.

1.4 Temperature

No measurable increase in receiving water temperature is allowed in stream segments that fail to meet temperature water quality standards. A significant increase in temperature is defined as an increase in ambient temperature at the edge of the mixing zone of 0.25 degrees Fahrenheit (°F) [0.14 degrees Celsius (°C)] or greater.

Tillamook's WWTP NPDES permit does not allow the effluent to increase the temperature of the Trask River by 0.14 °C (0.25 °F). Maximum effluent temperature taken during the dry season over a 4-year period was used in the model. Minimum ambient temperature was taken over 30 days containing the maximum ambient temperature. These temperatures were used for determining no significant increase.

Based on our discussion with DEQ, Kennedy /Jenks Consultants used the maximum 7-day average temperature and the highest monthly temperature of the Trask River to determine if there was no measurable increase.

1.4.1 Results

Using the 7-day ambient low flow and mixing zone lengths of 15 meters and 30 meters, the average difference in ambient temperature was 0.15 °C and a maximum temperature of 0.30 °C. Both the 15 meter and 30 meter mixing zone lengths had a measurable increase in temperature at the edge of the mixing zone. The 370-meter mixing zone length had an average difference in ambient temperature of 0.07 °C and a maximum temperature of 0.14 °C. The mixing zone length of 370 meters did not have a measurable increase in ambient temperature at the end of the mixing zone.

The maximum effluent temperature of 22.7 °C is also below the incipient lethal temperature of 25 °C (77 °F).

Section 2: Mixing Zone Study Methods

Kennedy/Jenks Consultants used the CORMIX GI version 4.1 (CORMIX) Hydrodynamic Mixing Zone Model to mathematically model effluent discharges from the Tillamook WWTP. The CORMIX model is a recognized modeling software package used by EPA, state agencies, and consultants for mixing zone studies. Cornell University developed the model under a cooperative agreement with EPA. The model is distributed by the EPA Center for Environmental Assessment Modeling located in Athens, Georgia. Dr. Robert Doneker, Oregon Graduate Institute, provides support for the model.

CORMIX modeled the effluent as buoyant surface discharge that enters the receiving water laterally through a pipe. The Tillamook WWTP has only one outfall (001), which was modeled for worst-case low flow and dry weather conditions.

The results of the modeling provide dilutions for the near field ZID and at the edge of the mixing zone. These results will be used in the RPA for the acute and chronic water quality criteria and for the temperature standards in the Tillamook NPDES permit.

2.1 Ambient Receiving Water Data

The geometry of the streambed was measured with staff gauges, measuring tapes, and other survey equipment, but not surveyed by a licensed surveyor. The measurements are accurate for the purposes of running the CORMIX model. Measurements of the streambed are relative to the outfall 001 and described as a cross section (see Figure 1). The average cross section and bank slopes were used in the CORMIX model (see Table 1).

2.1.1 Flow

Flow data was measured from 6 September through 4 October 2002 to represent dry weather flow conditions. Flow was measured by using a Unidata Starflow ultrasonic doppler instrument with a micro data logger. The Starflow meter measures depth, velocity, and with the cross sectional area of the Trask River, the Starflow calculated flow. Flow is summarized on Table 1; ambient velocity and plots are provided in Appendix A. Average flow during this period was 9.99 m^3/s. The rolling 1-hour average minimum flow was 0.19 m^3/s. This flow is in general agreement with the flow rate measured at the U.S. Geological Survey (USGS) Station No. 14302480 Trask River above Cedar Creek, about 9 miles upstream. The 4-day and 7-day rolling average minimum flows were 7.63 m^3/s and 8.59 m^3/s, respectively. These conditions were used to model worst-case receiving water conditions.

2.1.2 Temperature

Temperature was monitored at two locations in the Trask River. A VEMCO temperature datalogger recorded temperatures every 15 minutes. One datalogger was placed approximately 50 meters upstream and the other was placed approximately 30 meters downstream of the effluent discharge pipe.

Temperature was measured from 13 August through 12 November 2002 to capture maximum receiving water temperatures. The highest receiving water temperature recorded was 24.3 °C on 28 August 2002. Receiving water data used for the study encompassed 30 days of the maximum ambient temperature. The minimum receiving water temperature during this time was 16.5 °C, which was also the minimum 1-hour rolling average. The minimum 4-day and 7-day rolling average temperatures were 18.3 °C and 18.7 °C, respectively. These temperatures were used to model worst-case receiving water conditions. Ambient receiving water conditions used in the CORMIX model are described in Table 1.

2.2 Effluent Data

The geometry of the discharge pipe was measured for use in the CORMIX 3 model. The geometry of the effluent discharge area is shown on Figure 1. Effluent flow and temperature were measured during critical low flow and dry weather conditions.

2.2.1 Flow

Effluent flow was continuously monitored and reported in Tillamook's WWTP daily monitoring reports (DMR). Maximum effluent flow during dry weather conditions over the past four years was 0.105 cubic meters per second (m^3/s) and was used to model worst case conditions.

2.2.2 Temperature

Effluent temperature was measured every 15 minutes by a VEMCO temperature datalogger. Measurements were first taken on 13 August 2002 and are to be monitored continuously over the next two years. Temperature was monitored in the effluent discharge pipe before it reached the Trask River. Effluent data used during the study encompassed 30 days of the maximum effluent temperature. Effluent temperatures averaged 21.3 °C for the month that contained the maximum temperature. The maximum effluent temperature was 25.8 °C on 27 August 2002. The maximum 1-hour rolling average during the study was 23.4 °C and the maximum 4-day and 7-day rolling average during the study were 22.9 °C and 22.7 °C, respectively. Effluent data used for CORMIX modeling is described in Table 1.

Section 3: Modeling Conditions

Kennedy/Jenks Consultants used a 2000 version of the EPA-approved CORMIX mixing zone model to conduct this WWTP Effluent Water Quality Study. The 2000 version has improved hydrodynamic modeling and graphical output compared to the older version.

Direct measurements were made for stream and outfall characteristics. Minimum ambient water temperatures and maximum effluent temperatures were coupled with low stream velocities to model worst-case conditions. Because average velocities were used, tidal entrapment models featured within CORMIX was not used. One-hour rolling averages, along with 4-day and 7-day rolling averages for minimum stream temperature and flow rate, and maximum effluent temperature and flow were used to determine the extent of the mixing zone.

Section 4: Modeling Results

The near field mixing dilutions and size of the mixing area is summarized in Table 2. Complete session reports, prediction files, and graphical representations of the mixing zones are provided in Appendix B.

4.1 1-Hour Results

The 1-hour average flow was used to model the 1-hour acute effects and evaluate mixing within the ZID (see Table 2). The regulatory mixing zone is set at 5 feet upstream and 10 feet downstream. Since the Trask River is tidally influenced, the direction of the river is dependent on the tides. When the river changes directions, there is a point when the river is almost stagnant. Using the 1-hour average receiving water flow of 0.19 m^3/s, the ZID was predicted to have a length of 0.9 meters and a width of 10 meters. The centerline dilution was 7 (1:6) fold in the near field region or ZID.

4.2 4-Day Results

Kennedy/Jenks Consultants evaluated how higher flow rates affect the ZID (see Table 2). The worst-case mixing occurs when the minimum 4-day ambient flow rate of 7.63 m^3/s is combined with a 4-year maximum dry weather effluent flow rate of 0.105 m^3/s. Under these conditions, the centerline dilution is 10 (1:9) fold in the near field region or ZID. The ZID was predicted to have a length of 86 meters and a width of 18 meters from the effluent discharge pipe. The regulatory mixing zone is set at 5 feet upstream and 10 feet downstream. Using the regulatory ZID length of 1.5 meters (5 feet), the ZID was predicted to have a width of 4.8 meters and a centerline dilution factor of 3.2 (1:2.2). With a regulatory ZID length of 3.0 meters (10 feet), the ZID was predicted to have a width of 4.5 meters and a centerline dilution factor of 4.5 (1:3.5).

Four-day average low flow rates were used to evaluate chronic effects within the mixing zone. The Trask River is tidally influenced and changes direction according to the tides. The regulatory mixing zone is set at 15 meters (50 feet) upstream and 30 meters (100 feet) downstream. Using a 4-day average low flow and a mixing zone boundary of 15 meters long, the mixing zone was predicted to be 9 meters wide and have a centerline dilution factor of 8 (1:7). By changing the mixing zone boundary to 30 meters long, the mixing zone was predicted to be 10 meters wide and have a centerline dilution factor of 9 (1:8). For the dilution at the edge of the mixing zone to have the least chronic effects, the length of the mixing zone was predicted to be 2,500 meters long and 33 meters wide. The centerline dilution factor at the edge of the mixing zone would be 20 (1:19) and have a bulk dilution of 34.

4.3 7-Day Results

Kennedy/Jenks Consultants used 7-day average flows to model conditions of chronic exposure under tidal influence. We feel that the 7-day average flow more accurately represents conditions of exposure over several changing tidal cycles.

The 7-day average low flow rate of 8.59 m^3/s combined with a 4-year maximum dry weather effluent flow rate of 0.105 m^3/s was used to model the ZID (see Table 2). With a regulatory length of 1.5 meters (5 feet), the ZID was predicted to be 4.4 meters wide and have a centerline dilution factor of 3.4 (1:2.4). Using the regulatory length of 3.0 meters (10 feet), the model predicted the ZID to be 5.2 meters wide and have a centerline dilution factor of 5.0 (1:4). Complete mixing occurred at a length of 51 meters and a width of 11 meters. The centerline dilution factor was 10 (1:9).

The regulatory mixing zone was set at 15 meters (50 feet) upstream and 30 meters (100 feet) downstream. The 7-day average minimum ambient flow rate used was 8.59 m^3/s and the 4-year maximum dry weather effluent flow of 0.015 was used to model these conditions. Using a 7-day average low flow and a mixing zone boundary length of 15 meters, the mixing zone was predicted to be 7 meters wide and have a centerline dilution factor of 9 (1:8). By changing the mixing zone boundary to 30 meters in length, the mixing zone was predicted to be 7 meters wide and have a centerline dilution factor of 9 (1:8). For the dilution at the edge of the mixing zone to have the least chronic effects, the length of the mixing zone was predicted to be 370 meters long and 33 meters wide. The centerline dilution factor at the edge of the mixing zone would be 20 (1:19) and have a bulk dilution of 34.

Section 5: Reasonable Potential Analysis

Kennedy/Jenks Consultants conducted three rounds of effluent and background receiving water sampling for the RPA. All samples were collected and analyzed using ultra clean sampling methods. Effluent samples were collected at the last manhole before discharge. Background samples were collected at the end of low slack tide about 50 meters upstream of the outfall. Background sample collection times were carefully selected to obtain non-tidally influenced samples.

The RPA followed methods outlined in the Technical Support Document for Water Quality-based Toxics Control (EPA 1991). This is a statistical approach where a confidence interval is added to the highest concentration found during the three rounds of sampling. Based on the statistical table for 99 percent confidence level and 99 percent probability basis, and a coefficient of variation of 0.6, three data points would require a factor of 5.6 times the highest concentration detected in the three rounds of sampling. These results are compared against the CMC and CCC water quality criteria after taking account for the dilution in the zone of initial dilution and the edge of the mixing zone, which was determined in Section 3 of this report. The CCCs and CMCs are provided in Tables 3, 4, and 5. An example calculation is shown below.

Maximum Effluent Concentration (C_{EFMAX}) = highest concentration in effluent x 5.6 (from EPA table).

Concentration at Zone of Initial Dilution (ZID) = $[(DF_{ZID} - 1) \times C_{BG} + C_{EFMAX}] / DF_{ZID}$.

Concentration at Mixing Zone (MZ) = $[(DF_{MZ} - 1) \times C_{BG} + C_{EFMAX}] / DF_{MZ}$.

Where DF_{ZID} is the dilution factor for the zone of initial dilution.

DF_{MZ} is the dilution factor for the mixing zone.

C_{BG} is the background concentration.

C_{EFMAX} is the statistically derived maximum likely effluent concentration.

During the third round of sampling, background levels were significantly elevated compared to the previous sampling rounds. Kennedy/Jenks Consultants rejected this background dataset because we believed the samples may have been influenced by tidally enhanced effluent. Hence, background was determined using the highest background levels out of the first two rounds of sampling. The maximum effluent concentration was determined by the highest concentration found during all three rounds of sampling.

The RPA for acute CMC was conducted using four different scenarios: 1) 1-hour ZID length of 0.9 meters, 2) 4-day ZID lengths of 1.5 meters, 3) 3.0 meters, and 4) 86 meters (see Table 3). The CMC dilution factors were taken from the CORMIX 1-hour and 4-day low flow modeling results. Using the 1-hour, the ZID was predicted to be 0.9 meters in length and 12 meters wide, with a centerline dilution of 7. The 4-day low flow modeling results predicted the ZID to be 1.5 meters long and 4.8 meters wide with a centerline dilution factor of 3.2. The 4-day model of 3.0

meters long and 5.8 meters wide with a centerline dilution factor of 4.5 was used, along with a ZID 86 meters long and 18 meters wide with a centerline dilution of 10. The results of the acute RPA are shown in Table 3. There is a potential to exceed copper and silver standards in all four scenarios. There is also a reasonable potential to exceed total cyanide in all but the ZID of 86 meters long. The ZID of 1.5 meters long has the potential to exceed zinc water quality standards.

The chronic CCC was compared by using the 4-day and 7-day low flow modeling results. The dilution factors were taken from the CORMIX 4-day and 7-day low flow modeling results. Three mixing zone lengths were used in the RPA for the 4-day and 7-day analysis. The 4-day analysis results are shown in Table 4 and the 7-day analysis in Table 5.

The RPA for the 4-day CCC used mixing zone lengths of 15 meters, 30 meters, and 2,500 meters and their corresponding centerline dilution factors of 8, 9, and 20, respectively. The results are shown in Table 4. There is a potential to exceed water quality standards for aluminum, silver, and total cyanide for a mixing zone length of 2,500 meters. The 15 meter and 30 meter mixing zone lengths have a potential to exceed aluminum, copper, silver, mercury, and total cyanide. The 15 meter mixing zone length could exceed lead levels. Receiving water background concentrations of aluminum and silver exceed the 4-day CCC standard.

The RPA for the 7-day CCC used mixing zone lengths of 15 meters, 30 meters, and 370 meters and their corresponding centerline dilution factors of 9, 9, and 20, respectively. The results are shown in Table 5. There is a potential to exceed water quality standards for aluminum, silver, and total cyanide for the mixing zone length of 370 meters. The 15 meter and 30 meter mixing zone lengths have a potential to exceed aluminum, copper, silver, mercury, and total cyanide. Receiving water background concentrations of aluminum and silver exceed the 4-day CCC standard.

Complete water quality analysis results for three rounds of sampling are provided in Table 6. Full laboratory analysis results are provided in Appendix C.

5.1 Ammonia

DEQ prepared an Evaluation Review Report for the Tillamook WWTP on 6 January 2003. In their report, DEQ evaluated the ammonia toxicity at the ZID and at the edge of the mixing zone. DEQ used a pH of 7.4 and an ambient temperature ranging from 57.4 °F (14.1 °C) to 68 °F (20 °C). A dilution of 3.0 was used at the ZID and a dilution of 100 was used for the edge of the mixing zone.

Kennedy/Jenks Consultants used the 4-day ambient low flow to determine if there was reasonable potential to exceed water quality ammonia standards at the ZID and at the edge of the mixing zone (see Table 7). With a ZID length of 1.5 meters and 3.0 meters, the centerline dilution achieved was 3.2 and 4.5, respectively. When using a mixing zone length of 15 meters and 30 meters, the centerline dilution achieved was 8 and 9, respectively.

Based on the statistical table for 99 percent confidence level and 99 percent probability basis, and a coefficient of variation of 0.6, seven data points would require a factor of 2.5 times the highest concentration detected in the seven rounds of sampling by DEQ. These results are

compared against the CMC and CCC water quality criteria after taking account for the dilution at the ZID and the edge of the mixing zone.

There was no reasonable potential to exceed ammonia standards within the ZID, but there is a potential that the discharge may not meet ammonia standards at the edge of the mixing zone.

Section 6: Temperature

The Trask River is listed as being water quality limited for several constituents, including temperature. EPA approved the TMDL for the Tillamook basin, including the Trask River, on 31 July 2001. The TMDL assigned specific thermal load and waste allocations, including a temperature waste load allocation for the Tillamook WWTP. These allocations are designed to assure that there is no measurable increase beyond site potential temperature outside the mixing zone during critical periods of low dilution.

No measurable increase in surface water temperature is allowed in stream segments that contain threatened and endangered species, if that increase would impair the biological integrity of the threatened and endangered populations. Coho salmon is listed as a threatened species and the Oregon Department of Fish and Wildlife (ODFW) show Coho salmon rearing and migration on the Trask River in the area of the outfall. Therefore, no measurable increase in the temperature is allowed above the natural background temperatures, which may cause an adverse effect on fish. A significant increase in temperature is defined as a temperature change of 0.25 °F (0.14 °C) or greater at the edge of the mixing zone.

Tillamook's WWTP NPDES does not allow the effluent to increase the temperature of the Trask River by 0.14 °C (0.25 °F) at the edge of the mixing zone. Maximum effluent temperature taken during the dry season over a 4-year period was used in the model. Average ambient temperature was taken over the same 30-day period containing the maximum ambient temperature. These temperatures were used for determining no significant increase.

The following equation was used to determine if the effluent temperature from the facility would cause a measurable increase in the natural background temperature and/or the water quality criterion.

$$T_{MZ} = (T_E + DT_S) / (D + 1)$$

Where:

T_{MZ} = Temperature of the stream at the edge of the mixing zone
T_E = Effluent temperature
T_S = Temperature of the Trask River (Average monthly temperature = 20 °C during the month containing the highest temperature)
D = Dilution factor at the edge of the mixing zone = 8, 9, and 20.

The calculated temperatures at the end of the mixing zone are summarized in Table 7. Using the 4-day ambient low flow and mixing zone length of 15 meters, the average difference in temperature was 0.16 °C and a maximum of 0.36 °C. The 30 meter mixing zone length had an average difference in ambient temperature of 0.15 °C and a maximum of 0.32 °C. Both the 15 meter and 30 meter mixing zone lengths had a measurable increase in temperature at the edge of the mixing zone. The mixing zone length of 2,500 meters had a difference in ambient temperature of 0.07 °C and a maximum of 0.14 °C. With a mixing zone length of 2,500 meters, there was no measurable increase in ambient temperature at the end of the mixing zone.

Using the 7-day ambient low flow and mixing zone lengths of 15 meters and 30 meters, the average difference in ambient temperature was 0.15 °C and a maximum temperature of 0.30 °C. Both the 15 meter and 30 meter mixing zone lengths had a measurable increase in temperature at the edge of the mixing zone. The 370 meter mixing zone length had an average difference in ambient temperature of 0.07 °C and a maximum temperature of 0.14 °C. The mixing zone length of 370 meters did not have a measurable increase in ambient temperature at the end of the mixing zone.

The maximum effluent temperature of 22.7 °C (72.9 °F) is below the incipient lethal temperature of 25 °C (77 °F).

Section 7: Time of Travel Estimate

Kennedy/Jenks Consultants estimated the worst-case time of travel for flushing effluent from the Tillamook WWTP outfall to the mouth of the Trask River. This was requested by DEQ to assess the potential for dissolved oxygen sag in the Trask River from biochemical oxygen demand in the effluent. The distance between the WWTP outfall and the mouth of the Trask River was estimated using the USGS 7.5 Minute Quadrangle map. The mouth of the Trask River was assumed to be located at the convergence with the Tillamook River. This convergence is approximately 0.5 miles from Tillamook Bay. The total distance between the WWTP outfall and the mouth of the Trask River was estimated to be 7,180 feet (or 1.4 miles).

Using the Kennedy/Jenks Consultants' stream flow monitoring data collected in July 2002, Kennedy/Jenks Consultants calculated the lowest 25-hour average velocity (25 hours was used to be consistent with the tidal period) for the period of record. This velocity was used to estimate a worst-case time of travel for the Trask River. The results are presented in the table below.

Time of Travel

Lowest 25-Hour Average Velocity (ft/sec)	Distance to Mouth (ft)	Time of Travel (hrs)
0.2	7,180	10

The analysis assumed a constant cross-sectional area between the WWTP outfall and the Trask River mouth; there were no contributory flows.

Section 8: Habitat Assessment

DEQ requested additional information on the habitat potentially affected by the WWTP discharge.

8.1 Setting

The Tillamook WWTP is located at approximately mile 1.9 on the lower Trask River, as tributary to Tillamook Bay. The Trask River near and downstream of the outfall is characterized by its transition from a freshwater riverine environment to an estuarine environment. The Trask River has a low gradient, widens and flow rates slow, and is tidally influenced. The lower Tillamook Bay is generally well mixed and has lowered salinity ranging from 0 to 10 parts per thousand (Ellis 1999). Hence, salinity may rise and fall within the environment, but it is unlikely that a separate salt wedge occurs within the river near the outfall. The river bed environment is characterized by steep cut banks and constructed rip-rap and soft bottoms with fine sand and silts.

8.2 Fish Species

Fish species likely present in the lower Trask River and South Tillamook Bay estuarine area, based on the 1999 Monitoring Report for Tillamook Bay (ibid), are summarized in Table 8.

Of the fish species in Table 8, the only state or federally listed as threatened or endangered is the Oregon coast Coho salmon (*Oncorhynchus kisutch),* which has a federal threatened listing. Other fish species listed in the beneficial use analysis of the Tillamook Bay Watershed TMDL are spring and fall runs of Chinook (*Oncorhynchus tshawytscha*), chum (dog) (*Oncorhynchus keta*), and steelhead (*Oncorhynchus mykiss*) salmon. The only salmonid species capable of potentially spawning in brackish water is *Oncorhynchus keta.* However, the Oregon Department of Fish and Game, in a letter dated 9 March 2001, established that there is no suitable gravel spawning habitat in the lower Trask River near or downstream of the WWTP outfall. Hence, the beneficial uses of this habitat for salmonids include rearing and passage.

8.3 Invertebrates

Kennedy/Jenks Consultants found little published information on invertebrates potentially present in the lower Trask River. However, there have been benthic surveys of Tillamook Bay, which may include estuarian species that could be present in the estuarian environment of the lower Trask River.

Invertebrates potentially present in the lower Trask River estuary are derived from *A Biological Inventory of Benthic Invertebrates in Tillamook Bay* (Golden et al. 1998). ODFW surveyed Tillamook Bay in the summer of 1996 to inventory the Bay's benthic invertebrates. The emphasis of the survey was to estimate clam density and biomass in selected areas and habitats. The benthic invertebrate grab samples resulted in the identification of 154 taxa at 15 grab areas. At least eight of these are introduced species and six are cryptogenic or of unknown origin. The top five most abundant taxa were: Oligochaetes spp., 49%; the introduced

cumacean *Nippoleucon hinumensis*, 8%; the introduced gamarid amphipod *Grandidierella japonica*, 6%; the introduced spionid polychaete *Pseudopolydora kempi*, 5%; and the polychaete *Capitella capitata* 'hyperspecies,' 4%. Thirteen taxa of bivalves were identified through the grab samples, three of which were not seen, and two of which were not quantified in the clam survey. Of the superfamily Corophioidea, juvenile *Corophioidea sp.*, the introduced species *Corophium acherusicum,* and *Corophium spinicorne* were identified.

There are no federal or state listed threatened or endangered invertebrate species that could potentially inhabit the lower Trask River estuarine environment.

8.4 Amphibians

Kennedy/Jenks Consultants found little published information on amphibians present in the lower Trask River. However, amphibians are generally not tolerant of brackish or salt water. Hence, amphibians are not expected to be present in the tidally influenced portion of the lower Trask River. Moreover, there are no state or federal threatened or endangered amphibian species listed in Oregon.

8.5 Reptiles

According to ODFW, sea turtles occasionally come ashore on the Oregon coast, but the only turtles living year-round in Oregon are the painted turtle and the western pond turtle, both of which are found in parts of the Willamette Valley. There are no tortoises in Oregon.

Terrestrial lizards and skinks are not likely to be directly affected by the aquatic environment.

The only threatened or endangered reptiles in Oregon are sea turtles, which are occasional coastal visitors unlikely to be present in an estuarine environment.

8.6 Birds

Numerous bird species interact with the lower Trask River for feeding, congregating, resting, and habitat. A variety of grebe, pelagic, cormorant, pelican, heron, crane, duck, geese, swan, shorebird, gull, tern, and alcid species may spend time in this area. In addition, piscavorous raptors including accipiters, buteos, eagles, and osprey may hunt in the area. Of these, the Brown Pelican (*Pelecanus occidentalis*), Aleutian Canada goose (*Branta canadensis leucopareia*), bald eagle (*Haliaeetus leucocephalus*), American peregrine falcon (*Falco peregrinus anatum*), Arctic peregrine falcon (*Falco peregrinus tundrius*), western snowy plover (*Charadrius alexandrinus nivosus*), California least tern (*Sterna antillarum browni*), and marbled murrelet (*Brachyramphus marmoratus*) are listed as state or federal threatened or endangered.

8.7 Mammals

The lower Trask River is likely host to a variety of aquatic mammals, including mammals that prey upon fish and invertebrates, such as the river otter (*Lutra canadensis*), herbivorous beaver (*Castor canadensis*) and omnivors including muskrat (*Ondatra zibethicus*), raccoon (*Procyon lotor*), and nutria (Myocastor coypus). Bats may feed on insects and fish in the river. Marine

mammals including the California sea lion (*Zalophus californianus*) and harbor seals (*Phoca vitulia*) may occasionally venture up the Trask River in pursuit of fish prey, but are not likely to regularly be present. Cetatians are not normally present in riverine systems.

While several bat species are federal "species of concern", none of the above mammals are listed as state or federal threatened or endangered.

Section 9: Findings

Since the Trask River is tidally influenced, mixing zones will need to be applied both upstream and downstream of the outfall. This mixing zone study used under very severe conditions of extremely low ambient flow, high effluent flow, maximum effluent temperatures and minimum ambient stream conditions.

The worst-case ZID was identified to occur during the 4-day low ambient flow by the CORMIX model. Mixing in the near field was predicted to have a length of 86 meters and a width of 18 meters. The centerline dilution factor at the ZID under these conditions was 10. The regulatory ZID length of 1.5 meters had a width of 4.8 meters and a centerline dilution factor of 4.5. The other regulatory ZID length of 3.0 meters had a width of 5.8 meters and a centerline dilution factor of 4.5. These dilution factors were used in the acute RPA. There was reasonable potential to exceed water quality standards for copper and silver for all four scenarios. There was also reasonable potential to exceed total cyanide in all but the ZID of 86 meters long. The ZID of 1.5 meters long had the reasonable potential to exceed zinc water quality standards.

The regulatory mixing zone was determined by DEQ to be 15 meters upstream and 30 meters downstream. Since the Trask River is tidally influenced, the direction of the river is dependent on the tides. Therefore, the determination of upstream and downstream is also dependent on the tides. Kennedy/Jenks Consultants suggest that the size of the mixing zone should be based on the 7-day CORMX model, which predicted the width to be 7 meters when the regulatory lengths are 15 meters or 30 meters. The centerline dilution factor for both of these mixing zone sizes are 9 (1:8), which was used in the reasonable potential analysis. Since the centerline dilution factors are the same for both the 15 meter and 30 meter mixing zones, the reasonable potential analysis results are also the same. There was reasonable potential to exceed water quality standards for aluminum, copper, silver, mercury, and total cyanide. The Trask River has background concentrations of aluminum and silver that are higher than the chronic 4-day CCC.

The receiving water background concentrations of aluminum and silver exceeded the 4-day CCC standard.

The 7-day maximum average temperature does not cause a significant increase to the Trask River. This maximum temperature of 22.7 °C (72.9 °F) is also below the incipient lethal temperature of 25 °C (77 °F).

Changes to the effluent outfall design would significantly increase the mixing in the Trask River. The outfall is in an indentation at the bank of the river, not in the main channel. A submerged outfall with diffusers placed in the center of the channel would greatly improve mixing and reduce the size of the ZID and mixing zone. Multiport diffusers would be used to disperse effluent equally. A mixing zone study would be performed to determine the best location and configuration for the new outfall.

Section 10: References

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Table 1: CORMIX 3 Parameters

Ambient Receiving Water Conditions	1-Hour Average Ambient Flow	4 Day Average Ambient Flow	7-Day Average Ambient Flow
Average Depth (m)	1.44	1.81	1.81
Discharge Depth (m)	1.44	1.81	1.81
Flow Rate (m^3/s) Minimum	0.19	7.63	8.59
Bounded Stream Width (m)	33.2	33.2	33.2
Channel Appearance	2	2	2
Manning's N	0.035	0.035	0.035
Wind Speed (m/s)	2	2	2
Water Type	Fresh	Fresh	Fresh
Temperature (°C) Minimum[a]	16.5	18.3	18.7
Effluent Data			
Flow Rate (m^3/s) Maximum	0.105	0.105	0.105
Type	Fresh	Fresh	Fresh
Temperature (°C) Maximum[a]	23.4	22.9	22.7
Surface Discharge Geometry			
Discharge Location (looking downriver)	Right Bank	Right Bank	Right Bank
Configuration	Flush	Flush	Flush
Horizontal Angle σ (degrees)	90	90	90
Bottom Slope θ (degrees)	40	40	40
Discharge Type	Rectangular	Rectangular	Rectangular
Channel Width (m)	0.43	0.43	0.43
Channel Depth (m)	0.381	0.381	0.381
Depth at Discharge (m)	1.44	1.81	1.81
Mixing Zone Specification			
Water Quality Standard (mg/l)		Set at 100 for percent calculations	
Mixing Zone Area of Concern	100	100	100
Region of Interest (m)	10,000	10,000	10,000
Grid Intervals	25	25	25

Notes:

(a) Temperature used for calculations was from 13 August through 12 September 2002. This consisted of one month's data containing the maximum temperature during the study.

Based condition is worst-case condition required for mixing zone study in Tillamook's NPDES Permit.

Extreme condition is used for sensitivity analysis of CORMIX Model.

Table 2: Zone of Immediate Dilution and Mixing Zone Size and Dilution Factor

	1 Hour Average Receiving Water Flow				4-Day Average Receiving Water Flow				7-Day Average Receiving Water Flow			
	Length (m)	Width (m)	Centerline Dilution	Bulk Dilution	Length (m)	Width (m)	Centerline Dilution	Bulk Dilution	Length (m)	Width (m)	Centerline Dilution	Bulk Dilution
ZID	0.9	10.0	7.0	12.0	1.5	4.8	3.2	5.4	1.5	4.4	3.4	5.8
					3.0	5.8	4.5	7.7	3.0	5.2	5.0	8.5
					86	18	10	17	51	11	10	17
Mixing Zone	NA	NA	NA	NA	15	9	8	13	15	7	9	15
	NA	NA	NA	NA	30	10	9	14	30	7	9	16
	NA	NA	NA	NA	2,500	33	20	34	370	33	20	34

Table 3: Acute Reasonable Potential Analysis Results

Parameter	Background Concentration mg/l	Highest Concentration mg/l	Maximum Effluent Concentration Multiplier = 5.6	Maximum Concentration at ZID (1 Hour 0.9m) mg/l	Maximum Concentration at ZID (4 Day 1.5 m) mg/l	Maximum Concentration at ZID (4 Day 3.0 m) mg/l	Maximum Concentration at ZID (4 Day 86 m) mg/l	1 Hour (CMC) mg/l	Reasonable Potential			
									0.9 m	1.5 m	3.0 m	86 m
Cr (III)	0.0005	0.002	0.01120	0.00203	0.00384	0.00288	0.00207	0.58522	NO	NO	NO	NO
Cr (VI)	0.00013	0.00042	0.00235	0.00045	0.00082	0.00062	0.00048	0.01600	NO	NO	NO	NO
As (III)	0.0005	0.0005	0.00280	0.00083	0.00122	0.00101	0.00123	0.36000	NO	NO	NO	NO
As (V)	0.0005	0.00005	0.00028	0.00047	0.00043	0.00045	0.00098	0.85000	NO	NO	NO	NO
Al	0.156	0.121	0.67760	0.23051	0.31900	0.27191	0.36416	0.75000	NO	NO	NO	NO
Sb	0.0005	0.0005	0.00280	0.00083	0.00122	0.00101	0.00123	9.00000	NO	NO	NO	NO
Ba	0.00316	0.0113	0.06328	0.01177	0.02196	0.01654	0.01237	NA				
Be	0.0005	0.0005	0.00280	0.00083	0.00122	0.00101	0.00123	0.13000	NO	NO	NO	NO
Cd	0.00005	0.00005	0.00028	0.00008	0.00012	0.00010	0.00012	0.00068	NO	NO	NO	NO
Ca	12.4	21.6	120.96000	27.90857	46.32500	36.52444	35.65600	NA				
Cr (Total)	0.0005	0.002	0.00935	0.00176	0.00327	0.00247	0.00189	0.60100	NO	NO	NO	NO
Co	0.0005	0.0005	0.00280	0.00083	0.00122	0.00101	0.00123	NA				
Cu	0.00107	0.00935	0.05236	0.00840	0.01710	0.01247	0.00727	0.00507	YES	YES	YES	YES
Fe	0.22	0.376	2.10560	0.48937	0.80925	0.63902	0.62856	2.00000	NO	NO	NO	NO
Pb	0.0001	0.0008	0.00448	0.00073	0.00147	0.00107	0.00064	0.01538	NO	NO	NO	NO
Mg	11.5	5.0	28.11200	13.87314	16.69125	15.19156	24.66120	NA				
Mn	0.0279	0.183	1.02480	0.17031	0.33943	0.24943	0.15549	NA				
Ni	0.00054	0.0022	0.01210	0.00219	0.00415	0.00311	0.00224	0.46173	NO	NO	NO	NO
K	3.73	17.9	100.24000	17.51714	33.88938	25.17667	17.11100	NA				
Se	0.00354	0.003	0.01523	0.00521	0.00719	0.00614	0.00825	0.26000	NO	NO	NO	NO
Ag	0.00037	0.0044	0.02475	0.00385	0.00799	0.00579	0.00318	0.00041	YES	YES	YES	YES
Na	87.5	50.7	283.92000	115.56000	148.88125	131.14889	194.64200	NA				
Tl	0.0005	0.0005	0.00280	0.00083	0.00122	0.00101	0.00123	1.40000	NO	NO	NO	NO
V	0.0025	0.0025	0.01400	0.00414	0.00609	0.00506	0.00615	NA				
Zn	0.001	0.024	0.13496	0.02014	0.04286	0.03077	0.01540	0.03798	NO	YES	NO	NO
Hg	0.000005	0.00003	0.00014	0.00002	0.00005	0.00004	0.00002	0.00240	NO	NO	NO	NO
CN (Total)	0.0025	0.0276	0.15456	0.02422	0.05002	0.03629	0.02021	0.02200	YES	YES	YES	NO

Note:

Dilution for 1 hour	7
Dilution for 4 day 1.5 m	3.2
Dilution for 4 day 3.0 m	4.5
Dilution for 4 day 86 m	10

*Background exceeds CCC

Bold type indicated use of ½ the detection limit.

Table 4: 4-Day Chronic Reasonable Potential Analysis Results

Parameter	Background Concentration mg/l	Highest Concentration mg/l	Maximum Effluent Concentration Multiplier = 5.6	Maximum Concentration at 15 meters mg/l	Maximum Concentration at 30 meters mg/l	Maximum Concentration at 2,500 meters mg/l	4 Day (CCC) mg/l	Reasonable Potential to Exceed Water Quality Standards		
								15 meters	30 meters	2,500 meters
Cr (III)	0.0005	0.002	0.01120	0.00184	0.00169	0.00104	0.06975	NO	NO	NO
Cr (VI)	0.00013	0.00042	0.00235	0.00041	0.00038	0.00024	0.01100	NO	NO	NO
As (III)	0.0005	0.0005	0.00280	0.00079	0.00076	0.00062	0.19000	NO	NO	NO
As (V)	0.0005	0.00005	0.00028	0.00047	0.00048	0.00049	0.04800	NO	NO	NO
Al	0.156	0.121	0.67760	0.22120	0.21396	0.18208	0.08700	YES*	YES*	YES*
Sb	0.0005	0.0005	0.00280	0.00079	0.00076	0.00062	1.60000	NO	NO	NO
Ba	0.00318	0.0113	0.06328	0.01069	0.00986	0.00619	NA			
Be	0.0005	0.0005	0.00280	0.00079	0.00076	0.00062	0.00530	NO	NO	NO
Cd	0.00005	0.00005	0.00028	0.00008	0.00008	0.00006	0.00040	NO	NO	NO
Ca	12.4	21.6	120.96000	25.97000	24.46222	17.82800	NA			
Cr (Total)	0.0005	0.002	0.00935	0.00161	0.00148	0.00094	0.08100	NO	NO	NO
Co	0.0005	0.0005	0.00280	0.00079	0.00076	0.00062	NA			
Cu	0.00107	0.00935	0.05236	0.00748	0.00677	0.00363	0.00380	YES	YES	NO
Fe	0.22	0.376	2.10560	0.45570	0.42951	0.31428	1.00000	NO	NO	NO
Pb	0.0001	0.0008	0.00448	0.00065	0.00059	0.00032	0.00060	YES	NO	NO
Mg	11.5	5.0	28.11200	13.57650	13.34578	12.33060	NA			
Mn	0.0279	0.183	1.02480	0.15251	0.13867	0.07775	NA			
Ni	0.00054	0.0022	0.01210	0.00198	0.00182	0.00112	0.05128	NO	NO	NO
K	3.73	17.9	100.24000	15.79375	14.45333	8.55550	NA			
Se	0.00354	0.003	0.01523	0.00500	0.00484	0.00412	0.03500	NO	NO	NO
Ag	0.00037	0.0044	0.02475	0.00342	0.00308	0.00159	0.00012	YES*	YES*	YES*
Na	87.5	50.7	283.92000	112.05250	109.32444	97.32100	NA			
Tl	0.0005	0.0005	0.00280	0.00079	0.00076	0.00062	0.04000	NO	NO	NO
V	0.0025	0.0025	0.01400	0.00394	0.00378	0.00308	NA			
Zn	0.001	0.024	0.13496	0.01775	0.01588	0.00770	0.03440	NO	NO	NO
Hg	0.000005	0.00003	0.00014	0.00002	0.00002	0.00001	0.00001	YES	YES	NO
CN (Total)	0.0025	0.0276	0.15456	0.02151	0.01940	0.01010	0.00520	YES	YES	YES

Note:

Dilution at 15 meters 8
Dilution at 30 meters 9
Dilution at 2,500 meters 20
*Background exceeds CCC
Bold type indicated use of ½ the detection limit.

Table 5: 7-Day Chronic Reasonable Potential Analysis Results

Parameter	Background Concentration mg/l	Highest Concentration mg/l	Maximum Effluent Concentration Multiplier = 5.8	Maximum Concentration at 15 meters mg/l	Maximum Concentration at 30 meters mg/l	Maximum Concentration at 370 meters mg/l	4 Day (CCC) mg/l	Reasonable Potential to Exceed Water Quality Standards		
								15 meters	30 meters	370 meters
Cr (III)	0.0005	0.002	0.01120	0.00169	0.00169	0.00104	0.06875	NO	NO	NO
Cr (VI)	0.00013	0.00042	0.00235	0.00038	0.00038	0.00024	0.01100	NO	NO	NO
As (III)	0.0005	0.0005	0.00280	0.00076	0.00076	0.00062	0.19000	NO	NO	NO
As (V)	0.0005	0.00005	0.00028	0.00048	0.00048	0.00049	0.04800	NO	NO	NO
Al	0.158	0.121	0.67760	0.21396	0.21396	0.18208	0.08700	YES*	YES*	YES*
Sb	0.0005	0.0005	0.00280	0.00076	0.00076	0.00062	1.60000	NO	NO	NO
Ba	0.00318	0.0113	0.06328	0.00986	0.00986	0.00619	NA			
Be	0.0005	0.0005	0.00280	0.00076	0.00076	0.00062	0.00530	NO	NO	NO
Cd	0.00005	0.00005	0.00028	0.00008	0.00008	0.00006	0.00040	NO	NO	NO
Ca	12.4	21.8	120.96000	24.46222	24.46222	17.82800	NA			
Cr (Total)	0.0005	0.002	0.00935	0.00148	0.00148	0.00094	0.08100	NO	NO	NO
Co	0.0005	0.0005	0.00280	0.00076	0.00076	0.00062	NA			
Cu	0.00107	0.00935	0.05236	0.00677	0.00677	0.00363	0.00380	YES	YES	NO
Fe	0.22	0.376	2.10560	0.42951	0.42951	0.31428	1.00000	NO	NO	NO
Pb	0.0001	0.0008	0.00448	0.00059	0.00059	0.00032	0.00060	NO	NO	NO
Mg	11.5	5.0	28.11200	13.34578	13.34578	12.33060	NA			
Mn	0.0279	0.183	1.02480	0.13867	0.13867	0.07775	NA			
Ni	0.00054	0.0022	0.01210	0.00182	0.00182	0.00112	0.05126	NO	NO	NO
K	3.73	17.9	100.24000	14.45333	14.45333	8.55550	NA			
Se	0.00354	0.003	0.01523	0.00484	0.00484	0.00412	0.03500	NO	NO	NO
Ag	0.00037	0.0044	0.02475	0.00308	0.00308	0.00159	0.00012	YES*	YES*	YES*
Na	87.5	50.7	283.92000	109.32444	109.32444	97.32100	NA			
Tl	0.0005	0.0005	0.00280	0.00076	0.00076	0.00062	0.04000	NO	NO	NO
V	0.0025	0.0025	0.01400	0.00378	0.00378	0.00308	NA			
Zn	0.001	0.024	0.13498	0.01588	0.01588	0.00770	0.03440	NO	NO	NO
Hg	0.000005	0.00003	0.00014	0.00002	0.00002	0.00001	0.00001	YES	YES	NO
CN (Total)	0.0025	0.0276	0.15456	0.01940	0.01940	0.01010	0.00520	YES	YES	YES

Note:

Dilution at 15 meters 9
Dilution at 30 meters 9
Dilution at 370 meters 20
*Background exceeds CCC
Bold type indicated use of ½ the detection limit.

Table 6: Summary of Water Quality Analysis Results

Trask River Upstream

Date Sampled	Cr (III)	Cr (VI)	As (III)	As (V)	Al	Sb	Ba	Be	Cd	Ca	Cr	Co	Cu	Fe	Pb	Mg	Mn	Ni	K	Se	Ag	Na	Tl	V	Zn	Hg	CN (Total)
Units	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l
08/13/02	<0.001	0.00013	<0.001	<0.001	0.156	<0.001	0.0032	<0.001	<0.0001	12.4	<0.001	<0.001	0.00107	0.166	<0.0002	11.5	0.0279	0.0005	3.73	0.004	0.0004	87.5	<0.001	<0.005	<0.00200	<0.00001	<0.005
08/27/02	<0.001	0.00013	<0.001	<0.001	0.138	<0.001	0.0021	<0.001	<0.0001	11.8	<0.001	<0.001	0.00037	0.220	<0.0002	10.2	0.0202	<0.0002	3.10	0.003	<0.0001	77.2	<0.001	<0.005	<0.00200	<0.00001	<0.005
09/17/02	<0.001	0.10000	<0.001	<0.001	0.240	<0.001	0.0045	<0.001	<0.0001	15.3	<0.001	<0.001	0.00112	0.330	<0.0002	17.3	0.0620	0.0031	5.55	<0.001	<0.0001	143	<0.001	<0.005	<0.00200	<0.00001	<0.005

Wastewater Effluent

Date Sampled	Cr (III)	Cr (VI)	As (III)	As (V)	Al	Sb	Ba	Be	Cd	Ca	Cr	Co	Cu	Fe	Pb	Mg	Mn	Ni	K	Se	Ag	Na	Tl	V	Zn	Hg	CN (Total)
Units	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l
08/13/02	0.002	0.00008	<0.001	<0.001	0.118	<0.001	0.0077	<0.001	<0.0001	14.5	0.002	<0.001	0.00934	0.363	0.0008	0.4	0.183	0.0022	10.9	0.003	0.0044	50.7	<0.001	<0.005	0.023	0.00002	<0.005
08/27/02	<0.001	<0.00001	<0.001	<0.001	0.106	<0.001	0.0056	<0.001	<0.0001	14.9	<0.001	<0.001	0.00935	0.376	0.0008	4.3	0.098	0.0017	12	0.002	0.0013	49.6	<0.001	<0.005	0.020	0.00002	0.0086
09/17/02	<0.001	0.00042	<0.001	<0.001	0.121	<0.001	0.0113	<0.001	<0.0001	21.6	<0.001	<0.001	0.00740	0.297	0.0006	5.0	0.137	0.0018	17.9	0.002	0.0012	46.7	<0.001	<0.005	0.024	0.00003	0.0276

Notes:

Sampling and analysis done by using ultra clean analysis methods.

09/17/02 background data not used in RPA because of possible effluent entrainment.

Table 7: Reasonable Potential Analysis for Ammonia

Parameter	Background Concentration mg/l	Highest Concentration mg/l	Maximum Effluent Concentration Multiplier = 2.5	Maximum Concentration at ZID (4 Day 1.5 m) mg/l	Maximum Concentration at ZID (4 Day 3.0 m) mg/l	Maximum Concentration at MZ (4 Day 15 m) mg/l	Maximum Concentration at MZ (4 Day 30m) mg/l	1 Hour (CMC) mg/l	4 Day (CCC) mg/l	Reasonable Potential			
										1.5 m	3.0 m	15 m	30 m
Ammonia @ 14.1 C	0.06	14.5	36.25	11.44	8.08	4.56	4.15	13.97	1.80	NO	NO	YES	YES
Ammonia @ 20 C	0.06	14.5	36.25	11.44	8.08	4.56	4.15	13.57	1.23	NO	NO	YES	YES

Notes:

Dilution at 1.5 m	3.2
Dilution at 3.0 m	4.5
Dilution at 15 m	8
Dilution at 30 m	9

Table 8: Temperature at the Edge of the Mixing Zone

CORMIX Runs	Ambient Temperature	Effluent Temperature		Dilution Factor	Edge of Mixing Zone Temperature		Difference in Ambient Temperature	
	Average (C)	Average (C)	Maximum (C)		Average (C)	Maximum (C)	Average (C)	Maximum (C)
4 Day Ambient Flow	20.00	21.31	22.86					
15 meters				8	20.16	20.36	**0.16**	**0.36**
30 meters				9	20.15	20.32	**0.15**	**0.32**
2500 meters				20	20.07	20.14	0.07	0.14
7 Day Ambient Flow	20.00	21.35	22.70					
15 meters				9	20.15	20.30	**0.15**	**0.30**
30 meters				9	20.15	20.30	**0.15**	**0.30**
370 meters				20	20.07	20.14	0.07	0.14

Note:

No Measurable Increase = > 0.14 C of the ambient temperature.

Table 9: Fish Species Potentially Present in Lower Trask River Estuary

Family, Genus, and Species	Common Name
Petromyzontidae	
Entosphenus tridentatus	Pacific Lamprey
Rajidae	
Raja binoculata	Big Skate
Raja rhina	Longnose Skate
Acipenseridae	
Acipenser medirostris	Green Sturgeon
Clupeidae	
Alosa sapidissima	American Shad
Clupea harengus pallasi	Pacific Herring
Engraulidae	
Engraulis mordax	Northern Anchovy
Salmonidae	
Oncorhynchus tshawytscha	Chinook Salmon
Oncorhynchus clarki	Cutthroat Trout
Oncorhynchus kisutch	Coho Salmon
Oncorhynchus keta	Chum Salmon
Oncorhynchus mykiss	Steelhead Trout
Osmeridae	
Hypomesus pretiosus	Surf Smelt
Spirinchus thaleichthys	Longfin Smelt
Gadidae	
Microgadus proximus	Pacific Tomcod
Zoarcidae	
Lycodes sp.	Eelpout (small juvenile)
Atherinidae	
Atherinops affinis	Top Smelt
Gasterosteidae	
Aulorhynchus flavidus	Tubesnout
Gasterosteus aculeatus	Threespine Stickleback
Syngnathidae	
Syngnathus griseolineatus	Bay Pipefish
Embiotocidae	
Amphistichus rhododterus	Redtail Surfperch
Phanerodon furcatus	White Seaperch
Embiotoca lateralis	Striped Seaperch
Rhacochilus vacca	Pile Perch
Cymatogaster aggregata	Shiner Perch
Hyperporsopon argenteum	Walleye Surfperch

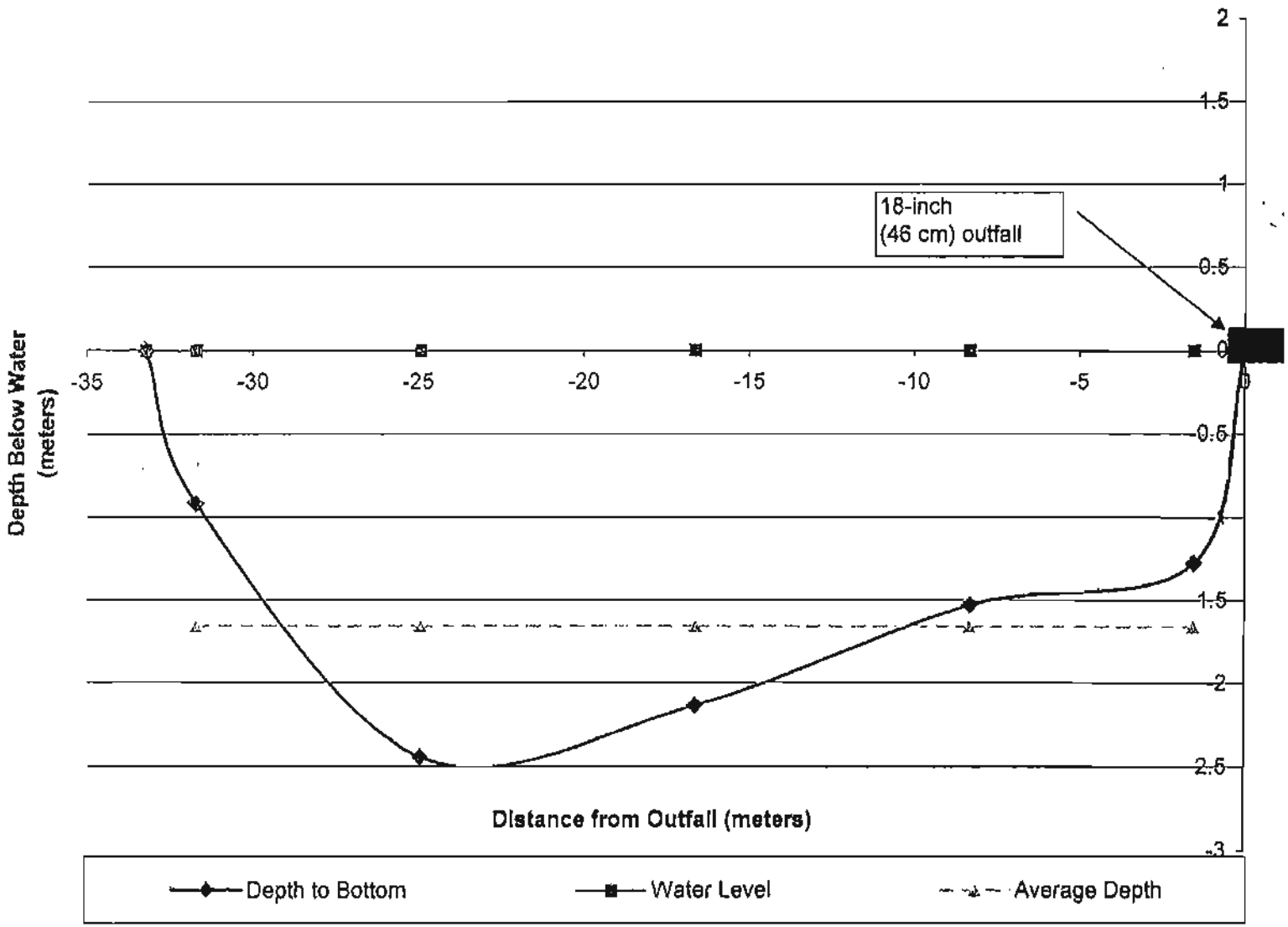
Table 9: Fish Species Potentially Present in Lower Trask River Estuary

Family, Genus, and Species	Common Name
Hyperprosopon ellipticum	Silver Surfperch
Stichaeidae	
Lumpenus sagitta	Snake Prickleback
Xiphister mucosus	Rock Prickleback
Anoplarchus purpurescens	High cockscomb
Pholidae	
Pholis ornata	Saddleback Gunnel
Xererpes fucorum	Rockweed Gunnel
Pholis schultzi	Red Gunnel
Apodichthys flavidus	Penpoint Gunnel
Anarhichadidae	
Anarrhichthys ocellatus	Wolf-eel
Ammodtidae	
Ammodytes hexapterus	Pacific Sand Lance
Gobiidae	
Clevelandia ios	Arrow Goby
Scorpaenidae	
Sebastes sp.	Rockfish sp.
Anoplopomatidae	
Anoplopoma fimbria	Sablefish
Hexagrammidae	
Hexarammos sp.	Greenling sp.
Ophodon elongatus	Lingcod
Cottidae	
Cottus asper	Prickley Sculpin
Leptocottus armatus	Pacific Staghorn Sculpin
Oligocottus maculosus	Tidepool Sculpin
Artedius fenestralis	Padded Sculpin
Artedius lateralis	Smoothhead sculpin
Hemilepidotus hemilepidotus	Red Irish Lord
Enophrys bison	Buffalo Sculpin
Scorpaenichthys marmoratus	Cabezon
Blepsias cirrhosus	Silverspotted Sculpin
Hemilepidotus hemilepidotus	Brown Irish Lord
Clinocottus acuticeps	Sharpnose sculpin
Agonidae	
Occella verrucosa	Warty Poacher
Pallasina barbata	Tubenose Poacher
Stellerina xyosterna	Pricklebreast Poacher

Table 9: Fish Species Potentially Present in Lower Trask River Estuary

Family, Genus, and Species	Common Name
Cyclopteridae	
Liparis fucensis	Slipskin snailfish
Liparis rutteri	Ringtail snailfish
Bothidae	
Clitharichthys sordidus	Pacific Sanddab
Clitharichthys stigmaeus	Speckled Sanddab
Pleuronectidae	
Parophrys vetulus	English Sole
Platichthys stellatus	Starry Flounder
Psettichthys melanostictus	Sand Sole
Isopsetta isolepis	Butter Sole

Figure 1. Profile of Trask River at Tillamook WWTP Outfall



City of Tillamook
Discharge Monitoring Reports

Year	Month	Influent BOD			Influent TSS			Loading (lb/d)		Influent Flow		
		Max	Wkly	Avg	Max	Wkly	Avg	BOD	TSS			
		mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	Avg	Avg	Max	Wkly	Avg
1999	Jan	91	74	59	91	90	56	1543	1465	4.66	4.42	3.14
1999	Feb	108	90	59	171	152	82	1790	2488	4.81	4.50	3.64
1999	Mar	130	92	178	561	385	227	3478	4436	4.08	3.14	2.34
1999	Apr	298	195	158	551	350	313	1464	2900	1.60	1.30	1.11
1999	May	280	207	160	372	312	293	1601	2932	2.60	1.20	1.20
1999	Jun	316	240	200	872	707	349	1284	2241	0.90	0.83	0.77
1999	Jul	387	360	270	764	541	414	1504	2306	0.80	0.74	0.67
1999	Aug	416	394	316	530	507	362	1591	1811	0.70	0.60	0.60
1999	Sep	334	311	245	372	351	134	1063	581	0.80	0.60	0.52
1999	Oct	390	318	288	709	643	556	1201	2319	0.90	0.70	0.50
1999	Nov	260	214	107	392	342	208	1976	3841	4.30	3.20	2.21
1999	Dec	197	168	34	448	399	225	766	5067	5.20	4.30	2.70
2000	Jan	147	119	98	545	433	283	1471	4248	2.40	2.20	1.80
2000	Feb	226	223	126	508	432	290	1576	3628	2.20	1.60	1.50
2000	Mar	341	271	169	781	528	266	1832	2884	1.90	1.40	1.30
2000	Apr	321	287	243	592	580	323	1621	2155	1.00	0.80	0.80
2000	May	514	381	245	546	410	283	2084	2407	1.70	1.30	1.02
2000	Jun	387	338	219	398	343	220	1626	1835	2.10	1.50	1.00
2000	Jul	438	420	356	304	275	245	1781	1226	0.80	0.67	0.60
2000	Aug	330	298	270	245	217	199	1193	880	0.60	0.53	0.53
2000	Sep	367	337	298	289	266	232	1292	1006	0.90	0.56	0.52
2000	Oct	398		264	403	363	225	1475	1257	1.20	0.77	0.67
2000	Nov	329	224	185	348	328	213	1296	1492	1.10	0.99	0.84
2000	Dec	279	240	176	394	284	256	1732	2519	2.10	1.49	1.18
2001	Jan	323	312	233	495	391	258	2332	2662	2.10	1.30	1.20
2001	Feb	571	428	252	638	503	305	2417	2925	1.40	1.30	1.15
2001	Mar	482	366	310	602	411	335	2844	3073	1.60	1.30	1.10
2001	Apr	461	375	286	408	330	288	3053	3074	1.80	1.56	1.28
2001	May	307	283	214	452	419	265	1765	2210	1.50	1.20	1.00
2001	Jun	545	409	316	593	414	317	1871	1677	0.80	0.80	0.71
2001	Jul	383	305	288	340	284	234	1288	1054	0.80	0.59	0.54
2001	Aug	454	433	334	450	392	313	1560	1482	1.20	0.73	0.56
2001	Sep	313	275	251	281	257	221	1005	885	0.60	0.50	0.48
2001	Oct	399	307	249	328	281	206	1085	898	0.80	0.57	0.52
2001	Nov	244	194	113	226	172	99	1731	1517	4.20	3.20	1.84
2001	Dec	133	104	58	121	121	61	1507	1585	4.50	3.67	3.12
2002	Jan	109	109	75	292	205	92	1801	2210	4.30	3.72	2.88
2002	Feb	106	90	77	110	98	92	1336	1596	3.40	2.80	2.08
2002	Mar	110	109	80	137	112	73	1328	1212	3.10	2.80	1.99
2002	Apr	144	116	99.6	182	121	98	1177	1158	2.50	2.06	1.42
2002	May	208	182	149	274	200	168	946	1068	1.20	0.87	0.76
2002	Jun	230	202	180	218	179	146	981	779	1.20	0.70	0.64
2002	Jul	304	256	207	280	241	185	968	866	1.10	0.68	0.56
2002	Aug	287	248	234	255	228	193	900	742	0.50	0.50	0.46
2002	Sep	315	278	235	315	273	208	907	795	0.60	0.50	0.46
2002	Oct	851	394	303	368	284	218	1112	800	0.60	0.53	0.44
2002	Nov	317	308	180	231	218	147	1021	834	1.20	0.90	0.68
2002	Dec	248	226	95	193	179	62	1641	1418	4.30	3.36	2.07
	Minimum	91	74	34	91	90	56	766	581	0.50	0.50	0.44
	Maximum	651	433	356	872	707	556	3478	5067	5.20	4.50	3.64
	Average	311	258	199	395	324	226	1563	1972	1.96	1.57	1.23
								BOD	TSS			
	Limit Dry						2001 Average	1175	1123		1.62	
	Limit Wet											

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Month	Effluent Temp (C)	Effluent pH Min	Effluent pH Max	Effluent Flow (mgd) Max	Effluent Flow (mgd) Wkly	Effluent Flow (mgd) Avg	Effluent BOD mg/l wkly	Effluent BOD mg/l Avg	Effluent BOD load (lb/d) Max	Effluent BOD load (lb/d) wkly	Effluent BOD load (lb/d) Avg	Effluent CBOD mg/l wkly	Effluent CBOD mg/l Avg	Effluent TSS mg/l wkly	Effluent TSS mg/l Avg	Effluent TSS load (lb/d) Max	Effluent TSS load (lb/d) wkly	Effluent TSS load (lb/d) Avg
Jan-99	13.5	5.4	6.3	4.7	4.4	3.1	13	11	457	423	257	11	9	14	10	582	348	228
Feb-99	13.0	5.4	6.0	4.8	4.5	3.6	13	11	456	409	330	11	8	14	12	601	548	369
Mar-99	13.8	5.6	6.1	4.1	3.1	2.3	11	9	245	195	172	10	8	14	12	349	324	227
Apr-99	15.0	6.0	6.3	1.8	1.4	1.2	15	13	174	133	106	12	10	15	14	200	183	125
May-99	15.7	6.0	6.3	1.7	1.5	1.3	15	12	192	170	136	13	10	16	14	206	183	158
Jun-99	17.3	6.2	6.4	1.1	0.9	0.8	35	25	287	261	175	32	22	21	18	180	156	131
Jul-99	17.0	6.3	6.5	1.1	1.1	0.9	40	30	429	289	255	38	30	33	25	271	220	165
Aug-99	20.0	6.3	6.6	1.0	1.0	0.9	25	20	196	194	151	16	16	16	14	135	124	101
Sep-99	20.0	6.4	6.8	0.9	0.8	0.8	17	15	130	122	100	16	12	17	11	130	119	77
Oct-99	18.5	6.2	6.8	1.2	1.0	0.9	30	18	299	201	128	28	17	15	13	133	126	90
Nov-99	15.9	5.8	6.4	5.2	4.1	2.7	23	16	371	352	285	26	17	38	19	647	598	341
Dec-99	14.0	5.4	6.2	5.7	5.1	3.5	15	10	448	366	266	15	11	20	14	772	658	439
Jan-00	13.0	5.6	6.4	3.1	2.9	2.0	12	11	304	299	236	16	13	13	12	405	395	251
Feb-00	13.8	5.5	6.2	3.0	2.2	2.0	19	13	301	273	230	18	12	14	12	353	323	235
Mar-00	13.7	5.8	6.2	2.2	1.6	1.7	23	14	323	307	197	25	14	18	11	256	246	159
Apr-00	15.8	6.0	6.7	1.4	1.3	1.3	28	20	261	187	123	18	11	12	10	130	117	103
May-00	16.2	6.0	6.1	2.3	1.8	1.4	18	15	386	307	162	19	14	20	14	449	343	177
Jun-00	18.0	6.0	6.7	2.4	1.8	1.4	30	23	715	460	276	30	20	20	15	302	234	178
Jul-00	19.6	6.3	6.7	1.2	1.2	1.1	17	14	149	142	125	16	13	12	11	117	103	93
Aug-00	20.4	6.5	6.7	1.1	1.0	1.0	43	25	377	368	203	40	23	32	18	308	270	146
Sep-00	20.6	6.5	6.7	1.3	1.0	1.0	17	15	147	137	115	16	14	14	12	119	108	32
Oct-00	18.8	6.3	6.8	1.5	1.2	1.1	17	15	235	154	132	19	16	17	13	176	158	114
Nov-00	11.2	6.3	6.8	1.4	1.5	1.2	30	22	355	283	219	29	21	23	18	284	257	186
Dec-00	14.6	6.0	6.7	2.1	1.5	1.2	20	15	187	181	135	16	15	19	15	269	222	140
Jan-01	13.8	6.2	6.7	1.7	1.6	1.5	35	19	390	347	199	34	18	22	16	239	221	170
Feb-01	14.0	6.2	6.6	1.7	1.6	1.5	22	16	192	184	151	22	16	21	17	257	205	199
Mar-01	15.0	6.1	6.6	1.8	1.6	1.5	18	15	238	218	146	21	17	29	21	425	353	200
Apr-01	15.4	6.1	6.4	1.9	1.8	1.6	37	24	655	505	302	35	24	37	23	585	541	270
May-01	17.0	6.2	6.5	1.8	1.5	1.4	10	16	228	217	139	19	16	17	12	225	200	115
Jun-01	18.0	6.1	6.6	1.3	1.2	1.2	18	15	107	102	87	19	15	16	13	99	94	74
Jul-01	19.7	6.5	6.8	1.1	1.1	1.0	24	21	110	100	92	19	18	13	12	63	54	53
Aug-01	20.9	6.8	6.9	1.4	1.1	1.0	33	25	440	280	136	32	23	31	20	400	255	113
Sep-01	20.1	6.5	7.0	1.0	1.0	0.9	30	22	129	123	91	26	19	18.5	15	96	77	62
Oct-01	18.4	6.3	6.9	1.1	0.9	0.9	21	17	127	121	80	18	14	14.5	13	85	83	58
Nov-01	16.2	5.8	6.7	4.5	3.6	2.1	31	20	618	568	366	24.5	16.6	28	20	642	524	369
Dec-01	13.4	5.8	6.8	4.9	4.0	3.5	23	15	480	440	369	11	8	15	12	547	444	335
Jan-02	13.0	6.0	6.7	4.6	2.9	3.5	22	14	432	390	318	17	12	15	13	494	421	301
Feb-02	13.6	6.0	6.4	3.9	3.3	2.5	14	11	338	284	176	11	9	14	10	360	286	165
Mar-02	13.4	6.4	7.1	3.7	3.4	2.5	16	15	400	394	267	14	12	12	11	325	297	196
Apr-02	15.2	6.4	6.9	3.0	2.5	1.9	30	23	605	505	298	25	18	23	18	396	336	218
May-02	16.4	6.4	7.0	1.6	1.3	1.2	20	14	260	183	93	16	11	19	12	260	172	85
Jun-02	18.9	6.6	7.0	1.5	1.1	1.1	17	13	111	91	67	13	10	16	12	99	85	61
Jul-02	20.6	6.5	7.1	1.1	1.1	1.0	20	18	99	96	83	18	15	16	15	93	88	67
Aug-02	21.1	6.4	7.1	1.0	0.9	0.9	23	21	92	91	79	19.5	18	16	14	75	66	54
Sep-02	20.6	6.4	7.7	1.0	1.0	0.9	22	19	88	76	73	19	16	17	12	75	51	46
Oct-02	19.8	6.7	7.1	1.0	0.9	0.8	18	17	79	75	62	16	14	14	11	63	57	52
Nov-02		6.3	7.2	1.4	1.2	1.0												
Dec-02		5.8	7.0	4.7	3.8	2.5												
Minimum	11.2	5.4	6.0	0.9	0.8	0.8	10	9	79	75	62	10	8	12	10	63	51	32
Maximum	21.1	6.7	7.7	5.7	5.1	3.6	43	30	715	568	369	40	30	38	25	772	658	439
Average	16.6	6.1	6.7	2.3	1.9	1.6	22	17	294	252	179	20	15	19	14	289	245	165
Limit Dry		6.0	9.0			1.1	30	20	354	265	177			30	20	354	265	177
Limit Wet		6.0	9.0				45	30	530	398	265			45	30	530	398	265

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Month	Total Chlorine Res. mg/l		Total Rainfall Inches			Coliform MPN		Percent Removal		Pumped	Hauled	% VSS
	Max	Avg	Max	Total	Avg	wkly	Avg	BOD	TSS	per day	per mo.	Red.
Jan-99	0.02	0.004	2.3	17.05	0.55	128	44	81.4	82.1	1040		
Feb-99	0.02	0.006	3.04	22.68	0.81	105	31	81.4	85.4	1300		70
Mar-99	0.02	0.006	1.55	9.92	0.32	63	12.7	94.9	94.7	1500		65
Apr-99	0.02	0.006	0.67	3.3	0.11	121	6	91.8	95.5	1954		
May-99	0.02	0.005	1.45	7.13	0.23	29	8.2	92.5	95.2	1287	74000	55.5
Jun-99	0.02	0.005	0.66	3.3	0.11	11	8	87.5	94.8	1543		
Jul-99			0.49	1.46	0.047			88.9	94.0	1855		
Aug-99	0.02	0.006	0.41	1.71	0.06	2.6	2	93.7	96.1	3023	132000	
Sep-99	0.02	0.005	0.07	0.3	0.01	23	3.5	93.9	91.8	2966	164000	
Oct-99	0.02	0.008	1	4.03	0.13	38	3.3	93.8	97.7	2928	40000	36
Nov-99	0.02	0.006	3.98	25.5	0.85	61	24	85.0	90.9	2452		
Dec-99	0.02	0.007	2.23	17.05	0.55	107	60	70.8	93.8	1284		67
Jan-00	0.02	0.005	1.35	12.09	0.39	76	51	88.8	95.8	1400		66
Feb-00	0.02	0.005	1.55	8.96	0.32	75	41	89.7	95.9	1400		58
Mar-00	0.02	0.003	0.88	5.58	0.18	32	16	91.7	95.9	1515		76
Apr-00			0.72	4.08	0.14			91.8	96.9	1163	25200	
May-00	0.02	0.006	1.4	7.03	0.23	20	7.2	93.9	95.1	1906	57600	66
Jun-00	0.02	0.007	1.53	3.9	0.13	57	37	89.5	93.2	2059		66
Jul-00	0.02	0.006	0.3	0.62	0.02	2.4	1.8	96.1	95.5	1589	54000	
Aug-00	0.02	0.004	0.039	0.59	0.1	2.5	1.4	90.7	91.0	2138	52200	60
Sep-00	0.02	0.005	1.1	3	0.1	11	5	95.0	94.8	2377	115600	48
Oct-00	0.02	0.004	1.25	6.2	0.2	18	9	94.3	94.2	2552		
Nov-00			0.68	4.72	0.18			88.1	91.5	2342	36500	76
Dec-00			2.61	6.86	0.22			91.5	94.1	2014		63
Jan-01	0.02	0.006	1.07	6.2	0.2	48	9	91.8	94.0			
Feb-01	0.02	0.01	0.85	3.92	0.14	10	6.7	93.7	94.4			
Mar-01	0.02	0.006	0.9	8.06	0.26	49	11	95.2	93.7	2165		33
Apr-01	0.02	0.007	1.14	8.06	0.26	40	27	91.6	92.0	1430	36000	43.2
May-01	0.02	0.005	1.17	3.41	0.11	74	84	92.5	95.5	1423	109400	64
Jun-01	0.02	0.004	0.7	3.3	0.11	8	3.3	95.3	95.9	2077		69
Jul-01	0.02	0.003	0.5	1.24	0.04	12	3.5	92.7	94.9	1793		54
Aug-01	0.02	0.001	2.2	3.41	0.11	44	3.7	92.5	93.6	2579		83
Sep-01	0.02	0.002	0.53	1.05	0.035	7.5	5.4	91.2	93.2		23000	42
Oct-01	1	0.035	1.1	5.58	0.18	61	5.4	93.2	93.7	2603	1800	66
Nov-01	0.02	0.0023	1.89	15.3	0.51	155	16.19	82.3	79.8	2026		58
Dec-01	0.02	0.0048	1.85	16.43	0.53	58	17.18	74.1	80.3	2176		
Jan-02	1	0.066	2.95	18.91	0.61	86.15	24	81.3	85.9	2213		57
Feb-02	0.02	0.001	1.02	6.72	0.24	26	8.97	85.7	89.1	1811		
Mar-02	0.01	0.001	2.75	10.54	0.34	29.5	15.3	81.3	84.9	1629		54
Apr-02	0.02	0.002	1.24	6.9	0.23	27	13	78.9	81.8	1670		61
May-02	0.02	0.0006	0.68	2.79	0.09	11.49	8.9	90.6	92.9	2523		
Jun-02	0.01	0.0003	0.97	2.7	0.09	29	8.4	92.8	91.6	2503	41250	
Jul-02	0.02	0.0006	0.27	0.31	0.01	6.9	6.3	91.3	91.9	2707	162000	71
Aug-02	0	0	0.18	0.186	0.006	15	9	91.0	92.7	2227	69000	53
Sep-02	0	0	0.51	1.458	0.0486	8.1	7.2	91.9	94.2	2218	29500	45
Oct-02	0	0	1.52	1.55	0.05	9	7	94.4	95.0	2443	66800	38
Nov-02			1.19	7.76	0.259							
Dec-02			2.03	16.35	0.527							
Minimum	0.00	0.00	0.04		0.01	2	1	71	80	1040	1800	33
Maximum	1.00	0.07	3.98		0.85	155	84	96	98	3023	164000	76
Average	0.08	0.01	1.26		0.23	43	16	90	92	1998	67887	60
										85901	1289850	
Limit Dry	0.02	0.01				200	49	85	85			
Limit Wet	0.02	0.01				400	200	85	85			

358494 TILLAMOOK, OR

LAT: 45 27N (DEG MIN) LONG: 123 52W (DEG MIN)

PERIOD: 1951-80

FREEZE DATA

PROBABILITY OF LATER DATE IN SPRING (THRU JULY 31) THAN INDICATED(*)

TEMP (F)	.90	.80	.70	.60	.50	.40	.30	.20	.10
	SPRING FREEZE DATES (MO/DAY)								
36	5/14	5/24	6/01	6/07	6/13	6/19	6/25	7/03	7/13
32	4/24	4/29	5/04	5/07	5/11	5/14	5/18	5/22	5/28
28	3/11	3/21	3/27	4/02	4/08	4/13	4/19	4/26	5/05
24	12/30	1/17	1/29	2/07	2/16	2/24	3/05	3/16	3/30
20	0/00	0/00	0/00	0/00	12/29	1/14	1/26	2/06	2/19
16	0/00	0/00	0/00	0/00	0/00	0/00	1/06	1/21	2/05

PROBABILITY OF EARLIER DATE IN FALL (BEGINNING AUG 1) THAN INDICATED(*)

TEMP (F)	.10	.20	.30	.40	.50	.60	.70	.80	.90
	FALL FREEZE DATES (MO/DAY)								
36	8/23	9/01	9/07	9/12	9/17	9/22	9/27	10/03	10/11
32	9/22	9/29	10/04	10/09	10/13	10/17	10/21	10/26	11/03
28	10/12	10/22	10/30	11/05	11/11	11/17	11/23	11/30	12/11
24	11/05	11/15	11/22	11/28	12/04	12/10	12/17	12/25	1/06
20	11/26	12/12	12/26	1/10	2/04	0/00	0/00	0/00	0/00
16	12/31	1/14	1/27	0/00	0/00	0/00	0/00	0/00	0/00

PROBABILITY OF LONGER THAN INDICATED FREEZE FREE PERIOD (DAYS)

TEMP (F)	.10	.20	.30	.40	.50	.60	.70	.80	.90
	FREEZE FREE PERIOD								
36	136	122	112	103	095	087	078	068	054
32	182	173	166	160	154	149	143	136	126
28	260	245	234	225	216	208	198	188	173
24	>365	324	306	294	285	275	266	255	241
20	>365	>365	>365	>365	>365	>365	>365	>365	320
16	>365	>365	>365	>365	>365	>365	>365	>365	348

(*)PROBABILITY OF OBSERVING A TEMPERATURE AS COLD, OR COLDER, LATER IN THE SPRING OR EARLIER IN THE FALL THAN THE INDICATED DATE. 0/00 INDICATES THAT THE PROBABILITY OF OCCURRENCE OF THRESHOLD TEMPERATURE IS LESS THAN INDICATED PROBABILITY.

GROWING DEGREE UNITS TO SELECTED BASE TEMPERATURES (F)

BASE		JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
		GROWING DEGREE UNITS												
40	M	127	148	162	231	370	474	569	588	534	404	224	153	3984
	S	127	275	437	668	1038	1512	2081	2669	3203	3607	3831	3984	
45	M	47	56	55	99	216	324	414	433	384	251	105	56	2440
	S	47	103	158	257	473	797	1211	1644	2028	2279	2384	2440	
50	M	7	13	11	24	85	176	259	278	236	119	30	10	1248
	S	7	20	31	55	140	316	575	853	1089	1208	1238	1248	
55	M	0	2	1	4	19	58	114	131	105	37	3	0	474
	S	0	2	3	7	26	84	198	329	434	471	474	474	
60	M	0	0	0	1	2	9	23	29	27	8	0	0	99
	S	0	0	0	1	3	12	35	64	91	99	99	99	

M = MONTHLY DATA S = SUM OF MONTHLY DATA

		JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
		GROWING DEGREE UNITS FOR CORN												
CORN	M	34	56	69	108	178	237	303	320	301	217	93	42	1959
	S	34	90	159	267	445	682	985	1305	1606	1823	1916	1958	

NOTE: FOR CORN THE BASE IS 50, AND THE DEGREE UNITS ARE ADJUSTED FOR TEMPERATURES BELOW 50 AND ABOVE 86

OTHER CLIMATOLOGICAL DATA ARE AVAILABLE IN A VARIETY OF SUMMARIES AND FORMATS, SUCH AS THE CLIMATOGRAPHY OF THE UNITED STATES: NO. 60 - CLIMATE OF STATES: NO. 81 - MONTHLY NORMALS (AND SUPPLEMENTS: ANNUAL DEGREE DAYS TO SELECTED BASES DERIVED FROM THE 1951-80 NORMALS: AND MONTHLY PRECIPITATION PROBABILITIES, SELECTED PROBABILITY LEVELS DERIVED FROM THE 1951-80 NORMALS): NO. 84 - DAILY NORMALS: NO. 85 - DIVISIONAL NORMALS. A VARIETY OF DATA IS AVAILABLE EITHER ON MAGNETIC TAPE, MICROFICHE, OR PAPER COPY.

TO OBTAIN INFORMATION ABOUT CLIMATOLOGICAL DATA AND RELATED PUBLICATIONS, CONTACT:

DIRECTOR
NATIONAL CLIMATIC DATA CENTER
FEDERAL BUILDING
ASHEVILLE, NC 28801-2696
(OR TELEPHONE: (704) 259-0682)

DEPARTMENT OF COMMERCE
NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION
NATIONAL ENVIRONMENTAL SATELLITE, DATA, AND INFORMATION SERVICE
NATIONAL CLIMATIC DATA CENTER
ASHEVILLE, NC

USCOMM-NOAA-ASHEVILLE, NC 7/84-200

CLIMATOGRAPHY OF THE UNITED STATES NO. 20

TILLAMOOK, OR

PERIOD: 1951-80
ELEVATION: 10 FT

CLIMATOLOGICAL SUMMARY

	TEMPERATURE (F)															PRECIPITATION TOTALS (INCHES)											
	MEANS			EXTREMES						MEAN NUMBER OF DAYS				DEGREE DAYS		*	*					SNOW			MEAN NUMBER OF DAYS		
										MAX		MIN															
	DAILY MAXIMUM	DAILY MINIMUM	MONTHLY	RECORD HIGHEST	YEAR	DAY	RECORD LOWEST	YEAR	DAY	90 AND ABOVE	32 AND BELOW	32 AND BELOW	0 AND BELOW	HEATING BASE 65	COOLING BASE 65	MEAN	GREATEST MONTHLY	YEAR	GREATEST DAILY	YEAR	DAY	MEAN	MAXIMUM MONTHLY	YEAR	.10 OR MORE	.50 OR MORE	1.00 OR MORE
JAN	48.8	35.5	42.2	69+	76	30	11	74	9	0	0	11	0	707	0	14.87	29.18	53	4.65	65	28	1.2	15.0	69	18	10	5
FEB	52.4	36.8	44.6	73+	68	28	15+	79	2	0	0	8	0	571	0	10.11	22.90	61	3.06	54	19	.2	2.5	69	15	7	3
MAR	53.4	35.9	44.6	73+	69	25	18+	60	1	0	0	10	0	632	0	10.43	17.42	61	3.14	74	05	1.4	19.0	51	17	8	3
APR	56.7	38.2	47.5	83	51	11	23+	80	6	0	0	6	0	525	0	6.13	11.22	72	2.92	62	27	.0	.8	75	12	4	1
MAY	60.9	42.3	51.7	87+	53	4	26+	54	1	0	0	2	0	412	0	4.07	8.87	77	1.68	75	03	.0	.0		9	3	0
JUN	64.5	46.6	55.6	91+	70	1	31+	76	3	0	0	0	0	282	0	3.04	6.80	68	4.10	54	16	.0	.0		6	2	0
JUL	67.3	49.0	58.2	102	61	11	34+	56	30	0	0	0	0	211	0	1.29	4.87	74	2.00	74	17	.0	.0		3	1	0
AUG	68.2	49.4	58.8	93+	72	7	33+	59	9	0	0	0	0	196	0	2.00	6.53	68	1.71	53	24	.0	.0		4	1	0
SEP	68.5	46.7	57.6	97+	79	14	27+	72	27	1	0	0	0	226	0	3.73	7.80	77	2.65	71	02	.0	.0		7	3	1
OCT	63.0	42.4	52.8	88	52	1	22+	71	28	0	0	3	0	378	0	7.64	15.29	55	3.02	55	09	.0	.0		11	6	2
NOV	55.0	38.6	46.9	80+	62	1	17+	55	14	0	0	7	0	543	0	12.52	22.07	73	4.26	75	30	.1	2.0	77	16	9	4
DEC	50.4	37.0	43.7	69	60	22	4+	72	8	0	0	8	0	660	0	15.07	22.41	71	4.92	77	02	1.0	11.0	64	19	11	5
YEAR	59.1	41.5	50.4	102	JUL 61	11	4	DEC 72	8	1	0	55	0	5343	0	90.90	29.18	JAN 53	4.92	DEC 77	02	3.9	19.0	MAR 51	137	65	24

FROM 1951-80 NORMALS

* ESTIMATED VALUE BASED ON DATA FROM SURROUNDING STATIONS

\+ ALSO ON EARLIER DATES.

DEGREE DAYS TO SELECTED BASE TEMPERATURES (F)

BASE	HEATING DEGREE DAYS												
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
[illegible]	707	571	632	525	412	282	211	196	226	378	543	660	5343
[illegible]	552	431	477	375	257	146	77	73	103	223	393	505	3612
[illegible]	459	347	384	285	168	84	29	28	54	139	303	412	2692
[illegible]	402	296	322	225	112	53	10	11	29	90	247	350	2147
[illegible]	260	172	180	9[illegible]	23	9	0	0	0	17	118	211	1087

BASE	COOLING DEGREE DAYS												
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
[illegible]	0	0	0	0	10	71	110	129	107	22	0	0	449
[illegible]	0	0	0	0	0	42	66	84	72	9	0	0	273
[illegible]	0	0	0	0	0	14	22	36	31	0	0	0	103
[illegible]	0	0	0	0	0	0	0	0	0	0	0	0	0
[illegible]	0	0	0	0	0	0	0	0	0	0	0	0	0

DERIVED FROM THE 1951-80 MONTHLY NORMALS

PROBABILITY THAT THE MONTHLY PRECIPITATION WILL BE EQUAL TO OR LESS THAN THE INDICATED PRECIPITATION AMOUNT

MONTHLY PRECIPITATION (INCHES)

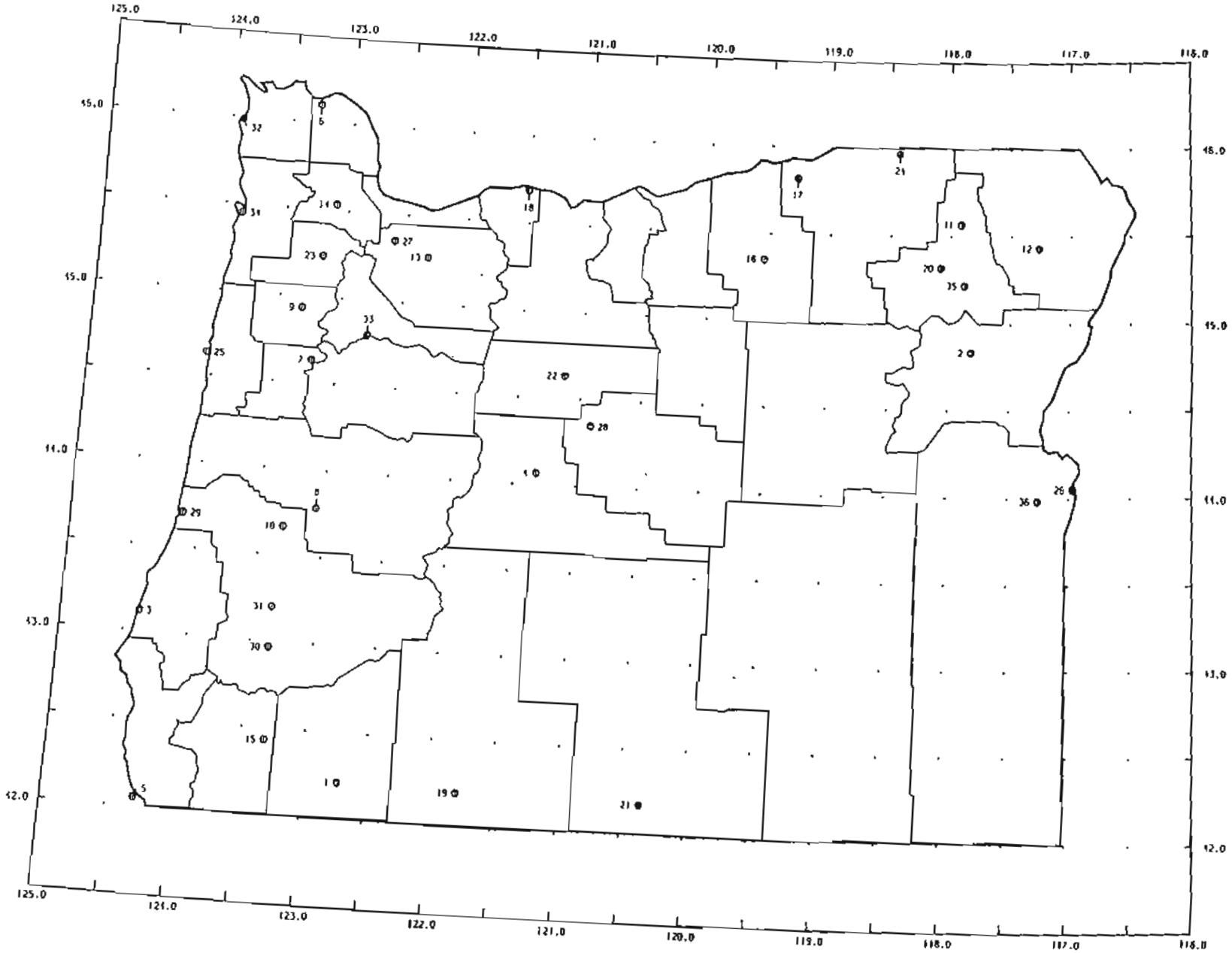
PROBABILITY LEVELS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
.05	5.04	4.78	4.23	2.51	1.57	.73	.00	.14	.53	2.05	4.72	7.03
.10	6.45	5.66	5.19	3.07	1.95	1.02	.05	.27	.84	2.77	5.89	8.34
.20	8.51	6.87	6.54	3.86	2.49	1.46	.21	.52	1.38	3.88	7.56	10.15
.30	10.25	7.84	7.66	4.51	2.93	1.85	.40	.80	1.90	4.85	8.96	11.62
.40	11.92	8.74	8.71	5.13	3.36	2.25	.61	1.11	2.44	5.80	10.28	12.98
.50	13.64	9.64	9.78	5.75	3.79	2.66	.86	1.46	3.03	6.80	11.63	14.34
.60	15.52	10.61	10.93	6.42	4.26	3.13	1.17	1.89	3.71	7.91	13.10	15.80
.70	17.71	11.71	12.26	7.20	4.80	3.68	1.56	2.43	4.55	9.22	14.80	17.47
.80	20.53	13.09	13.95	8.18	5.49	4.41	2.12	3.18	5.68	10.93	16.97	19.56
.90	24.89	15.17	16.52	9.68	6.54	5.55	3.08	4.44	7.53	13.61	20.31	22.72
.95	28.90	17.05	18.87	11.04	7.51	6.63	4.03	5.69	9.30	16.12	23.35	25.57

THESE VALUES WERE DETERMINED FROM THE INCOMPLETE GAMMA DISTRIBUTION.

CLIMATOGRAPHY OF THE UNITED STATES NO. 20
CLIMATE OF CITIES, STATE OF

OREGON

PERIOD OF RECORD : 1951-80



CLIMATOGRAPHY OF THE UNITED STATES NO. 20 (1951-80)
CITY NAME LIST, STATE OF:

OREGON

NAMES ARE IN ALPHABETICAL ORDER

✓ = name like crooked

MAP NUMBER	ST/STAN NUMBER	CITY NAME	LAT DEG	LAT MN	LONG DEG	LONG MN	ELEV (FT)
1	35 0304	ASHLAND	42	13N	122	43W	1780
2	35 0412	BAKER	44	50N	117	49W	3368
3	35 0471	BANDON	43	07N	124	23W	80
4	35 0694	BEND	44	04N	121	19W	3599
5	35 1055	BROOKINGS	42	03N	124	17W	80
6	35 1643	CLATSKANIE	46	06N	123	17W	92
7	35 1862	CORVALLIS	44	38N	123	12W	225
8	35 1897	COTTAGE GROVE	43	47N	123	04W	650
9	35 2112	DALLAS	44	56N	123	19W	325
10	35 2406	DRAIN	43	40N	123	19W	292
11	35 2597	ELGIN	45	34N	117	55W	2659
12	35 2672	ENTERPRISE	45	26N	117	16W	3790
13	35 2693	ESTACADA	45	16N	122	19W	410
14	35 2997	FOREST GROVE	45	32N	123	06W	180
15	35 3445	GRANTS PASS	42	26N	123	19W	925
16	35 3827	HEPPNER	45	21N	119	33W	1950
17	35 3847	HERMISTON	45	49N	119	17W	624
18	35 4003	HOOD RIVER	45	41N	121	31W	500
19	35 4506	KLAMATH FALLS	42	12N	121	47W	4098
20	35 4622	LA GRANDE	45	19N	118	05W	2755
21	35 4670	LAKEVIEW	42	11N	120	21W	4774
22	35 5139	MADRAS	44	38N	121	08W	2230
23	35 5384	MC MINNVILLE	45	14N	123	11W	148
24	35 5593	MILTON FREEWATER	45	58N	118	26W	839
25	35 6032	NEWPORT	44	38N	124	03W	154
26	35 6294	ONTARIO	44	03N	116	58W	2145
27	35 6334	OREGON CITY	45	21N	122	36W	167
28	35 6883	PRINEVILLE	44	21N	120	54W	2840
29	35 7082	REEDSPORT	43	42N	124	07W	60
30	35 7169	RIDDLE	42	58N	123	21W	663
31	35 7331	ROSEBURG	43	12N	123	21W	465
32	35 7641	SEASIDE	45	59N	123	55W	10
33	35 8095	STAYTON	44	48N	122	46W	465
34	35 8494	TILLAMOOK	45	27N	123	52W	10
35	35 8746	UNION	45	13N	117	53W	2765
36	35 8797	VALE	43	59N	117	15W	2240

(751
2709
5429

CLIMATOGRAPHY OF THE UNITED STATES NO. 20

Tables	Issue 1975	Issue 1985
Climatological Summary (Clim Sum) Means and Extremes for the Period	X	X
Freeze Probability Data	X	X
Precipitation Probabilities	X	X
Maximum Temperature, MO/YR	X	
Minimum Temperature, MO/YR	X	
Average Temperature, MO/YR	X	
Monthly/Annual Normals	1941-70	1951-80
Temperature		in Clim Sum
Precipitation		in Clim Sum
Heating and Cooling Degree Days (base 65°F)		in Clim Sum
Total Precipitation, MO/YR	X	
Total Snowfall, MO/YR	X	
Degree Days to Selected Base Temperatures		X
Growing Degree Units to Selected Base Temperatures		X
Growing Degree Units for Corn		X

INTRODUCTION

This issue (1985) of Climatography of the United States No. 20 summarizes observations from the 30-year period 1951 through 1980, for 1,879 locations. The locations were chosen on the basis of having a population of at least 5,000 persons. The previous issue (1975) was for sites which were selected on the basis of statewide location, without regard for population size. That issue was for 1,063 locations, with each summary based on at least a 20-year period-of-record from the overall period 1951 through 1975. There are 633 locations common to both issues. The contents of this issue of No. 20 differ from those previously issued; the change in content reflects the growing need by climatological data users for data which can be used when solving economic and social problems at the local level; for example, Growing Degree Units to Selected Base Temperatures. The following table shows the differences in content for the 1985 and 1975 issues. The deleted monthly temperature and precipitation values can be found in other National Climatic Data Center (NCDC) publications.

This publication contains the statewide collection of No. 20s from the current issue. The site locations are given on the state map, which is keyed to an alphabetically ordered station list. For comparative purposes, a similar map and station list for the 1975 issue are also included, following the climatological data. Detailed information about the observation sites and previously issued No. 20s can be obtained from the NCDC. The temperature data used in the construction of the Freeze Data and Growing Degree Units tables were extracted from a validated serially-complete data base (which also can be obtained from the NCDC, on magnetic tape). Previous Freeze Data tables were compiled from data extracted from the Climatological Data - Annual Summary publication. Also, for this issue, the base dates for the calculation of Freeze Date Probabilities were changed to July 31 and August 1 because they better represent the seasonal transition for many northern and mountainous sections of the United States. Further information about the calculation of the current Freeze Data and Growing Degree Units is available from the Statistical Climatology Branch, Climatological Analysis Division, NCDC.

CLIMATOGRAPHY OF THE UNITED STATES NO. 20

CLIMATIC SUMMARIES FOR SELECTED SITES, 1951—80

OREGON

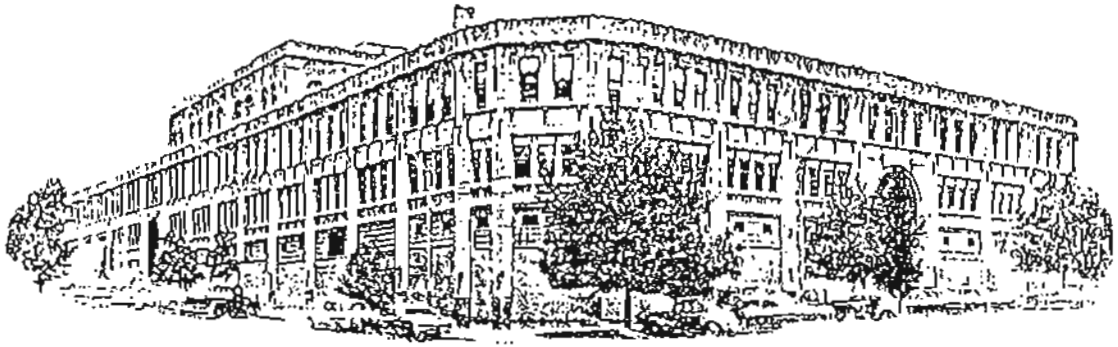
CLIMATOLOGICAL SUMMARY

DEGREE DAYS

PRECIPITATION PROBABILITY

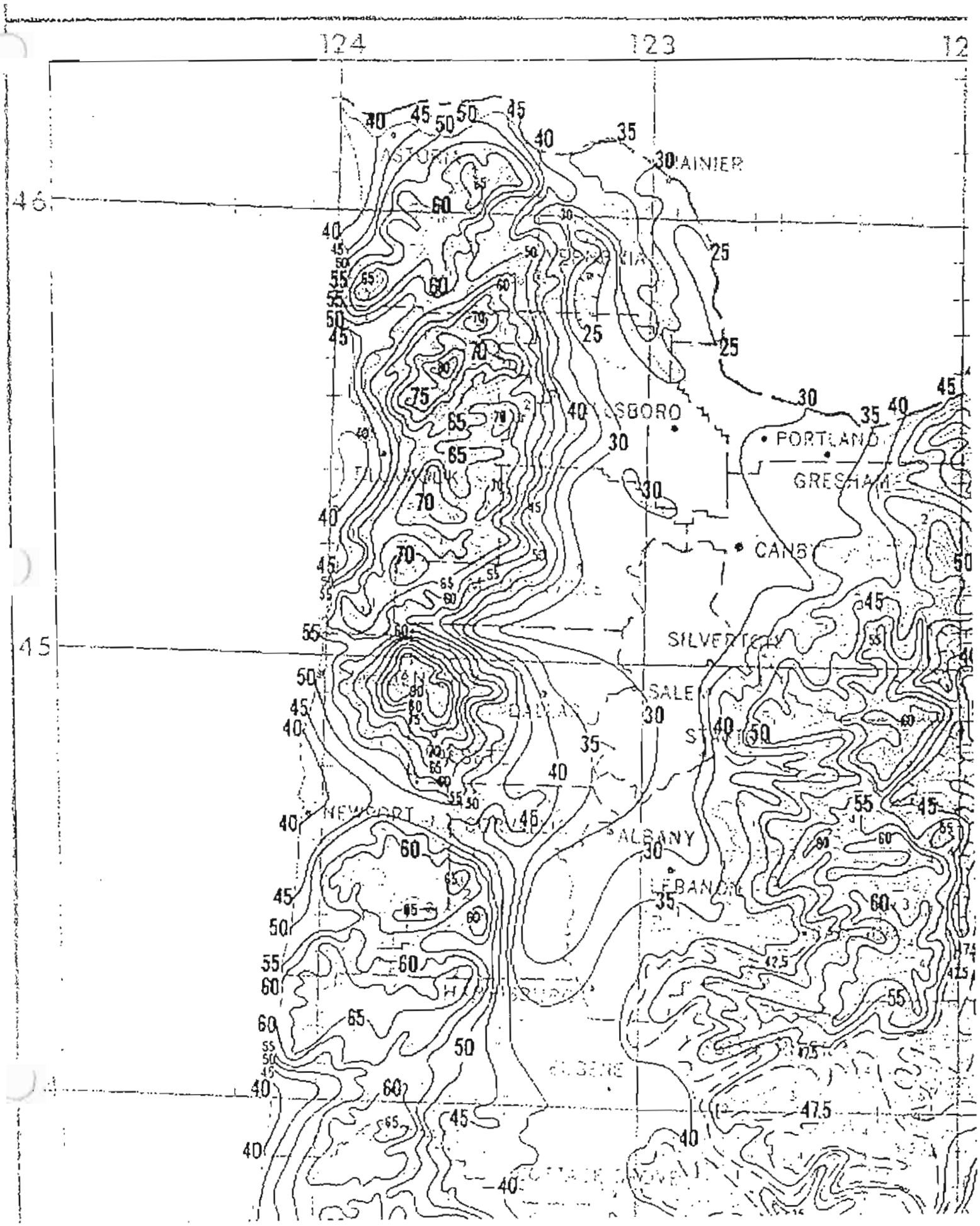
FREEZE DATA

GROWING DEGREE UNITS



NATIONAL CLIMATIC DATA CENTER
ASHEVILLE, N.C.

noaa NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION / NATIONAL ENVIRONMENTAL SATELLITE, DATA, AND INFORMATION SERVICE / NATIONAL CLIMATIC DATA CENTER ASHEVILLE, N.C.


124
123
46
45
RAINIER
PORTLAND
GRESHAM
CANBY
SILVERTON
SALEM
NEWPORT
ALBANY
LEBANON

Oregon
Theodore R. Kulongoski, Governor

Department of Environmental Quality
Northwest Region Portland Office
2020 SW 4th Avenue, Suite 400
Portland, OR 97201-4987
(503) 229-5263
FAX (503) 229-6945
TTY (503) 229-5471

September 23, 2004

MR ARLEY SULLIVAN
PUBLIC WORKS DEPARTMENT
CITY OF TILLAMOOK
210 LAUREL AVENUE
TILLAMOOK OR 97141

RE: City of Tillamook NPDES Permit
Modification No. 1
File No. 88665

Dear Mr. Sullivan:

Attached is the public notice document and proposed modification for the City of Tillamook's NPDES permit. The proposed modification is for the wet weather mass load increase. The comment period for this modification is from September 24th through October 29th. The City of Tillamook has the opportunity to comment during the public comment period.

If you have any questions or comments please let me know (503) 229-5586.

Sincerely,

Beth Moore

Beth Moore
Environmental Specialist
Water Quality Source Control Section

cc: John Harrison, Kennedy/Jenks Consultants 2828 SW Naito Parkway, Suite 250, Portland, OR 97201
Donita Parks, City of Tillamook

Nov 1st
Is when
new limits
Come

Public Notice: Request for Comments

Proposed Modification to NPDES Permit for the City of Tillamook (file no. 88665)

State of Oregon
Department of
Environmental
Quality

Water Quality
Source Control
Section
2020 SW 4th Avenue
Portland, OR 97201
Phone: (503) 229-5263
(800) 452-4011
Fax: (503) 229-6957
Contact: Beth Moore

Notice issued: September 24, 2004

Written comments due:
No later than 5:00 pm, October 29, 2004

Where can I get more information and send comments?
DEQ accepts comments by mail, fax and e-mail.

Name: Beth Moore

Phone: (503) 229-5586 or
toll free in Oregon (800) 452-4011

Mailing address:
DEQ Water Quality
2020 SW 4th Ave., Suite 400
Portland, OR 97201-4987

Fax: (503) 229-6957

E-mail: moore.beth@deq.state.or.us
(Receipt of E-mail comments will be acknowledged immediately. If there is a delay between servers, e-mails may not be received before the deadline.)

How can I review documents?
You can review the proposed permit and review report at the following locations:

Tillamook County Library
210 Ivy Avenue
Tillamook, OR 97141

DEQ's Northwest Region Office
2020 SW 4th Ave., Suite 400
Portland, OR 97201

For a review appointment at DEQ call
Jan Coomler (503) 229-5087

What are DEQ's responsibilities?
The Oregon Department of Environmental Quality (DEQ) is the regulatory agency that helps protect and preserve Oregon's environment. DEQ is responsible for protecting and enhancing Oregon's water and air quality, for cleaning up spills and releases of hazardous materials, and for managing the proper disposal of hazardous and solid wastes. One way DEQ does this is by requiring permits for certain activities. The purpose of this notice is to invite you submit written comments on this proposed permit by no later than 5:00pm October 29, 2004.

Who is the applicant?
City of Tillamook
210 Laurel Ave.
Tillamook, OR 97141

Where is the facility located?
City of Tillamook Wastewater Treatment Facility
710 Fifth Street
Tillamook, OR

What is proposed?
DEQ proposes to modify the National Pollutant Discharge Elimination System (NPDES) Permit for the City of Tillamook wastewater treatment facility.

Permit expiration: The expiration date is December 31, 2006

What are the special conditions of this permit?
The permit provided for the wet weather (November 1- April 30) mass load limits to be changed based on the design average wet weather flow and/or lower percent removal requirements if the permittee submitted a request with engineering studies in the facility plan in accordance with OAR 340-041-0061(10)(D). The City of Tillamook submitted a Wastewater Facility Plan in December 2003 which contained documentation describing and supporting the design average wet weather flow and percent removals. The Department approved the engineer study which supported the change in the wet weather mass load increase and percent removal requirements.

Compliance history:
The City of Tillamook entered into MAO No. WQ/M-NWR-02-144 and completed improvements to the wastewater treatment

facility as part of an expanded corrective action plan. The City of Tillamook completed the improvements by May 1, 2003.

Who is affected?

The proposed actions will affect those served by the wastewater treatment facility and those who use the Trask River in the vicinity of the facility.

What other DEQ permits are required?

Any other permits required by the DEQ for this facility are not affected by this permit.

What legal requirements apply?

A permit is issued pursuant to Oregon Revised Statute 468B.050 and the Federal Water Pollution Control Act Amendments of 1972.

What discretionary decisions did DEQ use in deciding to issue the permit?

The wet weather mass load increase is allowed for an existing facility that has not had its secondary treatment capacity expanded since 1992 provided the facility follow an approved program and time schedule for identifying and reducing inflow. Per Schedule C condition no. 1 of the permit, this requirement is in effect no later than 180 days after the permit modification is issued.

What happens next?

DEQ will review and consider all comments received during the comment period. Following this review, DEQ may issue the permit as proposed or modified, or deny the permit.

DEQ will hold a hearing if the written requests for a public hearing during the public comment period is received from at least 10 persons or from an organization or organizations representing at least ten persons.

Accessibility information

DEQ is committed to accommodating people with disabilities at our hearings. Please notify DEQ of any special physical or language accommodations or if you need information in large print, Braille or another format. To make these arrangements, contact DEQ Public Affairs at (503) 229-5696 or toll free in Oregon at (800) 452-4011.

People with hearing impairments may call DEQ's TTY number, (503) 229-5471.

What are the known health effects or environmental impacts of the permitted substances stored, disposed of, discharged or emitted by the facility?

Improperly treated wastewater can carry disease causing organisms and pollutants to the water body. Wastewater treatment facilities are designed to limit the environmental and public exposure.

How are the permitted substances measured?

Regular sampling of the treated effluent from this facility is required to document compliance with the permit.

MODIFICATION NO. 1
National Pollutant Discharge Elimination System
Waste Discharge Permit

Department of Environmental Quality
2020 SE 4th Avenue, Suite 400, Portland OR 97201
Telephone: (503) 229-5263

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

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ISSUED TO:

City of Tillamook
210 Laurel Avenue
Tillamook, OR 97141

SOURCES COVERED BY THIS PERMIT:

Type of Waste	Outfall Number	Outfall Location
Treated Wastewater	001	R.M. 1.9

FACILITY TYPE AND LOCATION:

Rotating Biological Contactors
710 Fifth Street
Tillamook

Treatment System Class: Level IV
Collection System Class: Level III

RECEIVING STREAM INFORMATION:

Basin: North Coast
Sub-Basin: Wilson/Trask/Nestucca

Receiving Stream: Trask River
Hydro Code: 11E-TRAS 1.9D
County: Tillamook

EPA REFERENCE NO: OR-002066-4

Issued in response to Schedule A, Condition 1.a. of the NPDES permit issued on January 6, 2003.

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

Neil Mullane, Administrator, Water Quality Program
Northwest Region

Date

SCHEDULE A

1. **Waste Discharge Limitations not to be exceeded after permit modification.**

a. Treated Effluent Outfall 001

(1) May 1 - October 31:

Parameter	Average Effluent Concentrations Monthly	Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
BOD_5	20 mg/L	30 mg/L	177	265	354
TSS	20 mg/L	30 mg/L	177	265	354

Other parameters	Limitations
Fecal Coliform Bacteria	Shall not exceed a monthly geometric mean of 200 organisms per 100 mL. No single sample shall exceed 400 organisms per 100 mL. (See Note 1)
BOD_5 and TSS Removal Efficiency	Shall not be less than 85% based on a monthly average for BOD_5 and shall not be less than 85% based on a monthly average for TSS.

(2) November 1 - April 30:

Parameter	Average Effluent Concentrations Monthly	Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum Lbs
BOD_5	30 mg/L	45 mg/L	530	795	1060
TSS	30 mg/L	45 mg/L	530	795	1060

Other parameters	Limitations
Fecal Coliform Bacteria	Shall not exceed a monthly geometric mean of 49 organisms per 100 mL. No single sample shall exceed 151 organisms per 100 mL. (See Note 1)
BOD_5 and TSS Removal Efficiency	Shall not be less than 85%, except that when the monthly average for BOD_5 or TSS is less than 167 mg/L the percent removal shall not apply, provided that the treatment plant shall be operated as effectively as possible.

*Average dry weather design flow to the facility equals 1.06 MGD. Per OAR 340-0061(10)(A) the summer mass load limits are based upon average dry weather design flow to the facility. Per OAR 340-041-0061(10)(B), the winter mass load limits are based upon average wet weather design flow to the facility equaling 2.12 MGD. Per OAR 340-041-0061(10)(C), the daily mass load limit is suspended on any day in which the flow to the treatment facility exceeds 2.12 MGD (twice the design average dry weather flow).

PERMIT MODIFICATION NO. 1
EVALUATION AND FACT SHEET

In the permit issued on January 6, 2003, the mass load limits for BOD and TSS were based on the dry weather design flow of 1.06 MGD. The mass load limits for BOD and TSS based on the dry weather design flow are shown below.

(1) May 1 - October 31:

Parameter	Average Effluent Concentrations Monthly	Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily Maximum lbs
BOD_5	20 mg/L	30 mg/L	177	265	354
TSS	20 mg/L	30 mg/L	177	265	354

(2) November 1 - April 30:

Parameter	Average Effluent Concentrations Monthly	Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily Maximum lbs
BOD_5	30 mg/L	45 mg/L	265	398	530
TSS	30 mg/L	45 mg/L	265	398	530

The permit provided for the wet weather (November 1- April 30) mass load limits to be changed based on the design average wet weather flow and/or lower percent removal requirements if the permittee submitted a request with engineering studies in the facility plan in accordance with OAR 340-041-0061(10)(D). The City of Tillamook submitted a Wastewater Facility Plan in December 2003 which contained documentation describing and supporting the design average wet weather flow and percent removals (1/12/2004 Section 6.5.3 Percent Removal Requirements). The Department approved the engineer study which supported the change in the wet weather mass load increase and percent removal requirements.

In addition, the wet weather mass load increase is allowed for an existing facility that has not had its secondary treatment capacity expanded since 1992 provided the facility follow an approved program and time schedule for identifying and reducing inflow. Per Schedule C condition no. 1 of the permit, this requirement is in effect no later than 180 days after the permit modification is issued.

The federal secondary treatment regulations allow for exceptions to the 85% removal efficiency limitation for wastewater treatment facilities with a separate sewer system under 40 CFR Part 133.103. To qualify for this exception, there are three conditions to meet.

1) The treatment facility is consistently meeting or will consistently meet, its concentration limits but cannot meet the percent removal limitations due to less concentrated wastewater.

The City of Tillamook recently added improvements to the wastewater treatment facility as part of a corrective action plan that was approved June 4, 2001. The City of Tillamook

entered into MAO No. WQ/M-NWR-02-144 with the Department to complete these improvements by May 1, 2003 as part of an expanded corrective action plan. Concentration limits are required to be met with these improvements in place.

The improvements that were made as part of the expanded corrective action plan include some of the following main elements:

- A secondary solids aeration recirculation system to "activate" the RBC's and improve treatment performance, especially in the summer.
- Replacement of leaking air piping for the RBC drives.
- Primary sludge pumping improvements to simplify operations, including variable speed drives, flow meters and electric control valves.
- Concrete work to eliminate dead zones, sludge accumulations, and odor sources throughout the new primary clarifier influent channels and headworks channel.

2) To meet the percent removal limitations, the facility would have to meet "significantly more stringent" concentration limits than would otherwise be required by the concentration based standards. EPA in 40 CFR Part133.101(m) defined "significantly more stringent limitations" to mean: BOD and TSS limitations necessary to meet the percent removal requirements of at least 5 mg/L more stringent than the otherwise applicable concentration based limits. Since the concentration standard is 30 mg/l, this definition would define any limit of 25 mg/l or below as "significantly more stringent." This means an influent baseline concentration for a secondary treatment plant of 166.66 mg/L or above (25 mg/L/0.15), is necessary in order to meet 85% removal.

The facility plan (Section 6.5.3 Percent Removal Requirements) provides influent data showing that the influent concentration drops below 166 mg/l when plant flows exceed 1.5 MGD, a routine occurrence. The plan provides an explanation of treatment levels that would be required when the influent concentration drops below 167 mg/L, in order to meet 85% removal. For example, to treat an influent concentration of 67 mg/L with 85% removal efficiency would require treatment to an effluent concentration of 10 mg/L (67 x 0.15 = 10 mg/L). This would require significant capital expenditure on advanced treatment technologies for the existing facility, such as, effluent filtration, chemical addition or other advanced treatment systems.

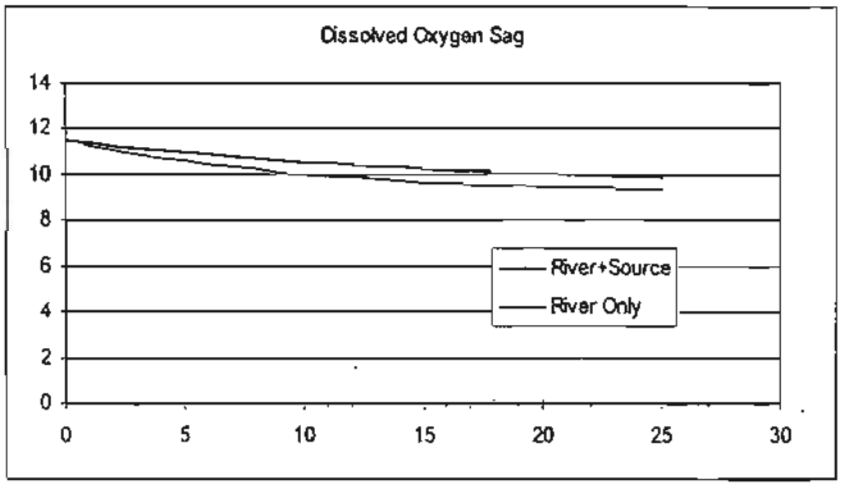
3) The less concentrated effluent is not the result of excessive infiltration and inflow. Excessive infiltration and inflow is defined as "the quantities of infiltration and inflow which can be economically eliminated from a sewer system as determined in a cost effective analysis that compares the costs for correcting the infiltration/inflow conditions to the total costs for transporting and treatment of the infiltrations and inflow."

In Section 7 Table 7.3 of the facility plan rough cost estimates are given that shows a system-wide rehabilitation to remove inflow and infiltration in the collection basins is $24,000,000. The facility plan goes on to state that "experience has shown that a 40% to 60% reduction of the inflow and infiltration can be removed following extensive rehabilitation of sewer lines and laterals." It has been shown in the facility plan that the less concentrated effluent is not the result of excessive infiltration and inflow.

The water quality impact of the increased mass load of BOD_5 discharge on the Trask River was analyzed for dissolved oxygen using the Streeter-Phelps dissolved oxygen model. This model is limited because it is a stream model and does not incorporate the tidal action of Tillamook Bay which has an influence on the Trask River. The head of the tide water on the Trask River can reach around river mile 4.8. This information is based on the 1972 North Coast Drainage Basin- Oregon State Water Resources Board map.

The Streeter-Phelps model was used to interpret the downstream effects from the point of discharge in the Trask River. The discharge moves downstream with the tide as it goes out to Tillamook Bay.

The outfall is located at river mile 1.9 on the Trask River. In a short distance, the discharge into the Trask River will reach the Tillamook Bay. The estuarine standard for dissolved oxygen is 6.5 mg/L. The Streeter-Phelps model shows that the dissolved oxygen that reaches Tillamook Bay is above 6.5 mg/L.



The inputs for the Streeter-Phelps modeling are provided below.

Effluent		
Temp (C)	19	Based on November 2002 temperature data.
DO(mg/L)	8	DO data from DMR's is 8.5 mg/l and higher
BOD_5(mg/L)	30	Based on monthly allowable. Daily 2003/ 2004 DMR winter data is lower.
NH_3-N(mg/L)	20	Reasonable Assumption
Q (mgd)	2.12	Based on average wet weather flow.
K_1(@20 C)	0.0673	Laboratory Reaction Rate
River		
Temp (C)	10	Laser Data
DO	11.5	Laser Data
BOD5	1.5	Laser Data
NH3	0.06	Laser Data
Q (cfs)	326	10th percentile of minimum 7 day flow November to April 30th 1996 to 2004 USGS Station 14302480 data
K_d	0.044	Deoxygenation rate(/day)
K_{oa}	0.325	Organic to NH_3 conversion rate (/day)
K_{ai}	0.324	NH_3 to NO_2 conversion rate (/day)
K_{in}	0.325	NO_2 to NO_3 conversion rate(/day)
K_r	0.044	K_d+K_s (/day), K_s = 0
Depth(ft)	8	Physical River data
Width (ft)	109	Physical River data
U (fps)	0.38	Calculation based on flow
U (mpd)	6.18	Calculation based on flow
K_a (see comment)	0.277	Reaeration O'conner Dobbin formula
S_B g O2/m2 day	1.065	Sediment oxygen demand estuarine mud

RM	Time	River Only	River&Source	Source Ambient
(mi)	(days)	DO	DO	DO Deficit
0.00	0.00	11.50	11.47	0.03
1.00	0.16	11.38	11.25	0.12
2.00	0.32	11.26	11.06	0.20
3.00	0.49	11.15	10.88	0.27
4.00	0.65	11.05	10.71	0.33
5.00	0.81	10.95	10.56	0.39
6.00	0.97	10.86	10.43	0.43

FINAL

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM
WASTE DISCHARGE PERMIT

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

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

ISSUED TO:

City of Tillamook
210 Laurel Avenue
Tillamook, OR 97141

SOURCES COVERED BY THIS PERMIT:

Type of Waste	Outfall Number	Outfall Location
Treated Wastewater	001	R.M. 1.9

FACILITY TYPE AND LOCATION:

Rotating Biological Contactors
710 Fifth Street
Tillamook

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

RECEIVING STREAM INFORMATION:

Basin: North Coast
Sub-Basin: Wilson / Trask / Nestucca
Receiving Stream: Trask River
Hydro Code: 11E-TRAS 1.9 D
County: Tillamook

EPA REFERENCE NO: OR-002066-4

Issued in response to Application No. 990099 received July 16, 1999.

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

Robert P. Baumgartner, Water Quality Manager
Northwest Region

01-06-2003
Date

PERMITTED ACTIVITIES

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

Page
Schedule A - Waste Discharge Limitations not to be Exceeded 3
Schedule B - Minimum Monitoring and Reporting Requirements 5
Schedule C - Compliance Conditions and Schedules 10
Schedule D - Special Conditions 12
Schedule F - General Conditions 15

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

SCHEDULE A

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

a. Treated Effluent Outfall 001

(1) May 1 - October 31:

Parameter	Average Effluent Concentrations Monthly	Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
BOD_5	20 mg/L	30 mg/L	177	265	354
TSS	20 mg/L	30 mg/L	177	265	354

Other parameters	Limitations
Fecal Coliform Bacteria	Shall not exceed a monthly geometric mean of 200 organisms per 100 mL. No single sample shall exceed 400 organisms per 100 mL. (See Note 1)

(2) November 1 - April 30:

Parameter	Average Effluent Concentrations Monthly	Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
BOD_5	30 mg/L	45 mg/L	265	398	530
TSS	30 mg/L	45 mg/L	265	398	530

Other parameters	Limitations
Fecal Coliform Bacteria	Shall not exceed a monthly geometric mean of 49 organisms per 100 mL. No single sample shall exceed 151 organisms per 100 mL. (See Note 1)

* Average dry weather design flow to the facility equals 1.06 MGD. Mass load limits based upon average dry weather design flow to the facility. If the permittee requests a winter mass load increase and/or a lower percent removal requirement, then the permittee is required to submit engineering studies as part of the Facilities Plan under MAO WQ/M-NWR-02-144. An increase will be evaluated as part of the Facility Planning Process. Upon review and approval of the engineering studies to determine the design average wet weather flow, pursuant to OAR 340-41-120 (9), and percent removal limits, the Department intends to modify this permit and include revised mass load limits.

(3) Year-Round

Other parameters (year-round)	Limitations
pH	Shall be within the range of 6.0 - 9.0
BOD_5 and TSS Removal Efficiency	Shall not be less than 85% monthly average for BOD_5 and 85% monthly for TSS.
Chlorine Residual	Shall not exceed a Maximum Daily Limit of 0.02 mg/L

Chlorine Residual	Shall not exceed a Monthly Average limit of 0.01 mg/L
Excess Thermal Load	Shall not exceed a 7 day moving average of the daily maximums of 141 million BTU/day Shall not exceed a monthly average of 62 million BTU/day

(4) Except as provided for in OAR 340-45-080, no wastes shall be discharged and no activities shall be conducted which violate Water Quality Standards as adopted in OAR 340-41-0205 except in the following defined mixing zone:

The allowable mixing zone is that portion of the Trask River, which extends 100 feet downstream and 50 feet upstream of the point of discharge. The Zone of Initial Dilution (ZID) shall be defined as that portion of the allowable mixing zone that is 10 feet downstream and 5 feet upstream of the point of discharge. (See Note 2)

NOTES:

1. May 1 to October 31
If a single sample exceeds 400 organisms per 100 mL, then five consecutive re-samples may be taken at four-hour intervals beginning within 28 hours after the original sample was taken. If the log mean of the five re-samples is less than or equal to 200 organisms per 100 mL, a violation shall not be triggered.

November 1 to April 30
If a single sample exceeds 151 organisms per 100 mL, then five consecutive re-samples may be taken at four-hour intervals beginning within 28 hours after the original sample was taken. If the log mean of the five re-samples is less than or equal to 49 organisms per 100 mL, a violation shall not be triggered.

2. When the permittee completes a dilution and mixing zone study as required in the Temperature Management Plan, the Department will reopen this permit and amend the conditions pertaining to limits as appropriate.

SCHEDULE B

1. Minimum Monitoring and Reporting Requirements (unless otherwise approved in writing by the Department).

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

a. Influent

The facility influent sampling locations are at the headworks:

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD)	Daily	Measurement
Flow Meter Calibration	Semi-Annual	Verification
BOD_5	2/Week	24- hour Composite
TSS	2/Week	24- hour Composite
pH	3/Week	Grab
Temperature	Weekly	Grab (see Note 6)/or Continuous

b. Treated Effluent Outfall 001

The facility effluent sampling locations are prior to the outfall:

Item or Parameter	Minimum Frequency	Type of Sample
BOD_5	2/Week	Composite
TSS	2/Week	Composite
pH	3/Week	Grab
Fecal Coliform	2/Week	Grab
Quantity Chlorine Used	Daily	Measurement
Chlorine Residual	Daily	Grab
Pounds Discharged (BOD_5 and TSS)	2/Week	Calculation
Average Percent Removed (BOD_5 and TSS)	Monthly	Calculation
Temperature	Weekly	Grab (see Note 6) or continuous
Excess Thermal Load	7-day moving average	Calculation (See Note1)
Excess Thermal Load	Monthly	Calculation
Nutrients		
TKN, NH3-N, NO2+NO3-N, Total Phosphorus	1 Month (May 1-Oct 31)	24-hour Composite
Toxics:		
Metals (Ag, Al, As, Cd, Cr, Cu, Fe, Pb, Hg, Ni, Zn, Se)	3 samples	24-hour composite

(See Note 9)		
Bioassay (See Note 2)	Semi-annually	Acute & chronic

c. Trask River - May 1 through October 31

Item or Parameter	Minimum Frequency	Type of Sample
Stream Temperature, Max Daily, Upstream	Daily	Continuous (See Note 6, 7 & 8)
Toxics		
Upstream Metals (Ag, Al, As, Cd, Cr, Cu, Fe, Pb, Hg, Ni, Zn, Se) Cyanide measured as total (See Note 9)	3 samples	Grab

d. Biosolids Management

Item or Parameter	Minimum Frequency	Type of Sample
Sludge analysis including: Total Solids (% dry wt.) Volatile solids (% dry wt.) Biosolids nitrogen for: NH_3-N; NO_3-N; & TKN (% dry wt.) Phosphorus (% dry wt.) Potassium (% dry wt.) pH (standard units) Sludge metals content for: As, Cd, Cu, Hg, Mo, Ni, Pb, Se & Zn, measured as total in mg/kg	Annually	Composite sample to be representative of the product to be land applied from the Digester withdrawal line (See Note 3)
Record of locations where biosolids are applied on each DEQ approved site. (Site location maps to be maintained at treatment facility for review upon request by DEQ)	Each Occurrence	Date, volume & locations where biosolids were applied recorded on site location map.
Record of % volatile solids reduction accomplished through stabilization	Monthly	Calculation (See Note 4)
Record of digestion days (mean cell residence time)	Monthly	Calculation (See Note 5)
Daily Minimum Sludge Temperature	Daily	Record

e. Continuous Temperature Monitoring (May 1- October 31)

Item or Parameter	Minimum Frequency	Type of Sample
Effluent Temperature, Daily Max (See Note 6)	Daily	Continuous
Effluent Temperature, 7 Day Moving Average of Daily Maximums (See Note 6)	Weekly	Calculation
Thermal Load (See Note 6)	Weekly	Calculation (See Note 7)
Audit Continuous Temperatutre Monitors (See Note 7)	June and September	Record
Check Temperature Monitors (See Note 8)	Monthly	Visual Observation and Record

2. Reporting Procedures

a. Monitoring results shall be reported on approved forms. With the exception of biosolids monitoring and continuous temperature monitoring, the reporting period is the calendar month. Reports must be submitted to the Department's Northwest Region - Portland office by the 15th day of the following month. The biosolids monitoring report shall be submitted annually.

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

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

3. Report Submittals

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

b. For any year in which biosolids are land applied, a report shall be submitted to the Department by February 19 of the following year that describes solids handling activities for the previous year and includes, but is not limited to, the required information outlined in OAR 340-50-035(6)(a)-(e).

NOTES:

1. Excess Thermal Load

Grab Sample:
The grab sample will be representative of daily maximum temperature.
If only one weekly grab sample is taken then that one sample will be used for the 7 day moving average.
The monthly average shall be the average of the weekly samples.

Continuous Monitor:
7 day moving average is calculated as follows:

Step 1: (daily maximum hourly average temperature °F – applicable stream temperature standard, °F) x (daily flow in MGD) x (8.34 lb/gal) x (specific heat of water 1BTU/lb°F) x (1,000,000) = daily Excess Thermal Load in British Thermal Units per day.

Step 2: Average each daily Excess Thermal Load with the daily Excess Thermal Loads from the previous 6 days

Monthly Average:
The Monthly Average is an average of the each daily Excess Thermal Load.

2. Beginning no later than April 2003, the permittee shall conduct bioassay testing for a period of one (1) year in accordance with the frequency specified above. If the bioassay tests show that the effluent samples are not toxic at the dilutions determined to occur at the Zone of Immediate Dilution and the Mixing Zone, no further bioassay testing will be required during this permit cycle. Note that bioassay test results will be required along with the next NPDES permit renewal application.

3. Composite samples from the digester withdrawal line shall consist of at least 4 aliquots of equal volume collected over an 8 hour period and combined.

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

4. Calculation of the % volatile solids reduction is to be based on comparison of a representative grab sample of total and volatile solids entering each digester (a weighted blend of the primary and secondary clarifier solids) and a representative composite sample of solids exiting each digester withdrawal line (as defined in note 3 above).

5. The days of digestion shall be calculated by dividing the effective digester volume by the average daily volume of sludge production.

6. For influent and effluent samples, report the results with each month's DMR. For continuous instream monitors and concomitant effluent temperatures, record the time between temperature readings. The results are to be tabulated and submitted in an annual report. After two full years of temperature monitoring, and if approved in writing by the Department, monitoring may be waived in accordance with Schedule C. 5. The first year of temperature data will be submitted with the Mixing Zone Study Report as required in Schedule C.5. The sites shall be located following the DEQ Procedural Guidance for Water Temperature Monitoring, [illegible] located just upstream from the point of discharge. The permittee shall keep a map clearly

showing the proposed monitoring points, a description of the stream conditions (e.g. pools or riffle) and the latitude and longitude of the sites, and have this information available for DEQ inspection upon request at the facility site.

7. Continuous temperature monitors that are used for the instream and effluent monitoring must be audited in June and September, following procedures described in DEQ Procedural Guidance for Water Temperature Monitoring.

8. Continuous temperature monitors are to be checked visually monthly to insure that the devices are still in place and submerged.

9. The metal samples are to be collected during the mixing zone study. The detection limits for the metals analysis should follow the USEPA methods that best provide the following "minimum quantitative level" as defined in standard methods.

Parameter	Minimum Quantitative Level (ug/L)
Aluminum	87
Arsenic	48
Cadmium	0.44
Chromium	11
Copper	4.23
Cyanide	5.2
Iron	500
Lead	0.7
Mercury	0.12
Nickel	57
Silver	0.12
Zinc	38
Selenium	35

SCHEDULE C

Compliance Schedules and Conditions

1. Within 180 days of permit modification to include higher winter mass load limits, the permittee shall submit to the Department for review and approval a proposed program and time schedule for identifying and reducing inflow. Within 60 days of receiving written Department comments, the permittee shall submit a final approvable program and time schedule. The program shall consist of the following:

   a. Identification of all overflow points and verification that sewer system overflows are not occurring up to a 24-hour, 5-year storm event or equivalent;

   b. Monitoring of all pump station overflow points;

   c. A program for identifying and removing all inflow sources into the permittee's sewer system over which the permittee has legal control; and

   d. If the permittee does not have the necessary legal authority for all portions of the sewer system or treatment facility, a program and schedule for gaining legal authority to require inflow reduction and a program and schedule for removing inflow sources.

2. By no later than ninety (90) days after permit issuance, the permittee shall submit to the Department a report which either identifies known sewage overflow locations and a plan for estimating the frequency, duration and quantity of sewage overflowing, or confirms that there are no overflow points. The report shall also provide a schedule to eliminate the overflow(s), if any.

3. Industrial Waste Survey/Pretreatment Program

   a. As soon as practicable, but by no later than six (6) months from permit issuance date, the permittee shall submit to the Department an industrial waste survey as described in 40 CFR 403.8(f)(2)(i-iii) suitable to make a determination as to the need for development of a pretreatment program.

   b. Should the Department determine that a pretreatment program is required, the permit shall be reopened and modified in accordance with 40 CFR 403.8(e)(1) to incorporate a compliance schedule to require development of a pretreatment program. The compliance schedule requiring program development shall be developed in accordance with the provisions of 40 CFR 403.12(k), and shall not exceed twelve (12) months.

4. By February 17, 2003, the permittee shall submit to the Department the results of a dilution and mixing zone study. The study shall conform to the approved scope of work that was submitted to the Department.

5. Every year for the first two years after permit issuance the permittee shall submit the results of the continuous temperature monitoring. The report shall include summary statistics by month for the daily maximum hourly average temperature and the maximum 7-day moving average of the daily maximums. The daily maximum temperature is defined as the maximum hourly average. Temperature data shall be provided to the Department in electronic format. The reports shall be due no later than February 17th of the year starting in 2003. Upon review of the data the Department may waive ambient temperature monitoring during the months the river does not violate the temperature standards or request an extended period of sampling if the temperature monitoring does not capture this transition in the river.

6. The permittee is expected to meet the compliance dates which have been established in this schedule. Either prior to or no later than 14 days following any lapsed compliance date, the permittee shall submit to the

Department a notice of compliance or noncompliance with the established schedule. The Director may revise a schedule of compliance if he determines good and valid cause resulting from events over which the permittee has little or no control.

SCHEDULE D

Special Conditions

1. The Temperature Management Plan is considered part of this permit and should be attached to the permit. The permittee is required to implement all elements of the Temperature Management Plan approved by the Department.

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

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

4. **Whole Effluent Toxicity Testing**

   a. The permittee shall conduct whole effluent toxicity tests as specified in Schedule B of this permit.

   b. Bioassay tests may be dual end-point tests, only for the fish tests, in which both acute and chronic end-points can be determined from the results of a single chronic test (the acute end-point shall be based upon a 48-hour time period).

   c. Acute Toxicity Testing - Organisms and Protocols

      (1) The permittee shall conduct 48-hour static renewal tests with the *Ceriodaphnia dubia* (water flea) and the *Pimephales promelas* (fathead minnow).

      (2) The presence of acute toxicity will be determined as specified in **Methods for Measuring the Acute Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms**, Fourth Edition, EPA/600/4-90/027F, August 1993.

      (3) An acute bioassay test shall be considered to show toxicity if there is a statistically significant difference in survival between the control and 100 percent effluent, unless the permit specifically provides for a Zone of Immediate Dilution (ZID) for biotoxicity. If the permit specifies such a ZID, acute toxicity shall be indicated when a statistically significant difference in survival occurs at dilutions greater than that which is found to occur at the edge of the ZID.

   d. Chronic Toxicity Testing - Organisms and Protocols

      (1) The permittee shall conduct tests with: *Ceriodaphnia dubia* (water flea) for reproduction and survival test endpoint, *Pimephales promelas* (fathead minnow) for growth and survival test endpoint, and *Raphidocelis subcapitata* (green alga formerly known as *Selanastrum capricornutum*) for growth test endpoint.

      (2) The presence of chronic toxicity shall be estimated as specified in **Short-Term Methods for Estimating the Chronic Toxicity of Effluents and Receiving Waters to Freshwater Organisms**, Third Edition, EPA/600/4-91/002, July 1994.

(3) A chronic bioassay test shall be considered to show toxicity if a statistically significant difference in survival, growth, or reproduction occurs at dilutions greater than that which is known to occur at the edge of the mixing zone. If there is no dilution data for the edge of the mixing zone, any chronic bioassay test that shows a statistically significant effect in 100 percent effluent as compared to the control shall be considered to show toxicity.

e. Quality Assurance

(1) Quality assurance criteria, statistical analyses and data reporting for the bioassays shall be in accordance with the EPA documents stated in this condition and the Department's Whole Effluent Toxicity Testing Guidance Document, January 1993.

f. Evaluation of Causes and Exceedances

(1) If toxicity is shown, as defined in sections c.(3) or d.(3) of this permit condition, another toxicity test using the same species and Department approved methodology shall be conducted within two weeks, unless otherwise approved by the Department. If the second test also indicates toxicity, the permittee shall follow the procedure described in section f.(2) of this permit condition.

(2) If two consecutive bioassay test results indicate acute and/or chronic toxicity, as defined in sections c.(3) or d.(3) of this permit condition, the permittee shall evaluate the source of the toxicity and submit a plan and time schedule for demonstrating compliance with water quality standards. Upon approval by the Department, the permittee shall implement the plan until compliance has been achieved. Evaluations shall be completed and plans submitted to the Department within 6 months unless otherwise approved in writing by the Department.

g. Reporting

(1) Along with the test results, the permittee shall include: 1. the dates of sample collection and initiation of each toxicity test; 2. the type of production; and 3. the flow rate at the time of sample collection. Effluent at the time of sampling for bioassay testing should include samples of required parameters stated under Schedule B, condition 1. of this permit.

(2) The permittee shall make available to the Department, on request, the written standard operating procedures they, or the laboratory performing the bioassays, are using for all toxicity tests required by the Department.

h. Reopener

(1) If bioassay testing indicates acute and/or chronic toxicity, the Department may reopen and modify this permit to include new limitations and/or conditions as determined by the Department to be appropriate, and in accordance with procedures outlined in Oregon Administrative Rules, Chapter 340, Division 45.

15. A priority pollutant scan shall be performed at least once during the term of this permit and must be submitted to the Department as part of the Permittee's NPDES permit renewal application. The permittee shall perform chemical analysis of its influent, effluent and biosolids to be beneficially used for the specific toxic pollutants listed in Tables II and III of Appendix D of 40 CFR Part 122. The influent and effluent samples shall be 24-hour daily composites, except where sampling volatile compounds. In this case, six (6) discrete samples (not less than 100 mL) collected over the operating day are acceptable. The permittee shall take special precautions in compositing the individual grab samples for the volatile organics to insure sample integrity (i.e.

separately and averaged. For biosolids analyses, a composite of weekly grab samples for the final product shall be used.

6. The permittee shall comply with Oregon Administrative Rules (OAR), Chapter 340, Division 49, "Regulations Pertaining To Certification of Wastewater System Operator Personnel" and accordingly:

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

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

   b. The permittee's wastewater system may not be without supervision (as required by Special Condition 6.a. above) for more than thirty (30) days. During this period, and at any time that the supervisor is not available to respond on-site (i.e. vacation, sick leave or off-call), the permittee must make available another person who is certified at no less than one grade lower then the system classification.

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

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

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

   f. Upon written request, the Department may grant the permittee reasonable time, not to exceed 120 days, to obtain the services of a qualified person to supervise the wastewater system. The written request must include justification for the time needed, a schedule for recruiting and hiring, the date the system supervisor availability ceased and the name of the alternate system supervisor(s) as required by 6.b. above.

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

8. An adequate contingency plan for the prevention and handling of spills and unplanned discharges shall be in force at all times. A continuing program of employee orientation and education shall be maintained to ensure awareness of necessity of good inplant control and quick and proper action in the event of a spill or accident.

NPDES GENERAL CONDITIONS
(SCHEDULE F)

SECTION A. STANDARD CONDITIONS

1. Duty to Comply

The permittee must comply with all conditions of this permit. Any permit noncompliance constitutes a violation of Oregon Revised Statutes (ORS) 468B.025 and is grounds for enforcement action; for permit termination, suspension, or modification; or for denial of a permit renewal application.

2. Penalties for Water Pollution and Permit Condition Violations

Oregon Law (ORS 468.140) allows the Director to impose civil penalties up to $10,000 per day for violation of a term, condition, or requirement of a permit.

In addition, a person who unlawfully pollutes water as specified in ORS 468.943 or ORS 468.946 is subject to criminal prosecution.

3. Duty to Mitigate

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

4. Duty to Reapply

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

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

5. Permit Actions

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

a. Violation of any term, condition, or requirement of this permit, a rule, or a statute;

b. Obtaining this permit by misrepresentation or failure to disclose fully all material facts; or

c. A change in any condition that requires either a temporary or permanent reduction or elimination of the authorized discharge.

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

6. Toxic Pollutants

The permittee shall comply with any applicable effluent standards or prohibitions established under Section 307(a) of the Clean Water Act for toxic pollutants within the time provided in the regulations that establish those standards or prohibitions, even if the permit has not yet been modified to incorporate the requirement.

7. Property Rights

The issuance of this permit does not convey any property rights of any sort, or any exclusive privilege.

8. Permit References

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

SECTION B. OPERATION AND MAINTENANCE OF POLLUTION CONTROLS

1. Proper Operation and Maintenance

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

2. Duty to Halt or Reduce Activity

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

3. Bypass of Treatment Facilities

a. Definitions

(1) "Bypass" means intentional diversion of waste streams from any portion of the treatment facility. The term "bypass" does not include nonuse of singular or multiple units or processes of a treatment works when the nonuse is insignificant to the quality and/or quantity of the effluent produced by the treatment works. The term "bypass" does not apply if the diversion does not cause effluent limitations to be exceeded, provided the diversion is to allow essential maintenance to assure efficient operation.

(2) "Severe property damage" means substantial physical damage to property, damage to the treatment facilities or treatment processes which causes them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of a bypass. Severe property damage does not mean economic loss caused by delays in production.

(1) Bypass is prohibited unless:

(a) Bypass was necessary to prevent loss of life, personal injury, or severe property damage;

(b) There were no feasible alternatives to the bypass, such as the use of auxiliary treatment facilities, retention of untreated wastes, or maintenance during normal periods of equipment downtime. This condition is not satisfied if adequate backup equipment should have been installed in the exercise of reasonable engineering judgement to prevent a bypass which occurred during normal periods of equipment downtime or preventative maintenance; and

(c) The permittee submitted notices and requests as required under General Condition B.3.c.

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

c. Notice and request for bypass.

(1) Anticipated bypass. If the permittee knows in advance of the need for a bypass, it shall submit prior written notice, if possible at least ten days before the date of the bypass.

(2) Unanticipated bypass. The permittee shall submit notice of an unanticipated bypass as required in General Condition D.5.

4. <u>Upset</u>

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

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

c. Conditions necessary for a demonstration of upset. A permittee who wishes to establish the affirmative defense of upset shall demonstrate, through properly signed, contemporaneous operating logs, or other relevant evidence that:

(1) An upset occurred and that the permittee can identify the causes(s) of the upset;

(2) The permitted facility was at the time being properly operated;

(3) The permittee submitted notice of the upset as required in General Condition D.5, hereof (24-hour notice); and

(4) The permittee complied with any remedial measures required under General Condition A.3 hereof.

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

5. Treatment of Single Operational Event

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

6. Overflows from Wastewater Conveyance Systems and Associated Pump Stations

a. Definitions

(1) "Overflow" means the diversion and discharge of waste streams from any portion of the wastewater conveyance system including pump stations, through a designed overflow device or structure, other than discharges to the wastewater treatment facility.

(2) "Severe property damage" means substantial physical damage to property, damage to the conveyance system or pump station which causes them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of an overflow.

(3) "Uncontrolled overflow" means the diversion of waste streams other than through a designed overflow device or structure, for example to overflowing manholes or overflowing into residences, commercial establishments, or industries that may be connected to a conveyance system.

b. Prohibition of overflows. Overflows are prohibited unless:

(1) Overflows were unavoidable to prevent an uncontrolled overflow, loss of life, personal injury, or severe property damage;

(2) There were no feasible alternatives to the overflows, such as the use of auxiliary pumping or conveyance systems, or maximization of conveyance system storage; and

(3) The overflows are the result of an upset as defined in General Condition B.4. and meeting all requirements of this condition.

c. Uncontrolled overflows are prohibited where wastewater is likely to escape or be carried into the waters of the State by any means.

d. Reporting required. Unless otherwise specified in writing by the Department, all overflows and uncontrolled overflows must be reported orally to the Department within 24 hours from the time the permittee becomes aware of the overflow. Reporting procedures are described in more detail in General Condition D.5.

7. Public Notification of Effluent Violation or Overflow

If effluent limitations specified in this permit are exceeded or an overflow occurs, upon request by the Department, the permittee shall take such steps as are necessary to alert the public about the extent and nature of the discharge. Such steps may include, but are not limited to, posting of the river at access points and other places, news releases, and paid announcements on radio and television.

8. Removed Substances

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

SECTION C. MONITORING AND RECORDS

1. Representative Sampling

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

2. Flow Measurements

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

3. Monitoring Procedures

Monitoring must be conducted according to test procedures approved under 40 CFR Part 136, unless other test procedures have been specified in this permit.

4. Penalties of Tampering

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

5. Reporting of Monitoring Results

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

6. Additional Monitoring by the Permittee

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

7. Averaging of Measurements

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

8. Retention of Records

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

9. Records Contents

Records of monitoring information shall include:

a. The date, exact place, time and methods of sampling or measurements;

b. The individual(s) who performed the sampling or measurements;

c. The date(s) analyses were performed;

d. The individual(s) who performed the analyses;

e. The analytical techniques or methods used; and

f. The results of such analyses.

10. Inspection and Entry

The permittee shall allow the Director, or an authorized representative upon the presentation of credentials to:

a. Enter upon the permittee's premises where a regulated facility or activity is located or conducted, or where records must be kept under the conditions of this permit;

b. Have access to and copy, at reasonable times, any records that must be kept under the conditions of this permit;

c. Inspect at reasonable times any facilities, equipment (including monitoring and control equipment), practices, or operations regulated or required under this permit, and

d. Sample or monitor at reasonable times, for the purpose of assuring permit compliance or as otherwise authorized by state law, any substances or parameters at any location.

SECTION D. REPORTING REQUIREMENTS

1. Planned Changes

The permittee shall comply with Oregon Administrative Rules (OAR) 340, Division 52, "Review of Plans and Specifications". Except where exempted under OAR 340-52, no construction, installation, or modification involving disposal systems, treatment works, sewerage systems, or common sewers shall be commenced until the plans and specifications are submitted to and approved by the Department. The permittee shall give notice to the Department as soon as possible of any planned physical alternations or additions to the permitted facility.

2. Anticipated Noncompliance

The permittee shall give advance notice to the Director of any planned changes in the permitted facility or activity that may result in noncompliance with permit requirements.

3. Transfers

This permit may be transferred to a new permittee provided the transferee acquires a property interest in the permitted activity and agrees in writing to fully comply with all the terms and conditions of the permit and the rules of the Commission. No permit shall be transferred to a third party without prior written approval from the Director. The permittee shall notify the Department when a transfer of property interest takes place.

4. Compliance Schedule

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

5. Twenty-Four Hour Reporting

The permittee shall report any noncompliance that may endanger health or the environment. Any information shall be provided orally (by telephone) within 24 hours, unless otherwise specified in this permit, from the time the permittee becomes aware of the circumstances. During normal business hours, the Department's Regional office shall be called. Outside of normal business hours, the Department shall be contacted at 1-800-452-0311 (Oregon Emergency Response System).

A written submission shall also be provided within 5 days of the time the permittee becomes aware of the circumstances. If the permittee is establishing an affirmative defense of upset or bypass to any offense under ORS 468.922 to 468.946, and in which case if the original reporting notice was oral, delivered written notice must be made to the Department or other agency with regulatory jurisdiction within 4 (four) calendar days. The written submission shall contain:

a. A description of the noncompliance and its cause;

b. The period of noncompliance, including exact dates and times;

d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance; and

e. Public notification steps taken, pursuant to General Condition B.7.

The following shall be included as information that must be reported within 24 hours under this paragraph:

a. Any unanticipated bypass which exceeds any effluent limitation in this permit.

b. Any upset which exceeds any effluent limitation in this permit.

c. Violation of maximum daily discharge limitation for any of the pollutants listed by the Director in this permit.

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

6. Other Noncompliance

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

a. A description of the noncompliance and its cause;

b. The period of noncompliance, including exact dates and times;

c. The estimated time noncompliance is expected to continue if it has not been corrected; and

d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance.

7. Duty to Provide Information

The permittee shall furnish to the Department, within a reasonable time, any information that the Department may request to determine compliance with this permit. The permittee shall also furnish to the Department, upon request, copies of records required to be kept by this permit.

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

8. Signatory Requirements

All applications, reports or information submitted to the Department shall be signed and certified in accordance with 40 CFR 122.22.

9. Falsification of Information

A person who supplies the Department with false information, or omits material or required information, as specified in ORS 468.953 is subject to criminal prosecution.

10. Changes to Indirect Dischargers - [Applicable to Publicly Owned Treatment Works (POTW) only]

The permittee must provide adequate notice to the Department of the following:

a. Any new introduction of pollutants into the POTW from an indirect discharger which would be subject to section 301 or 306 of the Clean Water Act if it were directly discharging those pollutants and;

b. Any substantial change in the volume or character of pollutants being introduced into the POTW by a source introducing pollutants into the POTW at the time of issuance of the permit.

c. For the purposes of this paragraph, adequate notice shall include information on (i) the quality and quantity of effluent introduced into the POTW, and (ii) any anticipated impact of the change on the quantity or quality of effluent to be discharged from the POTW.

11. Changes to Discharges of Toxic Pollutant - **[Applicable to existing manufacturing, commercial, mining, and silvicultural dischargers only]**

The permittee must notify the Department as soon as they know or have reason to believe of the following:

a. That any activity has occurred or will occur which would result in the discharge, on a routine or frequent basis, of any toxic pollutant which is not limited in the permit, if that discharge will exceed the highest of the following "notification levels:

(1) One hundred micrograms per liter (100 µg/L);

(2) Two hundred micrograms per liter (200 µg/L) for acrolein and acrylonitrile; five hundred micrograms per liter (500 µg/L) for 2,4-dinitrophenol and for 2-methyl-4,6-dinitrophenol; and one milligram per liter (1 mg/L) for antimony;

(3) Five (5) times the maximum concentration value reported for that pollutant in the permit application in accordance with 40 CFR 122.21(g)(7); or

(4) The level established by the Department in accordance with 40 CFR 122.44(f).

b. That any activity has occurred or will occur which would result in any discharge, on a non-routine or infrequent basis, of a toxic pollutant which is not limited in the permit, if that discharge will exceed the highest of the following "notification levels":

(1) Five hundred micrograms per liter (500 µg/L);

(2) One milligram per liter (1 mg/L) for antimony;

(3) Ten (10) times the maximum concentration value reported for that pollutant in the permit application in accordance with 40 CFR 122.21(g)(7); or

(4) The level established by the Department in accordance with 40 CFR 122.44(f).

SECTION E. DEFINITIONS

1. BOD means five-day biochemical oxygen demand.

2. TSS means total suspended solids.

4. kg means kilograms.

5. m^3/d means cubic meters per day.

6. MGD means million gallons per day.

7. Composite sample means a sample formed by collecting and mixing discrete samples taken periodically and based on time or flow.

8. FC means fecal coliform bacteria.

9. Technology based permit effluent limitations means technology-based treatment requirements as defined in 40 CFR 125.3, and concentration and mass load effluent limitations that are based on minimum design criteria specified in OAR 340-41.

10. CBOD means five day carbonaceous biochemical oxygen demand.

11. Grab sample means an individual discrete sample collected over a period of time not to exceed 15 minutes.

12. Quarter means January through March, April through June, July through September, or October through December.

13. Month means calendar month.

14. Week means a calendar week of Sunday through Saturday.

15. Total residual chlorine means combined chlorine forms plus free residual chlorine.

16. The term "bacteria" includes but is not limited to fecal coliform bacteria, total coliform bacteria, and E. coli bacteria.

17. POTW means a publicly owned treatment works.

FINAL

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM (NPDES) PERMIT
Evaluation Review Report

Oregon Department of Environmental Quality
Northwest Regional Office
2020 SW Fourth Avenue, Suite 400
Portland, Oregon 97201
503-229-5263 FAX 503-229-6945

Permittee: City of Tillamook 210 Laurel Avenue Tillamook, OR 97141	**Plant Location:** 710 Fifth Street Tillamook, OR 97141
Type of Waste: Domestic Sewage	**Receiving Streams:** Trask River
Source Category: Major Domestic	**Proposed Action:** NPDES renewal
File Information: WQ-Tillamook County File No. 88665 Permit Application No. 990099	**Source Contact:** Donita Parks (503) 824-4155
Preparer: Beth Moore Water Quality Source Control Section Northwest Region 503-229-5586	**Date Prepared:** January 6, 2003

Overview of Proposed Action

The proposed action is to renew the NPDES permit for wastewater treatment and disposal facilities serving the City of Tillamook. The wastewater treatment plant is located at 710 Fifth Street in the City of Tillamook.

The facility provides secondary treatment, which serves a mainly residential and commercial community with some industry. The average population equivalent loading over the past two years was 8000 persons and the reported flows in May through October averaged 0.71 million gallons per day. The average dry weather design flow of these facilities is 1.06 million gallons per day.

Treatment Facility Description

The wastewater treatment plant is a rotating biological contactor (RBC) plant. In 1998, a new headworks facility and two additional primary clarifiers were added to the facility. The equipment for the headworks consist of a Parshall flume, a Pista® grit removal unit, a grit pump, a grit cyclone, a grit screw dewatering unit and a Channel Monster® grinder. After leaving the headworks distribution box, flow enters a channel system that splits flow between the new primary clarifiers by way of two slide gates. The inlet channel allows the velocity to be reduced and provides an even distribution over the surface of the primary clarifier. The function of the primary clarifiers is to remove as much solid matter and floating scum as possible prior to biological treatment.

Settleable solids gravitate to the bottom of the clarifier and are pulled into a sludge collection sump by a collector mechanism consisting of a series of flights. Floatable materials such as grease or scum are removed through a rotating scum trough, which drains to the sludge pumps. The sludge and scum are pumped to either the primary or secondary digester. Treated wastewater effluent from the primary clarifiers flows into the existing RBC units.

In 1980 two trains of RBC's were installed to provide secondary biological treatment. The RBC's are fixed-film biological reactors. The process consists of a rotating cylindrical drum mounted on a horizontal shaft. Micro-organisms become attached to the surface area of the media on the drum. These micro-organisms absorb mainly the soluble BOD fraction of the wastewater and convert it to more micro-organisms. The excess biomass sloughs off the drum and combines with the suspended solids in the wastewater, which then flows to the secondary clarifiers. Polymer is added after the RBC's and prior to the secondary clarifiers.

Two 45-foot-diamer circular secondary clarifiers were added in 1980. The suspended solids in the wastewater and the excess biomass from the RBC's is removed in the secondary clarifiers. Effluent from the clarifiers is disinfected in the chlorine contact chamber and dechlorinated with a sodium bisulfite solution prior to discharge. The treated effluent from the facility is discharged to the Trask River at river mile 1.9.

Biosolids Management

Sludge digestion is provided by a two-stage anerobic digester system. Sludge from the primary and secondary clarifiers are removed to the primary digester. After digestion, the sludge is transferred to the secondary digester. The biosolids from the secondary digester are stabilized and land applied in accordance with a biosolids management plan and land application plan. Or the biosolids are transported to the Port of Tillamook.

Groundwater

There are no known or suspected environmental impacts on groundwater at this facility. No monitoring is proposed in the NPDES permit renewal.

Storm Water

The NPDES Storm Water discharge permit 1200-S for the facility was terminated August 12, 1999 because the catch basin collecting the run off water was sealed to stop all direct flow to the Trask River.

Compliance History

In the previous NPDES permit (Permit No. 101239), new water quality based permit limits for chlorine were established. The wastewater treatment facility was not able to meet the water quality based effluent chlorine limits when the permit was issued. The Department and the City of Tillamook entered into Stipulation and Final Order (SFO) No. WQMW-NWR-94-183, which included interim limits and a schedule for complying with the chlorine limits. The date for achieving compliance was July 1997. By April 1997 the dechlorination system was installed and by September 1997 the dechlorination system was on line and maintaining the permit limits.

In March 1996, the City of Tillamook entered into MAO No. WQMW-NWR-96-060. The MAO recognized that the City has not been able to meet the mass load limits during high flow events. Mass load limits for BOD and TSS and 85% removal efficiency were not being met (see notice of noncompliance WQ-NWR-95-010 and NON WQ-NWR-95-046). The MAO established a schedule to meet the permit limits. Under the terms and conditions of the MAO all permit conditions were required to be met by September 30, 1998. Although the facility has completed the headworks and primary clarifier modifications for this MAO in September of 1998, the facility continues to exceed the mass load limits in the wet weather season, has difficulty meeting effluent quality at other times, and has pH and chlorine limit violations. (see notice of noncompliance WQ-NWR-00-49, WQ-NWR-00-128,WQ-NWR-02-012 and WQ-NWR-02-048).

The City of Tillamook evaluated corrective measures under a Corrective Action Plan. The plan documented areas of improvement for chemical feed systems (pH, flocculation), RBC operation and maintenance, solids handling and disposal, strength and impact of the raw sewage influent and collection system, and odor control.

The Department has required that the Corrective Action Plan be expanded to improve reliability and redundancy. The City of Tillamook entered into MAO No. WQ/M-NWR-02-144 with the Department to complete the repairs and modifications in the expanded Corrective Action Plan by May 1, 2003 and to complete a Facilities Plan with additional engineering studies if a mass load increase or reduction in percent removal is requested.

COMPLIANCE WITH WATER QUALITY STANDARDS

Total Maximum Daily Load (TMDL)

DEQ is required by the Federal Clean Water Act, Section 303(d) to identify all water bodies in Oregon that do not meet water quality standards. Once identified, the stream segments are placed on DEQ's "303(d) list". Twelve river segments were listed for temperature and 15 river segments and the bay were listed for bacterial contamination in the Tillamook Bay Watershed. This listing means that the standards that protect the most sensitive beneficial uses of the water body are not being met and the Department is required to establish a plan with goals and pollution control targets for improving water quality.

The plan the Department developed is the Tillamook Bay Watershed Total Maximum Daily Load (TMDL). A TMDL is total amount of pollutant (from all sources) that a water body can receive without violating water quality standards. Waste load allocations are portions of the total load that are allotted to the discharge from the City of Tillamook's wastewater treatment facility and other point sources of pollution. The waste load allocations are used to establish effluent permit limits.

The Tillamook Bay Watershed TMDL plan was placed on public notice in February 2001 and a public hearing was held on March 22, 2001. The Department has reviewed the comments and revised the plan. The Plan was sent to EPA for approval. EPA accepted the TMDL on July 31, 2001.

The facility discharges to the Trask River, which is part of the Tillamook Bay watershed. The Tillamook Bay watershed is listed as water quality limited for bacteria and temperature. The facility is required to meet specific waste load allocations based on the Tillamook Bay Watershed Total Maximum Daily Load (TMDL).

Anti-degradation *(OAR 340-041-0026(1)(a))*

An anti-degradation review must be made for the NPDES permit renewal. The proposed wastewater treatment facility discharges to a water quality limited water body but the permit incorporates a temperature management plan and permit limits for thermal load and fecal coliform based on the assigned waste load allocation from the TMDL to maintain water quality standards. The proposed permit does not include for an increase in mass load limits from the previous permit.

Mixing Zone

The proposed mixing zone is the same as the previous permit. The mixing zone is defined as that portion of Trask River, which extends 100 feet downstream and 50 feet upstream of the point of discharge. The zone of initial dilution as defined includes that portion of the Trask River which extends 10 feet downstream and 5 feet upstream of the point of discharge. The Department conducted a mixing zone survey in July 1999. The data is sparse and was taken at a location further downstream than the assigned mixing zone. The data was used to estimate the dilution using the following equations.

$(Ce-Cr)/(Czid - Cr) = Dzid$ and $(Ce-Cr)/(Cmz - Cr) = Dmz$

where:

Ce = Concentration of the effluent

Cr = Concentration of the river

Czid or C mz = Concentration near the zone of initial dilution or edge of the mixing zone

Dzid or Dmz = Dilution near the zone of initial dilution or edge of the mixing zone

A dilution of 100 at the edge of the mixing zone is used in the reasonable potential analysis for ammonia, metals and temperature. The dilution at the edge of the zone of immediate (ZID) dilution was determined to be 3.0.

Parameter	Units	Effluent	20 ft d/s of Outfall	200 ft d/s of Outfall	Tillamook Boat Ramp	Dilution ZID	Dilution MZ
		Ce	Czid	Cmz	Cr	Dzid	Dmz
Potassium (total recoverable)	mg/L	17.1	6.78	0.818	0.753	2.7	251.5
Sodium (total recoverable)	mg/L	27.7	14.3	5.6	5.08	2.5	43.5
Chloride	mg/L	32	13	4.7	4.5	3.2	137.5
Sulfate (by Ion Chromatography)	mg/L	19.7	7.14	3.14	3.06	4.1	208.0
Total Dissolved Solids	mg/L	245	121	74	71	3.5	58.0
Field Conductivity	umhos/cm	418	200	94	90	3.0	82.0
					Std dev	0.6	84.7
					median	3.1	109.8

Due the tidal influence, the actual dynamics of the mixing zone are more complex than the July 1999 mixing zone survey was able to capture. The Department believes further information is appropriate to refine the measures of the dilution and size of the mixing zone and zone of initial dilution. Therefore, Schedule C of the proposed permit requires the City of Tillamook to perform a mixing zone study to evaluate the mixing characteristics of the effluent plume and to evaluate the protection of beneficial uses. The mixing zone for this permit will be reviewed by the Department after the mixing zone study is completed.

Ammonia

Water quality standards for ammonia (NH_3-N) are based on preventing both acute and chronic toxicity. A Reasonable Potential Analysis was performed to determine whether the discharge has the potential to cause or contribute to an ammonia criteria violation. Reasonable Potential Analysis for ammonia takes into consideration effluent ammonia data, effluent and ambient water temperature, pH, dilution and the presence or absence of salmonids in the receiving water. The reasonable potential analysis using the following ammonia data is shown below: a pH of 7.4 an ambient temperature ranging from 57.4°F(14.1 °C) to 68°F (20°C), a dilution of 3.0 at the zone of initial dilution, and a dilution of 100 at the mixing zone.

PARAMETER	Ammonia Data from the Facility			Maximum Effluent Conc.	Background Conc.	Maximum Conc. at ZID	Maximum Conc. at MZ	WQ Criteria 1 Hour (CMC)	WQ Criteria 4 Day (CCC)	REASONABLE POTENTIAL ?	
	No. of Ammonia Samples	Highest Conc.	Coef. of Variance							ACUTE	CHRONIC
		mg/L		mg/L	mg/L	mg/L	mg/L	mg/L	mg/L		
AMMONIA@ 14.1 °C	7	14.50	0.62	36.25	0.06	11.369	0.422	13.97	1.8	NO	NO
AMMONIA@ 20 °C	7	14.50	0.62	36.25	0.06	11.369	0.422	13.57	1.23	NO	NO

The reasonable potential analysis indicates that the facility does not have a reasonable potential to contribute to an in-stream excursion of freshwater ammonia. The conductivity data for the Trask River suggests that saltwater ammonia limits would not be appropriate.

Chlorine

The dechlorination process removes the total combined chlorine residual from disinfection with chlorine. The proposed permit includes the same permit limits as the previous permit. The water quality standards for chlorine are based on preventing both acute and chronic toxicity. The acute chlorine standard is 0.019 mg/L and the chronic chlorine standard is 0.011 mg/L. Once a mixing zone study is conducted this limit may change to reflect the amount of chlorine residual that would cause a water quality violation.

Fecal Coliform

The new permit limits are based on the seasonal allocations in the TMDL and measured at Outfall 001 prior to dilution. Discharges to the river were given an allowance for decay based on temperature and velocity in the rivers. The limits do not exceed the criterion set for recreational contact. The criterion is based on the fecal coliform standard used by DEQ prior to 1996.

Metals

The EPA method of reasonable potential analysis 40 CFR122.44(d)(1) was performed to evaluate whether the discharge has the reasonable potential to cause or contribute to an in-stream excursion above the allowable acute water quality criterion.

Only one data set from July 1999 is available for effluent and background water quality. Because the analysis was conducted with a single data point, the effluent data is multiplied by conservative statistical multiplier of 13.2 to estimate the maximum effluent concentration.

The result may be an artificially high effluent concentration on which the rest of the analysis is based. The Department has found that the maximum effluent concentrations used in the analysis often exceed the observed concentrations based on further sampling. Therefore the Department believes it is appropriate to require additional monitoring of this permit.

Although the results seem to indicate that there is a reasonable potential to cause or contribute to an in-stream excursion for some metals, the reasonable potential methodology is limited for small sample sizes. The amount of data and the detection level of the analytical results used in the reasonable potential analysis play an important part in determining whether there is an actual in-stream excursion above the allowable water quality criterion. The analysis is highly dependent on conservatively assumed distribution when less than 10 data points are available and without lower levels of detection the results of the reasonable potential analysis is artificially high.

PARAMETER	Effluent Data (July 1999)		Coef. of Varience	Max Effluent Conc.	Trask River (July 1999)	Max Conc. at ZID	Max Conc. at MZ	WATER QUALITY Criteria		REASONABLE POTENTIAL?	
	# of Samples	Highest Conc. µg/l		µg/l	Background Conc. µg/l	µg/l	µg/l	1 Hour Acute (CMC) µg/l	4 Day Chronic (CCC) µg/l	ACUTE	CHRONIC
ARSENIC V	1	4.20	0.60	55.44	1.00	19.15	1.54	850.00	48.00	NO	NO
ARSENIC III	1	4.20	0.60	55.44	1.00	19.15	1.54	360.00	190.00	NO	NO
CADMIUM *	1	0.25	0.60	3.30	0.05	1.13	0.08	1.01	0.44	YES	NO
CHROMIUM VI	1	1.08	0.60	14.26	0.10	4.82	0.24	16.00	11.00	NO	NO
CHROMIUM III *	1	1.08	0.60	14.26	0.10	4.82	0.24	647.80	77.21	NO	NO
TOTAL CHROMIUM * †	1	1.08	0.60	14.26	0.10	4.82	0.24	663.80	88.21	NO	NO
COPPER *	1	27.20	0.60	293.04	0.15	97.78	3.08	5.70	4.23	YES	NO
IRON ‡	1	943.00	0.60	12447.60	0.10	4149.27	124.58	2000.00	1000.00	YES	NO
LEAD *	1	1.50	0.60	19.80	0.35	6.80	0.50	17.99	0.70	NO	NO
NICKEL *	1	2.72	0.60	35.90	1.21	12.77	1.56	512.84	56.94	NO	NO
SILVER *	1	6.95	0.60	91.74	0.10	30.65	1.02	0.51	0.12	YES	YES
ZINC *	1	46.10	0.60	608.52	1.28	203.69	7.35	42.19	38.22	YES	NO

NOTES

* - Hardness Dependant 30 mg/L (based on Trask River Data upper 75th percentile)

Dilution at the ZID = 3.0

Dilution at the edge of the MZ = 100

‡ - No acute standard. The CMC is estimated as 2X's the CCC.

Bold type indicates use of ½ the detection limit

1/2 the WQ Criteria

Temperature

There are 12 river segments in the Tillamook Bay Watershed that do not meet the state standards for temperature. The temperature TMDL is based on what salmonid fish need to live and reproduce in the rivers. The waste load allocations established in the TMDL apply those sources, which discharge into the rivers in the Tillamook Bay Watershed. A temperature allocation was provided for this treatment facility because the treatment facility discharges into the Trask River. The allocations in the TMDL are based on rearing and spawning temperature criteria. The ODFW fish habitat distribution showed Chum salmon spawning in the Trask River in the in the reach that the facility discharges.

ODFW has recently modified its distributional data for chum salmon in the lower rivers of the Tillamook Bay Watershed. In prior versions, chum were represented as spawning from river mile zero up into freshwater areas of the subbasin. The current version indicates spawning in the lower Trask River above river mile 4.5. Because the outfall for the City of Tillamook is located at river mile 1.9, the 55°F(12.8°C) water quality criterion will not be applied.

The Temperature criterion are also based on the presence of Threatened and Endangered species. No measurable increase in the surface water temperature is allowed in stream segments that contain Threatened and Endangered species if that increase would impair the biological integrity of the threatened and endangered population. Coho is listed as a threatened species on the Oregon Coast. The OFDW fish habitat distribution shows Coho salmon rearing and migration on the Trask River in the area of the outfall. Therefore no measurable increase in temperature is allowed above the natural background temperatures, which may cause an adverse effect on fish. A significant increase in temperature is defined as a temperature change of 0.25°F (0.14°C) or greater.

The waste load allocation in the TMDL were derived based on the assumption that ¼ of the 7Q10 river flow was available for dilution. The waste load allocation can also be based on an appropriated proportion related to the mixing zone. The discharge occurs in a tidally influenced portion of the river which may influence available dilution. Greater dilution may occur due to tidal influence. The permit contains a requirement for a mixing zone study to be conducted.

The proposed permit contains excess thermal load limits that are based on a mixing zone study that was conducted by the Department. The survey was conducted during low tide.
The following equation was used to determine if the effluent temperature from the facility would cause a measurable increase in the natural background temperature and/or the 64°F water quality criterion.

$$T_{MZ}=(T_E+ DTs)/(D+1)$$

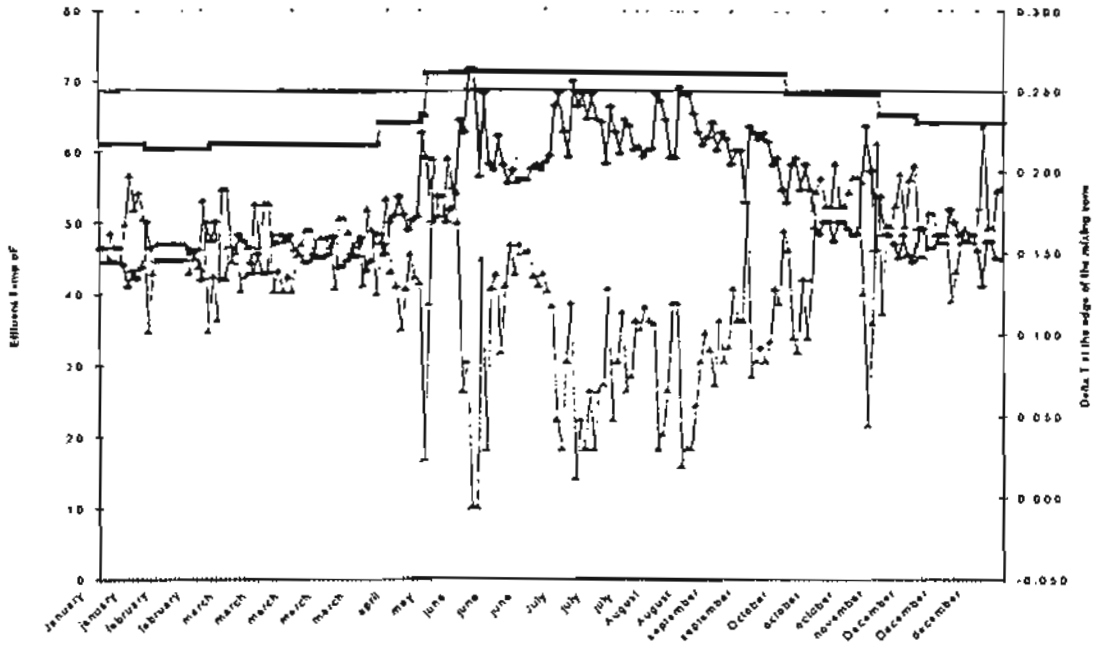
Where:

T_{MZ}= temperature of the stream at the edge of the mixing zone

T_E = effluent temperature = maximum monthly temperature from 1999-2001
May to September maximum monthly temperature = 71°F

T_S = Temperature (°F) data in the Trask River from DEQ database at HWY 101 Bridge

D = dilution at the edge of the mixing zone= 100



The analysis shows that with the background Trask River temperature (dark blue line) and a dilution of 100 the change in temperature (orange line) due to the effluent discharge (pink line) is below 0.25°F (light blue line).

The analysis shows that the discharge temperature does not cause a 0.25°F (0.14°C) increase in the background temperature at the edge of the mixing zone. The maximum effluent temperature of 71°F from the facility data (1999-2001) is below the incipient lethal temperatures of 77°F (25°C).

$$T_{MZ} = (T_E + DT_S)/(D+1)$$

Where:
T_{MZ}= temperature of the stream at the edge of the mixing zone
T_E = effluent temperature
May to September maximum monthly temperature = 71°F
T_S = 64°F in the Trask River.
D = dilution at the edge of the mixing zone= 100

PARAMETER	Units	Effluent	Edge of Mixing Zone
Temperature	° F	71	64.1

The analysis shows that even with high temperatures of 71°F from the effluent there does not appear to be a reasonable potential for this discharge to cause or contribute to any temperature standard violation.

A temperature management plan is required for sources that discharge to surface waters that are not meeting the state standards for temperature. The temperature management plan includes a mixing zone study and temperature monitoring. The mixing zone study will be used to determine if any modification of dilution is warranted. Knowing the effect of the mixing zone is important for identifying solutions. The temperature management plan will identify the options for thermal controls to be in effect at the end of 5 years if necessary.

These options will be further developed and evaluated in conjunction with the facilities plan. The temperature management plan did contain a requirement to conduct field observations with ODFW at least once within the permit cycle to determine the viability of the spawning habitat and the application of the 55°F(12.8°C) water quality criterion. However, the 55°F(12.8°C) water quality criterion will not be applied.

The Permit

Proposed Permit Limitations (Schedule A)

The proposed permit sets limits on the wastes disposed from this facility. Limits on the current discharges from the facility include Biochemical Oxygen Demand (BOD_5), Total Suspended Solids (TSS), pH, Fecal Coliform, chlorine, and excess thermal load.

BOD/TSS limitations

The mass load limits and removal efficiency for BOD/TSS will remain the same as in the previous discharge permit. Per OAR 340-041-0120(a)(4), the weekly average is 1.5 times the monthly average and daily maximum is 2 times the monthly average. The mass load limits are calculated as follows

May 1 – October 31
Monthly average mass load limit: 1.06 MGD X 20 mg/L X 8.34 = 177 lbs
Weekly average mass load limit: 1.06 MGD X 20 mg/L X 8.34 X 1.5 = 265 lbs
Daily mass load limit: 1.06 MGD X 20 mg/L X 8.34 X 2= 354 lbs

November 1 to April 30
Monthly average mass load limit: 1.06 MGD X 30 mg/L X 8.34 = 265 lbs
Weekly average mass load limit: 1.06 MGD X 30 mg/L X 8.34 X 1.5 = 398 lbs
Daily mass load limit: 1.06 MGD X 30 mg/L X 8.34 X 2= 530 lbs

pH

The current proposed limits shown below are the same as the previous permit. This limit is based on Federal wastewater secondary treatment regulations (40 CFR § 133.102) for sewage treatment facilities and is applied to the majority of the NPDES permits in the state. The higher pH limits assume there is a mixing zone for dilution that will allow the pH to meet the North Coast Basin Standard of 6.5 to 8.5. DEQ believes establishing an effluent pH permit limit of 6.0-9.0 will not cause or contribute to a violation of the water quality standard for pH.

Fecal Coliform

The new permit limits are based on the seasonal allocations in the TMDL and measured at Outfall 001 prior to dilution.

From November 1 to April 30, the permit limit for fecal coliform is a monthly geometric mean of 49 organisms per 100 ml with no single sample exceeding 151 organisms. The permit allows for a violation to be expunged by analyzing consecutive samples. If a single sample exceeds 151, then five consecutive samples shall be taken at four hour intervals beginning as soon a practicable (preferably within 28 hours) after the original was taken. If the geometric mean of the five re-samples is less than or equal to 49 organisms per 100 ml a violation shall not be triggered

From May 1 to October 31,the permit limit for fecal coliform is a monthly geometric mean of 200 organisms per 100 ml with no single sample exceeding 400 organisms. The permit allows for a violation to be expunged by analyzing consecutive samples. If a single sample exceeds 200, then five consecutive samples shall be taken at four hour intervals beginning as soon a practicable (preferably within 28 hours) after the original was taken. If the geometric mean of the five re-samples is less than or equal to 200 organisms per 100 ml a violation shall not be triggered

Chlorine

The proposed permit includes the same permit limits as the previous permit. The water quality standards for chlorine are based on preventing both acute and chronic toxicity.

Temperature

Thermal Load (BTU/day) = $C_p \times p_w \times \Delta T \times Q_e \times 1{,}000{,}000$

Where C_p = specific heat of water = 1 BTU/lb°F

p_w = density of water = 8.34 lb/gal

ΔT = Effluent Temperature – ambient temperature criterion = 71°F –64°F

Q_e = Effluent flow = 1.06 Mgd

The monthly average thermal load was calculated based on the average dry weather daily flow and the temperature difference between the maximum effluent temperature 71°F and the receiving stream criterion of 64°F.

1.06 Mgd x 1 BTU/lb°F x 8.34 lb/gal x (71°F –64°F) = 62 million BTU/day

For the maximum daily heat load the average dry weather daily flow is multiplied by a flow ratio. The flow ratio was derived using the 1999-2001 discharge monitoring report flow data. The flow ratio is the maximum daily flow divided by the average monthly flow. The flow ratio is 2.28

1.06 Mgd x 2.28 x 1 BTU/lb°F x 8.34 lb/gal x (71°F –64 °F) = 141 million BTU/day

Minimum Monitoring and Reporting Requirements (Schedule B)

Periodic monitoring and reporting is required for this facility. The frequency for monitoring generally follows a monitoring matrix developed by the Department in 1992. The monitoring parameters are summarized below.

Influent

Waste strength (based on pH, BOD_5 and TSS) shall be monitored entering the treatment facility. Other parameters include flow and temperature.

Effluent

The discharge (Outfall 001) to the Trask River will be monitored for, waste strength (pH, BOD_5 and TSS) and disinfection (E-coli). Other water quality parameters include total chlorine residual, and temperature.

Biosolids

The biosolids monitoring requirements established in the permit are consistent with the need to document compliance with State and federal regulations.

Compliance Conditions (Schedule C)

The proposed permit contains seven compliance conditions as follows:

1. If a permittee requests mass load limits based on average wet weather design flow then they must also complete a mandatory inflow identification and removal program as established in the rule. This condition defines the content of an inflow reduction plan

2. The permittee is required to verify that all overflow points are identified and alarmed so that the frequency and duration of raw sewage overflows can be documented. City is required to identify any pump stations or other points that may overflow no later than 90 days after the permit is issued. The overflow points that are identified will be included as emergency overflows on page one of the permit during the next permit renewal.

3. As a major source the facility is required to meet the requirements for the pretreatment program, which includes conducting an initial survey. The industrial waste survey is expected no later than 6 months from the permit issuance date.

4. The permittee is required to do a mixing zone study.

5. Requires annual reports for the temperature monitoring plan.

6. This condition requires the permittee, in the event that any compliance date is missed, to notify the Department no later than 14 days following the lapsed compliance date.

Special Conditions (Schedule D)

The proposed permit includes 8 special conditions as follows:

1. The permittee is required to have in place an adequate plan for preventing and handling spills and unplanned discharges.

2. This condition requires the management of biosolids in accordance with a Department approved biosolids management plan and site authorization letters.

3. This condition allows the permit to be reopened and modified if necessary for biosolids related issues.

4. A bioassay test, which is referred to as the whole effluent toxicity test, is required semi-annually for the first year and as part of the permit renewal. The protocols for conducting the acute and chronic testing are provided in this condition.

5. A priority pollutant scan is required as part of the permit renewal.

6. The permittee is required to comply with OAR 340-49, "Regulations pertaining to Certification of Wastewater System Operator Personnel" by ensuring that the wastewater treatment and collection systems are supervised by operators certified at the grade levels specified on page 1 of the permit.

7. The permittee is required to notify the Department's Regional office of any malfunction so corrective action can be coordinated between the permittee and the Department.

8. The permittee is required to have in place an adequate plan for preventing and handling spills and unplanned discharges.

General Conditions (Schedule F)

The General Conditions applicable to all holders of NPDES permits are listed.
These are divided into the following sections: Standard Conditions; Operation and Maintenance of Pollution Controls; Monitoring and Records; Reporting Requirements; and Definitions.

file

Oregon

Theodore Kulongoski, Governor

Department of Environmental Quality
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February 24, 2003

MAYOR ROBERT McPHEETERS
CITY OF TILLAMOOK
210 LAUREL AVENUE
TILLAMOOK, OR 97141

RE: City of Tillamook
Amendment #1
Mutual Agreement and Order
No. WQ/M-NWR-02-144
Tillamook County

Dear Mayor McPheeters:

Mutual Agreement and Order (MAO) No. WQ/M-NWR-02-144 negotiated between the City of Tillamook and the Department of Environmental Quality (DEQ), currently includes the following schedule for facility upgrades:

Paragraph 7A.(1) By February 1, 2003, submit a draft updated plant operations and maintenance manual meeting the DEQ guidelines.

Paragraph 7A.(2) By May 1, 2003, complete the construction of the modifications to the facilities and repairs to the wastewater treatment facilities, including the RBC air piping, as required in the approved plans and specifications for the expanded Corrective Action Plan.

Paragraph 7A.(3) Within 60 days after receiving comments from DEQ on the draft updated plant operations and maintenance manual, submit a complete final version of the operations and maintenance manual reflecting city staff and DEQ review comments.

Paragraph 7A.(4) By December 31, 2003, submit a complete facilities plan. The facilities plan may include a request for a mass load increase in the permit based on an engineering evaluation to demonstrate the design average wet weather flow. The engineering evaluation for a winter mass load increase must include a water quality analysis to show that the mass load increase will meet water quality standards and beneficial uses which apply. The facilities plan may include an engineering evaluation to demonstrate the maximum monthly flow at which 85% removal of BOD and TSS can be achieved.

Paragraph 7.C. Requiring the City to meet the following summer interim effluent limitations, measured as specified in the Permit, until May 1, 2003 when the

modifications and repairs as set forth in Paragraph 7.A.(2) will be complete.
(1) May 1-October 31

Parameter	Average Effluent Concentrations (mg/L)		Effluent Loadings (lbs)		
	Monthly	Weekly	Monthly Average	Weekly Average	Daily Maximum
BOD	25	37.5	221	332	442
TSS	25	37.5	221	332	442

Harry Ritter of Kennedy Jenks Consultants, who is the project consultant for the City of Tillamook, contacted David Mann of DEQ via e-mail on January 13, 2003 to request a change in the schedule for the construction schedule in Paragraph 7A.(1) & (2). In a letter dated January 28, 2003, the City formally submitted a request for an adjustment in the schedule. The following reasons were submitted for the requested amendment to the MAO.

1. Due to long equipment lead times, the project will require a longer contract time than originally anticipated to facilitate both delivery and installation of equipment.

2. With the alternative work schedule, the work requiring the temporary plant shutdown or treatment disruption would still be able to be accomplished prior to the May 1 change in permit limits.

The DEQ understands that the deviation and delay of the schedule dates in the MAO are caused by circumstances beyond the control and despite due diligence of the City of Tillamook. The DEQ concurs with the demonstration provided by the City and believes that changes in the schedule are needed to complete this work. Therefore, the DEQ approves the City's written request and MAO No. WQ/M-NWR-02-144 Paragraph 7 A. and C. are amended as follows:

Paragraph 7A.(1) For those items in the construction sequence at 50% completion, submit a section for the draft updated plant operations and maintenance manual meeting the DEQ guidelines.
Paragraph 7A.(2) By May 1, 2003, complete the construction of a new digester supernatant pump station and modifications and repairs for the RBCs as required in the approved plans and specifications for the expanded corrective action plan so as to meet applicable permit effluent limits.
Paragraph 7A.(3) By July 31, 2003, complete the construction of the modifications to the facilities and repairs to the wastewater treatment facilities as required in the approved plans and specifications for the expanded corrective action plan.
Paragraph 7A.(4) Within 60 days after receiving comments from DEQ on the draft updated plant operations and maintenance manual, submit a complete final version of the operations and maintenance manual reflecting city staff and DEQ review comments.

Paragraph 7A.(5) By December 31, 2003, submit a complete facilities plan. The facilities plan may include a request for a mass load increase in the permit based on an engineering evaluation to demonstrate the design average wet weather flow. The engineering evaluation for a winter mass load increase must include a water quality analysis to show that the mass load increase will meet water quality standards and beneficial uses which apply. The facilities plan may include an engineering evaluation to demonstrate the maximum monthly flow at which 85% removal of BOD and TSS can be achieved.

Paragraph 7.C. Requiring the City to meet the following summer interim effluent limitations, measured as specified in the Permit, until May 1, 2003 when a new digester supernatant pump station and the modifications and repairs for the RBCs as set forth in Paragraph 7.A.(2) will be complete.

(1) May 1-October 31

Parameter	Average Effluent Concentrations (mg/L)		Effluent Loadings (lbs)		
	Monthly	Weekly	Monthly Average	Weekly Average	Daily Maximum
BOD	25	37.5	221	332	442
TSS	25	37.5	221	332	442

Please attach this Amendment #1 to your original, signed MAO. Other than amended paragraphs, all other requirements and agreements of the original MAO remain in effect. If you have any questions about these documents, please call: Beth Moore at (503) 229-5586.

Sincerely,

Neil Mullane

Neil Mullane
Northwest Region Division Administrator

Cc: DEQ Office of Compliance and Enforcement
DEQ File No. 88665
Mark Gervasi, City of Tillamook
Arley Sullivan, City of Tillamook
Donita Parks, City of Tillamook

BEFORE THE ENVIRONMENTAL QUALITY COMMISSION

OF THE STATE OF OREGON

IN THE MATTER OF:	)	MUTUAL AGREEMENT
	)	AND ORDER
CITY OF TILLAMOOK	)	NO. WQ/M-NWR-02-144
PERMITTEE	)	TILLAMOOK COUNTY
	)	

WHEREAS:

1. On January 6, 2003, the Department of Environmental Quality (Department or DEQ) issued National Pollutant Discharge Elimination System Permit Number 101239 (Permit) to the City of Tillamook (Permittee). The Permit authorizes the Permittee to discharge adequately treated wastewater into the Trask River, waters of the state, in conformance with the requirements, limitations and conditions set forth in the Permit.

2. Condition 1 of Schedule A of the Permit does not allow the Permittee to exceed the following waste discharge limitations after the Permit issuance date:

Outfall Number 001

(1) May 1-October 31

Parameter	Average Effluent Concentrations (mg/L)		Effluent Loadings (lbs)		
	Monthly	Weekly	Monthly Average	Weekly Average	Daily Maximum
BOD	20	30	177	265	354
TSS	20	30	177	265	354
FC/100	200	400			

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(2) November 1-April 30:

Parameter	Average Effluent Concentrations (mg/L)		Effluent Loadings (lbs)		
	Monthly	Weekly	Monthly Average	Weekly Average	Daily Maximum
BOD	30	45	265	398	530
TSS	30	45	265	398	530
FC/100	49	200			

(3) Year-Around

Parameters	Limitations
a. pH	Shall be within the range 6.0-9.0
b. BOD and TSS removal efficiency	Shall not be less than 85% monthly average
c. Chlorine	Shall not exceed: Daily maximum limit = 0.02 mg/L Monthly average limit = 0.01 mg/L
Excess Thermal Load	Shall not exceed a 7 day moving average of the daily maximums of 141 million BTU/day Shall not exceed a monthly average of 62 million BTU/day

2. During the time period the Permit has been in effect, Permittee has not been able to consistently meet the above BOD and TSS limits as well as effluent mass loading and percent removal limitations in violation of water quality standards ORS 468B.025(2). The Department sent Notice of Noncompliance nos. WQ-NWR-00-49,WQ-NWR-00-128, WQ-NWR-02-012 and WQ-NWR-02-0048 to Permittee for these violations.

3. Under MAO WQMW-NWR-96-060, Permittee completed a headworks-primary clarifier project in 1998 that has not corrected the facility's problems with meeting these limits. The project received a Negative Certification in November 2000. The Department

approved a Corrective Action Plan in June 4, 2001 for additional facilities that are intended to ensure permit compliance. The Department required that the Permittee expand the CAP to improve reliability and redundancy. Permittee is in the process of completing plans and specifications under the expanded Corrective Action Plan.

4. DEQ and the Permittee recognize that until Permittee completes the actions required by this Mutual Agreement and Order (MAO), Permittee will continue to violate the Permit and Oregon law as described in Paragraph 2.

5. The Department and Permittee recognize that the Environmental Quality Commission has the power to impose a civil penalty and to issue an abatement order for violations of conditions of the Permit. Therefore, pursuant to ORS 183.415(5), the Department and Permittee wish to settle those past violations referred to in Paragraph 2; and to limit and resolve the future violations referred to in Paragraph 4 in advance by this MAO as follows:

i) violations of applicable summer limitations until May 1, 2003, and

ii) violations of applicable winter limitations until May 1, 2004.

6. This MAO is not intended to limit, in any way, the Department's right to proceed against Permittee in any forum for any past or future violations not expressly settled herein.

NOW THEREFORE, it is stipulated and agreed that:

7. The Environmental Quality Commission shall issue a final order:

A. Requiring Permittee to comply with the following schedule:

(1) By February 1, 2003, submit a draft updated plant operations and maintenance manual meeting the DEQ guidelines.

(2) By May 1, 2003, complete the construction of the modifications to the facilities and repairs to the wastewater treatment facilities, including the RBC air piping, as required in the approved plans and specifications for the expanded Corrective Action Plan.

(3) Within 60 days after receiving comments from DEQ on the draft updated plant operations and maintenance manual, submit a complete final version of the operations and

maintenance manual reflecting city staff and DEQ review comments.

(4) By December 31, 2003, submit a complete facilities plan. The facilities plan may include a request for a mass load increase in the permit based on an engineering evaluation to demonstrate the design average wet weather flow. The engineering evaluation for a winter mass load increase must include a water quality analysis to show that the mass load increase will meet water quality standards and beneficial uses which apply. The facilities plan may include an engineering evaluation to demonstrate the maximum monthly flow at which 85% removal of BOD and TSS can be achieved.

B. Requiring the City, at all times, to properly operate and maintain the wastewater collection, treatment, and disposal systems as effectively as possible.

C. Requiring the City to meet the following summer interim effluent limitations, measured as specified in the Permit, until May 1, 2003 when the modifications and repairs as set forth in Paragraph 7.A.(2) will be complete.

(1) May 1-October 31

Parameter	Average Effluent Concentrations (mg/L)		Effluent Loadings (lbs)		
	Monthly	Weekly	Monthly Average	Weekly Average	Daily Maximum
BOD	25	37.5	221	332	442
TSS	25	37.5	221	332	442

D. Requiring the City to meet the following winter interim effluent limitations, measured as specified in the Permit, until May 1, 2004 or the Permit is modified based upon the engineering evaluations in 7.A.(4) whichever is earlier. The MAO may be amended as described in Paragraphs 10 or 11 if the Department determines, based on the engineering evaluations in 7.A.(4), that additional facilities are needed to meet water quality standards and beneficial uses which apply.

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(2) November 1-April 30:

Parameter	Average Effluent Concentrations (mg/L)		Effluent Loadings (lbs)		
	Monthly	Weekly	Monthly Average	Weekly Average	Daily Maximum
BOD	45	67.5	500	900	1200
TSS	45	67.5	500	900	1200
BOD and TSS removal efficiency			Shall not be less than 85% monthly average*		
*Note: When the monthly average flow exceeds 1.5 MGD the percent removal shall not apply.					

The Permittee shall operate the wastewater treatment facility as effectively as possible during those times when the inflow exceeds 1.5 mgd. All BOD/TSS sample results shall be reported on the Permittee's discharge monitoring report. The discharge monitoring report shall contain information on progress with influent monitoring.

E. Requiring Permitee, upon receipt of a written Penalty Demand Notice from the Deparment to pay the following civil penalties:

(1) $250.00 for each violation of the compliance schedule set forth in Paragraph 7.A.

(2) $100.00 for each violation of each daily or weekly average waste discharge limitation set for the in Paragraph7. C.and D.

(3) $ 500.00 or each violation of each monthly average waste discharge limitation set forth in Paragraph 7.C. and D.

8. If any event occurs that is beyond Permittee's reasonable control and that causes or may cause a delay or deviation in performance of the requirements of this MAO, Permittee shall immediately notify the Department verbally of the cause of delay or deviation and its anticipated duration, the measures that have been or will be taken to prevent or minimize the delay or deviation, and the timetable by which Permittee proposes to carry out such measures. Permittee

shall confirm in writing this information within five (5) working days of the onset of the event. It is Permittee's responsibility in the written notification to demonstrate to the Department's satisfaction that the delay or deviation has been or will be caused by circumstances beyond the control and despite due diligence of Permittee. If Permittee so demonstrates, the Department shall extend times of performance of related activities under this MAO as appropriate. Circumstances or events beyond Permittee's control include, but are not limited to acts of nature, unforeseen strikes, work stoppages, fires, explosion, riot, sabotage, or war. Increased cost of performance or consultant's failure to provide timely reports may not be considered circumstances beyond Permittee's control.

9. Regarding the violations set forth in Paragraphs 2 and 4 above, which are expressly settled herein without penalty, Permittee and the Department hereby waive any and all of their rights to any and all notices, hearing, judicial review, and to service of a copy of the final MAO herein. The Department reserves the right to enforce this MAO through appropriate administrative and judicial proceedings.

10. The terms of this MAO may be amended by the mutual agreement of the Department and Permittee.

11. The Department may amend the compliance schedule and conditions in this MAO upon finding that such modification is necessary because of changed circumstances or to protect public health and the environment. The Department shall provide Permittee a minimum of thirty (30) days written notice prior to issuing an Amended Order modifying any compliance schedules or conditions. If Permittee contests the Amended Order, the applicable procedures for conduct of contested cases in such matters shall apply.

12. This MAO shall be binding on the parties and their respective successors, agents, and assigns. The undersigned representative of each party certifies that he or she is fully authorized to execute and bind such party to this MAO. No change in ownership or corporate or partnership status relating to the facility shall in any way alter Permittee's obligations under this MAO,

unless otherwise approved in writing by DEQ.

13. All reports, notices and other communications required under or relating to this MAO should be directed to Beth Moore, DEQ Water Quality Northwest Regional Office, 2020 SW Fourth Ave., Portland, Oregon 97034 phone number (503) 229-5586.

14. Permittee acknowledges that it has actual notice of the contents and requirements of the MAO and that failure to fulfill any of the requirements hereof would constitute a violation of this MAO and subject Permittee to payment of civil penalties pursuant to Paragraph 6.D. above.

15. Any stipulated civil penalty imposed pursuant to Paragraph 6.D. shall be due upon written demand. Stipulated civil penalties shall be paid by check or money order made payable to the "Oregon State Treasurer" and sent to: Business Office, Department of Environmental Quality, 811 S.W. Sixth Avenue, Portland, Oregon 97204. Within 21 days of receipt of a "Demand for Payment of Stipulated Civil Penalty" Notice from the Department, Permittee may request a hearing to contest the Demand Notice. At any such hearing, the issue shall be limited to Permittee's compliance or non-compliance with this MAO. The amount of each stipulated civil penalty for each violation and/or day of violation is established in advance by this MAO and shall not be a contestable issue.

16. Providing Permittee has paid in full all stipulated civil penalties pursuant to Paragraph 15 above, this MAO shall terminate 60 days after Permittee demonstrates full compliance with the requirements of the schedule set forth in Paragraph 6A above.

PERMITTEE

12/30/02
Date

Robert McPheeters
Mayor
City of Tillamook

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DEPARTMENT OF ENVIRONMENTAL QUALITY

1/06/03
Date

Neil Mullane
Neil Mullane
Division Administrator

FINAL ORDER

IT IS SO ORDERED:

ENVIRONMENTAL QUALITY COMMISSION

1/06/03
Date

Neil Mullane
Neil Mullane, Division Administrator
Department of Environmental Quality
Pursuant to OAR 340-11-136(1)

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October 24, 2001.

DEQ Northwest Region
Mr. Bruce Henderson, Environmental Specialist
2020 SW Fourth Avenue
Portland, Oregon 97201

Dear Mr. Henderson:

Enclosed is the Biosolids Management Plan for the City of Tillamook Wastewater Treatment Plant. I hope it will meet all of the requirements necessary for our permit renewal process.

Our plan is currently under a plant review and we are working with the engineers to correct some operating problems. Due to the recent retirement of our head operator the completion of this Biosolids Management Plan was delayed. I sincerely apologize for any inconveniences this may have caused. I hope this document will meet with your approval and you find it in order. I would like to take this moment to thank both you and Beth Moore for your assistance. If you have any questions please don't hesitate to inquire. You may contact me at the plant at 503-842-4155.

Respectfully submitted,

Donita Parks
Lead Operator-TWWT Plant.

BIOSOLIDS MANAGEMENT PLAN

Treatment Facility Description:

The City of Tillamook owns and operates a municipal sewage collection system and a 1.06 MGD Rotating Biological Contactor process facility under NPDES permit number 101239. The wastewater plant is a secondary treatment process located at 710 Firth Street in Tillamook Oregon. It serves a population of 4400 with approximately 95% of the contribution coming from domestic sources, the remainder comprised of commercial and light industrial contributors. The industrial contribution being primarily the pre-treated waste of the Hampton Lumber Mill. The City accepts no septage of any kind at the facility unless deemed necessary by special request by the Department of Environmental Quality. Our biosolids undergo anaerobic digestion and our sewage effluent is disinfected with chlorine and de-chlorinated with sodium bisulfite prior to discharge into the Trask river.

LIQUID STREAM:

Raw sewage enters the head works of the treatment plant via two main sewer inceptors. (Exhibit #1) The sewage entering the facility is pumped to the top of the cities recently constructed 4.8 MGD primary clarifier basins via 3 raw sewage pumps which deliver the flow into the primary inlet channel through a 73 foot long inlet channel, through a parshall flume, electronic flow meter, grit removal system, and a shredder prior to its entry into the inlet troughs of this larger two cell primary structure. At the end of this channel the flow may be split by two gate valves and delivered to either this larger primary via inlet troughs or to another smaller existing primary clarifier (1.06 MGD capacity) during particularly low flow periods.

The liquid flow stream from either primary structure is then carried to the inlet of the Rotating Biological Contactors. Two, air driven parallel trains, each train containing two, two-stage shafts in sequence, (1.0 MGD capacity each). At the outfall of the RBCS polymer is added in the warmer months to aid settling. Here the flow travels through approximately 170 feet of piping before the flow stream can be split to discharge into one or both of two secondary clarifiers (1.0 MGD each). Chlorine for disinfection is added at the outfall of these circular clarifiers before the flow stream travels approximately 360 feet to one of two chlorine contact chambers, 100,000 gallons and 12,000 gallons, where it has contact

time for disinfection before it enters the downstream manhole where sodium bisulfite is injected for dechlorination and the flow enters the out fall pipe to the Trask River.

SOLIDS PROCESS:

Sludge and floatable skimmings are removed from the larger primary via two100 GPM progressive cavity sludge pumps to an anaerobic digester. Secondary sludge and skimmings can be pumped either into the inlet channel of the larger primary or to the sludge bins of the smaller primary for removal. Primary sludge and skimmings from the smaller primary clarifier are pumped to the anaerobic digester via a 50 gallon a minute positive displacement pump. After anaerobic digestion for 17-22 days at 35-38 degrees Celsius the sludge is carried over to a secondary unheated 100,000 gallon digester for storage and further stabilization. Decant water from this digester is returned to the wet well.

For sludge removal the secondary digested sludge is delivered to the loading dock via the sewage pumps to be top loaded into a 4700 gallon tanker truck where it is delivered to the biosolids application sites. There it is broadcast via back spreader at the prescribed rate of application. Or it is top loaded into a 7000 gallon tanker truck and delivered to the application site where it is pumped into the application truck and broadcast with a back spreader.

SOLIDS TREATMENT PROCESS:

All the solids stream generated at the treatment plant after primary and secondary processing is pumped to one 63,000 gallon capacity methane/ propane heated anaerobic digester. Average theoretical detention time in the digester is 17-22 days based on fluctuations in primary sludge pumping rates. Primary sludge is then carried over to an unheated 100,000 gallon secondary digester for further stabilization and storage. Last year, biosolids were held in this unheated digester till early spring when it could be applied to the application fields when weather permitted.

Digested secondary sludge has an average solids concentration of 3.0 to 5% total solids. Average volatile solids reduction range in 2000 was 50%. Average operating temperature of the digester is maintained at temperatures ranging from 35 to 38 degrees C. PH of the digester is tested three times weekly with the average pH of 7.0 being maintained year round with an occasional addition of bicarbonate of soda added if the pH drops to 6.8 which is rare. We normally generate enough methane to heat the digester year round.

BIOSOLIDS APPLICATION:

Tillamook land applied 304,600 gallons of stabilized biosolids last year. Application is seasonal, May 15- November 15. All biosolids produced are land applied to our existing sites. (Exhibits #3A, #3B, #4, #5) We apply to approximately 21.5 acres in Tillamook County. The city is currently under contract with Zwald Transport to move our biosolids to the application sites. Ken Zwald, driver, transports the biosolids in both a 7000 gallon tanker truck or a 4700 gallon tanker truck. He applies the biosolids via back spreader. Our crew goes out in advance and stakes out the perimeters and the set backs in the field we will be applying biosolids on. Application is calculated on actual spread coverage. The applier is instructed in the application rates and applies the biosolids at the prescribed rate.

Tillamook will be hauling biosolids to the Port of Tillamook receiving station this year for year round biosolids removal. (Exhibit #11) We are currently looking for suitable sites for land application.

SOLIDS STORAGE:

At our current pumping rate the city has storage for approximately 2.5 months of biosolids on site. This is using the combined total of both primary and secondary digesters for storage.

PATHOGEN REDUCTION:

Tillamook meets the pathogen reduction requirements as specified in 503.32(b)(3) for Class B biosolids. " Sewage sludge is treated in the absence of air for a specific mean cell residence time at a specific temperature. Values for the mean cell residence time and temperature shall be between 15 days at 35 to 55 degrees Celsius and 60 days at 20 degrees Celsius."

VECTOR ATTRACTION REDUCTION:

Our anaerobic digestion process resulted in an average volatile solids reduction of 50% in 2000. This meets vector attraction reduction requirements as specified in 503.33(b)(1). "The mass of volatile solids in the sewage sludge shall be reduced by a minimum of 38 percent (see calculation procedures in "Environmental Regulations and Technology-Control of Pathogens and Vector Attraction in Sewage Sludge", EPA-625/R-92/013, 1992, U.S. Environmental Protection Agency, Cincinnati, Ohio 45268)."

CERTIFICATION STATEMENT:

I certify under penalty of law that the information that will be used to determine compliance with the Class B pathogen requirements in Section 503.32(b), the vector attraction reduction requirements in Section 503.33(b)(1) and the site restrictions contained in Section 503.32(b)(5) for each site on which Class B biosolids was applied, was prepared under my direction and supervision in accordance with the system designed to ensure that qualified personnel properly gather and evaluate this information. I am aware that there are significant penalties for false certification including the possibility of fine and imprisonment.

Name and Title: _____

Date: _____

Signature: _____

CONTINGENCY PLANS

Any spill occurring at the sludge loading dock shall be cleaned up immediately with the under drain run-off going back to the wet well of the plant. We have only had minor splashing in the past but a supply of dehydrated lime is on hand if any larger spills should occur. We shall contain any larger spill and suck up as much of the spilled material as possible with our 1500 gallon capacity Vactor truck. We will then sprinkle the remainder with lime and wash the area down and suck up the excess water with the truck.

If a spill should occur in transporting the biosolids the matter shall be reported to the proper authorities immediately after every attempt is made to contain the spilled material and prevent contact with human and animal population. We shall do as much as possible to limit the impact on the environment. Once again our policy is to send a crew to use shovels, brooms and the vacuum of our Vactor truck to pick up as much of the sludge as possible and other vacuum trucks in the area shall be enlisted to help if the spill is too large for us to pick up. Lime shall then be applied to the area and when possible we shall wash down the area and vacuum up any residual.

If a breakdown of our anaerobic digester should occur we shall report the matter immediately to the DEQ, then haul the contents of the secondary digester to the Port facilities and put the primary digester contents into that tank till it could be properly treated and disposed of.

SAMPLING AND MONITORING:

In the fall soil samples from the fields where biosolids are applied are collected and analyzed (Exhibit #6) according to protocol published by Oregon State University. We continually monitor our digester pH and temperature as well as maintaining adequate detention time in our digester to maintain a quality class B biosolids for land application. We calculate our volatile solids reduction (Exhibit #2) throughout the process on a monthly basis. We record and monitor gallons land applied per field based on data obtained from our annual biosolids analysis (Exhibit #7A) and Annual Biosolids Production Worksheet. (Exhibit #7B, 7C). Biosolids data is also submitted on the monthly DMR. All information on our biosolids program is on file at the wastewater plant.

LAND APPLICATION PLAN:

The crops grown on the cities biosolids application sites are grass hay and oats. This year a spring crop as well as an over winter crop of oats are being raised. The crop is non-irrigated and is cut for silage used for feed for beef cattle. We applied biosolids in the early spring as well as in September to help supply the crop with valuable fertilizer as well as moisture and soil conditioners.

The City of Tillamook is currently looking for land to purchase for biosolids application. In an area predominantly dairy based and with the impact of the confined animal feed regulations it is becoming difficult to find privately owned dairy land sites suitable for biosolids application. We are presently looking at a much broader geographic area from which to potentially obtain sites for biosolids application. Application site characteristics are of critical importance in liquid biosolids application. The site must meet all slope and topography criteria as well as specific set backs, groundwater depth, and other site criteria established by regulatory guidelines. We are looking for local sites but see the need to encompass a larger area through out the entire county and in so doing will incur higher costs for transportation. Therefore we are looking into the feasibility of thickening to reduce mass as well as hauling costs. All potential sites for biosolids application within Tillamook County (Exhibit #8) will be evaluated and selection determined according to specific site requirements as contained in site approval letters issued by the Department of Environmental Quality. If the sites meet this criteria the city will then request site approval from the DEQ in writing.

The city will notify adjacent landowners to any new proposed biosolids application sites (Exhibit #10 C). The notification process will include information mailed or hand delivered related to application, nature of the product, site address, as well as names and telephone numbers of city contacts for further information or questions. (Exhibits #10 A, #10 B). Documentation of public notice will be submitted to the Department of Environmental Quality along with other information required for land application authorization. No biosolids will be applied without a site approval letter (Exhibit #9) issued by the Department or verbal permission from their representative. All liquid biosolids applied by the city or its representatives will be applied according to agronomic loading rates. Agronomic loading rates are based on the crop requirements and the total available nitrogen concentration of the biosolids calculated from results of the sludge analysis. This shall be reviewed annually. All effort will be made during application to prevent ponding, run off,

erosion and groundwater contamination. Biosolids will not be applied to land used for food crops. All animal grazing on said lands will be restricted for a minimum of thirty days after the last application of biosolids. Metal concentrations on sites will be monitored. Buffer zones will be designated, flagged and monitored. Sampling will take place at the end of each season to determine cumulative carryover of nitrogen and application rates will be adjusted to reflect this.

An annual report shall be submitted to the DEQ and EPA by February 19th of each year. The report describes quality and quantity of biosolids applied calculated on results from annual figures.

The city maintains records of the area of application, gallons applied, dates of application as well as all pertinent information in regards to the biosolids program and tests conducted.

EXHIBITS

1. Drawing: Plant Flow Schematic.
2. Volatile Solids Worksheet.
3. A. map of application area and vicinity.

   B. map of approximate application site.
4. Agreement. (two pages).
5. Description of Real Property.
6. Soil Analysis Report.
7. A. Biosolids analysis Data- Coffey Lab.

   B. Annual Biosolids Production. (three pages).

   C. Bruce's calculations.
8. Map of Tillamook County. (potential application area).
9. Department of Enviromental Quality Site Approval Letter. (5 pages).
10. A. Adjacent property information "Intent to Apply "letter.

    B. Biosolids Data Sheet Class" B."

    C. Property Owner/Controller Agreement.
11. Contract for Septage Receiving "Port Of Tillamook." (3 pages).

July 8, 2002.

DEQ Northwest Region
Mr. Bruce Henderson, Environmental Specialist
2020 SW Fourth Avenue
Portland, Oregon 97201

Dear Mr. Henderson:

Enclosed is the addendum to the Biosolids Management Plan for the City of Tillamook. Wastewater Treatment Plant. I hope it will meet requirements necessary to be added to our existing plan for our permit renewal process.

I hope this document will meet any requirements you have. Thank you for your support and assistance. If you have any questions please feel free to call me at the Wastewater Treatment Plant at 503-842-4155.

Respectfully submitted,

Donita Parks
Biosolids Co-ordinator.
Tillamook Wastewater Treatment Plant.

Addendum to : City of Tillamook Biosolids Management Plan.

I would like to submit the following corrections and additions to the City of Tillamook Biosolids Management Plan:

Beginning under the heading entitled: **Solids Process:**
Page 3. Correction on line 8.
A correction in the sentence that reads: After anaerobic digestion for 17-22 days at 35-38 degrees Celsius the sludge is carried over to a secondary unheated **100,000** gallon digester. This digester volume is incorrect. The capacity of this digester should be corrected to read **90,400** gallon digester.

Correction under the heading entitled: **Solids Treatment Process:**
Page 3. Correction on line 5.
A correction to the following sentence that reads: Primary sludge is then carried over to an unheated **100,000** gallon secondary digester for further stabilization and storage. This volume is incorrect and should be corrected to read **90,400** gallon secondary digester for further stabilization and storage.

Addition under the heading: **Biosolids Application:**
Page 4. Line 3.
An addition to: **(Exhibits #3A, #3B, #4,#5)**, here I would like to add to these exhibits: **#13,#14,#15, #16.**

Correction under this same heading: **Biosolids Application:**
Page 4. Correction on the original biosolids management document, line 4.
The sentence read as follows: We apply to **approximately 21.5** acres in Tillamook County. This sentence should be corrected to read: We apply to **approximately 29.5 acres** in Tillamook County.
This now includes approximately 8 acres for application the city just acquired in June of 2002.

Addition under the Heading: **Solids Storage:**
Page 4. To be added as a second paragraph under this heading. To follow as a new paragraph after the following sentence: **This is using the combined total of both primary and secondary digesters for storage.**
Here I would like to add the following paragraph: " **Tillamook is currently looking at the feasibility of using our older primary clarifiers for winter biosolids storage. This is an available option for storage of approximately 117,000 gallons of secondary digested biosolids. This idea is currently being reviewed for the City by the engineering firm of Kennedy-Jenks.** "

An addition under the heading: **Solids Storage:**
Page 4 of the original document. To be added as a third paragraph under this heading. To be added immediately following the last sentence of the newly added paragraph from the previous page which reads as follows: " **This idea is currently being reviewed for the City by the engineering firm of Kennedy-Jenks."** Here, following the fore mentioned sentence ending with **Kennedy-Jenks**., I would like to add the following paragraph: **"The city is currently using the available space afforded by these empty clarifiers for temporary short term storage of anaerobically digested secondary digester biosolids when necessary. This material is covered with a water cap and is monitored for pathogen re-growth. Before these solids are delivered to the application site they will be tested within 48 hours of application to determine they meet the pathogen reduction requirements as set forth in Section 503.32(ii) (b)(2)(i).**
These biosolids have met vector attraction reduction standards required under 40 CFR 503.33(a)(1) and are re-tested for fecal coliform as set forth in "Standard Methods for the Examination of Water and Wastewater, 18th edition, (ASTM, 1992). The samples from these secondary storage basins must meet the standard as established in Table 3 "Indicator Organisms and Pathogens Monitored" Under 40 CFR Part 503.

Correction to the Cities: **Certification Statement:**

Page 6. Correction on line 2.
Correction is in the Class B pathogen requirements in **Section 503.32 (b)**, this should be corrected to read: Class B pathogen requirements in **Section 503.32(b) (3) (anaerobic digestion)**,. In this same certification statement, on line 4 in the portion that reads " site restrictions contained in **Section 503.32(b)(5)** this should be corrected to read: site restrictions contained in **Section 503.32(b)(5) iv,v,vii.**

Here the following sentence should be added:

(See newly revised certification statement- exhibit #17)

Addition under the heading: **Contingency Plans**
Page 7 of original document. Second paragraph. Line 3 of the original document. In this sentence it states that: If a spill should occur in transporting biosolids the matter shall be reported to the proper authorities immediately after every attempt is made to contain the spilled material and prevent contact with human and **animal population**. The proper authorities to contact should have been mentioned here, at the end of this sentence. Here, after **animal population.,** I would like to add the following "**Oregon Emergency Response Service or OERS will be called at 1-800-452-0311. OERS will be notified if any spill should occur whether at the treatment plant or in transport. Oregon Emergency Response Service will see that the DEQ and all other necessary agencies are notified and that proper corrective action is taken.**

Correction under the heading: **Contingency Plans**
Page 7. Last paragraph. Line 2 of original the document.
This reads as follows: " If a breakdown of our anaerobic digester should occur we shall report the matter immediately to the **DEQ**, then haul the contents of the secondary digester to the Port facilities and put the primary digester contents into that tank until it could be properly treated and disposed of. Here , in this sentence, it should be corrected to read: " If a breakdown of our anaerobic digester should occur we shall report the matter immediately to **OERS and the DEQ**, then haul the contents of the secondary digester to the Port facilities and put the primary digester contents into that tank until it could be properly treated and disposed of.

Addition under the heading: **Contingency Plans**
page 7. Last paragraph. Line 4 of original document. Here, at the end of this paragraph ending with" until it could be **properly treated and disposed of**. Here I would like to add another two paragraphs. The first one as follows: "**The city has the ability to use the old primary clarifier basins for temporary storage of approximately 118,000 gallons of biosolids if an emergency should arise. The contents of one of either digesters could be transferred to these basins via our pumps and stored there on a temporary basis if necessary.**

Second new paragraph under heading : **Contingency Plan**
page 7 of original document. To be added directly following the previously added paragraph ending with the sentence ending with, " **stored there on a temporary basis if necessary**."
Here I would like to add : "**The city may, when necessary, need to remove quantities of lime stabilized or otherwise treated material drawn from digesters, wet wells or grit basins for the purpose of cleaning or repair of these units. This material will be screened, washed and treated sufficiently to demonstrate that these solids meet pathogen reduction requirements required under 40 CFR 503.32(b)(4) and vector attraction reduction standards required under 40 CFR 503.33(a)(1). Prior approval will be obtained from the DEQ before removal of any such material from the treatment plant and all records and information in regard to testing this material will be kept on file at the plant and reported on the City's annual biosolids report.**

Additions under the Heading **"Sampling and Monitoring."**
Page 7 of original document. Under the above heading at the end of the paragraph ending on line10 of the original document which ends with the following sentence: All information on our biosolids program is on file **at the wastewater plant**. The new paragraph beginning immediately after this sentence ending with: **at the wastewater plant**. Here I would like to add the following paragraphs:
"**The Cities secondary digester solids have been reported varying from 3% solids to 6.7 % solids. This variation in the solids content in our biosolids may in part be due to sampling and lack of a mixing in this digester. Care is taken to collect representative samples of the entire contents of this digester but more frequent testing may be needed. More samples of the contents are now being analyzed at more frequent intervals to help reduce discrepancies so a more representative solids content can be obtained prior to establishing application rates for the year.**
We are now also testing samples taken out of the sample line from the fill hose of the application truck at frequent intervals to get a better mixed value of the total solids hauled to each specific site. This number will be recorded and compared with the solids on our biosolids analysis and noted on our annual biosolids report.
We are at present attempting re-circulating in this secondary digester by using our pumps to pump the tanks contents out from the bottom of this digester back into the top to initiate some mixing action within this tank.

This should help establish a more uniform mix in this secondary storage tank.

Additions under the heading titled: **Land Application Plan:**

Page 8, Second paragraph, Line 9 of the original document to be included in the application site characteristics. The biosolids application site must meet all slope and topography criteria as well as specific set backs, groundwater depth, and other site criteria established **by regulatory guidelines.** Here at the end of this sentence after: **by regulatory guidelines**. I would like to add: " **The biosolids application sites should not be subject to flooding or excessive runoff. Every effort shall be made to avoid sandy, rapidly draining soils and areas were ponding could occur. At the time biosolids are land applied the minimum depth to permanent groundwater should be four feet and the minimum depth to temporary groundwater should be one foot.**

Additions under the title: **Land Application Plan**
Page 8, Second paragraph, Line 12 of original document.
"Therefore we are looking into the feasibility of thickening to reduce mass as well as **hauling costs.** Here, after hauling **costs,** I would like to add this sentence: " **In the future the city may need to purchase additional equipment and/or add additional facilities in order to accomplish this. Additional storage facilities or modifications to existing structures may be necessary as well as the possibility of off-site storage. All designs for plant additions or modifications will be submitted to DEQ for prior approval.**

Additions to the: **Land Application Plan.**
Page 8, Second paragraph, Line 15 of the original document.
"All potential sites for biosolids application within Tillamook County (Exhibit #8) will be evaluated and selection determined according to specific site requirements as contained in site approval letters issued by the Department of **Environmental Quality**. Here, at the end of this sentence, after **Environmental Quality**, I would like to add: " **This will include but not be limited to: No grazing livestock shall be allowed on the site for a minimum of thirty days after application. No hay shall be cut from sites for a minimum of thirty days after application. All public access shall be restricted for one full year after application of biosolids to the site. Areas of application shall be clearly marked or staked and all setbacks shall be maintained.**

Additions under the heading: **Land Application Plan:**

Page 9. End of the last paragraph of the original document which reads: "The city maintains records of the area of application, gallons applied, dates of application as well as all pertinent information in regards to biosolids program **and tests conducted**. Here, at the end of this sentence ending with: **and tests conducted.** I wish to add a new paragraph. The paragraph reads as follows: **"The City of Tillamook has recently acquired another eight acres of land, through contract, for biosolids application.**
This property is part of an old farm site located about 6 miles South of Tillamook owned by Mel and Connie Kimmel. The address is 5555 Kimmel's Riverside Lane, Tillamook, 97141. (See exhibits #13 & #14.)
These sites are used for production of non-irrigated pasture hay. Under our contract, the owners agree to restrict public access to the application site for a year after application to the site, the hay to which the biosolids are applied will not be cut for a minimum of 30 days after application and no additional fertilizer will be used while biosolids are being applied to this site. There is no animal grazing on this property.
This is a small section of the 58 acre holdings that the Kimmel's own and in the future the city hopes to procure additional sections of land in this tract as they become available." (see Exhibit 14.)

Addition under the heading: **Land Application Plan**:

Page 9 of original document. To be added as another new paragraph at the end of the proceeding new paragraph of this addendum ending with the following sentence: **"This is a small section of the 58 acre holdings that the Kimmel's own and in the future the city hopes to procure additional sections of land in this tract as they become available." (see Exhibit 14.)** Add the following paragraph immediately following: **(see Exhibit 14.)**
"The city also contracts for biosolids application on the 21.5 acre farm owned by Mel and Pat Sump. The Sump property is located south of Tillamook. The address of the Sump property is 8840 South Prairie Rd. Tillamook.Oregon. 97141 (See exhibits #3A&B & #4.)

This concluded the additions and corrections to the Tillamook Biosolids Management Plan.

City of Tillamook
Required Sewer Interceptor Sizes

Interceptor	Dry Weather Flow Existing (mgd)	Dry Weather Flow Buildout (mgd)	Peak Instant Flow Existing (mgd)	Peak Instant Flow Buildout (mgd)	Existing Conditions Ground Slope (ft/ft)	Existing Conditions Pipe Slope (ft/ft)	Existing Conditions Pipe Size (in)	Existing Conditions Capacity (mgd)	Required Pipe Size Existing (in)	Required Pipe Size Buildout (in)	Required Pipe Size Nominal (in)
East Interceptor	0.23	0.58	3.53	5.94		0.004	8	0.5	17	21	21
Proposed Fairview Interceptor											
UGB/Fairview Service Area		0.36		1.90	0.0083					12	12
Connect some existing svcs		0.19		1.98							
Total		0.55		3.88	0.0034					18	
					0.0011					23	24
					0.0052					17	
Southeast Interceptor	0.24	1.15	3.65	10.06		0.002	18	4.3	20	29	30
Proposed Southwest Forcemain	0.24	1.15	3.65	10.06						24	
Southwest Interceptor w/ FM	0.09	0.22	1.46	3.02		0.0018	16 and 12	3.1	14	19	
Southwest Interceptor no FM	0.33	1.37	5.12	13.08		0.0018	16 and 12	3.1	23	32	30
North Interceptor*		0.24		0.48		0.004	8	0.5		8	
West Interceptor	0.20	0.78	3.07	6.96		0.004	16	3.1	16	22	

Assumptions

Mannings n	0.013
I/I Reduction	0%
peaking factor, old systems	15.4
%of Existing E. Interceptor connections rerouted to Fariview Interceptor	25%

All of contributing basin is introduced at upstream end of the basin (conservative)

* SubBasin 5 flows only

Surcharge Calcs

Interceptor		Southeast		Southwest	
Flow		Existing	Future	Existing	Future
Peak Instant Flow	mgd	3.65	10.06	5.12	3.02
Exist Pipe Size	in	18	18	16	16
Hydraulic Slope	ft/ft	0.0029	0.0220	0.0107	0.0037
Pump Station		12th St.	12th St.	Infl PS	Infl PS
US Manhole		MH D10	MH D10	MH 215	MH 215
EL Invert at PS	ft	-2	-2	-2	-2
Pipe Length to US Manhole	ft	3500	3500	3770	3770
EL Top US Manhole	ft	14	14	15	15
EL Hyd Grade at US Manhole	ft	8	75	38	12
Evaluation		OK	BUST	BUST	OK

City of Tillamook
Required Sewer Interceptor Sizes

Interceptor	Dry Weather Flow Existing (mgd)	Dry Weather Flow Buildout (mgd)	Peak Instant Flow Existing (mgd)	Peak Instant Flow Buildout (mgd)	Existing Conditions Ground Slope (ft/ft)	Existing Conditions Pipe Slope (ft/ft)	Existing Conditions Pipe Size (in)	Existing Conditions Capacity (mgd)	Required Pipe Size Existing (in)	Required Pipe Size Buildout (in)	Required Pipe Size Nominal (in)
East Interceptor	0.23	0.58	2.21	3.95		0.004	8	0.5	15	18	18
Proposed Fairview Interceptor											
UGB/Fairview Service Area		0.36		1.48	0.0083					11	12
Connect some existing svcs		0.19		1.32							
Total		0.55		2.80	0.0034					16	
					0.0011					20	21
					0.0052					15	
Southeast Interceptor	0.24	1.15	2.29	6.91		0.002	18	4.3	17	25	24
Proposed Southwest Forcemain	0.24	1.15	2.29	6.91						~~24~~ 20	
Southwest Interceptor w/ FM	0.09	0.22	0.92	1.93		0.0018	16 and 12	3.1	12	16	
Southwest Interceptor no FM	0.33	1.37	3.20	8.83		0.0018	16 and 12	3.1	19	28	30
North Interceptor*		0.24		0.48		0.004	8	0.5		8	
West Interceptor	0.20	0.78	1.92	4.66		0.004	16	3.1	14	19	

Assumptions

Mannings n	0.013
I/I Reduction	40%
peaking factor, old systems	9.64
%of Existing E. Interceptor connections rerouted to Fariview Interceptor	25%

All of contributing basin is introduced at upstream end of the basin (conservative)
* SubBasin 5 flows only

Surcharge Calcs

Interceptor		Southeast		Southwest	
Flow		Existing	Future	Existing	Future
Peak Instant Flow	mgd	2.29	6.91	3.20	1.93
Exist Pipe Size	in	18	18	16	16
Hydraulic Slope	ft/ft	0.0011	0.0104	0.0042	0.0015
Pump Station		12th St.	12th St.	Infl PS	Infl PS
US Manhole		MH D10	MH D10	MH 215	MH 215
EL Invert at PS	ft	-2	-2	-2	-2
Pipe Length to US Manhole	ft	3500	3500	3770	3770
EL Top US Manhole	ft	14	14	15	15
EL Hyd Grade at US Manhole	ft	2	34	14	4
Evaluation		OK	BUST	OK	OK

Hazen-Williams formula
Pressurized flow, full circular pipe
Headloss Calcs for Tillamook SubBasin 5 Forcemain (Proposed Grinder Systems)

		Pipe 1	Pipe 2	Pipe 3	Pipe 4	Pipe 5	Pipe 6		
Flow	(gpm)	55.6	111.1	166.7	222.2	277.8	333.3	= 0.48 MGD @ buildout	
Diameter	(in)	6	6	6	6	6	6		
HW Coeff		120	120	120	120	120	120		
Pipe L	(ft)	1000	1000	1000	1000	1000	1000		
Fitting K		0	0	0	0	0	0		
R = D/4	(ft)	0.125	0.125	0.125	0.125	0.125	0.125		
Velocity	(ft/sec)	0.630442	1.260884	1.891326	2.521768	3.15221	3.782652		
Pipe hL	(ft)	0.376748	1.358178	2.875581	4.896235	7.398535	10.36647		
Fitting hL (V2/2g)	(ft)	0	0	0	0	0	0		
Grinder hL	(psi)		0	0	0	0	0		
Static Mixer hL	(psi)		0	0	0	0	0		
Total hL	(ft)	0.376748	1.358178	2.875581	4.896235	7.398535	10.36647	27	OK

		Pipe 1	Pipe 2	Pipe 3	Pipe 4	Pipe 5	Pipe 6	
Flow	(gpm)	55.6	111.1	166.7	222.2	277.8	333.3	
Diameter	(in)	8	8	8	8	8	8	
HW Coeff		120	120	120	120	120	120	
Pipe L	(ft)	1000	1000	1000	1000	1000	1000	
Fitting K		0	0	0	0	0	0	
R = D/4	(ft)	0.166667	0.166667	0.166667	0.166667	0.166667	0.166667	
Velocity	(ft/sec)	0.354624	0.709247	1.063871	1.418494	1.773118	2.127742	
Pipe hL	(ft)	0.092931	0.335017	0.70931	1.207738	1.824973	2.557064	
Fitting hL (V2/2g)	(ft)	0	0	0	0	0	0	
Grinder hL	(psi)		0	0	0	0	0	
Static Mixer hL	(psi)		0	0	0	0	0	
Total hL	(ft)	0.092931	0.335017	0.70931	1.207738	1.824973	2.557064	7

City of Tillamook
Estimated Wastwater Flows by SubBasin
Developed and Future Lots, data from County

		ZONE											Dry Weather Flow		Peak Instantaneous Flow		Buildout Population	
													Existing Flow	Buildout Flow	Existing Flow	Buildout Flow	Actual	ADWF
Sub Basin	Data	C-C	C-H	C-N	I-G	I-L	OS	P & S-P	R-0	R-5.0	R-7.5	Grand Total	(mgd)	(mgd)	(mgd)	(mgd)		Equivalent
1	Developed Lots	186			19	24	4	16	112	330	251	942	0.20		3.07		1661.688	
	Future Lots	8			0	4	0	0	302	66	15	395		0.22		3.17	736.848	
	Total Acres	41.01			6.98	9	96.36	21.62	18.98	45.68	36.04	279.67		0.12		0.24	2399	2649
2	Developed Lots	31	1				2		0	173	141	348	0.09		1.46		791.28	
	Future Lots	1	0				0		58	41	19	117		0.11		1.53	249.984	
	Total Acres	5.45	0.12				1.67		1.89	25.73	21.63	56.39		0.01		0.02	1041	1035
3	Developed Lots						1	6		26		33	0.01		0.12		65.52	
	Future Lots						0	0		4		4		0.01		0.13	10.08	
	Total Acres						0.13	3.91		3.98		8.02		0.00		0.00	76	76
5	Developed Lots		58				0	1				59	0.00		0.00		0	
	Future Lots		18				0	0				18		0.00		0.00	0	
	Total Acres		120.3				3.56	6.58				130.44		0.24		0.48	0	2005
4A	Developed Lots	3		10	12	8		7	505	31	374	950	0.23		3.53		1911.42	
	Future Lots	0		0	1	0		0	2044	144	222	2411		0.34		3.98	4527.936	
	Total Acres	1.39		5.17	77.07	18.17		145.97	119.59	22.09	118.21	507.66		0.20		0.41	6439	4530
4B	Developed Lots		9			0		2	1	44	76	132	0.04		0.56		304.164	
	Future Lots		2			2		0	24	132	213	373		0.14		0.98	911.736	
	Total Acres		32.73			57		2.21	0.89	23.24	58.36	174.43		0.18		0.36	1218	2869
Total Developed Lots		220	68	10	31	32	7	32	618	604	842	2464						
Total Future Lots		9	20	0	1	6	0	0	2426	387	469	3318						
Total Total Acres		47.85	153.15	5.17	86.05	84.17	101.62	180.29	141.35	120.72	236.24	1156.61						
Total Flows													0.57	1.58	8.75	11.29	11,171	13,164

Assumptions

- 2.52 capita/residence, zones R-5 and R-7.5
- 1.764 capita/residence, zone R-0
- 120 gpd/capita, average dry weather flow
- 15.4 peaking factor, old systems
- 4 peaking factor, new systems
- 2 peaking factor, commercial and industrial
- 2000 gal/acre/day commercial and industrial flows

City of Tillamook
Alternative Analysis
May 1, 2003
Facilities Plan Design Data

Criteria	Common to All Alternatives		Alternative 1: RBCs/SAS Anaerobic Digestion	Alternative 2: Biotower/RBCs Anaerobic Digestion	Alternative 3: Oxidation Ditch no PC Aerobic Digestion	Alternative 4: Additional RBCs, Anaerobic Digestion	Alternative 5: Oxidation Ditch with PC Anaerobic Digestion	Alternative 6: ICEAS SBR Anaerobic Digestion	Alternative 7: NSAS with Anaerobic Digestion
Population	*7265*								
Average Annual Flow	*1.715*	*mgd*							
Average Dry Weather Flow	*0.9*	*mgd*							
Max Month Dry Weather Flow	*2.08*	*mgd*							
Average Wet Weather Flow	*2.53*	*mgd*							
Max Month Wet Weather Flow	*4.01*	*mgd*							
Peak Day Flow	*7.16*	*mgd*							
Peak Instantaneous Flow	*10.01*	*mgd*							
Annual Average BOD	*1503*	*lb/day*							
Annual Average TSS	*1503*	*lb/day*							
Influent Pump Station									
Proposed Submersible Station									
Depth	20	ft							
Type	Rectangular, Cast in Place								
Width	12	ft							
Length	16	ft							
Proposed Pumps									
Type	Centrifugal solids-handling, submerisble								
Number - duty	2								
Number - standby	1								
Flow capacity each	5.01	MGD							
Head capacity each	35	ft TDH							
Brake horsepower each	58.64	hp							
Estimate total hp each	75	hp							
Proposed Level Control	Ultrasonic level sensor								

Criteria	Common to All Alternatives	Alternative 1: RBCs/SAS Anaerobic Digestion	Alternative 2: Biotower/RBCs Anaerobic Digestion	Alternative 3: Oxidation Ditch no PC Aerobic Digestion	Alternative 4: Additional RBCs, Anaerobic Digestion	Alternative 5: Oxidation Ditch with PC Anaerobic Digestion	Alternative 6: ICEAS SBR Anaerobic Digestion	Alternative 7: NSAS with Anaerobic Digestion
Headworks								
Existing Channel Width	30 inches							
Existing Flow Measurement								
Type	Parshall Flume							
Opening Size	12-inch							
Peak Flow Capacity	10.4 MGD							
Existing Grit Removal								
Type	Vortex							
Size	8 ft diameter							
Peak Flow Capacity	6 MGD							
Manufacturer	Smith&Loveless Pista Model 4A							
Existing Comminutor								
Peak Flow Capacity	10 MGD							
Manufacturer	JWC Muffin Monster Model 30000-24							
Proposed Fine Screen								
Type	In-channel basket perforated plate							
Opening Size	1/4 inch							
Primary Clarification				none				
Existing Clarifiers								
Number		2	2		2	2	2	2
Width, each		20 ft	20 ft		20 ft	20 ft	20 ft	20 ft
Length		60 ft	60 ft		60 ft	60 ft	60 ft	60 ft
Depth		13.7 ft	13.7 ft		13.7 ft	13.7 ft	13.7 ft	13.7 ft
Loading criteria (Peak Day)		3000 gpd/sqft	3000 gpd/sqft		3000 gpd/sqft	3000 gpd/sqft	3000 gpd/sqft	3000 gpd/sqft
Capacity (Peak Day)		7.2 MGD	7.2 MGD		7.2 MGD	7.2 MGD	7.2 MGD	7.2 MGD
Primary Effluent Quality								
BOD Removal Rate		30%	30%		30%	30%	30%	30%
TSS Removal Rate		60%	60%		60%	60%	60%	60%
Primary BOD		*1052.1 lb/day*	*1052.1 lb/day*		*1052.1 lb/day*	*1052.1 lb/day*	*1052.1 lb/day*	*1052.1 lb/day*
Primary TSS		*601.2 lb/day*	*601.2 lb/day*		*601.2 lb/day*	*601.2 lb/day*	*601.2 lb/day*	*601.2 lb/day*
SBOD Criteria		90 *mg/L*	90 *mg/L*		90 *mg/L*	90 *mg/L*	90 *mg/L*	90 *mg/L*
Primary Soluble BOD		*675.5 lb/day*	*675.5 lb/day*		*675.5 lb/day*	*675.5 lb/day*	*675.5 lb/day*	*675.5 lb/day*
Biological Treatment								

Criteria	Common to All Alternatives	Alternative 1: RBCs/SAS Anaerobic Digestion	Alternative 2: Biotower/RBCs Anaerobic Digestion	Alternative 3: Oxidation Ditch no PC Aerobic Digestion	Alternative 4: Additional RBCs, Anaerobic Digestion	Alternative 5: Oxidation Ditch with PC Anaerobic Digestion	Alternative 6: ICEAS SBR Anaerobic Digestion	Alternative 7: NSAS with Anaerobic Digestion
Proposed Activated Sludge								
Type		Selector Activated Sludge (SAS) parallel to RBCs		Oxidation Ditch		Orbal Oxidation Ditch	ICEAS SBR	
People Served		*4,883 people*		*7,265 people*		*7,265 people*	*7,265 people*	*7,265 people*
% of Flow to AS		*67% at ADWF*		*100%*		*100%*	*100%*	*100%*
BOD to be treated		*707 lb/day*		*1,503 lb/day*		*1,052 lb/day*	*1,052 lb/day*	*1,052 lb/day*
Secondary Sludge criteria, Avg Annual		0.8 lb solids/lb BOD		1 lb solids/lb BOD		0.70 lb solids/lb BOD	0.70 lb solids/lb BOD	0.70 lb solids/lb BOD
Secondary Sludge Production, Avg Annual		566 lb/day		1,503 lb/day		736 lb/day	736 lb/day	736 lb/day
RAS flow rate criteria		67% of ADWF		100% of ADWF		100% of ADWF	100% of ADWF	100% of ADWF
Flow through Basin (Incl. RAS)								
ADWF		1.21 mgd		1.80 mgd				
MMWWF		3.30 mgd		4.91 mgd				
Peak Day		5.42 mgd		8.06 mgd				
Number Basins		2		2		2	2	2
MCRT, Total		6 days		10.0 days		10.0 days	40.9 days	12.0 days
MLSS		3,000 mg/l		3,000 mg/l		2,220 mg/l	2,231 mg/l	2,500 mg/l
Total mass of sludge		3,394 lb		15,030 lb		7,365 lb	30,122 lb	8,838 lb
Minimum total volume		135,649 gal		600,719 gal		397,774 gal	1,618,873 gal	423,868 gal
Decant Volume, each							0.27 mgal	
Decant depth, each							6 ft	
Basin width, each							68 ft	
Basin length							88 ft	
Average Decant Rate							5,867 gpm	
Peak Decant Rate							9,944 gpm	
Basin Volume, each		0.07 mgal		0.30 mgal		0.20 mgal	0.81 mgal	0.22 mgal
Basin depth, each		16 ft		13 ft		10.5 ft	18 ft	18.0 ft
Basin width, each		15 ft		20 ft		30 ft	68 ft	20 ft
Basin length		39 ft		86 ft, assumed oval		55 ft, assumed oval	88 ft	82 ft
Total volume		0.14 mgal		0.60 mgal		0.40 mgal	1.62 mgal	0.44 mgal
HDT at ADDWF (not incl RAS)		5.6 hours		16.0 hours		10.6 hours	43.2 hours	11.8 hours
HDT at MMWWF (not incl RAS)		1.2 hours		3.6 hours		2.4 hours	9.7 hours	2.7 hours
Loading		37.78		18.74 lb BOD/1000 cuft		19.82 lb BOD/1000 cuft	4.86 lb BOD/1000 cuft	17.72 lb BOD/1000 cuf
Oxygen demand criteria								
Estimated total from all demands		1.5 lb O2/lb BOD		1.5 lb O2/lb BOD		1.5 lb O2/lb BOD	1.5 lb O2/lb BOD	1.5 lb O2/lb BOD
Oxygen demand		1,061 lb O2/day		2,255 lb O2/day		1,578 lb O2/day	1,578 lb O2/day	1,578 lb O2/day
Field O2 xfer rate		1.5 lb O2/hr/hp		1.5 lb O2/hr/hp		1.5 lb O2/hr/hp	1.5 lb O2/hr/hp	1.5 lb O2/hr/hp
Total aeration hp required		29 hp		63 hp		44 hp	44 hp	44 hp
Mixing demand criteria		35 hp/MG		35 hp/MG		35 hp/MG	35 hp/MG	35 hp/MG
Basin mixing hp, total		5 hp		21 hp		14 hp	57 hp	16 hp
Secondary Settling								
Existing Clarifiers		Existing Secondary Clarifiers	Existing Secondary Clarifiers	Existing Primary Clarifiers	Existing Secondary Clarifiers	none	none	none
Number		2	2	2	2			
Diameter		45 ft	45 ft		45 ft			
Width, each				20 ft				
Length				60 ft				
Depth		10 ft	10 ft	13.7 ft	10 ft			
Loading criteria (Peak Day)		1000 gpd/sqft	1200 gpd/sqft	1060 gpd/sqft	1200 gpd/sqft			
Capacity (Peak Day)		3.181 MGD	3.817 MGD	2.544 MGD	3.817 MGD			

Criteria	Common to All Alternatives	Alternative 1: RBCs/SAS Anaerobic Digestion	Alternative 2: Biotower/RBCs Anaerobic Digestion	Alternative 3: Oxidation Ditch no PC Aerobic Digestion	Alternative 4: Additional RBCs, Anaerobic Digestion	Alternative 5: Oxidation Ditch with PC Anaerobic Digestion	Alternative 6: ICEAS SBR Anaerobic Digestion	Alternative 7: NSAS with Anaerobic Digestion
Proposed Clarifiers							none	
Number		2	2	2	2	2		2
Diameter		58 ft	45 ft	55 ft	45 ft	65 ft		65 ft
Width, each								
Length								
Depth		14 ft	14 ft	14 ft	14 ft	14 ft		14 ft
Loading criteria (Peak Day)		1200 gpd/sqft	1200 gpd/sqft	1060 gpd/sqft	1200 gpd/sqft	1060 gpd/sqft		1060 gpd/sqft
Capacity (Peak Day)		6.341 MGD	3.817 MGD	5.037 MGD	3.817 MGD	7.035 MGD		7.035 MGD
Total Secondary Clarifier Capacity (Peak Day)		9.5 MGD	7.6 MGD	7.6 MGD	7.6 MGD	7.0 MGD		7.0 MGD
Surface area with one new clarifier			4771.3 sqft	4775.8 sqft		3318.3 sqft		3318.3 sqft
estimated peak day flow capacity			5.7 MGD	5.1 MGD		3.5 MGD		3.5 MGD
Primary Solids Production		901.8 lb/day	901.8 lb/day		901.8 lb/day	901.8 lb/day	901.8 lb/day	901.8 lb/day
Secondary Biosolids Production		772.7 lb/day	736.5 lb/day	1503.0 lb/day	736.5 lb/day	736.5 lb/day	736.5 lb/day	736.5 lb/day
Total Solids Production								
TSS		*1674.5 lb TSS/day*	*1638.3 lb TSS/day*	*1503.0 lb TSS/day*	*1638.3 lb TSS/day*	*1638.3 lb TSS/day*	*1638.3 lb TSS/day*	*1638.3 lb TSS/day*
VSS Criteria		80% VSS/TSS	80% VSS/TSS	80% VSS/TSS	80% VSS/TSS	80% VSS/TSS	80% VSS/TSS	80% VSS/TSS
VSS		*1339.6 lb VSS/day*	*1310.6 lb VSS/day*	*1202.4 lb VSS/day*	*1310.6 lb VSS/day*	*1310.6 lb VSS/day*	*1310.6 lb VSS/day*	*1310.6 lb VSS/day*
Flow Equalization								
Critieria, % of total decant volume							75%	
Required Volume							0.20 mgal	
Depth							5.0 ft	
Width							88.0 ft	
Req'd Length							61.5 ft	
Disinfection								
Existing Contact Chamber								
Diameter	40 ft							
Depth	9 ft							
Volume	84600 gal							
Contact Time at ADWF	135 min							
Contact Time at PDF	17 min							
Existing Chlorination System								
Type	Automatic Gas, 150 lb cylinders							
Assumed dosage	10 mg/L							
Demand at MMWWF	334 ppd							
Capacity of Automatic Chlor	300 ppd							
Existing Dechlorination System								
Type	Automatic, liquid Sodium Bisulfite							
Assumed dosage	7.5 mg/L							
Demand at MMWWF	251 ppd							
Volume	3468 gal/mo							
Storage Volume	1100 gal							
Delivery frequency	9.65 days							

Criteria	Common to All Alternatives	Alternative 1: RBCs/SAS Anaerobic Digestion	Alternative 2: Biotower/RBCs Anaerobic Digestion	Alternative 3: Oxidation Ditch no PC Aerobic Digestion	Alternative 4: Additional RBCs, Anaerobic Digestion	Alternative 5: Oxidation Ditch with PC Anaerobic Digestion	Alternative 6: ICEAS SBR Anaerobic Digestion	Alternative 7: NSAS with Anaerobic Digestion
Effluent Flow Measurement								
Existing Meter Type	Velocity x Area Meter							
Solids								
Existing Primary Anaerobic Digester - not used								
Volume	62,000 gal							
Assumed solids	4%							
Existing Secondary Aerobic Digetser (Storage) - not used								
Volume	84,500 gal							
Assumed solids	6%							
Proposed Aerobic Digesters								
Reuse existing secondary clarifiers								
Number				2				
Diameter				45 ft				
Depth				10 ft				
Volume				237,928 gal				
New aerobic digester								
Number				1				
Diameter				65 ft				
Depth				20 ft				
Volume				496,418 gal				
Total Volume				734,347 gal				
Assumed influent solids				1.5%				
Solids Residence Time (SRT)				61 days				
Mixing Demand Criteria				1 hp/1000 cuft				
Mixing horsepower req'd				98 hp				
Solids loading				0.0122 lb VS/cuft/day				
Assumed volatile solids reduction				40%				
Oxygen Demand Criteria				1.5 lb O2/lb VS destroyed				
Oxygen Demand				721 lb/day				
Field O2 xfer rate				1.5 lb O2/hr/hp				
Total aeration hp required				20 hp				
Air Demand Criteria				15 scfm/1000 cuft				
Air Demand				1,473 scfm				
Proposed Gravity Thickener								
Number			1		1			
Diameter			12 ft		12 ft			
Depth			10 ft		10 ft			
Loading Rate			14.49 ppd/sqft		14.49 ppd/sqft			
Assumed influent sludge concentration			1.5%		1.5%			
Assumed effluent concentration			3.5%		3.5%			
Proposed Anaerobic Digester								
Number		1	1		1	1	1	1
Diameter		42 ft	28 ft		28 ft	43 ft	42 ft	43 ft
Depth		25 ft	25 ft		25 ft	25 ft	25 ft	25 ft
Volume		259,078 gal	115,146 gal		115,146 gal	271,561 gal	259,078 gal	271,561 gal
Assumed Influent Solids Concentration		1.5%	3.5%		3.5%	1.5%	1.5%	1.5%
Influent Solids volume		13,385 gal/day	5,612 gal/day		5,612 gal/day	13,096 gal/day	13,096 gal/day	13,096 gal/day
Hydraulic Retention Time (HRT)		19.36 days	20.52 days		20.52 days	20.74 days	19.78 days	20.74 days
Solids Loading		0.0387 lb VS/cuft/day	0.0851 lb VS/cuft/day		0.0851 lb VS/cuft/day	0.0361 lb VS/cuft/day	0.0378 lb VS/cuft/day	0.0361 lb VS/cuft/day
Assumed volatile solids reduction		50%	50%		50%	50%	50%	50%

Criteria	Common to All Alternatives	Alternative 1: RBCs/SAS Anaerobic Digestion	Alternative 2: Biotower/RBCs Anaerobic Digestion	Alternative 3: Oxidation Ditch no PC Aerobic Digestion	Alternative 4: Additional RBCs, Anaerobic Digestion	Alternative 5: Oxidation Ditch with PC Anaerobic Digestion	Alternative 6: ICEAS SBR Anaerobic Digestion	Alternative 7: NSAS with Anaerobic Digestion
Mixing								
Mixing Type		Draft Tube	Draft Tube		Draft Tube	Draft Tube	Draft Tube	Draft Tube
Mixing Demand Criteria		0.035 hp/1000 gal	0.035 hp/1000 gal		0.035 hp/1000 gal	0.035 hp/1000 gal	0.035 hp/1000 gal	0.035 hp/1000 gal
Mixing horsepower req'd		9.07 hp	4.03 hp		4.03 hp	9.50 hp	9.07 hp	9.50 hp
Heating								
Type		Methane / Propane Boiler	Methane / Propane Boiler		Methane / Propane Boiler	Methane / Propane Boiler	Methane / Propane Boiler	Methane / Propane Boiler
Approx Total Heating Req'd		316,500 Btu/hr	154,000 Btu/hr		154,000 Btu/hr	316,400 Btu/hr	312,400 Btu/hr	316,400 Btu/hr
Proposed Mechanical Dewatering								
Solids to be dewatered				*1022 lb TSS/day*				
Assumed solids concentration				2.0%				
Volume to be dewatered				6,127 gal/day				
Feed rate (1 day/wk)				89 gpm				
Type				Belt Filter Press				
Size				1 meter?				
Assumed Final Solids Concentration				14.0%				
Proposed Sludge Storage								
Solids to be stored		*1005 lb TSS/day*	*983 lb TSS/day*	*1022 lb TSS/day*	*983 lb TSS/day*	*983 lb TSS/day*	*983 lb TSS/day*	*983 lb TSS/day*
Assumed solids concentration		3.5%	4.5%	14.0%	4.5%	3.5%	3.5%	3.5%
Volume to be stored		3,442 gal/day	2,619 gal/day	117 cuft/day	2,619 gal/day	3,367 gal/day	3,367 gal/day	3,367 gal/day
Assumed storage time		122 days	122 days	122 days	122 days	122 days	122 days	122 days
Total Volume Required		418,762 gal	318,662 gal	14,238 cuft	318,662 gal	410,830 gal	409,708 gal	410,830 gal
Converted Volume Available								
Existing Clarifiers								
Number		1	1		1	2	2	2
Depth		20 ft	20 ft		20 ft	10 ft	10 ft	10 ft
Diameter		- ft	- ft		- ft	45 ft	45 ft	45 ft
Volume						237,928 gal	237,928 gal	237,928 gal
Existing Secondary Aerobic Digetser (Storage)								
Volume						84,500 gal	84,500 gal	84,500 gal
Total Existing Storage						322,428 gal	322,428 gal	322,428 gal
Time						95.75 days	95.75 days	95.75 days
Additional Volume Required						88,402 gal	87,279 gal	88,402 gal
Number		1	1		1	1	1	1
Assumed depth		20 ft	20 ft	5.5 ft	20 ft	20 ft	20 ft	20 ft
Width				30 ft				
Length				86 ft				
Diameter		60 ft	52 ft		52 ft	27 ft	27 ft	27 ft
Sludge Disposal								
Solids to be disposed		*1,005 lb TSS/day*	*983 lb TSS/day*	*1,022 lb TSS/day*	*983 lb TSS/day*	*983 lb TSS/day*	*983 lb TSS/day*	*983 lb TSS/day*
No Dewatering								
Assumed concentration		3.5%	4.5%	2.0%	4.5%	3.5%	3.5%	3.5%
Solids volume		104,691 gal/mo	79,665 gal/mo	186,373 gal/mo	79,665 gal/mo	102,427 gal/mo	102,427 gal/mo	102,427 gal/mo
# 2500 gallon loads during 2/3 of year		63 trips/mo	48 trips/mo	112 trips/mo	48 trips/mo	61 trips/mo	61 trips/mo	61 trips/mo
Assumed hours per load		4 hr	4 hr	4 hr	4 hr	4 hr	4 hr	4 hr
Required Staff during 8 mo		1.45 FTE	1.11 FTE	2.59 FTE	1.11 FTE	1.42 FTE	1.42 FTE	1.42 FTE
With Dewatering								
Assumed concentration		14.0%	14.0%	14.0%	14.0%	14.0%	14.0%	14.0%
Solids volume		26,173 gal/mo	25,607 gal/mo	26,625 gal/mo	25,607 gal/mo	25,607 gal/mo	25,607 gal/mo	25,607 gal/mo
# 2500 gallon loads during 2/3 of year		16 trips/mo	15 trips/mo	16 trips/mo	15 trips/mo	15 trips/mo	15 trips/mo	15 trips/mo
Assumed hours per load		4 hr	4 hr	4 hr	4 hr	4 hr	4 hr	4 hr
Required Staff during 8 mo		0.36 FTE	0.36 FTE	0.37 FTE	0.36 FTE	0.36 FTE	0.36 FTE	0.36 FTE
Disposal Criteria (from DEQ)		3.60 dry tons/acre/yr	3.60 dry tons/acre/yr	3.10 dry tons/acre/yr	3.60 dry tons/acre/yr	3.60 dry tons/acre/yr	3.60 dry tons/acre/yr	3.60 dry tons/acre/yr
Approx. Land Required		50.93 acres	49.83 acres	60.17 acres	49.83 acres	49.83 acres	49.83 acres	49.83 acres

Item	Alternative 1	2	3	4	5	6	7
	SAS/RBC	BT/RBC	OXDnoPC	RBC/RBC	OXDwPC	SBR	NSAS
Influent Pump Station	$ 1,340	$ 1,340	$ 1,340	$ 1,340	$ 1,340	$ 1,340	$ 1,340
Headworks	$ 230	$ 230	$ 230	$ 230	$ 230	$ 230	$ 230
Secondary Treatment	$ 3,172	$ 3,442	$ 3,644	$ 3,435	$ 2,784	$ 3,489	$ 3,045
Effluent Filtration	$ 800	$ 800		$ 800			
Disinfection	$ 396	$ 396	$ 396	$ 396	$ 396	$ 396	$ 396
Outfall	$ 35	$ 35	$ 35	$ 35	$ 35	$ 35	$ 35
Solids Processing	$ 2,238	$ 2,238	$ 2,161	$ 2,238	$ 2,583	$ 2,583	$ 2,583
Site Work	$ 469	$ 469	$ 469	$ 469	$ 469	$ 469	$ 469
Standby Power	$ 314	$ 314	$ 314	$ 314	$ 314	$ 314	$ 314
Wastewater Treatment (WWTP) Plant Cost	$ 9,000	$ 9,270	$ 8,590	$ 9,260	$ 8,160	$ 8,860	$ 8,420
Annual Comprative Operating Cost (COC)	111	102	121	96	92	89	90
Present Worth of Annual COC	$1,360	$1,250	$1,500	$1,180	$1,130	$1,100	$1,110
Total Comparative PW Cost of WWTP Upgrade	**$ 10,360**	**$ 10,520**	**$ 10,090**	**$ 10,440**	**$ 9,290**	**$ 9,960**	**$ 9,530**

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan
Alternative 1

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No.: 036043.00

SUMMARY BY AREA

ITEM NO.	ITEM DESCRIPTION	Phasing Schedule			Total
		Now	Phase II	Phase III	
	Secondary Treatment and Solids Handling Items				
	Selector Basins, 3rd Secondary Clarifier, Replace 1/2 RBCs	2,020,800			2,020,800.00
	4th Secondary Clarifier, Replace 1/2 RBCs		1,151,500		1,151,500
	Gravity Thickener, Anaerobic Digester, 1st Sludge Storage	1,728,100			1,728,100
	2nd Sludge Storage		510,100		510,100
	Effluent Filter and Pump Station			799,600	799,600
	Items Common to All Alternatives				
	Influent Pump Station - Submersible	1,340,400			1,340,400
	Headworks Improvements	230,500			230,500
	Disinfection			396,200	396,200
	Outfall	35,300			35,300
	Standby Power	314,300			314,300
	Site Work	469,100			469,100
	Total Estimate of Project Cost	$ 6,140,000	$ 1,660,000	$ 1,200,000	$ 9,000,000

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan
Alternative 2: Biotower

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No.: 036043.00

SUMMARY BY AREA

ITEM NO.	ITEM DESCRIPTION	Phasing Schedule			Total
		Now	Phase II	Phase III	
	Secondary Treatment and Solids Handling Items				
	Biotower, Biotower Pump Station, 3rd Secondary Clarifier, Replace 1/2 RBCs	2,359,000			2,359,000
	4th Secondary Clarifier, Replace 1/2 RBCs		1,082,700		1,082,700
	Gravity Thickener, Anaerobic Digester, 1st Sludge Storage	1,728,100			1,728,100
	2nd Sludge Storage		510,100		510,100
	Effluent Filter and Pump Station			799,600	799,600
	Items Common to All Alternatives				
	Influent Pump Station - Submersible	1,340,400			1,340,400
	Headworks Improvements	230,500			230,500
	Disinfection			396,200	396,200
	Outfall	35,300			35,300
	Standby Power	314,300			314,300
	Site Work	469,100			469,100
	Total Estimate of Project Cost	$ 6,480,000	$ 1,590,000	$ 1,200,000	$ 9,270,000

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan
Alternative 3: Oxidation Ditch no Primary Clarification

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No.: 036043.00

SUMMARY BY AREA

ITEM NO.	ITEM DESCRIPTION	Phasing Schedule			Total
		Now	Phase II	Phase III	
	Secondary Treatment and Solids Handling Items				
	Oxidation Ditch, 3rd Secondary Clarifier, Convert Primary	3,049,700			3,045,000
	4th Secondary Clarifier		594,100		594,100
	Aerobic Digester, Convert Secondaries to Storage	1,198,700			1,198,700
	Dewatering and Cake Storage		962,900		962,900
	Items Common to All Alternatives				
	Influent Pump Station - Submersible	1,340,400			1,340,400
	Headworks Improvements	230,500			230,500
	Disinfection			396,200	396,200
	Outfall	35,300			35,300
	Standby Power	314,300			314,300
	Site Work	469,100			469,100
	Total Estimate of Project Cost	$ 6,637,000	$ 1,557,000	$ 396,000	8,590,000

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan
Alternative 4 Series RBC

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No.: 036043.00

SUMMARY BY AREA

ITEM NO.	ITEM DESCRIPTION	Phasing Schedule			Total
		Now	Phase II	Phase III	
	Secondary Treatment and Solids Handling Items				
	Additional RBCs, 3rd Secondary Clarifier, Replace 1/2 RBCs	2,352,800			2,352,800
	4th Secondary Clarifier, Replace 1/2 RBCs		1,082,700		1,082,700
	Gravity Thickener, Anaerobic Digester, 1st Sludge Storage	1,728,100			1,728,100
	2nd Sludge Storage		510,100		510,100
	Effluent Filter and Pump Station			799,600	799,600
	Items Common to All Alternatives				
	Influent Pump Station - Submersible	1,340,400			1,340,400
	Headworks Improvements	230,500			230,500
	Disinfection			396,200	396,200
	Outfall	35,300			35,300
	Standby Power	314,300			314,300
	Site Work	469,100			469,100
	Total Estimate of Project Cost	$ 6,470,000	$ 1,590,000	$ 1,200,000	9,260,000

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan
Alternative 5 Oxidation Ditch with Primary Clarification

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No.: 036043.00

SUMMARY BY AREA

ITEM NO.	ITEM DESCRIPTION	Phasing Schedule			Total
		Now	Phase II	Phase III	
	Secondary Treatment and Solids Handling Items				
	Oxidation Ditch and new Secondary Clarifiers	2,784,300			2,784,300
	Anaerobic Digester, Convert existing tankage to storage	2,124,400			2,124,400
	New Sludge Storage Tank			458,800	458,800
	Items Common to All Alternatives				
	Influent Pump Station - Submersible	1,340,400			1,340,400
	Headworks Improvements	230,500			230,500
	Disinfection			396,200	396,200
	Outfall	35,300			35,300
	Standby Power	314,300			314,300
	Site Work	469,100			469,100
	Total Estimate of Project Cost	**$ 7,300,000**	**$ -**	**$ 860,000**	**8,160,000**

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan
Alternative 6 ICEAS SBR

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No.: 036043.00

SUMMARY BY AREA

ITEM NO.	ITEM DESCRIPTION	Phasing Schedule			Total
		Now	Phase II	Phase III	
	Secondary Treatment and Solids Handling Items				
	ICEAS SBR	3,489,500			3,489,500
	Anaerobic Digester, Convert existing tankage to storage	2,124,400			2,124,400
	New Sludge Storage Tank			458,800	458,800
	Items Common to All Alternatives				
	Influent Pump Station - Submersible	1,340,400			1,340,400
	Headworks Improvements	230,500			230,500
	Disinfection			396,200	396,200
	Outfall	35,300			35,300
	Standby Power	314,300			314,300
	Site Work	469,100			469,100
	Total Estimate of Project Cost	**$ 8,000,000**	**$ -**	**$ 860,000**	**8,860,000**

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan
Alternative 7 Nitrifying SAS

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No.: 036043.00

SUMMARY BY AREA

ITEM NO.	ITEM DESCRIPTION	Phasing Schedule			Total
		Now	Phase II	Phase III	
					0.00
#REF!	Secondary Treatment and Solids Handling Items				
	Selector Activated Sludge	3,044,100			3,044,100
	Anaerobic Digester, Convert existing tankage to storage	2,124,400			2,124,400
	New Sludge Storage Tank			458,800	458,800
		0	0		0
					0
#REF!	Items Common to All Alternatives				0
#REF!	Influent Pump Station - Submersible	1,340,400			1,340,400
	Headworks Improvements	230,500			230,500
	Disinfection			396,200	396,200
	Outfall	35,300			35,300
	Standby Power	314,300			314,300
	Site Work	469,100			469,100
					0.00
					0.00
					0.00
	Total Estimate of Project Cost	$ 7,560,000	$ -	$ 860,000	8,420,000

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Influent Pump Station, Wet Well / Dry Well

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @
- ☐ Construction
- ☐ Change Order
- % Complete

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Wet Well/ Dry Well Structure										
	1	Concrete - Walls	170	cuyd	150.00	25,500	500.00	85,000			110,500	quantity from cad
	2	Concrete - Slab on grade	60	cuyd	100.00	6,000	350.00	21,000			27,000	quantity from cad
	3	Concrete - Elevated slab	30	cuyd	100.00	3,000	600.00	18,000			21,000	quantity from cad
	4	Excavation	5,060	cuyd			15.00	75,900			75,900	quantity from cad
	5	Backfill	4,600	cuyd	10.00	46,000	10.00	46,000			92,000	quantity from cad
	6	Control of Water	1	ls			40,000.00	40,000			40,000	
	7	New Influent Manhole	1	ls	8,000.00	8,000	5,000.00	5,000			13,000	
	8	Tie-in to existing piping	2	ea			5,000.00	10,000			10,000	
	9	Pumps	3	ea	27,000.00	81,000	4,050.00	12,150			93,150	Pioneer SC1212S17, PumpTech
	10	Electrical, ea pump	3	ea	15,000.00	45,000	5,000.00	15,000			60,000	approx 20% of pump cost
	11	Valves	9	ea	10,000.00	90,000	2,500.00	22,500			112,500	
	12	Other Pipe & Fittings, ea pump	3	ea	10,000.00	30,000	10,000.00	30,000			60,000	
				ea								
		Architectural										
	13	Miscellaneous Metals	1	ls	15,000.00	15,000	5,000.00	5,000			20,000	
	14	Above-grade Building	900	sqft	80.00	72,000	70.00	63,000			135,000	
	15	HVAC & Odor Control	1	ls	18,000.00	18,000	5,000.00	5,000			23,000	
Subtotals						439,500		453,550			693,050	

1,701,000

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Influent Pump Station, Submersible

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Current at ENR _____
Escalated to ENR _____

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Submersible Structure										
	1	Concrete - Walls	78	cuyd	150.00	11,667	500.00	38,889			50,556	
	2	Concrete - Slab on grade	19	cuyd	100.00	1,933	350.00	6,767			8,700	
	3	Concrete - Elevated slab	15	cuyd	100.00	1,539	600.00	9,233			10,772	
	4	Excavation	2,929	cuyd			15.00	43,931			43,931	
	5	Backfill	2,677	cuyd	10.00	26,774	10.00	26,774			53,547	
	6	Control of Water	1	ls			20,000.00	20,000			20,000	
	7	New Influent Manhole	1	ls	8,000.00	8,000	5,000.00	5,000			13,000	
	8	Tie-in to existing piping	2	ea			5,000.00	10,000			10,000	
	9	Pumps	3	ea	32,000.00	96,000	6,400.00	19,200			115,200	Flowserve 12MSX174, APSCO
	10	Electrical, ea pump	3	ea	15,000.00	45,000	5,000.00	15,000			60,000	Approx 20% of pump cost
	11	Valves	6	ea	10,000.00	60,000	2,500.00	15,000			75,000	
	12	Other Pipe & Fittings, ea pump	3	ea	10,000.00	30,000	10,000.00	30,000			60,000	
		Architectural										
	13	Miscellaneous Metals	1	ls	30,000.00	30,000	15,000.00	15,000			45,000	
	14	Above-grade Building	620	sqft	80.00	49,600	70.00	43,400			93,000	
	15	Odor Control	1	ls	10,000.00	10,000	5,000.00	5,000			15,000	
	16	Monorail & Hoist	1	ls	20,000.00	20,000	10,000.00	10,000			30,000	
Subtotals						390,512		313,193			703,705	

1,340,000

ESTIMATE OF O&M COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
	Power					
30	Pumps	261,398	kW*hr	0 07	18,298	40 hp, 24hrs/day (180 hp connected)
Subtotal					18,300	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: Existing - Headworks (new Screen)

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Current at ENR _____
Escalated to ENR _____

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
	1	Headworks Screen & Controls	1	ea	90,000.00	90,000	18,000.00	18,000			108,000	Andritz/APSCO
	2	Modifications to channel	1	ls			3,000.00	3,000			3,000	
	3	Electrical Improvements	1	ls	5,000.00	5,000	5,000.00	5,000			10,000	approx 10% of screen
Subtotals						95,000		26,000			121,000	

230,000

ESTIMATE OF O&M COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
	Power					
	Screen	19,605	kW*hr	0.07	1,372	3 hp, 24hrs/day
Subtotal					1,400	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Selector Activated Sludge parallel to Existing RBCs

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		SAS - 2 basins, 151 x 401 (8 day MCRT)										
	1	Concrete - Walls	220	cuyd	150.00	33,000	500.00	110,000			143,000	quantity from cad
	2	Concrete - Slab on grade	110	cuyd	100.00	11,000	350.00	38,500			49,500	
	3	Excavation	1,730	cuyd			15.00	25,950			25,950	
	4	Backfill	1,070	cuyd	10.00	10,700	10.00	10,700			21,400	
	5	Diffusers - fine bubble	1	ls	23,200.00	23,200	6,960.00	6,960			30,160	$16/lb BOD avg, Sanitaire/Beaver
	6	Blowers, 100 hp	3	ea	30,000.00	90,000	6,000.00	18,000			108,000	$300/hp, Lamson/TEC
	7	Electrical	1	ls	5,000.00	5,000	5,000.00	5,000			10,000	approx 10% of blower cost
	8	Blower Building	456	sqft	80.00	36,480	70.00	31,920			68,400	
	9	Replacement RBC equipment	4	ea	100,000.00	400,000	20,000.00	80,000			480,000	
		Additional Secondary Clarifiers - 2, 58'dia										
	10	Concrete - Walls	230	cuyd	150.00	34,500	500.00	115,000			149,500	
	11	Concrete - Slab on grade	370	cuyd	100.00	37,000	350.00	129,500			166,500	
	12	Excavation	4,620	cuyd			15.00	69,300			69,300	
	13	Backfill	2,000	cuyd	10.00	20,000	10.00	20,000			40,000	
	14	Sludge Mechanisms	2	ea	87,000.00	174,000	17,400.00	34,800			208,800	$1500/ft/dia, Beaver Eq
	15	WAS/RAS pumping	1	ls	25,000.00	25,000	15,000.00	15,000			40,000	
	16	Electrical	1	ls	10,000.00	10,000	15,000.00	15,000			25,000	
	17	Piping	1	ls	7,500.00	7,500	7,500.00	7,500			15,000	
	18	Miscellaneous metals	1	ls	7,500.00	7,500	7,500.00	7,500			15,000	
Subtotals						924,880		740,630			1,665,510	

3,172,000

ESTIMATE OF AVERAGE ANNUAL POWER COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
	Power					
27	Blowers	326,748	kW*hr	0.07	$22,872	50 hp, 24hrs/day (150 hp connected)
28	RBC Blowers	163,374	kW*hr	0.07	$11,436	25 hp, 24hrs/day (75 hp connected)
29	Secondary Clarifier mechanisms	13,070	kW*hr	0.07	$915	4 mechanisms, 0.5 hp, 24hrs/day
30	WAS pumping	5,446	kW*hr	0.07	$381	5 hp, 4hrs/day
31	RAS pumping	13,070	kW*hr	0.07	$915	2.0 hp, 24hrs/day
Subtotal					$37,000	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Biotower before Existing RBCs

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Current at ENR _____
Escalated to ENR _____

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ ___ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Biotower - 45ft dia										
	1	Concrete - Walls	90	cuyd	150.00	13,500	500.00	45,000			58,500	quantity from cad
	2	Concrete - Slab on grade	30	cuyd	100.00	3,000	350.00	10,500			13,500	
	2	Concrete - Precast	100	cuyd	50.00	5,000	400.00	40,000			45,000	
	3	Structural Fill	1,100	cuyd	15.00	16,500	20.00	22,000			38,500	
	4	Media	26,710	cuft	4.00	106,840	1.20	32,052			138,892	Ric Tower
	5	Distributor Mechanism	1	ea	28,000.00	28,000	8,400.00	8,400			36,400	Walker, TEC, 42ft dia, 10-in mach
		Biotower Pump Station										
	6	Concrete - Walls	30	cuyd	150.00	4,500	500.00	15,000			19,500	
	7	Concrete - Slab on grade	10	cuyd	100.00	1,000	350.00	3,500			4,500	
	8	Concrete - Elevated slab	10	cuyd	100.00	1,000	600.00	6,000			7,000	
	9	Structural Fill	210	cuyd	15.00	3,150	20.00	4,200			7,350	
	10	Piping	1	ls	20,000.00	20,000	10,000.00	10,000			30,000	
	11	Pumps, 40hp	4	ea	12,000.00	48,000	2,400.00	9,600			57,600	Flowserve 14ENL, APSCO
		Cover	1,590	sqft	40.00	63,617	30.00	47,713			111,330	
		Odor Control	1	ls	10,000.00	10,000	5,000.00	5,000			15,000	Biofilter
	12	Electrical	1	ls	20,000.00	20,000	20,000.00	20,000			40,000	Approx 20% of pump cost
	13	Piping	1	ls	20,000.00	20,000	20,000.00	20,000			40,000	
	14	Misc metals	1	ls	5,000.00	5,000	2,000.00	2,000			7,000	
	15	Replacement RBC equipment	4	ea	100,000.00	400,000	20,000.00	80,000			480,000	
		Additional Secondary Clarifiers - 2, 45'dia										
	16	Concrete - Walls	210	cuyd	150.00	31,500	500.00	105,000			136,500	
	17	Concrete - Slab on grade	290	cuyd	100.00	29,000	350.00	101,500			130,500	
	18	Excavation	4,950	cuyd			15.00	74,250			74,250	
	19	Backfill	2,930	cuyd	10.00	29,300	10.00	29,300			58,600	
	20	Sludge Mechanisms	2	ea	67,500.00	135,000	13,500.00	27,000			162,000	$1500/ft dia, Beaver Eq
	21	WAS/RAS pumping	1	ls	25,000.00	25,000	15,000.00	15,000			40,000	
	22	Electrical	1	ls	10,000.00	10,000	15,000.00	15,000			25,000	
	23	Miscellaneous metals	1	ls	7,500.00	7,500	7,500.00	7,500			15,000	
	24	Piping	1	ls	7,500.00	7,500	7,500.00	7,500			15,000	
Subtotals						1,043,907		763,015			1,806,922	

3,442,000

ESTIMATE OF AVERAGE ANNUAL POWER COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
	Power					
33	Biotower distributor	6,535	kW*hr	0.07	$457	1 hp, 24hrs/day
34	Biotower pumps	196,049	kW*hr	0.07	$13,723	30 hp, 24hrs/day (60 hp connected at PIF)
35	Secondary Clarifier mechanisms	13,070	kW*hr	0.07	$915	4 mechanisms, 0.5 hp, 24hrs/day
36	RBC Blowers	163,374	kW*hr	0.07	$11,436	25 hp, 24hrs/day (75 hp connected)
37	WAS pumping	5,446	kW*hr	0.07	$381	5 hp, 4hrs/day
38	RAS pumping	13,070	kW*hr	0.07	$915	2.0 hp, 24hrs/day
Subtotal					$28,000	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Oxidation Ditch w/ conversion of existing Primary Clarifiers to Secondaries

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 034043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Oxidation Ditches - 2@0.30 mg ea (10 day MCRT)										
	1	Concrete - Walls	510	cuyd	150.00	76,500	500.00	255,000			331,500	
	2	Concrete - Slab on grade	420	cuyd	100.00	42,000	350.00	147,000			189,000	
	3	Concrete - Elevated slab		cuyd	100.00		600.00					
	4	Excavation	690	cuyd			15.00	10,350			10,350	
	5	Backfill	320	cuyd	10.00	3,200	10.00	3,200			6,400	
	6	Equipment	1	ls	210,000.00	210,000	42,000.00	42,000			252,000	Lakeside/Goble Sampson
	7	Electrical	1	ls	20,000.00	20,000	10,000.00	10,000			30,000	
	8	Piping	1	ls	50,000.00	50,000	15,000.00	15,000			65,000	
	9	Miscellaneous metals	1	ls	40,000.00	40,000	5,000.00	5,000			45,000	
		Convert Primary to Secondary Clarifier										
	10	Misc modifications	1	ls	100,000.00	100,000					100,000	
		HW modifications	1	ls	200,000.00	200,000					200,000	
		Additional Secondary Clarifiers - 2 @ 55ft dia										
	11	Concrete - Walls	230	cuyd	150.00	34,500	500.00	115,000			149,500	
	12	Concrete - Slab on grade	330	cuyd	100.00	33,000	350.00	115,500			148,500	
	13	Concrete - Elevated slab		cuyd	100.00		600.00					
	14	Excavation	1,800	cuyd			15.00	27,000			27,000	
	15	Backfill	940	cuyd	10.00	9,400	10.00	9,400			18,800	
	16	Sludge Mechanisms	2	ea	82,500.00	165,000	16,500.00	33,000			198,000	$1500/ft/dia, Beaver Eq
	17	WAS/RAS pumping	1.00	ls	30,000.00	30,000	20,000.00	20,000			50,000	
	18	Piping	1	ls	10,000.00	10,000	10,000.00	10,000			20,000	
	19	Miscellaneous metals	1	ls	10,000.00	10,000	2,000.00	2,000			12,000	
		Sludge Pump Building	400	sqft	80.00	32,000	70.00	28,000			60,000	
Subtotals						1,085,600		847,450			1,913,050	

3,644,000

ESTIMATE OF AVERAGE ANNUAL POWER COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
	Power					
30	Aerators	392,098	kW*hr	0.07	$27,447	60 hp, 24hrs/day (120 hp connected)
31	Secondary Clarifier mechanisms	19,605	kW*hr	0.07	$1,372	2 mechanisms @ 1.0 hp, 2 mechanisms @ 0.5 hp, 24hrs/day
32	WAS pumping	5,448	kW*hr	0.07	$381	5 hp, 4hrs/day
33	RAS pumping	13,070	kW*hr	0.07	$915	2.0 hp, 24hrs/day
Subtotal					$30,000	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Additional RBCs

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Current at ENR _____
Escalated to ENR _____

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Additional RBCs - 4, 1st stage										
	1	Concrete - Walls	80	cuyd	150.00	12,000	500.00	40,000			52,000	quanity from cad
	2	Concrete - Slab on grade	140	cuyd	100.00	14,000	350.00	49,000			63,000	
	3	Structural Fill	180	cuyd	15.00	2,700	20.00	3,600			6,300	
	4	Disks & Shafts, SD, motor op	4	ea	110,000.00	440,000	22,000.00	88,000			528,000	
	5	Piping	1	ls	10,000.00	10,000	7,500.00	7,500			17,500	
	6	Replacement RBC equipment	4	ea	100,000.00	400,000	20,000.00	80,000			480,000	
		Additional Secondary Clarifiers - 2, 45'dia										
	7	Concrete - Walls	210	cuyd	150.00	31,500	500.00	105,000			136,500	
	8	Concrete - Slab on grade	290	cuyd	100.00	29,000	350.00	101,500			130,500	
	9	Excavation	4,950	cuyd			15.00	74,250			74,250	
	10	Backfill	2,930	cuyd	10.00	29,300	10.00	29,300			58,600	
	11	Sludge Mechanisms	2	ea	67,500.00	135,000	13,500.00	27,000			162,000	$1500/fdia, Beaver Eq
	12	WAS/RAS pumping	1	ls	25,000.00	25,000	15,000.00	15,000			40,000	
	13	Electrical	1	ls	10,000.00	10,000	15,000.00	15,000			25,000	
	14	Miscellaneous metals	1	ls	7,500.00	7,500	7,500.00	7,500			15,000	
	15	Piping	1	ls	7,500.00	7,500	7,500.00	7,500			15,000	
Subtotals						1,153,500		650,150			1,803,650	

3,435,000

ESTIMATE OF AVERAGE ANNUAL POWER COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
	Power					
21	1st stage RBCs	130,689	kW*hr	0.07	$9,149	4 @ 5 hp ea, 24 hrs/day
22	RBC Blowers (2nd stage)	163,374	kW*hr	0.07	$11,436	25 hp, 24hrs/day (75 hp connected)
23	Secondary Clarifier mechanisms	13,070	kW*hr	0.07	$915	4 mechanisms, 0.5 hp, 24hrs/day
Subtotal					$22,000	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Oxidation Ditch and secondaries w/ Primary in service

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Oxidation Ditch - Orbal 0.4 mg (10 day MCRT w/ primaries)										
	1	Concrete - Walls	340	cuyd	150.00	51,000	500.00	170,000			221,000	
	2	Concrete - Slab on grade	360	cuyd	100.00	36,000	350.00	126,000			162,000	
	3	Concrete - Elevated slab		cuyd	100.00		600.00					
	4	Excavation	1,560	cuyd			15.00	23,400			23,400	
	5	Backfill	410	cuyd	10.00	4,100	10.00	4,100			8,200	
	6	Equipment	1	ls	80,000.00	80,000	24,000.00	24,000			104,000	Orbal, Reilly
	7	Electrical	1	ls	20,000.00	20,000	10,000.00	10,000			30,000	
	8	Piping	1	ls	50,000.00	50,000	15,000.00	15,000			65,000	
	9	Miscellaneous metals	1	ls	40,000.00	40,000	5,000.00	5,000			45,000	
		Secondary Clarifiers - 2 @ 65ft dia										
	11	Concrete - Walls	270	cuyd	150.00	40,500	500.00	135,000			175,500	
	12	Concrete - Slab on grade	460	cuyd	100.00	46,000	350.00	161,000			207,000	
	13	Concrete - Elevated slab		cuyd	100.00		600.00					
	14	Excavation	2,100	cuyd			15.00	31,500			31,500	
	15	Backfill	660	cuyd	10.00	6,600	10.00	6,600			13,200	
	16	Sludge Mechanisms	2	ea	97,500.00	195,000	19,500.00	39,000			234,000	$1500/ft dia, Beaver Eq
	17	WAS/RAS pumping	1.00	ls	30,000.00	30,000	20,000.00	20,000			50,000	
	18	Piping	1	ls	10,000.00	10,000	10,000.00	10,000			20,000	
	19	Miscellaneous metals	1	ls	10,000.00	10,000	2,000.00	2,000			12,000	
		Sludge Pump Building	400	sqft	80.00	32,000	70.00	28,000			60,000	
Subtotals						651,200		810,600			1,461,800	

2,764,000

ESTIMATE OF AVERAGE ANNUAL POWER COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
	Power					
30	Aerators	261,398	kW*hr	0.07	$18,299	40 hp, 24hrs/day (80 hp connected)
31	Secondary Clarifier mechanisms	19,605	kW*hr	0.07	$1,372	2 mechanisms @ 1.0 hp, 2 mechanisms @ 0.5 hp, 24hrs/day
32	WAS pumping	5,446	kW*hr	0.07	$381	5 hp, 4hrs/day
33	RAS pumping	13,070	kW*hr	0.07	$915	2.0 hp, 24hrs/day
Subtotal					$21,000	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - ICEAS SBR

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 038043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		ICEAS SBR and Equalization Basin										
	1	Concrete - Walls	730	cuyd	150.00	109,500	600.00	365,000			474,500	
	2	Concrete - Slab on grade	1,130	cuyd	100.00	113,000	350.00	395,500			508,500	
	3	Concrete - Elevated slab		cuyd	100.00		600.00					
	4	Excavation	4,630	cuyd			15.00	69,450			69,450	
	5	Backfill	3,480	cuyd	10.00	34,800	10.00	34,800			69,600	
	6	Equipment	1	ls	400,000.00	400,000	80,000.00	80,000			480,000	Sanitaire, includes RAS/WAS
	7	Electrical	1	ls	20,000.00	20,000	10,000.00	10,000			30,000	
	8	Piping	1	ls	50,000.00	50,000	15,000.00	15,000			65,000	
	9	Miscellaneous metals	1	ls	40,000.00	40,000	5,000.00	5,000			45,000	
		Blower and Control Building	600	sqft	80.00	48,000	70.00	42,000			90,000	
Subtotals						815,300		1,016,750			1,832,050	

3,489,000

ESTIMATE OF AVERAGE ANNUAL POWER COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
	Power					
30	Blowers	228,724	kW*hr	0.07	$16,011	35 hp, 24hrs/day (100 hp connected)
32	Secondary Clarifier mechanisms	13,070	kW*hr	0.07	$915	4 mechanisms, 0.5 hp, 24hrs/day
33	WAS pumping	5,446	kW*hr	0.07	$381	5 hp, 6hrs/day
34	RAS pumping	13,070	kW*hr	0.07	$915	2.5 hp, 24hrs/day
Subtotal					$18,000	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Dual Selector Activated Sludge Basins w Diffused Aeration, Selector, new Secondaries

Prepared By: H. Ritter
Date Prepared: 4-Aug-04
K/J Proj. No. 030043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
						0		0		0	0	
		Selector Activated Sludge w Diffused Aeration and Blowers (12-day MCRT)						0		0	0	
	1	Concrete - Walls	273	cuyd	150.00	40,977	500.00	136,589		0	177,566	
	2	Concrete - Slab on grade	122	cuyd	100.00	12,214	350.00	42,748		0	54,961	
	3	Concrete - Elevated slab	0	cuyd	100.00	0	600.00	0			0	
	4	Excavation	1,634	cuyd	0.00	0	15.00	24,510			24,510	
	5	Backfill	639	cuyd	10.00	6,392	10.00	6,392			10,784	
	6	Aeration Equipment	1	ls	135,000.00	135,000	33,750.00	33,750			168,750	EDI Aeration and Hoffman Blowers
	7	Blower Building	300	sf	100.00	30,000	100.00	30,000			60,000	
	8	Flow Control Gates and Weirs	1	ls	50,000.00	50,000	25,000.00	25,000			75,000	
	9	Electrical	1	ls	42,187.50	42,188	10,000.00	10,000			52,188	
	10	Piping	1	ls	10,000.00	10,000	5,000.00	5,000			15,000	
	11	Miscellaneous metals	1	ls	25,000.00	25,000	10,000.00	10,000			35,000	
						0		0		0	0	
		Anoxic Selector at 30-minutes Retention										
	1	Concrete - Walls	38	cuyd	150.00	5,782	500.00	19,274		0	25,056	
	2	Concrete - Slab on grade	14	cuyd	100.00	1,428	350.00	4,999		0	6,427	
	3	Excavation	150	cuyd		0	15.00	2,250			2,250	
	4	Submersible Mixers	2	ea	10,000.00	20,000	1,000.00	2,000			22,000	
	5	Gates and Misc Metals	1	ls	15,000.00	15,000	7,500.00	7,500			22,500	
	6	Piping	1	ls	10,000.00	10,000	5,000.00	5,000			15,000	
		Secondary Clarifiers - 2 @ 65ft dia				0		0		0	0	
	1	Concrete - Walls	270	cuyd	150.00	40,500	500.00	135,000		0	175,500	
	2	Concrete - Slab on grade	460	cuyd	100.00	46,000	350.00	161,000		0	207,000	
	3	Concrete - Elevated slab	0	cuyd	100.00	0	600.00	0		0	0	
	4	Excavation	2,100	cuyd	0.00	0	15.00	31,500		0	31,500	
	5	Backfill	660	cuyd	10.00	6,600	10.00	6,600		0	13,200	
	6	Sludge Mechanisms	2	ea	97,500.00	195,000	19,500.00	39,000		0	234,000	$1500/ft dia, Beaver Eq
	7	WAS/RAS pumping	1.00	ls	30,000.00	30,000	20,000.00	20,000		0	50,000	
	8	Piping	1	ls	10,000.00	10,000	10,000.00	10,000		0	20,000	
	9	Miscellaneous metals	1	ls	25,000.00	25,000	15,000.00	15,000		0	40,000	
	10	Sludge Pump Housing	400	sqft	80.00	32,000	70.00	28,000		0	60,000	
Subtotals						788,081		810,112		0	1,598,193	

3,045,000

ESTIMATE OF AVERAGE ANNUAL POWER COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
					0	
	Power				0	
30	Aerators	228,724	kW*hr	0.07	16,011	35 hp, 24hrs/day (100 hp connected)
31	Secondary Clarifier mechanisms	19,605	kW*hr	0.07	1,372	2 mechanisms @ 1.0 hp, 2 mechanisms @ 0.5 hp, 24hrs/day
32	WAS pumping	5,446	kW*hr	0.07	381	5 hp, 4hrs/day
33	RAS pumping	13,070	kW*hr	0.07	915	205 hp, 24hrs/day
					0	
					0	
					0	
					0	
					0	
					0	
Subtotal					19,000	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Effluent Filter for Biotower Alt, summer only

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Effluent Filter - 2.08 mgd (summer)										
		Concrete - Walls	50	cuyd	150.00	7,500	500.00	25,000			32,500	
		Concrete - Slab on grade	30	cuyd	100.00	3,000	350.00	10,500			13,500	
		Concrete - Elevated slab		cuyd	100.00		600.00					
		Excavation	320	cuyd			15.00	4,800			4,800	
		Backfill	250	cuyd	10.00	2,500	10.00	2,500			5,000	
		Equipment	1	ls	250,000.00	250,000	50,000.00	50,000			300,000	AquaAerobics/TEC
		Electrical	1	ls	5,000.00	5,000	2,000.00	2,000			7,000	
		Piping	1	ls	10,000.00	10,000	5,000.00	5,000			15,000	
		Miscellaneous metals										
		Pump Station										
		Concrete - included above										
		Pumps	3	ea	10,000.00	30,000	2,000.00	6,000			36,000	
		Electrical	1	ls	4,000.00	4,000	2,000.00	2,000			6,000	
Subtotals						312,000		107,800			419,800	

800,000

ESTIMATE OF O&M COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
	Power					
	Pumps	26,140	kW*hr	0.07	1,830	8 hp, 24 hrs/day, 6 mo/yr (40 hp connected at PIF)
	Filter	13,070	kW*hr	0.07	915	4 hp, 24 hr/sday, 6 mo/yr
Subtotal					2,800	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Effluent Filter for Oxidation Ditch Alt, summer only

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @
- ☐ Construction
- ☐ Change Order
- % Complete

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Effluent Filter - 2.08 mgd (summer)										
		Concrete - Walls	30	cuyd	150.00	4,500	500.00	15,000			19,500	
		Concrete - Slab on grade	10	cuyd	100.00	1,000	350.00	3,500			4,500	
		Concrete - Elevated slab		cuyd	100.00		600.00					
		Excavation	60	cuyd			15.00	900			900	
		Backfill	50	cuyd	10.00	500	10.00	500			1,000	
		Equipment	1	ls	250,000.00	250,000	50,000.00	50,000			300,000	AquaAerobics/TEC
		Electrical	1	ls	5,000.00	5,000	2,000.00	2,000			7,000	
		Piping	1	ls	10,000.00	10,000	5,000.00	5,000			15,000	
		Miscellaneous metals										
Subtotals						271,000		76,900			347,900	

663,000

ESTIMATE OF O&M COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
	Power					
	Filter	13,070	kW*hr	0.07	915	4 hp, 24 hr/sday, 6 mo/yr
Subtotal					1,000	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Liquid Chlorination, existing dechlorination

Prepared By: C Dummer
Date Prepared: 16-May-03
K/J Proj. No. 036043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Chlorine Modifications										
	1	Storage - 3,500 gal	2	ea	8,500.00	17,000	25,000.00	50,000			67,000	B2 Equipment
	2	Peristaltic Pumps w/ controls	3	ea	6,500.00	19,500	3,000.00	9,000			28,500	Watson/Marlow
	3	Chemical Fill Station	1	ea	5,000.00	5,000	1,500.00	1,500			6,500	
	4	Piping, Valving, and appurt	1	ls	15,000.00	15,000	10,000.00	10,000			25,000	
	5	Electrical	1	ls	4,000.00	4,000	2,000.00	2,000			6,000	
	6	New Chlorine Contact Tank	1	ls	400,000.00	400,000	125,000.00	125,000			525,000	
		Dechlorination Modifications										
	7	Storage - 550 gal HDPE	2	ea	5,000.00	10,000	12,000.00	24,000			34,000	
	8	Peristaltic Pumps w/ controls	2	ea	6,500.00	13,000	3,000.00	6,000			19,000	Watson/Marlow
	9	Piping, Valving, and appurt	1	ls	10,000.00	10,000	10,000.00	10,000			20,000	
	10	Electrical	1	ls	3,000.00	3,000	2,000.00	2,000			5,000	
Subtotals						496,500		239,500			736,000	

1,402,000

ESTIMATE OF O&M COSTS

Chlorine Liquid

ADF(mgd) = 1.87
Dose (mg/L) = 10.00 (15 mg/L to 20 mg/L typical)
Conc. (%) = 12.50
Usage (lb/day) = 156
Usage (gal/day) = 142
Usage (gal/mo) = 4312
Cost ($/gal) = 0.80 (from Sierra Chemical based on 3,0(
Annual Cost ($) = $41,400

Sodium Bisulfite

ADF(mgd) = 1.87
Dose (mg/L) = 7.30 (based on CL2 residual of 5 mg/L)
Conc. (%) = 25.00
Usage (lb/day) = 114
Usage (gal/day) = 52
Usage (gal/mo) = 1574
Cost ($/gal) = 0.70 (from Sierra Chemical based on 3,000 gal delivery)
Annual Cost ($) = $13,222

Now
5 ys
10 ys
15 ys

Total Annual Chemical Cost

Sodium Hypochlorite	$41,400
Sodium Bisulfite	$13,222
Total	$55,000

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Prepared By: C Dummer
Date Prepared: 28-Oct-03
K/J Proj. No. 036043.00

Building, Area: Existing Disinfection - Convert to ton cylinders & add scrubbing

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Chlorine Modifications										
	1	Convert back to ton cylinders	1	ls	2,000.00	2,000	2,000.00	2,000			4,000	
	2	Ton cylinder scale	1	ea	5,000.00	5,000	1,000.00	1,000			6,000	
	3	Chlorine Gas Scrubber	1	ea	90,000.00	90,000	18,000.00	18,000			108,000	APSCO
	4	Automatic Chlorinator - 300 lb	1	ea	10,000.00	10,000	2,000.00	2,000			12,000	
	5	New Chlorine Contact Tank	1	ls	400,000.00	400,000	125,000.00	125,000			525,000	
		Dechlorination Modifications										
	6	Storage - 550 gal HDPE	2	ea	5,000.00	10,000	12,000.00	24,000			34,000	
	7	Peristaltic Pumps w/ controls	2	ea	6,500.00	13,000	3,000.00	6,000			19,000	Walson/Marlow
	8	Piping, Valving, and appurt	1	ls	10,000.00	10,000	10,000.00	10,000			20,000	
	9	Electrical	1	ls	3,000.00	3,000	2,000.00	2,000			5,000	
Subtotals						543,000		190,000			733,000	

1,396,000

ESTIMATE OF O&M COSTS

Chlorine Gas

ADF(mgd) = 1.87
Dose (mg/L) = 17.50 (15 mg/L to 20 mg/L typical)
Conc. (%) = 100.00
Usage (lb/day) = 272

Usage (lb/mo) = 8279
Cost ($/lb) = 0.35
Annual Cost ($) = $34,773

Sodium Bisulfite

ADF(mgd) = 1.87
Dose (mg/L) = 7.30 (based on CL2 residual of 5 mg/L)
Conc. (%) = 25.00
Usage (lb/day) = 114
Usage (gal/day) = 52
Usage (gal/mo) = 1574
Cost ($/gal) = 0.70 (from Sierra Chemical based on 3,000 gal delivery)
Annual Cost ($) = $13,222

Total Annual Chemical Cost

Sodium Hypochlorite $34,773
Sodium Bisulfite $13,222
Total $48,000

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - UV System

Prepared By: C Dummer
Date Prepared: 16-May-03
K/J Proj. No. 036043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		UV System										
	1	New UV Basin and Controls Bld	1	ls	200,000.00	200,000	75,000.00	75,000			275,000	
	2	UV System	1	ls	420,000.00	420,000	84,000.00	84,000			504,000	assumes $40,000/MGD peak flow
	3	Electrical	1	ls	5,000.00	5,000	5,000.00	5,000			10,000	
	4	Piping modifications	1	ls	10,000.00	10,000	5,000.00	5,000			15,000	
Subtotals						635,000		169,000			804,000	

1,531,000

ESTIMATE OF O&M COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
5	Power	115,000	kW*hr	0.07	8,050	10 kW avg load, 24 hrs/day
	UV Bulbs	175	bulbs	215.00	37,625	
Subtotal					45,675	

Now
5 yrs
10 yrs
15 yrs

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Building, Area: New - Aerobic Digesters (1 new, 2 old 2° clair) and Mechanical Dewatering to 14%

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Aerobic Digester - 65'dia										
	1	Concrete - Walls	169	cuyd	150.00	25,300	500.00	84,333			109,633	
	2	Concrete - Slab on grade	208	cuyd	100.00	20,772	350.00	72,703			93,475	
	3	Concrete - Elevated slab		cuyd	100.00		600.00					
	4	Excavation	545	cuyd			15.00	8,168			8,168	
	5	Backfill	218	cuyd	10.00	2,182	10.00	2,182			4,363	
	6	Cover - existing secondaries	3,181	sqft	40.00	127,234	30.00	95,426			222,660	
	7	Odor Control	1	ls	20,000.00	20,000	10,000.00	10,000			30,000	biofilter
	8	Piping	1	ls	10,000.00	10,000	8,000.00	8,000			18,000	
		Convert Existing secondaries	1	ls	25,000.00	25,000	25,000.00	25,000			50,000	
	9	Diffusers, coarse bubble - all 3 ba	1	ls	40,000.00	40,000	12,000.00	12,000			52,000	EDI/APSCO
	10	Blowers, 25 hp - all 3 basins	3	ea	7,500.00	22,500	1,500.00	4,500			27,000	$300/hp, Lamsom/TEC
	11	Electrical	1	ls	10,000.00	10,000	4,000.00	4,000			14,000	20% of equip
	12	Centrifuge/Belt Filter Press w/ Pol	1	ea	150,000.00	150,000	45,000.00	45,000			195,000	Andritz BFP/Centrifuge, APSCO
	13	Building	750	sqft	80.00	60,000	70.00	52,500			112,500	
	14	Piping	1	ls	20,000.00	20,000	10,000.00	10,000			30,000	
	15	Conveyor	1	ls	15,000.00	15,000	3,000.00	3,000			18,000	
	16	Sludge Truck - cake spreader	1	ea	150,000.00	150,000					150,000	Ron est
Subtotals						697,988		436,812			1,134,800	

2,161,000

ESTIMATE OF O&M COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
	Power					
15	Digester blowers	326,748	kW*hr	0.07	22,872	50 hp, 24hrs/day
16	BFP	9,336	kW*hr	0.07	653	30 hp combined (incl dosing pump), 8hrs/wk
	Replace diffusers?					
Subtotal					23,600	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Gravity Thickener and Anaerobic Digester w/ Sludge Storage @ 4.5%

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Gravity Thickener - 12ft dia										
	1	Concrete - Walls	20	cuyd	150.00	3,000	500.00	10,000			13,000	
	2	Concrete - Slab on grade	10	cuyd	100.00	1,000	350.00	3,500			4,500	
	3	Concrete - Elevated slab		cuyd	100.00		600.00					
	4	Excavation	60	cuyd			15.00	900			900	
	5	Backfill	50	cuyd	10.00	500	10.00	500			1,000	
	6	Piping	1	ls	10,000.00	10,000	5,000.00	5,000			15,000	
		Anaerobic Digester - 0.11 MG, 28ft dia										
	7	Concrete - Walls	180	cuyd	150.00	27,000	500.00	90,000			117,000	
	8	Concrete - Slab on grade	50	cuyd	100.00	5,000	350.00	17,500			22,500	
	9	Concrete - Elevated slab	50	cuyd	100.00	5,000	600.00	30,000			35,000	
	10	Excavation	180	cuyd			15.00	2,700			2,700	
	11	Backfill	110	cuyd	10.00	1,100	10.00	1,100			2,200	
	12	Mixer - draft tube w/ HX	1	ea	57,000.00	57,000	17,100.00	17,100			74,100	OTI, TEC
	13	Boiler	1	ea	55,000.00	55,000	16,500.00	16,500			71,500	OTI, TEC
	14	Piping	1	ls	15,000.00	15,000	10,000.00	10,000			25,000	
	15	Electrical	1	ls	20,000.00	20,000	10,000.00	10,000			30,000	20% of equip
	16	Building	600	sqft	80.00	48,000	70.00	42,000			90,000	
		Cover - GT	113	sqft	40.00	4,524	30.00	3,393			7,917	
		Odor Control	1	ls	5,000.00	5,000	2,000.00	2,000			7,000	
		Sludge Storage - 2 @ 0.16 MG each										
	17	Concrete - Walls	320	cuyd	150.00	48,000	500.00	160,000			208,000	
	18	Concrete - Slab on grade	160	cuyd	100.00	16,000	350.00	56,000			72,000	
	19	Concrete - Elevated slab		cuyd	100.00		600.00					
	20	Excavation	480	cuyd			15.00	7,200			7,200	
	21	Backfill	260	cuyd	10.00	2,600	10.00	2,600			5,200	
	22	Floating cover	2	ea	63,900.00	127,800	12,780.00	25,560			153,360	OTI, TEC
	23	Piping and truck loading	1	ls	40,000.00	40,000	20,000.00	20,000			60,000	
	24	Sludge Truck - sprayer	1	ls	150,000.00	150,000					150,000	Ron est
Subtotals						641,524		533,553			1,175,077	

2,238,000

ESTIMATE OF O&M COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
	Power					
24	Mixer, 5 hp	32,675	kW*hr	0.07	2,287	5 hp, 24hrs/day
25	Boiler, 100,000 btu	100	mmBtu	8.00	800	could be zero if digester gas is enough
Subtotal					3,100	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Gravity Thickener and Anaerobic Digester w/ Mechanical Dewatering to 14%

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Gravity Thickener - 12ft dia										
	1	Concrete - Walls	20	cuyd	150.00	3,000	500.00	10,000			13,000	quantity from cad
	2	Concrete - Slab on grade	10	cuyd	100.00	1,000	350.00	3,500			4,500	
	3	Concrete - Elevated slab		cuyd	100.00		600.00					
	4	Excavation	60	cuyd			15.00	900			900	
	5	Backfill	50	cuyd	10.00	500	10.00	500			1,000	
	6	Piping	1	ls	10,000.00	10,000	5,000.00	5,000			15,000	
		Anaerobic Digester - 0.11 MG, 28ft dia										
	7	Concrete - Walls	180	cuyd	150.00	27,000	500.00	90,000			117,000	
	8	Concrete - Slab on grade	50	cuyd	100.00	5,000	350.00	17,500			22,500	
	9	Concrete - Elevated slab	50	cuyd	100.00	5,000	600.00	30,000			35,000	
	10	Excavation	180	cuyd			15.00	2,700			2,700	
	11	Backfill	110	cuyd	10.00	1,100	10.00	1,100			2,200	
	12	Mixer - draft tube w/ HX	1	ea	57,000.00	57,000	17,100.00	17,100			74,100	OTI, TEC
	13	Boiler	1	ea	55,000.00	55,000	16,500.00	16,500			71,500	OTI, TEC
	14	Piping	1	ls	15,000.00	15,000	10,000.00	10,000			25,000	
	15	Electrical	1	ls	20,000.00	20,000	10,000.00	10,000			30,000	20% of equip
	16	Building	600	sqft	80.00	48,000	70.00	42,000			90,000	
	17	Centrifuge/Belt Filter Press w/ Pol	1	ea	150,000.00	150,000	45,000.00	45,000			195,000	
	18	Building	750	sqft	80.00	60,000	70.00	52,500			112,500	
	19	Piping	1	ls	20,000.00	20,000	10,000.00	10,000			30,000	
	20	Sludge Truck - sprayer	1	ls	150,000.00	150,000					150,000	Ron est
Subtotals						627,600		364,300			991,900	

1,889,000

ESTIMATE OF O&M COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
	Power					
24	Mixer, 5 hp	32,675	kW*hr	0.07	2,287	5 hp, 24hrs/day
25	Boiler, 100,000 btu	100	mmBtu	8.00	800	could be zero if digester gas is enough
Subtotal					3,100	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Anaerobic Digester w/ Sludge Storage @ 3.5%

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Anaerobic Digester - 0.26 MG, 42ft dia										
		Concrete - Walls	270	cuyd	150.00	40,500	500.00	135,000			175,500	
		Concrete - Slab on grade	110	cuyd	100.00	11,000	350.00	38,500			49,500	
		Concrete - Elevated slab	100	cuyd	100.00	10,000	600.00	60,000			70,000	
		Excavation	310	cuyd			15.00	4,650			4,650	
		Backfill	160	cuyd	10.00	1,600	10.00	1,600			3,200	
		Mixer - draft tube w/ HX	1	ea	90,000.00	90,000	27,000.00	27,000			117,000	OTI, TEC
		Boiler	1	ea	85,000.00	85,000	25,500.00	25,500			110,500	OTI, TEC
		Piping	1	ls	15,000.00	15,000	10,000.00	10,000			25,000	
		Electrical	1	ls	30,000.00	30,000	10,000.00	10,000			40,000	20% of equip
		Building	600	sqft	80.00	48,000	70.00	42,000			90,000	
		Convert Existing to Sludge Storage										
		Existing Secondary Clarifiers										
		Floating Covers	2	ea	50,000.00	100,000	10,000.00	20,000			120,000	
		Mixers	2	ea	20,000.00	40,000	5,000.00	10,000			50,000	
		Existing Seondary Digester										
		Mixer	1	ea	20,000.00	20,000	5,000.00	5,000			25,000	
		Misc Modifications	1	ls	10,000.00	10,000	5,000.00	5,000			15,000	
		Piping Modifications	1	ls	50,000.00	50,000	20,000.00	20,000			70,000	
		Sludge Storage - 1 @ 0.09 MG										
		Concrete - Walls	120	cuyd	150.00	18,000	500.00	60,000			78,000	
		Concrete - Slab on grade	50	cuyd	100.00	5,000	350.00	17,500			22,500	
		Concrete - Elevated slab		cuyd	100.00		600.00					
		Excavation	170	cuyd			15.00	2,550			2,550	
		Backfill	90	cuyd	10.00	900	10.00	900			1,800	
		Floating cover	2	ea	40,000.00	80,000	8,000.00	16,000			96,000	OTI, TEC
		Piping and truck loading	1	ls	30,000.00	30,000	10,000.00	10,000			40,000	
		Sludge Truck - sprayer	1	ls	150,000.00	150,000					150,000	Ron est
Subtotals						835,000		521,200			1,356,200	

2,583,000

ESTIMATE OF O&M COSTS

Item No.	Description	Qty per Year	Units	Yearly Cost $/Unit	Yearly Cost Total	Assumptions
	Power					
24	Mixer, 5 hp	32,675	kW*hr	0.07	2,287	5 hp, 24hrs/day
25	Boiler, 100,000 btu	100	mmBtu	8.00	800	could be zero if digester gas is enough
Subtotal					3,100	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Outfall

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @
- ☐ Construction
- ☐ Change Order
- % Complete

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Outfall Pipe w/ 2 diffuser ports										
	1	Piping	1	ls	300.00	300	700.00	700			1,000	
		Encase in conc. set w/ backhoe										
	2	River containment - cofferdam	500	sqft	12.00	6,000	8.00	4,000			10,000	
	3	River bed excavation	1	ls			2,000.00	2,000			2,000	
	4	Setting encased pipe	1	ls	500.00	500	5,000.00	5,000			5,500	
Subtotals						6,800		11,700			18,500	

35,000

ALTERNATE INSTALLATION METHOD

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
		Outfall Pipe w/ 2 diffuser ports										
	1	Piping	1	ls	300.00	300	700.00	700			1,000	
	2	Directional Drill ports	2	ea					10,000	20,000	20,000	Michael Humm called
Subtotals						300		700		20,000	21,000	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: New - Standby Power, Diesel Generator

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
	1	Generator & ATS - 750 KW	1	ea	100,000.00	100,000	30,000.00	30,000			130,000	
	2	Above-grade Building	350	sqft	30.00	10,500	70.00	24,500			35,000	
Subtotals						110,500		54,500			165,000	

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook Wastewater Facility Plan

Building, Area: Site Work

Prepared By: CAD - HER
Date Prepared: 1-Nov-04
K/J Proj. No. 036043.00

Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @ % Complete
- ☐ Construction
- ☐ Change Order

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total	Source
	1	Site piping	200	lf	20.00	4,000	80.00	16,000			20,000	
	2	Fencing	500	lf	10.00	5,000	5.00	2,500			7,500	
	3	Landscaping	1	ls	2,000.00	2,000	5,000.00	5,000			7,000	
		Demolition										
	4	South Primary Clarifier	50,000	cuft					0.45	22,500	22,500	
	5	Primary Digester	10,000	cuft					0.45	4,500	4,500	
	6	Secondary Digester	15,000	cuft					0.65	9,750	9,750	
	7	SCADA system (simple)	1	ls	10,000.00	10,000	5,000.00	5,000			15,000	
	8	Site Electrical	1	ls	80,000.00	80,000	20,000.00	20,000			100,000	
	9	Transformer & MCC Panels	1	ls	40,000.00	40,000	20,000.00	20,000			60,000	
Subtotals						141,000		68,500		36,750	246,250	

Hwy. 101 - Step System Inventory

Name of business or property owner	1-Best Western	2-Fred Meyers	3-Shilo Inn	4-Shilo Inn Annex	5-Splash'n'Dash	6-KR Business Park	7-Taco Time	8-Shucks Auto Parts	9-Till. Coastal Plaza
Property address	1722 Main Ave. N.	2500 Main Ave. N	2535 Main Ave. N.	2535 Main Ave. N.	2200 Wilson RV Loop	2507 Main Ave. N	2211 Main Ave. N.		Ben Hathaway(Main St. Pizza) (Bagel shop)
Inspector	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole
Brief description of STEP system installation	4 float duplex	5 float duplex	5 float duplex	4 float duplex 2 stage	3 float simplex 3 stage (2 baffels)	3 float simplex single stage	4 float duplex 2 stage 1 baffle	triangle symplex	triangle duplex
Surface over tank (asphalt, soil, etc.)	concrete	concrete	soil, rock	concrete	concrete	concrete	asphalt	gravel	concrete
Describe the access to the tank (poor, fair, good).	fair	good	good	good	good	good	good	good	good
Is grease a problem at this location?		yes	yes, Shilo has dishwasher plumbed to trap. K&B deli has no trap	no	no, mud problem	no	yes	no	yes
Is a grease trap installed upstream of the STEP tank?		yes, 3	yes, Shilo has dishwasher plumbed to trap. K&B deli has no trap		2-500 gal separator tanks		yes		no traps in pizza parlor or bagle shop
Describe the grease trap.		3-50 gpm, 100 lb	20 gpm 40 lb				40 lb. 20 gm		
Location of grease trap.		deli, fish, & meat dept.	inside preparation area				inside under prep sink		
Date of last grease trap pumping		3 times/yr.	cleaned weekly by staff		pumped out 3-4x ann.		cleaned weekly by staff		
STEP TANK									
Approximate date installed				1990					
Material of construction	fiberglass	concrete	concrete	fiberglass	fiberglass	fiberglass no baffle 1 stage	fiberglass single stage	fiberglass	reinforced fiberglass
Description	1500 gal single	cylindrical single stage	cylindrical single stage	1500 gal single	1500 2 stage	1500 gal	1500 gal	1500 gal single stage	1800 gal 2 stage
Length	8	radius 6'	radius 5'	8	8	8'	8	8	12'
Width	6			6	6	6'	6	6	4'
Total depth	5	15'	12'	5	5	5	5	5	5'
Water depth	3' pump off	6'	6' pump off	3	2' pump	pump off 3'	3' pump off	3' pump off	4'
Liquid volume	12" working volume	2' working volume 3.00 total volume	working liquid level L3'	12" working volume	2 ' working level	12" working water level	12" working water volume	12" working water volume	2' working water volume
Tank condition	good	fair, some I&I	fair some I&I	fiar I&I around barrel	good	fair I&I through barrel	fair, I&I from barrel & bottom (flood plane)	fair I&I from barrel	fair. Some I&I around beams @ top (flood plane)
Date last pumped	9/03	05, 90 day cycle	05, 90 day cycle	10/21/03	05	05	4/03	05	04
PUMPS									
Number of pumps	2	2	2	2	1	1	2	1	2
Type of pump (solids handling, grinder)	solids	solids	grinder	solids	solids	solids	solids	solids	solids
Manufacturer	Myers	Gorman-Rupp	hydromatic	Myers	Myers	Myers	Myers	Myers	Myers
Model #	Whre.20-21c	JSV3C60-x2.7	SPGH 300	WHAE20-21C	WHRE20-21C	WhRE20-21C	WhRE20-21C	me1505-21	WhRE20-21C
Basket or floor mounted?	basket	floor	flo9or	basket	floor	basket	basket	basket	floor
Horsepower	2	2.7	3	2	2	2	2	1-½	2
Power (voltage/phase)	230/Ø	460 / 3Ø	297 3Ø	230 1Ø	230 1Ø	230 1Ø	230 1Ø	230 Ø1	230 Ø1
Approximate age		9 yrs	16 yrs	16 yrs					2 yrs
Condition	good	good	fair some I&I	fair, some I&I from barrel	good	good	good	good	good
Last rebuilt		03	1-04, 1-03			04			04
Power location (overhead or buried)	buried, fed from bldg breaker	buried, fed from inside breaker box	buried from bldg breaker box	buried, fed from breaker box in bldg.	buried from bldg breaker box	buried, fed from externally mounted circuit breaker	buried, fed from circuit breaker box	buried, fed brom inside breaker box	buried, fed brom outside breaker box
CONTROLS									
Description	bld mounted, NEMA 4x enclosure	30x24x12 NEMA 3 R steel inside a 5'4' reinforced fiberglass	pole mounted NEMA 4x enclosure	pole mounted type V5CSAENC4,5 Hoffman	bldg mounted 4x enclsoure	bldg mounted, 4x enclosure	inside bldg mounted, 4x enclosure	bldg mounted 4x enclosure	pole mounted, 4x NEMA enclosure
Manufacturer	Orenco	Gorman Rupp	hydronix	Orenco	Unipac	Unipac	triangle	triangle	triangle
Model number	DAX-2 ROH4	Serial # 95-214-ASUP	953	DA2			D-212-x	5-212-x	D-212-X
Install date		1997-98	1990?	1990					4
Alarms	yes, high, audible & light	yes, high, & low audible & light	yes, high & low audible & light, also inside anniciaton	yes, high audible & light	yes; high audible & light	yes, high audible & light	yes, high audible, & light	yes; high audible & light	yes, high audible & light
Condition (poor, fair, good)	good	good	poor box low enough to get floaded	fair	fair	fair I&I through barrel	good	fair I&I from barrel	good
Run time meter (yes/no)	yes	yes	yes	yes	yes	yes	yes	yes	yes
NOTES		no grease trap in bakery/ produce dept. Produce has industrial garbage disposal. Garb-no brand model #OL-56-FB 5 hp 208/3							

Hwy. 101 - Step System Inventory

Name of business or property owner	10-Debbie DS	11-House	12-House	13-House Bed & Brkfast	14-Abandoned	15-Sandy's house	16-Carquest; Pingles Muffler, Auto Repair	17-Hightide Espresso	18-Subway
Property address		2060 Main Ave. N.	2100 Main Ave. N.	2150 Main Ave. N.	1900 Goodspeed Road	1830 & 1850 Goodspeed Rd.	1865, 1867, 1885 Main Ave. N.	1810 Main Ave. N.	1863 Main Ave. N.
Inspector	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole
Brief description of STEP system installation	symplex 3 float	symplex	symplex	symplex	symplex	symplex 3 float	symplex 3 float	symplex 3 float	duplex
Surface over tank (asphalt, soil, etc.)	soil	soil	soil	soil	soil	soil	asphalt	concrete	concrete
Describe the access to the tank (poor, fair, good).	good	poor	poor	poor	fair	poor	good	good	good
Is grease a problem at this location?	yes	no	no	no	no	no	no	yes	
Is a grease trap installed upstream of the STEP tank?	yes							no	
Describe the grease trap.	100 gal								
Location of grease trap.	outside								
Date of last grease trap pumping	2004								
STEP TANK									
Approximate date installed								reactivated 2	
Material of construction	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass
Description	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage
Length	8	8	8	8	8	8	8	8	8
Width	6	6	6	6	6	6	6	6	6
Total depth	5	5	5	5	5	5	5	5	5
Water depth	3'	3' pump off	3' pump off	3' pump off		3' pump off	3' pump off	3' pump off	3' pump off
Liquid volume	12 working water valume	12" working water volume	12" working water volume	12" working water volume		12" working water volume	12" working water volume	12" working water volume	12" working water volume
Tank condition	fair. I&I through bottom (flood plane)	fair. I&I from barrel (flood plane)	fair. I&I from barrel (flood plane)	fair. I&I from barrel (flood plane)	haven't looked in 5 years (flood plane)	fair. I&I from barrel (flood plane)	fair. I&I from barrel (flood plane)	fair. I&I from barrel (flood plane)	good
Date last pumped	2005	4/04	2/5/04	11/98		6/05	05	05	05/02
PUMPS									
Number of pumps	1	1	1	1	0	1	1	1	2
Type of pump (solids handling, grinder)	Orecon 4" stacker pump, solids 3/16	solids	solids	solids		solids	solids	solids	solids (3/16)
Manufacturer	Orenco	Myers	Myers	Myers		Myers	Myers	Myers	Orenco
Model #	20SC2F-03	WhRE20-21C	WhRE20-21C	WhRE20-21C		WhRE20-21C	WhRE20-21C	WhRE20-21C	
Basket or floor mounted?	basket	basket	basket	basket		basket	basket	basket	basket
Horsepower	½	2	2	2		2	2	2	¾
Power (voltage/phase)	230 Ø1	230 Ø1	230 Ø1	230 Ø1		230 Ø1	230 Ø1	230 Ø1	230 Ø1
Approximate age									
Condition	fair	good	good	good		good	good	good	
Last rebuilt									
Power location (overhead or buried)	buried. Fed from bldg. breaker box	buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box
CONTROLS									
Description	bldg mounted UL Labs #5462720 Hoffman	bldg mounted 4x enclosure	bldg mounted 4x enclosure	bldg mounted 4x enclosure	bldg mounted 4x enclosure	bldg mounted 4x enclosure	bldg mounted 4x enclosure	pole mounted 4x enclosure	bldg mounted NEMA 4x enclosure
Manufacturer	Orenco	Triangle	Triangle	Triangle	Triangle	Triangle	Triangle	Triangle	Orenco
Model number	DAS-2	5-212-K	5-212-K	5-212-K	5-212-K	5-212-K	5-212-K	5-212-K	DAX-1R0-HT
Install date									
Alarms	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light
Condition (poor, fair, good)	good	fair	fair	fair	fair	fair	fair	good	good
Run time meter (yes/no)	yes	yes	yes	yes	yes	yes	yes	yes	yes

Hwy. 101 - Step System Inventory

Name of business or property owner	19-Hearth & Home	20-House	21-McDonalds	22-Buss Welding	23-Oil Can Henry's	24-Lindsay's Lattes #1	25-Co-Op	26-Duplex	27-Coast 105.5 Office
Property address	2180 Larson Rd.	2195 Larson Rd.	1855 Main Ave. N.	2175 Larson Rd.	1845 Main Ave. N.	1920 Main Ave. N.	1920 Main Ave. No.	1580 & 1610 Main Ave. N	1550 Main Ave. N.
Inspector	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole
Brief description of STEP system installation	symplex 3x float	symplex 3x float	Orenco 4 float	symplex 3x float	duplex 4 float	3 float symplex	3 float symplex	3 float symplex	3 float symplex
Surface over tank (asphalt, soil, etc.)	soil	gravel	soil	gravel	concrete	asphalt	asphalt	soil & 4x4 concrete pad	gravel
Describe the access to the tank (poor, fair, good).	fair	good	good	good	fair	good	good	fair	fair
Is grease a problem at this location?	no	no	yes	no	no. oil problem	yes	no	no	no
Is a grease trap installed upstream of the STEP tank?			yes		inceptor inside	no			
Describe the grease trap.			2500 gal inceptor						
Location of grease trap.			outside, before step tank						
Date of last grease trap pumping			05 90 day cycle						
STEP TANK									
Approximate date installed									
Material of construction	fiberglass	fiberglass	concrete	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass
Description	1500 gal single stage	1500 gal single stage	1000 gal pump vault 3000 gal 2 stage filter tanks	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage
Length	8	8		8	8	8	8	8	8
Width	6	6		6	6	6	6	6	6
Total depth	5	5		5	5	5	5	5	5
Water depth	3' pump off	3' pump off		3' pump off	3' pump off	3' pump off	3' pump off	3' pump off	3' pump off
Liquid volume	12" working water volume	12" working water volume		12" working water volume	12" working water volume	12" working water volume	12" working water volume	12" working level	12" working level
Tank condition	fair. I&I from barrel (flood plane)	fair. I&I from barrel (flood plane)	fair. I&I from barrel (flood plane)	fair. I&I from barrel (flood plane)	fair. I&I from barrel (flood plane)	good. No I&I (flood plane)	good. No I&I (flood plane)	fair. I&I around barrel (flood plane)	fair. I&I around barrel
Date last pumped	7/03	7/03	05	2/04	2/04	6/05	4/04	5/12/05	5/05
PUMPS									
Number of pumps	1	1	2	1	2	1	1	1	1
Type of pump (solids handling, grinder)	solids	solids	solids	solids	solids	solids	solids	solids	solids
Manufacturer	Myers	Myers	Orenco	Myers	Myers	Myers	Myers	Myers	Myers
Model #	WhRE20-21C	WhRE20-21C	205C1F-03	WhRE20-21C	WhRE20-21C	WhRE20-21C	WhRE20-21C	WhRE20-21C	WhRE20-21C
Basket or floor mounted?	basket	basket	basket	basket	basket	basket	basket	basket	basket
Horsepower	2	2	½	2	2	2	2	2	2
Power (voltage/phase)	230 Ø1	230 Ø1	120 Ø1	230 Ø1	280 Ø1	230 Ø1	230 Ø1	230 Ø1	230 Ø1
Approximate age									
Condition	good	good	fair	good	good	good	good	good	good
Last rebuilt								11/82	
Power location (overhead or buried)	buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box		buried. Fed brom bldg breaker box		buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box
CONTROLS									
Description	bldg mounted 4x enclosure	inside bldg mounted NEMA 4x enclosure	pole mounted, NEMA 4x enclosure	bldg mounted NEMA x enclosure	bldg mounted NEMA x enclosure	inside bldg mounted 4x enclosure	bldg mounted 4x enclosure	bldg mounted 4x enclosure	bldg mounted 4x enclosure
Manufacturer	Triangle	Triangle	Orenco	S.J.E. Rhombus	S.J.E. Rhombus	Triangle	UNIPAC	Triangle	Triangle
Model number	5-212-K	5-212-K	DAX1-tx	1157W	1221W110H	5-212-K		5-212-K	5-212-K
Install date									
Alarms	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light
Condition (poor, fair, good)	fair	good	good	good	good	good	good	fair	fair
Run time meter (yes/no)	yes	yes	yes	yes	yes	yes	yes	yes	yes

Hwy. 101 - Step System Inventory

Name of business or property owner	28-Ford Lot	29-Pizza Hut	30-Averil Trucking Office	31-Uhaul	32-#1 Main	33-#2 House	34-#3 House	35-Coldwell Banker	36-House (Kyook Rentals)
Property address	1695 Main Ave. N.	1605 Main Ave. N.	1500 Main Ave. N.	1565 Main Ave. N.	2001 Blue Heron Dr.	2001 Blue Heron Dr.	2001 Blue Heron Dr.	1180 Main Ave. N.	1240 Main Ave. N.
Inspector	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole
Brief description of STEP system installation	3 float symplex	4 float Orenco Duplex	3 float symplex	3 float symplex	4 float duplex	4 float duplex	Abandoned	3 float symplex	3 float symplex
Surface over tank (asphalt, soil, etc.)	gravel & soil	concrete	soil	gravel	concrete	concrete	soil	gravel	soil
Describe the access to the tank (poor, fair, good).	fair	good	poor	good	good	good	fair	good	fiar, in day, poor when wet
Is grease a problem at this location?	no	yes	no	no		no		no	no
Is a grease trap installed upstream of the STEP tank?									
Describe the grease trap.									
Location of grease trap.									
Date of last grease trap pumping									
STEP TANK									
Approximate date installed									
Material of construction	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass
Description	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage
Length	8	8	8	8	8	8	8	8	8
Width	6	6	6	6	6	6	6	6	6
Total depth	5	5	5	5	5	5	5	5	5
Water depth	3' pump off	3' pump off	3' pump off	3' pump off	3' pump off	3' pump off	-	3' pump off	3' pump off
Liquid volume	12" working level	12" working volume	12" working volume	12" working volume	12" working volume	12" working volume	-	12" working volume	12" working volume
Tank condition	fair. I&I around barrel	fair. I&I around barrel	fair. I&I around barrel	good	fair. I&I around barrel	fair. I&I around barrel	-	fair. I&I around barrel	fair. I&I around barrel
Date last pumped	very little use, 2003	05, 3-4 month cycle	5/11/05	6/2/05	7/02	7/02	05 puped out & filled with fresh water	3/05	05
PUMPS									
Number of pumps	1	2	1	1	2	2	0	1	1
Type of pump (solids handling, grinder)	solids	solids	solids	solids	solids	solids		solids	solids
Manufacturer	Myers	Orenco	Myers	Myers	Myers	Myers		Myers	Myers
Model #	WhRE20-21C	205C1F-03	WhRE20-21C	WhRE20-21C	WhRE20-21C	WhRE20-21C		WhRE20-21C	WhRE20-21C
Basket or floor mounted?	basket	basket	basket	basket	basket	basket		basket	basket
Horsepower	2	½	2	2	2	2		2	2
Power (voltage/phase)	230 Ø1	120 Ø1	230 Ø1	230 Ø1	230 Ø1	230 Ø1		230 Ø1	230 Ø1
Approximate age									
Condition	good	fair	fair	good	good	fair		good	good
Last rebuilt				05					
Power location (overhead or buried)	buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box		buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box		buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box
CONTROLS							No controls		
Description	pole mounted 4x enclosure	bldg mounted x enclosure	bldg mounted 4x enclosure	bldg mounted 4x enclosure	bldg mounted 4x enclosure	bldg mounted 4x enclosure		inside bldg mounted 4x enclosure	inside bldg mounted 4x enclosure
Manufacturer	Triangle	Orenco	Triangle	Triangle	Triangle	Triangle		UNIPAC	UNIPAC
Model number	5-212-K	DAX1-tx	5-212-K	5-212-K	D-212-K	D-212-K			
Install date									
Alarms	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light		yes, high audible & light	yes, high audible & light
Condition (poor, fair, good)	fair	good	fair	fair	fair	fair		fair	fair
Run time meter (yes/no)	yes	yes	yes	yes	yes	yes		yes	yes
NOTES				6/8/05-new wires pulled to tank from control panel					

Hwy. 101 - Step System Inventory

Name of business or property owner	37-LaCasa Medello	38-Locomotion	39-Western Royal Inn	40-RG Wyss	41-Tillamook Veterinary Hospital	42-Produce Stand Valley Fresh	43-North Port Plaza, Suite 1-9	44-North Port Plaza, Suites 10-13	45-Bluestar Espresso
Property address	1160 Main Ave. N.	1145 Main Ave. N.	1125 Main Ave. N.	1100 Main Ave. N.	1095 Main Ave. N.	1085 Main Ave. N.	1000 Main Ave. N.	1000 Main Ave. N.	940 Main Ave. N.
Inspector	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole
Brief description of STEP system installation	3 float symplex	3 float symplex	4 float symplex	3 float symplex	3 float symplex	3 float symplex	3 float symplex	3 float symplex	3 float symplex
Surface over tank (asphalt, soil, etc.)	asphalt	gravel	asphalt	soil	concrete	soil	asphalt	asphalt	soil
Describe the access to the tank (poor, fair, good).	good	fair	good	poor	good	fair	good	good	good
Is grease a problem at this location?	yes	yes		no	no	no	yes	yes	no
Is a grease trap installed upstream of the STEP tank?	yes (bad)	yes but dishwahers is plumbed to it					yes	yes	
Describe the grease trap.	40 lb 20 gpm	40 lb 20 gpm					20 gpm, 40 lb	Zurn 500, 50 gpm 80 lb.	
Location of grease trap.	outside (trap undersized)	outside (trap undersized)					inside, under prep sink	floor of bakery	
Date of last grease trap pumping									
STEP TANK									
Approximate date installed									
Material of construction	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass
Description	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage
Length	8	8	8	8	8	8	8	8	8
Width	6	6	6	6	6	6	6	6	6
Total depth	5	5	5	5	5	5	5	5	5
Water depth	3' pump off	3' pump off	3' pump off	3' pump off	3' pump off	3' pump off	3' pump off	3' pump off	3' pump off
Liquid volume	12" working volume	12" working volume	12" working volume	12" working volume	12" working volume	12" working volume	12" working volume	12" working volume	12" working volume
Tank condition	fair. I&I around barrel	fair. I&I around barrel	fair. I&I around barrel	fair. I&I around barrel	fair. I&I around barrel	good	Fair. I&I around tank & barrel	Fair. I&I around tank & barrel	Fair. I&I around tank & barrel
Date last pumped	04	04	1/29/04	5/20/98	05	new install	04	04	4/21/05
PUMPS									
Number of pumps	1	1	2	1	1	1	1	1	1
Type of pump (solids handling, grinder)	solids	solids	solids	solids	solids	solids	solids	solids	solids
Manufacturer	Myers	Myers	Myers	Myers	Myers	Orenco	Myers	Myers	Myers
Model #	WhRE20-21C	WhRE20-21C	WhRE20-21C	WhRE20-21C	WhRE20-21C		WhRE20-21C	WhRE20-21C	WhRE20-21C
Basket or floor mounted?	basket	basket	basket	basket	basket	basket	basket	basket	basket
Horsepower	2	2	2	2	2	2	2	2	1-½
Power (voltage/phase)	230 Ø1	230 Ø1	230 Ø1	230 Ø1	230 Ø1	230 Ø1	230 Ø1	230 Ø1	230 Ø1
Approximate age						installed 9/1/04			newly rebuilt
Condition	good	good	good	good	good	new	good	good	good
Last rebuilt						04			05
Power location (overhead or buried)	buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box	buried. Fed brom bldg breaker box	Buried. Fed brom bldg breaker box	Buried. Fed brom bldg breaker box	Buried. Fed brom bldg sewer	Buried. Fed brom bldg breaker box	Buried. Fed brom bldg breaker box	Buried. Fed brom bldg breaker box
CONTROLS									
Description	bldg mounted 4x enclosure	bldg mounted 4x enclosure	inside bldg mounted, x enclosure	bldg mounted 4x enclosure	bldg mounted 4x enclosure	bldg mounted 3x enclosure	Bldg mounted, 4 x enclosure	Bldg mounted, 4 x enclosure	Bldg mounted, 4 x enclosure
Manufacturer	UNIPAC	UNIPAC	Orenco	Triangle	Unipac	Orenco	Unipac	Unipac	Unipac
Model number			DA-2	5-212-x		5/2/2005			
Install date						9/1/2004			
Alarms	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light
Condition (poor, fair, good)	fair	fair	fair	fair	fair	good	fair	fair	fair
Run time meter (yes/no)	yes	yes	yes	yes	yes	yes	yes	yes	yes

Hwy. 101 - Step System Inventory

Name of business or property owner	46-Papa Murphy's	47-RV Tillamook Repair, Keen Edge (empty)	48-Safeway	49-Rental Center	50-RC Shop	51-Discount Roads, Mexican Food Rest, Kens Big Tire	52-Lins Restaurant	53-Texaco	54-Dairy Queen
Property address	2140 Hadley Rd.	2150 Hadley Rd.	955 Main Ave. N.	840 Main Ave. N	800 Main Ave. N.	540 & 542 Main Ave. N.	500 Main Ave. N.	416 Main Ave. N.	440 Main Ave. N.
Inspector	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole	Dave Cole
Brief description of STEP system installation	3 float symplex	3 float symplex	3 float symplex	3 float symplex	3 float symplex	3 float symplex	3 float symplex	3 float symplex	3 float symplex
Surface over tank (asphalt, soil, etc.)	asphalt	asphalt	soil	soil	soil	gravel	soil	gravel	concrete
Describe the access to the tank (poor, fair, good).	good	good	fair	fair	fair	fair	good	good	good
Is grease a problem at this location?	yes	no	yes	no	no		yes	no	yes
Is a grease trap installed upstream of the STEP tank?	no		yes				yes	yes	yes
Describe the grease trap.			3-40lb. 20 gpm				40 lb, 20 gpm	40 lb, 20 gpm	40 lb, 20 gpm
Location of grease trap.			Bakery, Meat Dept, Deli				outside building	outside building	inside
Date of last grease trap pumping									
STEP TANK									
Approximate date installed									
Material of construction	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass	fiberglass
Description	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage
Length	8	8	8	8	8	8	8	8	8
Width	6	6	6	6	6	6	6	6	6
Total depth	5	5	5	5	5	5	5	5	5
Water depth	3' pump off	3' pump off	3' pump off	3' pump off	3' pump off	3' pump off	3' pump off	3' pump off	3' pump off
Liquid volume	12" working volume	12" working volume	12" working level	12" working volume	12" working volume	12" working volume	12" working volume	12" working volume	12" working volume
Tank condition	fair. I&I around barrel	fair. I&I around barrel	Poor. Alot of I&I around tank. Bottom buckled.	Fair. I&I around barrel	Fair. I&I around barrel	Fair. I&I around barrel	Fair. I&I around barrel	Fair. I&I around barrel	Fair. I&I around barrel
Date last pumped	7/7/04	2/14/05	05	4/05	2/16/05	2000	04	9/23/02	9/02
PUMPS									
Number of pumps	1	1	1	1	1	1	1	1	1
Type of pump (solids handling, grinder)	solids	solids	solids	solids	solids	solids	solids	solids	solids
Manufacturer	Myers	Myers	Myers	Myers	Myers	Myers	Myers	Myers	Myers
Model #	WhRE20-21C	WhRE20-21C	WhRE20-21C	WhRE20-21C	WhRE20-21C	WhRE20-21C	WhRE20-21C	WhRE20-21C	WhRE20-21C
Basket or floor mounted?	basket	basket	basket	basket	basket	basket	basket	basket	basket
Horsepower	2	2	2	2	2	2	2	2	2
Power (voltage/phase)	230 Ø1	230 Ø1	230 Ø1	230 Ø1	230 Ø1	230 Ø1	230 Ø1	230 Ø1	230 Ø1
Approximate age				newly rebuilt					
Condition	good	good	good	good	good	good	good	good	good
Last rebuilt				05					
Power location (overhead or buried)	Buried. Fed brom bldg breaker	Buried. Fed brom bldg breaker	Buried. Fed brom bldg breaker	Buried. Fed brom bldg breaker	Buried. Fed brom bldg breaker	Buried. Fed brom bldg breaker	Buried. Fed brom bldg breaker	Buried. Fed brom bldg breaker	Buried. Fed brom bldg breaker
CONTROLS									
Description	bldg mounted 4x enclosure	bldg mounted 4x enclosure	Pole mounted, 4x enclosure	bldg mounted 4x enclosure	bldg mounted 4x enclosure	Bldg mounted NEMA 4x enclosure	Pole mounted 4x NEMA enclosure	Bldg mounted NEMA 4x enclosure	Bldg mounted NEMA 4x enclosure
Manufacturer	UNIPAC	UNIPAC	UNIPAC	UNIPAC	UNIPAC	UNIPAC	UNIPAC	UNIPAC	UNIPAC
Model number									
Install date									
Alarms	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light
Condition (poor, fair, good)	fair	fair	fair	fair	fair	fair	fair	fair	fair
Run time meter (yes/no)	yes	yes	yes	yes	yes	yes	yes	yes	yes

Hwy. 101 - Step System Inventory

Name of business or property owner	55-Grunders	56-Cash Co & Storage	57-Rosebergs	58-Power Motors					
Property address	405 Main Ave. N.	575 & 535 Main Ave. N.	#2 Main Ave. N.	#1 Main Ave. N.					
Inspector	Dave Cole	Dave Cole	Dave Cole	Dave Cole					
Brief description of STEP system installation	3 float symplex	3 float symplex	3 float symplex	3 float symplex					
Surface over tank (asphalt, soil, etc.)	gravel	soil	gravel	asphalt					
Describe the access to the tank (poor, fair, good).	good	poor	poor-fair	good					
Is grease a problem at this location?	no	no	no	no					
Is a grease trap installed upstream of the STEP tank?									
Describe the grease trap.									
Location of grease trap.									
Date of last grease trap pumping									
STEP TANK									
Approximate date installed									
Material of construction	fiberglass	fiberglass	fiberglass	fiberglass					
Description	1500 gal single stage	1500 gal single stage	1500 gal single stage	1500 gal single stage					
Length	8	8	8	8					
Width	6	6	6	6					
Total depth	5	5	5	5					
Water depth	3' pump off	3' pump off	3' pump off	3' pump off					
Liquid volume	12" working volume	12" working volume	12" working volume	12" working volume					
Tank condition	Fair. I&I around barrel	Fair. I&I around barrel	Fair. I&I around barrel	Fair. I&I around barrel					
Date last pumped	2/23/05	5/5/04	05	2/10/05					
PUMPS									
Number of pumps	1	1	1	1					
Type of pump (solids handling, grinder)	solids	solids	solids	solids					
Manufacturer	Myers	Myers	Myers	Myers					
Model #	WhRE20-21C	WhRE20-21C	WhRE20-21C	WhRE20-21C					
Basket or floor mounted?	basket	basket	basket	basket					
Horsepower	2	2	2	2					
Power (voltage/phase)	230 Ø1	230 Ø1	230 Ø1	230 Ø1					
Approximate age									
Condition	good	good	good	good					
Last rebuilt									
Power location (overhead or buried)	Buried. Fed brom bldg breaker	Buried. Fed brom bldg breaker	Buried. Fed brom bldg breaker	Buried. Fed brom bldg breaker					
CONTROLS									
Description	Bldg mounted NEMA 4x enclosure	Bldg mounted 4x enclosure	Bldg mounted 4x enclosure	Bldg mounted 4x enclosure					
Manufacturer	UNIPAC	UNIPAC	UNIPAC	UNIPAC					
Model number									
Install date									
Alarms	yes, high audible & light	yes, high audible & light	yes, high audible & light	yes, high audible & light					
Condition (poor, fair, good)	fair	fair	fair						
Run time meter (yes/no)	yes	yes	yes						

Appendix K

Revised Loading Criteria

Introduction

Early in the Facility Plan process, daily operating report (DOR) data from 1999 to 2003 was evaluated but rejected due to errors resulting from the recycle of digester supernatant and secondary solids to the sample point in the headworks channel compounded by the erroneous flows recorded by the existing flow meters. For this reason, the Facility Plan design criteria were based on the simplified approach of applying an average loading factor to the service area population. Additionally, the BOD and TSS loads were assumed to be constant year-round regardless of dry and wet weather flow patterns or seasonal variations in loading. This is a reasonable assumption since Tillamook has a consistent year-round population, is not significantly affected by seasonal tourism, and does not normally experience periodic or seasonal variations in sewage strength from its existing commercial customers. There are, however, treatment plant operation procedures that can cause variable loading to the treatment plant.

In the spring of 2003, a Corrective Action Plan (CAP) was completed. The CAP incorporated the following improvements for sampling and record keeping of influent loading criteria:

- Relocation of the influent sampler upstream of the secondary solids return point in the headworks.
- Installation of a new effluent flow meter and recalibration of the existing influent flow meter.
- Construction of a new digester supernatant pump station, allowing the digester recycle stream to return downstream of the influent sampler.
- Installation of electrical conduit to connect the influent flow meter signal to the influent sampler so that samples could be made proportional to plant flow.

To check the validity of the loading estimates made in the early version of the Facility Plan, data for the period following completion of the CAP improvements was examined. The results of this analysis are detailed in this document.

Flow and Loading Data

Flow and loading data from May 2003 through May 2005 DORs were summarized and graphed. The data appears in Table 1 and is shown graphically on Figures 1, 2 and 3. BOD and TSS are typically sampled nine to twelve days each month, for two or three consecutive days at a time.

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Revised Loading Criteria
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Table 1 – Flow and Loading Data (May 2003 to May 2005)

Month	Monthly Average Flow (mgd)	Monthly Average BOD (mg/l)	Monthly Average BOD (ppd)	Monthly Average TSS (mg/l)	Monthly Average TSS (ppd)
May-03	0.96	142	1017	113	1058
Jun-03	0.58	211	992	251	1156
Jul-03	0.41	316	1023	435	1427
Aug-03	0.40	304	1014	357	1178
Sep-03	0.40	299	998	307	1014
Oct-03	0.74	187	1238	177	1096
Nov-03	1.18	158	1311	187	1691
Dec-03	1.80	115	1625	118	1662
Jan-04	1.78	91	1079	124	1670
Feb-04	1.54	78	958	79	978
Mar-04	1.06	143	1225	141	1143
Apr-04	0.86	158	1172	170	1266
May-04	0.72	268	1567	268	1618
Jun-04	0.86	164	1078	159	1083
Jul-04	0.49	326	1310	263	1064
Aug-04	0.57	294	1370	314	1245
Sep-04	0.87	219	1240	208	1187
Oct-04	1.02	155	1354	173	1454
Nov-04	0.97	149	1209	149	1211
Dec-04	1.32	124	1189	110	1082
Jan-05	1.16	148	1350	184	1710
Feb-05	0.78	192	1203	158	978
Mar-05	0.90	163	1233	170	1306
Apr-05	1.30	123	1176	150	1595
May-05	0.90	172	1331	182	1372
Average	0.94	188	1210	198	1290
Minimum	0.40	78	958	79	978
Maximum	1.80	326	1625	435	1710

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Revised Loading Criteria
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Discussion

The DOR data indicate that the BOD and TSS concentration is inversely proportional to influent flow, as would be expected. The data also indicates that the average BOD loading to the plant is in the range of 1,000 ppd to 1,200 ppd with peaks over 1,600 ppd. The TSS concentration falls within a similar, but slightly higher range of values.

Inaccuracies in the reported values could occur as a result of the following:

- Test days may not coincide with peak flow or low flow days. If peak flow days do not coincide with test days, the average monthly loading may be overstated since the reported averages are a product of the average monthly flow and the average monthly loading. Conversely, if low flow days do not coincide with test days the opposite is true and the reported average loading value may be overstated. Examination of the DORs indicate that both scenarios are equally likely to occur, therefore, averaging the effect of the data.

- Operational situations can skew data. This is documented by the wide range of loading values recorded over a single month period when flows are consistent. Probably the most significant causes for wide variations in loading data under uniform flow conditions are the presence of rags caught on the sampling tube (resulting in under-reporting loads), inadvertent recycling of digester supernatant back into the influent pump station (possible if the valving is not set to direct flow to the supernatant pump station), and periodic recycling of stored solids (by accident or during routine transfer of stored solids from basin to basin).

The City maintains 63 STEP units. STEP tank disposal to the treatment plant is a potential source of very high strength BOD and TSS, depending on the customer. If disposal of a STEP tank occurs on a sample day, it will skew the data to the high side. Conversely, STEP tank disposal occurring when the sampler is inactive could result in under reported loads. Since both scenarios are equally likely to occur, it is assumed that the monthly average value represents an average.

Periodic loadings of extremely high strength waste are rare but do occur. On two occasions in 2004, a local dairy farmer may have dumped spoiled milk into the collection system. Other anomalies in sewage strength and characteristics have occurred over the years. While these may show up on the DOR as an extreme event, they do not significantly alter the overall loading to the plant.

Analysis of the DOR records from the past two years indicates that the original Facility Plan estimates (shown in Section 6) likely understated the daily BOD and TSS load estimate. Additionally, while the average BOD and TSS production load is independent of the season, STEP system pumping, changes in plant operations, and fluctuating commercial discharges could easily account for the approximate 25% increase in peak to average monthly values.

Appendix K

Revised Loading Criteria
Page 4

Conclusion

Based on the DOR information collected over the past two years, the average per capita BOD and TSS contribution including all residential and commercial sources is approximately 1,250 ppd. The peak month BOD and TSS is approximately 1,625 and 1,700 ppd respectively.

Based on a current service area population of approximately 4,625, this equates to a per capita BOD contribution of approximately 0.27 ppd on an annual average basis and 0.35 ppd for the peak month. Average annual monthly and peak values for TSS are 0.27 and 0.37 ppd per person, respectively. Influent ammonia criteria are based on 0.055 ppd per person.

Recommendations

It is recommended that the design loading for the WWTP expansion be based on the criteria presented in Table 2. These values are based on a current population of 4,625 and the 20-year design population of 7,500.

Table 2 – Design Criteria

Criteria	Unit	Startup	Design
Average Month BOD	ppd	1,250	2,100
Peak Month BOD	ppd	1,620	2,700
Average Month TSS	ppd	1,250	2,100
Peak Month TSS	ppd	1,710	2,850
Ammonia	ppd	255	413

FIGURE 1
MONTHLY AVERAGE INFLUENT FLOW
May 2003 - May 2005

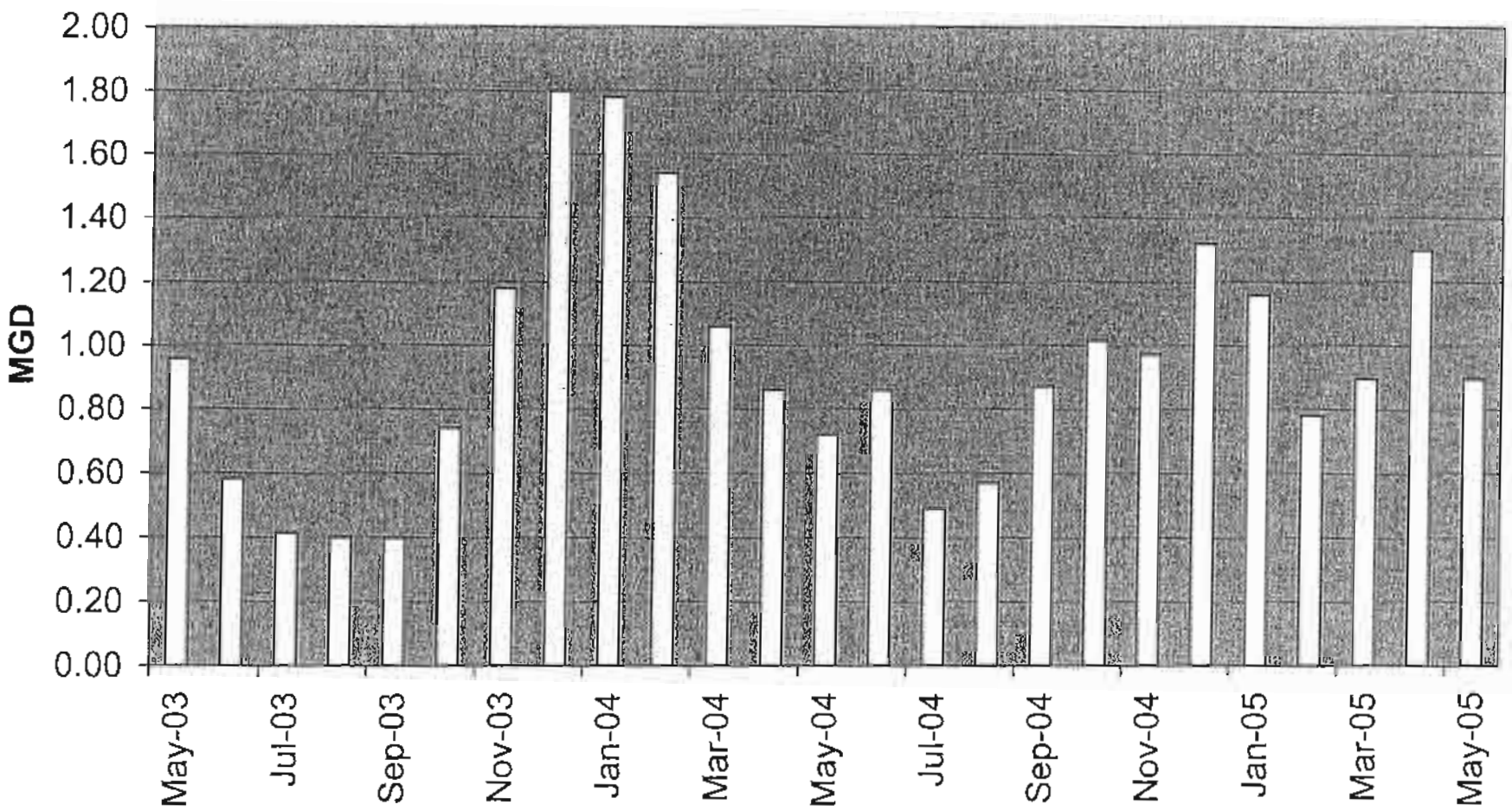

FIGURE 2
MONTHLY AVERAGE INFLUENT BOD & TSS
May 2003 - May 2005

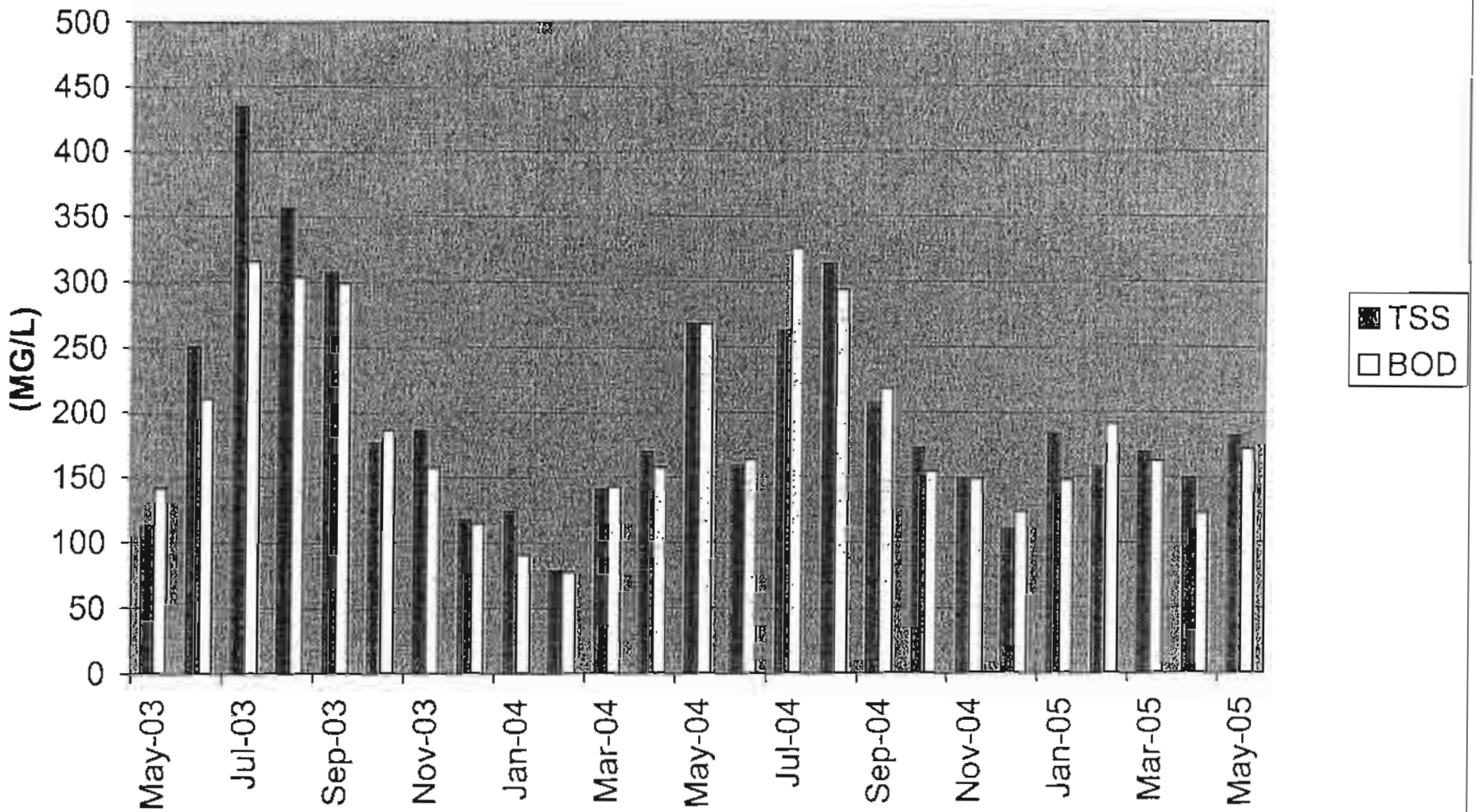
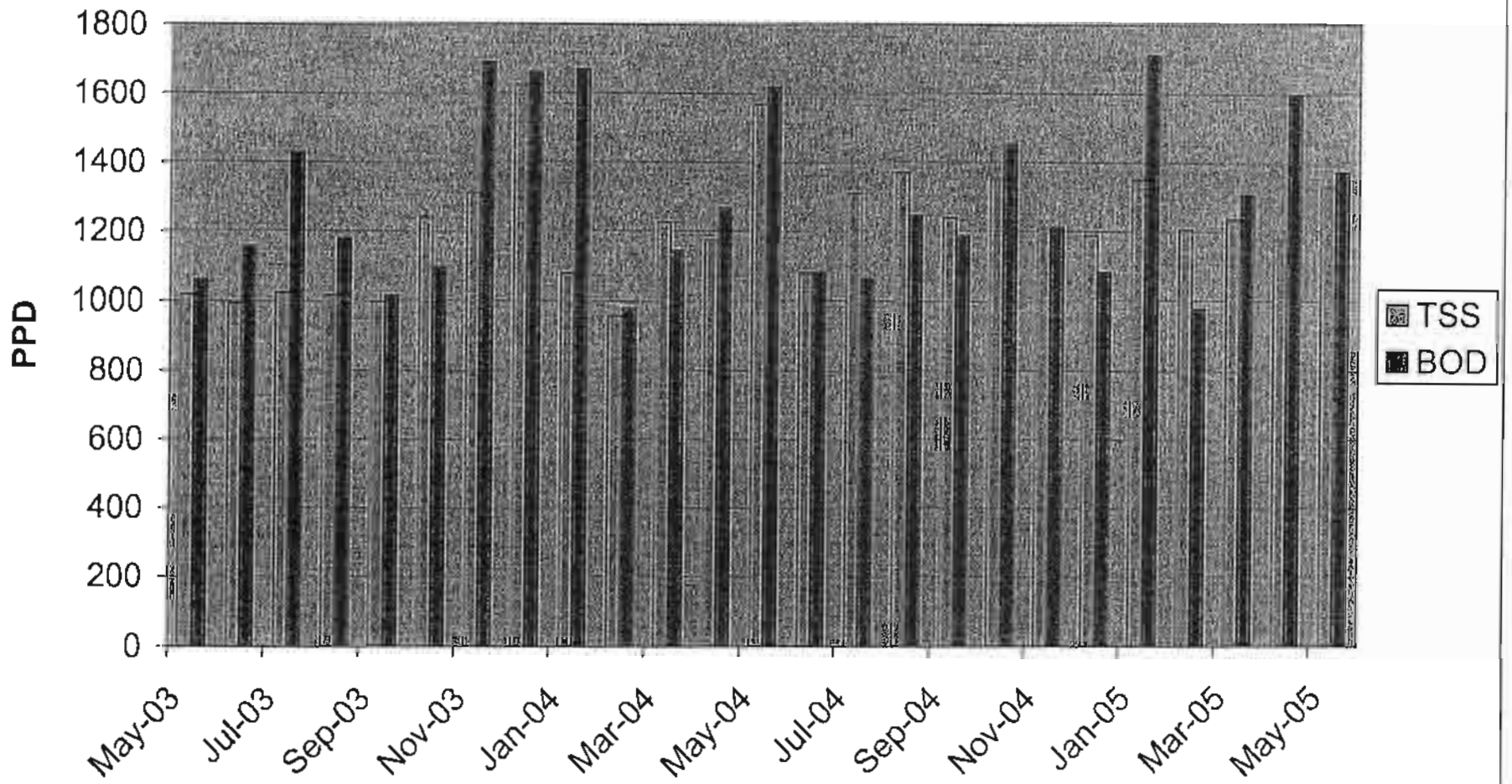
FIGURE 3
MONTHLY AVERAGE INFLUENT BOD & TSS
May 2003 - May 2005