Volume I – Public Facilities Plans (1995 through 2002)

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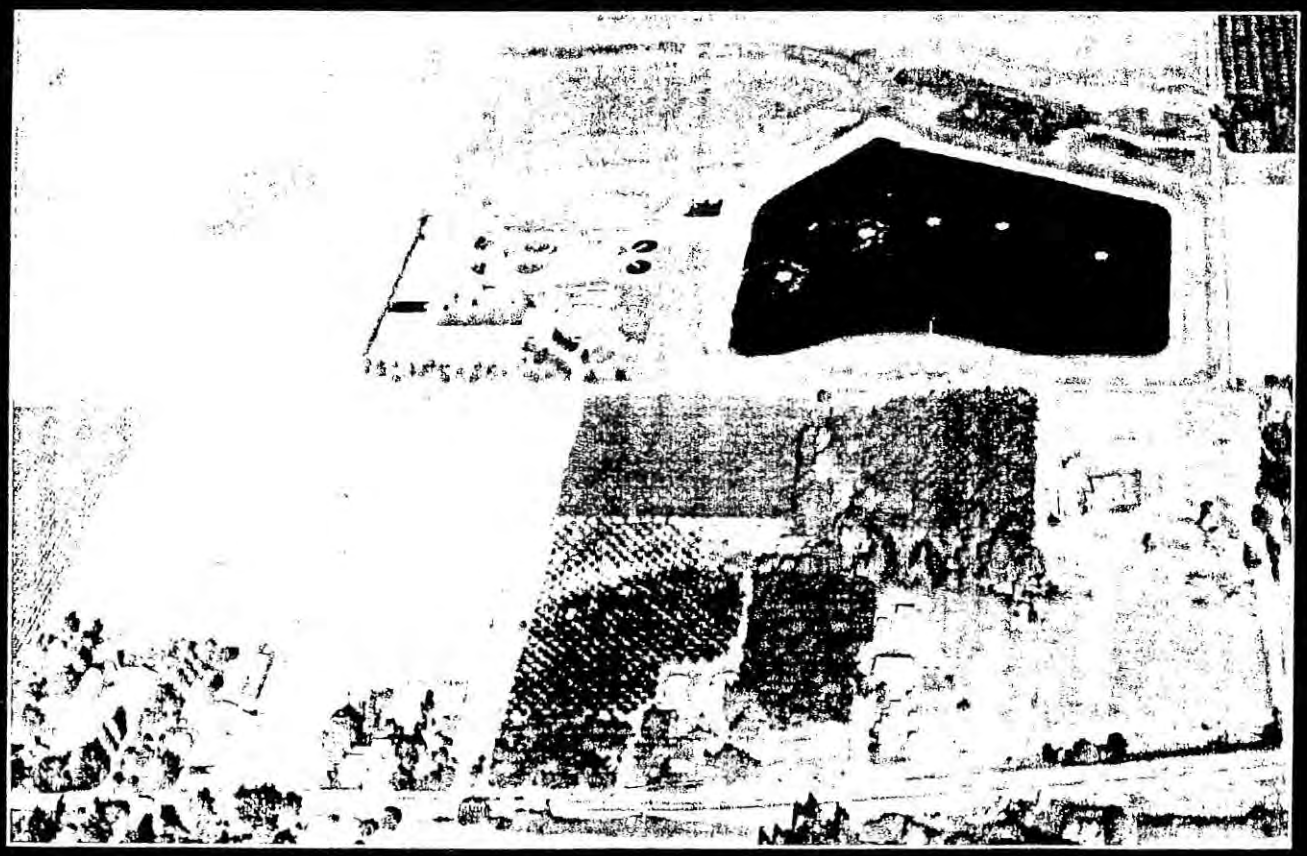
WASTEWATER FACILITIES PLAN

WOODBURN, OREGON
1889
CITY OF UNITY

Volume 1–Treatment



1995

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Executive Summary

In November 1993, the City of Woodburn (City) was notified by the U.S. Environmental Protection Agency (EPA) and the Oregon Department of Environmental Quality (DEQ) to develop a plan to meet the more stringent pollution limits that have been set for discharging effluent water from the sewage treatment plant into the Pudding River. The volume of water in the Pudding River during the summer (July and August) is so low that the river water cannot dilute the treatment plant effluent sufficiently to maintain the level of oxygen needed by certain aquatic life as well as it can during the winter months. This is the main reason that the Pudding River has been classified as a water-quality-limited stream.

The City has prepared a facilities plan for its wastewater treatment system. A facilities plan defines which steps the City should take to meet its future needs for wastewater treatment. It is the result of a planning process that looks at many possibilities, considers the advantages and disadvantages of each, and identifies the system that can most benefit the community. Once the plan is adopted by the City, it will guide operations and improvements to the City's treatment system through the year 2020. DEQ has set a deadline for submittal of this plan.

Improvements to the City's system are needed to meet new water quality regulations established by EPA and DEQ. These regulations designate wastewater treatment and discharge standards that must be met to protect the Pudding River and other bodies of water that receive treated wastewater. The City's current system is not able to meet these stringent new standards.

In addition to upgrading the existing system to meet regulatory requirements, the facilities plan will provide for increasing the system's capacity in order to accommodate planned growth in the City. Preparing now for both of these purposes is an efficient and cost-effective planning approach. Additional efficiency is built into the plan by providing for phased construction of the improvements. The plan will enable the City to look ahead to long-term needs through the year 2020, while implementing the improvements only as they are needed.

The City is committed to providing its citizens with a system that is environmentally responsible and cost-effective. Public review and comment is an essential part of the planning process.

A Wastewater Advisory Committee (WAC) of local citizens and community representatives was appointed by the governing body. The WAC evaluated several different alternatives on how to handle these problems and recommended one alternative over the others.

The City's public works staff has been working closely with DEQ, CH2M HILL, and the WAC in developing the plan. Several public workshops and an open house have been held. Another open house and public meeting are planned.

The planning process has the following basic steps:

- Define future wastewater treatment needs based on population, wastewater projections, and regulatory requirements.
- Identify alternative treatment systems that can meet these needs.
- Evaluate the alternatives on the basis of cost, environmental impacts, ability to construct and operate, and other advantages/disadvantages.
- Select the best treatment system for more detailed analysis, and develop a facilities plan.
- Conduct a financial analysis to determine who pays for the improvements, and how.

It is anticipated that the final facilities plan will be submitted to DEQ by July 18, 1995. Construction of the improvements to the existing wastewater treatment plant may begin in 1997. The first phase may be in operation by late 1999.

The wastewater treatment facilities will be required to meet a seasonal average effluent standard of 10 milligrams per liter (mg/L) of biochemical oxygen demand (BOD) and 10 mg/L of total suspended solids (TSS) at a design average dry weather flow of 5.0 million gallons per day (mgd) and provide ammonia removal. The estimated cost of these facilities is divided into two phases. Phase 1 estimated costs (in 1998 dollars) are $38.3 million; Phase 2 estimated costs (in 1998 dollars) are $11.9 million.

Organization of the Facilities Plan Report

The facilities plan is organized into three volumes. Volume I concerns wastewater treatment, Volume II contains the collection system evaluation, and Volume III contains the wastewater rate and system development charge study.

The findings of the facilities plan are summarized below.

Existing Conditions

The planning period is 1995 to 2020. The study area encompassed the area within the present urban growth boundary (UGB) of the City of Woodburn and several areas outside the boundary. The areas outside UGB were included in the study for public health reasons. Approval from the Department of Land use is needed before the areas are sewered. The Pudding River Regional Service Area was also considered in the evaluation of regional facilities.

The wastewater collection system conveys wastewater from residences, businesses, industries, and public facilities to the wastewater treatment plant. The hydraulic design capacity of the treatment plant is 3.14 mgd average dry weather flow, and 8.4 mgd peak hourly flow. The plant, however, has treated higher flows. The average total biochemical oxygen demand (BOD5) capacity is 3,350 lb/day BOD5. Currently, the plant has an average daily flow of 1.987 mgd and a wet weather peak hourly flow of 11.2 mgd. For BOD5, the plant has an average daily load of 3,562 lb/day and a maximum daily load of 10,575 lb/day. The plant has adequate hydraulic capacity. The BOD5 loads occasionally exceed the treatment capability of the plant as a result of a change in the BOD5 characteristics due to higher than anticipated commercial and industrial loads. At times, the rotating biological contactor (RBC) process does not effectively treat the current BOD5 load. The RBC units are also showing evidence of deterioration and will soon reach their expected service life.

Future Wastewater Flows and Loads

The City of Woodburn provided the population equivalent projections to the year 2020. The WAC and the City Council approved these projections:

- 3.4 percent growth for the summer residential population equivalent
- 3.4 percent growth for the summer commercial population equivalent
- 0.5 to 1.0 percent growth for the permitted population equivalent

The City projected the total summer population equivalent to grow from 29,085 (1993) to 61,430 (2020). The projected 2020 average dry weather flow is 5.037 mgd, and the design peak month BOD5 load is 14,781 lb/day. The City has planned for treatment of septage from the Pudding River Regional Service Area. The design flows and loads are summarized in Table ES-1.

Treatment Requirements

The Pudding River is a water-quality-limited stream. Total maximum daily loads are set for the Pudding River. Waste load allocations are set for the Woodburn WWTP. The treatment requirements are 10 mg/L BOD5 and 10 mg/L TSS. Effluent ammonia limits vary with Pudding River flow. Effluent to be irrigated must be disinfected and meet a turbidity limit of 2 NTUs.

Liquid Treatment and Effluent Disposal Alternatives

Liquid treatment alternatives were analyzed for preliminary wastewater treatment, secondary treatment, filtration, disinfection, and effluent disposal.

The preliminary treatment alternatives are:

- No action, leaving the existing headworks as it is

Table ES-1
Recommended Design Flows and Loads
Woodburn WWTP
2020

Flow Condition	*Flow (mgd)*
Annual	
Average Day	5.661
Dry Weather	
Average Day	5.037
Maximum Month	6.228
Maximum Day	11.426
Wet Weather	
Average Day	6.285
Maximum Month	8.772
Maximum Day	14.747
Peak	22.121

Load Condition	*BOD (lb/day)*	*TSS (lb/day)*
Total Annual	10,485	10,285
Peak Month	14,781	13,396
Worst Case Scenario	34,288	26,175

- New headworks, including additional hydraulic capacity, flow measurement, screenings removal, grit removal, and odor control

The secondary treatment alternatives include:

- No action, leaving the existing facilities as they are
- Expand and upgrade the existing RBCs
- Activated sludge with selector technology
- Sequencing batch reactor

The filtration alternatives are:

- No action, leaving the existing facilities as they are
- Expand the existing shallow filters
- Deep filters

The disinfection alternatives are:

- No action, leaving the existing facilities as they are
- Chlorination, including all compounds that produce free chlorine residuals
- Ultraviolet radiation

The effluent disposal alternatives are:

- Discharge to the Pudding River, the current method used at the existing plant
- Irrigation reuse during July and August, with discharge to the Pudding River the rest of the year

These alternatives were evaluated for cost and noncost factors such as flexibility and operational ease. The recommended plant configuration includes:

- New headworks
- Maintaining the existing primary treatment facilities
- Activated sludge with selector technology for secondary treatment and biological ammonia removal
- Deep bed filtration
- Ultraviolet light disinfection for Pudding River discharge and sodium hypochlorite for irrigation dosing
- Pudding River discharge with 2 months of effluent reuse on poplar trees

In addition to analyzing the treatment components available to meet water quality objectives, "Big Picture" alternatives were evaluated that considered treatment plant siting issues,

wetland tertiary treatment, Willamette River Disposal, crop irrigation, treating industrial loads separately, and Sidestream Elevated Pool Aeration. The consensus of the Wastewater Advisory Committee, the Council and the public was to recommend upgrading and expanding at the existing site and recommended plant configuration.

Solids Management Alternatives

Digested sludge (biosolids) is currently applied to agricultural land as a soil amendment. New sludge regulations require an increase in the digestion capacity and sludge storage capacity at the plant. Seven sludge processing and disposal/reuse alternatives were evaluated:

- Thickening
- Stabilization
- Dewatering
- Composting
- Incineration
- Heat drying
- Land application

These alternatives were evaluated for cost and noncost factors. The recommended alternative is for land application of liquid digested biosolids on the same site where reclaimed water is used to irrigate poplar trees. Storage lagoons will be provided to store solids during wet winter months when land application cannot occur.

Recommended Wastewater Management Program

The recommended wastewater management program for the City of Woodburn, as shown in Figures ES-1 and ES-2, includes the following facilities:

Liquid Treatment and Effluent Disposal

- New headworks with screening and vortex grit removal
- One additional primary clarifier
- Modified RBC basins to become phosphorus removal selector basins, if needed
- Three new activated sludge basins with anoxic selectors for ammonia removal
- New blower building
- Three new secondary clarifiers

- Return and waste sludge pump station
- Chemical addition
- New deep bed filters
- UV disinfection and sodium hypochlorite
- Effluent reaeration
- Plant water system
- Effluent flume
- New outfall
- Poplar tree reclaimed water irrigation system for July and August
- New odor control for headworks and solids building
- New administration building
- Modifications to the existing administration/laboratory building

Solids Management

- Waste-activated sludge thickening
- Anaerobic digestion
- Biosolids storage and land application

The estimated order-of-magnitude total project cost is $50.246 million in 1998 dollars (with an accuracy range of plus 50 percent to minus 30 percent). The cost for Phase 1 is $38.3 million, and the cost for Phase 2 is $11.9 million, both in 1998 dollars. A separate rate study in Volume III addresses financing issues.

The facilities plan approval process by DEQ includes:

- A dilution waiver for the Pudding River, with possible waivers required for mercury and silver
- Mass load increases for BOD_5 and TSS
- Grit and Secondary Clarification Redundancy and Reliability waiver
- Mixing zone ammonia limit resolution
- Permit drafted that will address new plant operation

- Determination of Phase I ammonia removal requirements
- Determination of how river flow monitoring data will determine effluent quality

The plant improvements would be made within the existing plant site, so no adverse environmental impacts are expected. Assuming regulatory agencies' approval by January 1996, the following schedule is anticipated, based on the schedule dictated by the SFO:

• Facilities plan approval	January 1996
• Design complete	July 1997
• Construction complete	September 1999

Environmental Assessment

No significant environmental impacts are expected. The improvements for the treatment facilities will be made at the existing plant site. An additional environmental assessment will be needed after identifying the beneficial reuse irrigation site.

Public Involvement

Public involvement in the facilities planning process has been continuous since August 1993, when the Woodburn WAC was started. The WAC has provided input in the facilities planning process. An open house was held with the public on the alternatives. Another open house is planned to present the facilities plan to the public. A public hearing is scheduled to be held before the final plan is adopted by the Woodburn City Council.

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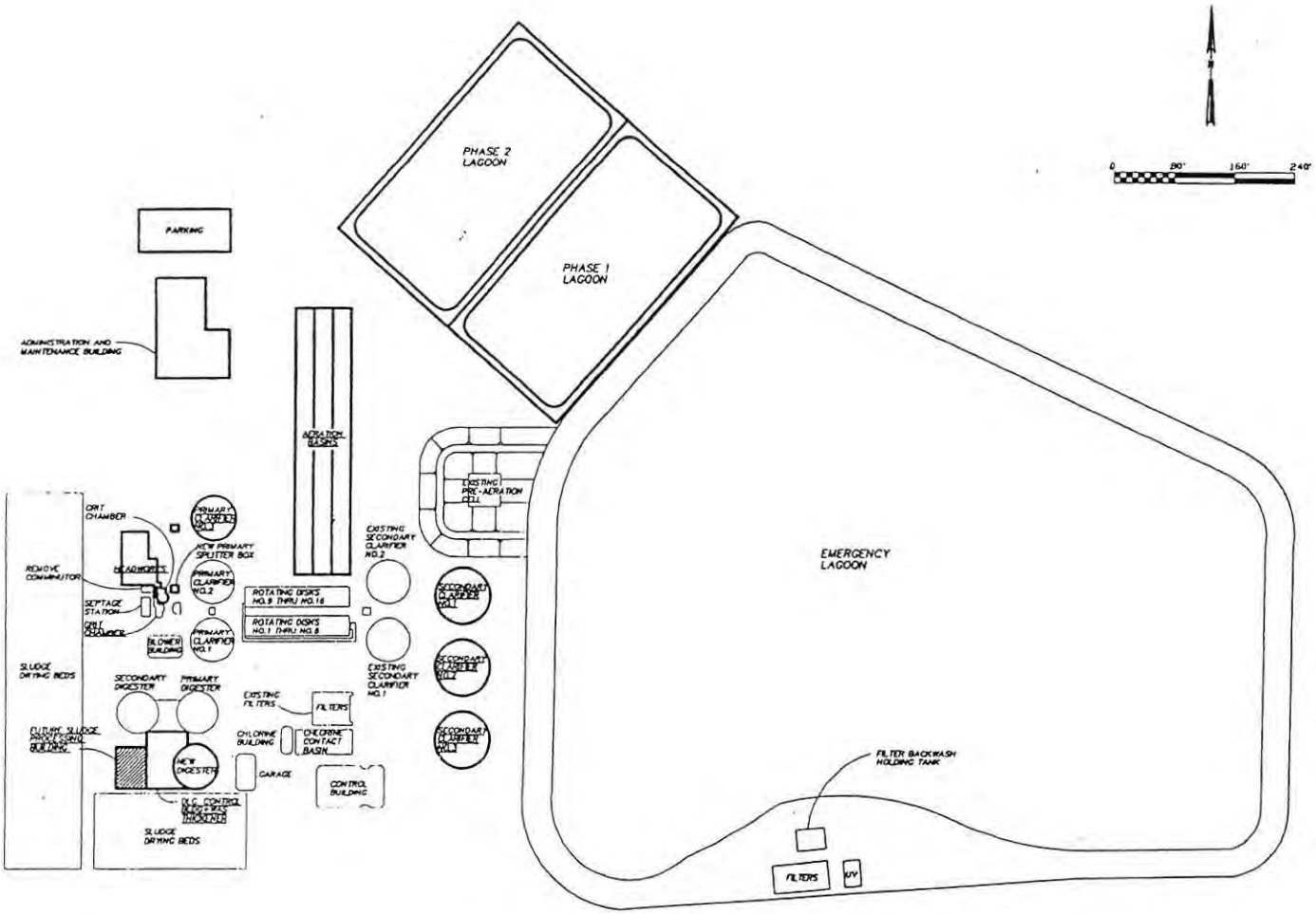

EXISTING WWTP SITE CONCEPTUAL LAYOUT

Figure ES-1
CONCEPTUAL LAYOUT OF RECOMMENDED WOODBURN WWTP FACILITIES

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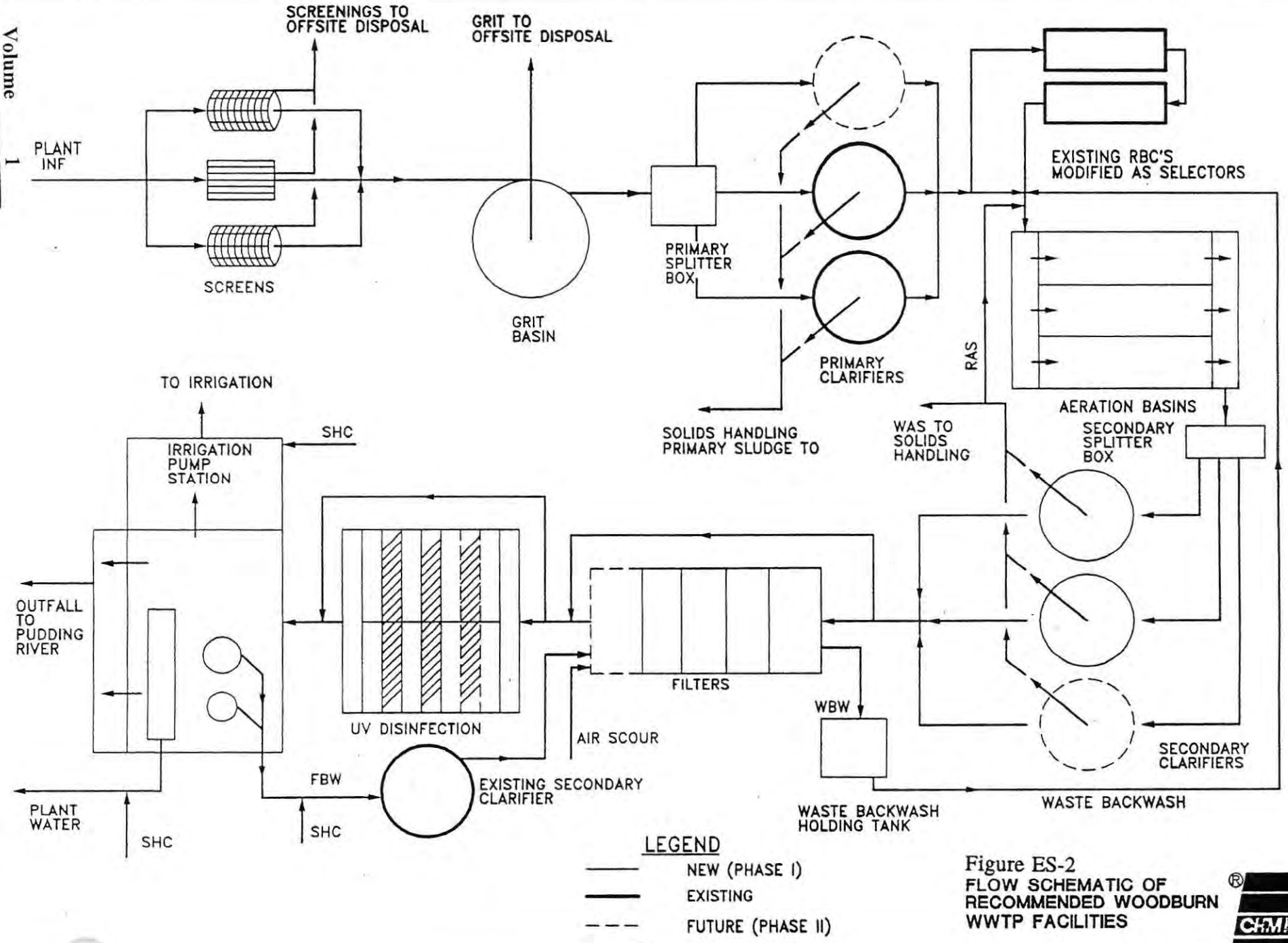

Figure ES-2
FLOW SCHEMATIC OF RECOMMENDED WOODBURN WWTP FACILITIES

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WASTEWATER FACILITIES PLAN

Volume I–Treatment

February 1995

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Executive Summary

In November 1993, the City of Woodburn (City) was notified by the U.S. Environmental Protection Agency (EPA) and the Oregon Department of Environmental Quality (DEQ) to develop a plan to meet the more stringent pollution limits that have been set for discharging effluent water from the sewage treatment plant into the Pudding River. The volume of water in the Pudding River during the summer (July and August) is so low that the river water cannot dilute the treatment plant effluent sufficiently to maintain the level of oxygen needed by certain aquatic life as well as it can during the winter months. This is the main reason that the Pudding River has been classified as a water-quality-limited stream.

The City has prepared a facilities plan for its wastewater treatment system. A facilities plan defines which steps the City should take to meet its future needs for wastewater treatment. It is the result of a planning process that looks at many possibilities, considers the advantages and disadvantages of each, and identifies the system that can most benefit the community. Once the plan is adopted by the City, it will guide operations and improvements to the City's treatment system through the year 2020. DEQ has set a deadline for submittal of this plan.

Improvements to the City's system are needed to meet new water quality regulations established by EPA and DEQ. These regulations designate wastewater treatment and discharge standards that must be met to protect the Pudding River and other bodies of water that receive treated wastewater. The City's current system is not able to meet these stringent new standards.

In addition to upgrading the existing system to meet regulatory requirements, the facilities plan will provide for increasing the system's capacity in order to accommodate planned growth in the City. Preparing now for both of these purposes is an efficient and cost-effective planning approach. Additional efficiency is built into the plan by providing for phased construction of the improvements. The plan will enable the City to look ahead to long-term needs through the year 2020, while implementing the improvements only as they are needed.

The City is committed to providing its citizens with a system that is environmentally responsible and cost-effective. Public review and comment is an essential part of the planning process.

A Wastewater Advisory Committee (WAC) of local citizens and community representatives was appointed by the governing body. The WAC evaluated several different alternatives on how to handle these problems and recommended one alternative over the others.

The City's public works staff has been working closely with DEQ, CH2M HILL, and the WAC in developing the plan. Several public workshops and an open house have been held. Another open house and public meeting are planned.

The planning process has the following basic steps:

- Define future wastewater treatment needs based on population, wastewater projections, and regulatory requirements.
- Identify alternative treatment systems that can meet these needs.
- Evaluate the alternatives on the basis of cost, environmental impacts, ability to construct and operate, and other advantages/disadvantages.
- Select the best treatment system for more detailed analysis, and develop a facilities plan.
- Conduct a financial analysis to determine who pays for the improvements, and how.

It is anticipated that the final facilities plan will be submitted to DEQ by July 18, 1995. Construction of the improvements to the existing wastewater treatment plant may begin in 1997. The first phase may be in operation by late 1999.

The wastewater treatment facilities will be required to meet a seasonal average effluent standard of 10 milligrams per liter (mg/L) of biochemical oxygen demand (BOD) and 10 mg/L of total suspended solids (TSS) at a design average dry weather flow of 5.0 million gallons per day (mgd) and provide ammonia removal. The estimated cost of these facilities is divided into two phases. Phase 1 estimated costs (in 1998 dollars) are $38.3 million; Phase 2 estimated costs (in 1998 dollars) are $11.9 million.

Organization of the Facilities Plan Report

The facilities plan is organized into three volumes. Volume I concerns wastewater treatment, Volume II contains the collection system evaluation, and Volume III contains the wastewater rate and system development charge study.

The findings of the facilities plan are summarized below.

Existing Conditions

The planning period is 1995 to 2020. The study area encompassed the area within the present urban growth boundary of the City of Woodburn and several areas outside the boundary. The Pudding River Regional Service Area was also considered in the evaluation of regional facilities.

The wastewater collection system conveys wastewater from residences, businesses, industries, and public facilities to the wastewater treatment plant. The hydraulic design capacity of the treatment plant is 3.14 mgd average dry weather flow, and 8.4 mgd peak hourly flow. The plant, however, has treated higher flows. The average total biochemical oxygen demand (BOD_5) capacity is 3,350 lb/day BOD_5. Currently, the plant has an average daily flow of 1.987 mgd and a wet weather peak hourly flow of 11.2 mgd. For BOD_5, the plant has an average daily load of 3,562 lb/day and a maximum daily load of 10,575 lb/day. The plant has adequate hydraulic capacity. The BOD_5 loads occasionally exceed the treatment capability of the plant as a result of a change in the BOD_5 characteristics due to higher than anticipated commercial and industrial loads. At times, the rotating biological contactor (RBC) process does not effectively treat the current BOD_5 load. The RBC units are also showing evidence of deterioration and will soon reach their expected service life.

Future Wastewater Flows and Loads

The City of Woodburn provided the population equivalent projections to the year 2020. The WAC and the City Council approved these projections:

- 3.4 percent growth for the summer residential population equivalent
- 3.4 percent growth for the summer commercial population equivalent
- 0.5 to 1.0 percent growth for the permitted population equivalent

The City projected the total summer population equivalent to grow from 29,085 (1993) to 61,430 (2020). The projected 2020 average dry weather flow is 5.037 mgd, and the design peak month BOD_5 load is 14,781 lb/day. The City has planned for treatment of septage from the Pudding River Regional Service Area. The design flows and loads are summarized in Table ES-1.

Treatment Requirements

The Pudding River is a water-quality-limited stream. Total maximum daily loads are set for the Pudding River. Waste load allocations are set for the Woodburn WWTP. The treatment requirements are 10 mg/L BOD_5 and 10 mg/L TSS. Effluent ammonia limits vary with Pudding River flow. Effluent to be irrigated must be disinfected and meet a turbidity limit of 2 NTUs.

Liquid Treatment and Effluent Disposal Alternatives

Liquid treatment alternatives were analyzed for preliminary wastewater treatment, secondary treatment, filtration, disinfection, and effluent disposal.

Table ES-1
Recommended Design Flows and Loads
Woodburn WWTP
2020

Flow Condition	*Flow (mgd)*
Annual	
Average Day	5.661
Dry Weather	
Average Day	5.037
Maximum Month	6.228
Maximum Day	11.426
Wet Weather	
Average Day	6.285
Maximum Month	8.772
Maximum Day	14.747
Peak	22.121

Load Condition	*BOD (lb/day)*	*TSS (lb/day)*
Total Annual	10,485	10,285
Peak Month	14,781	13,396
Worst Case Scenario	34,288	26,175

The preliminary treatment alternatives are:

- No action, leaving the existing headworks as it is
- New headworks, including additional hydraulic capacity, flow measurement, screenings removal, grit removal, and odor control

The secondary treatment alternatives include:

- No action, leaving the existing facilities as they are
- Expand and upgrade the existing RBCs
- Activated sludge with selector technology
- Sequencing batch reactor

The filtration alternatives are:

- No action, leaving the existing facilities as they are
- Expand the existing shallow filters
- Deep filters

The disinfection alternatives are:

- No action, leaving the existing facilities as they are
- Chlorination, including all compounds that produce free chlorine residuals
- Ultraviolet radiation

The effluent disposal alternatives are:

- Discharge to the Pudding River, the current method used at the existing plant
- Irrigation reuse during July and August, with discharge to the Pudding River the rest of the year

These alternatives were evaluated for cost and noncost factors such as flexibility and operational ease. The recommended plant configuration includes:

- New headworks
- Maintaining the existing primary treatment facilities
- Activated sludge with selector technology for secondary treatment and biological ammonia removal
- Deep bed filtration
- Ultraviolet light disinfection for Pudding River discharge and sodium hypochlorite for irrigation dosing

- Pudding River discharge with 2 months of effluent reuse on poplar trees

In addition to analyzing the treatment components available to meet water quality objectives, "Big Picture" alternatives were evaluated that considered treatment plant siting issues, wetland tertiary treatment, Willamette River Disposal, crop irrigation, treating industrial loads separately, and Sidestream Elevated Pool Aeration. The consensus of the Wastewater Advisory Committee, the Council and the public was to recommend upgrading and expanding at the existing site and recommended plant configuration.

Solids Management Alternatives

Digested sludge (biosolids) is currently applied to agricultural land as a soil amendment. New sludge regulations require an increase in the digestion capacity and sludge storage capacity at the plant. Seven sludge processing and disposal/reuse alternatives were evaluated:

- Thickening
- Stabilization
- Dewatering
- Composting
- Incineration
- Heat drying
- Land application

These alternatives were evaluated for cost and noncost factors. The recommended alternative is for land application of liquid digested biosolids on the same site where reclaimed water is used to irrigate poplar trees. Storage lagoons will be provided to store solids during wet winter months when land application cannot occur.

Recommended Wastewater Management Program

The recommended wastewater management program for the City of Woodburn includes the following facilities:

Liquid Treatment and Effluent Disposal

- New headworks with screening and vortex grit removal
- One additional primary clarifier
- Modified RBC basins to become phosphorus removal selector basins, if needed
- Three new activated sludge basins with anoxic selectors for ammonia removal
- New blower building
- Three new secondary clarifiers
- Return and waste sludge pump station
- Chemical addition
- New deep bed filters

- UV disinfection and sodium hypochlorite
- Effluent reaeration
- Plant water system
- Effluent flume
- New outfall
- Poplar tree reclaimed water irrigation system for July and August
- New odor control for headworks and solids building
- New administration building
- Modifications to the existing administration/laboratory building

Solids Management

- Waste-activated sludge thickening
- Anaerobic digestion
- Biosolids storage and land application

The estimated order-of-magnitude total project cost is $50.246 million in 1998 dollars (with an accuracy range of plus 50 percent to minus 30 percent). The cost for Phase 1 is $38.3 million, and the cost for Phase 2 is $11.9 million, both in 1998 dollars. A separate rate study in Volume III addresses financing issues.

The facilities plan approval process by DEQ includes:

- A dilution waiver for the Pudding River, with possible waivers required for mercury and silver
- Mass load increases for BOD_5 and TSS
- Grit and Secondary Clarification Redundancy and Reliability waiver
- Mixing zone ammonia limit resolution
- Permit drafted that will address new plant operation

The plant improvements would be made within the existing plant site, so no adverse environmental impacts are expected. Assuming regulatory agencies' approval by January 1996, the following schedule is anticipated, based on the schedule dictated by the SFO:

• Facilities plan approval	January 1996
• Design complete	July 1997
• Construction complete	September 1999

Bond election

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Environmental Assessment

No significant environmental impacts are expected. The improvements for the treatment facilities will be made at the existing plant site. An additional environmental assessment will be needed after identifying the beneficial reuse irrigation site.

Public Involvement

Public involvement in the facilities planning process has been continuous since August 1993, when the Woodburn WAC was started. The WAC has provided input in the facilities planning process. An open house was held with the public on the alternatives. Another open house is planned to present the facilities plan to the public. A public hearing is scheduled to be held before the final plan is adopted by the Woodburn City Council.

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Section 1
Introduction

The City of Woodburn prepared this facilities plan in order to define the wastewater system improvements that are needed to serve the area. The plan addresses the following important issues:

- New water quality standards for the Pudding River; for example, the Oregon Department of Environmental Quality (DEQ) issued final total maximum daily loads (TMDLs) for the Pudding River that require ammonia removal, and a discharge permit with chlorine residual effluent limits that the existing plant cannot meet
- New federal sludge processing regulations
- DEQ's Stipulation and Final Order (SFO) that defines a schedule for plant improvements and provides interim discharge limits until the plant improvements are operational
- Capacity to serve future planned growth
- Changes in the characteristics of influent sewage since the treatment plant was designed
- Some of the existing equipment has reached its expected service life

Organization of the Facilities Plan Report

The facilities plan is organized into three volumes. Volume I, which concerns wastewater treatment, is organized as follows:

- Existing Conditions
- Future Wastewater Flows and Loads
- Treatment Requirements (analysis of water quality and regulatory standards)
- Liquid Treatment and Effluent Disposal Alternatives
- Solids Management Alternatives
- Recommended Wastewater Management Alternatives
- Environmental Assessment
- Public Involvement

Volume II contains the collection system evaluation, and Volume III contains the wastewater rate and system development charge study.

The findings of the treatment plan are summarized below. See Appendix D, References, for a list of reports that were used in the preparation of this plan.

Existing Conditions

The planning period is 1995 to 2020. The study area encompassed the area within the present urban growth boundary of the City of Woodburn and several areas outside the boundary. The Pudding River Regional Service Area was also considered in the evaluation of regional facilities.

The wastewater collection system conveys wastewater from residences, businesses, industries, and public facilities to the wastewater treatment plant. The design capacity of the treatment plant is 3.14 mgd average dry weather flow, and 8.4 mgd peak hourly flow. The plant, however, has treated higher flows. The average total biochemical oxygen demand (BOD_5) capacity is 3,350 lb/day BOD_5. Currently, the plant has an average daily flow of 1.987 mgd and a wet weather peak hourly flow of 11.2 mgd. For BOD_5, the plant has an average daily load of 3,562 lb/day and a maximum day load of 10,575 lb/day. The plant has adequate hydraulic capacity. The BOD_5 loads occasionally exceed the treatment capability of the plant as a result of a change in the BOD_5 characteristics due to higher than anticipated commercial and industrial loads. At times, the rotating biological contactor (RBC) process does not effectively treat the current BOD_5 load. The RBC units are also showing evidence of deterioration and will soon reach their expected service life.

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The City projected the total summer population equivalent to grow from 29,085 (1993) to 61,430 (2020). The projected 2020 average dry weather flow is 5.037 mgd, and the design peak month BOD_5 load is 14,781 lb/day. The City has planned for treatment of septage from the Pudding River Regional Service Area.

Treatment Requirements

Regulations for effluent discharge to the Pudding River, effluent reuse, sludge processing, and plant reliability/redundancy requirements are summarized in Section 4. The Pudding River is a water quality limited stream. Total maximum daily loads are set for the Pudding River. Waste load allocations are set for the Woodburn WWTP. Reuse regulations governing irrigation of reclaimed water are summarized in Section 4.

Liquid Treatment and Effluent Disposal Alternatives

Liquid treatment alternatives were analyzed for preliminary wastewater treatment, secondary treatment, filtration, disinfection, and effluent disposal.

The preliminary treatment alternatives are:

- No action, leaving the existing headworks as it is
- New headworks, including additional hydraulic capacity, flow measurement, screenings removal, grit removal, and odor control

The secondary treatment alternatives include:

- No action, leaving the existing facilities as they are
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- Activated sludge with selector technology
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- Irrigation reuse during July and August, with discharge to the Pudding River the rest of the year

These alternatives were evaluated for cost and noncost factors such as flexibility and operational ease. The recommended plant configuration includes:

- New headworks
- Maintaining the existing primary treatment facilities
- Activated sludge with selector technology for secondary treatment
- Deep filtration
- Ultraviolet radiation and sodium hypochlorite
- Pudding River discharge with 2 months of effluent reuse on poplar trees

Solids Management Alternatives

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- Land application

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- New blower building
- Three new secondary clarifiers
- Return and waste sludge pump station
- Chemical addition
- New deep bed filters
- UV disinfection and sodium hypochlorite
- Effluent reaeration
- Plant water system
- Effluent flume
- New outfall
- Poplar tree reclaimed water irrigation system for July and August
- New odor control for headworks and solids building
- New administration building

- Modifications to the existing administration/lab building

Solids Management

- Waste activated sludge thickening
- Anaerobic digestion
- Biosolids storage and land application

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Public Involvement

Public involvement in the facilities planning process has been continuous since August 1993, when the Woodburn Wastewater Advisory Committee (WAC) was started. The WAC has provided input in the facilities planning process. An open house was held with the public on the alternatives. Another open house is planned to present the facilities plan to the public. A public hearing is scheduled to be held before the final plan is adopted by the Woodburn City Council.

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Se
Existing

Intro

This section discusses the characteristics of
facilities and operations, existing wastewat
toring study results, and the deficiencies of th

Study Area (

Study Area Definition

The City of Woodburn is situated on the Pu
City, incorporated in 1889, is located 30 m
Salem along the I-5 Interstate Freeway.

The Woodburn Wastewater Treatment Plant
Woodburn urban growth boundary (UGB).
that are served by the Woodburn WWTP or
include the mobile home park north of M
MacLaren School, an RV park (located at
apartment complex (located on Carl Road),
Interstate Freeway.

It is assumed that the UGB area will remain c
2020. It is also assumed that the existing an
collection and treatment systems is equal to th
system. Figure 2-1 is a map that shows the cu

Regional Service Provider Study Area

The Regional Service Provider Study Area
Merseth Engineers. The Regional Service Pro
regional handling of septage, sludge, and labor

Topography

The City of Woodburn is located in the Pu
Oregon. The City lies at the north end of
northeastern corner of the French Prairie, whi
elevation from 170 to 200 feet above sea leve

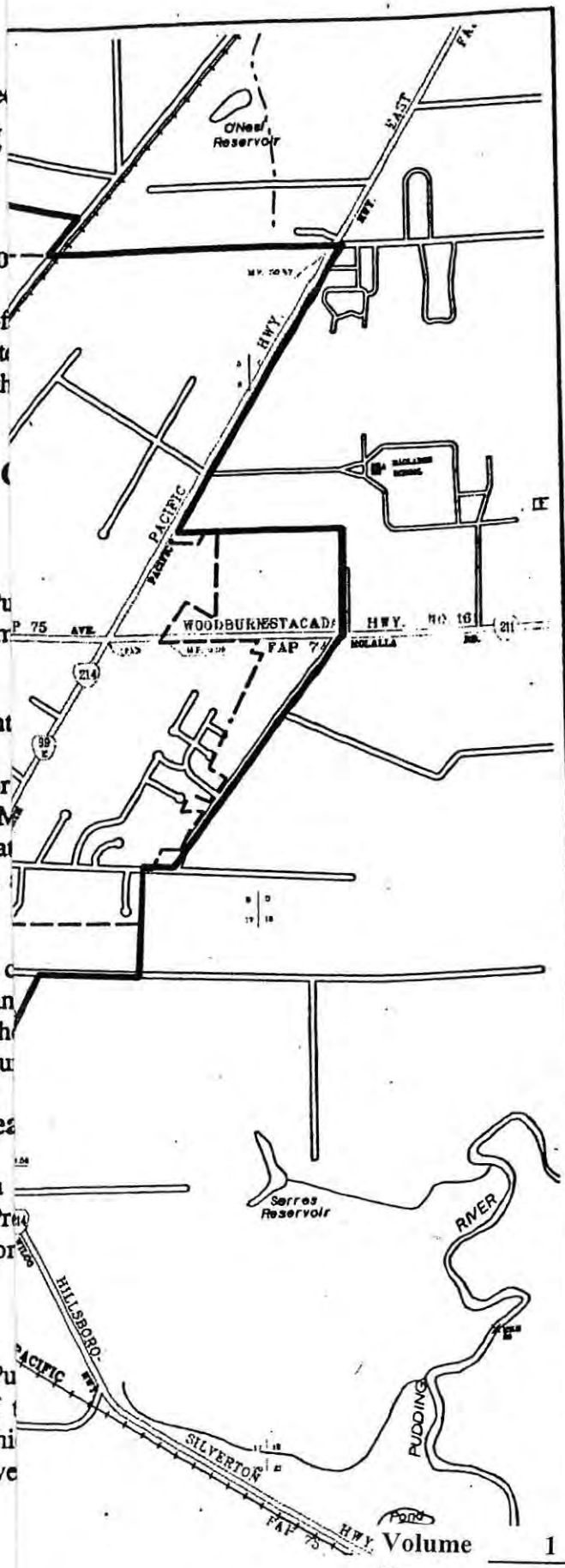

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FIGURE 2-1
URBAN GROWTH
BOUNDARY FOR
CITY OF WOODBURN

CH2M HILL

Figure 2-1
UGB - ~~In graphics/L. Hunt~~
(11 x 17)

Figure 2-1

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narrow ravines with deeply cut streams. The study area drains generally north-northeast through Mill Creek and the Pudding River.

For the most part, the City of Woodburn is bounded by the Pudding River east of the City and the I-5 Interstate Freeway and Northern Pacific Railroad to the west of the City. Mill Creek bisects the City as it flows to the northeast.

Geology

Because the site of the treatment plant has not changed, the following geological information is repeated from the *City of Woodburn, Oregon, Wastewater Facilities Plan*, October 1976, prepared by CH2M HILL.

Troutdale formation materials and Willamette silts overlie Columbia River basalt in the Woodburn area. Depth to the basalt is unknown, but is estimated to be about 600 feet.

The Troutdale formation consists of alternate layers of clay, silt, sand, and gravel. These materials were deposited by streams entering the valley from the south and east.

The Willamette silt formation consists of stratified silt, sandy silt, clayey silt and silty clay, and frequently has poor drainage capabilities. Ground water is occasionally found at or very near the surface in these areas and can cause construction problems and septic tank malfunctions.

Soils

Because the site of the treatment plant and has not changed, the following soils information is repeated from the *City of Woodburn, Oregon, Wastewater Facilities Plan*, October 1976, prepared by CH2M HILL.

Woodburn silt loams and Amity silt loams are the two major soil types in the study area. The Woodburn series consists of moderately well-drained soils formed in silty alluvium and loess. The surface layer is typically about 17 inches thick and dark brown. The subsoil is 37 inches thick and a dark yellowish brown. The substratum is dark-brown silt loam which extends to a depth of 68-inches or more. These soils are suitable for small grains, pasture, hay, orchards, berries, and vegetables.

The Amity series consists of poorly drained soils formed in mixed alluvial silts. These soils occur on the broad valley terraces and the surface layer is generally about 17 inches thick. The subsurface layer is dark gray silt loam about 7 inches thick. The subsoil is silty clay loam about 13-inches thick. These soils are utilized primarily for cereal grains, grass grown for seed, and pasture. Irrigated areas can support other crops.

The course of Mill Creek is etched in Bashaw clay, consisting of poorly drained soils formed in alluvium. The surface layer in a typical profile is 31-inches thick and consists of mottled dark gray and black clay. The Bashaw soils are used mainly for pasture, but support perennial grasses, wild blackberries, sedges, rushes, willows, and some ash and oak trees.

Dayton soils and terrace escarpments are also found in a few isolated areas throughout the study area.

Surface Waters

Pudding River

The Pudding River is the main surface water feature in the Woodburn area. The 62-mile-long river originates in the Waldo Hills and flows sluggishly in a northerly direction in a meandering channel with little slope. It has numerous tributaries. The upper 7 miles are typical of foothill drainage, and the lower 56-miles are typical of flat valley drainage. The Pudding River empties into the Molalla River, which flows into the Willamette River at river mile 36 near Wilsonville.

The drainage area for the Pudding River is 480 square miles. Average monthly flows for the Pudding River range from 63 cubic feet per second (cfs) in the summer to about 2,600 cfs in the winter. Streamflow responds to both rainfall and snowmelt, with snowmelt maintaining high flows into late spring. The river occasionally floods severely in winter and spring months. There is a marked decline in streamflow during the drier summer months which impacts (among other things) the assimilation of waste from waste treatment sources.

The entire length of the Pudding River is zoned for Exclusive Farm Use and its land-use designation is Primary Agriculture. The Marion County Comprehensive Plan designates the Pudding River as warm water habitat. Many species of fish use the Pudding River year-round, as well as seasonal salmon and cutthroat trout runs.

The Pudding River is the receiving water for the Woodburn WWTP. WWTP effluent discharges to the river through a 24-inch outfall. The National Pollutant Discharge Elimination System (NPDES) permit stipulates the allowable mixing zone shall not extend beyond 270 feet downstream from the point of discharge.

Mill Creek

Mill Creek is the major drainage course through Woodburn; it bisects the City. The creek flows 8 miles to the northeast and empties into the Pudding River just south of Aurora. Mill Creek is generally confined to the area between Highway 99E and the I-5 Interstate Freeway. Mill Creek is deeply incised in an otherwise generally flat plain.

Senecal Creek

Senecal Creek drains a small portion of the study area west of the I-5 Interstate Freeway. Senecal Creek discharges to Mill Creek near Aurora. It is poorly defined in some areas and clogged by brush.

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Climate

The City of Woodburn is located at the north end of the Willamette Valley (east of the Coast Range) and thus enjoys a drier, more continental climate than the coastal areas. The area has a mild and temperate climate with a dry summer season and a rainy winter. Westerly winds generally pick up moisture from the Pacific Ocean. As a result of the orographic effect of the Coast Range, precipitation decreases as the winds flow eastward into the Willamette Valley. On the east side of the range, the amount of rainfall decreases sharply on the lower slopes and on the valley floors.

Historical climatic data for the Woodburn area are summarized in Table 2-1. The data in this table are from the closest National Weather Service reporting station, located at Salem.

Although summer days can be consistently sunny, hot weather that is continuous and prolonged is rare and nights are generally cool. Similarly, continuous and prolonged subfreezing weather is rare during the winter; snowfall is usually light. Approximately 77 percent of the mean annual precipitation falls during the wet season, November through April (based on data from Table 2-1).

Demographics

In almost 100 years, the City of Woodburn population has increased from 405 to 14,055 residents (Table 2-2). Beginning in 1890 a population census was performed every 10 years. Annual population censuses began in 1956. According to this information, which was obtained from the Portland State University Population Research and Census Center, the City of Woodburn's population has increased every year from 1956 to 1993, except for two years: 1978 and 1982. The population trend in Woodburn and nearby areas has been one of continual growth with periods of relatively large increases. There was a growth spurt during the early- and mid- 1960s, with double-digit annual increases for many of the years. Population growth slowed during the late 1970s through the late 1980s; during this period the only two years of negative population growth in the City's history occurred.

While the City of Woodburn has maintained continuous growth, the age distribution of the population has changed (see Table 2-3 and Figure 2-2). As seen by a comparison of population age groups in 1970, 1980, and 1990, the number and percent of 25- to 44-year olds has increased. The 0-to 5-year old group also increased both in number and percent of the total population. At the same time, decreases occurred in the 45-to 64-year old and the 65 and over categories, as a percent of the total population. Even so, the 65 and over group is still the largest.

Growth in Woodburn is projected to continue on the basis of a diversifying local economy, continued growth in the agricultural processing industry, attractiveness as a place to live, and proximity to the Salem and Portland metropolitan areas.

Table 2-1
Historical Climatic Data
Salem, Oregon
1961-1990

Month	*Average Maximum Temperature (deg F)*	*Average Minimum Temperature (deg F)*	*Average Mean Temperature (deg F)*	*Mean Precipitation (inches)*
January	46.4	32.7	39.6	5.92
February	51.5	34.1	42.8	4.50
March	55.7	35.6	45.7	4.17
April	60.4	37.7	49.1	2.42
May	67.0	42.2	54.6	1.88
June	74.5	48.3	61.4	1.34
July	81.6	50.9	66.3	0.56
August	82.0	51.4	66.7	0.76
September	76.0	47.1	61.6	1.55
October	64.2	41.1	52.7	2.98
November	52.4	37.5	45.0	6.28
December	46.4	33.6	40.0	6.80
Annual Average	63.2	41.0	52.1	39.16

Note: Dry weather season data are shaded.

Table 2-2
Historical Population, Woodburn, Oregon
1890-1993

Year	*Population*	*Annual Percent Increase*
1890	405	---
1900	828	---
1910	1,616	---
1920	1,656	---
1930	1,675	---
1940	1,982	---
1950	2,395	---
1956	2,700	---
1957	2,750	2%
1958	2,810	2%
1959	2,825	1%
1960	3,105	10%
1961	3,184	3%
1962	3,527	11%
1963	3,966	12%
1964	4,439	12%
1965	4,800	8%
1966	5,809	21%
1967	6,200	7%
1968	6,550	6%
1969	6,850	5%
1970	7,495	9%
1971	7,810	4%

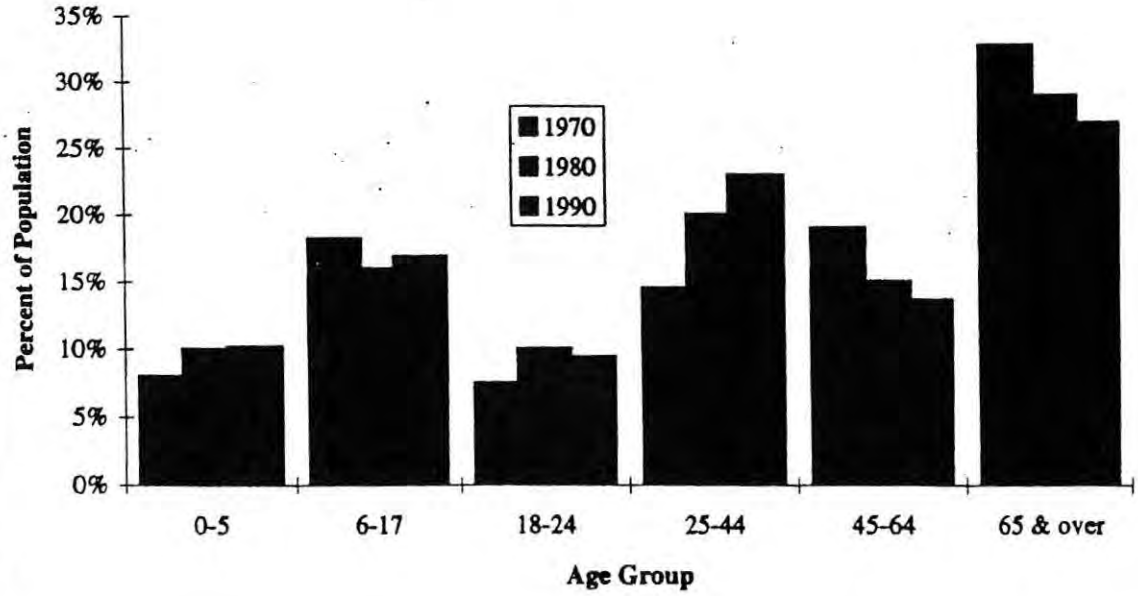
Year	*Population*	*Annual Percent Increase*
1972	8,260	6%
1973	9,015	9%
1974	9,560	6%
1975	9,575	0%
1976	10,200	7%
1977	10,500	3%
1978	10,000	-5%
1979	10,625	6%
1980	11,196	5%
1981	11,390	2%
1982	11,230	-1%
1983	11,270	0%
1984	11,300	0%
1985	11,700	4%
1986	11,785	1%
1987	11,990	2%
1988	12,080	1%
1989	12,445	3%
1990	13,404	8%
1991	13,525	1%
1992	14,005	4%
1993	14,055	0%

Table 2-3
Population Distribution by Age Group
Woodburn, Oregon
1970-1990

Age Group	1970 Population	1970 Percent of Population	1980 Population	1980 Percent of Population	1990 Population (a)	1990 Percent of Population
0-5	600	8%	1,120	10%	1,364	10%
6-17	1,364	18%	1,791	16%	2,260	17%
18-24	562	7%	1,120	10%	1,261	9%
25-44	1,087	15%	2,239	20%	3,077	23%
45-64	1,424	19%	1,679	15%	1,828	14%
65 & over	2,458	33%	3,247	29%	3,614	27%
Total	7,495	100%	11,196	100%	13,404	100%

Note:
(a) The populations for the first three age groups for 1990 are estimated because the compositions of the age groups were changed for that year.

Figure 2-2
Population Distribution by Age Group
Woodburn, Oregon
1970-1990



Land Use

Like other Oregon cities, the City of Woodburn maintains an Urban Growth Program. According to the City's Comprehensive Plan, expansion in Woodburn in an orderly and efficient manner will aid in giving the population the commerce and industry it has always historically desired. The program provides a guide for expansion and sets limits within a reasonable planning period. Decisions about allocation of public resources are the principal determinants for where and when development takes place.

Table 2-4 and Figure 2-1 show the UGB and the areas within the UGB that are zoned as residential, commercial, and industrial. The total area encompassed by the UGB is 2,855 acres (this includes 318 acres outside of the current city limits). Approximately 53 percent (1,506 acres) of the total UGB acreage is zoned low or high density residential. Seventeen percent (489 acres) of the UGB is zoned commercial, and 24 percent (690 acres) is zoned industrial. There are 170 acres of miscellaneous and public use lands within the UGB (6 percent). All of these figures are from the February 1989 *Woodburn Comprehensive Plan* and represent conditions in 1988.

There are 595 acres vacant and available within the UGB. Over one half of this land is zoned for low or high density housing (56 percent or 334 acres). The remaining vacant and available land is evenly split between commercial (133 acres) and industrial (128 acres) zoning.

With the expected continued growth of Woodburn, land will continue to be developed within the UGB in accordance with zoning requirements. Detailed data on zoning and development can be found in the *Woodburn Comprehensive Plan, Land Use Element*, December 1978 (amended March 1981 and February 1989).

Existing Wastewater Treatment Facility

Design Criteria

The existing Woodburn WWTP is located in the northeast part of Woodburn and is adjacent to the Pudding River, just east of Highway 99 and MacLaren School. The treatment plant site is on MacLaren School property and is leased from the State of Oregon. An aerial view of the WWTP is shown in Figure 2-3. The existing plant was built at the site of two large lagoons. One still exists as an emergency lagoon.

Design criteria for the existing plant and design factors for unit treatment processes and equipment are shown in Table 2-5.

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Table 2-4
Land Use Inventory
Woodburn, Oregon
1988

Land Use Category	Land Within the City Limits of Woodburn and the UGB					Land Outside the City Limits Within the UGB (acres)	UGB Total (acres)	Percent of UGB
	Land in Use (acres)	Vacant & Available (acres)	Vacant & Not Available (acres)	Total (acres)	Percent Vacant & Available			
Low density residential	950	235	60	1,245	19%	0	1,245	44%
High density residential	153	99	9	261	38%	0	261	9%
Commercial	211	133	0	344	39%	145	489	17%
Industrial	202	128	187	517	25%	173	690	24%
Miscellaneous	170	0	0	170	0%	0	170	6%
Total	1,686	595	256	2,537	23%	318	2,855	100%

Table 2-5

Design Criteria and Design Factors
Existing Woodburn WWTP

Design Parameter	*Value*	
Design Flows (mgd)	Wet	Dry
Average Daily	3.4	3.1
Diurnal Peak	5.1	4.7
Sustained Peak	6.8	6.1
Peak Storm	8.4	8.4
Design Loads (mg/L)		
Influent BOD_5	195	215
Influent Soluble BOD_5	60	70
Influent Suspended Solids	170	185
Design Effluent Requirements (mg/L)		
Maximum Monthly Average TSS	30	10
Maximum Monthly Average BOD_5	30	10
Headworks		
Comminutors		
Type	Oscillating	
Number	2	
In Use	1	
Standby	1	
Capacity	8.4 mgd	
Grit Chamber		
Type	Rotary induced vortex	
Capacity	8.4 mgd	
Design Grit Removal	150 mesh	
Grit Pump Capacity	75 gpm at 75 cfm	
Primary Clarifiers		
Type	Center feed, scraper	
Radius	27.5 ft	
Number	2	
Surface Area (each)	2,376 ft^2	
Overflow Rate at 3.4 mgd	715 gpd/ ft^2	
Rotating Biological Contactors		
Type	High density, polyethylene	
Number	16	
Diameter	12 ft	
Surface Loading Rate at 3.4 mgd	1.7 gpd/ft^2	
Air Requirement	2,700 cfm	
Secondary Clarifier		
Type	Center feed, scraper	
Radius	27.5 ft	
Number	2	
Surface Area (each)	2,376 ft^2	
Overflow Rate at 3.4 mgd	715 gpd/ft^2	
Filters		
Type	Gravity sand	

Table 2-5
Design Criteria and Design Factors
Existing Woodburn WWTP

Design Parameter	*Value*
Bed Depth	11 in
Sand Size	0.6-0.65 mm
Hydraulic Loading Rate at 3.1 gpm	2 gpm/ft^2
Backwash	Continuous
Backwash Rate	18 gpm/ft^2
Chlorine Contact Chamber	
Type	Serpentine
Number of Basins	2
Detention Time at 3.4 mgd	1 hr
Length-to-Width Ratio	40:1
Chlorination	
Number	2
In Use	1
Standby	1
Chlorinator Capacity (each)	500 lb/day
Chlorine Storage	8,000 lb
Anaerobic Digesters	
Primary Digester	
Type	Mesophilic
Mixing Mode	Compressed digester gas
Diameter	50 ft
Side Wall Depth	23 ft
Volatile Solids Loading	0.08 lb VSS/ft^3/day
Cover Type	fixed
Secondary Digester	
Thickening Mode	Supernatant drawoff
Mixing Mode	Compressed digester gas
Diameter	50 ft
Side Wall Depth	21 ft
Cover Type	Gasholder
Sludge Drying Beds	
Type	Paved, uncovered
Number	14
Solids Loading Rate	15 lb/ft^2/yr
Dimensions	50 ft x 100 ft x 18 in SWD
Drainage Area Per Bed	510 ft^2
Drainage Media	
Sawdust, Minimum Depth	6 in
Gravel, Minimum Depth	14 in
Emergency Storage and Evaporation Pond	
Type	Earth-lined, aerated
Area	14.2 acres
Storage Capacity	37 million gallons

FIGURE 2-3
Aerial View of Woodburn WWTP

Facility Description

The existing Woodburn WWTP was constructed in 1980 and is located outside the City limits on the west bank of the Pudding River. The WWTP is located on 4.5 acres of MacLaren School land leased from the State of Oregon. Rotating biological contactor (RBC) units provide secondary treatment and sand filters provide tertiary treatment of domestic and industrial wastewaters provide secondary treatment. The plant is operated under two modes: summer and winter process modes. A summer flow schematic of the existing WWTP is shown in Figure 2-4.

Raw wastewater (domestic and industrial) is pumped from the Mill Creek pump station to the headworks, which consists of two mechanical shredders and a grit removal chamber. A flowmeter was installed during the summer of 1991 at the influent to the headworks. A septage receiving station also empties into the grit removal chamber. Degritted wastewater is split between two primary clarifiers. These units are circular (27.5 ft radius), center feed clarifiers.

Primary clarifier effluent follows one of two paths before entering the RBC units; the plant is operated both ways. The primary clarifier effluent flows to either the storage lagoon and then to the pre-aeration cell, or to the pre-aeration cell and then to the storage lagoon. From the pre-aeration cell or lagoon, the wastewater then flows to the RBCs.

The RBC units are high-density polyethylene, 12-foot diameter, air-driven units. The sixteen air-driven RBC units are arranged in two eight-unit basins. The wastewater enters the first RBC basin, and flows into a contact aeration channel followed by the second RBC basin. This operational mode (RBC operation in series) provides additional reduction of soluble BOD during the cannery season. The RBCs can also be operated in parallel.

Mixed liquor from the biological system flows into two secondary clarifiers. The secondary clarifiers are similar to the primary clarifiers. Secondary effluent is filtered through two gravity sand filters with continuous backwash. The sand beds are 11 inches deep.

The plant effluent flow is measured through a Parshall flume and the filter effluent is chlorinated. The chlorine contact basin is a serpentine, twin-cell basin with a detention time of 1 hour at 3.4 mgd. The length to width ratio is 40 to 1. Plant effluent flows through an approximately 1-mile-long, 24-inch outfall and is discharged into the Pudding River.

The winter process configuration is schematically illustrated in Figure 2-5. The winter operational mode is similar to the summer configuration, except for the operation of the rotating biological system. Primary clarifier effluent bypasses the storage pond and pre-aeration cell and flows directly into the RBC basins, which operate in parallel. Effluent from the RBC basins flows into the secondary clarifiers. Filtration and chlorination follow before outfall discharge into the Pudding River.

Sludge and scum from the secondary clarifiers are returned to the raw influent wastewater stream just downstream of the grit chamber or to the lagoon. Sludge and scum are removed

Figure 2-4
Woodburn Wastewater Treatment Plant
(Summer Process Config.)

Figure 2-4

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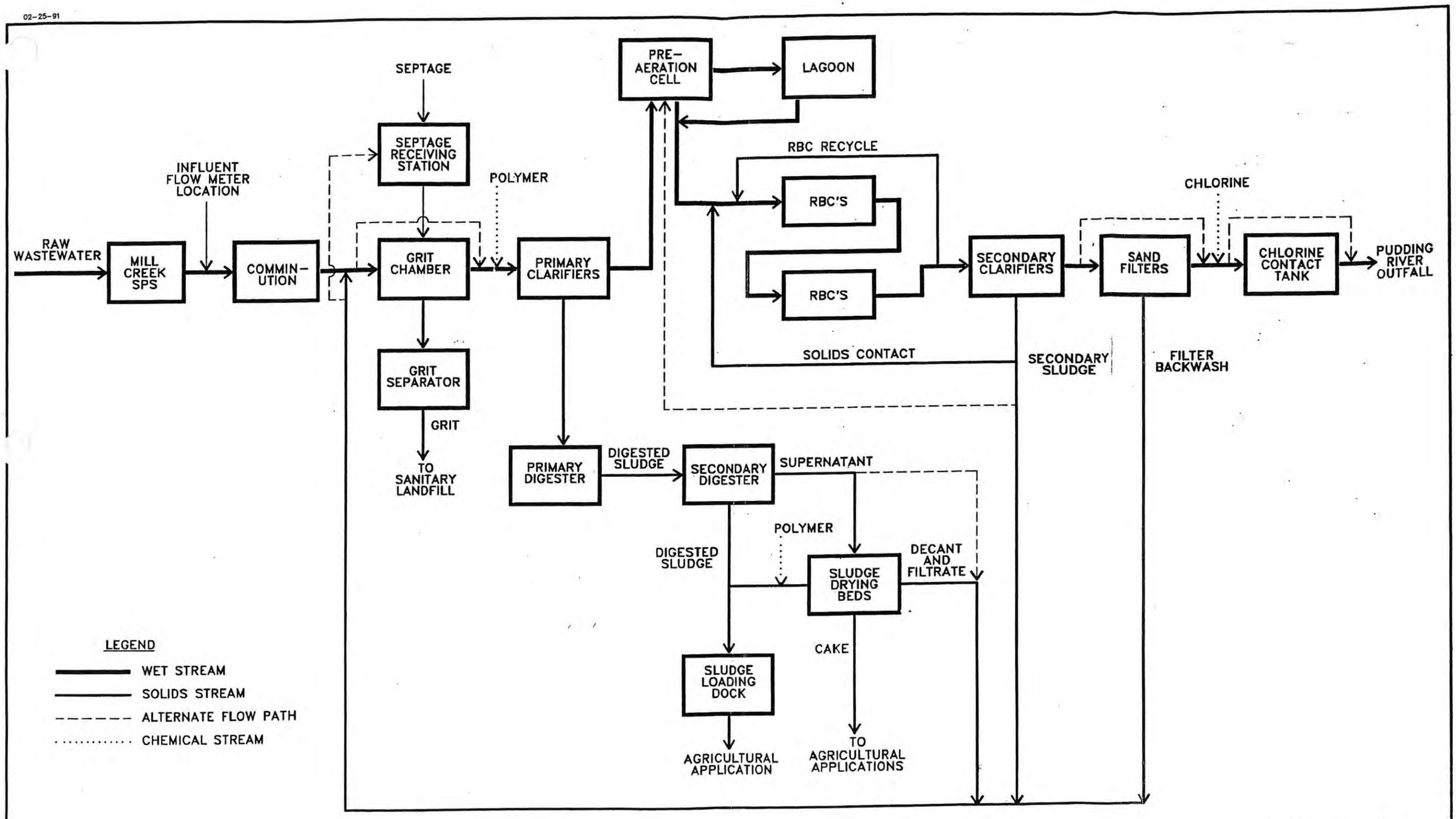

NOTE:
LAGOON ONLY USED DURING CANNING SEASON (JUNE, JULY, AND AUGUST)

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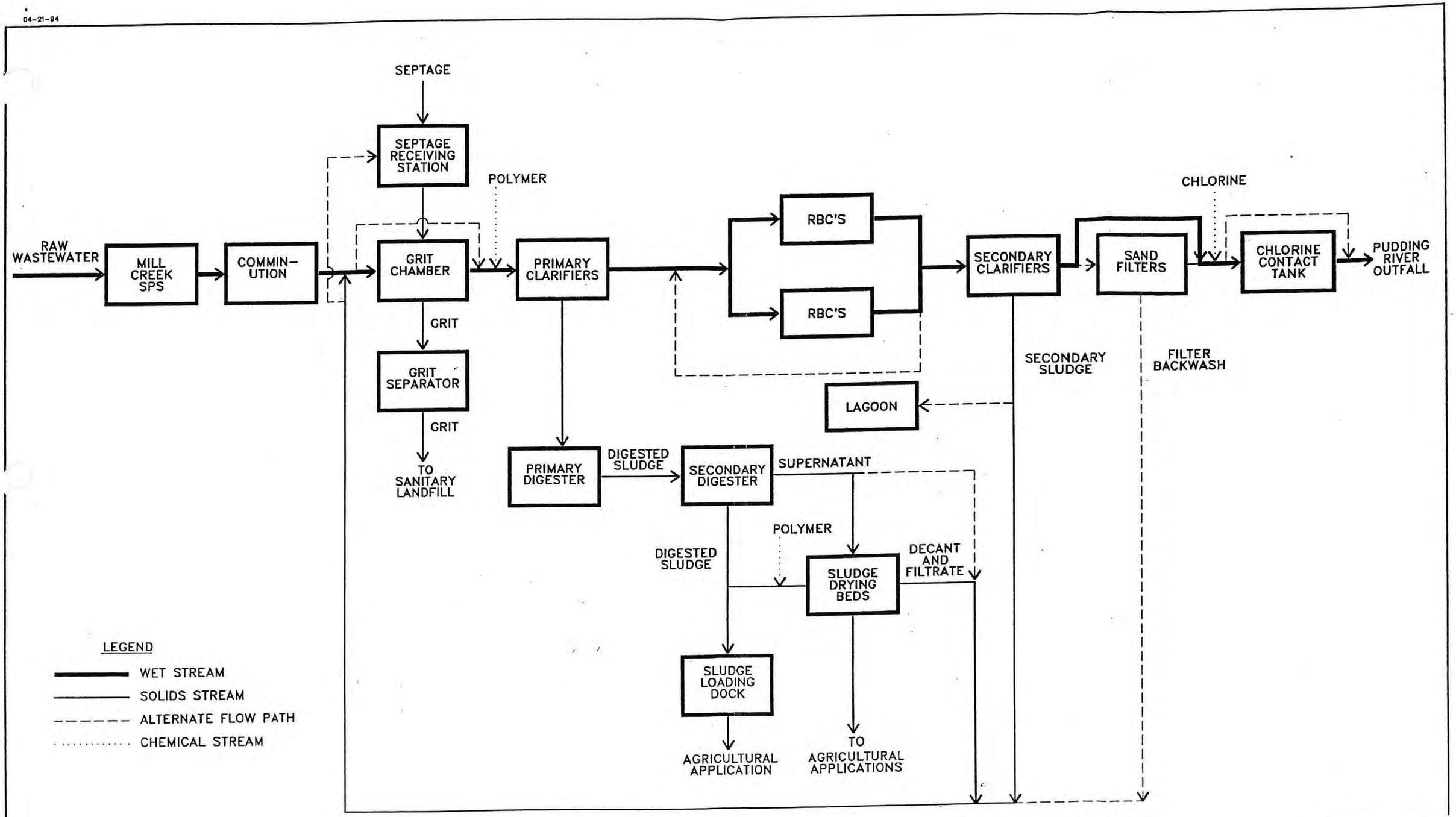
Figure 2-4
WOODBURN WASTEWATER TREATMENT PLANT
SUMMER PROCESS CONFIGURATION

CH2M HILL

04-21-94



NOTE:
LAGOON ONLY USED DURING 1990 CANNING SEASON (JUNE, JULY, AND AUGUST)

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Figure 2-5
WOODBURN WASTEWATER TREATMENT PLANT
WINTER PROCESS CONFIGURATION

CH2M HILL

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Figure 2-5
WWTP
(Winter Process Config.)

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Figure 2-5

from the primary clarifiers and pumped to primary and secondary digesters. The digested sludge (biosolids) can then be pumped to three locations:

- Sludge loading dock for loading into a pumper truck
- Sludge drying beds
- Emergency storage lagoon

Ultimate disposal of the sludge is by land application to agricultural fields at the treatment plant site.

Existing Wastewater Flows and Characteristics

Plant Flows

The flows to the Woodburn WWTP follow a typical wet season/dry season pattern. The dry and wet seasons are defined in the National Pollutant Discharge Elimination System Permit (NPDES) for Woodburn. The dry season is defined as the months of May through October. The wet season is defined as the months of November through April. The following definitions for wastewater flow rates for design of treatment facilities are used in this facilities plan:

- Average Dry Weather Flow (ADWF): the average of the daily flows for the months of May through October.

- Average Wet Weather Flow (AWWF): the average of the daily flows for the months of November through April.

- Dry Weather Maximum Daily Flow (DWMDF): the highest daily flow recorded during the months of May through October.

- Wet Weather Maximum Daily Flow (WWMDF): the highest daily flow recorded during the months of November through April.

- Dry Weather Maximum Month Average Daily Flow (DWMMADF): the highest 30-day average flow, calculated on a calendar month basis (for May through October) consistent with NPDES reporting requirements.

- Wet Weather Maximum Month Average Daily Flow (WWMMADF): the highest 30-day average flow, calculated on a calendar month basis (for November through April) consistent with NPDES reporting requirements.

- Dry Weather Minimum Daily Flow (DWMiDF): the lowest daily flow recorded during the months of May through October.

- Dry Weather Minimum Month Average Daily Flow (DWMiMADF): the lowest 30-day average flow, calculated on a monthly basis (for May through October) consistent with NPDES reporting requirements.

- Annual Average Daily Flow (AADF): total flow during the year divided by 365 days.

Influent and Effluent Flow Comparison

Wastewater flow data from January 1989 through December 1993 are used to determine historical flow variations. Effluent flow measurements are used for 1989, 1990, and the first half of 1991. An influent flowmeter was installed during the summer of 1991 and both influent and effluent flow measurements are available after July 1991. However, the influent flowmeter was out for repairs for 7 months during 1992. Additionally, there are several days throughout 1991 through 1993 where no influent flow measurements are available.

Daily influent and effluent flows are compared (on tables and graphs) in Appendix 2.1. Comparisons of influent and effluent flows on a monthly basis are also contained in Appendix 2.1. From these data it can be seen that the effluent flow is similar to the influent flow except during the times when the plant discharges to the holding pond because of high loads. For this analysis, influent flow data are used whenever they are available to determine historical flow variations. Effluent flows are used if there are no records of the influent flows.

Historical Flows

Historical WWTP flows for the 5 years 1989 through 1993 (period of record, POR) are presented in Table 2-6. Average dry weather daily flow (ADWF) is 1.78 mgd; average wet weather daily flow (AWWF) is 2.19 mgd. These data are from relatively dry years.

Plant Influent Load Characterization

Historical WWTP influent BOD_5 and TSS loads from 1990 to 1993 are summarized in Table 2-7 (see Appendix 2.1 for more detail). The average dry weather BOD_5 load is 3,876 lb/day; the maximum dry weather TSS load is 10,575 lb/day. Wet weather BOD_5 loads are slightly lower for the period of record. The average dry weather TSS load is 2,673 lb/day); the maximum dry weather TSS load is 7,499 lb/day. The wet weather average TSS load is slightly lower (2,530 lb/day); however, the maximum TSS load is 9,025 lb/day for the period of record, which occurs during wet weather. The maximum loads for both dry and wet weather are greater than capacity of the secondary system (3,350 lb/day BOD_5).

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Table 2-6
Historical Flow Summary
Woodburn WWTP
1989-1993

Monthly Average Flows (mgd) (a)

Month	*1989*	*1990*	*1991*	*1992*	*1993*	*Average*
January	1.895	2.597	1.776	2.716	2.118	2.220
February	2.057	3.318	2.090	3.528	1.824	2.563
March	3.117	2.472	2.358	2.317	2.585	2.570
April	2.191	1.914	2.513	2.158	2.767	2.309
May	1.785	2.035	2.649	1.819	2.258	2.109
June	1.869	2.081	2.011	1.083	2.455	1.900
July	1.826	1.801	1.727	1.825	1.831	1.802
August	1.829	1.792	1.625	1.461	1.663	1.674
September	1.546	2.219	1.520	1.428	1.533	1.649
October	1.557	1.833	1.613	1.249	1.486	1.548
November	1.657	1.653	1.790	1.537	1.580	1.643
December	1.862	1.501	2.291	1.862	1.758	1.855

Flow Conditions	*1989*	*1990*	*1991*	*1992*	*1993*	*Average*	*Maximum*
Annual Average (AADF)	1.933	2.101	1.997	1.915	1.988	1.987	—
Dry Weather Average	1.736	1.960	1.857	1.478	1.871	1.780	—
Wet Weather Average	2.130	2.242	2.136	2.353	2.105	2.193	—
Dry Wthr Max Month	1.869	2.219	2.649	1.825	2.455	2.204	2.649
Wet Wthr Max Month	3.117	3.318	2.513	3.528	2.767	3.048	3.528

Dry Weather Flows (mgd) - May through October	*Average*	*Max/Min*
Average Dry Weather Flow (ADWF)	1.780	—
Dry Weather Maximum Daily Flow (b) (DWMDF)	—	6.141
Dry Weather Minimum Daily Flow (DWMiDF)	—	0.408
Dry Weather Maximum Month Average Daily Flow (DWMMADF)	2.204	2.649
Dry Weather Minimum Month Average Daily Flow (DWMiMADF)	—	1.083

Wet Weather Flows (mgd) - November through April	*Average*	*Maximum*
Average Wet Weather Flow (AWWF)	2.193	—
Wet Weather Maximum Daily Flow (WWMDF) (c)	—	7.920
Wet Weather Maximum Month Average Daily Flow (WWMMADF)	3.048	3.528

Notes:
(a) Shaded cells are influent flows. Non-shaded cells are effluent flows.
(b) May 17, 1991 (effluent flow measurement).
(c) February 19, 1992 (effluent flow measurement).

Table 2-7
Influent Loading
Woodburn WWTP
1990-1993

	Influent BOD_5 (mg/L)	*Influent BOD_5 (lb/day)*	*Influent TSS (mg/L)*	*Influent TSS (lb/day)*
Summary: 1990 - 1993				
Period of Record				
Maximum	855	10,575	490	9,025
Minimum	12	190	22	219
1990 Average	206	3,302	157	2,555
1991 Average	185	3,058	148	2,478
1992 Average	258	3,609	182	2,736
1993 Average	244	4,279	153	2,637
POR Average	223	3,547	160	2,600
Dry Weather				
Maximum	855	10,575	490	7,499
Minimum	78	190	54	797
Average	258	3,876	180	2,673
Wet Weather				
Maximum	468	10,028	308	9,025
Minimum	12	398	22	219
Average	188	3,248	140	2,530

Notes:
POR = Period of Record.

Plant Biosolids Data

The City has recorded biosolids nutrient and metals data at least yearly for over 10 years as part of its ongoing biosolids management program. Data for 1986-94 are summarized in Table 2-8. Average values (from Table 2-8) are presented in Table 2-9 and compared with the ceiling concentration limits and exceptional quality values that have recently become part of the Federal (Part 503) regulations governing biosolids land application. From Table 2-9 it can be seen that the Woodburn WWTP biosolids are of good enough quality to be land applied (that is, they are less than the ceiling values). The average biosolids concentration values are better than the Part 503 exceptional quality criteria, and only two cadmium and one silver data points exceed them. These exceedances appear to be anomalies. It should also be noted that all data points are for digested primary sludge only, which would be expected to have higher metals concentrations than if the plant's secondary sludge were included.

Effluent Biomonitoring Results

CH2M HILL performed two acute screening bioassays and two chronic definitive bioassays on Woodburn WWTP effluent. The effluent samples were provided by City of Woodburn personnel. The tests were conducted on February 15 through 22, 1991. Test organisms used were the water flea (*Ceriodaphnia dubia*) and the fathead minnow (*Pimephales promelas*). EPA test methods were followed for all procedures.

Sample Collection And Preparation

CH2M HILL's Corvallis Aquatic Toxicology Laboratory conducted the acute screening and chronic definitive testing of the effluent samples. The effluent samples were collected and delivered to the lab by City of Woodburn personnel. The samples were collected on February 15, 18, and 20, 1991. Upon arrival at the lab, the samples were dechlorinated with sodium thiosulfate, filtered through a 60-micron net and temperature adjusted before each daily renewal. The samples were stored at 4 degrees C in the dark until the test solutions were prepared.

Acute Toxicity Tests

The concentrations tested in the acute tests were 100 percent effluent and dilution water for the control. The concentrations were run in duplicate with ten organisms per replicate. The response that was measured during the acute tests was survival over the exposure period.

There was no reduction in the survival of the *Pimephales promelas* over the 96 hour duration of the test in either the control or the 100 percent effluent. Acute toxicity was observed in the *Ceriodaphnia dubia* test within the first 24 hours.

Table 2-8
Biosolids Nutrient and Metals Data
Woodburn WWTP
1987-1994

	7/94 DB	10/93 DB	2/93 DB	9/93 DB	8/92 DB	7/92 PRI	7/92 SEC	7/92 DB	3/92 -	12/91 -	9/91 -	3/90 -	10/89 -	3/88 -	9/87 -
%TS	53	81		54	5				3	5	6	6	5	4	80
TKN	10,000	14,700		1,000							24,400	18,370	22,560	19,500	7,300
P		13,300		11,000							2,050	1,420	3,800	1,800	
K	1,220	800		48							166	1,993	2,080	39	
Arsenic	13	9		62	54	9	14	14	24						
Cadmium	3	3		67	5	2	4	5	4	4	24		9	5	5
Chromium	25	42		26	17	16	28	26	40	33	75	105	138	142	36
Copper	223	239		178	313	141	217	312	289	117	187	276	203	276	215
Lead	37	72		27	75	34	35	84	49	4	1	17	140	80	103
Mercury	4	0		1	5	1	2	1	3						
Molybdenum	3	12	6	1											
Nickel	23	38		1	24	18	25	30	32	4	1	21	147	39	21
Selenium	5	3	1	19											
Silver	37	19		1	2	10	20	18	3						
Zinc	738	677		598	1,078	552	4,676	1,347	996	626	51	640	972	1,140	

Notes:
DB=Drying Beds
PRI=Primary Digestion
SEC=Secondary Digester

Table 2-9 Biosolids Metals Data Woodburn WWTP			
Constituent	Available Woodburn WWTP Data (mg/kg)	Part 503 Ceiling Values[a] (mg/kg)	Part 503 Exceptional Quality Values (mg/kg)
Arsenic	25	75	41
Cadmium	11	85	39
Chromium	54	3,000	1,200
Copper	228	4,300	1,500
Lead	54	840	300
Mercury	2	57	17
Molybdenum	5	75[b]	18[b]
Nickel	30	420	420
Selenium	7	100	36
Silver	14	--	--
Zinc	1084	7,500	2,800

Notes:

[a]Maximum concentration allowed for land application.

[b]Initially proposed values currently under review.

Bio-monitoring

Chronic Definitive Tests

The concentrations tested in the chronic definitive tests were 100, 30, 10, 3, and 1 percent effluent and dilution water for control. For the *Ceriodaphnia dubia* test, 10 organisms per concentration were used, each in a separate vessel. For the *Pimephales promelas* test, 40 organisms per concentration were used (10 in each of four chambers).

The effects measured during the *Ceriodaphnia dubia* chronic test included survival and reproduction over the 7-day exposure period. The response measured in the *Pimephales promelas* tests included survival and growth over the 7-day exposure period.

The results of the *Ceriodaphnia dubia* test show a statistically significant reduction in survival at the 100 percent effluent concentration. The lowest observed effect concentration (LOEC) for survival is 100 percent effluent; the no observed effect concentration (NOEC) for survival is 30 percent effluent. The chronic value (the geometric mean of the LOEC and NOEC values) is 54.8 percent effluent.

The results of the *Pimephales promelas* test show no statistically significant reduction in survival at any of the concentrations. However, there is a statistically significant reduction in growth at the 3, 10, and 30 percent effluent concentrations. The LOEC for growth is 3 percent effluent; the NOEC for growth is 1 percent effluent. The chronic value for growth is 1.7 percent effluent.

The results of the *Ceriodaphnia dubia* test showed a NOEC for survival of 30 percent effluent. The results of the *Pimephales promelas* survival test showed an NOEC for survival at any of the tested concentrations.

Summary Of December 1994 Tests

More recent chronic and acute testing was done with samples collected December 6 through 13, 1994. The following is a quote from a report in progress from the CH2M HILL Corvallis Aquatic Toxicology Laboratory (to be published):

> The Ceriodaphnia dubia showed no statistically significant reduction in survival or reproduction when compared to the laboratory control at any effluent concentration tested. The no observed effect concentration (NOEC) and the lowest observed effect concentration (LOEC) for survival and reproduction were 100 and greater than 100 percent effluent, respectively.
>
> The IC25 (the concentration of effluent causing a 25 percent reduction in biological measurement, e.g., growth) values calculated for *Ceriodaphnia dubia* survival and reproduction were 90.0 and greater than 100 percent effluent, respectively.

Chronic testing was also done on the fathead minnow. The results are reported as follows:

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The fathead minnows showed a statistically significant reduction in weight at the 100 percent effluent concentration when compared to the laboratory control. The NOEC and LOEC were 50 and 100 percent effluent, respectively.

The IC25 value was calculated on the fathead minnow survival and growth. The IC25 values for survival and growth were both greater than 100 percent effluent.

Thus, this most recent testing indicates the dilutions in the mixing zone (15 and 26 percent effluent for 3.1 and 5.10 mgd effluent flow, respectively) would have no statistical reduction in weight or survival of the fathead minnow, as well as no statistical reduction in reproduction or survival of the *Ceriodaphnia dubia*.

Deficiencies of Existing Wastewater Treatment Facility

The deficiencies of the existing wastewater treatment facilities are divided into four categories: capacity limitations, water quality regulations, process equipment, and building codes and safety.

The Woodburn WWTP has been in service since the early 1980's with year-round discharge to the Pudding River (except during the 1990 summer canning season). The WWTP was upgraded in 1987 to improve the capacity and treatment of peak industrial loadings. The improvements included:

- Addition of a recirculation channel to allow the two RBC basins to operate in series during time of peak loadings (summer canning season) and low flow in the Pudding River
- Addition of a pre-aeration pond for aerating primary effluent prior to its introduction to the RBC units

The Oregon Department of Environmental Quality (DEQ) established new treatment plant effluent requirements for the Pudding River in November 1993 by setting Pudding River Total Maximum Daily Loads (TMDLs) for the City of Woodburn.

Building and safety codes have changed since the WWTP was built. Some equipment may warrant replacement or modification.

Capacity Limitations

According to the *Woodburn Wastewater Facilities Capital Improvements Plan* (CH2M HILL, January 1987), the following capacity limitations apply:

> The Mill Creek Pump Station and treatment system has a current treatment capacity (including RBC recirculation and pre-aeration) of approximately 3.4 mgd of average daily wastewater flow, 8.4 mgd peak hourly flow, 7,200 lb/day of BOD_5, 5,000 lb/day of $SBOD_5$, and 6,200 lb/day of TSS. Average daily flows are limited

by the capacities of the primary and secondary clarifiers and chlorine contact chamber. Organic loading is limited by the capacity of the anaerobic digesters and sludge drying beds. Peak hourly flows are limited by the capacities of the Mill Creek Pump Station, plant headworks, plant process piping, and clarifiers.

The City of Woodburn WWTP was designed in the late 1970's. At that time, the RBC process was considered an attractive alternative which saved on energy costs and received support from the U.S. Environmental Protection Agency's (EPA) innovative and alternative technology program. CH2M HILL designed the original plant, following design criteria recommended by the EPA and the RBC manufacturer. Since the RBC system was designed and built, the RBC design criteria have been derated by EPA, thus affecting the design capacity of the treatment plant.

The RBC system was designed with the assumption that the wastewater characteristics would be predominantly municipal, based on existing plant daily monitoring reports, the current facilities plan, and small scale pilot testing.

Plant staff members have indicated that recent operating data show the secondary treatment system with the pre-air cell and lagoon online is actually limited to 3,350 lb/day influent BOD_5. It is currently not able to treat summer loads.

It was assumed that industrial contributions would increase at the same rate as domestic contributions and the wastewater characteristics would stay proportional to the predominantly domestic wastewater characteristics at the time of the 1987 facilities plan. Contrary to expectations, industrial and commercial contributions grew significantly. Industrial contributions vary seasonally with the operation schedules of the canneries and loads are currently higher than anticipated at design. The current secondary treatment system (including the pre-aeration cell) can handle only 3,350 lb/day BOD. Because of the secondary capacity limitations, the City entered into an SFO agreement with DEQ effective January 1, 1993. The interim limits for effluent BOD and TSS in the dry season are relaxed from 10 mg/L to 20 mg/L BOD and TSS until new facilities are built.

The SFO agreement (see Appendix B) contained the following schedule to address the noncompliance issues:

- 12 months after establishment of the TMDL, the City must submit a final Facilities plan that includes an evaluation of alternatives for complying with the TMDL and the water quality standards for chlorine and ammonia. The facilities plan shall also include an evaluation of the mixing zone.

- 18 months after approval of the facilities plan, the City must submit engineering plans and specifications for construction of necessary improvements.

- 24 months after approval of the facilities plan, the City must award construction contracts for the completion of the necessary improvements.

- 44 months after approval of the facilities plan, the City must complete the construction of the necessary improvements.

An amendment to the SFO extended the facilities plan submittal to 20 months after establishment of final TMDLs and added a requirement for a predesign submittal 4 months after facilities plan approval. Additional capacity limitations include the secondary clarification and filtration system. Current design criteria for secondary clarifier overflow rates indicate the existing clarifiers are too small. The filters have been derated by plant staff from the design capacity of 3.1 mgd to 2.2 mgd because of aging equipment and flow short-circuiting.

Water Quality Regulations

The Oregon Administrative Rules (OAR 340-41) define a State-Wide Water Quality Management Plan. Beneficial Uses, Water Quality Standards, and Minimum Design Criteria for Treatment and Control of Wastes related to the Willamette River are included under the Willamette Basin Plan (OAR 340-41-442, 445, 455).

The City's NPDES Waste Discharge Permit was renewed in 1992 following expiration of the previous permit. The permit was renewed in September 1992 and is designed to be effective until September 30, 1997. A copy of the current permit is provided in Appendix A. Table 2-10 summarizes the current discharge criteria and mass load limits of the permit. The permit, however, is intended to be opened for review and modification during the course of the facilities planning process. Variances are allowed as defined in the SFO. On January 1, 1993, DEQ approved an SFO giving the City a discharge limit of 20 mg/L $CBOD_5$ and 20 mg/L TSS. These effluent limitations have been consistently met by the Woodburn WWTP. The limits are effective only until the City completes construction of a new or expanded wastewater treatment facility.

Effluent Limits

The current plant permit requires an effluent $CBOD_5$ concentration of 25 mg/L (monthly) and 40 mg/L (weekly) during the winter (November through April). Effluent $CBOD_5$ concentrations of 10 mg/L (monthly) and 15 mg/L (weekly) are required during the summer (May through October). The current permit also requires an effluent TSS concentration of 30 mg/L (monthly) and 45 mg/L (weekly) during the winter (November through April). Effluent TSS concentrations of 10 mg/L (monthly) and 15 mg/L (weekly) are required during the summer (May through October). The SFO relaxes the effluent concentrations to 20 mg/L for both $CBOD_5$ and TSS.

Wet Weather Bypasses

Any bypass of secondary treatment is a violation of federal water quality regulations.

Table 2-10 Current Discharge Criteria and Mass Load Limits Woodburn WWTP					
	Average Effluent Concentrations*		Mass Load Limitations		
Parameters	Monthly mg/L	Weekly mg/L	Monthly Average lb/day	Weekly Average lb/day	Daily Maximum lb/day
May 1 through October 31					
$CBOD_5$	10	15	259	388	517
TSS	10	15	259	388	517
FC/100 mL	200	400			
November 1 through April 30					
$CBOD_5$	25	40	646	970	1,293
TSS	30	45	776	1,163	1,551
FC/100 mL	200	400			
Other Parameters:					
pH	Shall be within the range 6.0-9.0.				
$CBOD_5$, BOD_5, TSS	Shall not be less than removal efficiency 85% monthly average.				
Total Residual Chlorine	Shall not exceed daily maximum of 0.03 mg/L.				
Mixing Zone	Shall not extend beyond 270 feet downstream from discharge.				
*Based on average dry weather design flow equaling 3.1 mgd.					

Monitoring and Reporting Requirements

Both the influent and effluent flow streams have monitoring requirements. The influent flow must be monitored for:

- Total flow (daily)
- Flow meter calibration (semi-annually)
- $CBOD_5$ (two times per week)
- TSS (two times per week)
- pH (three times per week)
- Metals (semi-annually)
- Total phenols (semi-annually)

The effluent flow must be monitored for:

- Total flow (daily)
- Flowmeter calibration (semi-annually)
- $CBOD_5$ (two times per week)
- TSS (two times per week)

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- Fecal coliform (two times per week)
- pH (three times per week)
- Quantity of chlorine used (daily)
- Chlorine residual (daily)
- Average percent removed (CBOD, and TSS, monthly)
- Nutrients (ammonia, nitrogen, nitrite, and nitrate, two times per week)
- Total phosphorus (weekly between June 1 and October 31)
- Metals (semi-annually)
- Total phenols (semi-annually)
- Toxics removal (semi-annually)
- Bioassay of effluent from outfall (quarterly)

In addition to the influent and effluent monitoring requirements, there are groundwater and sludge management monitoring requirements. These requirements include groundwater level elevation measurement, sludge analysis (including metals content), and maintaining a record of locations where sludge is applied on land.

Compliance Schedules

Schedules for completion of plans and implementation of the plans are included for identifying and reducing inflow, identification and monitoring of overflow points, and for the prevention and handling of spills.

Pretreatment Program

A pretreatment program is identified and outlined. The City is required to conduct and enforce a pretreatment program that includes the following items:

- Maintaining an updated inventory of industrial users
- Issuing individual discharge permits to significant industrial users
- Randomly sampling and analyzing industrial user effluents
- Reviewing reports submitted by industrial users
- Investigating all instances of industrial user noncompliance
- Publishing (annually) any industrial users that were in significant noncompliance with the pretreatment requirements
- Developing and maintaining a data management system designed to track the status of the industrial user inventory, discharge characteristics, and compliance

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- Reporting to DEQ (annually) the status of the pretreatment program during the previous year

Outfall Mixing Zone

The current NPDES permit defines the allowable mixing zone as the region within 270 feet downstream of the point of discharge. According to OAR 240-41-445, the water outside the mixing zone boundary must be free of materials in concentrations that will cause chronic toxicity. Additionally, the water must meet all other water quality standards under normal annual low-flow periods. The NPDES permit also defines the Zone of Immediate Dilution (ZID); it must be within 10 percent of the allowable mixing zone. Water at the edge of the ZID must be free of materials in concentrations that will cause acute toxicity. DEQ may redefine the ZID after completion of the WWTP modifications.

The present average dry weather plant flow (ADWF) is 3.1 mgd (4.8 cfs) and the future ADWF is estimated to be 5.0 mgd (7.7 cfs). A seven-day-average-low flow of approximately 50 cfs occurs every other year in the Pudding River near Aurora (7Q2). DEQ has also indicated that the 7Q10 flow is equal to 15 cfs. Given a Pudding River flow of 50 cfs, the maximum dilution ratio that can be achieved with complete mixing is 11.4 (for effluent flow of 3.1 mgd) and 7.5 (for effluent flow of 5.0 mgd). At a Pudding River flow of 15 cfs, the dilution ratios are even lower. The dilution does not meet the dilution requirement of a 15:1 dilution for BOD and TSS and a dilution waiver is required from DEQ.

The effect of these dilutions on the predicted concentration levels of various constituents is presented in Tables 2-11 and 2-12. As can be seen, mercury and silver are at unacceptable levels. The levels of these constituents will be reduced by additional pretreatment programs. Additional sampling and analysis for mercury and silver are recommended with clean sampling and testing techniques after the new plant upgrades are completed.

The approved pretreatment program recognizes that silver and mercury need to be reduced by tighter source control. Silver and mercury are controlled by local limits in the approved pretreatment program. The local limits will need to be reevaluated after the treatment plant improvements are implemented.

Chlorine Toxicity

For chlorine, acute and chronic toxicity concentrations are defined as 0.019 and 0.011 mg/L, respectively. In order to meet the requirements of OAR 340-41-445, the discharge shall not exceed a daily maximum of 0.10 mg/L and a monthly average of 0.04 mg/L. Additionally, adequate dilution and mixing must take place in the river for concentrations outside the mixing zone to be 0.011 mg/L. The practical limit for measurement of chlorine concentration is 0.1 mg/L. The Woodburn WWTP does not currently dechlorinate and residual chlorine concentrations are above the acute toxicity limit.

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Table 2-11
Predicted Concentration Levels
Pudding River at Woodburn
Pudding River = 15 cfs

Constituent	*Chronic Criteria (must be met outside the mixing zone) (a) (ug/l)*	*The WWTP effluent has the following concentrations of the listed constituents (f) (ug/l)*	*To meet the Chronic Criteria at the edge of the mixing zone, the effluent must have a concentration less than the amounts listed for the following constituents (WWTP flow = 3.1 mgd, Dilution = 4.1, River flow = 15 cfs) (ug/l)*	*To meet the Chronic Criteria at the edge of the mixing zone, the effluent must have a concentration less than the amounts listed for the following constituents (WWTP flow = 5.0 mgd, Dilution = 2.9, River flow = 15 cfs) (ug/l)*
Arsenic III (b)	190	4.667 (d)	779	551
Cadmium (c)	1.13	2.944	4.633	3.277
Chromium III (c)	206.98	(e)	848.618	600.242
Chromium VI (b)	11	7.000 (e)	45.1	31.9
Total Chromium (c)	217.98	(e)	893.718	632.142
Copper (c)	11.82	13.333	48.462	34.278
Cyanide (b)	5.2	2.917	21.32	15.08
Lead (c)	3.22	2.778	13.202	9.338
Mercury (b)	**0.012**	**0.706**	**0.0492**	**0.0348**
Nickel (c)	157.67	9.222	646.447	457.243
Silver (b)	**0.12**	**2.778**	**0.492**	**0.348**
Zinc (c)	105.99	45.444	434.559	307.371

Notes:
(a) From Industrial Pretreatment Program, Local Discharge Limitations, City of Woodburn, Oregon, Appendix A.
(b) Not dependent on hardness.
(c) Dependent on hardness (100 mg/l used).
(d) Valence of arsenic not indicated. Arsenic III assumed.
(e) Valence of cadmium not indicated (ie Cadmium VI or Cadmium III). Cadmium VI assumed.
(f) From Industrial Pretreatment Program, Local Discharge Limitations, City of Woodburn, Oregon, Table C.

Table 2-12
Predicted Concentration Levels
Pudding River at Woodburn
Pudding River = 21 cfs

Constituent	*Chronic Criteria (must be met outside the mixing zone) (a) (ug/l)*	*The WWTP effluent has the following concentrations of the listed constituents (f) (ug/l)*	*To meet the Chronic Criteria at the edge of the mixing zone, the effluent must have a concentration less than the amounts listed for the following constituents (WWTP flow = 3.1 mgd, Dilution = 5.4, River flow = 21 cfs) (ug/l)*	*To meet the Chronic Criteria at the edge of the mixing zone, the effluent must have a concentration less than the amounts listed for the following constituents (WWTP flow = 5.0 mgd, Dilution = 3.7, River flow = 21 cfs) (ug/l)*
Arsenic III (b)	190	4.667 (d)	1026	703
Cadmium (c)	1.13	2.944	6.102	4.181
Chromium III (c)	206.98	(e)	1117.692	765.826
Chromium VI (b)	11	7.000 (e)	59.4	40.7
Total Chromium (c)	217.98	(e)	1177.092	806.526
Copper (c)	11.82	13.333	63.828	43.734
Cyanide (b)	5.2	2.917	28.08	19.24
Lead (c)	3.22	2.778	17.388	11.914
Mercury (b)	**0.012**	**0.706**	**0.0648**	**0.0444**
Nickel (c)	157.67	9.222	851.418	583.379
Silver (b)	**0.12**	**2.778**	**0.648**	**0.444**
Zinc (c)	105.99	45.444	572.346	392.163

Notes:
(a) From Industrial Pretreatment Program, Local Discharge Limitations, City of Woodburn, Oregon, Appendix A.
(b) Not dependent on hardness.
(c) Dependent on hardness (100 mg/l used).
(d) Valence of arsenic not indicated. Arsenic III assumed.
(e) Valence of cadmium not indicated (ie Cadmium VI or Cadmium III). Cadmium VI assumed.
(f) From Industrial Pretreatment Program, Local Discharge Limitations, City of Woodburn, Oregon, Table C.

Process Equipment

The City of Woodburn's current wastewater treatment system will complete its designed useful life in the year 2000.

The RBCs are deteriorating and will need replacement media by the year 2000. Plant staff have indicated that the filters require regular maintenance and the parts are hard to obtain. The filters often plug from oil and grease, and experience short circuiting. Their performance is limited by the influent quality. For their design, it was assumed that the filter influent would be 15 mg/L; however, this is not always achieved. Sampling has shown that metals concentrations actually increase through the filters. Current capacity is estimated to be 2.2 mgd.

The existing chlorine gas disinfection system adequately disinfects effluent to meet the existing 200 fecal coliform standard. However, because of chlorine toxicity to the Pudding River , a dechlorination system would be required if chlorine disinfection were continued, or an alternative disinfection process such as ultraviolet (UV) light should be considered.

More laboratory space in the Control Building is needed because of the increase in analytical testing required by current and future discharge permits.

Building Codes And Safety

Various building codes and safety regulations have changed since the Woodburn WWTP was constructed. New municipal wastewater plants are required to be designed in accordance with National Fire Protection Act (NFPA) 820, which stipulates various ventilation rates and electrical classifications to be used for safety. While the unremodeled portions of existing plants are not required to be upgraded to NFPA 820 standards, all new or significantly remodeled portions of the plant would need to be upgraded to meet the standard. For those areas not requiring remodeling, NFPA 820 does represent a good set of recommendations for possible upgrades to improve system safety.

The existing chlorination facility may be in need of safety improvements, especially since the facility is in close proximity to neighborhoods. Recent revisions to the Uniform Fire Code, Article 80, require emergency chemical scrubbing equipment for toxic compressible gases. A sprinkler system is also required if the building is constructed of flammable materials. The local fire marshall may require these improvements if the facility is upgraded. Adding dechlorination may also trigger upgrading. Other regulations that may affect the facility upgrade are the Disabilities Act and OSHA requirements.

Most of the structures at the plant were designed and built before the January 1, 1993, effective date of the Oregon Structural Specialty Code (OSSC) that upgrades western Oregon to Uniform Building Code Seismic Zone 3. Previously, this same vicinity had been designated as Zone 2 or 2B with code forces two-thirds as large, and much less stringent regulations for connection details. Therefore, we would not expect the details of reinforcement and connections for these older structures to conform to current requirements of the building code for regions of high seismic risk (a structural inspection of the buildings

and/or plans is not within the scope of this report). If an earthquake of a magnitude higher than those that occurred in Woodburn in 1993 were to occur in the Woodburn area, we would expect many of these structures to suffer major structural damage.

Existing Wastewater Collection System

System Description

The City's existing sanitary wastewater collection system collects wastewater from residences, businesses, industries, and public facilities and conveys it to the Woodburn WWTP near the Pudding River. For the most part, the existing wastewater collection system was built in the 1950's and conforms to the topography of the Mill Creek basin. Nine pump stations pump wastewater from areas that cannot flow by gravity to the treatment plant because of the flat topography and low-lying sections at the extremities of the collection system.

Approximately 2,087 acres within the UGB are currently sewered. The total length of municipal sewer, excluding private service laterals, is approximately 52 miles and pipe sizes range in diameter from 8 to 36 inches.

Two major interceptors comprise the majority of the City's gravity collection system:

- The Mill Creek interceptor follows Mill Creek from the Mill Creek pump station south to Cleveland Street, then west to Settlemier Avenue. It intercepts flow from smaller collector systems both east and west of Mill Creek.

- The Front Street interceptor serves areas west of Front Street by collecting main and lateral lines south to Cleveland Street and west to the Interstate I-5 freeway.

The existing collection system occasionally exceeds its capacity. The Mill Creek pump station and the Mill Creek interceptor are particularly troublesome areas. According to the collection system analysis (see Volume II of this report), peak flow resulting from a 5-year storm event will exceed the capacity of the existing collection system. Pump station and pipeline improvements currently required include:

- An upgrade of the Mill Creek Pump Station

- Hydraulic relief for the Front Street Interceptor through the downtown area to Lincoln Street

- Hydraulic relief for the trunkline along Highway 214 and Astor Way serving the north part of town

- Monitoring of four additional pump stations to determine whether they should be upgraded in the immediate future:

- Pump Station 1 near Vanderbeck Way
- Pump Station 2 near Jansen Way
- Pump Station 3 near Rainer Road
- Pump Station 4 near Industrial Avenue

Infiltration/Inflow Evaluation

Introduction

Wastewater collection systems, although constructed to collect and convey wastewater, also inevitably convey a certain quantity of extraneous clear water. This water, commonly referred to as infiltration/inflow (I/I), can originate as groundwater or surface runoff. I/I includes water that leaks into the system from cracks (infiltration) or cross connections (inflow). The entry of groundwater and stormwater runoff into the wastewater collection system increases the cost of operating the wastewater conveyance and treatment facilities. Excessive I/I is defined as the quantity of I/I that can be economically eliminated from a sewer system as determined in a cost effective analysis. This analysis compares the cost of sewer rehabilitation plus transporting and treating the remaining I/I to the total cost for transportation and treatment of all the I/I.

Regulatory Requirements

EPA facilities planning guidelines include criteria for evaluating excessive I/I. A target level of 120 gallons per capita per day (gpcpd) has been established to assess impacts of groundwater infiltration. If per capita flow levels for a 7- to 14-day period of high groundwater are higher than 120 gpcpd, excessive groundwater infiltration may exist. A similar target level of 275 gpcpd has been established for peak I/I conditions. This target reflects the impact of inflow caused by a storm event. If either or both of these targets are exceeded, an I/I cost effective analysis is warranted.

Excessive I/I Analysis

An analysis to determine whether excessive I/I is present in the Woodburn collection system was performed. The 1993 wet season flow records were used to develop the domestic wastewater flow in gallons per capita per day (gpcpd). The City began correcting identified inflow sources before the 1993 wet season.

Woodburn has significant industrial wastewater flows. These flows were subtracted from the total plant flows to determine the domestic and I/I flows. Industrial permits were used to determine the total industrial flow.

The first column in Table 2-13 represents a period of high groundwater. The flow is for the month of February 1993. The second column represents peak I/I conditions during a storm event. The peak I/I event from Table 2-6 is 7.92 mgd (WWMDF). The per capita flow for groundwater infiltration is less than the EPA target values. The per capita flow for peak I/I conditions exceeds the EPA value; therefore, additional investigation of inflow is necessary. In a separate evaluation, approximately 2 mgd of inflow has been identified as being cost

effective to remove, as discussed in *Volume II—Collection System Evaluation, Wastewater Facilities Plan, City of Woodburn, Oregon.*

Table 2-13
Infiltration and Inflow Analysis
City of Woodburn
1993

	Units	*High Ground-water*	*Peak I/I*
Population Equivalent	Est. 1/1/93	20,401	20,401
Flow Period	1993	Feb-93	Mar-93
Total Flow	gpd	2.3	7.92
Industrial Flow	mgd	0.13	0.13
Sanitary and I/I Flow	mgd	2.17	7.79
Per Capita Sanitary & I/I Flow	gpcpd	106	382
EPA Target Values	gpcpd	120	275

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Appendix 2.1

Historical Woodburn WWTP Flow and Load Data

Table 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993

Date	*Daily Influent Flow (mgd)*	*Daily Effluent Flow (mgd)*	*Flow Difference (mgd)*	*Percent Difference (%)*
30-Jul-91	1.733	0.807	0.926	53%
31-Jul-91	1.758	0.948	0.810	46%
1-Aug-91	1.758	0.948	0.810	46%
2-Aug-91	1.739	0.408	1.331	77%
3-Aug-91	1.612	0.775	0.837	52%
4-Aug-91	1.777	0.654	1.123	63%
5-Aug-91	1.796	0.524	1.272	71%
6-Aug-91	1.509	0.512	0.997	66%
7-Aug-91	1.788	0.500	1.288	72%
8-Aug-91	1.692	0.587	1.105	65%
9-Aug-91	1.743	0.477	1.266	73%
10-Aug-91	1.829	0.486	1.343	73%
11-Aug-91	1.622	0.533	1.089	67%
12-Aug-91	1.682	0.665	1.017	60%
13-Aug-91	1.549	0.583	0.966	62%
14-Aug-91	1.621	0.564	1.057	65%
15-Aug-91	1.628	1.649	-0.021	-1%
16-Aug-91	1.658	1.670	-0.012	-1%
17-Aug-91	1.621	1.561	0.060	4%
18-Aug-91	1.608	1.689	-0.081	-5%
19-Aug-91	1.322	1.556	-0.234	-18%
20-Aug-91	1.614	1.690	-0.076	-5%
21-Aug-91	1.711	1.663	0.048	3%
22-Aug-91	1.711	1.622	0.089	5%
23-Aug-91	1.616	1.491	0.125	8%
24-Aug-91	1.370	1.439	-0.069	-5%
25-Aug-91	1.543	1.329	0.214	14%
26-Aug-91	1.485	1.272	0.213	14%
27-Aug-91	1.578	1.507	0.071	4%
28-Aug-91	1.580	1.481	0.099	6%
29-Aug-91	1.570	1.616	-0.046	-3%
30-Aug-91	1.566	1.478	0.088	6%
31-Aug-91	1.470	1.350	0.120	8%
1-Sep-91	1.268	1.261	0.007	1%
2-Sep-91	1.255	1.128	0.127	10%
3-Sep-91	1.562	1.416	0.146	9%
4-Sep-91	1.690	2.223	-0.533	-32%
5-Sep-91	1.523	2.188	-0.665	-44%
6-Sep-91	1.688	2.214	-0.526	-31%
7-Sep-91	1.670	2.219	-0.549	-33%
8-Sep-91	1.445	2.076	-0.631	-44%
9-Sep-91	1.410	1.428	-0.018	-1%
10-Sep-91	1.625	2.676	-1.051	-65%
11-Sep-91	1.605	1.563	0.042	3%
12-Sep-91	1.495	1.471	0.024	2%
13-Sep-91	1.658	1.735	-0.077	-5%
14-Sep-91	1.632	1.588	0.044	3%
15-Sep-91	1.350	1.490	-0.140	-10%
16-Sep-91	1.279	1.328	-0.049	-4%
17-Sep-91	1.671	1.715	-0.044	-3%
18-Sep-91	1.598	1.690	-0.092	-6%
19-Sep-91	1.622	1.691	-0.069	-4%
20-Sep-91	1.507	1.649	-0.142	-9%
21-Sep-91	1.467	1.598	-0.131	-9%
22-Sep-91	1.358	1.440	-0.082	-6%

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Table 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993

Date	*Daily Influent Flow (mgd)*	*Daily Effluent Flow (mgd)*	*Flow Difference (mgd)*	*Percent Difference (%)*
23-Sep-91	1.350	1.464	-0.114	-8%
24-Sep-91	1.620	1.707	-0.087	-5%
25-Sep-91	1.577	1.646	-0.069	-4%
26-Sep-91	1.524	1.699	-0.175	-11%
27-Sep-91	1.609	1.661	-0.052	-3%
28-Sep-91	1.607	1.666	-0.059	-4%
29-Sep-91	1.574	1.633	-0.059	-4%
30-Sep-91	1.346	1.522	-0.176	-13%
1-Oct-91	1.665	1.686	-0.021	-1%
2-Oct-91	1.645	1.623	0.022	1%
3-Oct-91	1.727	1.610	0.117	7%
4-Oct-91	1.585	1.624	-0.039	-2%
5-Oct-91	1.724	1.617	0.107	6%
6-Oct-91	1.385	1.520	-0.135	-10%
7-Oct-91	1.630	1.453	0.177	11%
8-Oct-91	1.601	1.586	0.015	1%
9-Oct-91	1.660	1.512	0.148	9%
10-Oct-91	1.751	1.671	0.080	5%
11-Oct-91	1.579	1.634	-0.055	-3%
12-Oct-91	1.463	1.676	-0.213	-15%
13-Oct-91	1.571	1.725	-0.154	-10%
14-Oct-91	1.588	1.675	-0.087	-5%
15-Oct-91	1.735	1.818	-0.083	-5%
16-Oct-91	1.804	1.943	-0.139	-8%
17-Oct-91	1.958	1.886	0.072	4%
18-Oct-91	1.471	1.791	-0.320	-22%
19-Oct-91	1.383	1.613	-0.230	-17%
20-Oct-91	1.371	1.583	-0.212	-15%
21-Oct-91	1.437	1.563	-0.126	-9%
22-Oct-91	1.477	1.604	-0.127	-9%
23-Oct-91	1.537	1.777	-0.240	-16%
24-Oct-91	1.595	1.597	-0.002	0%
25-Oct-91	1.561	1.825	-0.264	-17%
26-Oct-91	2.095	2.317	-0.222	-11%
27-Oct-91	2.017	1.822	0.195	10%
28-Oct-91	1.624	1.514	0.110	7%
29-Oct-91	1.524	1.688	-0.164	-11%
30-Oct-91	1.524	1.458	0.066	4%
31-Oct-91	1.307	1.469	-0.162	-12%
1-Nov-91	1.407	1.540	-0.133	-9%
2-Nov-91	1.383	1.511	-0.128	-9%
3-Nov-91	1.300	1.488	-0.188	-14%
4-Nov-91	1.470	1.405	0.065	4%
5-Nov-91	1.967	2.001	-0.034	-2%
6-Nov-91	2.030	2.205	-0.175	-9%
7-Nov-91	1.545	1.696	-0.151	-10%
8-Nov-91	1.624	1.742	-0.118	-7%
9-Nov-91	1.488	1.669	-0.181	-12%
10-Nov-91	1.392	1.511	-0.119	-9%
11-Nov-91	1.436	1.369	0.067	5%
12-Nov-91	1.758	1.714	0.044	3%
13-Nov-91	1.677	1.718	-0.041	-2%
14-Nov-91	1.862	1.841	0.021	1%
15-Nov-91	1.596	2.082	-0.486	-30%
16-Nov-91	1.620	2.231	-0.611	-38%

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Table 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993

Date	*Daily Influent Flow (mgd)*	*Daily Effluent Flow (mgd)*	*Flow Difference (mgd)*	*Percent Difference (%)*
17-Nov-91	2.096	2.759	-0.663	-32%
18-Nov-91	2.162	2.915	-0.753	-35%
19-Nov-91	1.929	2.552	-0.623	-32%
20-Nov-91	2.348	3.021	-0.673	-29%
21-Nov-91	2.237	2.911	-0.674	-30%
22-Nov-91	2.035	2.706	-0.671	-33%
23-Nov-91	1.739	2.280	-0.541	-31%
24-Nov-91	2.021	2.551	-0.530	-26%
25-Nov-91	2.090	2.678	-0.588	-28%
26-Nov-91	2.158	2.863	-0.705	-33%
27-Nov-91	2.599	2.140	0.459	18%
28-Nov-91	1.908	1.772	0.136	7%
29-Nov-91	1.501	1.772	-0.271	-18%
30-Nov-91	1.329	1.529	-0.200	-15%
1-Dec-91	1.441	1.566	-0.125	-9%
2-Dec-91	1.577	1.996	-0.419	-27%
3-Dec-91	1.696	2.175	-0.479	-28%
6-Dec-91	3.997	4.457	-0.460	-12%
7-Dec-91	2.808	2.857	-0.049	-2%
8-Dec-91	2.351	2.259	0.092	4%
9-Dec-91	2.236	2.486	-0.250	-11%
10-Dec-91	2.099	2.341	-0.242	-12%
11-Dec-91	1.953	3.105	-1.152	-59%
12-Dec-91	1.837	2.404	-0.567	-31%
13-Dec-91	1.858	2.163	-0.305	-16%
14-Dec-91	1.823	1.774	0.049	3%
15-Dec-91	1.738	1.659	0.079	5%
16-Dec-91	1.656	1.933	-0.277	-17%
17-Dec-91	1.873	1.875	-0.002	0%
18-Dec-91	3.074	2.970	0.104	3%
19-Dec-91	2.513	2.459	0.054	2%
20-Dec-91	2.487	2.536	-0.049	-2%
21-Dec-91	3.643	3.647	-0.004	0%
22-Dec-91	2.848	2.863	-0.015	-1%
23-Dec-91	2.450	2.626	-0.176	-7%
24-Dec-91	2.461	2.670	-0.209	-8%
25-Dec-91	2.258	2.201	0.057	3%
26-Dec-91	2.192	2.397	-0.205	-9%
27-Dec-91	2.205	2.299	-0.094	-4%
28-Dec-91	2.158	2.003	0.155	7%
29-Dec-91	2.816	1.864	0.952	34%
30-Dec-91	2.045	1.932	0.113	6%
31-Dec-91	2.342	1.780	0.562	24%
1-Jan-92	1.728	1.775	-0.047	-3%
2-Jan-92	2.063	2.274	-0.211	-10%
3-Jan-92	2.247	2.550	-0.303	-13%
4-Jan-92	1.536	2.398	-0.862	-56%
5-Jan-92	2.190	2.340	-0.150	-7%
6-Jan-92	2.273	2.252	0.021	1%
7-Jan-92	2.072	2.119	-0.047	-2%
8-Jan-92	2.042	2.040	0.002	0%
10-Jan-92	2.393	2.439	-0.046	-2%
11-Jan-92	2.281	2.296	-0.015	-1%
12-Jan-92	2.595	2.076	0.519	20%
13-Jan-92	3.035	2.301	0.734	24%

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Table 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993

Date	*Daily Influent Flow (mgd)*	*Daily Effluent Flow (mgd)*	*Flow Difference (mgd)*	*Percent Difference (%)*
14-Jan-92	3.035	2.233	0.802	26%
15-Jan-92	4.041	2.215	1.826	45%
16-Jan-92	2.736	3.676	-0.940	-34%
17-Jan-92	2.557	2.515	0.042	2%
18-Jan-92	2.412	2.249	0.163	7%
19-Jan-92	2.329	2.117	0.212	9%
20-Jan-92	2.368	2.314	0.054	2%
21-Jan-92	2.337	2.190	0.147	6%
22-Jan-92	2.194	2.231	-0.037	-2%
23-Jan-92	2.283	2.291	-0.008	0%
24-Jan-92	2.304	2.366	-0.062	-3%
25-Jan-92	2.334	2.389	-0.055	-2%
26-Jan-92	3.602	2.275	1.327	37%
27-Jan-92	4.810	3.379	1.431	30%
28-Jan-92	3.401	3.600	-0.199	-6%
29-Jan-92	4.188	3.262	0.926	22%
30-Jan-92	4.480	4.188	0.292	7%
31-Jan-92	3.611	3.900	-0.289	-8%
11-Jun-92	1.835	1.131	0.704	38%
12-Jun-92	2.060	0.858	1.202	58%
13-Jun-92	1.893	1.016	0.877	46%
16-Jun-92	1.518	0.980	0.538	35%
17-Jun-92	1.636	0.846	0.790	48%
18-Jun-92	1.643	0.917	0.726	44%
19-Jun-92	1.646	0.584	1.062	65%
20-Jun-92	1.624	0.651	0.973	60%
21-Jun-92	1.609	0.523	1.086	67%
22-Jun-92	1.766	0.627	1.139	64%
23-Jun-92	1.662	0.721	0.941	57%
24-Jun-92	1.613	0.973	0.640	40%
25-Jun-92	1.291	0.795	0.496	38%
26-Jun-92	1.471	0.764	0.707	48%
27-Jun-92	1.446	0.857	0.589	41%
28-Jun-92	1.579	0.792	0.787	50%
29-Jun-92	1.699	0.877	0.822	48%
30-Jun-92	1.757	0.990	0.767	44%
1-Jul-92	2.258	0.798	1.460	65%
2-Jul-92	2.351	0.807	1.544	66%
3-Jul-92	2.271	0.895	1.376	61%
4-Jul-92	2.736	0.819	1.917	70%
5-Jul-92	2.798	0.794	2.004	72%
6-Jul-92	1.608	0.757	0.851	53%
7-Jul-92	1.581	0.665	0.916	58%
8-Jul-92	1.627	0.828	0.799	49%
9-Jul-92	1.757	0.583	1.174	67%
10-Jul-92	1.617	0.598	1.019	63%
11-Jul-92	2.818	0.600	2.218	79%
12-Jul-92	3.116	0.504	2.612	84%
13-Jul-92	1.582	0.555	1.027	65%
14-Jul-92	1.615	0.569	1.046	65%
15-Jul-92	1.577	0.633	0.944	60%
16-Jul-92	1.549	0.526	1.023	66%
17-Jul-92	1.396	0.979	0.417	30%
18-Jul-92	1.220	0.955	0.265	22%
19-Jul-92	1.225	1.100	0.125	10%

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Table 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993

Date	Daily Influent Flow (mgd)	Daily Effluent Flow (mgd)	Flow Difference (mgd)	Percent Difference (%)
20-Jul-92	1.823	1.267	0.556	30%
21-Jul-92	1.872	1.124	0.748	40%
22-Jul-92	1.940	1.998	-0.058	-3%
23-Jul-92	1.451	2.048	-0.597	-41%
24-Jul-92	1.651	2.025	-0.374	-23%
25-Jul-92	1.735	1.887	-0.152	-9%
26-Jul-92	1.860	1.903	-0.043	-2%
27-Jul-92	1.811	2.061	-0.250	-14%
28-Jul-92	1.687	2.089	-0.402	-24%
29-Jul-92	1.362	2.056	-0.694	-51%
30-Jul-92	1.410	1.958	-0.548	-39%
31-Jul-92	1.279	1.903	-0.624	-49%
1-Aug-92	1.172	1.762	-0.590	-50%
2-Aug-92	1.261	1.757	-0.496	-39%
3-Aug-92	1.513	1.641	-0.128	-8%
4-Aug-92	1.417	1.563	-0.146	-10%
5-Aug-92	1.479	1.614	-0.135	-9%
6-Aug-92	1.535	1.721	-0.186	-12%
7-Aug-92	1.422	1.587	-0.165	-12%
8-Aug-92	1.222	1.373	-0.151	-12%
9-Aug-92	1.338	1.377	-0.039	-3%
10-Aug-92	1.431	1.677	-0.246	-17%
11-Aug-92	1.412	1.562	-0.150	-11%
12-Aug-92	1.399	1.662	-0.263	-19%
13-Aug-92	1.509	1.760	-0.251	-17%
14-Aug-92	1.273	1.504	-0.231	-18%
15-Aug-92	1.209	1.432	-0.223	-18%
16-Aug-92	1.955	1.337	0.618	32%
17-Aug-92	1.363	1.682	-0.319	-23%
18-Aug-92	2.291	1.605	0.686	30%
19-Aug-92	2.029	1.613	0.416	21%
20-Aug-92	1.400	1.621	-0.221	-16%
21-Aug-92	1.271	1.513	-0.242	-19%
22-Aug-92	1.033	1.334	-0.301	-29%
23-Aug-92	1.246	1.338	-0.092	-7%
24-Aug-92	1.507	1.507	0.000	0%
25-Aug-92	1.547	1.547	0.000	0%
26-Aug-92	1.627	1.627	0.000	0%
27-Aug-92	1.665	1.665	0.000	0%
28-Aug-92	1.382	1.382	0.000	0%
29-Aug-92	1.430	1.430	0.000	0%
30-Aug-92	1.369	1.369	0.000	0%
31-Aug-92	1.583	1.583	0.000	0%
1-Nov-92	1.757	1.410	0.347	20%
2-Nov-92	1.676	0.926	0.750	45%
3-Nov-92	1.554	1.540	0.014	1%
4-Nov-92	1.458	1.645	-0.187	-13%
5-Nov-92	1.526	1.427	0.099	6%
6-Nov-92	1.522	1.553	-0.031	-2%
7-Nov-92	1.498	1.610	-0.112	-7%
8-Nov-92	1.523	1.454	0.069	5%
9-Nov-92	1.441	1.461	-0.020	-1%
10-Nov-92	1.619	1.342	0.277	17%
11-Nov-92	1.508	1.481	0.027	2%
12-Nov-92	1.409	1.215	0.194	14%

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Table 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993

Date	*Daily Influent Flow (mgd)*	*Daily Effluent Flow (mgd)*	*Flow Difference (mgd)*	*Percent Difference (%)*
13-Nov-92	1.363	1.128	0.235	17%
14-Nov-92	1.420	1.161	0.259	18%
15-Nov-92	1.429	1.394	0.035	2%
16-Nov-92	1.415	1.320	0.095	7%
17-Nov-92	1.323	1.351	-0.028	-2%
18-Nov-92	1.663	1.541	0.122	7%
19-Nov-92	1.109	1.349	-0.240	-22%
20-Nov-92	2.082	1.569	0.513	25%
21-Nov-92	2.464	2.398	0.066	3%
22-Nov-92	1.574	1.473	0.101	6%
23-Nov-92	1.455	1.180	0.275	19%
24-Nov-92	1.426	1.211	0.215	15%
25-Nov-92	1.369	1.250	0.119	9%
26-Nov-92	1.327	1.194	0.133	10%
27-Nov-92	1.478	1.309	0.169	11%
28-Nov-92	1.217	1.138	0.079	6%
29-Nov-92	1.542	1.332	0.210	14%
30-Nov-92	1.959	1.819	0.140	7%
1-Dec-92	2.507	2.263	0.244	10%
2-Dec-92	1.576	1.384	0.192	12%
3-Dec-92	1.654	1.496	0.158	10%
4-Dec-92	1.467	1.309	0.158	11%
5-Dec-92	1.629	1.404	0.225	14%
6-Dec-92	1.500	1.235	0.265	18%
7-Dec-92	1.660	1.390	0.270	16%
8-Dec-92	1.983	1.819	0.164	8%
9-Dec-92	2.887	2.588	0.299	10%
10-Dec-92	2.867	1.727	1.140	40%
11-Dec-92	1.804	1.727	0.077	4%
12-Dec-92	1.762	1.682	0.080	5%
13-Dec-92	1.671	1.473	0.198	12%
14-Dec-92	1.779	1.665	0.114	6%
15-Dec-92	1.715	1.459	0.256	15%
16-Dec-92	2.055	1.736	0.319	16%
17-Dec-92	2.219	1.959	0.260	12%
18-Dec-92	1.839	1.642	0.197	11%
19-Dec-92	1.899	1.403	0.496	26%
20-Dec-92	1.798	0.225	1.573	87%
21-Dec-92	1.870	1.827	0.043	2%
22-Dec-92	1.757	1.483	0.274	16%
23-Dec-92	1.693	2.047	-0.354	-21%
24-Dec-92	1.472	2.365	-0.893	-61%
25-Dec-92	1.633	2.642	-1.009	-62%
26-Dec-92	1.834	2.812	-0.978	-53%
27-Dec-92	1.857	2.851	-0.994	-54%
28-Dec-92	1.948	2.679	-0.731	-38%
29-Dec-92	1.816	1.794	0.022	1%
30-Dec-92	1.751	1.626	0.125	7%
31-Dec-92	1.813	1.681	0.132	7%
1-Jan-93	1.680	1.717	-0.037	-2%
2-Jan-93	1.643	1.320	0.323	20%
3-Jan-93	2.101	2.000	0.101	5%
4-Jan-93	2.439	2.246	0.193	8%
5-Jan-93	2.390	2.262	0.128	5%
6-Jan-93	2.182	2.090	0.092	4%

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Table 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993

Date	*Daily Influent Flow (mgd)*	*Daily Effluent Flow (mgd)*	*Flow Difference (mgd)*	*Percent Difference (%)*
7-Jan-93	2.020	1.808	0.212	10%
8-Jan-93	1.840	1.694	0.146	8%
9-Jan-93	1.868	1.725	0.143	8%
10-Jan-93	1.633	1.466	0.167	10%
11-Jan-93	1.737	1.612	0.125	7%
12-Jan-93	1.620	1.591	0.029	2%
13-Jan-93	1.650	1.571	0.079	5%
14-Jan-93	1.660	1.590	0.070	4%
15-Jan-93	1.720	1.683	0.037	2%
16-Jan-93	1.710	1.629	0.081	5%
17-Jan-93	1.648	1.566	0.082	5%
18-Jan-93	1.749	1.761	-0.012	-1%
19-Jan-93	3.053	2.886	0.167	5%
20-Jan-93	3.211	3.138	0.073	2%
21-Jan-93	2.894	2.815	0.079	3%
22-Jan-93	2.802	2.666	0.136	5%
23-Jan-93	2.651	2.519	0.132	5%
24-Jan-93	2.477	2.401	0.076	3%
25-Jan-93	2.276	2.420	-0.144	-6%
26-Jan-93	2.163	2.292	-0.129	-6%
27-Jan-93	2.212	2.216	-0.004	0%
28-Jan-93	2.128	2.211	-0.083	-4%
29-Jan-93	2.581	2.072	0.509	20%
30-Jan-93	1.997	2.020	-0.023	-1%
31-Jan-93	1.918	1.907	0.011	1%
1-Feb-93	1.957	2.811	-0.854	-44%
2-Feb-93	1.953	3.030	-1.077	-55%
3-Feb-93	1.116	2.911	-1.795	-161%
4-Feb-93	1.854	2.993	-1.139	-61%
5-Feb-93	1.776	2.926	-1.150	-65%
6-Feb-93	1.932	3.039	-1.107	-57%
7-Feb-93	1.632	2.944	-1.312	-80%
8-Feb-93	1.790	2.446	-0.656	-37%
9-Feb-93	1.704	2.039	-0.335	-20%
10-Feb-93	1.766	2.018	-0.252	-14%
11-Feb-93	1.679	2.073	-0.394	-23%
12-Feb-93	1.808	2.023	-0.215	-12%
13-Feb-93	1.675	1.839	-0.164	-10%
14-Feb-93	1.633	1.734	-0.101	-6%
15-Feb-93	1.919	1.832	0.087	5%
16-Feb-93	1.739	1.649	0.090	5%
17-Feb-93	1.675	1.667	0.008	0%
18-Feb-93	1.744	1.720	0.024	1%
19-Feb-93	1.780	1.725	0.055	3%
20-Feb-93	2.252	2.150	0.102	5%
21-Feb-93	2.300	2.161	0.139	6%
22-Feb-93	2.245	2.182	0.063	3%
23-Feb-93	2.089	2.056	0.033	2%
24-Feb-93	2.019	1.952	0.067	3%
25-Feb-93	1.811	1.952	-0.141	-8%
26-Feb-93	1.847	1.913	-0.066	-4%
27-Feb-93	1.676	1.720	-0.044	-3%
28-Feb-93	1.691	1.741	-0.050	-3%
1-Mar-93	2.305	2.197	0.108	5%
2-Mar-93	2.285	2.249	0.036	2%

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Table 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993

Date	*Daily Influent Flow (mgd)*	*Daily Effluent Flow (mgd)*	*Flow Difference (mgd)*	*Percent Difference (%)*
3-Mar-93	2.273	2.329	-0.056	-2%
4-Mar-93	2.307	2.327	-0.020	-1%
5-Mar-93	2.385	2.496	-0.111	-5%
6-Mar-93	1.948	2.043	-0.095	-5%
7-Mar-93	1.978	2.096	-0.118	-6%
8-Mar-93	1.954	2.221	-0.267	-14%
9-Mar-93	2.033	2.138	-0.105	-5%
10-Mar-93	2.042	2.074	-0.032	-2%
11-Mar-93	1.767	2.103	-0.336	-19%
12-Mar-93	1.890	1.980	-0.090	-5%
13-Mar-93	1.995	2.075	-0.080	-4%
14-Mar-93	2.258	2.210	0.048	2%
15-Mar-93	2.714	3.606	-0.892	-33%
16-Mar-93	3.466	3.334	0.132	4%
17-Mar-93	3.693	3.688	0.005	0%
18-Mar-93	3.137	3.093	0.044	1%
19-Mar-93	2.880	2.938	-0.058	-2%
20-Mar-93	2.750	2.872	-0.122	-4%
21-Mar-93	2.340	2.637	-0.297	-13%
22-Mar-93	4.054	4.063	-0.009	0%
23-Mar-93	3.963	3.920	0.043	1%
24-Mar-93	3.488	3.407	0.081	2%
25-Mar-93	3.030	3.045	-0.015	0%
26-Mar-93	2.618	2.870	-0.252	-10%
27-Mar-93	2.624	2.788	-0.164	-6%
28-Mar-93	2.515	2.659	-0.144	-6%
29-Mar-93	2.473	2.562	-0.089	-4%
30-Mar-93	2.457	2.650	-0.193	-8%
31-Mar-93	2.512	2.625	-0.113	-4%
1-Apr-93	2.452	2.588	-0.136	-6%
2-Apr-93	2.523	2.642	-0.119	-5%
3-Apr-93	3.172	3.268	-0.096	-3%
4-Apr-93	2.406	2.489	-0.083	-3%
5-Apr-93	2.535	2.711	-0.176	-7%
6-Apr-93	2.448	2.587	-0.139	-6%
7-Apr-93	2.354	2.628	-0.274	-12%
8-Apr-93	2.702	2.842	-0.140	-5%
9-Apr-93	3.258	3.256	0.002	0%
10-Apr-93	3.138	3.049	0.089	3%
11-Apr-93	2.949	2.897	0.052	2%
12-Apr-93	2.770	2.811	-0.041	-1%
13-Apr-93	2.759	2.694	0.065	2%
14-Apr-93	2.475	2.687	-0.212	-9%
15-Apr-93	2.754	2.818	-0.064	-2%
16-Apr-93	3.091	3.172	-0.081	-3%
17-Apr-93	3.480	3.531	-0.051	-1%
18-Apr-93	3.130	3.210	-0.080	-3%
19-Apr-93	2.483	3.049	-0.566	-23%
20-Apr-93	2.737	2.909	-0.172	-6%
21-Apr-93	2.665	2.858	-0.193	-7%
22-Apr-93	2.605	2.764	-0.159	-6%
23-Apr-93	2.779	2.968	-0.189	-7%
24-Apr-93	2.740	2.942	-0.202	-7%
25-Apr-93	3.017	3.075	-0.058	-2%
26-Apr-93	3.212	3.458	-0.246	-8%

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Table 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993

Date	*Daily Influent Flow (mgd)*	*Daily Effluent Flow (mgd)*	*Flow Difference (mgd)*	*Percent Difference (%)*
27-Apr-93	2.155	4.286	-2.131	-99%
28-Apr-93	2.787	4.492	-1.705	-61%
29-Apr-93	2.820	3.197	-0.377	-13%
30-Apr-93	2.610	2.961	-0.351	-13%
1-May-93	2.694	3.019	-0.325	-12%
2-May-93	2.222	2.739	-0.517	-23%
3-May-93	2.742	3.173	-0.431	-16%
4-May-93	2.618	2.757	-0.139	-5%
5-May-93	2.517	2.907	-0.390	-15%
6-May-93	2.690	3.595	-0.905	-34%
7-May-93	2.714	3.695	-0.981	-36%
8-May-93	2.547	3.600	-1.053	-41%
9-May-93	2.405	3.675	-1.270	-53%
10-May-93	2.597	3.500	-0.903	-35%
11-May-93	2.511	3.506	-0.995	-40%
12-May-93	2.427	3.367	-0.940	-39%
13-May-93	2.359	3.442	-1.083	-46%
14-May-93	2.263	3.236	-0.973	-43%
15-May-93	2.242	3.288	-1.046	-47%
16-May-93	2.235	3.276	-1.041	-47%
17-May-93	2.297	2.821	-0.524	-23%
18-May-93	2.170	2.432	-0.262	-12%
21-May-93	2.249	2.480	-0.231	-10%
22-May-93	2.275	2.538	-0.263	-12%
23-May-93	2.169	2.447	-0.278	-13%
24-May-93	2.330	2.581	-0.251	-11%
25-May-93	2.386	2.736	-0.350	-15%
26-May-93	2.267	2.518	-0.251	-11%
27-May-93	2.570	2.818	-0.248	-10%
28-May-93	2.363	2.579	-0.216	-9%
29-May-93	2.210	2.443	-0.233	-11%
30-May-93	2.364	2.647	-0.283	-12%
31-May-93	2.572	2.795	-0.223	-9%
1-Jun-93	3.102	3.245	-0.143	-5%
2-Jun-93	2.783	2.936	-0.153	-5%
3-Jun-93	2.653	2.931	-0.278	-10%
4-Jun-93	2.898	3.076	-0.178	-6%
5-Jun-93	3.331	3.352	-0.021	-1%
6-Jun-93	2.754	2.878	-0.124	-5%
7-Jun-93	2.652	2.820	-0.168	-6%
8-Jun-93	2.728	2.924	-0.196	-7%
9-Jun-93	2.651	2.814	-0.163	-6%
10-Jun-93	2.675	2.621	0.054	2%
11-Jun-93	2.658	2.575	0.083	3%
12-Jun-93	2.613	2.516	0.097	4%
13-Jun-93	2.451	2.390	0.061	2%
14-Jun-93	2.549	2.490	0.059	2%
15-Jun-93	2.442	2.449	-0.007	0%
16-Jun-93	2.312	2.380	-0.068	-3%
17-Jun-93	2.307	2.377	-0.070	-3%
18-Jun-93	2.297	2.359	-0.062	-3%
19-Jun-93	2.300	2.303	-0.003	0%
20-Jun-93	2.206	2.240	-0.034	-2%
21-Jun-93	2.240	2.299	-0.059	-3%
22-Jun-93	2.324	2.256	0.068	3%

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Table 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993

Date	*Daily Influent Flow (mgd)*	*Daily Effluent Flow (mgd)*	*Flow Difference (mgd)*	*Percent Difference (%)*
23-Jun-93	2.160	2.252	-0.092	-4%
24-Jun-93	2.240	2.324	-0.084	-4%
25-Jun-93	2.147	2.259	-0.112	-5%
26-Jun-93	2.012	2.064	-0.052	-3%
27-Jun-93	1.979	1.915	0.064	3%
28-Jun-93	2.016	1.949	0.067	3%
29-Jun-93	2.110	2.158	-0.048	-2%
30-Jun-93	2.073	2.134	-0.061	-3%
1-Jul-93	2.032	2.109	-0.077	-4%
2-Jul-93	1.949	2.042	-0.093	-5%
3-Jul-93	1.452	1.826	-0.374	-26%
4-Jul-93	1.618	1.674	-0.056	-3%
5-Jul-93	1.736	1.784	-0.048	-3%
6-Jul-93	1.854	1.845	0.009	0%
7-Jul-93	1.894	1.006	0.888	47%
8-Jul-93	1.916	1.676	0.240	13%
9-Jul-93	1.844	1.471	0.373	20%
10-Jul-93	1.864	1.497	0.367	20%
11-Jul-93	1.732	1.461	0.271	16%
12-Jul-93	1.810	1.486	0.324	18%
13-Jul-93	1.881	1.480	0.401	21%
14-Jul-93	1.909	1.495	0.414	22%
15-Jul-93	1.911	1.726	0.185	10%
16-Jul-93	2.111	1.840	0.271	13%
17-Jul-93	1.968	1.753	0.215	11%
18-Jul-93	1.824	1.620	0.204	11%
19-Jul-93	2.079	1.826	0.253	12%
20-Jul-93	1.872	1.895	-0.023	-1%
21-Jul-93	1.866	2.124	-0.258	-14%
22-Jul-93	1.834	2.206	-0.372	-20%
23-Jul-93	1.853	2.174	-0.321	-17%
24-Jul-93	1.788	2.107	-0.319	-18%
25-Jul-93	1.690	2.014	-0.324	-19%
26-Jul-93	1.825	2.070	-0.245	-13%
27-Jul-93	1.490	1.628	-0.138	-9%
28-Jul-93	1.900	1.674	0.226	12%
29-Jul-93	1.753	0.996	0.757	43%
30-Jul-93	1.829	0.886	0.943	52%
31-Jul-93	1.676	0.762	0.914	55%
1-Aug-93	1.606	0.815	0.791	49%
2-Aug-93	1.894	1.068	0.826	44%
3-Aug-93	1.881	1.292	0.589	31%
4-Aug-93	1.922	1.651	0.271	14%
5-Aug-93	1.952	1.140	0.812	42%
6-Aug-93	1.662	1.636	0.026	2%
7-Aug-93	1.552	1.523	0.029	2%
8-Aug-93	1.534	1.490	0.044	3%
9-Aug-93	1.695	1.613	0.082	5%
10-Aug-93	1.750	1.645	0.105	6%
11-Aug-93	1.766	1.172	0.594	34%
12-Aug-93	1.770	1.443	0.327	18%
13-Aug-93	1.561	1.481	0.080	5%
14-Aug-93	1.533	1.433	0.100	7%
15-Aug-93	1.602	1.458	0.144	9%
16-Aug-93	1.690	1.651	0.039	2%

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Table 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993

Date	*Daily Influent Flow (mgd)*	*Daily Effluent Flow (mgd)*	*Flow Difference (mgd)*	*Percent Difference (%)*
17-Aug-93	1.760	1.655	0.105	6%
18-Aug-93	1.699	1.738	-0.039	-2%
19-Aug-93	1.705	1.570	0.135	8%
20-Aug-93	1.754	1.625	0.129	7%
21-Aug-93	1.492	1.517	-0.025	-2%
22-Aug-93	1.539	1.447	0.092	6%
23-Aug-93	1.744	1.634	0.110	6%
24-Aug-93	1.657	1.568	0.089	5%
25-Aug-93	1.720	1.618	0.102	6%
26-Aug-93	1.661	1.628	0.033	2%
27-Aug-93	1.593	1.528	0.065	4%
28-Aug-93	1.526	1.450	0.076	5%
29-Aug-93	1.373	1.330	0.043	3%
30-Aug-93	1.534	1.547	-0.013	-1%
31-Aug-93	1.434	1.396	0.038	3%
1-Sep-93	1.528	1.342	0.186	12%
2-Sep-93	1.623	1.373	0.250	15%
3-Sep-93	1.439	1.245	0.194	13%
4-Sep-93	1.433	1.243	0.190	13%
5-Sep-93	1.176	1.001	0.175	15%
6-Sep-93	1.484	1.263	0.221	15%
7-Sep-93	1.656	1.370	0.286	17%
8-Sep-93	1.423	1.236	0.187	13%
9-Sep-93	1.515	1.340	0.175	12%
10-Sep-93	1.643	1.401	0.242	15%
11-Sep-93	1.571	1.348	0.223	14%
12-Sep-93	1.466	1.273	0.193	13%
13-Sep-93	1.553	1.351	0.202	13%
14-Sep-93	1.555	1.327	0.228	15%
15-Sep-93	1.530	1.348	0.182	12%
16-Sep-93	1.598	1.645	-0.047	-3%
17-Sep-93	1.546	1.236	0.310	20%
18-Sep-93	1.565	1.254	0.311	20%
19-Sep-93	1.477	1.154	0.323	22%
20-Sep-93	1.784	1.390	0.394	22%
21-Sep-93	1.601	1.245	0.356	22%
22-Sep-93	1.551	1.373	0.178	11%
23-Sep-93	1.528	1.225	0.303	20%
24-Sep-93	1.574	1.318	0.256	16%
25-Sep-93	1.545	2.188	-0.643	-42%
26-Sep-93	1.545	2.188	-0.643	-42%
27-Sep-93	1.417	4.116	-2.699	-190%
28-Sep-93	1.555	3.692	-2.137	-137%
29-Sep-93	1.533	2.585	-1.052	-69%
30-Sep-93	1.581	1.593	-0.012	-1%
1-Oct-93	1.460	1.561	-0.101	-7%
2-Oct-93	1.462	1.469	-0.007	0%
3-Oct-93	1.405	1.399	0.006	0%
4-Oct-93	1.499	1.493	0.006	0%
5-Oct-93	1.505	1.503	0.002	0%
6-Oct-93	1.464	1.473	-0.009	-1%
7-Oct-93	1.456	1.486	-0.030	-2%
8-Oct-93	1.569	1.492	0.077	5%
9-Oct-93	1.632	1.556	0.076	5%
10-Oct-93	1.496	1.422	0.074	5%

Table 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993

Date	*Daily Influent Flow (mgd)*	*Daily Effluent Flow (mgd)*	*Flow Difference (mgd)*	*Percent Difference (%)*
11-Oct-93	1.485	1.504	-0.019	-1%
12-Oct-93	1.445	1.525	-0.080	-6%
13-Oct-93	0.592	0.624	-0.032	-5%
14-Oct-93	1.327	1.216	0.111	8%
15-Oct-93	1.681	1.655	0.026	2%
16-Oct-93	1.715	1.640	0.075	4%
17-Oct-93	1.525	1.461	0.064	4%
18-Oct-93	1.582	1.530	0.052	3%
19-Oct-93	1.630	1.562	0.068	4%
20-Oct-93	1.517	1.505	0.012	1%
21-Oct-93	1.625	1.536	0.089	5%
22-Oct-93	1.600	1.527	0.073	5%
23-Oct-93	1.508	1.499	0.009	1%
24-Oct-93	1.421	0.900	0.521	37%
25-Oct-93	1.527	0.856	0.671	44%
26-Oct-93	1.508	1.310	0.198	13%
27-Oct-93	1.555	0.531	1.024	66%
28-Oct-93	1.483	1.359	0.124	8%
29-Oct-93	1.562	1.546	0.016	1%
30-Oct-93	1.606	1.415	0.191	12%
31-Oct-93	1.228	1.228	0.000	0%
1-Nov-93	1.609	1.254	0.355	22%
2-Nov-93	1.549	1.308	0.241	16%
3-Nov-93	1.527	1.312	0.215	14%
4-Nov-93	1.543	1.316	0.227	15%
5-Nov-93	1.540	1.320	0.220	14%
6-Nov-93	1.550	1.350	0.200	13%
7-Nov-93	1.484	1.269	0.215	14%
8-Nov-93	1.686	1.574	0.112	7%
9-Nov-93	1.720	1.582	0.138	8%
10-Nov-93	1.590	1.500	0.090	6%
11-Nov-93	1.577	1.475	0.102	6%
12-Nov-93	1.530	1.454	0.076	5%
13-Nov-93	1.562	1.478	0.084	5%
14-Nov-93	1.496	1.393	0.103	7%
15-Nov-93	1.581	1.479	0.102	6%
16-Nov-93	1.592	1.393	0.199	13%
17-Nov-93	1.623	1.435	0.188	12%
18-Nov-93	1.620	1.438	0.182	11%
19-Nov-93	1.576	1.484	0.092	6%
20-Nov-93	1.566	1.471	0.095	6%
21-Nov-93	1.515	1.454	0.061	4%
22-Nov-93	1.912	1.886	0.026	1%
23-Nov-93	1.670	1.659	0.011	1%
24-Nov-93	1.473	1.446	0.027	2%
25-Nov-93	1.369	1.367	0.002	0%
26-Nov-93	1.430	1.228	0.202	14%
27-Nov-93	1.396	1.244	0.152	11%
28-Nov-93	1.440	1.337	0.103	7%
29-Nov-93	1.627	1.523	0.104	6%
30-Nov-93	2.045	2.127	-0.082	-4%
1-Dec-93	2.211	1.279	0.932	42%
2-Dec-93	1.552	1.395	0.157	10%
3-Dec-93	1.788	1.727	0.061	3%
4-Dec-93	1.680	1.459	0.221	13%

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Table 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993

Date	*Daily Influent Flow (mgd)*	*Daily Effluent Flow (mgd)*	*Flow Difference (mgd)*	*Percent Difference (%)*
5-Dec-93	1.325	1.173	0.152	11%
6-Dec-93	1.758	1.528	0.230	13%
7-Dec-93	2.131	1.985	0.146	7%
8-Dec-93	2.733	2.506	0.227	8%
9-Dec-93	2.050	1.893	0.157	8%
10-Dec-93	3.011	2.723	0.288	10%
11-Dec-93	2.348	2.103	0.245	10%
12-Dec-93	1.807	1.644	0.163	9%
13-Dec-93	1.941	1.730	0.211	11%
14-Dec-93	1.946	1.730	0.216	11%
15-Dec-93	1.760	1.573	0.187	11%
16-Dec-93	1.824	1.636	0.188	10%
17-Dec-93	1.705	1.493	0.212	12%
18-Dec-93	1.625	1.405	0.220	14%
19-Dec-93	1.558	1.297	0.261	17%
20-Dec-93	1.531	1.332	0.199	13%
21-Dec-93	1.470	1.249	0.221	15%
22-Dec-93	1.413	1.216	0.197	14%
23-Dec-93	1.409	1.090	0.319	23%
24-Dec-93	1.441	1.183	0.258	18%
25-Dec-93	1.476	1.208	0.268	18%
26-Dec-93	1.163	1.002	0.161	14%
27-Dec-93	1.458	1.134	0.324	22%
28-Dec-93	1.468	1.198	0.270	18%
29-Dec-93	1.397	1.194	0.203	15%
30-Dec-93	1.516	1.369	0.147	10%
31-Dec-93	2.001	1.795	0.206	10%

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Table 2.1-2
Monthly Average Influent and Effluent Flow Comparison
Woodburn WWTP
August 1991-December 1993

Month-Year	*Influent Flow (mgd)*	*Effluent Flow (mgd)*	*Percent Difference*
Aug-91	1.625	1.107	32%
Sep-91	1.520	1.693	-11%
Oct-91	1.613	1.674	-4%
Nov-91	1.790	2.072	-16%
Dec-91	2.291	2.402	-5%
Jan-92	2.716	2.525	7%
Feb-92	(a)	3.528	(a)
Mar-92	(a)	2.317	(a)
Apr-92	(a)	2.158	(a)
May-92	(a)	1.819	(a)
Jun-92	(a)	1.083	(a)
Jul-92	1.825	1.170	36%
Aug-92	1.461	1.553	-6%
Sep-92	(a)	1.428	(a)
Oct-92	(a)	1.249	(a)
Nov-92	1.537	1.406	9%
Dec-92	1.862	1.839	1%
Jan-93	2.118	2.029	4%
Feb-93	1.824	2.187	-20%
Mar-93	2.585	2.687	-4%
Apr-93	2.767	3.028	-9%
May-93	2.258	2.968	-31%
Jun-93	2.455	2.510	-2%
Jul-93	1.831	1.682	8%
Aug-93	1.663	1.476	11%
Sep-93	1.533	1.589	-4%
Oct-93	1.486	1.380	7%
Nov-93	1.580	1.452	8%
Dec-93	1.758	1.524	13%

Notes:
(a) Influent flow meter out for repairs.

Table 2.1-3
Historical BOD_5 Loading
Woodburn WWTP
1990-1993

Month/ Year	*Average Influent BOD_5 (mg/L)*	*Average Influent BOD_5 (lb/day)*	*Maximum Influent BOD_5 (mg/L)*	*Maximum Influent BOD_5 (lb/day)*
Jan-90	164	3,235	274	4,337
Feb-90	114	3,168	182	4,352
Mar-90	136	2,742	192	3,535
Apr-90	175	2,807	195	3,339
May-90	183	3,161	210	5,030
Jun-90	200	3,145	292	4,105
Jul-90	357	5,030	432	6,021
Aug-90	241	3,198	288	4,263
Sep-90	233	3,889	257	4,739
Oct-90	226	3,437	288	3,952
Nov-90	229	3,104	303	3,894
Dec-90	211	2,702	238	3,261
Dry Wthr Avg/Max	240	3,643	432	6,021
Wet Wthr Avg/Max	172	2,960	303	4,352
1990 Average/Max	206	3,302	432	6,021
Jan-91	209	2,911	468	6,261
Feb-91	110	1,936	236	3,868
Mar-91	139	2,653	230	3,586
Apr-91	118	2,527	172	3,802
May-91	136	2,904	195	4,054
Jun-91	167	2,839	217	3,759
Jul-91	269	3,936	498	7,198
Aug-91	256	3,456	370	4,656
Sep-91	267	3,980	366	5,912
Oct-91	220	3,043	250	3,669
Nov-91	179	3,301	266	4,758
Dec-91	144	3,208	233	4,461
Dry Wthr Avg/Max	219	3,360	498	7,198
Wet Wthr Avg/Max	150	2,756	468	6,261
1991 Average/Max	185	3,058	498	7,198
Jan-92	172	3,569	214	4,966
Feb-92	120	3,881	169	10,028
Mar-92	168	3,269	207	4,188
Apr-92	190	3,418	270	4,227
May-92	252	3,877	290	4,276
Jun-92	333	4,441	535	7,840
Jul-92	391	3,021	623	10,546
Aug-92	282	3,653	372	5,460
Sep-92	293	3,653	389	4,519
Oct-92	357	3,653	471	4,675
Nov-92	310	3,460	441	5,664

Table 2.1-3
Historical BOD_5 Loading
Woodburn WWTP
1990-1993

Month/ Year	*Average Influent BOD_5 (mg/L)*	*Average Influent BOD_5 (lb/day)*	*Maximum Influent BOD_5 (mg/L)*	*Maximum Influent BOD_5 (lb/day)*
Dec-92	224	3,415	336	4,732
Dry Wthr Avg/Max	318	3,716	623	10,546
Wet Wthr Avg/Max	197	3,502	441	10,028
1992 Average/Max	258	3,609	623	10,546
Jan-93	220	3,947	278	6,691
Feb-93	253	4,604	338	6,440
Mar-93	176	3,769	286	5,016
Apr-93	144	3,482	184	4,361
May-93	177	4,301	228	5,913
Jun-93	192	3,986	270	4,859
Jul-93	330	4,999	480	5,965
Aug-93	241	3,427	295	4,168
Sep-93	267	3,507	342	4,602
Oct-93	328	8,486	855	10,575
Nov-93	329	2,520	411	7,419
Dec-93	276	4,321	410	5,776
Dry Wthr Avg/Max	256	4,784	855	10,575
Wet Wthr Avg/Max	233	3,774	411	7,419
1993 Average/Max	244	4,279	855	10,575
POR Dry Wthr Avg/Max	258	3,876	855	10,575
POR Wet Wthr Avg/Max	188	3,248	468	10,028
POR Average/Max	223	3,562	855	10,575
Dry Weather Summary				
1990	240	3,643	432	6,021
1991	219	3,360	498	7,198
1992	318	3,716	623	10,546
1993	256	4,784	855	10,575
POR Dry Avg/Max	258	3,876	855	10,575
Wet Weather Summary				
1990	172	2,960	303	4,352
1991	150	2,756	468	6,261
1992	197	3,502	441	10,028
1993	233	3,774	411	7,419
POR Wet Avg/Max	188	3,248	468	10,028

Notes:
POR = Period of Record.

Table 2.1-4
Historical TSS Loading
Woodburn WWTP
1990-1993

Month/ Year	*Average Influent TSS (mg/L)*	*Average Influent TSS (lb/day)*	*Maximum Influent TSS (mg/L)*	*Maximum Influent TSS (lb/day)*
Jan-90	124	2,449	226	3,709
Feb-90	97	2,738	146	4,271
Mar-90	109	2,173	172	2,768
Apr-90	172	2,762	261	4,469
May-90	159	2,881	252	3,747
Jun-90	146	2,315	172	3,015
Jul-90	186	2,556	227	3,731
Aug-90	195	2,566	287	3,736
Sep-90	187	3,073	210	3,652
Oct-90	177	2,739	232	3,835
Nov-90	160	2,213	204	2,758
Dec-90	169	2,190	308	4,184
Dry Wthr Avg/Max	175	2,688	287	3,835
Wet Wthr Avg/Max	139	2,421	308	4,469
1990 Average/Max	157	2,555	308	4,469
Jan-91	146	2,027	208	2,782
Feb-91	146	2,585	260	4,261
Mar-91	103	1,975	162	2,901
Apr-91	131	2,825	252	4,972
May-91	113	2,460	150	4,223
Jun-91	131	2,235	166	2,772
Jul-91	191	2,795	254	3,671
Aug-91	198	2,683	302	4,428
Sep-91	222	3,300	342	5,036
Oct-91	137	1,884	204	2,881
Nov-91	137	2,679	228	5,444
Dec-91	116	2,282	186	3,987
Dry Wthr Avg/Max	165	2,560	342	5,036
Wet Wthr Avg/Max	130	2,396	260	5,444
1991 Average/Max	148	2,478	342	5,444
Jan-92	109	2,290	140	4,292
Feb-92	97	3,172	144	9,025
Mar-92	123	2,423	156	3,449
Apr-92	142	2,521	194	3,037
May-92	188	2,887	242	3,552
Jun-92	223	2,916	490	7,499
Jul-92	239	3,574	324	6,101
Aug-92	240	3,283	317	4,653
Sep-92	237	2,929	373	4,987
Oct-92	305	3,103	454	5,165
Nov-92	138	1,535	230	2,574
Dec-92	142	2,196	218	4,518

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Table 2.1-4
Historical TSS Loading
Woodburn WWTP
1990-1993

Month/ Year	*Average Influent TSS (mg/L)*	*Average Influent TSS (lb/day)*	*Maximum Influent TSS (mg/L)*	*Maximum Influent TSS (lb/day)*
Dry Wthr Avg/Max	239	3,115	490	7,499
Wet Wthr Avg/Max	125	2,356	230	9,025
1992 Average/Max	182	2,736	490	9,025
Jan-93	165	3,025	272	6,306
Feb-93	184	3,335	230	4,397
Mar-93	164	3,518	286	5,550
Apr-93	109	2,616	168	3,348
May-93	126	3,116	174	5,217
Jun-93	143	2,888	259	4,873
Jul-93	188	2,865	244	3,669
Aug-93	133	1,892	223	3,273
Sep-93	129	1,704	217	2,924
Oct-93	119	1,499	195	2,385
Nov-93	200	2,701	248	3,786
Dec-93	180	2,479	229	3,118
Dry Wthr Avg/Max	140	2,327	259	5,217
Wet Wthr Avg/Max	167	2,946	286	6,306
1993 Average/Max	153	2,637	286	6,306
POR Dry Wthr Avg/Max	180	2,673	490	7,499
POR Wet Wthr Avg/Max	140	2,530	308	9,025
POR Average/Max	160	2,601	490	9,025
Dry Weather Summary				
1990	175	2,688	287	3,835
1991	165	2,560	342	5,036
1992	239	3,115	490	7,499
1993	140	2,327	259	5,217
POR Dry Avg/Max	180	2,673	490	7,499
Wet Weather Summary				
1990	139	2,421	308	4,469
1991	130	2,396	260	5,444
1992	125	2,356	230	9,025
1993	167	2,946	286	6,306
POR Dry Avg/Max	140	2,530	308	9,025

Notes:
POR = Period of Record.

Figure 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993

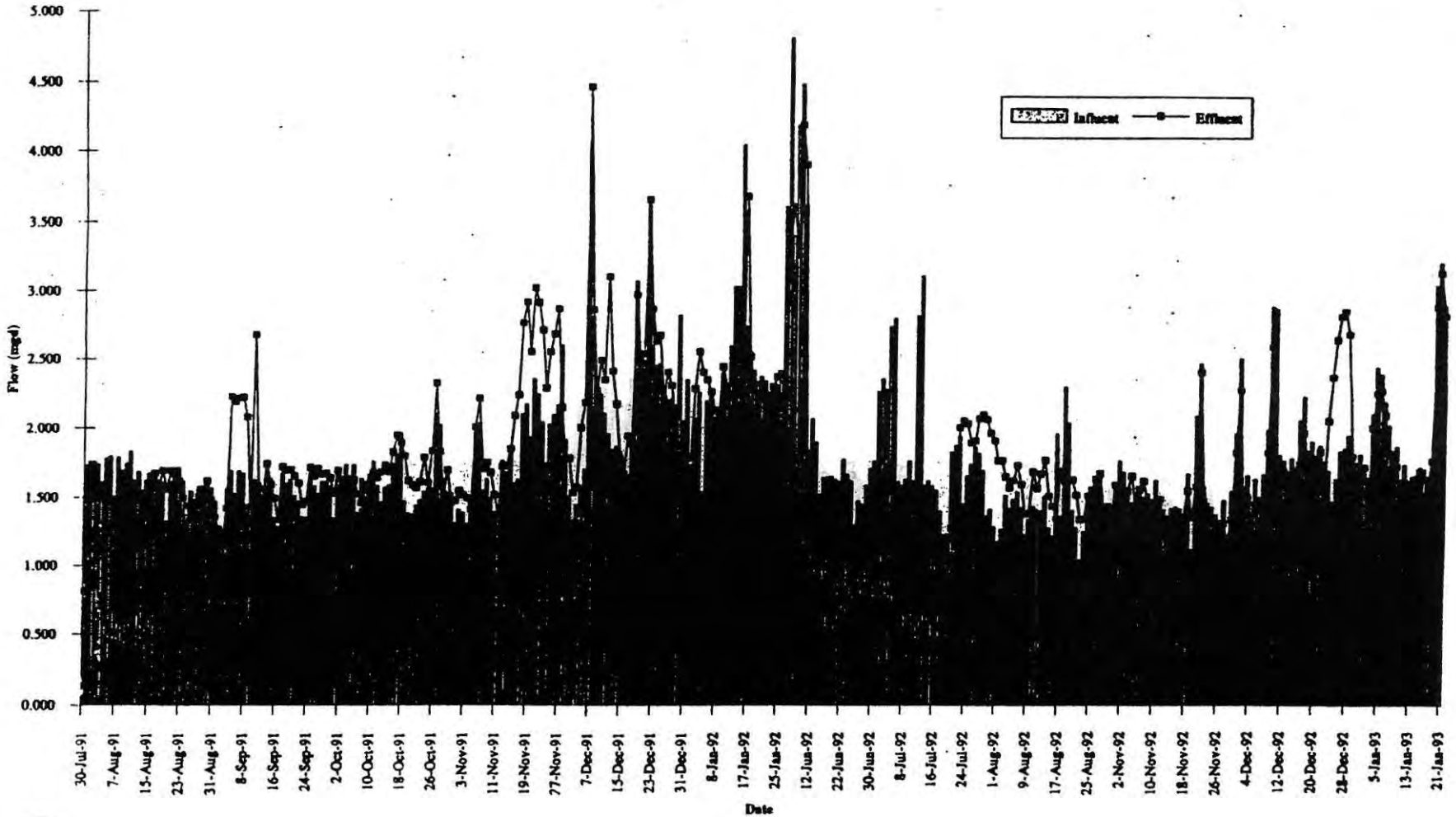

Figure 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993

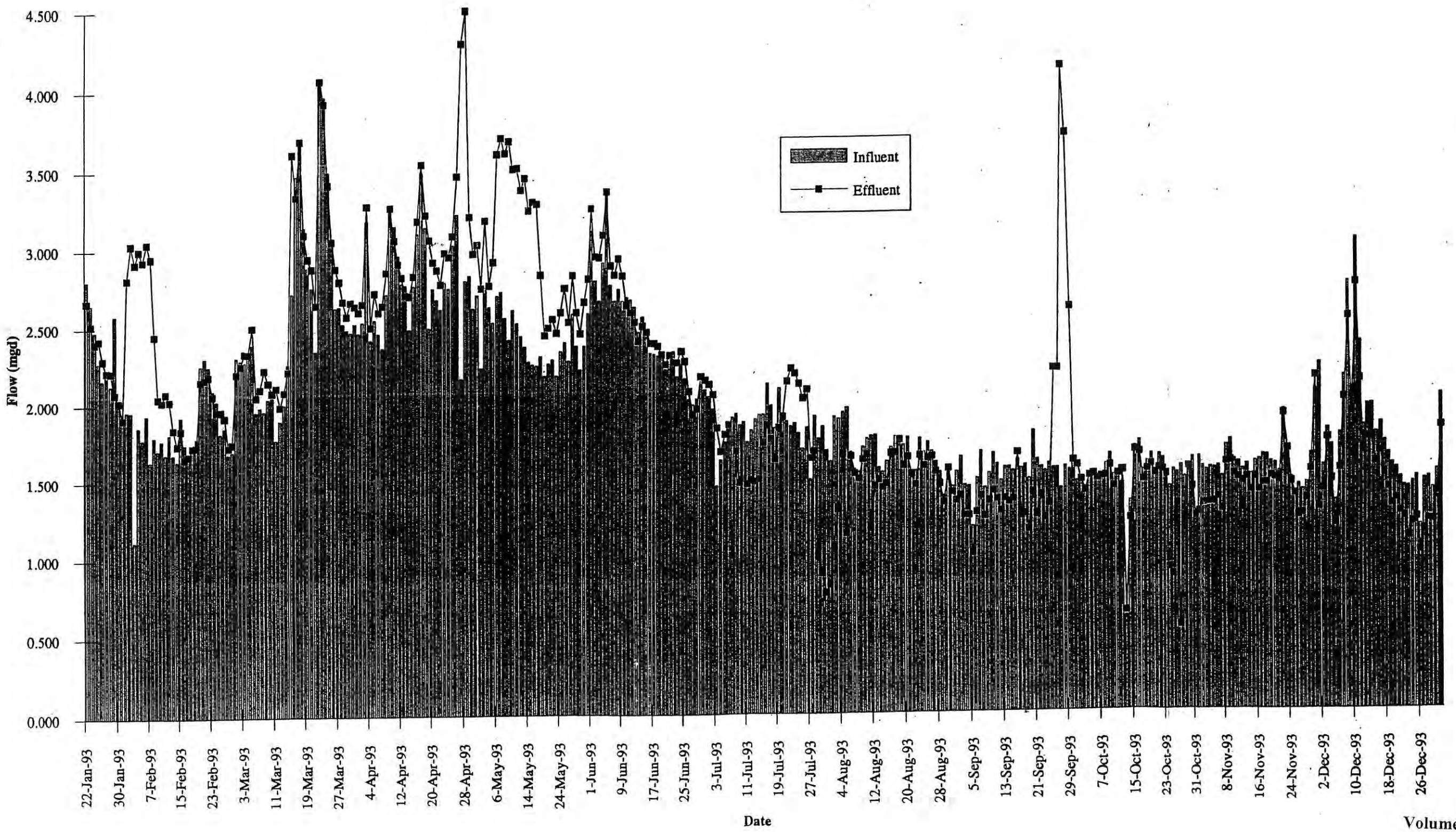

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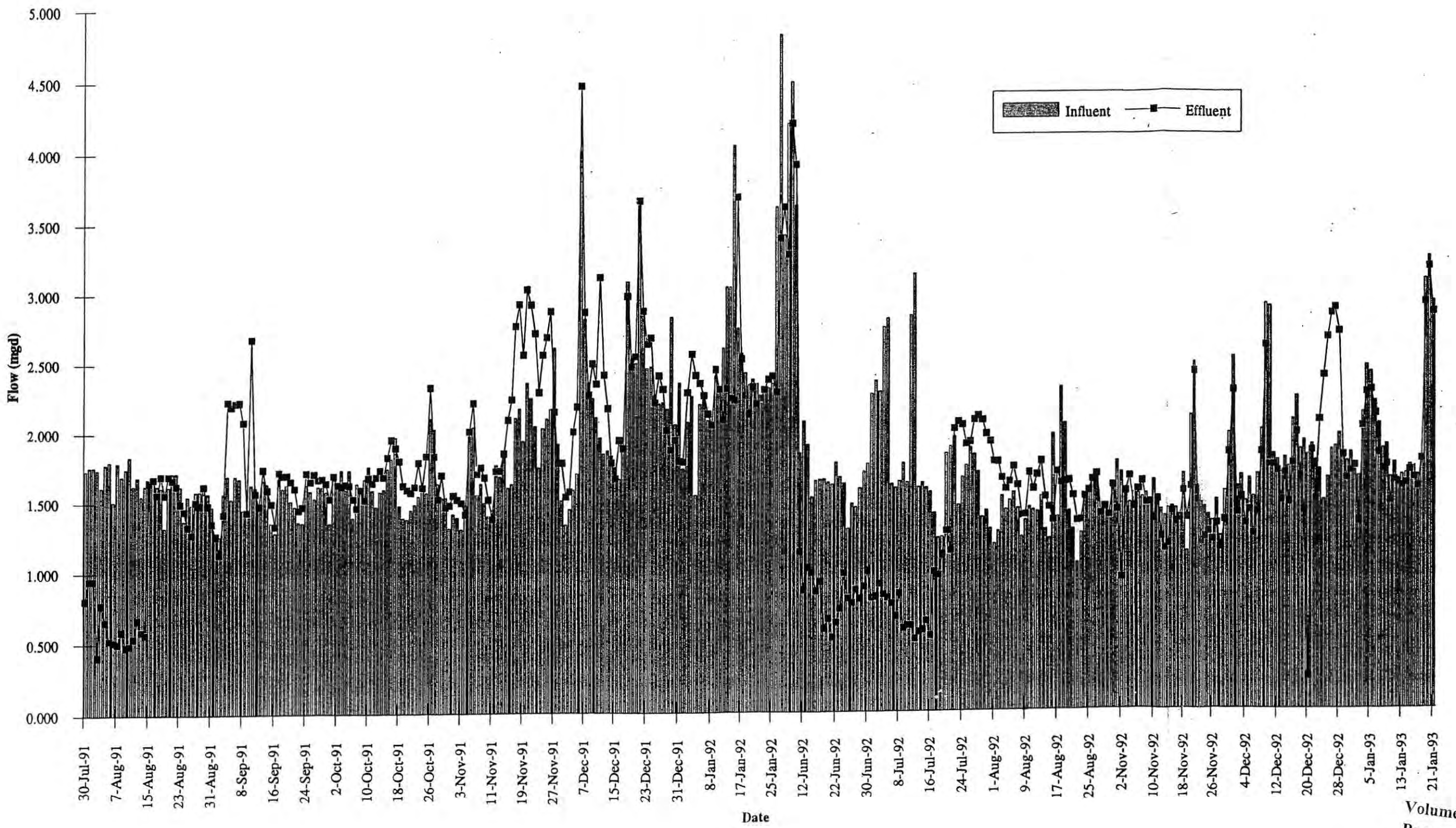
Figure 2.1-1
Daily Influent and Effluent Flow Comparison
Woodburn WWTP
1991-1993



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Appendix 2.2

Woodburn Outfall Memorandum

MEMORANDUM

CH2M HILL

TO: Daria Wightman/PDX

FROM: Roger Lindquist/CVO
Lorin Davis/CVO
Paula Mountjoy/PDX

DATE: February 6, 1995

SUBJECT: Woodburn Outfall

PROJECT: OPW27874.P2

Outfall Mixing Zone

The Pudding River in the vicinity of the Woodburn outfall is a crooked, meandering river. Low flows and velocities occur during the late summer and early fall. During this time of the year, the river is shallow with logs, roots and snags exposed. The low flow channel is about 25 feet wide and an average depth of about 1 foot, based on measurements made in August 1990 when the flow was about 20 cfs.

Two stream gaging stations have been operated by the USGS on the Pudding River. One gage is near Mt. Angel with 204 square miles of drainage and a 7Q10 of 8.7 cfs. The other gage is located at Aurora with 479 square miles of drainage area and a 7Q10 of 35 cfs. DEQ estimated a 7Q10 at the Woodburn equal to 15 cfs (Water Quality Report--Pudding River, Appendix C) which appears reasonable based on the existing gage data. A seven-day-average-low-flow of approximately 50 cfs occurs every other year in the Pudding River near Aurora (7Q2). Using the same ratio as the 7Q10, the approximate 7Q2 flow at Woodburn is 21 cfs (15 cfs at Woodburn : 35 cfs at Aurora = 21 cfs at Woodburn : 50 cfs at Aurora).

The existing 24-inch diameter outfall discharges horizontally through a 90 degree elbow pointed downstream. Based on the design drawings, this elbow is located near the center of the low flow channel and is surrounded by a riprap apron. The invert of the elbow is shown at elevation 92.4. Based on observations and photographs of the outfall elbow during low flow, it appears riprap or sediment is exposed on the left (north) side of the elbow and the top of the elbow is exposed above the water surface. The elbow appears to discharge into a narrow, shallow channel formed in the riprap/sediment, and then flows into the river. The effluent plume gradually moves out into the river channel and flows with the river current.

The plume configuration and dilution cannot be accurately estimated using available computer models. An approximation of the dilution and plume dimensions discharging directly into the river from the elbow can obtained by the PLUMES shallow river model. This model indicates the plume will encounter the river banks about 40 feet downstream. An approximation of the behavior of the plume discharging from the shallow channel can be obtained from a surface water discharge plume model called PSD. This model predicts the

plume will encounter the left river bank about 45 feet downstream. However both of these models assume the river can furnish unlimited dilution water. This is not the case in the Pudding River, and only rough assumptions can be made for the time required for complete mixing with the river water and the plume characteristics. The velocity of the discharge will pump river water into the plume near the point of discharge. However, the rate of mixing gradually decreases as the velocity of the plume slows down, depending only on difusion to complete the mixing. The maximum dilution is obtained when the plume and river are completely mixed, sometime after the plume fills the river width. It is expected the river and effluent will be well mixed after the flow is past the 90 degree bend located about 300 feet downstream from the outfall. This compares with the NPDES permit mixing zone of 270 feet.

A dye study, made by releasing dye into the effluent and measuring the concentration downstream, can be used to measure the actual dilution and plume dimensions.

The low flow characteristics of the outfall and river are presented in Table 1.

The current NPDES permit defines the allowable mixing zone as the region within 270 feet downstream of the point of discharge. According to OAR 340-41-445, the water outside the mixing zone boundary must be free of materials in concentrations that will cause chronic toxicity. Additionally, the water must meet all other water quality standards under normal annual low flow periods. The NPDES permit also defines the Zone of Immediate Dilution (ZID); it must be within 10 percent of the allowable mixing zone. Water at the edge of the ZID must be free of materials in concentrations that will cause acute toxicity. DEQ may redefine the ZID after completion of the WWTP modifications.

Table 1
Assumed River and Outfall Characteristics
Pudding River
Near Woodburn WWTP Outfall

River/Outfall Parameter	*Value*
River flow 7Q10 (cfs)	15.0
Average river current (ft/sec)	0.9
Flow area (sq ft)	24
Average depth (ft)	1
River width (ft)	25
Present ADWF (mgd)	3.1
Present ADWF (cfs)	4.8
Future ADWF (mgd)	5.0
Future ADWF (cfs)	7.74
Outfall pipe diameter (in)	24
Outfall channel area (sq ft)	15
Outfall channel width (ft)	10
Outfall Channel depth (ft)	1.5
Outfall channel discharge angle (degrees)	30
Outfall channel velocity (present ADWF, fps)	0.32
Outfall channel velocity (future ADWF, fps)	0.33

Notes:
(a) 7Q10 is the lowest flow for seven consecutive days over a ten year period.

Section 3
Future Wastewater Flows And Loads

Introduction

This section presents the projected flows and waste loads for the study area. Historical flow and load information and weather information are also presented in this section, supplementing the discussion in Section 2. This information is used as the basis for the flow analyses.

Planning Period

The time period for wastewater facilities planning is typically about 20 years, which is generally consistent with the design life of wastewater treatment facilities. For the Woodburn plan, the planning period extends to the year 2020, which is 22 years beyond an approximate 1999 construction date for improvements to the Woodburn Wastewater Treatment Plant (WWTP).

Population and Land Use Projections

Population

The population projections form the basis of the projected flows and loads (BOD_5 and TSS) for the Woodburn WWTP. Separate population equivalent estimates for commercial and industrial contributions are developed for flow and loading projections. These estimates are based on conversations with City of Woodburn staff for the design of the wastewater treatment facilities.

Population Equivalents for Flow Projections

The population equivalents for flow projections developed by the City of Woodburn are summarized in Table 3-1. The City's original projections are presented in Appendix 3.2. The projections reflect average summer (average dry weather) conditions. The permitted industrial flow is 26.8 percent. Residential flow is 60.9 percent and commercial is 12.3 percent.

Figure 3-1 shows the projected population as developed by four different methods:

- Highest historical growth rate (4.2 percent)
- Facilities plan growth rate (3.4 percent) as approved by the Wastewater Advisory Committee and City Council

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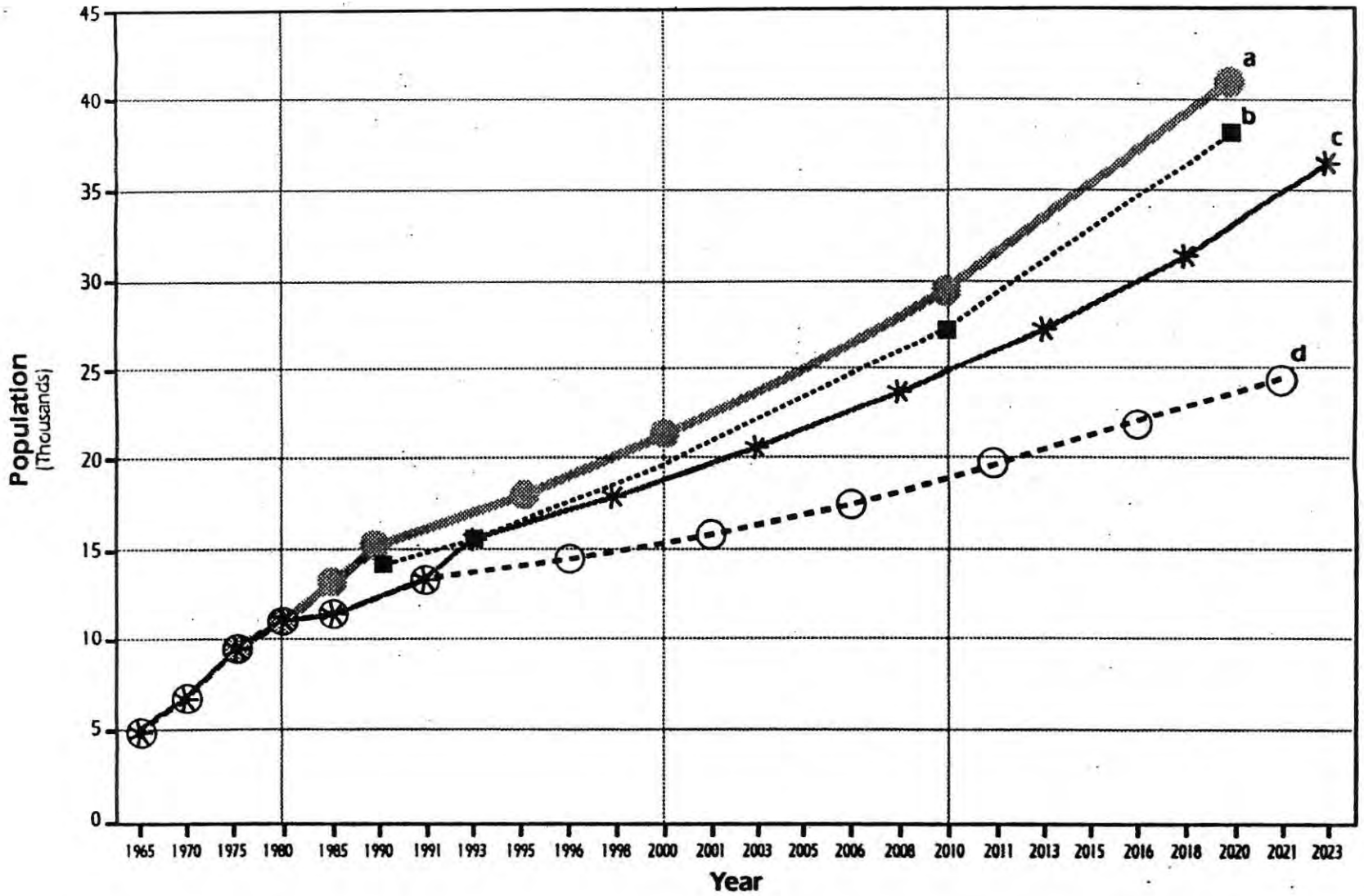
Table 3-1
Flow Population Equivalent Projections
Woodburn WWTP
1993-2020

Year	*Residential Summer Average Population (a)*	*Commercial Summer Average Pop. Equivalent (a)*	*Permitted Industrial Mo. Avg. Pop. Equivalent (b)*	*Total Summer Population Equivalent*	*Permitted Industrial Pk. Month Avg. Pop. Equiv. (c)*	*Total Peak Month Population Equivalent*
1993	17,707	3,585	7,793	29,085	9,634	30,926
1994	18,309	3,707	7,832	29,848	9,730	31,746
1995	18,932	3,833	7,871	30,636	9,828	32,592
1996	19,575	3,963	7,910	31,449	9,926	33,465
1997	20,241	4,098	7,950	32,289	10,025	34,364
1998	20,929	4,237	7,990	33,156	10,126	35,292
1999	21,641	4,381	8,030	34,052	10,227	36,249
2000	22,376	4,530	8,070	34,977	10,329	37,236
2001	23,137	4,684	8,110	35,932	10,432	38,254
2002	23,924	4,844	8,151	36,918	10,537	39,304
2003	24,737	5,008	8,192	37,937	10,642	40,388
2004	25,578	5,179	8,232	38,989	10,749	41,505
2005	26,448	5,355	8,274	40,076	10,856	42,659
2006	27,347	5,537	8,315	41,199	10,965	43,848
2007	28,277	5,725	8,357	42,359	11,074	45,076
2008	29,238	5,920	8,398	43,556	11,185	46,343
2009	30,232	6,121	8,440	44,794	11,297	47,650
2010	31,260	6,329	8,483	46,072	11,410	48,999
2011	32,323	6,544	8,525	47,392	11,524	50,391
2012	33,422	6,767	8,568	48,757	11,639	51,828
2013	34,559	6,997	8,610	50,166	11,755	53,311
2014	35,734	7,235	8,654	51,622	11,873	54,841
2015	36,949	7,481	8,697	53,126	11,992	56,421
2016	38,205	7,735	8,740	54,680	12,112	58,051
2017	39,504	7,998	8,784	56,286	12,233	59,735
2018	40,847	8,270	8,828	57,945	12,355	61,472
2019	42,236	8,551	8,872	59,659	12,479	63,265
2020	43,672	8,842	8,916	61,430	12,603	65,117

Notes:
(a) Residential and commercial flow growth rate assumed to be 3.4% per year.
(b) Average month industrial flow growth rate assumed to be 0.5% per year.
(c) Peak month industrial flow growth rate assumed to be 1% per year.



[a] High historical growth rate held constant

[b] Facilities plan growth rate (3.4%) as approved by the Wastewater Advisory Committee and City Council

[c] Average historical growth rate

[d] PSU Population Research and Census Center projection

FIGURE 3-1

City of Woodburn Population Projections

CH2M HILL

- Average historical growth rate
- Portland State University (PSU) Population Research and Census Center projection

For the purposes of this facilities plan, a 3.4 percent population growth rate was approved by the Wastewater Advisory Committee and City Council because it represents a conservative and reasonable projection that is greater than the average but less than the maximum growth rate and reflects the high population growth rate projections for the Portland metropolitan area.

The residential population equivalent is expected to grow at 3.4 percent per year over the 25 year planning period (from 17,707 in 1993 to 43,659 in 2020). The commercial component is also expected to grow at 3.4 percent per year (from 3,585 in 1993 to 8,841 in 2020).

The permitted industrial population equivalent is expected to grow at 1.0 percent per year (from 7,793 in 1993 to 8,915 in 2020). The current food processing industries have indicated they do not expect to expand their operations. The local area fields that support the food processing industries are at capacity and crop yield depends on weather conditions.

Population Equivalents for Load Projections

The City of Woodburn wastewater system receives a relatively large industrial input during the summer months. This is due mainly to seasonal cannery operations. Population equivalents for the BOD_5 and TSS loading conditions were developed by the City of Woodburn (see Appendix 3.2). The permanent and migrant residential and commercial population growth rates are 3.4 percent per year over the 25-year planning period; the industrial growth rate is 1 percent per year for the same planning period. The projections are summarized in Table 3-2 (BOD_5) and Table 3-3 (TSS).

Land Use

Land use within the UGB of Woodburn is dictated by the City urban growth program. The current UGB (Figure 2-1) represents the likely limits to urban expansion. Residential, commercial, and industrial lands are available within the UGB for future development. The City does not anticipate any significant changes in the percentages for the types of land use in the area. Therefore, for future projections, residential and commercial growth is assumed to increase proportionally with population growth. Industrial growth is assumed to grow at 1 percent annually. With future growth, land will continue to be developed within the UGB in accordance with zoning requirements. The City will expand the wastewater collection system as necessary to serve the additional growth.

Wastewater Flows and Projections

Wastewater flows for the Woodburn WWTP were projected using two methods: the conventional average-dry-weather-flow method and the method currently recommended by

Table 3-2
BOD 5 Population Equivalent Projections
Woodburn WWTP
1993-2020

	(A)	(B)	(C)	(D)=(A)+(B)+(C)	(E)=(A)+(C)	(F)	(G)=(D)+(F)	(H)=(E)+(F)	(I)	(J)=(G)+(I)	(K)=(H)+(I)	(L)	(M)=(G)+(L)	(N)	(O)=(G)+(N)
Year	*Permanent Residential Population (a) (summer & winter)*	*Migrant Population (a) (summer only)*	*Other Population (b) (summer & winter)*	*Summer Residential Population*	*Winter Residential Population*	*Commercial Population Equivalent (a)*	*Summer Residential & Commercial Population Equivalent*	*Winter Residential & Commercial Population Equivalent*	*Industrial Yearly Average Population Equivalent (c)*	*Total Summer Average Population Equivalent*	*Total Winter Average Population Equivalent*	*Industrial Peak Month Population Equivalent (c)*	*Total Summer Peak Month Average Population Equivalent*	*Industrial Worst Case Scenario Population Equivalent (c)*	*Total Summer Worst Case Scenario Population Equivalent*
1993	15,645	1,662	400	17,707	16,045	3,507	21,214	19,552	3,711	24,925	23,263	7,351	28,565	65,389	86,603
1994	16,177	1,719	400	18,295	16,577	3,626	21,922	20,203	3,748	25,670	23,951	7,425	29,346	66,043	87,965
1995	16,727	1,777	400	18,904	17,127	3,750	22,653	20,876	3,786	26,439	24,662	7,499	30,152	66,703	89,357
1996	17,296	1,837	400	19,533	17,696	3,877	23,410	21,573	3,823	27,233	25,396	7,574	30,984	67,370	90,780
1997	17,884	1,900	400	20,184	18,284	4,009	24,192	22,293	3,862	28,054	26,154	7,649	31,842	68,044	92,236
1998	18,492	1,964	530	20,986	19,022	4,145	25,131	23,167	3,900	29,032	27,067	7,726	32,857	68,724	93,856
1999	19,120	2,031	530	21,682	19,650	4,286	25,968	23,937	3,939	29,907	27,876	7,803	33,771	69,412	95,379
2000	19,771	2,100	530	22,401	20,301	4,432	26,833	24,732	3,979	30,811	28,711	7,881	34,714	70,106	96,938
2001	20,443	2,172	980	23,594	21,423	4,582	28,177	26,005	4,018	32,195	30,024	7,960	36,137	70,807	98,984
2002	21,138	2,246	980	24,363	22,118	4,738	29,102	26,856	4,059	33,160	30,915	8,040	37,141	71,515	100,617
2003	21,857	2,322	980	25,158	22,837	4,899	30,058	27,736	4,099	34,157	31,835	8,120	38,178	72,230	102,288
2004	22,600	2,401	980	25,980	23,580	5,066	31,046	28,646	4,140	35,187	32,786	8,201	39,248	72,952	103,999
2005	23,368	2,482	980	26,830	24,348	5,238	32,069	29,586	4,182	36,250	33,768	8,283	40,352	73,682	105,751
2006	24,163	2,567	980	27,709	25,143	5,416	33,126	30,559	4,223	37,349	34,782	8,366	41,492	74,419	107,544
2007	24,984	2,654	980	28,618	25,964	5,600	34,219	31,565	4,266	38,484	35,830	8,450	42,668	75,163	109,382
2008	25,834	2,744	980	29,558	26,814	5,791	35,349	32,604	4,308	39,657	36,913	8,534	43,883	75,915	111,263
2009	26,712	2,838	980	30,530	27,692	5,988	36,517	33,680	4,351	40,869	38,031	8,620	45,137	76,674	113,191
2010	27,620	2,934	980	31,534	28,600	6,191	37,726	34,791	4,395	42,121	39,186	8,706	46,431	77,440	115,166
2011	28,559	3,034	980	32,573	29,539	6,402	38,975	35,941	4,439	43,414	40,380	8,793	47,768	78,215	117,190
2012	29,530	3,137	980	33,647	30,510	6,620	40,267	37,130	4,483	44,750	41,613	8,881	49,148	78,997	119,264
2013	30,534	3,244	980	34,758	31,514	6,845	41,602	38,359	4,528	46,131	42,887	8,970	50,572	79,787	121,389
2014	31,572	3,354	980	35,906	32,552	7,077	42,984	39,630	4,573	47,557	44,203	9,059	52,043	80,585	123,569
2015	32,646	3,468	980	37,094	33,626	7,318	44,412	40,944	4,619	49,031	45,563	9,150	53,562	81,391	125,802
2016	33,756	3,586	980	38,322	34,736	7,567	45,888	42,302	4,665	50,554	46,968	9,241	55,130	82,205	128,093
2017	34,903	3,708	980	39,591	35,883	7,824	47,415	43,707	4,712	52,127	48,419	9,334	56,749	83,027	130,442
2018	36,090	3,834	980	40,904	37,070	8,090	48,994	45,160	4,759	53,753	49,919	9,427	58,421	83,857	132,851
2019	37,317	3,964	980	42,262	38,297	8,365	50,627	46,662	4,807	55,433	51,469	9,521	60,148	84,696	135,322
2020	38,586	4,099	980	43,665	39,566	8,649	52,315	48,216	4,855	57,169	53,070	9,617	61,931	85,542	137,857

Notes:
(a) Residential and commercial population equivalent growth rate assumed to be 3.4% per year.
(b) Includes areas outside the Woodburn city limits and inside the UGB that are served by the Woodburn WWTP: Carl Road Mobile Home units, MacLaren School, and Shalimar Mobile Home units.
(c) Industrial population equivalent growth rate assumed to be 1% per year.

Table 3-2

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Table 3-3
TSS Population Equivalent Projections
Woodburn WWTP
1993-2020

	(A)	(B)	(C)	(D)=(A)+(B)+(C)	(E)=(A)+(C)	(F)	(G)=(D)+(F)	(H)=(E)+(F)	(I)	(J)=(G)+(I)	(K)=(H)+(I)	(L)	(M)=(G)+(L)	(N)	(O)=(G)+(N)
Year	*Permanent Residential Population (a) (summer & winter)*	*Migrant Population (a) (summer only)*	*Other Population (b) (summer & winter)*	*Summer Residential Population*	*Winter Residential Population*	*Commercial Population Equivalent (a)*	*Summer Residential & Commercial Population Equivalent*	*Winter Residential & Commercial Population Equivalent*	*Industrial Yearly Average Population Equivalent (c)*	*Total Summer Average Population Equivalent*	*Total Winter Average Population Equivalent*	*Industrial Peak Month Population Equivalent (c)*	*Total Summer Peak Month Average Population Equivalent*	*Industrial Worst Case Scenario Population Equivalent (c)*	*Total Summer Worst Case Scenario Population Equivalent*
1993	15,645	1,662	400	17,707	16,045	2,694	20,401	18,739	2,000	22,401	20,739	3,857	24,258	11,916	32,317
1994	16,177	1,719	400	18,295	16,577	2,786	21,081	19,363	2,020	23,101	21,383	3,896	24,977	12,035	33,116
1995	16,727	1,777	400	18,904	17,127	2,880	21,784	20,007	2,040	23,824	22,047	3,935	25,719	12,156	33,940
1996	17,296	1,837	400	19,533	17,696	2,978	22,511	20,674	2,061	24,572	22,735	3,974	26,485	12,277	34,788
1997	17,884	1,900	400	20,184	18,284	3,079	23,263	21,363	2,081	25,344	23,444	4,014	27,277	12,400	35,663
1998	18,492	1,964	530	20,986	19,022	3,184	24,170	22,206	2,102	26,272	24,308	4,054	28,224	12,524	36,694
1999	19,120	2,031	530	21,682	19,650	3,292	24,974	22,943	2,123	27,097	25,066	4,094	29,068	12,649	37,623
2000	19,771	2,100	530	22,401	20,301	3,404	25,805	23,705	2,144	27,950	25,849	4,135	29,940	12,776	38,581
2001	20,443	2,172	980	23,594	21,423	3,520	27,115	24,943	2,166	29,280	27,109	4,177	31,291	12,903	40,018
2002	21,138	2,246	980	24,363	22,118	3,640	28,003	25,758	2,187	30,191	27,945	4,218	32,222	13,032	41,036
2003	21,857	2,322	980	25,158	22,837	3,764	28,922	26,600	2,209	31,131	28,809	4,261	33,183	13,163	42,085
2004	22,600	2,401	980	25,980	23,580	3,892	29,872	27,471	2,231	32,103	29,703	4,303	34,175	13,294	43,166
2005	23,368	2,482	980	26,830	24,348	4,024	30,854	28,372	2,254	33,108	30,626	4,346	35,200	13,427	44,282
2006	24,163	2,567	980	27,709	25,143	4,161	31,870	29,303	2,276	34,146	31,579	4,390	36,260	13,562	45,432
2007	24,984	2,654	980	28,618	25,964	4,302	32,920	30,266	2,299	35,219	32,565	4,434	37,354	13,697	46,617
2008	25,834	2,744	980	29,558	26,814	4,448	34,006	31,262	2,322	36,328	33,584	4,478	38,484	13,834	47,840
2009	26,712	2,838	980	30,530	27,692	4,600	35,129	32,292	2,345	37,474	34,637	4,523	39,652	13,972	49,102
2010	27,620	2,934	980	31,534	28,600	4,756	36,290	33,356	2,369	38,659	35,725	4,568	40,858	14,112	50,402
2011	28,559	3,034	980	32,573	29,539	4,918	37,491	34,457	2,392	39,883	36,849	4,614	42,104	14,253	51,744
2012	29,530	3,137	980	33,647	30,510	5,085	38,732	35,595	2,416	41,148	38,011	4,660	43,392	14,396	53,128
2013	30,534	3,244	980	34,758	31,514	5,258	40,016	36,772	2,440	42,456	39,212	4,706	44,722	14,540	54,556
2014	31,572	3,354	980	35,906	32,552	5,437	41,343	37,989	2,465	43,808	40,454	4,753	46,096	14,685	56,028
2015	32,646	3,468	980	37,094	33,626	5,621	42,715	39,247	2,489	45,205	41,737	4,801	47,516	14,832	57,547
2016	33,756	3,586	980	38,322	34,736	5,813	44,134	40,548	2,514	46,649	43,063	4,849	48,983	14,980	59,115
2017	34,903	3,708	980	39,591	35,883	6,010	45,602	41,894	2,539	48,141	44,433	4,897	50,499	15,130	60,732
2018	36,090	3,834	980	40,904	37,070	6,215	47,119	43,285	2,565	49,684	45,850	4,946	52,065	15,281	62,400
2019	37,317	3,964	980	42,262	38,297	6,426	48,687	44,723	2,591	51,278	47,314	4,996	53,683	15,434	64,122
2020	38,586	4,099	980	43,665	39,566	6,644	50,309	46,210	2,616	52,926	48,827	5,046	55,355	15,589	65,898

Notes:
(a) Residential and commercial population equivalent growth rate assumed to be 3.4% per year.
(b) Includes areas outside the Woodburn city limits and inside the UGB that are served by the Woodburn WWTP: Carl Road Mobile Home units, MacLaren School, and Shalimar Mobile Home units.
(c) Industrial population equivalent growth rate assumed to be 1% per year.

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the Oregon Department of Environmental Quality (DEQ) that incorporates historical rainfall data into the analysis. The results of these two methods are discussed and compared below.

Conventional Average-Dry-Weather-Flow Method

Historical Data

Wastewater flow data from January 1989 through December 1993 are used to determine historical flow variations. Effluent flow measurements are used for 1989, 1990, and the first half of 1991. An influent flowmeter was installed during the summer of 1991; measurements from this meter are used after July 1991. The meter, however, was out of service for repairs for 7 months during 1992; during this time, the effluent flow measurements are used. Additionally, there are several days throughout 1991 through 1993 where effluent flows are used because no influent flow measurements are available (an expanded discussion of influent and effluent flow measurements is contained in Section 2). Influent data are used whenever they are available. They are highlighted in Table 2-6, which summarizes the historical flow data. These data are from relatively dry years.

Peaking Factors

Using average dry weather flow (ADWF) as the foundation, other design flow conditions for wastewater treatment facilities are estimated by using the peaking factors developed from the existing flows. Peaking factors are derived from the ratios of different flow conditions to the average dry weather flow (e.g., the maximum month peaking factor equals the ratio of the maximum month flow to the average dry weather flow). A comparison of historical peaking factors over the 5-year period 1989 through 1993 is summarized in Table 3-4. The maximum peaking factor for each flow condition is calculated, and these peaking factors are shown in bold. The extreme nature (and subsequent high flows) of these peaking factors is due in part to the extreme weather the Willamette Valley has experienced in the past several years. To minimize the effect of extreme peaking factors and take into account attenuation of peak flows as the system grows, the five peaking factors for each flow condition are averaged. These average peaking factors (Table 3-4, highlighted and in bold) are used to determine the different flow conditions.

In 1994 the influent flowmeter recorded a peak flow of 11.2 mgd when the Mill Creek Pump Station was pumping at maximum capacity. This flow was recorded after partial inflow correction had been implemented.

An instantaneous peak flow event was developed in Technical Memorandum F4-1 Collection System Evaluation–Flow Monitoring Program (CH2M HILL, June 11, 1993). Measurement of peak flows at the treatment plant have been hindered by flow measuring device failures and other limitations. The 1992 instantaneous peak wet weather flow event was estimated to be 12.00 mgd (see Table 10, Technical Memorandum F4-1 in Volume II of this facilities plan). This event occurred during a storm and when the Mill Creek Pump Station was malfunctioning. The City of Woodburn believes it can economically remove 2.00 mgd of I/I flow from this instantaneous flow event, reducing it to 10.00 mgd.

Table 3-4
Historical Flow Peaking Factors
Woodburn WWTP
1989-1993

Flow Condition	1989 Flow (mgd)	1989 Peaking Factor	1990 Flow (mgd)	1990 Peaking Factor	1991 Flow (mgd)	1991 Peaking Factor	1992 Flow (mgd)	1992 Peaking Factor	1993 Flow (mgd)	1993 Peaking Factor	Average Peaking Factor
Average Dry Weather Flow	1.736	1.00	1.960	1.00	1.857	1.00	1.478	1.00	1.871	1.00	1.00
Average Wet Weather Flow	2.130	1.23	2.242	1.14	2.136	1.15	2.353	1.59	2.105	1.13	1.25
Annual Average Daily Flow	1.933	1.11	2.101	1.07	1.997	1.08	1.915	1.30	1.988	1.06	1.12
Dry Wthr Max Month Avg Daily Flow	1.869	1.08	2.219	1.13	2.649	1.43	1.825	1.23	2.455	1.31	1.24
Wet Wthr Max Month Avg Daily Flow	3.117	1.80	3.318	1.69	2.513	1.35	3.528	2.39	2.767	1.48	1.74
Annual Max Month Avg Daily Flow	3.117	1.80	3.318	1.69	2.649	1.43	3.528	2.39	2.767	1.48	1.76
Dry Weather Maximum Daily Flow	2.421	1.39	5.897	3.01	6.141	3.31	2.115	1.43	4.116	2.20	2.27
Wet Weather Maximum Daily Flow	4.000	2.30	4.262	2.17	4.457	2.40	7.920	5.36	4.492	2.40	2.93
Peak Hour Wet Weather Flow	n/a	n/a	n/a	n/a	n/a	n/a	12.00	8.12	n/a	n/a	n/a (1)

Notes:
(1) Assumes peak hour is attenuated in large collection system and I/I program will reduce peaking factor. Peaking factor will be peak hour divided by wet weather maximum daily flow (12/7.92 = 1.5).

If 12 mgd were used to determine the peak event peaking factor, the peaking factor would be 8.12. This would result in projecting extremely high flows, over-estimating the magnitude of the future peak event. Since peak events are attenuated as the collection system increases in size, a more realistic number is estimated by calculating a peaking factor based on the peak hour flow to the peak day flow. This peaking factor is 1.5 (12.0 mgd divided by 7.92 mgd = 1.5).

Flow Components

No historical division of flows is available for the Woodburn WWTP. Therefore, the division of historical flows into residential, commercial, and industrial components is based on flow split estimates provided by the City of Woodburn (see Appendix 3.1). The average daily flow for the 6-month summer period is divided as follows:

- 74.2 percent residential
- 15.0 percent commercial
- 10.7 percent actual industrial

These percentages are based on actual flows seen at the WWTP, not on the permitted industrial flows.

Flow Components–Per Capita Flow

The average dry weather per capita flow rate was estimated as 82 gpcpd for 1993, by the City of Woodburn. For comparison, per capita flow numbers were calculated for the years 1989 - 1993 using historical plant data (see Table 3-1 in the appendix to this section). The per capita numbers ranged from 68 to 94 gpcpd. The City plans to maintain a per capita of 82 gpcpd by imposing water conservation measures if necessary. A per capita of 82 gpcpd is used for planning and design calculations.

Projection of Flow to 2020

The projected average dry weather flows at the WWTP for the year 2020 are summarized on Table 3-5. To determine these flows the following approach was used:

1. Population equivalent projections through the year 2020 were estimated to increase by:

   - 3.4 percent per year for residential and commercial
   - 0.5 percent per year for permitted industrial monthly average
   - 1.0 percent per year for permitted industrial peak monthly average

2. The dry weather per capita flow rate was assumed to remain constant during the planning period at 82 gpcpd.

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Projected Average Dry Weather Flow
Woodburn WWTP
1993-2020

Year	*Summer Population Equivalent (a)*	*Average Dry Weather Per Capita Flow (b) (gpcpd)*	*Average Dry Weather Projected Flow (mgd)*
1993	29,085	82	2,385[c]
1994	29,834	82	2.446
1995	30,608	82	2.510
1996	31,407	82	2.575
1997	32,232	82	2.643
1998	33,213	82	2.723
1999	34,093	82	2.796
2000	35,001	82	2.870
2001	36,389	82	2.984
2002	37,358	82	3.063
2003	38,358	82	3.145
2004	39,392	82	3.230
2005	40,459	82	3.318
2006	41,561	82	3.408
2007	42,700	82	3.501
2008	43,876	82	3.598
2009	45,091	82	3.697
2010	46,346	82	3.800
2011	47,642	82	3.907
2012	48,982	82	4.016
2013	50,365	82	4.130
2014	51,795	82	4.247
2015	53,271	82	4.368
2016	54,797	82	4.493
2017	56,373	82	4.623
2018	58,002	82	4.756
2019	59,685	82	4.894
2020	61,423	82	5.037

Notes:
(a) Population projections based on City of Woodburn estimates.
(b) Per capita flow based on historical flow information for 1989-1993.
(c) Actual 1993 average dry weather flow = 1.871; difference between actual and projected = 0.51.

3. The per capita flow rate was multiplied by the projected population equivalent to calculate the average dry weather projected flow. The ADWF in 2020 was projected to be 5.037 mgd at 82 gpcpd.

4. Average peaking factors were used to determine other flow conditions. These flows are summarized in Table 3-6.

The actual flows for 1993 and the projected flows for 1993 vary. This is because the projected flow includes the total industrial permitted flow and several mobile homes accounted for in the projections that are not yet hooked up. It is unlikely all of the industries will discharge at their maximum permitted flow at the same time, so the projected flows are conservative. When the difference between the actual industrial flows and permitted industrial flows (0.51 mgd) is subtracted from the projected flows, it is estimated the 5 mgd ADWF may occur between 2020 and 2025, depending on the industrial discharges (Table 3-6 and Figure 3-2). Figure 3-1 shows a 5 year range for all the projected flows. Actual growth will determine when 5 mgd ADWF occurs.

The future peaking factors and flows should be attenuated somewhat by the additional sewered areas which will have additional storage capacity. Therefore, the peaking factor values are conservative.

DEQ Rainfall Method

Another methodology employed to predict future design flows is to use both rainfall and the corresponding treatment plant flow in an effort to more accurately predict the future design flows. This method was developed by DEQ and is outlined in *Guidelines for Making Flow Projections for Sewage Treatment Plants* (August 1994). The rationale for using both rainfall and historical treatment plant flows is that high plant flows are usually associated with rain and high groundwater in western Oregon. Additionally, a 6-month wet- or dry-season average (as required by NPDES permitting) has a 50 percent chance of being exceeded each year, leading to further treatment plant failures. By using the cumulative monthly rainfall and a 5-year storm in this method, a storm that has only a 20 percent chance of being exceeded can be used to predict flows. The plant must be designed at a minimum to treat these flows projected by the DEQ rainfall method.

Maximum Month Dry-Weather Design Flow (MMDWF)

The maximum month dry-weather flow is the monthly average flow in the rainiest summer month with high groundwater. In western Oregon (west of the Cascades in areas not affected by irrigation and snowmelt) this flow usually occurs during May. The May monthly precipitation that will not be exceeded 80 percent of the time is 3.64 inches (for Oregon City, the closest reporting station contained in *Climatology of the United States No. 20*, U.S. Weather Bureau). Monthly cumulative rainfall (for Oregon City) and plant flows for the high groundwater season (January through May, 1989 through 1993) are tabulated in Table 3-7 and plotted in Figure 3-3. The May rainfall of 3.64 inches corresponds to a flow of 2.323 mgd (MMDWF).

Projected Flows
Woodburn WWTP
1993-2020

Years	Dry Weather: Average Dry Weather Flow (a) (ADWF) (mgd)	Dry Weather: Max Month Avg Daily Flow (ADWF x 1.24) (mgd)	Dry Weather: Maximum Daily Flow (ADWF x 2.27) (mgd)	Wet Weather: Average Wet Wthr Flow (ADWF x 1.25) (mgd)	Wet Weather: Maximum Month Avg Daily Flow (ADWF x 1.74) (mgd)	Wet Weather: Maximum Daily Flow (WWMDF) (ADWF x 2.93) (mgd)	Wet Weather: Peak Hour Flow (WWMDF x 1.5) (mgd)	Annual Average Daily Flow (ADWF x 1.12) (mgd)
1993 - 1998	2.385	2.949	5.410	2.976	4.153	6.982	10.473	2.680
1994 - 1999	2.448	3.026	5.552	3.054	4.262	7.165	10.748	2.751
1995 - 2000	2.512	3.106	5.698	3.134	4.375	7.355	11.032	2.823
1996 - 2001	2.579	3.189	5.849	3.217	4.491	7.550	11.325	2.898
1997 - 2002	2.648	3.274	6.006	3.303	4.611	7.751	11.627	2.975
1998 - 2003	2.719	3.362	6.167	3.392	4.735	7.960	11.939	3.055
1999 - 2004	2.792	3.452	6.333	3.484	4.863	8.175	12.262	3.138
2000 - 2005	2.868	3.546	6.505	3.578	4.995	8.397	12.595	3.223
2001 - 2006	2.946	3.643	6.683	3.676	5.131	8.626	12.939	3.311
2002 - 2007	3.027	3.743	6.867	3.777	5.272	8.863	13.294	3.402
2003 - 2008	3.111	3.846	7.056	3.881	5.418	9.107	13.661	3.496
2004 - 2009	3.197	3.953	7.252	3.989	5.568	9.360	14.040	3.593
2005 - 2010	3.286	4.063	7.454	4.100	5.723	9.621	14.431	3.693
2006 - 2011	3.378	4.177	7.663	4.215	5.883	9.890	14.836	3.797
2007 - 2012	3.473	4.295	7.878	4.334	6.049	10.169	15.253	3.903
2008 - 2013	3.572	4.416	8.101	4.456	6.220	10.456	15.685	4.014
2009 - 2014	3.673	4.542	8.331	4.583	6.397	10.753	16.130	4.128
2010 - 2015	3.778	4.671	8.569	4.713	6.579	11.060	16.591	4.246
2011 - 2016	3.886	4.805	8.815	4.849	6.768	11.377	17.066	4.367
2012 - 2017	3.998	4.943	9.068	4.988	6.963	11.705	17.557	4.493
2013 - 2018	4.114	5.086	9.331	5.132	7.164	12.043	18.065	4.623
2014 - 2019	4.233	5.234	9.601	5.281	7.372	12.393	18.589	4.757
2015 - 2020	4.356	5.386	9.881	5.435	7.587	12.754	19.131	4.896
2016 - 2021	4.484	5.544	10.170	5.594	7.808	13.127	19.690	5.039
2017 - 2022	4.615	5.707	10.469	5.758	8.038	13.512	20.268	5.187
2018 - 2023	4.751	5.875	10.777	5.928	8.275	13.911	20.866	5.340
2019 - 2024	4.892	6.049	11.096	6.103	8.519	14.322	21.483	5.498
2020 - 2025	5.037	6.228	11.426	6.285	8.772	14.747	22.121	5.661

Notes:
(a) Assume equal to City of Woodburn Summer Average Flow.

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Table 3-7
Monthly Cumulative Rainfall and Average Monthly Plant Flow
Woodburn WWTP
January - May, 1989-1993

	Actual Data Points		*Trend Line*	
Date	*Average Monthly Flow (a) (mgd)*	*Monthly Cumulative Rainfall (b) (inches)*	*Average Monthly Flow (a) (mgd)*	*Monthly Cumulative Rainfall (b) (inches)*
Jan-89	1.895	5.18	1.872	0.00
Feb-89	2.057	3.18	1.995	1.00
Mar-89	3.117	6.78	2.119	2.00
Apr-89	2.191	2.02	2.243	3.00
May-89	1.795	2.23	2.367	4.00
Jan-90	2.597	9.65	2.491	5.00
Feb-90	3.318	4.58	2.615	6.00
Mar-90	2.472	2.78	2.739	7.00
Apr-90	1.914	2.59	2.863	8.00
May-90	2.035	1.92	2.987	9.00
Jan-91	1.776	3.70	3.110	10.00
Feb-91	2.090	4.26	3.234	11.00
Mar-91	2.358	3.71	3.358	12.00
Apr-91	2.513	4.63	2.323	3.64
May-91	2.649	4.40	3.304	11.56
Jan-92	2.628	5.24		
Feb-92	3.528	4.48		
Mar-92	2.317	1.12		
Apr-92	2.158	5.10		
May-92	1.819	0.16		
Jan-93	2.118	3.74		
Feb-93	1.824	1.11		
Mar-93	2.585	5.24		
Apr-93	2.767	6.15		
May-93	2.433	4.24		

Notes:
(a) Both influent and effluent data were available for some months. Influent data were used wherever possible.
(b) Rainfall data are from Oregon City, Oregon.

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Figure 3-2
Projected Range of Average Dry Weather Flows
Woodburn WWTP
1993-2020

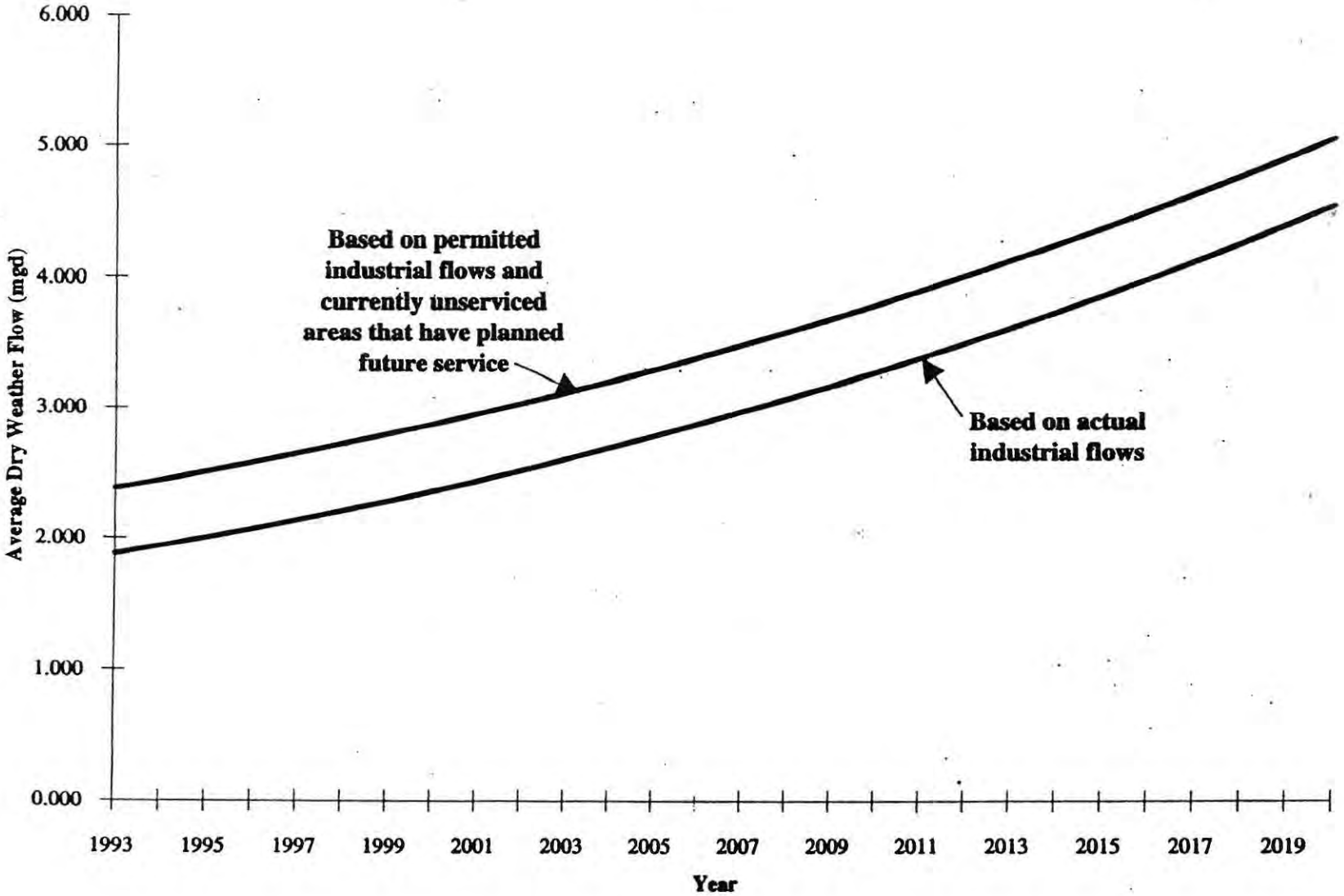
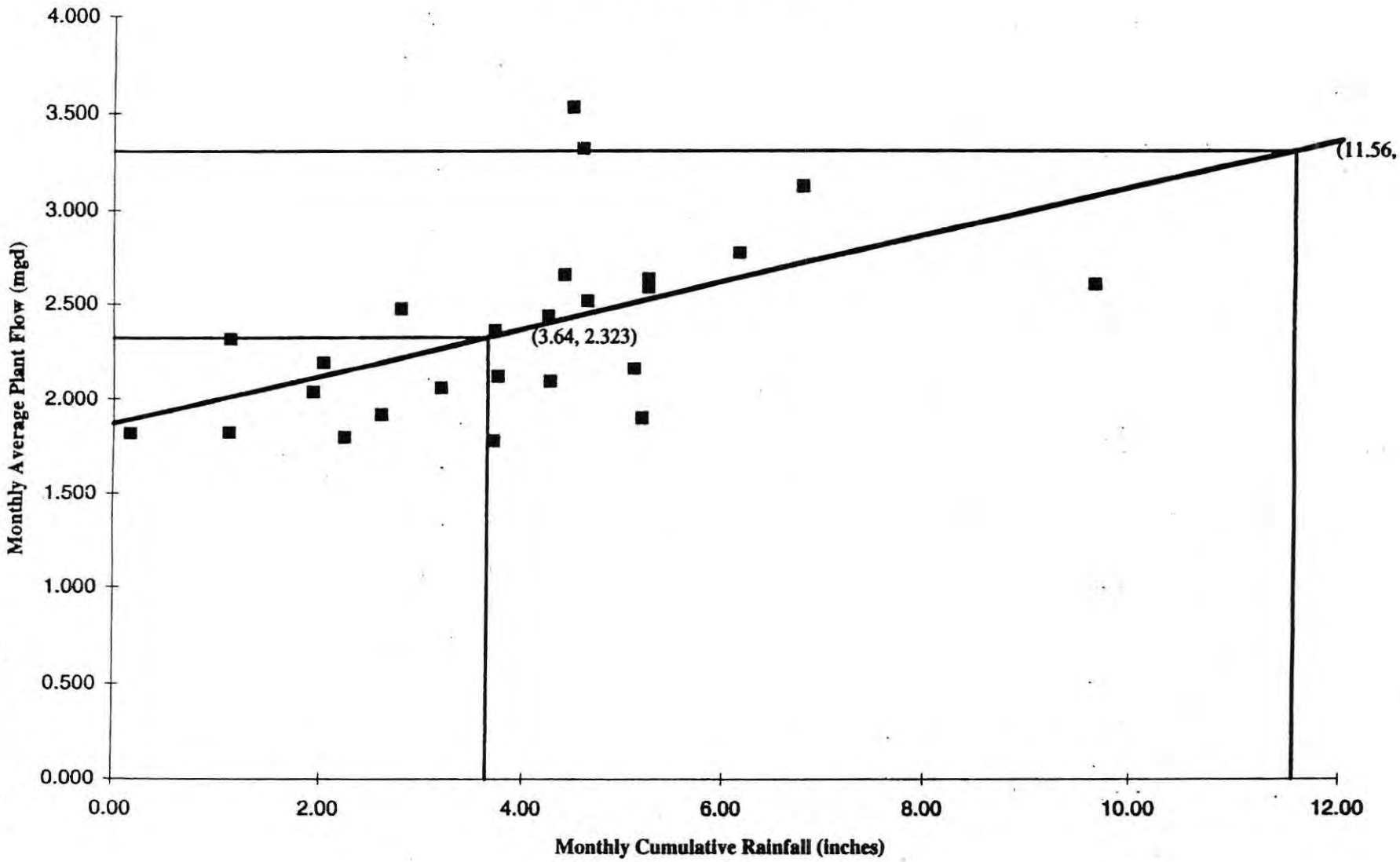


Figure 3-3
Monthly Cumulative Rainfall vs Average Monthly Plant Flow
Woodburn WWTP
January - May, 1989-1993



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Maximum Month Wet-Weather Design Flow (MMWWF)

The MMWWF design flow is calculated as described above for the MMDWF, using January precipitation data instead of May. West of the Cascades in Oregon, January is the maximum wet-weather month with high groundwater, and the maximum month wet-weather flow (MMWWF) usually occurs in January. The January monthly precipitation that will not be exceeded 80 percent of the time is 11.56 inches (for Oregon City, the closest reporting station contained in *Climatology of the United States No. 20*, U.S. Weather Bureau). The January rainfall of 11.56 inches corresponds to a flow of 3.304 mgd (MMWWF).

Peak Daily Average Flow and Peak Instantaneous Flow (PIF)

Daily rainfall, plant flow data (Table 3.1-2 in appendix to this section), and a plot of daily rainfall versus daily plant flow (Figure 3-4) are used to predict the peak daily average flow. The flow that corresponds to the 5-year, 24-hour storm is used as the peak daily average flow. This storm is 3.0 inches for Woodburn/Oregon City, as estimated from an isopluvial map (Figure 26 contained in the *NOAA Atlas 2, Volume X*). The plant flow that corresponds to this storm is 6.45 mgd.

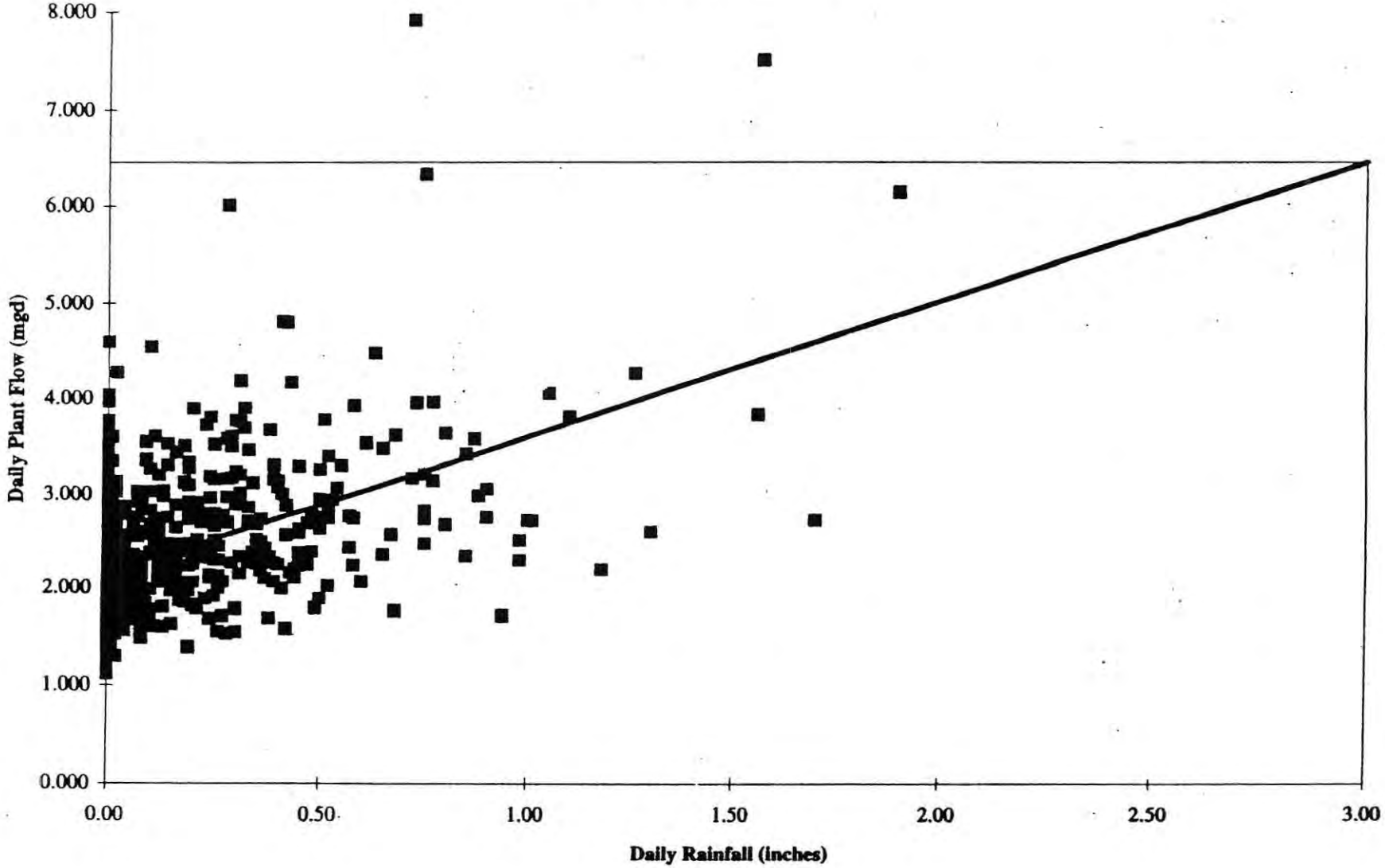
Peaking factors, as discussed above, from observed historical flows are used to predict the peak instantaneous flow. A peaking factor of 1.5 (12 mgd peak hour divided by 7.92 mgd wet weather maximum day flow) is applied to the 6.45 mgd flow to obtain a peak instantaneous flow of 9.68 mgd.

Projection of Flows to 2020

City of Woodburn flow population projections were used to project the flows developed by the rainfall method. The peak month flow population equivalent (total population equivalent including industry) and the maximum monthly dry weather flow were used to determine a maximum month dry weather per capita flow rate (Table 3-8) for the rainfall method. The maximum month per capita flow by this method is 75 gpcpd. The 75 gpcpd is held constant for the planning period 1993 through 2020. Projected flows by the DEQ rainfall method ranged from 2.323 mgd in 1993 to 4.891 mgd in 2020.

The winter population equivalent projection for the City of Woodburn is determined by subtracting the migrant summer population. The winter population equivalent (including industry) and the maximum monthly wet weather flow were used to determine a wet weather maximum month per capita flow rate (Table 3-8). The maximum month wet weather per capita flow of 142 gpcpd is held constant for the planning period 1993 through 2020. Projected flows by the DEQ rainfall method ranged from 3.304 mgd in 1993 to 7.537 mgd in 2020.

Figure 3-4
Daily Rainfall vs Daily Plant Flow
Woodburn WWTP
January - May, 1989-1993



Projected Flow - DEQ Rainfall Method
Woodburn WWTP
1993-2020

Year	*Summer Population Equivalent (a)*	*Maximum Monthly Dry Weather Per Capita (b) (gpcpd)*	*Projected Flow (mgd)*	*Winter Population Equivalent (a)*	*Maximum Monthly Wet Weather Per Capita (c) (gpcpd)*	*Projected Flow (mgd)*
1993	30,926	75	2.323	23,263	142	3.304
1994	31,746	75	2.385	23,951	142	3.402
1995	32,592	75	2.448	24,662	142	3.503
1996	33,465	75	2.514	25,396	142	3.607
1997	34,364	75	2.581	26,154	142	3.715
1998	35,292	75	2.651	27,067	142	3.844
1999	36,249	75	2.723	27,876	142	3.959
2000	37,236	75	2.797	28,711	142	4.078
2001	38,254	75	2.873	30,024	142	4.264
2002	39,304	75	2.952	30,915	142	4.391
2003	40,388	75	3.034	31,835	142	4.521
2004	41,505	75	3.118	32,786	142	4.657
2005	42,659	75	3.204	33,768	142	4.796
2006	43,848	75	3.294	34,782	142	4.940
2007	45,076	75	3.386	35,830	142	5.089
2008	46,343	75	3.481	36,913	142	5.243
2009	47,650	75	3.579	38,031	142	5.401
2010	48,999	75	3.681	39,186	142	5.566
2011	50,391	75	3.785	40,380	142	5.735
2012	51,828	75	3.893	41,613	142	5.910
2013	53,311	75	4.004	42,887	142	6.091
2014	54,841	75	4.119	44,203	142	6.278
2015	56,421	75	4.238	45,563	142	6.471
2016	58,051	75	4.361	46,968	142	6.671
2017	59,735	75	4.487	48,419	142	6.877
2018	61,472	75	4.617	49,919	142	7.090
2019	63,265	75	4.752	51,469	142	7.310
2020	65,117	75	4.891	53,070	142	7.537

Notes:

(a) Total population equivalent projections (including industry) based on City of Woodburn estimates (from Table 3-1).

(b) Per capita based on 1993 total population equivalents (including industry) and MMDWF by the DEQ rainfall method.

(c) Per capita based on 1993 total population equivalents (including industry) and MMWWF by the DEQ rainfall method.

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Comparison of the Two Methods' Flow Projections

Table 3-9 compares the following aspects of the two methods of flow prediction:

- Dry weather maximum month average day flow with maximum month dry weather design flow
- Wet weather maximum month average day flow with maximum month wet weather design flow

When comparing the dry weather maximum month average daily flow of 6.228 mgd with the maximum month dry weather design flow of 4.891 mgd and the corresponding wet weather values of 8.772 mgd with 7.537 mgd, it can be seen that the flow conditions projected using the conventional average-dry-weather-flow method with peaking factors are more conservative than those of the DEQ rainfall method. The projections for the conventional method are also more consistent with flows observed at the plant. Therefore, the projections from the conventional method will be used.

Waste Loading Projections

As with wastewater treatment plant flows, the development of waste loading projections (BOD_5 and TSS) is predicated on historical data and population projections.

Biochemical Oxygen Demand Loads

Historical Loads and Per Capita Loads

Four years of WWTP influent BOD_5 and TSS data were used in the analysis. These data were taken from the WWTP monitoring records for January 1990 through December 1993 (see Table 3.1-3 in the appendix to this section). Table 3-10 summarizes 1993 loads for the Woodburn WWTP.

The City of Woodburn provided population equivalents based on a 1993 calculated per capita load of 0.177 ppcpd for BOD_5 and 0.162 for TSS. The "textbook" values for average dry weather loadings range from 0.17 to 0.20 ppcpd for residential and commercial BOD_5 and TSS loads. To calculate future loads, the industrial contribution is separated from the residential and commercial components for analysis.

Peaking Factors

Table 3-11 summarizes the BOD_5 data on an annual basis under different loading conditions. The peaking factors for each BOD_5 load condition are summarized in Table 3-12. As with the flow analysis, the four years worth of peaking factors are averaged. These maximum and averaged peaking factors are highlighted in Table 3-12.

Table 3-9

Flow Summary
Woodburn WWTP
2020

Flow Condition	Conventional ADWF Method: Average Peaking Factor	Conventional ADWF Method: 2020 Flow (mgd)	DEQ Rainfall Method 2020 Flow (mgd)
Average Dry Weather Flow (ADWF)	1.00	5.037	—
Average Wet Weather Flow (AWWF)	1.25	6.285	—
Annual Average Daily Flow (AADF)	1.12	5.661	—
Dry Weather Max Month Avg Daily Flow (DWMMADF)	1.24	6.228	4.891[a]
Wet Weather Max Month Avg Daily Flow (WWMMADF)	1.74	8.772	7.537[b]
Annual Maximum Month Avg Daily Flow (AMMADF)	1.76	8.846	—
Dry Weather Maximum Daily Flow (DWMDF)	2.27	11.426	—
Wet Weather Maximum Daily Flow (WWMDF)	2.93	14.747	—

Notes:

[a]Maximum Month Dry Weather Design Flow (MMDWDF)	—	—
[b]Maximum Month Wet Weather Design Flow (MMWWDF)	—	—

Table 3-10
BOD 5 and TSS Summary
Woodburn WWTP
1993

Load Condition	*BOD_5 Load (lb/day)*	*TSS Load (lb/day)*
Average Dry Weather Load	4,784	2,327
Dry Weather Max Daily Load	10,575	5,217
Dry Weather Min Daily Load	2,154	797
Dry Weather Max Month Avg Daily Load	8,486	3,116
Dry Weather Min Month Avg Daily Load	3,427	1,499
Average Wet Weather Load	3,774	2,946
Wet Weather Max Daily Load	7,419	6,306
Wet Weather Max Month Avg Daily Load	4,604	3,518

BOD 5 Historical Summary
Woodburn WWTP
1990-1993

Load Condition	*1990 Load (lb/day)*	*1991 Load (lb/day)*	*1992 Load (lb/day)*	*1993 Load (lb/day)*
Average Dry Weather Load	3,643	3,360	3,716	4,784
Dry Weather Max Daily Load	6,021	7,198	10,546	10,575
Dry Weather Min Daily Load	1,625	1,961	190	2,154
Dry Weather Max Month Avg Daily Load	5,030	3,980	4,441	8,486
Dry Weather Min Month Avg Daily Load	3,145	2,839	3,021	3,427
Average Wet Weather Load	2,960	2,756	3,502	3,774
Wet Weather Max Daily Load	4,352	6,261	10,028	7,419
Wet Weather Max Month Avg Daily Load	3,235	3,301	3,881	4,604

Table 3-12
Historical BOD s Load Peaking Factors
Woodburn WWTP
1990-1993

BOD Load Condition	1990 Load (lb/day)	1990 Peaking Factor	1991 Load (lb/day)	1991 Peaking Factor	1992 Load (lb/day)	1992 Peaking Factor	1993 Load (lb/day)	1993 Peaking Factor	Average Load (lb/day)	Average Peaking Factor
Average Dry Weather	3,643	1.00	3,360	1.00	3,716	1.00	4,784	1.00	3,876	1.00
Average Wet Weather	2,960	0.81	2,756	0.82	3,502	0.94	3,774	0.79	3,248	0.84
Annual Average Daily	3,302	0.91	3,058	0.91	3,609	0.97	4,279	0.89	3,562	0.92
Dry Wthr Max Month Avg Daily	5,030	1.38	3,980	1.18	4,441	1.19	8,486	1.77	8,486	1.38
Wet Wthr Max Month Avg Daily	3,235	0.89	3,301	0.98	3,881	1.04	4,604	0.96	4,604	0.97
Annual Max Month Avg Daily	5,030	1.38	3,980	1.18	4,441	1.19	8,486	1.77	8,486	1.38
Dry Weather Maximum Daily	6,021	1.65	7,198	2.14	10,546	2.84	10,575	2.21	10,575	2.21
Wet Weather Maximum Daily	4,352	1.19	6,261	1.86	10,028	2.70	7,419	1.55	10,028	1.83

Population Equivalent Projections for BOD_5

The projections of population equivalents for BOD_5 loading were developed by the City of Woodburn. These projections are summarized in Table 3-2. See Appendix 3.2 for more detail. Projections are included for summer and winter residential, commercial, industrial yearly average, industrial peak month, and the industrial worst case scenario.

BOD_5 Load Projections

The annual BOD_5 load projections are contained in Table 3-13. The annual residential and commercial load is determined by multiplying the summer residential and commercial population equivalent by the textbook value of 0.2 ppcpd and the historic annual peaking factor of 0.92 (to calculate average dry weather from annual average). The industrial yearly average population equivalent is multiplied by 0.177 ppcpd (provided by the City of Woodburn) to obtain the annual industrial BOD_5 load. These two figures are added together to obtain the total annual BOD_5 load. The calculated annual BOD_5 load in 1993 is 4,560 lb/day compared with an actual reported 4,279 lb/day in 1993.

The peak month BOD_5 load projections are contained in Table 3-14. The peak month residential and commercial load is determined by multiplying the summer residential and commercial population equivalent by the textbook value of 0.25 ppcpd. The industrial peak month population equivalent is multiplied by 0.177 ppcpd (provided by the City of Woodburn and used in the industrial population equivalent analysis for the industrial load) to obtain the peak month industrial BOD_5 load. These two figures are added together to obtain the peak month BOD_5 load. The calculated peak month BOD_5 for 1993 is 6,605 lb/day compared with the actual dry weather maximum month BOD_5 of 8,486 lb/day in October and wet weather maximum month of 4,604 lb/day in February. The period of record average of all the peak months from 1990 to 1993 is 5,484 lb/day. It appears that 1993 had an unusually high peak month. The projected load is in line with the normal peak months.

The worst case scenario BOD_5 load projections are contained in Table 3-15. The peak day residential and commercial load is determined by multiplying the summer residential and commercial population equivalent by the textbook value of 0.2 ppcpd and the historic peaking factor of 1.83 (peak day without industry). The industrial worst case scenario population equivalent is multiplied by 0.177 ppcpd (provided by the City of Woodburn) to obtain the worst case scenario industrial BOD_5 load. The worst case scenario assumes all the industries discharge at their permitted loads. These two figures are added together to obtain the worst case scenario BOD_5 load which is assumed to occur for at most a 2-week period. The calculated worst case scenario for 1993 BOD_5 is 19,338 lb/day compared with an actual 1993 BOD_5 of 10,575 lb/day in October (which is also the maximum day for the period of record).

Table 3-13
Projected Annual BOD 5 Loads
Woodburn WWTP
1993-2020

	Summer Residential & Commercial Population Equivalent (a)	*Annual Residential & Commercial BOD_5 Load (b) (lb/day)*	*Industrial Yearly Average Population Equivalent (a)*	*Annual Industrial BOD_5 Load (c) (lb/day)*	*Total Annual BOD_5 Load (d) (lb/day)*
1993	21,214	3,903	3,711	657	4,560
1994	21,922	4,034	3,748	663	4,697
1995	22,653	4,168	3,786	670	4,838
1996	23,410	4,307	3,823	677	4,984
1997	24,192	4,451	3,862	684	5,135
1998	25,131	4,624	3,900	690	5,315
1999	25,968	4,778	3,939	697	5,475
2000	26,833	4,937	3,979	704	5,641
2001	28,177	5,185	4,018	711	5,896
2002	29,102	5,355	4,059	718	6,073
2003	30,058	5,531	4,099	726	6,256
2004	31,046	5,713	4,140	733	6,445
2005	32,069	5,901	4,182	740	6,641
2006	33,126	6,095	4,223	748	6,843
2007	34,219	6,296	4,266	755	7,051
2008	35,349	6,504	4,308	763	7,267
2009	36,517	6,719	4,351	770	7,489
2010	37,726	6,942	4,395	778	7,719
2011	38,975	7,171	4,439	786	7,957
2012	40,267	7,409	4,483	794	8,203
2013	41,602	7,655	4,528	801	8,456
2014	42,984	7,909	4,573	809	8,718
2015	44,412	8,172	4,619	818	8,989
2016	45,888	8,443	4,665	826	9,269
2017	47,415	8,724	4,712	834	9,558
2018	48,994	9,015	4,759	842	9,857
2019	50,627	9,315	4,807	851	10,166
2020	52,315	9,626	4,855	859	10,485

Notes:
(a) From City of Woodburn.
(b) Based on 0.2 ppcpd (textbook value) BOD_5 load and 0.92 historic peaking factor.
(c) Based on 0.177 ppcpd from the City of Woodburn from which the industrial population equivalents were derived (Appendix 3.2).
(d) Septage is not accounted for in this table.

Projected Peak Month BOD_5 Loads
Woodburn WWTP
1993-2020

	Summer Residential & Commercial Population Equivalent (a)	*Peak Month Residential & Commercial BOD_5 Load (b) (lb/day)*	*Industrial Peak Month Population Equivalent (a)*	*Peak Month Industrial BOD_5 Load (c) (lb/day)*	*Peak Month BOD_5 Load (d) (lb/day)*
1993	21,214	5,304	7,351	1,301	6,605
1994	21,922	5,480	7,425	1,314	6,795
1995	22,653	5,663	7,499	1,327	6,991
1996	23,410	5,853	7,574	1,341	7,193
1997	24,192	6,048	7,649	1,354	7,402
1998	25,131	6,283	7,726	1,367	7,650
1999	25,968	6,492	7,803	1,381	7,873
2000	26,833	6,708	7,881	1,395	8,103
2001	28,177	7,044	7,960	1,409	8,453
2002	29,102	7,275	8,040	1,423	8,698
2003	30,058	7,514	8,120	1,437	8,952
2004	31,046	7,762	8,201	1,452	9,213
2005	32,069	8,017	8,283	1,466	9,483
2006	33,126	8,281	8,366	1,481	9,762
2007	34,219	8,555	8,450	1,496	10,050
2008	35,349	8,837	8,534	1,511	10,348
2009	36,517	9,129	8,620	1,526	10,655
2010	37,726	9,431	8,706	1,541	10,972
2011	38,975	9,744	8,793	1,556	11,300
2012	40,267	10,067	8,881	1,572	11,639
2013	41,602	10,401	8,970	1,588	11,988
2014	42,984	10,746	9,059	1,603	12,349
2015	44,412	11,103	9,150	1,620	12,722
2016	45,888	11,472	9,241	1,636	13,108
2017	47,415	11,854	9,334	1,652	13,506
2018	48,994	12,249	9,427	1,669	13,917
2019	50,627	12,657	9,521	1,685	14,342
2020	52,315	13,079	9,617	1,702	14,781

Notes:

(a) From City of Woodburn.

(b) Based on 0.25 ppcpd (textbook value) BOD_5 load.

(c) Based on 0.177 ppcpd from the City of Woodburn from which the industrial population equivalents were derived (Appendix 3.2).

(d) Septage is not accounted for in this table.

Table 3-15
Projected Worst Case Scenario BOD 5 Loads
Woodburn WWTP
1993-2020

	Summer Residential & Commercial Population Equivalent (a)	*Peak Day (w/o Industry) Res & Comm BOD_5 Load (b) (lb/day)*	*Industrial Worst Case Scenario Population Equivalent (a)*	*Worst Case Scenario Industrial BOD_5 Load (c) (lb/day)*	*Worst Case Scenario BOD_5 Load (lb/day)*
1993	21,214	7,764	65,389	11,574	19,338
1994	21,922	8,023	66,043	11,690	19,713
1995	22,653	8,291	66,703	11,806	20,098
1996	23,410	8,568	67,370	11,925	20,493
1997	24,192	8,854	68,044	12,044	20,898
1998	25,131	9,198	68,724	12,164	21,362
1999	25,968	9,504	69,412	12,286	21,790
2000	26,833	9,821	70,106	12,409	22,229
2001	28,177	10,313	70,807	12,533	22,846
2002	29,102	10,651	71,515	12,658	23,309
2003	30,058	11,001	72,230	12,785	23,786
2004	31,046	11,363	72,952	12,913	24,276
2005	32,069	11,737	73,682	13,042	24,779
2006	33,126	12,124	74,419	13,172	25,296
2007	34,219	12,524	75,163	13,304	25,828
2008	35,349	12,938	75,915	13,437	26,375
2009	36,517	13,365	76,674	13,571	26,937
2010	37,726	13,808	77,440	13,707	27,515
2011	38,975	14,265	78,215	13,844	28,109
2012	40,267	14,738	78,997	13,982	28,720
2013	41,602	15,227	79,787	14,122	29,349
2014	42,984	15,732	80,585	14,264	29,996
2015	44,412	16,255	81,391	14,406	30,661
2016	45,888	16,795	82,205	14,550	31,345
2017	47,415	17,354	83,027	14,696	32,050
2018	48,994	17,932	83,857	14,843	32,775
2019	50,627	18,529	84,696	14,991	33,520
2020	52,315	19,147	85,542	15,141	34,288

Notes:
(a) From City of Woodburn.
(b) Based on 0.2 ppcpd (textbook value) BOD_5 load and 1.83 historic peaking factor (peak day without industry).
(c) Based on 0.177 ppcpd from the City of Woodburn which equals the current permitted pounds per day allowable discharge (Appendix 3.2). This number was decreased by a percent chance of everyone discharging at once.

Suspended Solids Loads

ical Loads and Per Capita Loads

Four years of WWTP influent TSS data were used in the analysis, from January 1990 through December 1993 (see Table 3.1-3 in the appendix to this section). Table 2-8 summarizes the TSS data on a monthly and annual basis. The average dry weather TSS influent load is 2,673 lb/day for the period of record.

Peaking Factors

The peaking factors for each TSS load condition are summarized in Table 3-16. As with the flow and BOD_5 analysis, the 4 years worth of data are averaged. The maximum and averaged peaking factors are highlighted in Table 3-16.

Population Projections

The projections of population equivalents for TSS loading were developed by the City of Woodburn. These projections are summarized in Table 3-3. Projections are included for summer and winter residential, commercial, industrial yearly average, industrial peak month, and the industrial worst case scenario.

TSS Load Projections

The annual TSS load projections are contained in Table 3-17. The annual residential and commercial load is determined by multiplying the summer residential and commercial population equivalent by the textbook value of 0.2 ppcpd and the historic annual peaking factor of 0.98 (to change average dry weather to annual average). The industrial yearly average population equivalent is multiplied by 0.1623 ppcpd (provided by the City of Woodburn since 0.1623 ppcpd was used to derive the industrial population equivalent) to obtain the annual industrial TSS load. These two figures are added together to obtain the total annual TSS load. The 1993 calculated annual TSS load is 4,323 lb/day compared with an actual reported TSS load of 2,637 in 1993.

The peak month TSS load projections are contained in Table 3-18. The peak month residential and commercial load is determined by multiplying the summer residential and commercial population equivalent by the textbook value of 0.25 ppcpd. The industrial peak month population equivalent is multiplied by 0.1623 ppcpd (provided by the City of Woodburn) to obtain the peak month industrial TSS load. These two figures are added together to obtain the peak month TSS load. The calculated peak month TSS load for 1993 is 5,726 lb/day compared to the actual dry weather maximum month TSS load of 3,518 lb/day.

The worst case scenario TSS load projections are contained in Table 3-19. The peak day residential and commercial load is determined by multiplying the summer residential and commercial population equivalent by the textbook value of 0.2 ppcpd and the historic

Table 3-16
Historical TSS Load Peaking Factors
Woodburn WWTP
1990-1993

TSS Load Condition	1990 Load (lb/day)	1990 Peaking Factor	1991 Load (lb/day)	1991 Peaking Factor	1992 Load (lb/day)	1992 Peaking Factor	1993 Load (lb/day)	1993 Peaking Factor	Average Load (lb/day)	Average Peaking Factor
Average Dry Weather	2,688	1.00	2,560	1.00	3,115	1.00	2,327	1.00	2,673	1.00
Average Wet Weather	2,421	0.90	2,396	0.94	2,356	0.76	2,946	1.27	2,530	0.96
Annual Average Daily	2,555	0.95	2,478	0.97	2,736	0.88	2,637	1.13	2,601	0.98
Dry Weather Max Month Average Daily	3,073	1.14	3,300	1.29	3,574	1.15	3,116	1.34	3,266	1.23
Wet Weather Max Month Average Daily	2,762	1.03	2,825	1.10	3,172	1.02	3,518	1.51	3,069	1.17
Annual Max Month Average Daily	3,073	1.14	3,300	1.29	3,574	1.15	3,518	1.51	3,366	1.27
Dry Weather Maximum Daily	3,835	1.43	5,036	1.97	7,499	2.41	5,217	2.24	5,397	2.01
Wet Weather Maximum Daily	4,469	1.66	5,444	2.13	9,025	2.90	6,306	2.71	6,311	2.35

Table 5-17
Projected Annual TSS Loads
Woodburn WWTP
1993-2020

	Summer Residential & Commercial Population Equivalent (a)	*Annual Residential & Commercial TSS Load (b) (lb/day)*	*Industrial Yearly Average Population Equivalent (a)*	*Annual Industrial TSS Load (c) (lb/day)*	*Total Annual TSS Load (d) (lb/day)*
1993	20,401	3,999	2,000	325	4,323
1994	21,081	4,132	2,020	328	4,460
1995	21,784	4,270	2,040	331	4,601
1996	22,511	4,412	2,061	334	4,747
1997	23,263	4,560	2,081	338	4,897
1998	24,170	4,737	2,102	341	5,079
1999	24,974	4,895	2,123	345	5,240
2000	25,805	5,058	2,144	348	5,406
2001	27,115	5,314	2,166	351	5,666
2002	28,003	5,489	2,187	355	5,844
2003	28,922	5,669	2,209	359	6,027
2004	29,872	5,855	2,231	362	6,217
2005	30,854	6,047	2,254	366	6,413
2006	31,870	6,247	2,276	369	6,616
2007	32,920	6,452	2,299	373	6,826
2008	34,006	6,665	2,322	377	7,042
2009	35,129	6,885	2,345	381	7,266
2010	36,290	7,113	2,369	384	7,497
2011	37,491	7,348	2,392	388	7,736
2012	38,732	7,592	2,416	392	7,984
2013	40,016	7,843	2,440	396	8,239
2014	41,343	8,103	2,465	400	8,503
2015	42,715	8,372	2,489	404	8,776
2016	44,134	8,650	2,514	408	9,058
2017	45,602	8,938	2,539	412	9,350
2018	47,119	9,235	2,565	416	9,652
2019	48,687	9,543	2,591	420	9,963
2020	50,309	9,861	2,616	425	10,285

Notes:
(a) From City of Woodburn.
(b) Based on 0.2 ppcpd (textbook value) TSS load and 0.98 historic peaking factor.
(c) Based on 0.1623 ppcpd from the City of Woodburn.
(d) Septage is not accounted for in this table.

Table 3-18
Projected Peak Month TSS Loads
Woodburn WWTP
1993-2020

	Summer Residential & Commercial Population Equivalent (a)	*Peak Month Residential & Commercial TSS Load (b) (lb/day)*	*Industrial Peak Month Population Equivalent (a)*	*Peak Month Industrial TSS Load (c) (lb/day)*	*Peak Month TSS Load (lb/day)*
1993	20,401	5,100	3,857	626	5,726
1994	21,081	5,270	3,896	632	5,903
1995	21,784	5,446	3,935	639	6,085
1996	22,511	5,628	3,974	645	6,273
1997	23,263	5,816	4,014	651	6,467
1998	24,170	6,043	4,054	658	6,701
1999	24,974	6,244	4,094	665	6,908
2000	25,805	6,451	4,135	671	7,122
2001	27,115	6,779	4,177	678	7,457
2002	28,003	7,001	4,218	685	7,685
2003	28,922	7,230	4,261	691	7,922
2004	29,872	7,468	4,303	698	8,166
2005	30,854	7,714	4,346	705	8,419
2006	31,870	7,968	4,390	712	8,680
2007	32,920	8,230	4,434	720	8,950
2008	34,006	8,502	4,478	727	9,228
2009	35,129	8,782	4,523	734	9,516
2010	36,290	9,073	4,568	741	9,814
2011	37,491	9,373	4,614	749	10,121
2012	38,732	9,683	4,660	756	10,439
2013	40,016	10,004	4,706	764	10,768
2014	41,343	10,336	4,753	771	11,107
2015	42,715	10,679	4,801	779	11,458
2016	44,134	11,034	4,849	787	11,821
2017	45,602	11,400	4,897	795	12,195
2018	47,119	11,780	4,946	803	12,582
2019	48,687	12,172	4,996	811	12,983
2020	50,309	12,577	5,046	819	13,396

Notes:
(a) From City of Woodburn.
(b) Based on 0.25 ppcpd (textbook value) TSS load.
(c) Based on 0.1623 ppcpd from the City of Woodburn (Appendix 3.2).
(d) Septage is not accounted for in this table.

Table 3-19
Projected Worst Case Scenario TSS Loads
Woodburn WWTP
1993-2020

	Summer Residential & Commercial Population Equivalent (a)	*Peak Day (w/o Industry) Res & Comm TSS Load (b) (lb/day)*	*Industrial Worst Case Scenario Population Equivalent (a)*	*Worst Case Scenario Industrial TSS Load (c) (lb/day)*	*Worst Case Scenario TSS Load (d) (lb/day)*
1993	20,401	9,588	11,916	1,934	11,522
1994	21,081	9,908	12,035	1,953	11,861
1995	21,784	10,239	12,156	1,973	12,211
1996	22,511	10,580	12,277	1,993	12,573
1997	23,263	10,934	12,400	2,012	12,946
1998	24,170	11,360	12,524	2,033	13,393
1999	24,974	11,738	12,649	2,053	13,791
2000	25,805	12,128	12,776	2,073	14,202
2001	27,115	12,744	12,903	2,094	14,838
2002	28,003	13,161	13,032	2,115	15,277
2003	28,922	13,593	13,163	2,136	15,730
2004	29,872	14,040	13,294	2,158	16,198
2005	30,854	14,502	13,427	2,179	16,681
2006	31,870	14,979	13,562	2,201	17,180
2007	32,920	15,473	13,697	2,223	17,696
2008	34,006	15,983	13,834	2,245	18,228
2009	35,129	16,511	13,972	2,268	18,778
2010	36,290	17,056	14,112	2,290	19,347
2011	37,491	17,621	14,253	2,313	19,934
2012	38,732	18,204	14,396	2,336	20,541
2013	40,016	18,807	14,540	2,360	21,167
2014	41,343	19,431	14,685	2,383	21,815
2015	42,715	20,076	14,832	2,407	22,483
2016	44,134	20,743	14,980	2,431	23,174
2017	45,602	21,433	15,130	2,456	23,888
2018	47,119	22,146	15,281	2,480	24,626
2019	48,687	22,883	15,434	2,505	25,388
2020	50,309	23,645	15,589	2,530	26,175

Notes:
(a) From City of Woodburn.
(b) Based on 0.2 ppcpd (textbook value) BOD load and 2.35 historic peaking factor (peak day without industry).
(c) Based on 0.1623 ppcpd from the City of Woodburn (Appendix 3.2).
(d) Septage is not accounted for in this table.

peaking factor of 2.35 (peak day without industry). The industrial worst case scenario population equivalent is multiplied by 0.1623 ppcpd (provided by the City of Woodburn) to obtain the worst case scenario industrial TSS load. These two figures are added together to obtain the worst case scenario TSS load. The calculated worst case scenario for 1993 TSS load is 11,522 lb/day compared with the actual 1993 TSS load of 5,217 lb/day and the maximum TSS load of 9,025 lb/day for the period of record (February 1992).

Load projections for the year 2020 were also calculated using peaking factors times the annual average loads projected by population equivalents. These are shown in Table 3-20. Similar results were obtained with the two methods. The population equivalent method will be used for the loads. However, the worst case scenario of the population equivalent method will be considered unlikely but possible for short periods.

Septage

The *Regional Service Provider Study* conducted by Gordon Merseth Engineering (now Crane/Merseth Engineering) concluded that the planning criteria should include septage from the region. The quantity of septage recommended from the report can be contained in two 25,000 gallon tanks. The loading is assumed to be 1,549 lb/day BOD_5 (1,100 lb/day carbonaceous BOD) and 1,549 lb/day TSS. For the year 2020, the design loadings for BOD_5 including septage are:

- BOD_5 Maximum Month 14,781 lb/day
- BOD_5 Septage 1,549 lb/day
- BOD_5 Total 16,330 lb/day
- TSS Maximum Month 13,396 lb/day
- TSS Septage 1,549 lb/day
- TSS Total 14,985 lb/day

Recommended Design Flows and Loadings

The recommended design flows through 2020 are summarized in Table 3-21. The recommended design loads are shown in Table 3-22.

The average dry weather and the maximum month values for both BOD_5 and TSS will be used in designing the secondary treatment and solids handling facilities.

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Comparison of Loads
Woodburn WWTP
2020

	2020 Using Population Equivalents (lb/day)	*2020 Using Peaking Factors (lb/day)*
Total Annual		
BOD	10,485	10,485
TSS	10,285	10,285
Peak Month		
BOD	14,781	15,726
TSS	13,396	12,909
Worst Case Scenario		
BOD	34,288	25,185
TSS	26,175	24,663

Table 3-21
Recommended Design Flows
Woodburn WWTP
1993-2020

Flow Condition (mgd)	*Existing*	*1995*	*2000*	*2005*	*2010*	*2015*	*2020*
Annual							
Average Day							
(ADWF x 1.12)	2.680	2.823	3.223	3.693	4.246	4.896	5.661
Dry Weather							
Average Day							
(ADWF)	2.385	2.512	2.868	3.286	3.778	4.356	5.037
Maximum Month							
(ADWF x 1.24)	2.949	3.106	3.546	4.063	4.671	5.386	6.228
Maximum Day							
(ADWF x 2.27)	5.410	5.698	6.505	7.454	8.569	9.881	11.426
Wet Weather							
Average Day							
(ADWF x 1.25)	2.976	3.134	3.578	4.100	4.713	5.435	6.285
Maximum Month							
(ADWF x 1.74)	4.153	4.375	4.995	5.723	6.579	7.587	8.772
Maximum Day (WWMDF)							
(ADWF x 2.93)	6.982	7.355	8.397	9.621	11.060	12.754	14.747
Peak							
(WWMDF x 1.5)	10.473	11.032	12.595	14.431	16.591	19.131	22.121

Table 3-22

Recommended Design Loads
Woodburn WWTP
1993-2020

Load Condition (lb/day)	*Actual 1993*	*Calculated 1993*	*2020*
Total Annual			
BOD_5	—	4,560	10,485
TSS	—	4,323	10,285
Annual Average Daily			
BOD_5	4,279	—	—
TSS	2,637	—	—
Peak Month			
BOD_5	—	6,605	14,781
TSS	—	5,726	13,396
Annual Max Month Avg Daily			
BOD_5	8,486	—	—
TSS	3,518	—	—
Worst Case Scenario			
BOD_5	—	19,338	34,288
TSS	—	11,522	26,175
Maximum Daily			
BOD_5 (Dry Weather)	10,575	—	—
TSS (Wet Weather)	6,306	—	—
Septage			1,549

Appendix 3.1

Historical Woodburn WWTP Data Used to Project Flows and Loads

Appendix 3.1

The following steps were followed to obtain an estimate of the per capita flow for comparison with the City of Woodburn's 1993 estimate (Table 3.1-1).

1. The total flow (column 2) is multiplied by 74.2% (see "Flow Components" in Section 3) to obtain the residential flow (column 3).

2. The annual permanent population (column 4) is provided by the Portland State University (P.S.U.) Population Research Center. This population added to the migrant (summer) population and the population served by the WWTP but outside the UGB (column 5) results in the residential population for the dry weather flow period (column 6).

3. The residential component of the total flow is divided by this summer population to obtain the dry weather per capita flow rate (column 7). The average dry weather per capita flow rate for 1989 through 1993 is 85 gallons per capita per day (gpcpd).

4. Using the City of Woodburn population estimate for 1993 (column 8), results in a slightly lower average of 84 gpcpd (column 9) for the 5 year period. These two per capita figures compare well with the City of Woodburn per capita estimate of 82 gpcpd. This per capita estimate is used in the flow projections.

5. Peak per capita loadings of 94 gpcpd occurred in 1990. If peak conditions were realized in the future, the design flow of 5.0 mgd would be reached in the year 2020, 20 years from 1995.

Table 3.1-1
Dry Weather Per Capita Flow
Woodburn WWTP
1989-1993

Month/Year	*Average Daily Flow (a) (mgd)*	*Avg. Daily Residential Flow (b) (mgd)*	*P.S.U. Census Population*	*Other (c) Population*	*Total Population*	*Residential Per Capita Flow (gpcpd)*	*Population from the City of Woodburn*	*Residential Per Capita Flow (gpcpd)*
May-89	1.785	1.324	12,445	2,062	14,507	91	14,507	91
Jun-89	1.869	1.387	12,445	2,062	14,507	96	14,507	96
Jul-89	1.826	1.355	12,445	2,062	14,507	93	14,507	93
Aug-89	1.829	1.357	12,445	2,062	14,507	94	14,507	94
Sep-89	1.546	1.147	12,445	2,062	14,507	79	14,507	79
Oct-89	1.557	1.155	12,445	2,062	14,507	80	14,507	80
1989 Dry Avg.	**1.735**	**1.288**	**12,445**	**2,062**	**14,507**	**89**	**14,507**	**89**
May-90	2.035	1.510	13,404	2,062	15,466	98	15,466	98
Jun-90	2.081	1.544	13,404	2,062	15,466	100	15,466	100
Jul-90	1.801	1.336	13,404	2,062	15,466	86	15,466	86
Aug-90	1.792	1.330	13,404	2,062	15,466	86	15,466	86
Sep-90	2.219	1.646	13,404	2,062	15,466	106	15,466	106
Oct-90	1.833	1.360	13,404	2,062	15,466	88	15,466	88
1990 Dry Avg.	**1.960**	**1.454**	**13,404**	**2,062**	**15,466**	**94**	**15,466**	**94**
May-91	2.649	1.966	13,525	2,062	15,587	126	15,587	126
Jun-91	2.011	1.492	13,525	2,062	15,587	96	15,587	96
Jul-91	1.727	1.281	13,525	2,062	15,587	82	15,587	82
Aug-91	1.625	1.206	13,525	2,062	15,587	77	15,587	77
Sep-91	1.520	1.128	13,525	2,062	15,587	72	15,587	72
Oct-91	1.613	1.197	13,525	2,062	15,587	77	15,587	77
1991 Dry Avg.	**1.858**	**1.378**	**13,525**	**2,062**	**15,587**	**88**	**15,587**	**88**
May-92	1.819	1.350	14,005	2,062	16,067	84	16,067	84
Jun-92	1.083	0.804	14,005	2,062	16,067	50	16,067	50
Jul-92	1.825	1.354	14,005	2,062	16,067	84	16,067	84
Aug-92	1.461	1.084	14,005	2,062	16,067	67	16,067	67
Sep-92	1.428	1.060	14,005	2,062	16,067	66	16,067	66
Oct-92	1.249	0.927	14,005	2,062	16,067	58	16,067	58
1992 Dry Avg.	**1.478**	**1.096**	**14,005**	**2,062**	**16,067**	**68**	**16,067**	**68**
May-93	2.258	1.675	14,055	2,062	16,117	104	17,707	95
Jun-93	2.455	1.822	14,055	2,062	16,117	113	17,707	103
Jul-93	1.831	1.359	14,055	2,062	16,117	84	17,707	77
Aug-93	1.663	1.234	14,055	2,062	16,117	77	17,707	70
Sep-93	1.533	1.137	14,055	2,062	16,117	71	17,707	64
Oct-93	1.486	1.103	14,055	2,062	16,117	68	17,707	62
1993 Dry Avg.	**1.871**	**1.388**	**14,055**	**2,062**	**16,117**	**86**	**17,707**	**78**
Average	**1.780**	**1.321**	**13,487**	**2,062**	**15,549**	**85**	**15,867**	**84**

Notes:
(a) Based on historical records.
(b) Based on a residential flow percentage of 74.2% from the City of Woodburn.
(c) Includes migrant population, Carl Road Mobile Home units, Maclaren School and Shalimar Mobile Home units.

Woodburn WWTP
January - May, 1989-1993

Date	Daily Influent Flow (a) (mgd)	Daily Effluent Flow (a) (mgd)	Monthly Cumulative Flow (b) (mgd)	Daily Rainfall (c) (inches)	Monthly Cumulative Rainfall (c) (inches)	Rainfall-Flow Trend Line: Daily Flow (mgd)	Rainfall-Flow Trend Line: Daily Rainfall (inches)
1-Jan-89		1.481		0.08		2.17	0.00
2-Jan-89		1.572		0.42		2.89	0.50
3-Jan-89		1.664		0.01		3.60	1.00
4-Jan-89		2.149		0.31		4.31	1.50
5-Jan-89		1.789		0.49		5.03	2.00
6-Jan-89		1.613		0.08		5.74	2.50
7-Jan-89		1.515		0.01		6.45	3.00
8-Jan-89		1.798		0.21			
9-Jan-89		2.988		0.88			
10-Jan-89		2.121		0.00			
11-Jan-89		1.911		0.00			
12-Jan-89		2.046		0.00			
13-Jan-89		2.471		0.75			
14-Jan-89		2.266		0.38			
15-Jan-89		2.113		0.06			
16-Jan-89		2.218		0.07			
17-Jan-89		1.819		0.20			
18-Jan-89		1.955		0.00			
19-Jan-89		1.850		0.00			
20-Jan-89		1.847		0.00			
21-Jan-89		2.357		0.65			
22-Jan-89		1.998		0.26			
23-Jan-89		1.933		0.00			
24-Jan-89		1.651		0.01			
25-Jan-89		1.756		0.00			
26-Jan-89		1.647		0.00			
27-Jan-89		1.675		0.05			
28-Jan-89		1.637		0.00			
29-Jan-89		1.555		0.00			
30-Jan-89		1.805		0.00			
31-Jan-89		1.543	58.730	0.26	5.180		
1-Feb-89		1.603		0.11			
2-Feb-89		1.383		0.19			
3-Feb-89		1.625		0.00			
4-Feb-89		1.621		0.00			
5-Feb-89		1.665		0.00			
6-Feb-89		1.586		0.00			
7-Feb-89		1.622		0.00			
8-Feb-89		1.665		0.00			
9-Feb-89		1.741		0.00			
10-Feb-89		1.501		0.01			
11-Feb-89		1.534		0.01			
12-Feb-89		1.412		0.01			
13-Feb-89		1.595		0.02			
14-Feb-89		1.674		0.00			
15-Feb-89		1.621		0.00			
16-Feb-89		2.338		0.85			
17-Feb-89		2.750		0.11			
18-Feb-89		3.305		0.55			
19-Feb-89		2.902		0.30			
20-Feb-89		2.854		0.00			
21-Feb-89		2.682		0.10			
22-Feb-89		2.882		0.42			
23-Feb-89		2.635		0.50			
24-Feb-89		2.406		0.00			
25-Feb-89		2.331		0.00			
26-Feb-89		2.897		0.00			
27-Feb-89		2.266		0.00			
28-Feb-89		2.287	57.585	0.00	3.180		
1-Mar-89		2.321		0.24			
2-Mar-89		2.181		0.00			
3-Mar-89		2.046		0.00			
4-Mar-89		2.070		0.03			
5-Mar-89		3.809		1.10			
6-Mar-89		3.788		0.51			
7-Mar-89		3.138		0.02			
8-Mar-89		3.036		0.00			
9-Mar-89		3.163		0.25			
10-Mar-89		2.853		0.01			
11-Mar-89		2.780		0.27			

Table 3.1-2
Daily Rainfall and Daily Plant Flow
Woodburn WWTP
January - May, 1989-1993

Date	Daily Influent Flow (a) (mgd)	Daily Effluent Flow (a) (mgd)	Monthly Cumulative Flow (b) (mgd)	Daily Rainfall (c) (inches)	Monthly Cumulative Rainfall (c) (inches)	Rainfall-Flow Trend Line: Daily Flow (mgd)	Rainfall-Flow Trend Line: Daily Rainfall (inches)
12-Mar-89		3.536		0.61			
13-Mar-89		3.295		0.45			
14-Mar-89		3.103		0.00			
15-Mar-89		3.727		0.23			
16-Mar-89		3.774		0.30			
17-Mar-89		3.903		0.32			
18-Mar-89		3.530		0.14			
19-Mar-89		3.133		0.01			
20-Mar-89		3.375		0.09			
21-Mar-89		3.172		0.28			
22-Mar-89		2.901		0.01			
23-Mar-89		2.873		0.00			
24-Mar-89		3.161		0.39			
25-Mar-89		3.636		0.80			
26-Mar-89		3.168		0.00			
27-Mar-89		3.200		0.19			
28-Mar-89		3.231		0.30			
29-Mar-89		3.030		0.00			
30-Mar-89		2.891		0.16			
31-Mar-89		2.762	96.618	0.07	6.780		
1-Apr-89		2.801		0.22			
2-Apr-89		2.769		0.57			
3-Apr-89		2.555		0.02			
4-Apr-89		2.847		0.01			
5-Apr-89		2.561		0.00			
6-Apr-89		2.598		0.00			
7-Apr-89		2.547		0.00			
8-Apr-89		2.483		0.00			
9-Apr-89		2.370		0.00			
10-Apr-89		2.278		0.00			
11-Apr-89		2.158		0.00			
12-Apr-89		2.204		0.00			
13-Apr-89		2.180		0.00			
14-Apr-89		2.087		0.00			
15-Apr-89		1.756		0.00			
16-Apr-89		1.954		0.00			
17-Apr-89		2.239		0.00			
18-Apr-89		2.190		0.00			
19-Apr-89		2.315		0.16			
20-Apr-89		1.983		0.13			
21-Apr-89		2.037		0.02			
22-Apr-89		1.910		0.25			
23-Apr-89		2.057		0.17			
24-Apr-89		1.952		0.04			
25-Apr-89		1.882		0.23			
26-Apr-89		1.818		0.20			
27-Apr-89		1.873		0.00			
28-Apr-89		1.876		0.00			
29-Apr-89		1.848		0.00			
30-Apr-89		1.796	65.744	0.00	2.020		
1-May-89		2.421		0.01			
2-May-89		1.968		0.00			
3-May-89		1.978		0.00			
4-May-89		1.978		0.00			
5-May-89		2.000		0.00			
6-May-89		1.810		0.00			
7-May-89		1.804		0.00			
8-May-89		1.941		0.00			
9-May-89		1.970		0.02			
10-May-89		1.843		0.00			
11-May-89		1.414		0.00			
12-May-89		1.886		0.00			
13-May-89		1.658		0.00			
14-May-89		1.715		0.00			
15-May-89		1.595		0.00			
16-May-89		1.633		0.00			
17-May-89		1.598		0.13			
18-May-89		1.849		0.18			
19-May-89		1.704		0.01			
20-May-89		1.621		0.00			

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Daily Rainfall and Daily Plant Flow
Woodburn WWTP
January - May, 1989-1993

Date	Daily Influent Flow (a) (mgd)	Daily Effluent Flow (a) (mgd)	Monthly Cumulative Flow (b) (mgd)	Daily Rainfall (c) (inches)	Monthly Cumulative Rainfall (c) (inches)	Rainfall-Flow Trend Line: Daily Flow (mgd)	Rainfall-Flow Trend Line: Daily Rainfall (inches)
21-May-89		1.586		0.00			
22-May-89		1.895		0.00			
23-May-89		1.680		0.38			
24-May-89		1.880		0.50			
25-May-89		1.816		0.05			
26-May-89		1.829		0.00			
27-May-89		1.705		0.94			
28-May-89		1.511		0.00			
29-May-89		1.590		0.01			
30-May-89		1.939		0.00			
31-May-89		1.827	55.644	0.00	2.230		
1-Jan-90		2.098		0.00			
2-Jan-90		1.898		0.01			
3-Jan-90		1.848		0.08			
4-Jan-90		1.968		0.19			
5-Jan-90		2.061		0.27			
6-Jan-90		2.946		0.53			
7-Jan-90		3.827		1.56			
8-Jan-90		3.423		0.85			
9-Jan-90		3.578		0.87			
10-Jan-90		3.006		0.41			
11-Jan-90		2.567		0.05			
12-Jan-90		2.544		0.00			
13-Jan-90		2.330		0.07			
14-Jan-90		2.285		0.02			
15-Jan-90		2.346		0.05			
16-Jan-90		2.258		0.20			
17-Jan-90		2.146		0.00			
18-Jan-90		2.189		0.02			
19-Jan-90		2.107		0.00			
20-Jan-90		2.008		0.00			
21-Jan-90		1.986		0.00			
22-Jan-90		2.272		0.29			
23-Jan-90		2.157		0.43			
24-Jan-90		2.122		0.00			
25-Jan-90		2.752		0.90			
26-Jan-90		2.592		0.45			
27-Jan-90		2.742		0.27			
28-Jan-90		4.262		1.26			
29-Jan-90		3.809		0.24			
30-Jan-90		3.181		0.29			
31-Jan-90		3.123	80.511	0.34	9.650		
1-Feb-90		3.671		0.38			
2-Feb-90		3.211		0.12			
3-Feb-90		3.956		0.73			
4-Feb-90		3.357		0.01			
5-Feb-90		3.584		0.28			
6-Feb-90		3.508		0.18			
7-Feb-90		3.606		0.29			
8-Feb-90		3.930		0.58			
9-Feb-90		3.900		0.20			
10-Feb-90		3.551		0.09			
11-Feb-90		3.196		0.24			
12-Feb-90		3.312		0.39			
13-Feb-90		3.191		0.00			
14-Feb-90		2.992		0.00			
15-Feb-90		3.618		0.68			
16-Feb-90		3.762		0.31			
17-Feb-90		3.283		0.00			
18-Feb-90		2.880		0.00			
19-Feb-90		3.872		0.00			
20-Feb-90		3.533		0.00			
21-Feb-90		3.272		0.10			
22-Feb-90		3.175		0.00			
23-Feb-90		3.013		0.00			
24-Feb-90		3.000		0.00			
25-Feb-90		2.923		0.00			
26-Feb-90		2.893		0.00			
27-Feb-90		2.867		0.00			
28-Feb-90		2.635	92.891	0.00	4.580		

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Table 3.1-2
Daily Rainfall and Daily Plant Flow
Woodburn WWTP
January - May, 1989-1993

Date	Daily Influent Flow (a) (mgd)	Daily Effluent Flow (a) (mgd)	Monthly Cumulative Flow (b) (mgd)	Daily Rainfall (c) (inches)	Monthly Cumulative Rainfall (c) (inches)	Rainfall-Flow Trend Line: Daily Flow (mgd)	Rainfall-Flow Trend Line: Daily Rainfall (inches)
1-Mar-90		2.561		0.00			
2-Mar-90		2.610		0.00			
3-Mar-90		2.740		0.19			
4-Mar-90		2.973		0.31			
5-Mar-90		2.316		0.03			
6-Mar-90		2.858		0.02			
7-Mar-90		3.144		0.77			
8-Mar-90		2.768		0.26			
9-Mar-90		3.446		0.16			
10-Mar-90		3.314		0.14			
11-Mar-90		3.046		0.13			
12-Mar-90		3.014		0.00			
13-Mar-90		2.448		0.11			
14-Mar-90		2.493		0.36			
15-Mar-90		2.346		0.00			
16-Mar-90		2.361		0.00			
17-Mar-90		2.282		0.01			
18-Mar-90		2.122		0.00			
19-Mar-90		2.312		0.00			
20-Mar-90		2.240		0.08			
21-Mar-90		2.261		0.00			
22-Mar-90		2.523		0.21			
23-Mar-90		2.308		0.00			
24-Mar-90		2.036		0.00			
25-Mar-90		2.122		0.00			
26-Mar-90		2.132		0.00			
27-Mar-90		2.255		0.00			
28-Mar-90		1.929		0.00			
29-Mar-90		1.930		0.00			
30-Mar-90		1.957		0.00			
31-Mar-90		1.798	76.645	0.00	2.780		
1-Apr-90		1.790		0.00			
2-Apr-90		1.988		0.00			
3-Apr-90		2.041		0.00			
4-Apr-90		2.102		0.00			
5-Apr-90		1.901		0.00			
6-Apr-90		1.892		0.00			
7-Apr-90		1.744		0.00			
8-Apr-90		1.675		0.00			
9-Apr-90		1.651		0.00			
10-Apr-90		1.770		0.00			
11-Apr-90		1.670		0.07			
12-Apr-90		1.936		0.00			
13-Apr-90		1.777		0.00			
14-Apr-90		1.879		0.00			
15-Apr-90		1.849		0.00			
16-Apr-90		1.849		0.00			
17-Apr-90		1.968		0.15			
18-Apr-90		1.911		0.07			
19-Apr-90		2.053		0.00			
20-Apr-90		2.157		0.00			
21-Apr-90		1.919		0.00			
22-Apr-90		2.236		0.45			
23-Apr-90		2.198		0.13			
24-Apr-90		1.781		0.08			
25-Apr-90		1.678		0.24			
26-Apr-90		1.868		0.01			
27-Apr-90		2.594		1.30			
28-Apr-90		1.841		0.09			
29-Apr-90		1.815		0.00			
30-Apr-90		1.898	57.431	0.00	2.590		
1-May-90		2.297		0.00			
2-May-90		1.648		0.00			
3-May-90		1.821		0.00			
4-May-90		1.829		0.00			
5-May-90		1.740		0.00			
6-May-90		1.730		0.00			
7-May-90		1.798		0.00			
8-May-90		1.862		0.00			
9-May-90		1.729		0.00			

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Daily Rainfall and Daily Plant Flow
Woodburn WWTP
January - May, 1989-1993

Date	Daily Influent Flow (a) (mgd)	Daily Effluent Flow (a) (mgd)	Monthly Cumulative Flow (b) (mgd)	Daily Rainfall (c) (inches)	Monthly Cumulative Rainfall (c) (inches)	Rainfall-Flow Trend Line: Daily Flow (mgd)	Rainfall-Flow Trend Line: Daily Rainfall (inches)
10-May-90		1.783		0.00			
11-May-90		1.795		0.00			
12-May-90		1.628		0.00			
13-May-90		1.628		0.15			
14-May-90		1.792		0.00			
15-May-90		1.655		0.00			
16-May-90		1.632		0.00			
17-May-90		1.637		0.00			
18-May-90		1.517		0.02			
19-May-90		1.510		0.00			
20-May-90		1.520		0.28			
21-May-90		1.770		0.10			
22-May-90		2.969		0.24			
23-May-90		3.524		0.25			
24-May-90		3.341		0.19			
25-May-90		2.480		0.19			
26-May-90		2.407		0.01			
27-May-90		2.329		0.22			
28-May-90		2.396		0.19			
29-May-90		2.425		0.04			
30-May-90		2.452		0.04			
31-May-90		2.434	63.078	0.00	1.920		
1-Jan-91		1.292		0.02			
2-Jan-91		1.242		0.00			
3-Jan-91		1.289		0.00			
4-Jan-91		1.245		0.00			
5-Jan-91		1.280		0.00			
6-Jan-91		1.556		0.04			
7-Jan-91		1.759		0.68			
8-Jan-91		1.604		0.02			
9-Jan-91		1.892		0.08			
10-Jan-91		2.016		0.52			
11-Jan-91		2.337		0.31			
12-Jan-91		2.632		0.45			
13-Jan-91		2.038		0.05			
14-Jan-91		2.975		0.28			
15-Jan-91		2.506		0.98			
16-Jan-91		2.241		0.00			
17-Jan-91		2.086		0.00			
18-Jan-91		1.982		0.00			
19-Jan-91		1.871		0.00			
20-Jan-91		1.611		0.00			
21-Jan-91		1.882		0.00			
22-Jan-91		1.701		0.00			
23-Jan-91		1.602		0.00			
24-Jan-91		1.608		0.00			
25-Jan-91		1.583		0.00			
26-Jan-91		1.436		0.00			
27-Jan-91		1.447		0.00			
28-Jan-91		1.526		0.00			
29-Jan-91		1.510		0.00			
30-Jan-91		1.617		0.00			
31-Jan-91		1.702	55.068	0.27	3.700		
1-Feb-91		1.827		0.01			
2-Feb-91		2.202		0.44			
3-Feb-91		2.139		0.24			
4-Feb-91		2.758		0.51			
5-Feb-91		2.413		0.23			
6-Feb-91		2.152		0.00			
7-Feb-91		2.104		0.00			
8-Feb-91		1.983		0.00			
9-Feb-91		1.854		0.00			
10-Feb-91		1.710		0.00			
11-Feb-91		1.947		0.05			
12-Feb-91		2.272		0.33			
13-Feb-91		2.181		0.36			
14-Feb-91		2.121		0.03			
15-Feb-91		2.091		0.06			
16-Feb-91		1.942		0.18			
17-Feb-91		1.758		0.00			

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Table 3.1-2
Daily Rainfall and Daily Plant Flow
Woodburn WWTP
January - May, 1989-1993

Date	Daily Influent Flow (a) (mgd)	Daily Effluent Flow (a) (mgd)	Monthly Cumulative Flow (b) (mgd)	Daily Rainfall (c) (inches)	Monthly Cumulative Rainfall (c) (inches)	Rainfall-Flow Trend Line: Daily Flow (mgd)	Rainfall-Flow Trend Line: Daily Rainfall (inches)
18-Feb-91		1.986		0.10			
19-Feb-91		1.965		0.02			
20-Feb-91		2.716		1.70			
21-Feb-91		2.308		0.00			
22-Feb-91		2.158		0.00			
23-Feb-91		1.964		0.00			
24-Feb-91		1.936		0.00			
25-Feb-91		2.016		0.00			
26-Feb-91		2.004		0.00			
27-Feb-91		1.949		0.00			
28-Feb-91		1.975	58.511	0.00	4.260		
1-Mar-91		1.983		0.03			
2-Mar-91		2.244		0.58			
3-Mar-91		2.745		0.58			
4-Mar-91		2.823		0.75			
5-Mar-91		3.104		0.02			
6-Mar-91		2.918		0.21			
7-Mar-91		2.678		0.10			
8-Mar-91		2.609		0.00			
9-Mar-91		2.734		0.25			
10-Mar-91		2.745		0.19			
11-Mar-91		2.924		0.19			
12-Mar-91		2.838		0.10			
13-Mar-91		2.718		0.00			
14-Mar-91		2.558		0.07			
15-Mar-91		2.421		0.00			
16-Mar-91		2.316		0.00			
17-Mar-91		2.218		0.00			
18-Mar-91		2.207		0.15			
19-Mar-91		2.139		0.00			
20-Mar-91		2.185		0.12			
21-Mar-91		1.864		0.17			
22-Mar-91		2.128		0.00			
23-Mar-91		2.256		0.00			
24-Mar-91		2.178		0.16			
25-Mar-91		2.214		0.04			
26-Mar-91		2.181		0.00			
27-Mar-91		2.067		0.00			
28-Mar-91		1.864		0.00			
29-Mar-91		1.819		0.00			
30-Mar-91		1.575		0.00			
31-Mar-91		1.846	73.099	0.00	3.710		
1-Apr-91		1.937		0.03			
2-Apr-91		2.924		0.08			
3-Apr-91		2.901		0.08			
4-Apr-91		3.507		0.29			
5-Apr-91		2.718		1.00			
6-Apr-91		2.864		0.33			
7-Apr-91		2.685		0.35			
8-Apr-91		3.198		0.24			
9-Apr-91		3.078		0.40			
10-Apr-91		2.925		0.07			
11-Apr-91		2.770		0.00			
12-Apr-91		2.596		0.00			
13-Apr-91		2.541		0.00			
14-Apr-91		2.984		0.13			
15-Apr-91		2.658		0.02			
16-Apr-91		2.690		0.00			
17-Apr-91		2.476		0.16			
18-Apr-91		2.350		0.00			
19-Apr-91		2.366		0.00			
20-Apr-91		2.200		0.00			
21-Apr-91		2.076		0.00			
22-Apr-91		2.060		0.00			
23-Apr-91		2.085		0.00			
24-Apr-91		2.293		0.98			
25-Apr-91		2.241		0.19			
26-Apr-91		2.096		0.15			
27-Apr-91		2.061		0.01			
28-Apr-91		2.059		0.12			

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Daily Rainfall and Daily Flow
Woodburn WWTP
January - May, 1989-1993

Date	Daily Influent Flow (a) (mgd)	Daily Effluent Flow (a) (mgd)	Monthly Cumulative Flow (b) (mgd)	Daily Rainfall (c) (inches)	Monthly Cumulative Rainfall (c) (inches)	Rainfall-Flow Trend Line: Daily Flow (mgd)	Rainfall-Flow Trend Line: Daily Rainfall (inches)
29-Apr-91		2.062		0.00			
30-Apr-91		2.058	75.379	0.00	4.630		
1-May-91		1.939		0.00			
2-May-91		1.909		0.00			
3-May-91		1.768		0.00			
4-May-91		1.736		0.00			
5-May-91		1.969		0.05			
6-May-91		2.099		0.20			
7-May-91		2.861		0.52			
8-May-91		3.068		0.54			
9-May-91		2.704		0.22			
10-May-91		2.236		0.00			
11-May-91		2.320		0.00			
12-May-91		2.346		0.00			
13-May-91		2.213		0.01			
14-May-91		2.186		0.00			
15-May-91		2.172		0.00			
16-May-91		3.376		0.00			
17-May-91		6.141		1.90			
18-May-91		4.803		0.42			
19-May-91		3.968		0.00			
20-May-91		3.669		0.00			
21-May-91		3.015		0.00			
22-May-91		2.661		0.00			
23-May-91		2.603		0.00			
24-May-91		2.690		0.00			
25-May-91		2.420		0.16			
26-May-91		2.268		0.12			
27-May-91		1.541		0.00			
28-May-91		2.481		0.00			
29-May-91		2.512		0.25			
30-May-91		2.347		0.01			
31-May-91		2.160	82.128	0.00	4.400		
1-Jan-92	1.728	1.775		0.05			
2-Jan-92	2.063	2.274		0.60			
3-Jan-92	2.247	2.550		0.04			
4-Jan-92	1.536	2.398		0.30			
5-Jan-92	2.190	2.340		0.03			
6-Jan-92	2.273	2.252		0.00			
7-Jan-92	2.072	2.119		0.00			
8-Jan-92	2.042	2.040		0.00			
9-Jan-92	0.000	2.020		0.00			
10-Jan-92	2.393	2.439		0.48			
11-Jan-92	2.281	2.296		0.13			
12-Jan-92	2.595	2.076		0.00			
13-Jan-92	3.035	2.301		0.07			
14-Jan-92	3.035	2.233		0.10			
15-Jan-92	4.041	2.215		0.00			
16-Jan-92	2.736	3.676		0.75			
17-Jan-92	2.557	2.515		0.00			
18-Jan-92	2.412	2.249		0.00			
19-Jan-92	2.329	2.117		0.00			
20-Jan-92	2.368	2.314		0.00			
21-Jan-92	2.337	2.190		0.00			
22-Jan-92	2.194	2.231		0.07			
23-Jan-92	2.283	2.291		0.00			
24-Jan-92	2.304	2.366		0.25			
25-Jan-92	2.334	2.389		0.38			
26-Jan-92	3.602	2.275		0.01			
27-Jan-92	4.810	3.379		0.41			
28-Jan-92	3.401	3.600		0.52			
29-Jan-92	4.188	3.262		0.31			
30-Jan-92	4.480	4.188		0.63			
31-Jan-92	3.611	3.900	83.497	0.11	5.240		
1-Feb-92		3.362		0.09			
2-Feb-92		2.951		0.00			
3-Feb-92		3.233		0.00			
4-Feb-92		3.185		0.00			
5-Feb-92		2.505		0.00			
6-Feb-92		2.500		0.00			

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Table 3.1-2
Daily Rainfall and Daily Plant Flow
Woodburn WWTP
January - May, 1989-1993

Date	Daily Influent Flow (a) (mgd)	Daily Effluent Flow (a) (mgd)	Monthly Cumulative Flow (b) (mgd)	Daily Rainfall (c) (inches)	Monthly Cumulative Rainfall (c) (inches)	Rainfall-Flow Trend Line: Daily Flow (mgd)	Rainfall-Flow Trend Line: Daily Rainfall (inches)
7-Feb-92		2.474		0.01			
8-Feb-92		2.338		0.07			
9-Feb-92		2.431		0.11			
10-Feb-92		2.463		0.00			
11-Feb-92		2.406		0.04			
12-Feb-92		2.530		0.00			
13-Feb-92		2.407		0.14			
14-Feb-92		2.288		0.03			
15-Feb-92		2.271		0.01			
16-Feb-92		2.060		0.00			
17-Feb-92		2.458		0.11			
18-Feb-92		7.515		1.57			
19-Feb-92		7.920		0.72			
20-Feb-92		6.330		0.75			
21-Feb-92		6.012		0.28			
22-Feb-92		4.171		0.43			
23-Feb-92		4.275		0.02			
24-Feb-92		4.551		0.10			
25-Feb-92		3.275		0.00			
26-Feb-92		4.600		0.00			
27-Feb-92		3.773		0.00			
28-Feb-92		3.216		0.00			
29-Feb-92		2.818	102.318	0.00	4.480		
1-Mar-92		2.759		0.00			
2-Mar-92		3.113		0.00			
3-Mar-92		2.826		0.19			
4-Mar-92		3.240		0.30			
5-Mar-92		3.100		0.19			
6-Mar-92		2.840		0.04			
7-Mar-92		2.587		0.00			
8-Mar-92		2.478		0.00			
9-Mar-92		2.535		0.00			
10-Mar-92		2.350		0.00			
11-Mar-92		2.476		0.00			
12-Mar-92		2.651		0.00			
13-Mar-92		2.063		0.00			
14-Mar-92		2.377		0.00			
15-Mar-92		2.107		0.24			
16-Mar-92		2.211		0.08			
17-Mar-92		2.118		0.08			
18-Mar-92		2.117		0.00			
19-Mar-92		2.023		0.00			
20-Mar-92		2.195		0.00			
21-Mar-92		2.008		0.00			
22-Mar-92		1.976		0.00			
23-Mar-92		2.006		0.00			
24-Mar-92		1.961		0.00			
25-Mar-92		1.938		0.00			
26-Mar-92		1.974		0.00			
27-Mar-92		1.974		0.00			
28-Mar-92		1.892		0.00			
29-Mar-92		1.967		0.00			
30-Mar-92		2.070		0.00			
31-Mar-92		1.900	71.832	0.00	1.120		
1-Apr-92		1.900		0.00			
2-Apr-92		1.877		0.00			
3-Apr-92		1.797		0.00			
4-Apr-92		1.991		0.41			
5-Apr-92		2.063		0.39			
6-Apr-92		2.020		0.07			
7-Apr-92		1.858		0.00			
8-Apr-92		2.543		0.04			
9-Apr-92		2.674		0.80			
10-Apr-92		2.219		0.00			
11-Apr-92		2.326		0.08			
12-Apr-92		2.320		0.02			
13-Apr-92		2.430		0.37			
14-Apr-92		2.202		0.00			
15-Apr-92		2.254		0.00			
16-Apr-92		2.434		0.57			

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Daily Rainfall and Daily Plant Flow
Woodburn WWTP
January - May, 1989-1993

Date	Daily Influent Flow (a) (mgd)	Daily Effluent Flow (a) (mgd)	Monthly Cumulative Flow (b) (mgd)	Daily Rainfall (c) (inches)	Monthly Cumulative Rainfall (c) (inches)	Rainfall-Flow Trend Line: Daily Flow (mgd)	Rainfall-Flow Trend Line: Daily Rainfall (inches)
17-Apr-92		2.378		0.45			
18-Apr-92		2.236		0.00			
19-Apr-92		2.189		0.00			
20-Apr-92		2.236		0.06			
21-Apr-92		2.153		0.00			
22-Apr-92		2.060		0.00			
23-Apr-92		2.136		0.04			
24-Apr-92		2.138		0.00			
25-Apr-92		2.013		0.00			
26-Apr-92		1.910		0.00			
27-Apr-92		1.976		0.18			
28-Apr-92		2.126		0.00			
29-Apr-92		2.106		0.44			
30-Apr-92		2.191	64.753	1.18	5.100		
1-May-92		1.908		0.00			
2-May-92		1.859		0.00			
3-May-92		1.865		0.00			
4-May-92		2.118		0.00			
5-May-92		1.972		0.00			
6-May-92		2.036		0.00			
7-May-92		1.960		0.00			
8-May-92		1.845		0.03			
9-May-92		1.872		0.00			
10-May-92		1.664		0.00			
11-May-92		1.923		0.00			
12-May-92		1.934		0.00			
13-May-92		1.859		0.00			
14-May-92		1.883		0.00			
15-May-92		1.857		0.00			
16-May-92		1.723		0.00			
17-May-92		1.717		0.00			
18-May-92		1.748		0.00			
19-May-92		1.770		0.00			
20-May-92		1.768		0.00			
21-May-92		1.752		0.07			
22-May-92		1.854		0.00			
23-May-92		1.778		0.00			
24-May-92		1.650		0.00			
25-May-92		1.814		0.01			
26-May-92		1.754		0.05			
27-May-92		1.774		0.00			
28-May-92		1.760		0.00			
29-May-92		1.713		0.00			
30-May-92		1.654		0.00			
31-May-92		1.618	56.399	0.00	0.160		
1-Jan-93	1.680	1.717		0.05			
2-Jan-93	1.643	1.320		0.08			
3-Jan-93	2.101	2.000		0.37			
4-Jan-93	2.439	2.246		0.20			
5-Jan-93	2.390	2.262		0.10			
6-Jan-93	2.182	2.090		0.00			
7-Jan-93	2.020	1.808		0.00			
8-Jan-93	1.840	1.694		0.00			
9-Jan-93	1.868	1.725		0.00			
10-Jan-93	1.633	1.466		0.00			
11-Jan-93	1.737	1.612		0.00			
12-Jan-93	1.620	1.591		0.00			
13-Jan-93	1.650	1.571		0.02			
14-Jan-93	1.660	1.590		0.00			
15-Jan-93	1.720	1.683		0.02			
16-Jan-93	1.710	1.629		0.03			
17-Jan-93	1.648	1.566		0.00			
18-Jan-93	1.749	1.761		0.00			
19-Jan-93	3.053	2.886		0.90			
20-Jan-93	3.211	3.138		0.75			
21-Jan-93	2.894	2.815		0.31			
22-Jan-93	2.702	2.666		0.25			
23-Jan-93	2.651	2.519		0.16			
24-Jan-93	2.477	2.401		0.14			
25-Jan-93	2.276	2.420		0.00			

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Table 3.1-2
Daily Rainfall and Daily Plant Flow
Woodburn WWTP
January - May, 1989-1993

Date	Daily Influent Flow (a) (mgd)	Daily Effluent Flow (a) (mgd)	Monthly Cumulative Flow (b) (mgd)	Daily Rainfall (c) (inches)	Monthly Cumulative Rainfall (c) (inches)	Rainfall-Flow Trend Line: Daily Flow (mgd)	Rainfall-Flow Trend Line: Daily Rainfall (inches)
26-Jan-93	2.163	2.292		0.00			
27-Jan-93	2.212	2.216		0.00			
28-Jan-93	2.128	2.211		0.26			
29-Jan-93	2.581	2.072		0.10			
30-Jan-93	1.997	2.020		0.00			
31-Jan-93	1.918	1.907	65.653	0.00	3.740		
1-Feb-93	1.957	2.811		0.00			
2-Feb-93	1.953	3.030		0.00			
3-Feb-93	1.116	2.911		0.00			
4-Feb-93	1.854	2.993		0.00			
5-Feb-93	1.776	2.926		0.00			
6-Feb-93	1.932	3.039		0.00			
7-Feb-93	1.632	2.944		0.00			
8-Feb-93	1.790	2.446		0.00			
9-Feb-93	1.704	2.039		0.00			
10-Feb-93	1.766	2.018		0.00			
11-Feb-93	1.679	2.073		0.10			
12-Feb-93	1.808	2.023		0.02			
13-Feb-93	1.675	1.839		0.00			
14-Feb-93	1.633	1.734		0.00			
15-Feb-93	1.919	1.832		0.00			
16-Feb-93	1.739	1.649		0.00			
17-Feb-93	1.675	1.667		0.00			
18-Feb-93	1.744	1.720		0.00			
19-Feb-93	1.788	1.725		0.30			
20-Feb-93	2.252	2.150		0.40			
21-Feb-93	2.300	2.161		0.26			
22-Feb-93	2.245	2.182		0.03			
23-Feb-93	2.089	2.056		0.00			
24-Feb-93	2.019	1.952		0.00			
25-Feb-93	1.811	1.952		0.00			
26-Feb-93	1.847	1.913		0.00			
27-Feb-93	1.676	1.720		0.00			
28-Feb-93	1.691	1.741	51.062	0.00	1.110		
1-Mar-93	2.385	2.197		0.36			
2-Mar-93	2.285	2.249		0.00			
3-Mar-93	2.273	2.329		0.39			
4-Mar-93	2.307	2.327		0.20			
5-Mar-93	2.385	2.496		0.00			
6-Mar-93	1.948	2.043		0.00			
7-Mar-93	1.978	2.096		0.00			
8-Mar-93	1.954	2.221		0.00			
9-Mar-93	2.033	2.138		0.00			
10-Mar-93	2.042	2.074		0.00			
11-Mar-93	1.767	2.103		0.00			
12-Mar-93	1.890	1.980		0.00			
13-Mar-93	1.995	2.075		0.00			
14-Mar-93	2.258	2.210		0.35			
15-Mar-93	2.714	3.606		1.01			
16-Mar-93	3.466	3.334		0.33			
17-Mar-93	3.693	3.688		0.32			
18-Mar-93	3.137	3.093		0.00			
19-Mar-93	2.880	2.938		0.04			
20-Mar-93	2.750	2.872		0.20			
21-Mar-93	2.340	2.637		0.00			
22-Mar-93	4.854	4.063		1.05			
23-Mar-93	3.963	3.920		0.77			
24-Mar-93	3.488	3.407		0.00			
25-Mar-93	3.030	3.045		0.02			
26-Mar-93	2.618	2.870		0.00			
27-Mar-93	2.624	2.788		0.00			
28-Mar-93	2.515	2.659		0.00			
29-Mar-93	2.473	2.562		0.00			
30-Mar-93	2.457	2.650		0.18			
31-Mar-93	2.512	2.625	80.134	0.02	5.240		
1-Apr-93	2.452	2.588		0.26			
2-Apr-93	2.523	2.642		0.35			
3-Apr-93	3.172	3.268		0.72			
4-Apr-93	2.406	2.489		0.00			
5-Apr-93	2.535	2.711		0.00			

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Daily Rainfall and Daily Plant Flow
Woodburn WWTP
January - May, 1989-1993

Date	Daily Influent Flow (a) (mgd)	Daily Effluent Flow (a) (mgd)	Monthly Cumulative Flow (b) (mgd)	Daily Rainfall (c) (inches)	Monthly Cumulative Rainfall (c) (inches)	Rainfall-Flow Trend Line: Daily Flow (mgd)	Rainfall-Flow Trend Line: Daily Rainfall (inches)
6-Apr-93	**2.448**	2.587		0.04			
7-Apr-93	**2.354**	2.628		0.00			
8-Apr-93	**2.782**	2.842		0.28			
9-Apr-93	**3.258**	3.256		0.50			
10-Apr-93	**3.138**	3.049		0.40			
11-Apr-93	**2.949**	2.897		0.50			
12-Apr-93	**2.770**	2.811		0.02			
13-Apr-93	**2.799**	2.694		0.14			
14-Apr-93	**2.475**	2.687		0.00			
15-Apr-93	**2.754**	2.818		0.27			
16-Apr-93	**3.091**	3.172		0.00			
17-Apr-93	**3.480**	3.531		0.65			
18-Apr-93	**3.130**	3.210		0.18			
19-Apr-93	**2.483**	3.049		0.00			
20-Apr-93	**2.737**	2.909		0.06			
21-Apr-93	**2.665**	2.858		0.25			
22-Apr-93	**2.605**	2.764		0.08			
23-Apr-93	**2.779**	2.968		0.48			
24-Apr-93	**2.740**	2.942		0.12			
25-Apr-93	**3.017**	3.075		0.31			
26-Apr-93	**3.212**	3.458		0.31			
27-Apr-93	**2.155**	4.286		0.00			
28-Apr-93	**2.787**	4.492		0.03			
29-Apr-93	**2.820**	3.197		0.20			
30-Apr-93	**2.610**	2.961	83.006	0.00	6.150		
1-May-93	**2.694**	3.019		0.06			
2-May-93	**2.222**	2.739		0.00			
3-May-93	**2.742**	3.173		0.52			
4-May-93	**2.618**	2.757		0.02			
5-May-93	**2.517**	2.907		0.00			
6-May-93	**2.690**	3.595		0.47			
7-May-93	**2.714**	3.695		0.33			
8-May-93	**2.547**	3.600		0.12			
9-May-93	**2.485**	3.675		0.00			
10-May-93	**2.597**	3.500		0.00			
11-May-93	**2.511**	3.506		0.00			
12-May-93	**2.427**	3.367		0.00			
13-May-93	**2.359**	3.442		0.16			
14-May-93	**2.263**	3.236		0.00			
15-May-93	**2.242**	3.288		0.00			
16-May-93	**2.235**	3.276		0.00			
17-May-93	**2.297**	2.821		0.00			
18-May-93	**2.170**	2.432		0.00			
19-May-93	n/a	**2.670**		0.12			
20-May-93	n/a	**2.738**		0.36			
21-May-93	**2.249**	2.480		0.47			
22-May-93	**2.275**	2.538		0.00			
23-May-93	**2.169**	2.447		0.00			
24-May-93	**2.330**	2.581		0.00			
25-May-93	**2.386**	2.736		0.34			
26-May-93	**2.267**	2.518		0.00			
27-May-93	**2.570**	2.818		0.42			
28-May-93	**2.363**	2.579		0.06			
29-May-93	**2.210**	2.443		0.01			
30-May-93	**2.364**	2.647		0.11			
31-May-93	**2.572**	2.795	75.413	0.67	4.240		

Notes:
(a) Both influent and effluent data were available for some months. Influent data were used wherever possible. Flow data used are in bold.
(b) Cumulative monthly data are for the data in bold.
(c) Rainfall data are from Oregon City, Oregon.

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Date printed: 1/30/93

Table 3.1-3
Influent Loading
Woodburn WWTP
1990-1993

	Influent Flow (a,b) (mgd)	*Influent BOD_5 (mg/L)*	*Influent BOD_5 (lb/day)*	*Influent TSS (mg/L)*	*Influent TSS (lb/day)*
Jan-90					
Total	n/a	1,478	29,117	1,114	22,041
Max.	n/a	274	4,337	226	3,709
Min.	n/a	88	2,157	68	1,456
Avg.	n/a	164	3,235	124	2,449
Feb-90					
Total	n/a	912	25,345	775	21,907
Max.	n/a	182	4,352	146	4,271
Min.	n/a	60	1,755	61	1,867
Avg.	n/a	114	3,168	97	2,738
Mar-90					
Total	n/a	1,229	24,678	983	19,562
Max.	n/a	192	3,535	172	2,768
Min.	n/a	99	2,021	72	1,643
Avg.	n/a	136	2,742	109	2,173
Apr-90					
Total	n/a	1,402	22,456	1,374	22,100
Max.	n/a	195	3,339	261	4,469
Min.	n/a	145	2,140	145	2,306
Avg.	n/a	175	2,807	172	2,762
May-90					
Total	n/a	1,644	28,452	1,591	28,809
Max.	n/a	210	5,030	252	3,747
Min.	n/a	136	2,252	123	2,075
Avg.	n/a	183	3,161	159	2,881
Jun-90					
Total	n/a	1,603	25,163	1,171	18,520
Max.	n/a	292	4,105	172	3,015
Min.	n/a	167	2,117	124	1,462
Avg.	n/a	200	3,145	146	2,315
Jul-90					
Total	n/a	3,210	45,273	1,671	23,000
Max.	n/a	432	6,021	227	3,731
Min.	n/a	257	2,846	96	1,590
Avg.	n/a	357	5,030	186	2,556
Aug-90					
Total	n/a	2,165	28,782	1,755	23,098
Max.	n/a	288	4,263	287	3,736
Min.	n/a	204	1,625	157	1,410
Avg.	n/a	241	3,198	195	2,566
Sep-90					
Total	n/a	1,862	31,110	1,494	24,583
Max.	n/a	257	4,739	210	3,652
Min.	n/a	198	2,198	166	1,822
Avg.	n/a	233	3,889	187	3,073
Oct-90					
Total	n/a	2,034	30,937	1,597	24,649
Max.	n/a	288	3,952	232	3,835
Min.	n/a	185	2,702	152	2,066
Avg.	n/a	226	3,437	177	2,739
Nov-90					
Total	n/a	2,062	27,935	1,436	19,914
Max.	n/a	303	3,894	204	2,758
Min.	n/a	151	2,126	96	1,799
Avg.	n/a	229	3,104	160	2,213
Dec-90					
Total	n/a	1,690	21,612	1,650	17,519
Max.	n/a	238	3,261	308	4,184
Min.	n/a	146	2,166	128	1,382
Avg.	n/a	211	2,702	169	2,190

1990 Influent BOD_5 Load Summary

Average Dry Weather Load 3,643 lb/day	2,688
Dry Weather Max Daily Load 6,021 lb/day	3,835
Dry Weather Min Daily Load 1,625 lb/day	1,410
Dry Weather Max Month Avg Daily Load 5,030 lb/day	3,073
Dry Weather Min Month Avg Daily Load 3,145 lb/day	2,315
Average Wet Weather Load 2,960 lb/day	2,421
Wet Weather Max Daily Load 4,352 lb/day	4,469
Wet Weather Max Month Avg Daily Load 3,235 lb/day	2,762

Table 3.1-3
Influent Loading
Woodburn WWTP
1990-1993

	Influent Flow (a,b) (mgd)	*Influent BOD_5 (mg/L)*	*Influent BOD_5 (lb/day)*	*Influent TSS (mg/L)*	*Influent TSS (lb/day)*
Jan-91					
Total	n/a	2,093	29,108	1,460	20,268
Max.	n/a	468	6,261	208	2,782
Min.	n/a	112	2,102	74	1,547
Avg.	n/a	209	2,911	146	2,027
Feb-91					
Total	n/a	879	15,486	1,168	20,678
Max.	n/a	236	3,868	260	4,261
Min.	n/a	17	398	112	2,072
Avg.	n/a	110	1,936	146	2,585
Mar-91					
Total	n/a	1,115	21,221	820	15,799
Max.	n/a	230	3,586	162	2,901
Min.	n/a	80	2,068	54	1,055
Avg.	n/a	139	2,653	103	1,975
Apr-91					
Total	n/a	1,064	22,742	1,181	25,425
Max.	n/a	172	3,802	252	4,972
Min.	n/a	70	1,707	67	1,720
Avg.	n/a	118	2,527	131	2,825
May-91					
Total	n/a	1,226	26,133	1,018	22,140
Max.	n/a	195	4,054	150	4,223
Min.	n/a	78	1,961	74	1,669
Avg.	n/a	136	2,904	113	2,460
Jun-91					
Total	n/a	1,339	22,714	1,046	17,880
Max.	n/a	217	3,759	166	2,772
Min.	n/a	86	2,017	82	1,849
Avg.	n/a	167	2,839	131	2,235
Jul-91					
Total	n/a	2,424	35,423	1,718	25,159
Max.	n/a	498	7,198	254	3,671
Min.	n/a	189	2,915	148	2,411
Avg.	n/a	269	3,936	191	2,795
Aug-91					
Total	50.37	2,303	31,108	1,778	24,145
Max.	1.83	370	4,656	302	4,428
Min.	1.32	190	2,711	84	1,199
Avg.	1.63	256	3,456	198	2,683
Sep-91					
Total	45.59	2,136	31,839	1,778	36,401
Max.	1.69	366	5,912	342	5,036
Min.	1.26	165	2,504	110	1,299
Avg.	1.52	267	3,980	222	3,300
Oct-91					
Total	49.99	2,201	30,433	1,374	18,835
Max.	2.10	250	3,669	204	2,881
Min.	1.31	190	2,675	66	1,038
Avg.	1.61	220	3,043	137	1,884
Nov-91					
Total	53.71	1,434	26,411	1,096	21,432
Max.	2.60	266	4,758	228	5,444
Min.	1.30	119	2,253	102	1,458
Avg.	1.79	179	3,301	137	2,679
Dec-91					
Total	66.44	1,149	22,459	927	18,253
Max.	4.00	233	4,461	186	3,987
Min.	1.44	12	1,726	35	718
Avg.	2.29	144	3,208	116	2,282

1991 Influent BOD_5 Load Summary

Average Dry Weather Load 3,360 lb/day	2,560
Dry Weather Max Daily Load 7,198 lb/day	5,036
Dry Weather Min Daily Load 1,961 lb/day	1,038
Dry Weather Max Month Avg Daily Load 3,980 lb/day	3,300
Dry Weather Min Month Avg Daily Load 2,839 lb/day	1,884
Average Wet Weather Load 2,756 lb/day	2,396
Wet Weather Max Daily Load 6,261 lb/day	5,444
Wet Weather Max Month Avg Daily Load 3,301 lb/day	2,825

Table 3.1-3
Influent Loading
Woodburn WWTP
1990-1993

	Influent Flow (a,b) (mgd)	*Influent BOD_5 (mg/L)*	*Influent BOD_5 (lb/day)*	*Influent TSS (mg/L)*	*Influent TSS (lb/day)*
Jan-92					
Total	81.48	1,716	35,693	1,090	22,899
Max.	4.81	214	4,966	140	4,292
Min.	1.54	110	2,656	64	1,718
Avg.	2.72	172	3,569	109	2,290
Feb-92					
Total	n/a	961	31,050	772	25,378
Max.	n/a	169	10,028	144	9,025
Min.	n/a	60	2,045	54	1,647
Avg.	n/a	120	3,881	97	3,172
Mar-92					
Total	n/a	1,340	26,156	985	19,382
Max.	n/a	207	4,188	156	3,449
Min.	n/a	111	2,491	86	1,416
Avg.	n/a	168	3,269	123	2,423
Apr-92					
Total	n/a	1,520	27,346	1,132	20,167
Max.	n/a	270	4,227	194	3,037
Min.	n/a	150	2,672	106	2,130
Avg.	n/a	190	3,418	142	2,521
May-92					
Total	n/a	2,264	34,891	1,692	25,981
Max.	n/a	290	4,276	242	3,552
Min.	n/a	187	2,976	128	2,037
Avg.	n/a	252	3,877	188	2,887
Jun-92					
Total	n/a	2,994	39,966	2,005	26,243
Max.	n/a	535	7,840	490	7,499
Min.	n/a	134	2,064	75	979
Avg.	n/a	333	4,441	223	2,916
Jul-92					
Total	56.68	3,907	60,411	2,394	35,740
Max.	3.12	623	10,546	324	6,101
Min.	1.22	235	190	190	2,320
Avg.	1.83	391	3,021	239	3,574
Aug-92					
Total	45.29	2,253	29,224	1,922	26,265
Max.	2.29	372	5,460	317	4,653
Min.	1.03	176	3,136	177	2,346
Avg.	1.46	282	3,653	240	3,283
Sep-92					
Total	n/a	3,225	42,593	2,608	32,218
Max.	n/a	389	4,519	373	4,987
Min.	n/a	190	3,136	85	1,010
Avg.	n/a	293	3,653	237	2,929
Oct-92					
Total	n/a	3,217	35,401	2,742	27,925
Max.	n/a	471	4,675	454	5,165
Min.	n/a	256	2,802	164	2,060
Avg.	n/a	357	3,653	305	3,103
Nov-92					
Total	46.11	2,483	27,678	1,106	12,276
Max.	2.46	441	5,664	230	2,574
Min.	1.11	205	2,302	22	219
Avg.	1.54	310	3,460	138	1,535
Dec-92					
Total	57.72	2,238	34,153	1,416	21,960
Max.	2.89	336	4,732	218	4,518
Min.	1.47	147	2,110	38	717
Avg.	1.86	224	3,415	142	2,196

1992 Influent BOD_5 Load Summary

Average Dry Weather Load 3,716 lb/day	3,115
Dry Weather Max Daily Load 10,546 lb/day	7,499
Dry Weather Min Daily Load 190 lb/day	979
Dry Weather Max Month Avg Daily Load 4,441 lb/day	3,574
Dry Weather Min Month Avg Daily Load 3,021 lb/day	2,887
Average Wet Weather Load 3,502 lb/day	2,356
Wet Weather Max Daily Load 10,028 lb/day	9,025
Wet Weather Max Month Avg Daily Load 3,881 lb/day	3,172

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Influent Loading
Woodburn WWTP
1990-1993

	Influent Flow (a,b) (mgd)	Influent BOD_5 (mg/L)	Influent BOD_5 (lb/day)	Influent TSS (mg/L)	Influent TSS (lb/day)
Jan-93					
Total	65.65	1,760	31,576	1,319	24,198
Max.	3.21	278	6,691	272	6,306
Min.	1.62	154	2,984	43	570
Avg.	2.12	220	3,947	165	3,025
Feb-93					
Total	51.06	2,027	36,830	1,470	26,680
Max.	2.30	338	6,440	230	4,397
Min.	1.12	214	3,273	158	2,270
Avg.	1.82	253	4,604	184	3,335
Mar-93					
Total	80.13	1,581	33,923	1,314	28,143
Max.	4.05	286	5,016	286	5,550
Min.	1.77	79	2,286	73	2,387
Avg.	2.59	176	3,769	164	3,518
Apr-93					
Total	83.01	1,299	31,341	982	23,542
Max.	3.18	184	4,361	168	3,348
Min.	2.16	100	2,666	56	1,493
Avg.	2.77	144	3,482	109	2,616
May-93					
Total	70.01	1,596	67,359	1,130	28,044
Max.	2.74	228	5,913	174	5,217
Min.	2.17	107	3,129	66	1,470
Avg.	2.26	177	4,301	126	3,116
Jun-93					
Total	73.66	1,724	63,949	1,283	25,995
Max.	3.33	270	4,859	259	4,873
Min.	1.98	110	2,977	54	1,414
Avg.	2.46	192	3,986	143	2,888
Jul-93					
Total	54.95	2,968	82,719	1,695	25,784
Max.	2.11	480	5,965	244	3,669
Min.	1.45	244	4,135	126	1,927
Avg.	1.77	330	4,999	188	2,865
Aug-93					
Total	51.56	1,687	57,248	1,328	18,924
Max.	1.95	295	4,168	223	3,273
Min.	1.45	198	3,106	62	813
Avg.	1.66	241	3,427	133	1,892
Sep-93					
Total	46.00	2,403	59,722	1,165	15,337
Max.	1.78	342	4,602	217	2,924
Min.	1.45	156	2,154	63	840
Avg.	1.53	267	3,507	129	1,704
Oct-93					
Total	46.07	2,623	67,887	953	11,990
Max.	1.72	855	10,575	195	2,385
Min.	0.59	199	2,247	66	797
Avg.	1.48	328	8,486	119	1,499
Nov-93					
Total	47.40	2,958	79,747	1,796	24,305
Max.	2.13	411	7,419	248	3,786
Min.	1.23	285	3,906	109	1,364
Avg.	1.57	329	2,520	200	2,701
Dec-93					
Total	54.50	2,755	75,985	1,797	24,791
Max.	3.01	410	5,776	229	3,118
Min.	1.45	228	3,575	64	1,137
Avg.	1.76	276	4,321	180	2,479

1993 Influent BOD_5 Load Summary	tss
Average Dry Weather Load 4,784 lb/day	2,327
Dry Weather Max Daily Load 10,575 lb/day	5,217
Dry Weather Min Daily Load 2,154 lb/day	797
Dry Weather Max Month Avg Daily Load 8,486 lb/day	3,116
Dry Weather Min Month Avg Daily Load 3,427 lb/day	1,499
Average Wet Weather Load 3,774 lb/day	2,946
Wet Weather Max Daily Load 7,419 lb/day	6,306
Wet Weather Max Month Avg Daily Load 4,604 lb/day	3,518

Table 3.1-3
Influent Loading
Woodburn WWTP
1990-1993

	Influent Flow (a,b) (mgd)	*Influent BOD_5 (mg/L)*	*Influent BOD_5 (lb/day)*	*Influent TSS (mg/L)*	*Influent TSS (lb/day)*
Summary: 1990 - 1993					
Period of Record					
Maximum	4.81	855	10,575	490	9,025
Minimum	0.59	12	190	22	219
1990 Average	n/a	206	3,302	157	2,555
1991 Average	1.77	185	3,058	148	2,478
1992 Average	1.88	258	3,609	182	2,736
1993 Average	1.98	244	4,279	153	2,637
POR Average	1.91	223	3,547	160	2,600
Dry Weather					
Maximum	3.33	855	10,575	490	7,499
Minimum	0.59	78	190	54	797
Average	1.75	258	3,876	180	2,673
Wet Weather					
Maximum	4.81	468	10,028	308	9,025
Minimum	1.11	12	398	22	219
Average	2.07	188	3,248	140	2,530

Notes:
(a) Influent flow data were not reported until July 1991.
(b) The influent flowmeter was out for repairs February - June and September - October, 1992.

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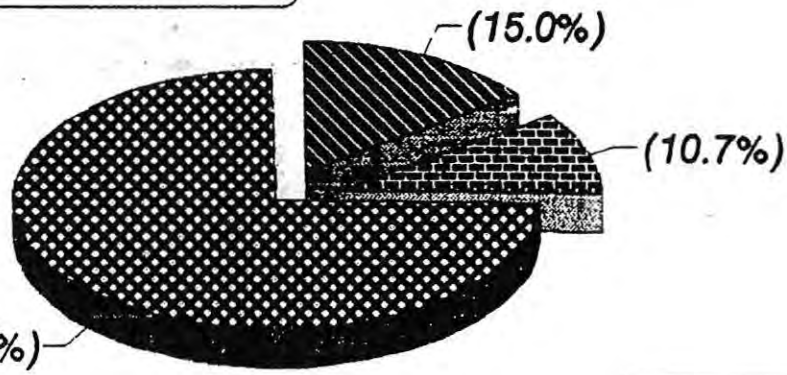
Appendix 3.2

Treatment Plant Population Projections by City of Woodburn

CITY OF WOODBURN
SUMMER FLOW

AVERAGE DAILY FLOW FOR SIX MONTH PERIOD

COM .294 MGD



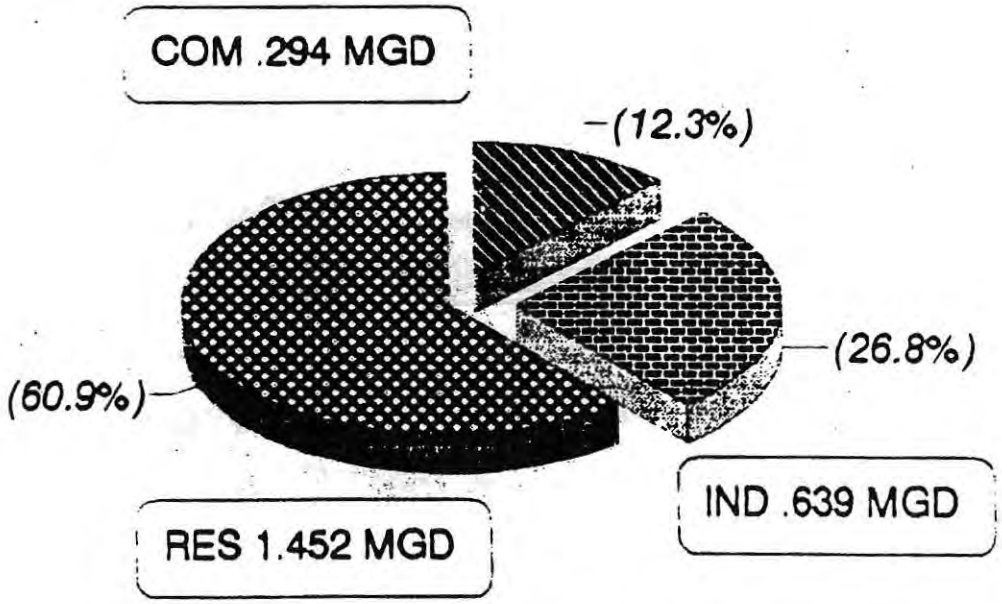
RES 1.452 MGD

IND .210 MGD

INDUSTRIAL ACTUAL AVERAGE MONTHLY FLOWS

CITY OF WOODBURN
SUMMER FLOW

Average Daily Flow For Six Month Period



INDUSTRIAL MONTHLY AVERAGE PERMITTED FLOWS

TREATMENT PLANT SUMMER MONTH FLOW PROJECTIONS

BASED on FLOW

TABLE 1A

Year	Resid. Summer Average Flow MGD	Comm. Sum. Aver. Flow MGD	Ind. Monthly Average Permited Flow MGD	Total Sum. Ave. Flow MGD	Summer Peak Mo Avg Ind Flow MGD	Actual Summer Ave Ind Flow MGD
1993	1.452	.294	.639	2.385	.790	.210
1994	1.501	.304	.642	2.447	.798	.212
1995	1.552	.314	.645	2.481	.806	.214
1996	1.605	.325	.649	2.579	.814	.216
1997	1.660	.336	.652	2.648	.822	.218
1998	1.716	.347	.655	2.718	.830	.221
1999	1.774	.359	.658	2.791	.839	.223
2000	1.834	.371	.662	2.867	.847	.225
2001	1.897	.384	.665	2.946	.855	.227
2002	1.961	.397	.668	3.026	.864	.230
2003	2.028	.411	.672	3.111	.873	.232
2004	2.097	.425	.675	3.197	.881	.234
2005	2.168	.439	.678	3.285	.890	.239
2006	2.242	.454	.682	3.378	.899	.239
2007	2.318	.469	.685	3.472	.908	.241
2008	2.397	.485	.689	3.571	.917	.244
2009	2.478	.502	.692	3.672	.926	.246
2010	2.562	.519	.695	3.776	.936	.249
2011	2.650	.537	.699	3.886	.945	.251
2012	2.740	.554	.702	3.996	.954	.254
2013	2.833	.574	.706	4.113	.964	.256
2014	2.929	.593	.710	4.232	.974	.259
2015	3.029	.613	.713	4.355	.983	.261
2016	3.132	.634	.717	4.783	.993	.264
2017	3.238	.656	.720	4.614	1.00	.267
2018	3.348	.678	.724	4.750	1.01	.269
2019	3.462	.701	.727	4.890	1.02	.272
2020	3.580	.725	.731	5.036	1.03	.275

Base Data From 1993

Treatment Plant Flow Capacity Needs

For Year 2020

Total Summer Average Resid. Commercial & Indus. Permitted Flow MGD	Summer Peak Month Flow MGD	Actual Summer Average Flow From Residential, Comm. & Industrial.
5.036 MGD For 2020	5.34 MGD For 2020	4.58 MGD For 2020

TREATMENT PLANT POPULATION PROJECTIONS

BASED on FLOW

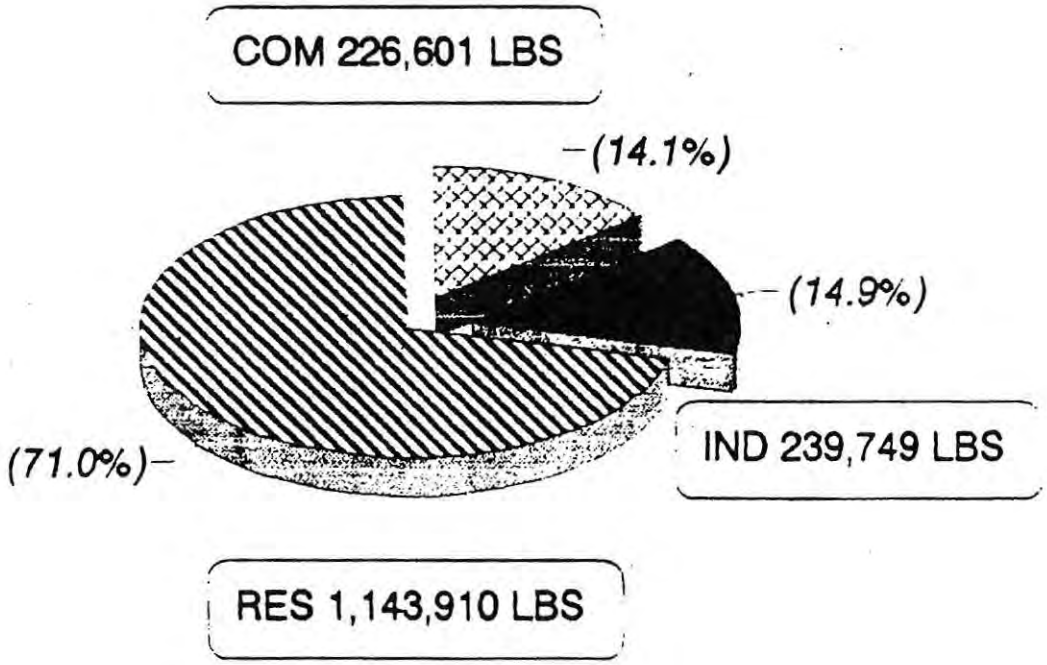
Table 2A

Year	Residential	Commercial	Industrial	Total
1993	17,707	3,585	7,793	29,085
1994	18,305	3,707	7,829	29,841
1995	18,927	3,829	7,866	30,622
1996	19,573	3,963	7,915	31,451
1997	20,244	4,097	7,951	32,292
1998	20,927	4,232	7,988	33,238
1999	21,634	4,278	8,024	34,036
2000	22,366	4,524	8,073	34,963
2001	23,134	4,683	8,110	35,927
2002	23,915	4,841	8,146	36,902
2003	24,732	5,012	8,195	37,939
2004	25,573	5,183	8,232	38,988
2005	26,439	5,354	8,268	40,061
2006	27,341	5,537	8,317	41,195
2007	28,268	5,720	8,354	43,242
2008	29,232	5,915	8,402	43,549
2009	30,220	6,122	8,439	44,781
2010	31,244	6,329	8,476	46,049
2011	32,317	6,549	8,524	47,390
2012	33,415	6,756	8,561	48,732
2013	34,549	7,000	8,610	20,159
2014	35,720	7,232	8,659	51,702
2015	36,939	7,476	8,695	53,110
2016	38,195	7,732	8,744	54,671
2017	39,488	8,000	8,780	56,268
2018	40,829	8,263	8,829	57,921
2019	42,220	8,549	8,866	59,635
2020	43,659	8,841	8,915	61,415

Flow population Equivalent Based on 82 Gals/Person/Day
Residential and commercial growth is based on 3.4% growth per year.

Industrial permitted growth and summer peak average is based on 1.0% growth per year.

CITY OF WOODBURN
YEARLY TBOD LBS



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TREATMENT PLANT POPULATION PROJECTIONS

BASED on TBOD

Table 1B

Year	Resid. Popul.	Temp. Pop. (Migrant)	Other Popul.	Sub-Total Resid. Popul.	Commer. Popul. Equiv.	Total Resid. & Comm. Pop. Equivalents
1993	15,645	1,662	400	17,707	3507	21,214
1994	16,176	1,718	400	18,294	3626	21,920
1995	16,726	1,776	400	18,902	3750	22,652
1996	17,295	1,837	400	19,532	3870	23,402
1997	17,883	1,899	400	20,182	4009	24,191
1998	18,491	1,964	530	20,985	4145	25,130
1999	19,120	2,031	530	21,681	4296	25,977
2000	19,770	2,100	530	22,400	4432	26,832
2001	20,442	2,171	980	23,539	4582	28,121
2002	21,137	2,245	980	24,362	4738	29,100
2003	21,856	2,321	980	25,157	4899	30,056
2004	22,599	2,400	980	25,979	5066	31,045
2005	23,368	2,482	980	26,830	5238	32,068
2006	24,162	2,566	980	27,708	5416	33,124
2007	24,984	2,654	980	28,618	5600	34,218
2008	25,833	2,744	980	29,557	5791	35,348
2009	26,711	2,837	980	30,528	5988	36,516
2010	27,620	2,934	980	31,534	6191	37,725
2011	28,559	3,033	980	32,572	6402	38,974
2012	29,530	3,137	980	33,647	6620	40,267
2013	30,534	3,243	980	34,757	6845	41,602
2014	31,572	3,353	980	35,905	7077	42,982
2015	32,645	3,468	980	37,093	7318	44,411
2016	33,755	3,585	980	38,320	7567	45,887
2017	34,903	3,707	980	39,590	7824	47,414
2018	36,090	3,833	980	40,903	8090	48,993
2019	37,317	3,964	980	42,261	8365	50,626
2020	38,586	4,099	980	43,665	8649	52,314

Base data from 1993

Residential, commercial and temporary population growth are based on 3.4% growth per year.
Other populations include Carl Road Mobile Home Units, Maclaren School and Shalimar Mobile Home Units.
Industrial Worst case scenario is based on permitted loads, actual worst case data and 1% growth per year.

Based on TBOD

Table 2B

Year	Residential & Commercial Population Equivalents	Industrial Yearly Ave. Population Equivalents	Industrial Peak Month Ave. Pop. Equiv.	Industrial W.C.S. Population Equiv.
1993	21,214	3,711	7,351	65,389
1994	21,920	3,748	7,425	66,043
1995	22,652	3,786	7,499	66,703
1996	23,402	3,823	7,574	67,370
1997	24,191	3,862	7,649	68,044
1998	25,130	3,900	7,726	68,724
1999	25,977	3,939	7,803	69,412
2000	26,832	3,979	7,881	70,106
2001	28,121	4,018	7,960	70,807
2002	29,100	4,059	8,040	71,515
2003	30,056	4,099	8,120	72,230
2004	31,045	4,140	8,201	72,952
2005	32,068	4,182	8,283	73,682
2006	33,124	4,223	8,366	74,419
2007	34,218	4,266	8,450	75,163
2008	35,348	4,308	8,534	75,914
2009	36,516	4,351	8,620	76,674
2010	37,725	4,395	8,706	77,440
2011	38,974	4,439	8,793	78,215
2012	40,267	4,483	8,881	78,997
2013	41,602	4,528	8,970	79,787
2014	42,982	4,573	9,059	80,585
2015	44,411	4,519	9,150	81,391
2016	45,887	4,665	9,241	82,205
2017	47,414	4,712	9,334	83,027
2018	48,993	4,758	9,427	83,857
2019	50,626	4,807	9,521	84,696
2020	52,314	4,855	9,617	85,542

Industrial worst case scenerio, peak monthly average and summer average is based on 1.0% growth per year.

Residential Commercial and Industrial Population Equivalents

Based on TBOD

Table 3B

Year	Total Summer Average Population Equivalents	Total Summer Peak Month Ave. Pop. Equiv.	Total W.C.S Population Equivalents
1993	24,925	28,565	86,603
1994	25,668	29,345	87,963
1995	26,438	30,151	89,355
1996	27,225	30,976	90,772
1997	28,053	31,840	92,235
1998	29,030	32,856	93,854
1999	29,916	33,780	95,389
2000	30,811	34,713	96,938
2001	32,139	36,081	98,928
2002	33,159	37,140	100,615
2003	34,155	38,176	102,286
2004	35,185	39,246	103,997
2005	36,250	40,351	105,750
2006	37,347	41,490	107,543
2007	38,484	42,669	109,381
2008	39,656	43,882	111,262
2009	40,867	45,136	113,190
2010	42,120	46,431	115,165
2011	43,413	47,767	117,189
2012	44,750	49,148	119,264
2013	46,130	50,572	121,389
2014	47,555	52,041	123,567
2015	48,930	53,561	125,802
2016	50,552	55,128	128,092
2017	52,126	56,748	130,441
2018	53,751	58,420	132,850
2019	55,433	60,147	135,322
2020	57,169	61,931	137,856

Treatment Plant TBOD Capacity Needs in Population Equivalents

For Year 2020

Summer Average Loadings	Peak Month Average Loadings	W.C.S. Loadings
57,169 For 2020	61,931 For 2020	137,856 For 2020

Averages are based on a 22 day work month.
Population equivalent based on .177 lbs BOD/Capita)
Residential & Commercial population based on 3.4% growth per year.

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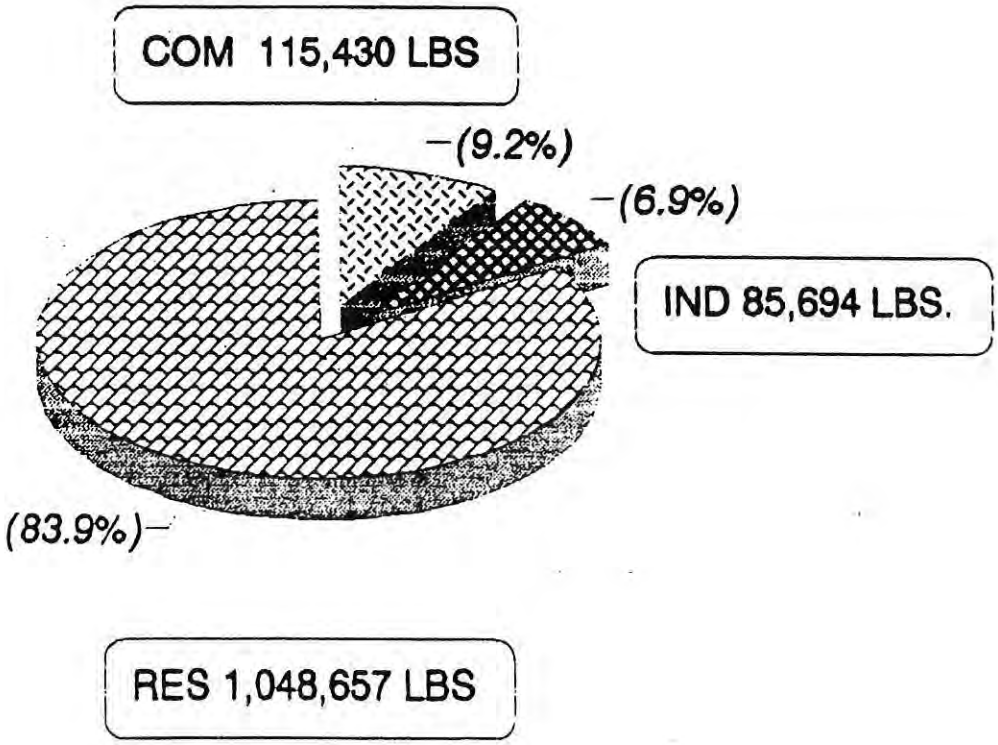
CITY OF WOODBURN

YEARLY TSS LBS



TREATMENT PLANT POPULATION PROJECTIONS
BASED ON TSS

Table 1C

Year	Residential Population	Commercial Population Equivalent	Industrial Population Equivalent	Total Resid & Comm Pop Equivalent
1993	17707	2694	2000	20401
1994	18305	2786	2020	21091
1995	18927	2880	2040	21807
1996	19573	2978	2061	22551
1997	20244	3079	2081	23323
1998	20927	3184	2102	24111
1999	21634	3292	2123	24926
2000	22366	3404	2144	25770
2001	23134	3520	2166	26654
2002	23915	3640	2187	27558
2003	24732	3764	2209	28496
2004	25573	3892	2231	29465
2005	26439	4024	2254	30463
2006	27341	4161	2276	31503
2007	28268	4303	2299	32571
2008	29232	4449	2322	33681
2009	30220	4600	2345	34820
2010	31244	4757	2359	36001
2011	32317	4918	2392	37235
2012	33415	5086	2416	38501
2013	34549	5258	2440	39807
2014	35720	5437	2465	41157
2015	36939	5622	2489	42561
2016	38195	5813	2574	44008
2017	39488	6011	2539	45499
2018	40829	6215	2565	47044
2019	42220	6426	2591	48646
2020	43659	6645	2616	50304

22 Days per month
.1623 = 1 Population Equivalent

TREATMENT PLANT POPULATION PROJECTIONS
BASED ON TSS

Table 2C

Year	Residential & Commercial Population Equivalents	Industrial Yearly Avg. Population Equivalents	Industrial Peak Month Avg. Pop. Equivalents	Industrial W.C.S. Population Equivalents
1993	20,401	2,000	3,857	11,916
1994	21,091	2,020	3,896	12,035
1995	21,807	2,040	3,935	12,156
1996	22,551	2,061	3,974	12,277
1997	23,323	2,081	4,014	12,400
1998	24,111	2,102	4,054	12,524
1999	24,926	2,123	4,094	12,649
2000	25,770	2,144	4,135	12,776
2001	26,654	2,166	4,177	12,903
2002	27,555	2,187	4,218	13,032
2003	28,496	2,209	4,261	13,163
2004	29,465	2,231	4,303	13,294
2005	30,463	2,254	4,346	13,427
2006	31,502	2,276	4,390	13,562
2007	32,571	2,299	4,434	13,697
2008	33,681	2,322	4,478	13,834
2009	34,820	2,345	4,523	13,972
2010	36,001	2,359	4,568	14,112
2011	37,235	2,392	4,614	14,253
2012	38,501	2,416	4,660	14,396
2013	39,807	2,440	4,706	14,540
2014	41,157	2,465	4,753	14,685
2015	42,561	2,489	4,801	14,832
2016	44,008	2,574	4,849	14,980
2017	45,499	2,539	4,897	15,130
2018	47,044	2,565	4,946	15,281
2019	48,646	2,591	4,996	15,434
2020	50,304	2,616	5,046	15,589

Industrial worst case scenerio, peak monthly average and summer average is based on 1.0% growth per year.

RESIDENTIAL, COMMERCIAL AND INDUSTRIAL POPULATION EQUIVALENTS

Table 3C

Year	Total Summer Average Population Equivalents	Total Summer Peak Month Average Pop. Equiv.	Total W.C.S. Population Equivalents
1993	22,401	24,258	32,317
1994	23,111	24,987	33,045
1995	23,847	25,742	33,963
1996	24,612	26,525	34,828
1997	25,404	27,337	35,600
1998	26,213	28,165	36,635
1999	27,049	29,020	37,575
2000	27,914	29,905	38,546
2001	28,820	30,831	39,557
2002	29,705	31,773	43,587
2003	30,705	32,757	41,659
2004	31,696	33,768	42,759
2005	32,717	34,809	43,890
2006	33,778	35,892	45,064
2007	34,870	37,005	46,268
2008	36,003	38,159	47,515
2009	37,165	39,343	48,792
2010	38,360	50,569	50,113
2011	39,627	41,849	51,488
2012	40,917	43,161	50,897
2013	42,247	44,513	54,347
2014	43,622	45,910	55,842
2015	45,050	47,362	57,393
2016	46,582	48,857	58,955
2017	48,038	50,396	60,629
2018	49,609	51,990	62,325
2019	51,237	53,642	64,080
2020	52,920	55,350	65,893

Treatment Plant TSS Capacity Needs in Population Equivalents

FOR THE YEAR 2020

Summer Average Loadings	Peak Month Average Loadings	W.C.S. Loadings
52,920	55,350	65,893

Averages are based on 22 day work month.
Population equivalents based on .1623 lbs TSS/Capita.
Residential & Commercial population based on 3.4% growth per year.

TREATMENT PLANT LBS PROJECTION
BASED On TSS Lbs.

Table 4C

DATE	RESIDENTIAL	COMMERCIAL	INDUSTRIAL YEARLY AVG LBS	TOTAL RES & COMM LBS
1993	2,874	437	325	3,311
1994	2,972	452	328	3,424
1995	3,073	467	332	3,540
1996	3,177	483	335	3,660
1997	3,285	500	338	3,785
1998	3,397	517	342	3,914
1999	3,512	534	345	4,046
2000	3,632	552	348	4,184
2001	3,755	571	352	4,326
2002	3,883	590	355	4,473
2003	4,015	611	359	4,626
2004	4,152	631	363	4,783
2005	4,293	653	366	4,946
2006	4,439	675	370	5,114
2007	4,590	698	374	5,288
2008	4,746	722	377	5,468
2009	4,907	746	381	5,653
2010	5,074	771	385	5,845
2011	5,246	798	389	6,044
2012	5,424	825	393	6,249
2013	5,609	853	397	6,462
2014	5,800	882	401	6,682
2015	5,997	912	405	6,909
2016	6,201	943	409	7,144
2017	6,412	975	413	7,387
2018	6,630	1,008	417	7,638
2019	6,855	1,042	421	7,897
2020	7,088	1,078	425	8,166

Residential & Commercial based on 1.034% growth.
Residential & Commercial Average figured at 365 days per year.
Industrial average figured on 264 days per year.

TREATMENT PLANT LBS PROJECTIONS
BASED ON TSS

Table 5C

DATE	Res & Comm lbs.	Industrial yearly avg lbs	Industrial peak month avg lbs	Industrial W.C.S. lbs
1993	3311	325	626	1934
1994	3424	328	632	1953
1995	3540	332	639	1973
1996	3660	335	645	1993
1997	3785	338	651	2012
1998	3914	342	658	2033
1999	4046	345	665	2053
2000	4184	348	671	2073
2001	4326	352	678	2094
2002	4473	355	685	2115
2003	4626	359	691	2136
2004	4783	363	698	2158
2005	4946	366	705	2179
2006	5114	370	712	2201
2007	5288	374	720	2223
2008	5468	377	727	2245
2009	5653	381	734	2268
2010	5845	385	741	2290
2011	6044	389	749	2313
2012	6249	393	756	2336
2013	6462	397	764	2360
2014	6682	401	771	2383
2015	6909	405	779	2407
2016	7144	409	787	2431
2017	7387	413	795	2456
2018	7638	417	803	2480
2019	7897	421	811	2505
2020	8166	425	819	2530

TREATMENT PLANT LBS PROJECTIONS
BASED ON TSS

Table 6C

Date	Total summer average TSS lbs	Total Summer peak month avg TSS lbs	Total W.C.S. TSS lbs
1993	3,636	3,937	5,245
1994	3,751	4,056	5,377
1995	3,872	4,179	5,513
1996	3,995	4,305	5,653
1997	4,123	4,436	5,797
1998	4,256	4,572	5,947
1999	4,391	4,711	6,099
2000	4,532	4,855	6,257
2001	4,678	5,004	6,420
2002	4,828	5,158	6,588
2003	4,985	5,317	6,762
2004	5,146	5,481	6,941
2005	5,312	5,651	7,125
2006	5,484	5,826	7,315
2007	5,662	6,008	7,511
2008	5,845	6,195	7,713
2009	6,034	6,387	7,921
2010	6,230	6,586	8,135
2011	6,433	6,793	8,357
2012	6,642	7,005	8,585
2013	6,859	7,226	8,822
2014	7,083	7,453	9,065
2015	7,314	7,688	9,316
2016	7,553	7,931	9,575
2017	7,800	8,182	9,843
2018	8,055	8,441	10,118
2019	8,318	8,708	10,402
2020	8,519	8,985	10,696,

Treatment plant TSS loadings

FOR THE YEAR 2020

Summer Average Loadings	Peak Month Average Loadings	Total W.C.S. Loadings
8,519	8,985	10,696

Residential and Commercial based on 3.4% growth per year.
Industrial based on 1.0% per year.

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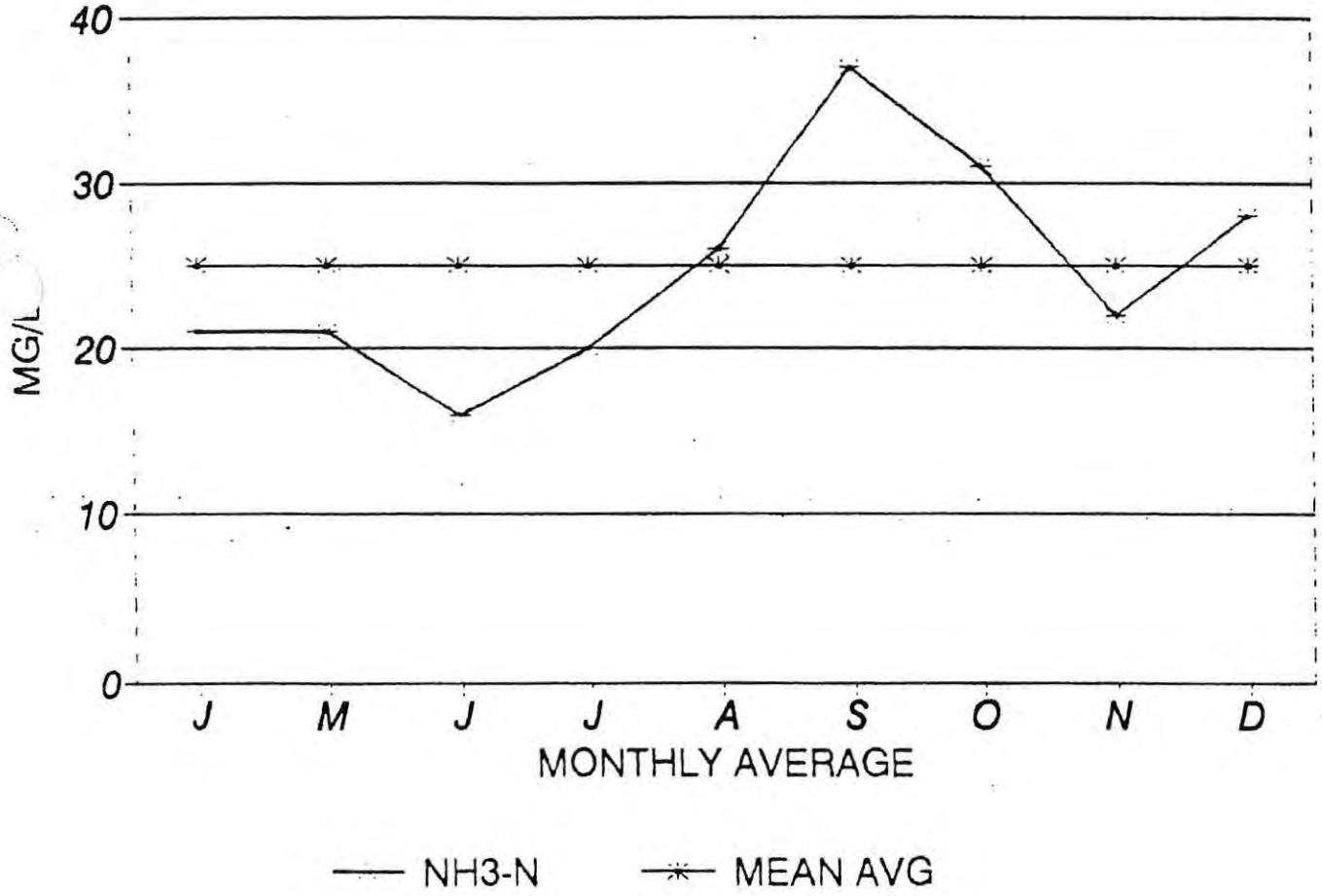
PLANT PROFILE 1989



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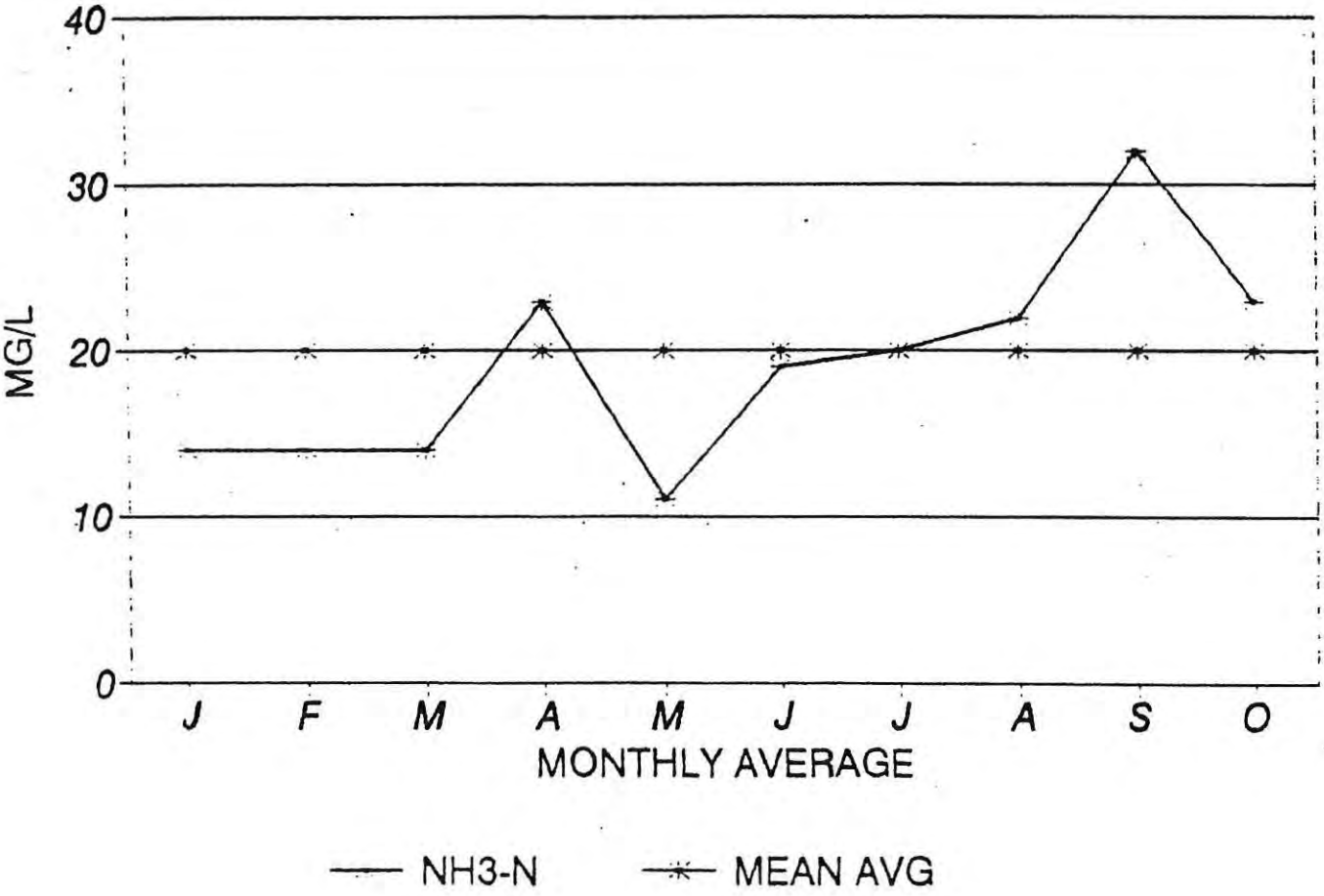
PLANT PROFILE 1990



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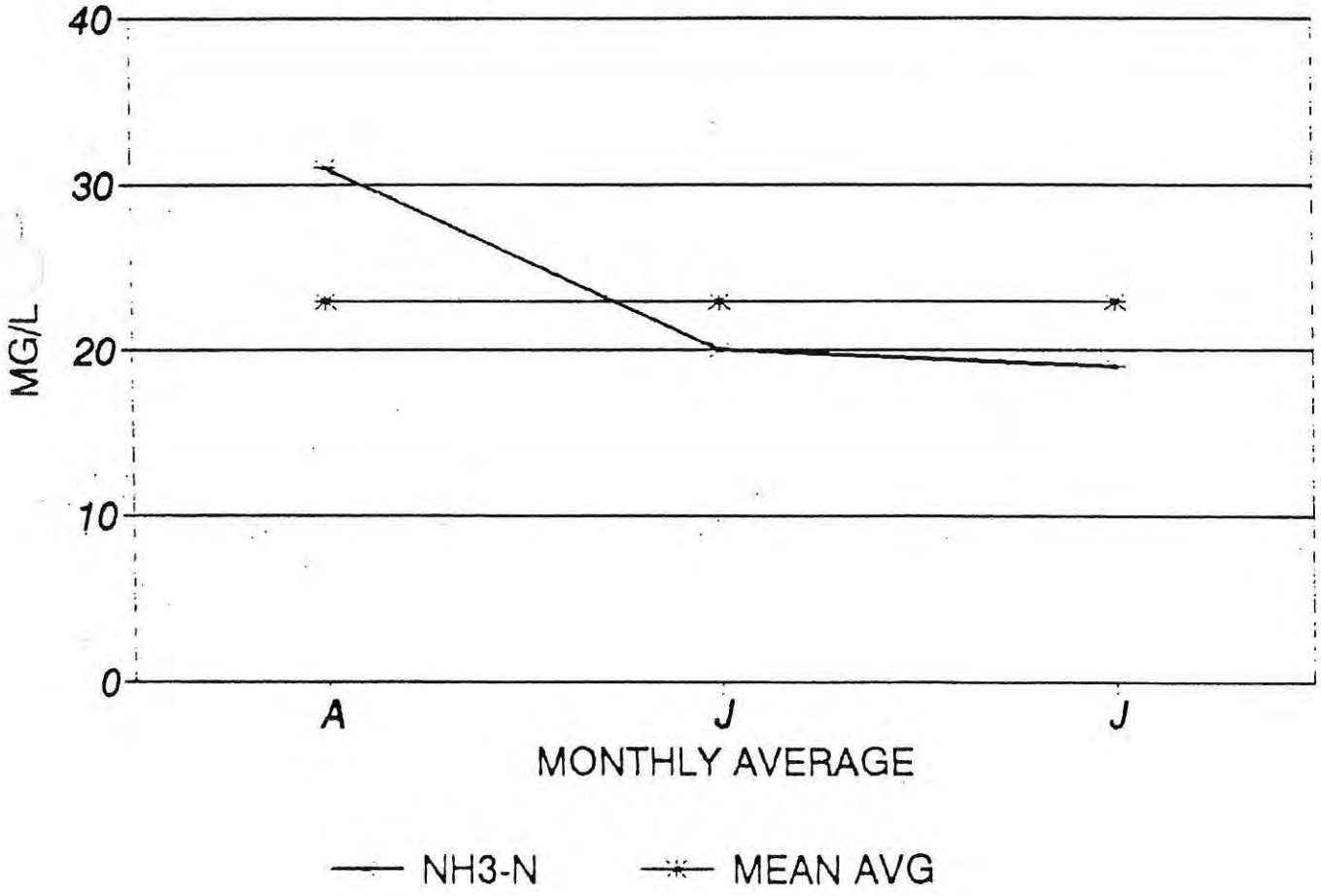
PLANT PROFILE 1991



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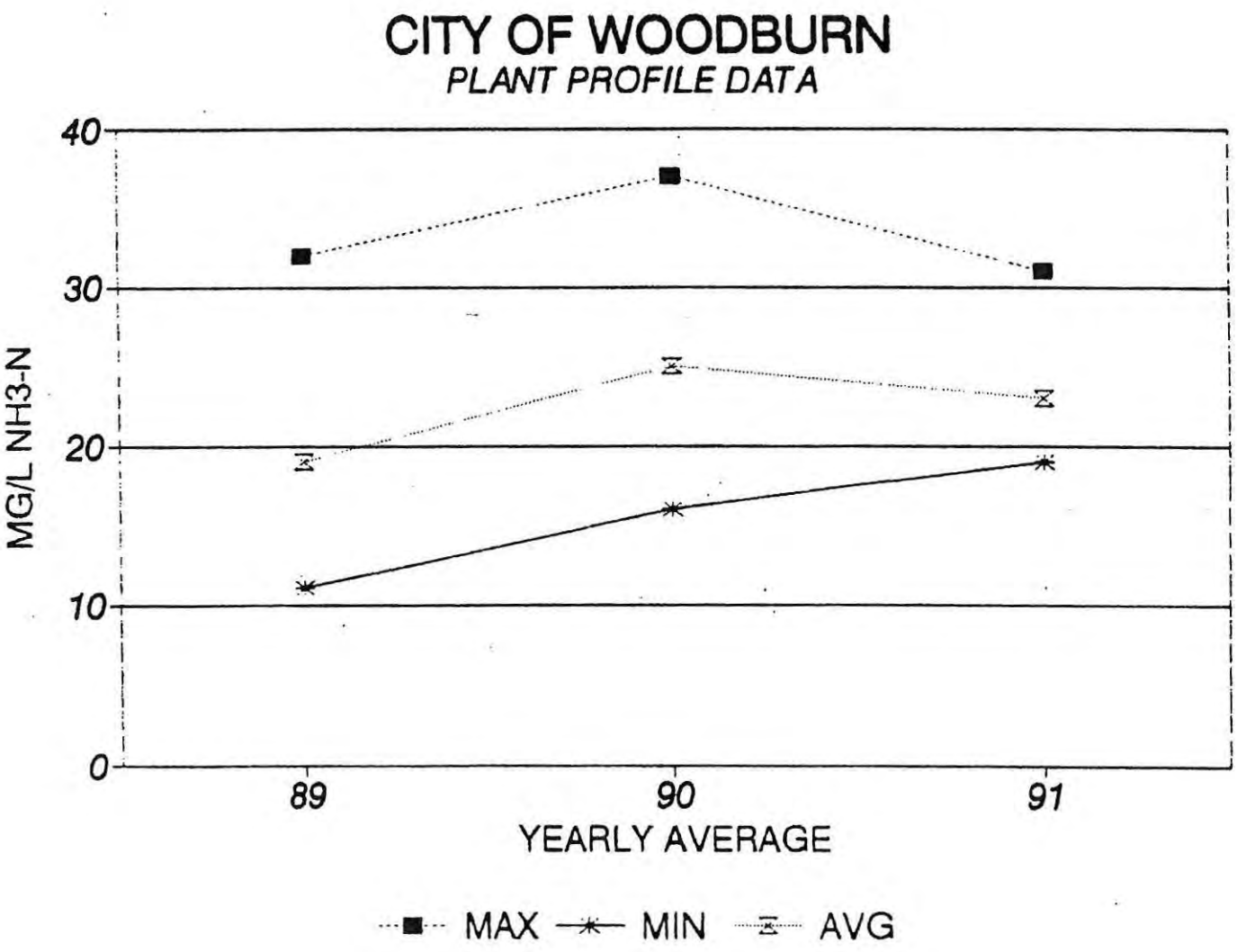

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	MAX	MIN	AVG
89	32	11	19
90	37	16	25
91	31	19	23

1989 AMMONIA PLANT PROFILE

J	21	25
M	21	25
J	16	25
J	20	25
A	26	25
S	37	25
O	31	25
N	22	25
D	28	25

TOTAL	222
MAX	37
MIN	16
AVG	25

1990 AMMONIA PLANT PROFILE

	MG/L	MG/L
J	14	20
F	14	20
M	14	20
A	23	20
M	11	20
J	19	20
J	20	20
A	22	20
S	32	20
O	23	20

TOTAL	192
MAX	32
MIN	11
AVG	19

1991 AMMONIA PLANT PROFILE

A	31	23
J	20	23
J	19	23
TOTAL	70	
MAX	31	
MIN	19	
AVG	23	

Section 4
Treatment Requirements

Introduction

This section summarizes current and proposed regulations, and establishes the design criteria to be used in the development of the various treatment and disposal alternatives for the City of Woodburn wastewater treatment system. The criteria listed include the Willamette Basin standards, Willamette River, and Pudding River discharge criteria, reuse criteria for land application of effluent and biosolids, and EPA criteria for reliability and redundancy.

Willamette Basin Water Quality Standards

The standards for river basins in the State of Oregon are established by the Department of Environmental Quality (DEQ) through the Oregon Administrative Rules (OAR) 340-41-445. These rules are reviewed on a yearly basis for setting new or modifying existing standards. The following presents a discussion of State water quality standards for specific reaches of the Willamette River and its tributaries.

Dissolved Oxygen (DO). Table 4-1 presents the DO standards as a function of specific river location and classification:

Table 4-1
Dissolved Oxygen Standards*

Location of Outfall	Standard
Willamette River from Willamette Falls to Newberg, river mile 50:	The DO concentrations shall not be less than 6 mg/L.

*From OAR 340-41.

Temperature. Table 4-2 presents temperature standards as a function of specific river location and classification. The temperature standard is currently under review as part of the Trienniel Review Process. Current discussions indicate that there may be additional best practicable treatment standards imposed if river levels exceed temperatures necessary for fish survival.

Turbidity. No more than a 10 percent cumulative increase in natural stream turbidities shall be allowed, as measured relative to a control point immediately upstream of the turbidity causing activity. However, limited duration activities necessary to address an emergency or to accommodate essential dredging, construction, or other legitimate activities that cause the standard to be exceeded, may be authorized by DEQ provided all practicable turbidity control techniques have been applied.

pH. pH values shall not fall outside the range 6.5 to 8.5.

Table 4-2 Temperature Standards*	
Outfall Location	**Temperature Standards**
Willamette River from mouth to Newberg, river mile 50:	No measurable increase shall be allowed outside of the assigned mixing zone (T>=70°F); or more than 0.5°F increase due to a single source discharge when receiving water temperatures are 69.5°F or less; or more than 2°F increase due to all sources combined when stream temperatures are 68°F or less.
*From OAR 340-41.	

Bacteria. Bacteria of the *coliform group* associated with fecal sources and bacteria of the *enterococci group* shall not exceed the criteria values described below. However, the DEQ can designate site-specific bacteria criteria on a case-by-case basis to protect beneficial uses. Site-specific values shall be described in and included as part of a water quality management plan. The criteria are as follows:

- Freshwaters: A geometric mean of 33 enterococci per 100 mL based on no fewer than five samples, representative of seasonal conditions, collected over a period of at least 30 days. No single sample shall exceed 61 enterococci per 100 mL.

- Existing permit effluent limitations for fecal coliform will remain in effect until permit renewal, or until the DEQ reopens existing permits to include an effluent limit and compliance schedule for enterococci.

- Bacterial pollution or other conditions deleterious to waters used for domestic purposes, livestock watering, irrigation, bathing, or shellfish propagation, or otherwise injurious to public health shall not be allowed.

These regulations are currently being reviewed as part of the Trienniel Review Process. Current discussions indicate that the enterococci standard will be replaced with a fecal coliform standard.

Total Dissolved Solids. A concentration of 100 mg/L shall not be exceeded in the Willamette River and tributaries unless otherwise specifically authorized by DEQ upon such conditions as it may deem necessary to carry out the general intent of the rules and to protect the beneficial uses presented in OAR 340-41-442.

Toxic Substances. Toxic substances shall not to be introduced in the waters of the state in amounts, concentrations, or combinations which may be harmful, may chemically change to harmful forms in the environment, or may accumulate in sediments or bio-accumulate in aquatic life or wildlife to levels that adversely affect public health, safety, or welfare; aquatic

life; wildlife; or other designated beneficial uses. Toxic substances are controlled through the pretreatment program.

Mixing Zone. A mixing zone is defined as a designated portion of a receiving water that serves as a zone of dilution where wastewaters and receiving waters mix thoroughly. The DEQ may suspend all or part of the water quality standards, or set less restrictive standards, in the defined mixing zone under the following conditions:

1. The water within the mixing zone shall be free of:

   - Materials in concentrations that will cause acute toxicity to aquatic life (bioassay testing required and approved by DEQ). Acute toxicity is lethality to aquatic life as measured by significant difference in lethal concentration between the control and 100 percent effluent in an acute bioassay test. Lethality in 100 percent effluent may be allowed due to ammonia and chlorine only when it is demonstrated on a case-by-case basis that immediate dilution of the effluent within the mixing zone reduces toxicity below lethal concentrations.

   - Materials that will settle to form objectionable deposits.

   - Floating debris, oil, scum, or other materials that cause nuisance conditions.

   - Substances in concentrations that produce deleterious amounts of fungal or bacterial growths.

2. The water outside the boundary of the mixing zone shall:

   - Be free of materials in concentrations that will cause chronic (sublethal) toxicity. Chronic toxicity is measured as the concentration that causes long-term sublethal effects, such as significantly impaired growth or reproduction in aquatic organisms, during a testing period based on test species life cycle.

   - Meet all other water quality standards under normal annual low flow conditions.

The DEQ will describe the mixing zone in the wastewater discharge permit. The mixing zone will be defined by DEQ on the basis of receiving water and effluent characteristics. The mixing zone limits or outfall location may be changed if DEQ determines that the water within the mixing zone adversely affects any existing beneficial uses in the receiving waters.

Pudding River Discharge Criteria

The Pudding River is a tributary of the Willamette River. In addition to being subject to the criteria for the Willamette River, the Pudding River has also been defined as water quality limited.

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In an effort to analyze the effects of the Woodburn WWTP discharge on the Pudding River, DEQ developed a model to predict waste loads that could be discharged from the Woodburn WWTP and still meet the water quality standards for the Pudding River. The model assumed the discharge flow from the Woodburn WWTP would be 5 million gallons per day (mgd).

From instream data collected by DEQ, DEQ found that during summer low flow conditions that water quality criteria, specifically the dissolved oxygen standard, are violated on a diurnal basis. Based on the data and the model, DEQ has defined the allowable discharge loads.

The dissolved oxygen content of the wastewater shall not be less than 6.5 mg/L on a daily average to meet the 6.0 mg/L minimum standard.

Table 4-3 presents the effluent concentrations corresponding to the ammonia mass loads for the summer time average day flow of 5.0 mgd.

The effluent concentrations for $CBOD_5$ and TSS would be 10 and 10 mg/L for a monthly average, respectively, and 20 mg/L for a daily maximum.

For the winter period from November 1 through April 30, DEQ concluded that the Pudding River has assimilative capacity; therefore, no waste load allocations are necessary for winter discharge.

Wastewater Effluent Reuse Criteria

An alternative to direct river discharge of treated effluent during the dry weather period is to apply the treated effluent for beneficial reuse, such as irrigation. Effluent reuse can also be achieved by providing reclaimed water for specific nonagricultural industrial uses such as cooling water. The standards for effluent reuse in the State of Oregon are established by the DEQ through OAR 340-55.

Application Rates

The goal of a wastewater beneficial reuse program for agricultural use is to beneficially reuse the treated effluent by applying at rates to meet the crop's gross irrigation and nutrient requirements, which are commonly referred to as agronomic rates.

The gross irrigation requirement is the total crop water demand adjusted for effective precipitation, irrigation application efficiency, and soil moisture storage. Table 4-4 summarizes the gross irrigation required for crops commonly grown in the Willamette Valley near the City of Woodburn. The gross irrigation required was calculated from the net irrigation requirement with an 80 percent application efficiency. The net irrigation requirement for the various crops was obtained from the Oregon State University report, Oregon Crop Water Use and Irrigation Requirements, October 1992. Because of the intense management and appearance requirements for turf grass at golf courses, a 25 percent increase to the net irrigation requirement for pasture grass was used. The application efficiency for irrigation systems at golf courses is typically higher because of night irrigation; therefore, an efficiency of

85 percent was assumed. Gross irrigation requirements for popular trees are 45 acre-inches per acre (ac-in/ac).

Table 4-3
City of Woodburn WWTP
Ammonia-N Waste Load Allocations for Pudding River

June		
Pudding River Monthly Average Flow, cfs	**Monthly Average Effluent Ammonia-N Concentration Not to be Exceeded mg/L**	**Daily Maximum Effluent Ammonia-N Concentration Not to be Exceeded mg/L**
>150	No WLA	
100-150	5.7	8.6
50-100	3.1	4.7
<50	1.1	1.7
July and August		
Pudding River Monthly Average Flow, cfs	**Monthly Average Effluent Ammonia-N Concentration Not to be Exceeded mg/L**	**Daily Maximum Effluent Ammonia-N Concentration Not to be Exceeded mg/L**
>60	1.0	1.5
30-60	0.50	0.75
<30	0.10	0.18
September and October		
Pudding River Monthly Average Flow, cfs	**Monthly Average Effluent Ammonia-N Concentration Not to be Exceeded mg/L**	**Daily Maximum Effluent Ammonia-N Concentration Not to be Exceeded mg/L**
>100	No WLA	
60-100	6.8	10.2
30-60	3.3	5.0
<30	1.6	2.4

Note:

Waste load allocations displayed above represent oxygen-demanding mass load limits for specific flow scenarios in the Pudding River. These allocations will be fixed. The actual concentration and mass load limits may be represented differently in the waste discharge permit, however, depending on the specific wastewater control alternatives selected by the City or Agripac. For example, if either of the sources choose to irrigate a portion of their effluent, higher concentration limits may be permitted because effluent discharge flows would be less.

Also, the waste load allocations were derived by distributing the loads based upon an equal percent removal of influent ultimate oxygen demand from both sources. DEQ considered several other options, but believes this approach is as equitable as any. Load allocations for nonpoint sources are assumed to stay unchanged.

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The nutrient requirement is the amount of fertilizer, such as available nitrogen, phosphorus, and potassium, that is needed to obtain an optimum crop yield. The available nitrogen is made up of organic nitrogen, ammonia-nitrogen, and nitrate-nitrite nitrogen. Organic nitrogen is a long-term, slow-release fertilizer. As organic matter decomposes in the soil, microorganisms convert the organic nitrogen to inorganic ammonium nitrogen, a process called mineralization. Other organisms then convert the ammonium to nitrate; this process is called nitrification. The assumptions that will be used to determine the available nitrogen are:

- Ninety percent of mineralized organic nitrogen (TKN-ammonia nitrogen) will be available to the crop in the first year. The balance will be lost to volatilization.
- There will be a 50 percent loss of ammonia from volatilization as a result of application with sprinklers; therefore, 50 percent will remain available.
- One hundred percent of nitrate-nitrite nitrogen will be available for crop use.

Table 4-5 summarizes the fertilizer requirements typically used in the Willamette Valley for various crops.

Table 4-4
Gross Irrigation Requirements of Various Crops for Willamette Valley Region
(ac-in/ac)

Month	Turf Grass	Pasture Grass	Field Corn	Alfalfa Hay	Spring Grains	Winter Grains	Grass Seed (fall)
January	0.0	0.0	0.0	0.0	0.0	0.0	0.0
February	0.0	0.0	0.0	0.0	0.0	0.0	0.0
March	0.06	0.05	0.0	0.0	0.0	0.0	0.05
April	0.30	0.25	0.0	0.15	0.1	0.05	0.25
May	2.3	1.96	0.05	1.78	1.88	0.3	2.71
June	4.5	3.84	1.28	3.45	4.58	1.96	4.58
July	8.2	6.94	7.39	6.45	6.89	4.73	7.3
August	6.5	5.51	6.25	5.08	0.64	3.59	5.66
September	3.1	2.61	1.18	2.31	0.0	1.18	0.54
October	0.24	0.20	0.0	0.15	0.0	0.1	0.10
November	0.0	0.0	0.0	0.0	0.0	0.0	0.0
December	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	**25.13**	**21.36**	**16.14**	**19.36**	**14.08**	**11.9**	**21.8**

Notes:
1. Net irrigation requirement is based on meeting 5 out of 10 years for the various crops, Willamette Valley Region. Source: *Oregon Crop Water Use and Irrigation Requirements*, October 1992.
2. Irrigation efficiency (E) = 80 percent for crops other than turf grass; 85 percent for turf grass.
3. Gross irrigation required = Net Irrigation Required/Efficiency.
4. Turf grass irrigation requirement applies to golf courses only and assumes a 25 percent increase over pasture grass because of the intense management of a golf course.

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Table 4-5 Fertilizer Requirements of Various Crops for Willamette Valley Region (lb/ac)						
Nutrient	Pasture Grass/ Turf Grass	Field Corn	Alfalfa Hay	Spring Grains	Winter Grains	Grass Seed (fall)
Nitrogen	180-250	150-180	200-480	40-50	100-140	100-140
Phosphorus	50-75	20-30	20-30	40-60	30-60	30-60
Potassium	240-290	100	160-200	40	30-100	60

Notes:

1. Nutrient uptake rates for pasture/turf grass, alfalfa, and field corn were taken from EPA's *Process Design Manual for Land Treatment of Municipal Wastewater.*
2. Nutrient uptake rates for spring grains, winter grains, and grass seed were taken from Oregon State University fertilizer guides.
3. Alfalfa hay does not require N fertilization, but is capable of utilizing the rates indicated.

Seasonal Limitations/Storage Requirements

As seen in Table 4-4, there are seasonal limitations to an effluent reuse system because there is no crop water requirement during parts of the year (November through February) and very little crop water requirement in October. Most of the crops grown in the Woodburn area will typically have a growing season from April through October.

Effluent that is not used for irrigation or discharged can be stored in reservoirs. Some municipalities do this to avoid advanced wastewater treatment during the dry season. The stored effluent is typically discharged during the wet season when effluent requirements are less stringent. DEQ requires reservoirs designed for treated effluent to be lined to prevent any potential leaching into groundwater.

Treatment and Monitoring Requirements

Through OAR 340-55, DEQ has established treatment and monitoring requirements for potential agricultural and nonagricultural uses of the treated effluent. DEQ has classified reclaimed water into four categories and assigned a minimum degree of treatment required:

- Level I: Less than biological treatment or biological treatment without disinfection.
- Level II: Biological treatment plus disinfection.
- Level III: Biological treatment plus disinfection.
- Level IV: Biological treatment, clarification, coagulation, and filtration treatment plus disinfection.

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Limits for total coliform (organisms/100 mL) and turbidity (NTU) have been established for the four categories. These standards serve as a general guideline for defining the anticipated water quality required for the various uses. In addition to the water quality limits, DEQ has provided standards for the minimum monitoring required for total coliform and turbidity based on the four categories. Table 4-6 summarizes the treatment and monitoring requirements for the four reuse categories. DEQ may include additional permit effluent limitations and/or other permit conditions other than those shown in Table 4-6 if DEQ has reason to believe that the reclaimed water may contain physical or chemical contaminants that would impose potential hazards to the public or environment.

General Requirements

A number of general requirements have been outlined in OAR 345-55. These requirements address agricultural and nonagricultural uses that are acceptable based on the effluent water quality level, irrigation system, public access requirements, and buffer zones for irrigation. Table 4-7 summarizes these general requirements based on the different levels of reclaimed water quality.

Table 4-6
Treatment and Monitoring Requirements for Agricultural Use of Reclaimed Water*

	Reuse Category Level Minimum Degree of Treatment Required			
	I	**II**	**III**	**IV**
	Less than biological treatment or biological treatment without disinfection	**Biological treatment plus disinfection**	**Biological treatment plus disinfection**	**Biological, clarification, coagulation, and filtration treatment plus disinfection**
Reclaimed Water Quality				
Total coliform (#/100 mL)				
7-day median	No limit	23	2.2	2.2
Two consecutive samples	No limit	240	No limit	No limit
Maximum	No limit	No limit	23	23
Turbidity (NTU)				
24-hour mean	No limit	No limit	No limit	2
5% of the time during any 24-hour period	No limit	No limit	No limit	5
Minimum Monitoring Requirements				
Total coliform	Not required	One sample/week	Three samples/ week	Daily
Turbidity	Not required	Not required	Not required	Hourly or continuous

* From OAR 340-55.

Table 4-7
Agricultural Use Allowed with Different Levels of Reclaimed Water Quality (from OAR 340-55)

	Reuse Category Level Minimum Degree of Treatment Required			
	I	**II**	**III**	**IV**
	Less than Biological Treatment or Biological Treatment without Disinfection	**Biological Treatment Plus Disinfection**	**Biological Treatment Plus Disinfection**	**Biological, Clarification, Coagulation, and Filtration Treatment Plus Disinfection**
General Agricultural Uses - Irrigation Method Allowed				
Food crops	Not allowed	Not allowed	Not allowed	Surface or spray
Processed food crops[b]	Not allowed	Surface or spray	Surface or spray	Surface or spray
Orchards and vineyards	Surface[c]	Surface[c]	Surface[c]	Surface or spray
Fodder, fiber, and seed crops[d]	Surface or spray	Surface or spray	Surface or spray	Surface or spray
Pasture for animals	Not allowed	Surface or spray[e]	Surface or spray[e]	Surface or spray
Specific Agricultural Uses - Irrigation Method Allowed				
Produce–general (lettuce, carrots, etc.)	Not allowed	Not allowed	Not allowed	Surface or spray
Tomatoes (unprocessed)	Not allowed	Not allowed	Surface[c]	Surface or spray
Tomatoes (processed–no gleaning)	Not allowed	Surface or spray	Surface or spray	Surface or spray
Strawberries	Not allowed	Not allowed	Not allowed	Surface or spray
Sugar beets	Not allowed	Surface or spray	Surface or spray	Surface or spray
Grain–for human consumption	Not allowed	Not allowed	Surface[c]	Surface or spray
Trees and vines	Surface	Surface or spray[f]	Surface or spray[f]	Surface or spray
Nuts	Not allowed	Not allowed	Surface or spray	Surface or spray
Other crops: sod	Not allowed	Not allowed	Surface or spray	Surface or spray
Ornamental nursery stock	Not allowed	Not allowed	Surface or spray	Surface or spray
Christmas trees	Not allowed	Surface or spray	Surface or spray	Surface or spray
Firewood: customer cut	Not allowed	Surface or spray	Surface or spray	Surface or spray
Firewood: not customer cut	Surface or spray	Surface or spray	Surface or spray	Surface or spray

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Table 4-7
Agricultural Use Allowed with Different Levels of Reclaimed Water Quality (from OAR 340-55)

	A			
	I	II	III	IV
	Less than Biological Treatment or Biological Treatment without Disinfection	Biological Treatment Plus Disinfection	Biological Treatment Plus Disinfection	Biological, Clarification, Coagulation, and Filtration Treatment Plus Disinfection
Nonagricultural Uses - Irrigation Method Allowed				
Parks, playgrounds, schoolyards, golf courses with contiguous residences	Not allowed	Not allowed	Not allowed	Surface or spray[g,h]
Golf courses without contiguous residences	Not allowed	Surface or spray[g,h]	Surface or spray[g,h]	Surface or spray[g,h]
Cemeteries, highway medians, landscapes without frequent public access	Not allowed	Surface or spray[g,h]	Surface or spray[g,h]	Surface or spray[g,h]
Unrestricted Impoundments	Not allowed	Not allowed	Not allowed	Surface or spray[g,j]
Restricted Impoundments	Not allowed	Not allowed	Surface or spray[g,j,k]	Surface or spray[g,j]
Landscape Impoundments	Not allowed	Surface or spray[g,j,k]	Surface or spray[g,j,k]	Surface or spray[g,j]
Other Requirements				
Public access	"Prevented" (fences, gates, locks)	"Controlled" (signs, rural or nonpublic lands)	"Controlled" (signs, rural or nonpublic lands)	No direct public contact during irrigation cycle
Buffers for Irrigation	Surface: 10 ft Spray: Site-specific	Surface: 10 ft Spray: 70 ft	Surface: 10 ft Spray: 10 ft	None Required

[a]Not acceptable for root crops or crops where edible parts touch the ground.

[b]Processed food crops must undergo extensive commercial, physical, or chemical processing sufficient to destroy pathogenic agents. Processing does not include washing, pickling, fermenting, or milling.

[c]Edible portion of plant does not contact the ground.

[d]Not for human ingestion.

[e]No animals shall be on the pasture during irrigation.

[f]No spraying within 30 days of fruit formation.

[g]Signs shall be posted around the perimeter and other locations indicating that reclaimed water is used and is not safe for drinking, and in the case of effluent quality Levels II and III, for body contact.

[h]Reclaimed water shall be applied in a manner so that it is not sprayed onto areas where food is prepared or served or onto drinking fountains.

[i]Reclaimed water shall be applied in a manner so that it is not sprayed within 100 feet from areas where food is prepared or served or where drinking fountains are located.

[j]There shall be no disposal of reclaimed waters into surface or groundwaters without authorization by an NPDES or WPCF permit.

[k]Aerators or decorative fixtures that may generate aerosols shall not be used unless approved in writing by DEQ.

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Agricultural and Nonagricultural Uses

Agricultural uses are divided into general agricultural and specific agricultural uses. General agricultural uses cover irrigation for food crops, processed food crops, orchards and vineyards, fodder, fiber, and seed crops, and pasture for animals. Specific agricultural uses range from general produce (such as lettuce and carrots) to poplar trees. Nonagricultural uses cover irrigation at parks, playgrounds, golf courses, cemeteries, highway medians, and other landscape irrigation.

Because the existing WWTP could produce a Level II quality effluent, the potential uses range from the irrigation of agricultural crops processed before human consumption or crops not for human consumption, to irrigation at golf courses without "contiguous" residences. Level IV effluent is the least restrictive with respect to the types of uses for which the treated effluent can be beneficially reused and is the most costly to produce.

DEQ provides guidelines on public access and buffer zones for irrigation systems depending on the effluent water quality level beneficially reused. As illustrated in Table 4-7, public access requirements for the different effluent levels range from "prevented" (fences, gates, locks) to no direct public contact during the irrigation cycle. The current level of effluent from the City of Woodburn WWTP approaches Level II. The disinfection limit is the only criterion the plant is not currently required to meet, although the plant is capable of meeting the Level II standard. The public access under a Level II effluent quality reuse program must be "controlled." This means that irrigation using this effluent can only occur on rural or nonpublic lands that limit the potential for direct public contact. The site used would also require signs indicating the use of reclaimed water in the irrigation system. This level of public access control would be similar for Level III effluent quality. It would be reduced to no restrictions except prevention of direct public contact during the irrigation cycle under a reuse program using Level IV effluent quality.

Buffer zones for surface and spray irrigation systems are intended to protect public health and the environment. As with the public access requirements, the buffer zones are least restrictive for Level IV effluent quality. Assuming the City of Woodburn WWTP achieves Level II effluent quality, the buffer zones for surface (flooding and overland flow) and spray irrigation systems would be 10 and 70 feet, respectively. DEQ may reduce the buffer distances, as identified in Table 4-7, if it determines that alternative controls would adequately protect public health and the environment.

To achieve Level IV, additional treatment such as coagulation, and more stringent disinfection and turbidity effluent levels would be required.

Other Reuse Requirements

Other requirements to consider in designing a wastewater reuse system are alarm devices, standby power, redundancy, cross-connection, and construction and marking of piping, valves, and other portions of the reclaimed water system. As outlined in OAR 340-55, alarm devices are used to provide the necessary warning of loss of power and/or failure of process equipment essential to the proper operation of the WWTP. This requirement is consistent

with the design guidelines of any WWTP whether or not a wastewater effluent reuse system is implemented. In addition to the alarms, appropriate redundancy is required to have a sufficient level of treatment facilities and monitoring equipment available to effectively prevent inadequately treated water from being used or discharged to public waters.

There is no cross-connection allowed between a potable water system and the distribution system carrying the reclaimed water unless the connection is through either an unrestricted air gap or a reduced pressure principle backflow preventer. This backflow preventer must be tested and serviced professionally at least once per year. Unless approved by DEQ, construction and marking of piping, valves, and other portions of the reclaimed water system must conform with requirements outlined in the "Guidelines for Distribution of Nonpotable Water" of the California-Nevada Section of the American Water Works Association. In general, the requirements that have not already been discussed are:

- Pipe Separation: Potable pipelines must maintain a separation of 10 feet horizontally and one foot vertically with parallel reclaimed water (nonpotable) pipelines. When potable pipelines cross reclaimed water pipelines, the potable water pipeline must maintain a separation of one foot above the reclaimed water pipeline.

- Pipe and Valve Identification: Reclaimed water pipeline must be adequately marked with a warning tape. The warning tape should be prepared with specified purple color and printing with the words CAUTION: RECLAIMED WATERLINE. Aboveground or exposed facilities should be marked to differentiate reclaimed water pipelines from potable water systems or wastewater facilities.

Biosolids Management Criteria

Both federal and state regulations apply to land application of biosolids from WWTPs. Federal regulations include 40 CFR 257 and newly approved Part 503 regulations. The Oregon regulations include OAR 340-50. The State of Oregon also publishes guidance documents for interpreting and following the regulations. These materials include *Guidelines for Land Application of Wastewater and Sludge, May 18, 1981*, and a *Sludge/Septage Management Plan Submittal Checklist.*

For disposal of sludge as interim cover or as fill at a solid waste landfill, federal regulations 40 CFR 258 apply. If the sludge were incorporated in the final cover for the landfill, the 503 regulations would still apply.

State regulations take precedence over federal regulations, where applicable. In some instances, state regulations may impose more stringent requirements than federal regulations. However, federal regulations apply if no state regulations are declared.

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Regulations

Federal Regulations

Current federal regulations for land treatment of biosolids are listed in the Federal Register under 40 CFR 257, "Criteria for Classification of Solid Waste Disposal Facilities and Practices," dated September 13, 1979. In the past, Part 257.3-5 has regulated solid waste application to food chain crops; however, these regulations have been considered too general. Therefore, new regulations under 40 CFR 503 were required by Section 405 (d) of the Clean Water Act of 1977 (as amended by the Water Quality Act of 1987).

The new regulations under 40 CFR 503 have gone through several scientific community and public reviews and were released as final in late 1992.

State Regulations

In December 1984, DEQ defined rules for the land application and disposal of sewage treatment plant biosolids and biosolids-derived products, including septage (OAR 340-50). These regulations remain current for the State of Oregon although the State may be in the process of updating the rules to conform to recently adopted federal regulations.

Biosolids Quality

According to current state and new federal regulations (40 CFR 503), biosolids samples should be analyzed for the parameters listed in Table 4-8.

The nitrogen, phosphorus, and potassium content of the sludge are important when applying biosolids at agronomic rates. Nitrogen content can vary significantly in the biosolids depending on its source, age, and history. The concentration levels of these nutrients should be determined from samples taken immediately before biosolids are applied because stored biosolids can lose nitrogen rapidly. Therefore, it is important that the real nitrogen content of the biosolids is known to avoid under- or over-application. The assumptions used to determine the available nitrogen in the biosolids were:

- 30 percent of the organic nitrogen will be available
- 50 percent of the ammonia nitrogen will be available
- 100 percent of the nitrate-nitrite nitrogen will be available

Under the new federal regulations Part 503, ceiling concentrations, cumulative pollutant loading rates, exceptional quality or "clean biosolids," and annual pollutant loading rate have been established for heavy metals. Table 4-9 shows the acceptable levels for land application based on federal regulations. These regulations are somewhat different from the state regulations. Table 4-10 shows the acceptable levels of metals in biosolids for land application based on the state regulations. Cumulative loading limits for the metals are also established and are dependent on the soil cation exchange capacity (CEC) (see Table 4-9). These rates are used to determine site life, which is the number of years that biosolids with a

uniform metal content could be applied to a specific site. Regardless of CEC, if soil pH is less than 6.5, cumulative loading of cadmium cannot exceed 5 kg/ha (4.5 lb/ac). The soil can be limed to increase the soil pH and thereby increase the site life.

Table 4-8
Sampling Requirements for the EPA 503 Sludge Regulations[a]

Parameter	Units
Arsenic	mg/kg dry weight
Beryllium	mg/kg dry weight
Cadmium	mg/kg dry weight
Chromium	mg/kg dry weight
Copper	mg/kg dry weight
Lead	mg/kg dry weight
Mercury	mg/kg dry weight
Molybdenum	mg/kg dry weight
Nickel	mg/kg dry weight
Selenium	mg/kg dry weight
Zinc	mg/kg dry weight
Total Nitrogen	% dry weight
Nitrate Nitrogen	% dry weight
Ammonia Nitrogen	% dry weight
Phosphorus	% dry weight
Potassium	% dry weight
pH	standard units
Total Solids	%
Volatile Solids	%
PCBs[b]	µg/kg

[a]From 40 CFR 503.
[b]PCBs include PCB-1016,-1221,-1232,-1242,-1248,-1254, and -1260.

Table 4-9
New Federal Regulations (Part 503) for Heavy Metals*

Parameter	Ceiling (mg/kg)	Cumulative Loading (kg/ha)	Exceptional Quality (mg/kg)	Annual Pollutant Loading Rate (kg/ha/yr)
Arsenic	75	41	41	2.0
Cadmium	85	39	39	1.9
Chromium	3,000	3,000	1,200	150
Copper	4,300	1,500	1,500	75
Lead	840	300	300	15
Mercury	57	17	17	0.85
Molybdenum	75	18	18	0.90
Nickel	420	420	420	21
Selenium	100	100	36	5.0
Zinc	7,500	2,800	2,800	140

* From 40 CFR 503.

Table 4-10 State Regulations for Land Application of Biosolids[a]				
		Maximum Metal Addition (kg/ha)[b]-Cumulative		
Metal	Acceptable Concentrations (mg/kg)	CEC[c] Less than 5 meq[d]/100 g	CEC 5 to 15 meq/100 g	CEC greater than 15 meq/100 g
Lead (Pb)	1,000	500	1,000	2,000
Zinc (Zn)	2,000	250	500	1,000
Copper (Cu)	800	125	250	500
Nickel (Ni)	100	50	100	200
Cadmium (Cd)[e]	25	5	15	20

[a]From OAR 340-50.
[b]kg/ha is equivalent to 0.89 lb/acre.
[c]Soil cation exchange equivalent.
[d]Milliequivalent.
[e]The maximum application of cadmium (Cd) for soils with pH values of 6.5 or less is 4.5 lb/acre, regardless of the CEC.

Site Identification and Approval

Before approving any potentially sensitive application site (with respect to residential housing, runoff potential, or groundwater threat), DEQ may require an opportunity for public comment and public hearing. A statement of land use compatibility from the responsible planning jurisdiction should accompany requests for approval of biosolids land application sites. New sites or expansion of existing sites must be proposed to DEQ before use. Newly approved sites become part of the sludge management plan.

Site criteria for land applying biosolids includes physical geographical features (geological formation, floodplain proximity, and groundwater and surface water proximity, topography, and soils), and method of application. DEQ's specific criteria are outlined in Table 4-11.

Special Management Considerations

Land receiving biosolids for agricultural use requires special management considerations. These relate to access to the site, types of crops grown, plant nutrient rates, timing and duration of biosolids land application (site life and seasonal constraints), and grazing restrictions.

Access

Controlled access to municipal biosolids application sites is required for 12 months following a surface application. Controlled access means that public entry or traffic is unlikely. Rural private land is assumed to have controlled access while parks or other public lands may require fencing to ensure control.

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Table 4-11 Oregon DEQ Site Criteria for Biosolids Application[a]	
Parameter	**Criteria**
Geology	Must have a stable formation
Floodplain	Restricted period of application and incorporate biosolids if in a floodplain
Groundwater	At time of application, the minimum depth to permanent groundwater is 4 feet; the minimum depth to temporary groundwater is 1 foot
Topography • Slope less than or equal to 12 percent • Slope greater than 12 percent but less than 20 percent	• Liquid biosolid application with appropriate management to eliminate surface runoff • Surface application of dewatered or dried biosolids • Direct incorporation of liquid biosolids into the soil
Soils	• Minimum rooting depth of 24 inches • No rapid leaching • Avoid saline or alkali soil • pH of 6.5 to 8.2 where heavy metal accumulator crops[b] are grown (pH can be raised by liming the soil)
Method of application and proximity to water bodies	Buffer strips may be required to protect water bodies. Size depends on method of application and proximity to sensitive area (variable with local conditions and left to discretion of DEQ), as described below: • Direct injection: no limit required • Truck spreading: less than 50-foot buffer strip • Spray irrigation: 300- to 500-foot buffer strip • Near ditch, pond, channel, or waterway; greater than 50-foot buffer strip • Near domestic water source or well; greater than 200-foot buffer strip

[a]From OAR 340-50.
[b]Heavy metal accumulator crops are crops such as Swiss chard, lettuce, spinach, carrots, and other leaf and root crops that have been shown to readily accumulate heavy metals.

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Crops

No biosolids or biosolids-derived product is to be used directly on fruits or vegetables that may be eaten raw, and as a general rule, crops grown for human consumption should not be planted until 18 months after municipal biosolids application. If the edible parts will not be in contact with the biosolids-amended soil, or if the crop is to be treated or processed before marketing such that pathogen contamination is not a concern, this requirement may be waived.

No restrictions on planting times have been placed on crops not grown for direct human consumption.

Nutrient Loading

Biosolids application to agricultural land should not exceed the annual nitrogen loading required for maximum crop yield and is, therefore, managed according to its fertilizer value. Biosolids may be applied to approved sites above agronomic rates on a one-time basis or less than once per year as long as runoff, nuisance conditions, or groundwater contamination do not occur. Nitrogen accumulation from higher than agronomic rates and annual nitrogen use will determine the acceptable loading rate and frequency.

Site Life

Site life is important in planning because sites generally have a limited application life based on the chemistry of the soil and the metals loading from the biosolids. Site life is calculated by dividing lifetime biosolids loading limits based on the most limiting constituent by the annual application rate.

Seasonal Constraints

In western Oregon, where soil damage may occur from application equipment traffic in the wet season, biosolids application should be restricted to the dry season. The main consideration in land applying on sloping ground is to avoid surface runoff and soil erosion.

Grazing Restrictions

Grazing animals should not be allowed on pasture or forage for 30 days after application of digested biosolids, 180 days after application of nondigested biosolids, and 7 days after application of air-dried biosolids.

Site Monitoring and Reporting

No site monitoring is required where biosolids are applied at or below agronomic rates based on crop nitrogen requirements. However, if the biosolids contain high concentrations of heavy metals or other toxic elements, or if crop nitrogen requirements are exceeded on an annual basis, additional soil monitoring and special management practices may be required. Monitoring wells may be required on any site on a case-by-case basis at the discretion of

DEQ. Also, groundwater background characterization and/or monitoring may be required at the discretion of DEQ.

Reliability and Redundancy Criteria

New or expanding treatment works are required to meet minimum standards for mechanical, electrical, fluid systems, and component reliability in accordance with EPA's policy. This helps treatment facilities operate effectively on a day-to-day basis and provides the capability for satisfactory operation during power failures, flooding, peak loads, equipment failures, and maintenance shutdowns. These reliability and redundancy standards are important to ensure that unacceptable degradation of the receiving water will not occur as a result of the interrupted operation of specific treatment units or processes. In that regard, standards have been established for three classes of wastewater treatment works.

The reliability class appropriate for the Woodburn WWTP will depend on the effluent disposal receiving stream or body of water. For discharge to the Pudding River, it is anticipated that reliability Class I would be appropriate for the Woodburn WWTP, since the discharge from the existing and future facility is to a water quality limited stream.

Table 4-12 contains the minimum backup requirements for plant components that may be provided at the Woodburn WWTP in accordance with the EPA's Works Design Criteria, Reliability Class I, for sewage treatment plants. In addition to the standards listed in the table, unit operations must be designed to pass the peak hydraulic flow with one unit out of service. Also, mechanical components in the facility must be designed to enable repair or replacement without violating the effluent limitations or causing control diversion.

The reliability criteria for sludge processes presented in Table 4-13 are also based on the guidance offered in the EPA's Works Design Criteria. Table 4-13 is not specific to the Woodburn WWTP, and all elements presented are not necessarily included in the existing or future facilities. The most significant difference between Class I and Class II reliability is that for secondary sedimentation only 50 percent design capacity is required with one unit out of service for Level II reliability. Also, backup components are not mandatory for wastewater treatment systems used to provide treatment in excess of typical biological treatment and disinfection.

Table 4-12 Reliability Class I Requirements	
Plant Component	**Requirement**
Raw Sewage Pumps	Peak flow with largest unit out of service. Peak flow is defined as the maximum wastewater flow expected during the design period of the treatment works.
Mechanical Bar Screens	One backup with either manual or mechanical cleaning (manual cleaning if only two screens)
Grit Removal	Minimum of two units.
Primary Sedimentation	50% of design flow capacity with largest unit out of service. Design flow is defined as the flow used as the design basis of the component.
Activated Sludge Process	A minimum of two equal volume basins; no backup basin required.
Aeration Blowers	Supply the design air capacity with the largest unit out of service; provide a minimum of two units.
Air Diffusers	Isolation of largest section of diffusers (within a basin) without measurably impairing oxygen transfer.
Secondary Sedimentation	75% of design flow capacity with largest unit out of service. Design flow is defined as the flow used as the design basis of the component.
Disinfectant Contact Basin	50% of the design flow with largest unit out of service. Design flow is defined as the flow used as the design basis of the component.
Effluent Pumps	Peak flow with largest unit out of service. Peak flow is defined as the maximum wastewater flow expected during the design period of the treatment works.
Electrical Power	Two separate and independent sources of electrical power shall be provided, either from two separate utility substations or from a single substation and a works-based generator. Designated backup source shall have sufficient capacity to operate all vital components, critical lighting, and ventilation during peak flow conditions, except that components used to support the secondary processes need not be included as long as treatment equivalent to sedimentation and disinfection is provided.

Table 4-13 Sludge Handling System Reliability	
System Components	**Required Capacity/Backup**
Sludge Holding Tanks	The volume of the holding tank shall be based on the expected time necessary to perform maintenance and repair of the component in question.
Anaerobic Sludge Digestion	At least two digestion tanks shall be provided. At least two of the digestion tanks provided shall be designed to permit processing all types of sludges normally digested.
Aerobic Sludge Digestion	A backup basin is not required. At least two blowers or mechanical aerators shall be provided. Isolation of largest section of diffusers without measurably impairing oxygen transfer is allowed.
Sludge Pumping	Pumps sized to pump peak sludge quantity and maintain velocities above 2 fps. Provide a minimum of 2 pumps.

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Section 5

Liquid Treatment and Effluent Disposal Alternatives

Introduction

This section discusses the multiple alternatives available to meet water quality objectives and balance the wastewater system needs of the City of Woodburn, now and in the future. Alternatives are available for each component of the treatment system.

As an initial stage of the facilities planning process, the Woodburn Wastewater Advisory Committee evaluated Big Picture alternatives that considered treatment plant siting issues, wetland tertiary treatment, Willamette River disposal, and crop irrigation. Before this, an analysis was also conducted on treating industrial loads separately, which was found to be more costly than keeping them combined with domestic flows for treatment. The consensus of the committee, the Council, and the public was to recommend upgrading and expanding the existing plant. Wetlands tertiary treatment, discharge to the Willamette River, and storage for 6 months were seen to be too costly. A summary of the Big Picture Analysis is found in Appendix 5.1. The selected Big Picture alternative was then broken down into components for step-wise screening.

In addition, Sidestream Elevated Pool Aeration (SEPA) was evaluated to determine if in-stream re-aeration would minimize the required treatment plant improvements. The analysis showed that the SEPA alternative was not cost effective. This section deals exclusively with liquid processing and effluent disposal components of the selected Big Picture alternative; the solids management components are discussed in Section 6.

Preliminary Treatment

No-Action Alternative

The No-Action alternative would be to leave the existing headworks as it is. This would result in insufficient hydraulic capacity for future flows which may cause sewage spills. It would result in noncompliance with the City's NPDES permit.

New Headworks Alternative

A new headworks would allow sufficient hydraulic capacity for preliminary treatment and flow measurement. It would also provide screenings removal, and more efficient grit removal. Odor control would also be provided. Although two grit units are recommended for redundancy and reliability, the vortex type of grit removal contains no submerged moving parts and is therefore very reliable. A waiver will be requested.

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Primary Treatment

No-Action Alternative

The No-Action alternative would leave existing primaries as they are. The existing primaries are sufficient to treat flow up to approximately 3.3 mgd. The no-action alternative would require increased secondary treatment capacity in the future.

Additional Primary Treatment Alternative

One additional primary clarifier is needed to treat flows up to 5.0 mgd.

Secondary Treatment

Screening of secondary treatment alternatives mainly involves selection of the appropriate biological secondary treatment process. The alternatives considered were: activated sludge with selector technology (ASST) and sequencing batch reactors (SBR). These alternatives were selected on the basis of reliability, site-constraint considerations and the need to remove ammonia by nitrification and denitrification. No action and expanding/upgrading the existing rotating biological contactor (RBC) process were also considered.

Alternative 1: Rotating Biological Contactors

No-Action Option

The No-Action option would be to leave the existing facilities as they are. This would result in overloading of the RBCs and would result in noncompliance with the City's NPDES permit and DEQ's Stipulation and Final Order (SFO). The SFO states new facilities will be constructed 44 months after the facilities plan is approved. New total maximum daily loads (TMDLs), as well as current plant performance, necessitate plant upgrades. As discussed in Section 3, Future Conditions, the recent overloading and performance problems are believed to be primarily a result of higher-than-expected loads from the service area. Treatment improvements are therefore required based on current conditions, regardless of future growth. The No-Action option is therefore not recommended and is not considered further.

Expand/Upgrade Existing Rotating Biological Contactors Option

This option would involve replacement of deteriorating components of the existing shafts and the addition of more RBC units to accommodate the high industrial and slug loadings experienced at the plant. A schematic diagram of this option is shown in Figure 5-1. The option was not considered further because RBCs cannot treat high strength industrial loads well, nor can they remove ammonia.

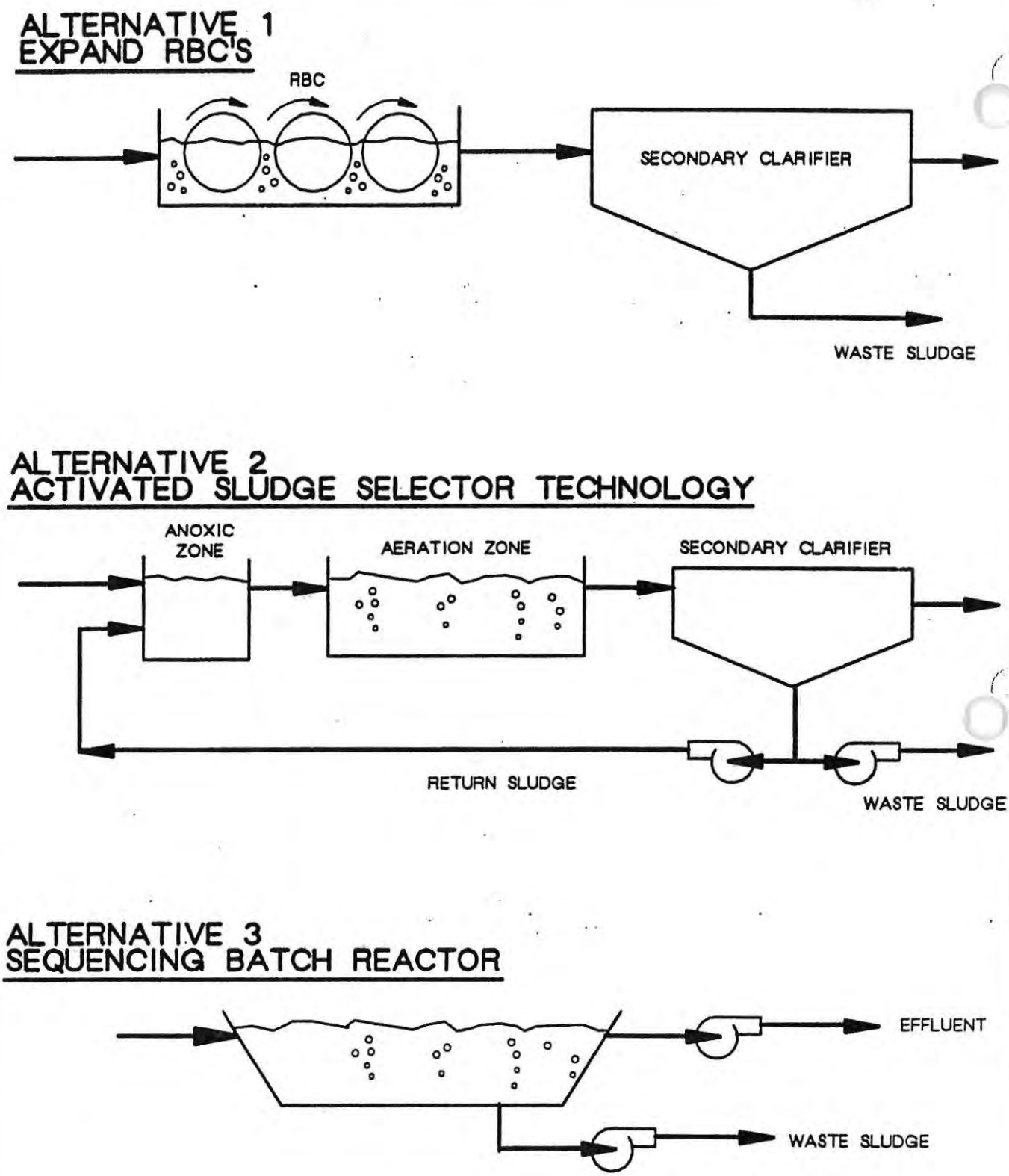

FIGURE 5-1
SECONDARY TREATMENT ALTERNATIVES
WOODBURN WWTP FACILITIES

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CH2M HILL

Alternative 2: Activated Sludge with Selector Technology

The ASST option is a modification of the conventional activated sludge process in which an anoxic (oxygen absent, nitrite/nitrate present) zone is placed upstream of an aerated basin. A plug-flow pattern through the basin is recommended for the process. A baffle wall usually separates the anoxic and aerobic zones. The anoxic selector provides the following benefits over conventional complete mix activated sludge:

- Improved sludge settleability
- Provides, through denitrification:
  - Nitrogen removal
  - Lower oxygen requirement
  - Alkalinity recovery

New secondary clarifiers would be required with this alternative. A schematic of this alternative is shown in Figure 5-1.

Alternative 3: Sequencing Batch Reactor

An SBR is a fill-and-draw activated sludge treatment system. The unit processes involved in the SBR and conventional activated sludge systems are identical. Aeration and sedimentation/clarification are part of both systems. However, there is one important difference. In conventional systems, the processes are carried out simultaneously in separate tanks, whereas in SBR systems, the processes are carried out sequentially in the same tank. As currently used, all SBR systems have five steps in common that are carried out in sequence as follows: (1) fill, (2) react (aeration), (3) settle (sedimentation/clarification), (4) draw (decant), and (5) idle. A feature of the SBR system is that there is no need for return activated sludge (RAS) pumping. Because both aeration and settling occur in the same chamber, no sludge is lost in the react step, and none has to be returned from the clarifier to maintain the sludge content in the aeration chamber. An anoxic selector can also be incorporated into the SBR process by inserting a mixing period without air between steps 1 and 2. A schematic SBR is shown in Figure 5-1.

Comparison of Secondary Treatment Alternatives

Because of their limited organic loading capacity, inability to remove ammonia, and a high odor generation potential, RBCs (Alternative 1) are inherently unsuitable for Woodburn's current and future high-strength waste, as demonstrated by recent experiences. Significant additions occupying a large amount of space would be required to provide the necessary capacity. The existing RBC mechanical equipment (shafts, media, blowers) are close to the end of their service life and an upgrade or expansion would not be cost-effective. It may be possible to salvage the existing concrete RBC structures for use as the anoxic selector or to provide temporary sideline storage for high-strength slugs.

Table 5-1 presents an evaluation of the secondary biological treatment process Alternatives 2 and 3. This evaluation is based on CH2M HILL experience and literature

information. Each alternative is scored on a scale of 1 through 10, with 1 representing the greatest disadvantage and 10 representing the greatest advantage. Each criterion is assigned a weight from 1 through 5 to reflect its significance to the City. Scores and weights were assigned in consultation with the City and based on previous experience. Weighted scores were obtained by multiplying the raw scores by the appropriate weight. The sum of the weighted scores for each alternative may be used to determine the relative overall desirability of the alternative. It can be seen that ASST scored better than the SBR process.

The SBR may have increased costs for hydraulics. A downstream surge tank may be required because of the intermittent nature of SBR effluent discharge and the need for continuous flow through UV disinfection. An intermediate pump station may also be required to compensate for the varying SBR water surface level.

Table 5-1
Comparison of Secondary Biological Process Alternatives 2 and 3

Criteria	Weight	Raw Rank		Weighted Rank	
		Alternative 2 ASST	Alternative 3 SBR	Alternative 2 ASST	Alternative 3 SBR
Operational ease	2	8	6	16	12
Operational flexibility	2	10	6	20	12
Environmental concerns	1	9	9	9	9
Knowledge of process	5	10	5	50	25
Plant size	5	7	8	35	40
Effluent aesthetics	1	10	10	10	10
Phased construction	1	8	6	8	6
Startup ease	1	7	6	7	6
Control	1	8	6	8	6
Automation	1	9	10	9	10
Sampling requirements	1	7	7	7	7
Noise and odor	1	6	6	6	6
Potential BNR flexibility	2	10	9	20	18
Effluent flow impact to downstream processes	2	10	6	20	12
Total	26			225	179
Weighted average				8.7	6.9

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Because of legal compliance deadlines and in keeping with the overall plant upgrade objectives, the City and plant staff favor a well established, proven process with flexibility to meet changing effluent requirements. ASST meets these requirements, whereas SBR represents relatively new technologies with limited full-scale operating experience.

Based on the above considerations, ASST is selected as the biological secondary treatment process for upgrade of the Woodburn WWTP.

Secondary Clarification

No-Action Alternative

The existing clarifiers are too shallow for an activated sludge process. In addition, the existing clarifiers have insufficient solids handling capacity. The No-Action Alternative would result in unsatisfactory secondary effluent. This alternative is no longer considered.

Replace Existing Secondary Clarifiers

The preferred alternative is to replace the existing secondary clarifiers with deeper clarifiers. The deeper clarifiers will have appropriate solids handling capabilities, which include recycle of the return activated sludge (RAS) back to the activated sludge process.

Filtration

Stringent DEQ limits on the effluent TSS and pollutants of concern (POCs) monitored by the pretreatment program necessitate filter modifications to the Woodburn WWTP. The treatment requirements are becoming more stringent. The pretreatment report in 1994 showed that mercury, silver, cadmium, and zinc cannot be adequately removed by the existing secondary treatment and filtration system. Interim limits based on categorical limits have been established until new treatment facilities are constructed. In addition, City staff are also taking measures to reduce these pollutants at their source.

No-Action Alternative

The No-Action alternative would be to leave the existing facilities as they are. The current filters are experiencing capacity limitations. This would result in noncompliance with the City's NPDES permit and DEQ's SFO for TSS limitations and POCs. On January 1, 1993, the Oregon DEQ approved the SFO giving the City a limit for TSS of 20 mg/L (from 10 mg/L) until the improvements to the WWTP are constructed. This alternative is not recommended and is not considered further.

Expand Existing Shallow Bed Filters Alternative

The existing filters are old and maintenance intensive and are inadequate to treat current peak dry weather flows. They are also shallow filters that are not able to treat the

Sodium Bisulfite. Higher storage volumes and doses are required for dechlorination with sodium bisulfite than sulfur dioxide, but it is much safer than sulfur dioxide. It is also more expensive than sulfur dioxide. However, the cost difference is lower than that between chlorine gas and sodium hypochlorite.

Local Fire Marshal Chlorine/Sulfur Dioxide Safety Requirements

If chlorine is selected as the preferred alternative, a meeting with the Fire Marshal for Woodburn should be held to discuss the Fire Department's interpretation of Article 80 of the Uniform Fire Code. A scrubber system as called for by the Uniform Fire Code may be required. A single gas scrubbing system could be used for both chlorine and sulfur dioxide.

Ultraviolet Light Alternative

There are approximately 750 operating ultraviolet (UV) light wastewater disinfection facilities in the United States, ranging from 1 mgd to 256 mgd at the largest installation in Atlanta, Georgia. UV radiation provides no residual. Typical wastewater disinfection installations use low-energy lamps, while medium-energy lamps are more common for stormwater applications. Three banks of lamps and turbulent flow are generally required.

The two main factors affecting UV disinfection performance are suspended solids concentration and transmittance. Transmittance is affected by suspended as well as dissolved materials such as iron and humics. Woodburn WWTP effluent samples should be collected to determine the design transmittance value. Although the secondary treatment process will influence the transmittance, the current value will provide a preliminary indication. A sample should also be taken to test for iron. In addition to process water, iron can reduce the lamp sleeve transmittance by depositing a coating on the sleeve surface.

The UV dose required is controlled by the water quality and the target inactivation level. Recent research shows that the water quality has little effect on the dose for target effluent total coliform levels above approximately 50 MPN/100 mL, but the dose increases rapidly with deteriorating water quality for effluent total coliform below approximately 5 MPN/100 mL. Therefore, for the current and anticipated future disinfection levels required by DEQ, UV can provide effective disinfection at a reasonable dose.

Particle size distribution is also important to UV disinfection. Particles greater than 20 micrometers in diameter are difficult to disinfect using UV radiation, and larger particles cause a shielding effect. Thus, in addition to the suspended solids concentration, the size distribution contributes to the overall water quality. Particle size distribution will be an important design criterion for future wastewater treatment facilities. Effluent that has been filtered is easier to disinfect with UV radiation than nonfiltered effluent.

A major advantage of UV disinfection is its greater potential for compliance with future disinfection standards. This is because UV is more effective against viruses and other indicator organisms (such as Cryptosporidium) likely to appear in future permits. Also, because of the small size of viruses, filtration may become mandatory in the future, and UV

can exploit this advantage more fully than other disinfection methods. Partial or near-complete nitrification does not affect UV effectiveness significantly.

Evaluation of Disinfection Alternatives

The increasingly stringent effluent chlorine residual limits, fire code requirements, and safety and handling and TDS concerns discussed above in relation to chlorine disinfection make the chlorination/dechlorination option highly undesirable for continued use and/or expansion at the Woodburn WWTP. The City, plant staff, and Woodburn Wastewater Advisory Committee have expressed a strong interest in UV disinfection because of its many advantages. UV radiation is therefore selected as the disinfection alternative. Other alternatives are not considered further.

However, there is still a need for in-plant and irrigation use of a disinfectant with chlorine residual. Sodium hypochlorite is recommended for these disinfection needs.

Sustaining Required Dissolved Oxygen

The new TMDLs require that the daily average of the effluent dissolved oxygen concentration should not go below 6.5 mg/L. In order to meet this standard, it will be necessary to reaerate the effluent after secondary treatment and before discharge.

Storage and Effluent Disposal

The major economically feasible effluent disposal alternatives available to the City of Woodburn are discharge to the Pudding River (currently in practice), storage, or beneficial reuse irrigation. The effluent storage and reuse alternatives are evaluated for the summer months when discharge to the Pudding River is not allowed because of the TMDLs.

Discharge to the Pudding River (No-Action) Alternative

This is the current discharge method used at the existing plant. Plant improvements to provide additional treatment to meet TMDLs and new permit requirements will be needed to continue this method. Because there are new stringent ammonia limits as part of the TMDLs, discharge to the Pudding River is prohibitive during July and August. Therefore, other alternatives also need to be considered in addition to Pudding River discharge. An effluent chlorine residual limit is in the new permit for the Woodburn WWTP, with interim limits and schedules for implementation in the SFO. A dilution waiver and mass load increase will be required to continue discharging to the Pudding River.

Storage/Seasonal Discharge Alternative

Storage is an alternative considered as cost-effective for eliminating river discharge during July and August. An 88-acre lagoon would be required. The lagoon would be emptied during the winter months. Mass load increases would be required in the winter.

Irrigation Reuse Alternative

Wastewater reclamation through irrigation eliminates river discharge during July and August when ammonia limits are too stringent to meet with current treatment technology. It also provides other benefits to the community and environment during the dry season. The proposed irrigation beneficial reuse is to apply reclaimed water to 400 acres of poplar trees. Recently, an innovative technology of land application to poplar trees, called the Ecolotree Buffer, has shown promise as a positive effluent beneficial reuse technique. The poplar tree root systems provide water and nutrient uptake capacity greater than conventionally irrigated crops. Thus, poplar trees are more economical to irrigate than other local crops. The poplar tree irrigation falls under agricultural reuse regulations. The poplar trees can be sold to wood products industries on a 6- to 10-year cycle.

Comparison of Alternatives

Continued discharge to the Pudding River is the preferred alternative except during July and August when discharges are prohibited because of TMDLs. During July and August, beneficial reuse irrigation of a poplar tree farm would provide more flexibility than storage because the tree root systems can provide additional treatment. Reuse at a poplar tree farm is attractive for sludge utilization and yard debris utilization as well. A pilot demonstration project is planned.

Summary

A new headworks will be included to handle future flows and improve screenings and grit removal. The primary clarifiers will be maintained with an additional primary clarifier added of equal size to the existing clarifier. Activated sludge with selector technology is the recommended secondary treatment process with ultraviolet radiation as the recommended disinfection process. New secondary clarifiers are required with the new secondary treatment process. New deep-bed filters are planned to meet summer effluent requirements. Reaeration will be required. The primary method of effluent disposal will be discharge to the Pudding River with provisions for 2 months of effluent reuse on poplar trees. These recommendations are further developed in Section 7 and during predesign.

Appendix 5.1
Big Picture Alternatives

Planning for Woodburn's Wastewater Treatment Facilities

Introduction

The City of Woodburn is preparing the facilities plan for its wastewater treatment system. A facilities plan defines the steps that the City should take to meet its future needs for wastewater treatment. It is the result of a planning process that looks at many possibilities, considers the advantages and disadvantages of each, and identifies the system that can most benefit the community. Once the plan is adopted by the City, it will guide operations and improvements to the City's wastewater treatment system through the year 2020.

Improvements to the City's system are needed to meet new water quality regulations established by the U.S. Environmental Protection Agency (EPA) and the Oregon Department of Environmental Quality (DEQ) and to protect against citizen lawsuits. These regulations designate wastewater treatment and discharge standards that must be met to protect the Pudding River and other bodies of water that receive treated wastewater. The City's current system will complete its 20-year design life in the year 2000 and it will not be able to meet these stringent new standards or the projected growth requirements of the City.

In addition to upgrading the existing system to meet regulatory requirements, the facilities plan will provide for increasing the system's capacity in order to accommodate planned growth in the City. Preparing now for both of these purposes is an efficient and cost-effective planning approach. The plan will enable the City to look ahead to long-term needs through the year 2020.

The City of Woodburn is committed to providing its citizens with a system that is environmentally responsible and cost-effective. Public review and comment is an essential part of the planning process. A number of possible systems are being considered for meeting the City's treatment needs. Optional new technology and mitigation measures can also provide additional advantages, such as reduced odor and noise impacts in the plant vicinity. All of these alternatives have cost and benefits that must be carefully weighed. We welcome your participation in making these decisions.

The Study Process and Schedule

The City of Woodburn has hired CH2M HILL, an engineering and environmental consulting firm, to help prepare the facilities plan. The planning process has the following basic steps:

1. Define future wastewater treatment needs, based on population and wastewater projections and on regulatory requirements.

2. Identify alternative treatment systems that can meet these needs.

3. Evaluate the alternatives on the basis of cost, environmental impacts, ability to construct and operate, and other advantages/disadvantages.

4. Select the best treatment system for more detailed analysis, and develop a facilities plan.

5. Conduct a financial analysis to determine who pays for the improvements and how.

The planning process is currently in the third step: evaluation of the alternatives. All public comments will be considered in selecting the alternatives that will be carried forward for further analysis. The draft facilities plan is anticipated to be available for public review by November 1994.

Recommendations

The Wastewater Advisory Committee evaluated the treatment plant siting alternatives and the tertiary treatment and effluent disposal options (see Figures 1 through 3). The consensus of the committee was to recommend the following big-picture alternative with two disposal options for further development and analysis:

Issue	Alternative
Treatment Plant Site	Alternative I—Upgrade and Expand Existing Plant
Tertiary Treatment	Mechanical Filtration
Effluent Disposal during the Summer	Option A—Discharge to the Pudding River, with 88-acre storage lagoon for July and August
	Option B—Irrigation at Agronomic Application Rates for May through October, with 58-acre storage lagoon and 300 acres irrigable land
Effluent Disposal during the Winter	Discharge to Pudding River

Treatment Plant Site

Upgrading and expanding the existing plant (Alternative I) was the preferred treatment plant siting alternative because of costs and practicality. The other siting alternatives were shown to have many disadvantages. For instance, adding a second, parallel treatment plant to the same site (Alternative II) would require additional pumping for separate treatment of industrial flows. Retaining the existing plant and building a separate plant at a new site (Alternative III) would require new pumping facilities for sludge and industrial flows, a new force main, and another effluent pipeline. Furthermore, operations and maintenance of two sites would be less efficient than for one. Abandonment of the existing plant and building a new plant at a new site (Alternative IV) would mean losing the value of the existing plant and would entail dismantling the existing facilities and restoring the site to farmland at a significant cost. Moreover, the siting of a new plant would likely face public opposition.

Tertiary Treatment

Mechanical filtration was found to be a better tertiary treatment option in this case than wetland treatment. A wetland treatment system would be approximately 50 percent more expensive because of land costs and the cost of a lining for the wetland (which is required by DEQ) without significant added benefit. Uncertainties regarding long-term permitting for control of the wetlands as a treatment facility and concerns about possible effects on the groundwater (City of Woodburn's main drinking water source) also make this a less attractive option.

Mechanical filtration is a more reliable treatment process and the capacity of a filtration facility can be increased more easily. It is estimated to be the least expensive alternative. The operations staff preferred mechanical filtration.

Effluent Disposal during the Summer

Pudding River and Storage. With mechanical filtration provided, the effluent can be discharged to the Pudding River during May, June, September, and October of the summer period. During July and August of each year, when the effluent ammonia requirements preclude discharge to the Pudding River, the effluent would be stored in an 88-acre lagoon.

Irrigation at Agronomic Application Rates. Agricultural irrigation was selected as another viable alternative for disposal of treatment plant effluent during the summer period. This alternative provides nutrient rich water for irrigation in the area at a time when water is becoming a scarce and valuable resource. A 58-acre storage lagoon would be required to store effluent, assuming roughly 300 acres of land are available for irrigation during July and August. Filtration would be required for disposal of excess wastewater to the Pudding River during the other summer months.

Other Alternatives. The other summer disposal alternatives considered—discharge to the Willamette River and a 5-month-capacity storage lagoon—were seen to be too costly.

Effluent Disposal during the Winter

Because the DEQ effluent requirements for the winter period are less stringent, it will be possible to discharge to the Pudding River with minor modifications to the current treatment processes, upgraded and expanded as necessary to replace aging components and meet new capacity needs.

Costs

Cost opinions were developed based on Environmental Protection Agency (EPA) and industry guidelines. The costs were further adjusted to reflect recent local projects such as wastewater facilities for the City of McMinnville, Oregon. They costs are comparative and not to be used for budgetary purposes.

The present worth cost in 1997 dollars for Option A—Summer Effluent Disposal to Pudding River and Storage was estimated to be $53,300,000. For Option B—Summer Effluent Disposal to Irrigation it was estimated to be $55,850,000. These present worth costs are within 15 percent of each other, and, therefore, are not considered by EPA to be significantly different. The non-cost analyses need to be considered in the selection process. The range of costs for all the alternatives was from $55,300,000 to $92,335,000.

Public Review and Comment

The City of Woodburn would like to hear public comments on the alternatives now under consideration to help decide which should be carried forward for further study. An open house will be held for this purpose on June 6, 1994. A Public Hearing will be held on November 15, 1994, to hear public comment on the draft facilities plan. The public will be notified, and additional information about the project will be available before this meeting.

If you would like more information or have any comments about the facilities plan, please call Frank Sinclair, Woodburn Wastewater Treatment Plant Superintendent, at 982-5281.

Recommended by Wastewater Advisory Committee for further evaluation

ALTERNATIVE I	Upgrade and expand existing plant at existing plant site
ALTERNATIVE II	Existing site co-location with dual treatment process
ALTERNATIVE III	Existing site and new site; new plant located at new site
ALTERNATIVE IV	New plant at new site; abandon existing plant and existing site

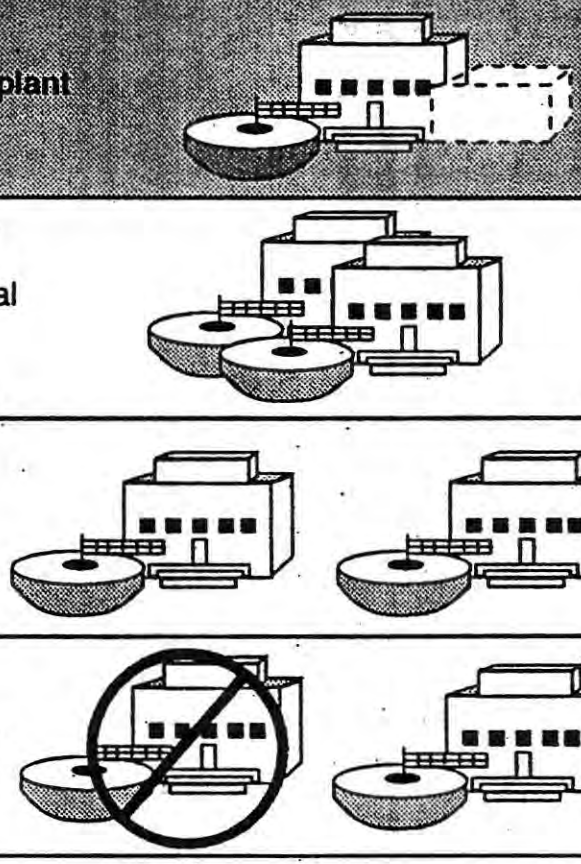

Figure 1
Siting Alternatives
City of Woodburn WWTP
Woodburn, Oregon

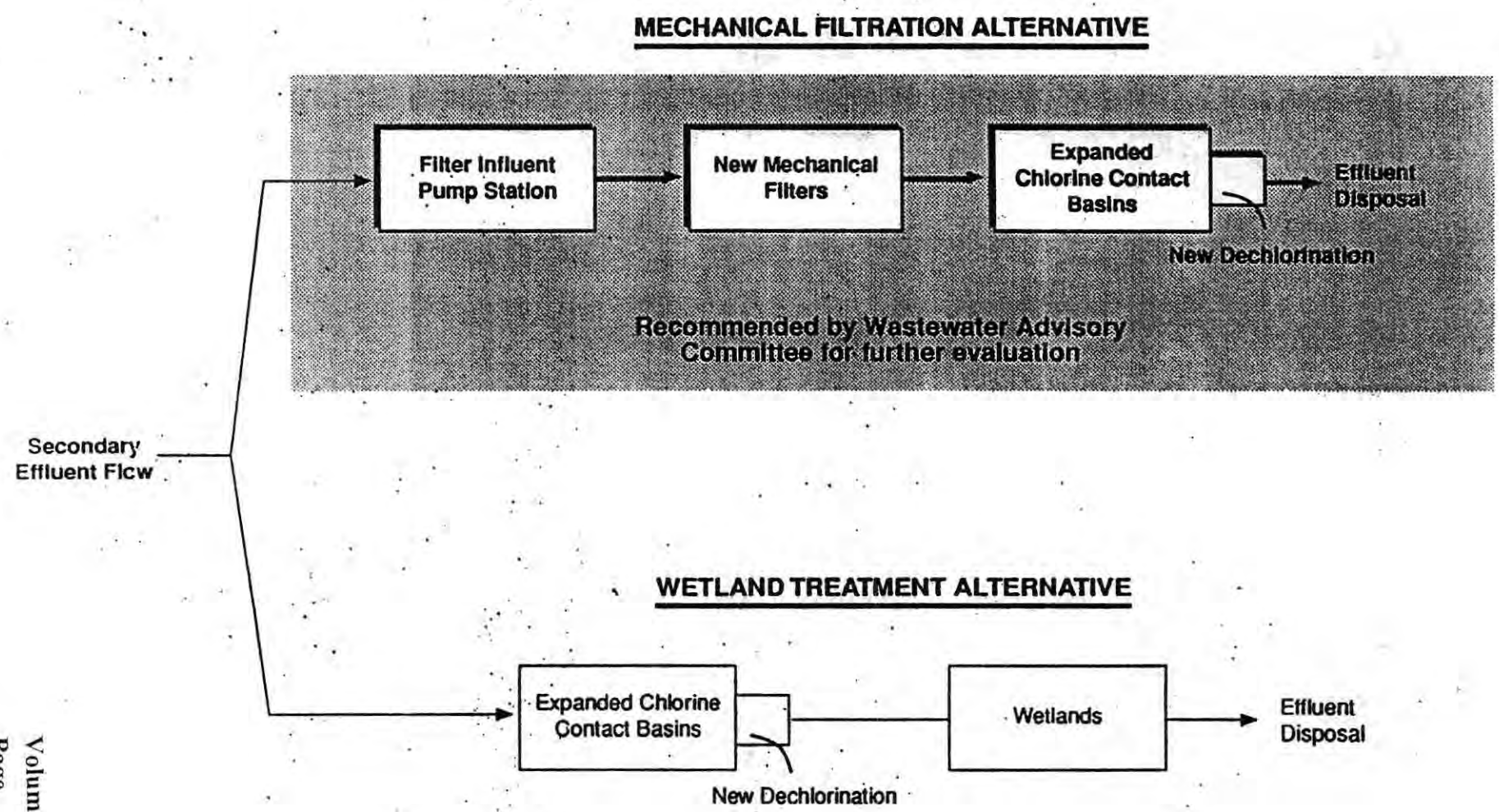

Figure 2
Tertiary Treatment Alternatives
City of Woodburn WWTP
Woodburn, Oregon

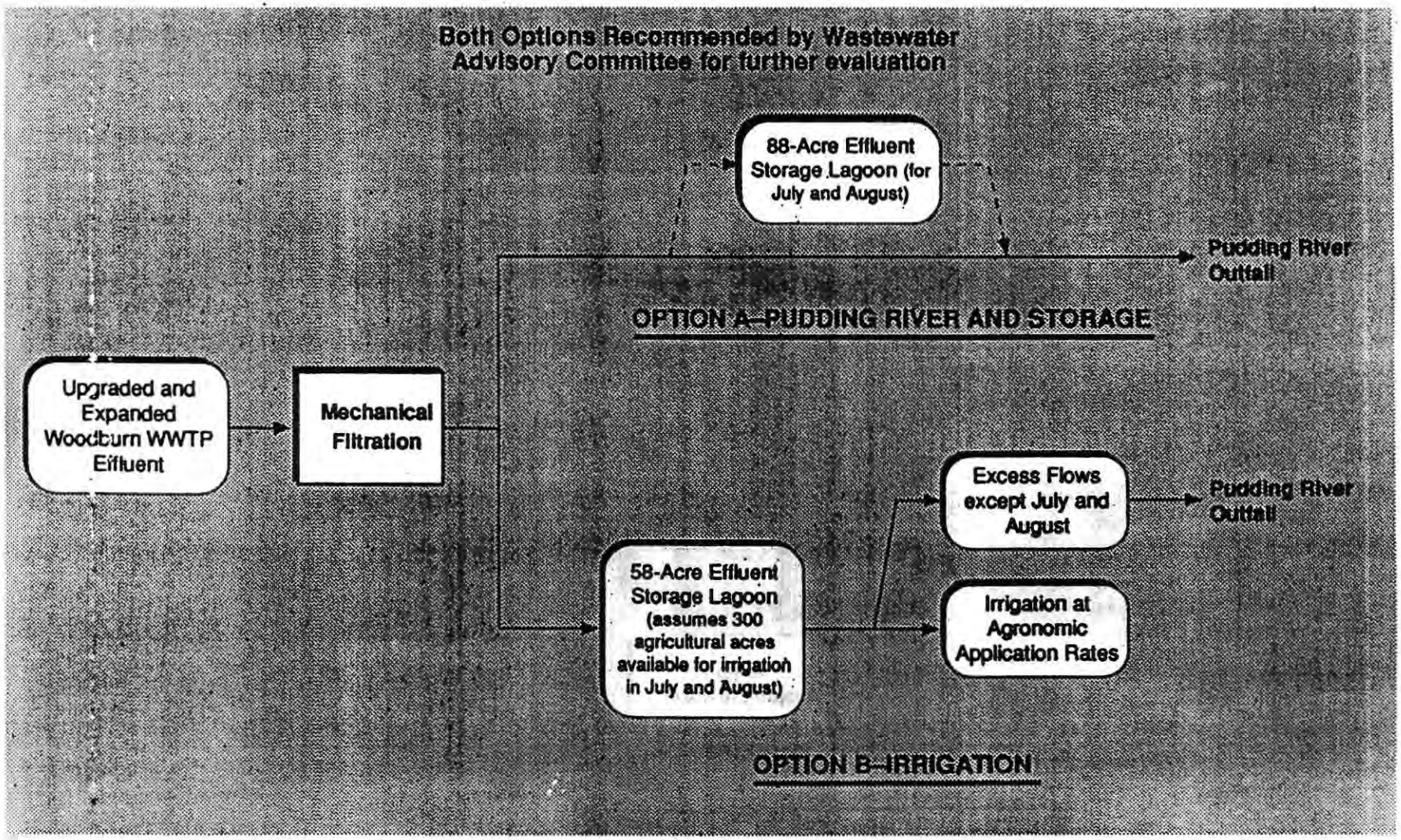

Figure 3
Options for Effluent Disposal During the Summer (May-October)
City of Woodburn WWTP
Woodburn, Oregon

TABLE F2-1. Big Picture Alternatives

Alternative I	Upgrade and Expand Existing Plant at Existing Plant Site
Alternative II	Existing Site Co-location with Dual Treatment Process
Alternative III	Existing and New Site; New Plant Located at New Site
Alternative IV	New Plant at New Site; Abandon Existing Plant and Existing Site

Table F2-2
Advantages and Disadvantages of Tertiary Treatment Options
City of Woodburn Facilities Plan

Tertiary Treatment Options	Advantages	Disadvantages
Mechanical Filtration	• Reliably meets stringent effluent requirements (BOD/TSS less than 10 mg/L) • Less land requirements than other tertiary/disposal treatment systems • Least expensive of tertiary treatment options • Easily expandable	• May require chemical addition for better particle flocculation • High operation and maintenance costs • Requires a clearwell and backwash pumps • Effectiveness depends heavily on media selected
Wetlands Treatment System	• Longer lifespan than a standard effluent filtration system • The system can readily reach discharge standards for land application or winter discharge • Great recreational and educational opportunities for the community • Enhances local water quality • Wildlife benefits and use • Potential creation of parks with passive recreational opportunities and considerable community support	• More expensive than a standard effluent filtration system because of land costs • Permitting issues need to be clarified to ensure long-term control of the wetlands as a treatment component • Heavily dependent on site specifics such as surface and groundwater hydrology, soils, adjacent land use, natural resources, and cultural resources • DEQ requires lining • Vectors (mosquitoes) • Concerns about management of wetlands above groundwater

Table F2-3
Advantages and Disadvantages of Disposal and Reuse Options
City of Woodburn Facilities Plan

Disposal and Reuse Options	Advantages	Disadvantages
Effluent Irrigation at Agronomic Application Rates	• Provides an additional source of irrigation water rich in nutrients • Reduces nutrient additions to the Pudding River, improving the water quality • Lower than average operation costs • Nonfiltered wastewater can be used. However, filtered effluent can be applied to more crops with less restrictions. • Closer to getting flow out of river	• Public concern about consumption of crops irrigated with level IV water • In the future DEQ may require filtration; requires filtration for discharge of excess flows to the Pudding River • Assumes up to 300 acres of land available to reuse the effluent • Requires "demand" for water • Requires a storage pond for effluent not used during July and August • Pipelines and reservoir required • Local food processors are reluctant to buy crops that have had wastewater applied • If no filtration provided, storage reservoir size would be excessive
Willamette River Discharge	• No effluent requirements for ammonia • Larger dilution available	• 11-mile pipeline • 42-inch diameter to meet 50 year design life • Outfall costs • Willamette River may eventually become water quality limited
Pudding River Discharge	• Shorter effluent pipeline (approximately 3/4 mile long)	• Storage requirements • Stringent effluent ammonia limits

Table F2-4
Advantages and Disadvantages of Alternatives
City of Woodburn Facilities Plan

New Treatment Plant Location Alternatives	Advantages	Disadvantages
Alternative I – Upgrade and Expand Existing Plant	• Retains the value of the existing structures: digesters, drying beds, roads, clarifiers, emergency lagoon • Does not require a sludge pump station and force main to the existing plant • Close to Pudding River for effluent discharge • Can pipe industrial wastewater with domestic • Cost savings of overall plant upgrade and expansion versus maintenance and gradual replacement of 20 year old plant components • Public acceptance of existing treatment site	• Greater disturbance of downtown area during collection system expansion
Alternative II – Existing Site Co-Location with Dual Treatment Process	• Retains the value of the existing structures • Does not require a sludge pump station and force main to the existing plant • Close to existing Pudding River outfall • Public acceptance of existing site	• Requires pumping from industries for separate treatment of industrial flows at new process • Greater disturbance of downtown area during collection system expansion
Alternative III – Existing and New Site; New Plant Located at New Site	• Retains value of the existing structures • Close to existing Pudding River outfall for existing plant • Closer to Willamette River • Closer to other communities who might share services • Could be fed by gravity • Closer to the new customers of the Pudding River Service Area	• Requires pumping from industries for separate treatment of majority of industrial flows at the site • Requires pumping flows from existing plant (new force main) to new location • Requires a sludge pump station and force main to the existing plant • Requires a separate effluent pipeline to dispose with existing plant effluent • Requires operations and maintenance staff to work at two plant sites • Public opposition to new site (not in my backyard)
Alternative IV – New Plant and New Site; Abandon Existing Plant and Existing Site	• Closer to Willamette River • Closer to other communities who might share services • Can pipe industrial wastewater with domestic	• Requires pumping all flows (new force main) to new location • Farther away from Pudding River outfall • Value of existing plant lost; also must return site to pre-existing conditions • Public opposition to new site (not in my backyard)

Table F2-5. Summary of Preliminary Cost Opinions in 1994 Dollars

Alternatives	Capital Costs	Present Worth O&M Costs	Total Present Worth Costs
I - Upgrade and Expand Existing Plant			
Mechanical Filtration,Pudding River Outfall	$40,865,200	$5,925,245	$46,790,445
Wetlands, Pudding River Outfall	$62,943,900	$5,740,945	$68,684,845
Effluent Irrigation	$41,352,700	$7,639,235	$48,991,935
Willamette River Discharge	$56,341,600	$8,505,445	$64,847,045
II - Existing Site, Co-locate (Dual Treatment)			
Mechanical Filtration,Pudding River Outfall	$41,791,477	$6,092,037	$47,883,513
Wetlands, Pudding River Outfall	$63,179,791	$5,907,737	$69,087,528
Effluent Irrigation	$45,012,043	$7,806,027	$52,818,070
Willamette River Discharge	$53,319,913	$8,672,237	$61,992,150
III.1 - New Plant Located at New Site, Maintain Existing			
Mechanical Filtration,Pudding River Outfall	$45,600,119	$7,820,125	$53,420,245
Wetlands, Pudding River Outfall	$63,886,689	$7,346,198	$71,232,887
Effluent Irrigation	$46,005,259	$9,741,177	$55,746,436
Willamette River Discharge	$51,889,759	$10,138,343	$62,028,102
III.2 - New Plant Located at New Site, Upgrade Existing			
Mechanical Filtration,Pudding River Outfall	$47,820,869	$8,138,043	$55,958,912
Wetlands, Pudding River Outfall	$67,028,139	$7,664,116	$74,692,255
Effluent Irrigation	$50,070,109	$10,059,094	$60,129,203
Willamette River Discharge	$55,954,609	$9,488,686	$65,443,295
IV - New Plant At New Site, Abandon Existing Site			
Mechanical Filtration,Pudding River Outfall	$57,442,761	$5,925,245	$63,368,006
Wetlands, Pudding River Outfall	$75,255,461	$5,740,945	$80,996,406
Effluent Irrigation	$54,366,261	$7,639,235	$62,005,496
Willamette River Discharge	$63,242,800	$8,505,445	$71,748,245

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Table F2-6. Summary of Preliminary Cost Opinions in 1997 Dollars

Alternatives	Capital Costs	Present Worth O&M Costs	Total Present Worth Costs
I - Upgrade and Expand Existing Plant			
I - Mechanical Filtration,Pudding River Outfall	$46,586,328	$6,754,779	$53,341,107
I - Wetlands, Pudding River Outfall	$71,756,046	$6,544,677	$78,300,723
I - Effluent Irrigation	$47,142,078	$8,708,728	$55,850,806
I - Willamette River Discharge	$64,229,424	$9,696,207	$73,925,631
II - Existing Site, Co-locate (Dual Treatment)			
II - Mechanical Filtration,Pudding River Outfall	$47,642,284	$6,944,922	$54,587,205
II - Wetlands, Pudding River Outfall	$72,024,962	$6,734,820	$78,759,782
II - Effluent Irrigation	$51,313,730	$8,898,870	$60,212,600
II - Willamette River Discharge	$60,784,701	$9,886,350	$70,671,051
III.1 - New Plant Located at New Site, Maintain Existing			
III.1 - Mechanical Filtration,Pudding River Outfall	$51,984,136	$8,914,943	$60,899,079
III.1 - Wetlands, Pudding River Outfall	$72,830,826	$8,374,666	$81,205,491
III.1 - Effluent Irrigation	$52,445,995	$11,104,941	$63,550,937
III.1 - Willamette River Discharge	$59,154,325	$11,557,711	$70,712,036
III.2 - New Plant Located at New Site, Upgrade Existing			
III.2 - Mechanical Filtration,Pudding River Outfall	$54,515,791	$9,277,369	$63,793,160
III.2 - Wetlands, Pudding River Outfall	$76,412,079	$8,737,092	$85,149,170
III.2 - Effluent Irrigation	$57,079,924	$11,467,367	$68,547,292
III.2 - Willamette River Discharge	$63,788,254	$10,817,101	$74,605,356
IV - New Plant At New Site, Abandon Existing Site			
IV - Mechanical Filtration,Pudding River Outfall	$65,484,747	$6,754,779	$72,239,526
IV - Wetlands, Pudding River Outfall	$85,791,225	$6,544,677	$92,335,902
IV - Effluent Irrigation	$61,977,537	$8,708,728	$70,686,265
IV - Willamette River Discharge	$72,096,792	$9,696,207	$81,792,999

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Table F2-7. Ranking of Alternatives by Ascending Preliminary Cost Opinions in 1997 Dollars

Ranking By 1997 Cost Opinions	Capital Costs	Present Worth O&M Costs	Total Present Worth Costs	Percent Difference	Rank Number
I - Mechanical Filtration,Pudding River Outfall	$46,586,328	$6,754,779	$53,341,107	0%	1
II - Mechanical Filtration,Pudding River Outfall	$47,642,284	$6,944,922	$54,587,205	-2%	2
I - Effluent Irrigation	$47,142,078	$8,708,728	$55,850,806	-5%	3
II - Effluent Irrigation	$51,313,730	$8,898,870	$60,212,600	-13%	4
III.1 - Mechanical Filtration,Pudding River Outfall	$51,984,136	$8,914,943	$60,899,079	-14%	5
III.1 - Effluent Irrigation	$52,445,995	$11,104,941	$63,550,937	-19%	6
III.2 - Mechanical Filtration,Pudding River Outfall	$54,515,791	$9,277,369	$63,793,160	-20%	7
III.2 - Effluent Irrigation	$57,079,924	$11,467,367	$68,547,292	-29%	8
II - Willamette River Discharge	$60,784,701	$9,886,350	$70,671,051	-32%	9
IV - Effluent Irrigation	$61,977,537	$8,708,728	$70,686,265	-33%	10
III.1 - Willamette River Discharge	$59,154,325	$11,557,711	$70,712,036	-33%	11
IV - Mechanical Filtration,Pudding River Outfall	$65,484,747	$6,754,779	$72,239,526	-35%	12
I - Willamette River Discharge	$64,229,424	$9,696,207	$73,925,631	-39%	13
III.2 - Willamette River Discharge	$63,788,254	$10,817,101	$74,605,356	-40%	14
I - Wetlands, Pudding River Outfall	$71,756,046	$6,544,677	$78,300,723	-47%	15
II - Wetlands, Pudding River Outfall	$72,024,962	$6,734,820	$78,759,782	-48%	16
III.1 - Wetlands, Pudding River Outfall	$72,830,826	$8,374,666	$81,205,491	-52%	17
IV - Willamette River Discharge	$72,096,792	$9,696,207	$81,792,999	-53%	18
III.2 - Wetlands, Pudding River Outfall	$76,412,079	$8,737,092	$85,149,170	-60%	19
IV - Wetlands, Pudding River Outfall	$85,791,225	$6,544,677	$92,335,902	-73%	20

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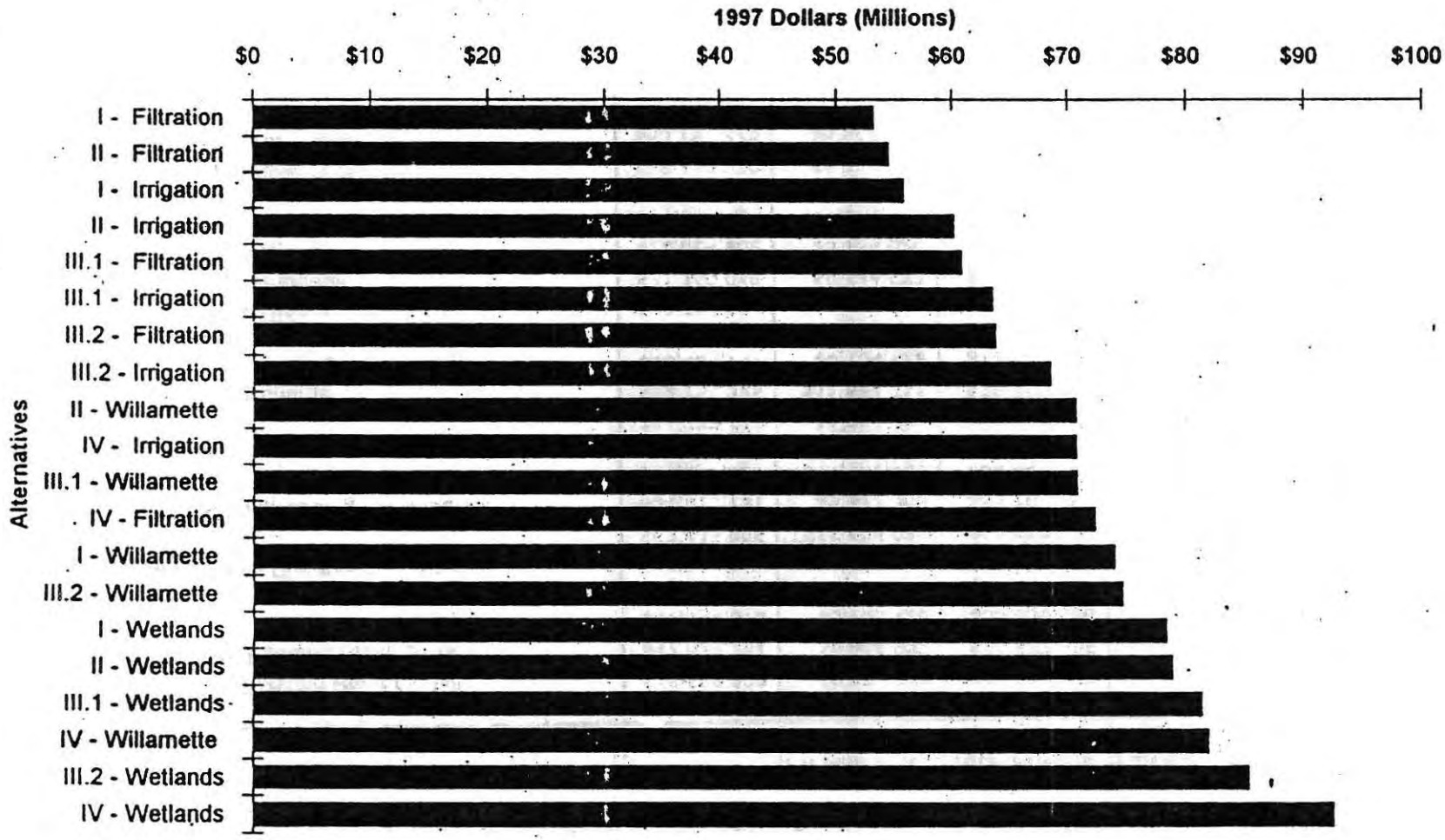
Figure F2-7. 1997 Present Worth Costs of Alternatives in Ascending Order of Cost Opinion



SUMMARY OF ALTERNATIVE I:
UPGRADE AND EXPAND EXISTING PLANT AT EXISTING SITE

COMPONENT INSTALLED CONSTRUCTION COSTS
MECHANICAL FILTRATION - OUTFALL TO PUDDING RIVER

UNIT PROCESS	ALT. I
HEADWORKS	$1,900,000
PRIMARY CLARIFIERS	$1,400,000
AERATION BASINS	$2,500,000
SECONDARY CLARIFIERS	$2,200,000
EFFLUENT FILTERS	$1,900,000
CHLORINATION/DECHLOR	$1,100,000
SUMMER STORAGE (2 mo)	$4,500,000
SOLIDS HANDLING	$3,200,000
DOWNTOWN COLLECTION IMP	$800,000
PUDDING RIVER OUTFALL	$850,000
SUBTOTAL 1:	$20,350,000
NON-COMPONENT COSTS	
PIPING @ 12%	$2,442,000
ELECTRICAL , I&C @ 15%	$3,052,500
SITE PREPARATION @ 5%	$1,017,500
PLANT REHAB @ 20%	$3,170,000
SUBTOTAL 2:	$9,682,000
NON-CONSTRUCTION COSTS	
SUPERVISION @ 15%	$4,504,800
CONTINGENCIES @ 20%	$6,006,400
SUBTOTAL 3:	$10,511,200
LAND COST: SUBTOTAL 4:	$322,000
CAPITAL COST (INCL LAND):	$40,865,200
P.W. O&M:	$5,925,245
TOTAL PROJECT (1994 $):	$46,790,445

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SUMMARY OF ALTERNATIVE I:
UPGRADE AND EXPAND EXISTING PLANT AT EXISTING SITE

COMPONENT INSTALLED CONSTRUCTION COSTS
EFFLUENT IRRIGATION WITH PUDDING RIVER OUTFALL

UNIT PROCESS	ALT. I
HEADWORKS	$1,900,000
PRIMARY CLARIFIERS	$1,400,000
AERATION BASINS	$2,500,000
SECONDARY CLARIFIERS	$2,200,000
CHLORINATION/DECHLOR	$1,100,000
EFFLUENT FILTERS	$1,900,000
EFFLUENT IRRIGATION	$4,450,000
SOLIDS HANDLING	$3,200,000
DOWNTOWN COLLECTION IMP	$800,000
PUDDING RIVER OUTFALL	$850,000
SUBTOTAL 1:	$20,300,000
NON-COMPONENT COSTS	
PIPING @ 12%	$1,902,000
ELECTRICAL AND I&C @ 15%	$2,377,500
SITE PREPARATION @ 5%	$792,500
PLANT REHAB @ 20%	$3,170,000
SUBTOTAL 2:	$8,242,000
NON-CONSTRUCTION COSTS	
SUPERVISION @ 15%	$4,281,300
CONTINGENCIES @ 20%	$5,708,400
SUBTOTAL 3:	$9,989,700
LAND COST: SUBTOTAL 4:	$2,821,000
CAPITAL COST (INCL LAND):	$41,352,700
P.W. O&M:	$7,639,235
TOTAL PROJECT (1994 $):	$48,991,935

Section 6

Solids Management Alternatives

Introduction

The solids management alternatives available to the Woodburn Wastewater Treatment Plant (WWTP) are discussed in this section. These alternatives encompass in-plant solids processing and biosolids disposal or reuse. A broad range of potential methods were screened according to their applicability to Woodburn. Those that passed the screening were developed into alternatives and evaluated further. The evaluation included:

- Compatibility with the viable liquid treatment processes
- Potential to reuse existing plant facilities
- Compatibility with the surrounding predominant agricultural land uses

Screening of Methods

Thickening

Thickening of primary treatment sludge and secondary treatment waste sludge will be part of any overall solids processing scheme. Previous experience has shown that thickening is a cost-effective component for most solids management programs because it minimizes the sizes of the other components. The plant currently allows primary sludge to thicken in the primary clarifiers; this thickened sludge is pumped directly to anaerobic digestion. Secondary waste sludge can either be co-thickened in the primary clarifier or wasted directly to the lagoon. Currently the waste sludge is normally wasted to the lagoon and allowed to accumulate. Because allowing primary sludge to thicken in the primary clarifiers has been successful at Woodburn, and this practice eliminates a separate thickening process step, this analysis assumes that the practice will continue. Because of the emergency-only aspect of diverting waste sludge to the lagoon, and the difficulty of co-settling by gravity a waste activated sludge (WAS) such as will be produced by the new secondary treatment facilities, this analysis also assumes that separate WAS thickening facilities will be provided. Candidate WAS thickening methods include gravity belt, centrifuge, and rotary screen thickeners. These are all mechanical methods and require similar facilities construction. Preliminary design of facilities should consider the options, keeping in mind associated facilities such as stabilization and dewatering, if used. For cost estimating purposes, the evaluation of overall solids management options assumed gravity belt WAS thickening.

Stabilization

Aerobic and anaerobic digestion and lime stabilization are the three candidate stabilization methods for achieving pathogen and vector attraction reduction criteria to allow biosolids to be beneficially reused through methods such as agricultural land application. Anaerobic digestion is

currently used at Woodburn and is generally cost effective over aerobic digestion for this size plant. Because of the existing investment in anaerobic digestion that can be reused in the future, and because of odor concerns associated with lime stabilization, continuing anaerobic digestion through construction of additional tankage is recommended.

Dewatering

Dewatering consists of converting biosolids from a liquid to a solid material consistency. Common methods of dewatering include natural methods such as the plant's existing drying beds and mechanical methods such as belt filter presses, centrifuges, and chamber filter presses. Certain of the overall solids management alternatives require dewatering to support ultimate reuse or disposal methods. In this evaluation it was assumed that, where dewatering was required, a mechanical method such as belt filter presses or centrifuges would be used. Additional drying beds are considered impractical because of the large site area required and because of odor problems associated with the existing drying beds. Continuous processes such as belt presses and centrifuges are also preferred over a batch process such as a chamber press and are generally cost effective for a plant the size of Woodburn. For cost estimating purposes, the evaluation of overall solids management alternatives assumes belt filter presses.

Composting

Several methods of composting are commonly used to stabilize sludge at municipal wastewater treatment facilities. These methods include aerated static pile, windrow, and in-vessel composting. Composting involves dewatering sludge and then adding amendments (for example, sawdust, wood chips, or ground yard debris) to the sludge to produce a material that allows air to penetrate into the pile. Sludge stabilization occurs within windrows or a compost pile by aerobic bacteria generating heat as they stabilize the sludge.

Sludge composting adds to the cost of sludge processing because of the material handling of the amendment and compost, but composting produces a Class A product that has a marketable value. The City of Woodburn currently participates in a yard debris program where citizens are allowed to bring yard debris, primarily leaves, to the treatment plant where the debris is ground and stored on a portion of the plant's drying beds. A biosolids/yard debris co-composting operation is therefore a realistic overall solids management alternative for the City. The primary technology candidates for such a program include either windrow or static pile composting of mechanically dewatered biosolids and the ground yard debris using the existing drying beds as a composting surface.

Incineration

Incineration involves high temperature combustion of sludge. In addition to air quality and ash disposal permitting difficulties, and the fact incineration is best suited for continuous (24 hour per day) operation, incineration is generally not economically feasible for a facility the size of the Woodburn WWTP. Also, DEQ discourages incineration as—evidenced by their stated goal of

100 percent beneficial reuse of biosolids in Oregon. For these reasons, incineration is not considered further.

Heat Drying

This option would involve dewatering and drying biosolids with a high-temperature dryer to remove moisture. This option would involve air permitting, would significantly increase the complexity of current operations, and (like incineration) is best suited to continuous operation. Also, heat drying is not generally economically feasible for a facility the size of the Woodburn WWTP. For these reasons, heat drying is not considered further.

Evaluation of Alternatives

Development of Alternatives

The City and CH2M HILL met on October 17, 1994, to discuss candidate solids processing, reuse, and disposal methods and to combine the various methods into realistic overall solids management alternatives, which included:

- Liquid Land Application of Digested Biosolids
- Land Application of Dewatered Digested Biosolids
- Composting

The most realistic configuration for each alternative was developed based on the compatibility of the proposed solids processes with proposed liquids processes, available land area at the plant site, and cooperative reuse/disposal potential in conjunction with the proposed poplar tree water reclamation irrigation program. These alternatives and a No-Action Alternative were evaluated and compared, as described below.

No-Action Alternative

The existing solids handling method includes anaerobic digestion, drying bed storage, and dewatering of primary treatment sludge followed by land application on MacLaren fields. The design intent was for waste sludge from the secondary treatment system to be co-settled with primary sludge in the primary clarifiers and digested, dewatered, and land applied as is done with primary sludge. However, higher than anticipated wastewater loadings have overloaded various plant processes, including the ability to handle secondary treatment waste sludge in the intended manner. As a result, secondary waste sludge has historically been sent to the lagoon and allowed to accumulate.

Continuation of the existing system in the short- and long-term is not possible for the following reasons. First, the lagoon is needed for emergency wastewater storage and treatment currently and in the future wastewater plan, thus precluding its continued use for waste sludge handling. Second, although the existing digesters are adequate for existing primary sludge loads, they will

require expansion to handle future primary and waste sludge loads in order to be in compliance with federal and state stabilization requirements. Third, additional land application area will also be required for biosolids anticipated from future wasteloads and from current waste sludge loadings if they are included.

Because of these capacity limitations, a No-Action Alternative is not possible. Alternatives 1, 2, and 3 below depict various scenarios that provide the necessary capacity expansion of the existing digestion and land application methods.

Alternative 1: Land Application of Liquid Digested Biosolids

This alternative applies land application of liquid digested biosolids on the reclaimed water irrigation site for poplar trees. New facilities at the treatment plant would include WAS thickening, a new anaerobic digestion tank, and a liquid sludge storage lagoon. WAS thickening could be constructed in part of the building associated with the addition of one new 612,000-gallon primary anaerobic digestion tank. The new digester would work in conjunction with the existing primary digester; the existing secondary digester would remain in service as a secondary tank. Liquid digested biosolids would be pumped directly to land application or to a lagoon for storage during wet weather when land application is not possible. Land application of biosolids could either be performed by truck, or, if the logistics of the land application site are favorable, by sprinkler system.

A significant design consideration will be the actual land application site configuration and how it will determine the optimum method of biosolids application. Preliminary calculations indicate that including biosolids may push the nutrient loading of the land application site near maximum limits. Therefore, design should consider both wastewater and biosolids considerations carefully. Inclusion of biosolids would, however, be beneficial to the site because it would lessen reliance on commercial fertilizer, particularly for younger trees. Phasing opportunities include the WAS thickening process and storage lagoon.

Alternative 2: Land Application of Dewatered Biosolids

This alternative includes WAS thickening and expanded anaerobic digestion facilities identical to Alternative 1. Digested biosolids would then be mechanically dewatered using equipment such as belt filter presses or centrifuges. The resulting dewatered cake would then either be loaded directly into a land application vehicle for spreading or deposited on existing drying beds for storage during wet weather. The cake would be applied to the poplar tree water reclamation site by spreader truck or manure spreader.

As with Alternative 1, the siting details of the land application area will determine the details of the land application program. Although nutrient loading will not be as critical with this alternative because of the lower nitrogen content of a dewatered cake, application area design must still carefully consider both wastewater and biosolids.

Alternative 3: Composting

This alternative includes WAS thickening, expanded anaerobic digestion, and mechanical dewatering facilities identical to Alternative 2. Dewatered biosolids would be mixed with ground yard debris, which then would be composted by windrow or aerated static pile on the existing asphalt drying beds. The composted product could be spread on the wastewater land application site or marketed in the community. The material would have value as a mulch, soil conditioner or a component in topsoil mixtures.

Other Process Issues

WAS thickening and anaerobic digestion facilities are identical for Alternatives 1, 2, and 3. As discussed previously, several candidate WAS thickening technologies exist. This evaluation and its associated cost estimate assume gravity belt thickening will be used because of its general cost effectiveness and simplicity and because it does not have to be operated continually. Further project implementation phases could evaluate alternative technologies if the City desires. Similarly, alternative anaerobic digester configurations could be evaluated for providing the additional 612,000 gallons of tankage required at design conditions

Sludge storage is a necessary component of any land application alternative during wet weather when field access is restricted. The alternatives, therefore, employ liquid biosolids storage in lagoons or dewatered cake storage (piled mechanically) in the existing drying beds. Exact storage configuration is another design related issue that should be considered early in the predesign phase of the proposed facilities.

Alternative 3 includes co-composting of biosolids and yard debris, and addresses the City's yard debris disposal program which the City plans on continuing. Under Alternatives 1 and 2, yard debris could be applied to the poplar tree water reclamation site directly as a method of disposal. The cost of yard debris application, however, was not included in Alternatives 1 and 2. Alternative 3 does not include the costs of land spreading the compost product because of the potential value of the product for other uses.

Cost Evaluation

Table 6-1 presents current day capital, mid-point of study period O&M, and present worth values for the three alternatives. Alternative 1, liquid land application, is the most cost effective of the three alternatives. Comparison of Alternatives 1 and 2 illustrates that the additional capital cost of dewatering is not offset by a significant reduction in land application costs. Alternative 3, composting, has the highest capital and O&M cost. Costs include 23 percent for legal, administration, and engineering and a 15 percent contingency.

Non-Cost Evaluation

Solids management Alternatives 1, 2, and 3 were evaluated and compared with regard to the following non-cost issues: ease of implementation, operations and maintenance (O&M),

Table 6-1
Woodburn Solids Management Alternatives Cost Summary

	WAS Thickening	Anaerobic Digestion	Dewater	Composting	Storage	Land Application	Totals	Present-Worth Cost
Alternative 1–Land Application of Liquid Digested Biosolids								
Capital	1,924,000	3,016,000			722,000		5,662,000	5,662,000
O&M	25,500	17,000			20,200	181,000	243,700	2,244,000
								7,906,000
Alternative 2–Land Application of Dewatered Digested Biosolids								
Capital	1,924,000	3,016,000	2,855,000			125,000	7,920,000	7,920,000
O&M	25,500	17,000	70,600		2,500	61,100	176,700	1,627,000
								9,547,000
Alternative 3–Composting								
Capital	1,924,000	3,016,000	2,855,000	345,000			8,140,000	8,140,000
O&M	25,500	17,000	70,600	181,600			294,700	2,714,000
								10,854,000

performance reliability, flexibility, energy use and resource recovery, and environmental impacts. The results are discussed below and summarized in Table 6-2.

Ease of Implementation

All alternatives are relatively easy to implement given existing practices at the plant and are rated equally. Alternatives 1 and 2 rely on the development of the poplar tree water reclamation site, but they would be easy to implement once the site were developed. Similarly, Alternative 3 would be easy to implement if wastewater reuse were on the poplar tree site. All alternatives would be impacted negatively if a wastewater land application site were not implemented because other land application sites or markets for compost would have to be developed.

Operations and Maintenance Characteristics

Alternatives 2 and 3 are more O&M intensive because of the additional process complexity of dewatering and composting.

Performance Reliability

The component unit processes of all the alternatives are fairly simple and are, therefore, rated equally. Tradeoffs exist between more unit processes in Alternatives 2 and 3 versus the larger volume of material handled in Alternative 1.

Flexibility

Flexibility is rated higher for Alternatives 2 and 3 because the biosolids end products, cake or compost, have more potential reuse opportunities.

Energy Use And Resource Recovery

The alternatives are similar in energy use because of tradeoffs between the energy requirements for volume reduction and the energy required for transport of end product to reuse.

Environmental Impacts

All alternatives include beneficial reuse of biosolids. Odor at the plant site would be the major concern in alternatives 2 and 3 because of cake storage and/or composting activities. Odor at the land application site will be the major concern for Alternative 1.

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Table 6-2 Non-Cost Evaluation Summary			
Impact Criteria	Alternative 1 Liquid Land Application	Alternative 2 Dewatering Land Application	Alternative 3 Composting
Ease of implementation	0	0	0
O&M characteristics	+	0	0
Performance reliability	0	0	0
Flexibility	0	+	+
Energy use	0	0	0
Environmental impacts	0	0	0
Overall system option impact	**+1**	**+1**	**+1**

Alternative Selection

Alternative 1, Liquid Land Application of Biosolids, is the recommended alternative because it is the least costly alternative on a capital and total present worth basis and it is equivalent to the other alternatives from the standpoint of the Non-Cost Evaluation (see Table 6-2).

Biosolids Information

Solids quantities were projected based on the recommended liquids treatment alternative and design wastewater characteristics. Table 6-3 presents a summary of the projected solids quantities.

The Woodburn WWTP biosolids are of good quality with regard to metals, which enhances their land application potential. See Section 2 for a discussion of solids quality information. Only limited biosolids indicator organism testing for pathogens is available to demonstrate compliance with pathogen reduction requirements. The anaerobic digestion process does, however, meet the minimum processing requirement of a 15-day digestion time, and the expanded facilities must be designed to do so as well.

Table 6-3 Solids Quantity Projections					
Condition	Plant Flow (mgd)	Primary (lb/dry)	WAS (lb/dry)	Total Undigested Sludge (lb/dry)	Digested (lb/dry)
Average Dry Weather	5.037	7432	5451	12883	7896
Maximum Month Dry Weather	6.228	10282	7549	17831	10694
Maximum Month Wet Weather	8.772	6069	5803	11872	7277
Peak Day	11.747	12100	12801	24901	15400

Allowable biosolids application rates are determined based on the nutrient content of the biosolids and the nutrient uptake rate of the crop. The nutrient content of a biosolid is significantly impacted by the way in which it is processed. For instance, dewatering will significantly reduce the nitrogen content of a biosolid because a majority of the ammonia will be lost with the liquid fraction as it is removed. Also, the addition of secondary sludge into the solids processing scheme will impact the resultant nutrient content. The appropriate loading rate and corresponding site requirements will be determined based on actual nutrient content that is produced and the crop that is being grown.

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Section 7

Recommended Wastewater Management Plan

Introduction

This section summarizes the findings and conclusions of the wastewater facilities planning and presents the recommended wastewater management program for the City of Woodburn.

Included is a summary of the recommended wastewater system improvements, a recommended phasing plan, a summary of the implementation issues, and a schedule to complete the program.

Summary of Recommended Facilities

Section 5 presented the evaluation of wastewater treatment and disposal options considered appropriate to satisfy the facility planning criteria outlined in Section 4 and to meet the overall needs and objectives of the City. Section 6 presented a similar evaluation of solids processing and disposal alternatives. Each alternative was evaluated considering technical feasibility, environmental impacts, and costs in order to select the most cost-effective and environmentally sound system for the City. The recommended wastewater treatment and disposal program for the City includes the following facilities:

Liquid Treatment and Effluent Disposal

- New headworks with screening and vortex grit removal
- One additional primary clarifier
- Modified RBC basins to become future phosphorus removal selector basins, if needed
- Three new activated sludge basins with anoxic selectors for ammonia removal
- New blower building
- Three new secondary clarifiers
- Return and waste sludge pump station
- Chemical addition
- New deep bed filters
- UV disinfection and sodium hypochlorite
- Effluent reaeration
- Plant water system
- Effluent flume
- New outfall
- Poplar tree reclaimed water irrigation system for July and August
- New odor control for headworks and solids building
- New administration building

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- Modifications to the existing administration/lab building

Solids Management

- Waste activated sludge thickening
- Anaerobic digestion
- Biosolids storage and land application

Project Phasing

The facilities identified for the wastewater treatment plant will ultimately be necessary to meet the effluent discharge requirements. The facilities will be implemented in two phases as recommended by the Wastewater Advisory Committee, because of funding constraints and unknowns related to future growth. Two out of three units will be implemented in Phase 1 whenever possible.

Description of Recommended Facilities

Preliminary Treatment

Existing preliminary treatment consists of in-channel comminutors followed by a vortex grit unit. Plant staff have reported consistently low grit removal efficiencies as indicated by accumulation of large quantities of grit in the digesters. The headworks is already experiencing flows greater than design. A new headworks facility is recommended with two in-channel coarse rotary screens with screw conveyors and compactors, one mechanical screen, and a new 16-foot diameter vortex grit unit to serve both Phases 1 and 2 a waiver from the redundancy and reliability requirement for a minimum of 2 grit units is required (the smaller existing vortex grit unit may be used for septage). A building would be provided to enclose the screens and screenings and grit handling equipment in order to control odors. Odor scrubbing of offgases is provided for this building. The structure would include a drive-through facility to load grit and screenings onto trucks for offsite disposal.

Primary Treatment

The existing clarifiers have adequate capacity for Phase 1. One additional unit of similar dimensions is recommended for Phase 2. During predesign, the reliability and condition of the primary clarifier mechanisms and other primary treatment equipment should be evaluated to determine the remaining life and need for repairs.

Biological Secondary Treatment

An activated sludge system with anoxic and aerobic zones and flexible primary effluent and recycle routing is recommended for the Woodburn WWTP. The flexibility to provide an anaerobic zone in the future is also recommended. The existing RBC basins would be

converted into anaerobic reactors, intended to be part of a biological phosphorus removal process, if needed by future effluent phosphorus limits. A total of three new aeration basins (two in Phase 1 and one more in Phase 2) would be constructed, with approximately 30 percent of the new volume allocated to anaerobic/anoxic zones. Provision of additional anaerobic/anoxic volume in the new basins is necessary because the combined volume of the existing RBC basins is only about 280,000 gallons, which may not be adequate for nutrient removal purposes. Flexibility would be provided to route the primary effluent and RAS either directly to the new aeration basins or to the RBC basins converted to anaerobic zones. Similarly, both aeration and mixing capabilities would be provided in a portion of the new basins to provide limited variation in the aerated and non-aerated volume fractions. Sufficient aerobic volume would be maintained to achieve near-complete nitrification year-round, except in July and August when there would be no discharge to the river, should ammonia be an issue in the mixing zone year-round.

It is expected that the system would initially be operated as an activated sludge process with an anoxic selector designed to remove ammonia and to promote sludge settleability by controlling filamentous bulking. The anaerobic zone (existing RBC basins) would be by-passed in this mode, and mixed liquor recycle (MLR) may or may not be required (this should be determined during predesign based on reliable influent nitrogen characterization). The anaerobic zones and MLR would be included in the process for phosphorus and nitrogen removal, respectively, as required to comply with anticipated future regulations. MLR pumps may be installed at a later date. Some flexibility would be provided in the MLR discharge location to allow variation of anaerobic/anoxic volume fractions.

The diurnal peak oxygen requirements are based on an assumed loading peaking factor of 1.5. This factor is subject to change pending determination of diurnal variation in influent flow and characteristics. Sizing of process air blowers will be based on the diurnal peak oxygen requirement. Centrifugal blowers with adjustable speed drives are recommended along with a fine bubble diffuser system. Anaerobic and anoxic zones would be equipped with submersible mixers in Phases 1 and 2. The anaerobic zones will need mixers whenever they are implemented, which will probably not occur in Phases 1 or 2. Additional analysis will be conducted during predesign.

Retrofit of the existing RBC basins will involve removal of the shafts and the media and installation of new mixers to keep solids in suspension without aeration. It is desirable to leave the existing covers and aeration equipment in place to provide odor containment and process flexibility.

Secondary Clarification

The existing secondary clarifiers have a sidewater depth of only about 10 feet and are therefore not effective at higher overflow rates. These will be replaced with three new deep circular secondary clarifiers (two in Phase 1 and one more in Phase 2) having a sidewater depth around 16 feet. Each clarifier would have a diameter of 75 feet. The proposed three clarifiers do not meet the redundancy and reliability criteria of "75 percent of design flow

capacity with largest unit out of service." However, three clarifiers are more economical to construct than four.

Chemical Addition/Coagulation

Modular inline chemical injection equipment would be incorporated at appropriate locations to allow addition of various coagulation/flocculation chemicals to the secondary clarifier influent or filter influent, now or in the future. Flexibility to vary chemical dose(s) according to the filter influent quality and the reuse/disposal method would be provided. Metering pumps and blending equipment will be installed and will discharge to chemical injection nozzles. An in-line static mixer would be provided downstream of the nozzles.

Tertiary Filtration

Conventional deep bed mixed media downflow filters are selected for the facility upgrade. Filter sizing information is shown in Table 7-1. Simultaneous backwash and air-scour capability is recommended. Based on hydraulics considerations (to provide sufficient driving head), it is recommended that the filters (and downstream processes) be located in the ravine south of the storage lagoon. It is estimated that this would provide about 15 feet of head for gravity flow. This arrangement would also allow use of the existing secondary clarifiers for backwash supply storage. Backwash pumps would be located downstream of disinfection and would pump to the clarifier storage over a period of time at a rate much lower than the actual required backwash flow rate. This would reduce the size of the backwash pumps required. Stored backwash water would then be conveyed to the filters by gravity during an actual backwash event. A waste backwash surge tank would also be provided downstream of the filters. Waste backwash would be gradually pumped to the head of the plant via the plant drain system. Dosing the filters with disinfectant or oxidant can be done during the backwash cycle.

UV Disinfection

The UV disinfection design information, listed in Table 7-1, is based on previous experience and the expected effluent quality of the upgraded Woodburn WWTP, and should be verified before the system is designed. Some amount of testing will be required to provide the additional design data required. Ideally, comprehensive pilot testing is recommended with the actual filtered and/or unfiltered effluent. At a minimum, however, testing for iron and UV transmittance will be required for predesign. Two channels with three UV banks per channel are recommended based on current data. Additional analyses will be conducted during predesign.

Reuse Effluent and Plant Water Chlorination

Limited chlorination capability will have to be maintained at the plant to meet residual requirements for plant water and for dosing filters and irrigation reuse during July and August. It is recommended that sodium hypochlorite be used for this purpose. Design information for hypochlorite storage and feeding equipment is shown in Table 7-1.

Table 7-1
Recommended Wastewater Treatment Facilities
Preliminary Design Information

Item	Description			
	Dry Weather Average	Dry Weather Max. Month	Dry Weather Peak Day	Wet Weather Max Month
Flow (mgd)	5.036	6.227	11.426	8.844
Screens and Screen Channels				
Number of Channels				
Meachnical		2		
Manual		1		
Single Channel Width (feet)				
Mechanical		3.5		
Manual		7		
Channel Length (feet)		20		
Channel depth (feet)		5		
Screens		2 Mechanical, 1 Manual		
Mechanical screen type		In-channel rotary with screw conveyor/compactor		
Bar spacing		0.25-0.5 inches		
Grit Unit		One new 16-foot diameter vortex type (Pista/Jeta) 18 feet structure OD to 9.5 feet depth from TOC 6 feet OD between 9.5 feet to 17 feet deep Grit conveyor and classifier		
Primary Clarifiers				
Number of units				
Total	3	3	3	3
In service	3	3	3	3
Single clarifier dimensions (feet)				
Diameter	55	55	55	55
Sidewater depth	10	10	10	10
Total concrete perimeter (feet)	518	518	518	518
Total area (ft^2)	7127	7127	7127	7127
Area in service (ft^2)	7127	7127	7127	7127
SOR (gpd/ft^2)	707	874	1603	1241
Mechanism		Scraper		

Note: No additional primary clarifier required for Phase 1. One additional clarifier in Phase 2.

OD = Outside diameter
TOC = Top of concrete
SOR = Surface overflow rate

Table 7-1
Recommended Wastewater Treatment Facilities
Preliminary Design Information

Item	Description: Dry Weather Average	Dry Weather Max. Month	Dry Weather Peak Day	Wet Weather Max Month
Flow (mgd)	5.036	6.227	11.426	8.844
Aeration Basins (Phased)				
Number of basins				
Total	3	3	3	3
In service	2	3	3	3
Single basin dimensions (feet)				
Length Aerobic	250	250	250	250
Anoxic/Anaerobic	107	107	107	107
Total	357	357	357	357
Width	25	25	25	25
Water depth	17.5	17.5	17.5	17.5
Aerobic volume (MG)	2.455	2.455	2.455	2.455
Aerobic volume in service (MG)	1.636	2.455	2.455	2.455
Total volume (MG)	3.507	3.507	3.507	3.507
Nominal aerobic HRT (hours)	7.80	9.46	5.16	6.66
Aerobic sludge age (days)	Scraper	8	5	10
MLSS (mg/L)	3195	2950	3127	2835
Min. RAS flow rate (mgd)	3.36	4.15	7.62	5.90
OD = (WAS flow rate (mgd)	0.082	0.123	0.196	0.098
TOC = WAS production (lb/d)	5451	7549	12801	5803
SOR = Primary sludge production (lb/d)	7432	10282	12100	6096
Diurnal peak O_2 requirement (lb/d)	11655	16169	16086	11769
Blower Sizing				
Fine bubble clean water SOTE (%)	25%	25%	25%	25%
Typical AOTE/SOTE	0.35	0.35	0.35	0.35
AOTE (%)	9%	9%	9%	9%
Air (scfm)/O_2 (lb/d) ratio	0.04	0.04	0.04	0.04
Total Blower Capacity (scfm)	5328	7392	7354	5380
Number of blowers (incl 1 spare)		6		
Capacity, each (scfm)		1500		
Blower horsepower, each		125		
Mixed Liquor Recycle Pumps				
Assumed recycle ratio		3		
Total capacity (mgd)		27		
Total number		3		
Capacity, each (gpm)		6200		
TDH (feet)		6		
Horsepower, each		13 (Use 15 hp motors)		
Pump type		Vertical turbine centrifugal, adjustable speed		

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Table 7-1
Recommended Wastewater Treatment Facilities
Preliminary Design Information

Anoxic/Anaerobic Submersible Mixers	
Number per basin	4
Number per RBC tank	2
Total number	16
Pumping capacity, each (gpm)	10000
Horsepower, each	10

Notes:

1 Use existing RBC concrete volume as anaerobic selector for future biological phosphorus removal. Existing covers could be retained to contain odor.

2 Combined volume of the two RBC tanks is approximately 280,000 gallons, which may not be adequate. Will need to provide additional anaerobic/anoxic volume in new aeration basins (about 30% of total). Above table shows minimum aerobic volume required for nitrification plus 1 MG anaerobic/anoxic.

3 Flexibility in future mixed liquor recycle suction and discharge locations should be provided.

4 Methanol addition may be required to meet future effluent nitrate/total nitrogen limits.

Table 7-1
Recommended Wastewater Treatment Facilities
Preliminary Design Information

Item	Description			
	Dry Weather Average	Dry Weather Max. Month	Dry Weather Peak Day	Wet Weather Max Month
Flow (mgd)	5.036	6.227	11.426	8.844
Secondary Clarifiers (Phased)				
Number of units				
Total	3	3	3	3
In service	3	3	3	3
Single clarifier dimensions (feet)				
Diameter	75	75	75	75
Sidewater depth	16	16	16	16
Total area (ft^2)	13254	13254	13254	13254
Area in service (ft^2)	13254	13254	13254	13254
SOR (gpd/ft^2)	380	470	862	667
SLR ($lb/d/ft^2$)	16.88	19.27	37.47	26.29
Mechanism		Tow-Bro		
OD = Outside diameter				
TOC = Number of cells				
SOR = Surface over Total	4	4	4	4
In service	2	3	3	3
Single cell dimensions (feet)				
Width	20	20	20	20
Length/Width ratio	1	1	1	1
Length	20	20	20	20
Media bed depth	5	5	5	5
Underdrain depth	2.5	2.5	2.5	2.5
Water above media	7	7	7	7
Freeboard	1.5	1.5	1.5	1.5
Total cell depth	16	16	16	16
Single cell area (ft^2)	400	400	400	400
Total area (ft^2)	1600	1600	1600	1600
Area in service (ft^2)	800	1200	1200	1200
Filtration rate (gpm/ft^2)	4.37	3.60	6.61	5.12
Waste Backwash Surge Tank Sizing				
Number of backwashes		2		
Single backwash duration (minutes)		12		
Required tank capacity (gal)		144000		
(ft^3)		19249		
Tank dimensions (feet)		8		
Length		44		

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Table 7-1
Recommended Wastewater Treatment Facilities
Preliminary Design Information

Width	44
Depth	10

Filter Backwash Pumps (Gravity Backwash from Exst Sec Clar)

Max total backwashes per day	8
Max consecutive backwashes	2
Average tank filling time (minutes)	336
Safety factor	1.5
Total pump capacity (gpm)	650
Single pump capacity (gpm)	650
Number of pumps (incl 1 spare)	2
Estimated TDH (feet)	25
Single pump BHP	6 (Use 7.5 hp motor)
Pump type	Vertical turbine centrifugal, variable speed
Wet well dimensions (feet)	10Lx5Wx12D

Note: *Provide 2 additional identical pumps for waste backwash pumping to headworks.*

Filter Air Scour Blowers

Scour rate required (scfm/ft^2)	5
Safety factor	1.25
Total blower capacity (scfm)	2500
Single blower capacity (scfm)	1250
Number of blowers (incl 1 spare)	3
Estimated discharge pressure (psig)	7.5
Single blower BHP	75
Blower type	Centrifugal, variable speed

Table 7-1
Recommended Wastewater Treatment Facilities
Preliminary Design Information

Item	Description
Disinfection Design Criteria	
AAF (mgd)	5.66
Max. monthly avg. flow (mgd)	8.84
Peak flow (mgd)	28
Plant water flow (mgd)	1.5
Max. monthly avg. reuse flow (mgd)	6.23
Peak reuse flow (mgd)	15.60
Chlorine dose (mg/L)	1.5
UV dose (mW-s/cm2)	30
Chemical Properties	
Chlorine compound purity (w/w)	12.5%
UV transmittance	55.0%
Hypochlorite bulk density (lb/gal)	10.1
Feed Equipment Sizing	
Pure chlorine peak feed rate (lb/d)	195
Deliv chlorine peak feed rate (lb/d)	1562
Deliv chlorine peak feed rate (gpd)	155
Deliv chlorine peak feed rate (gph)	6.444
Storage Sizing	
Pure chlorine 30-d use (lb)	2340
Deliv chlorine 30-d use (lb)	18716
Deliv chlorine 30-d use (gal)	1853
UV System Sizing	
	Scraper
Std int, 55% tr, 100 h (mW/cm2)	4.5
Std int, 55% tr, 8760 h (mW/cm2)	2.93
OD = Outside diameter	10.3
TOC = Top of concrete	6.36E-03
SOR = Surface overflow rate	1613
Number of redundant lamps	0
Number of channels	**2**
Banks/channel	3
Lamps/bank reqd	269
Lamps/module	16
Modules/bank	17
Actual number of lamps	**1632**

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Table 7-1
Recommended Wastewater Treatment Facilities
Preliminary Design Information

Module width=lamp spacing (inches)	3
Channel width (inches)	**51**
Channel depth (inches)	**84**
Minimum channel length (feet)	**36**
Concrete on grade	**60'x30'**
Cleaning tank	**8'lx5'wx7.5'd**

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A package chlorination system is recommended. Flexible piping would be provided to allow inclusion or bypass of the existing chlorine contact tanks in the flow configuration.

Effluent Reaeration

The upgraded plant will include effluent reaeration facilities to meet the minimum DO requirements for discharge to the Pudding River. The location will be determined during predesign.

Figure 7-1 shows a schematic of the recommended wastewater treatment facilities. The proposed layout of the facilities in shown in Figure 7-2. Table 7-1 summarizes the proposed design criteria for the new facilities. The design criteria may be refined during the predesign of the facilities.

Plant Water System

The in-plant water system will use secondary effluent disinfected with sodium hypochlorite.

Effluent Flow Measurement

The effluent flow measurement will be provided to flow pace the UV disinfection system.

New Outfall

A new outfall is proposed that will also reuse the existing outfall. The alignment of the new outfall will be evaluated during predesign. Improvements to the outfall access are also needed. Increased monitoring requirements will necessitate river access near the outfall.

Poplar Tree Irrigation

During the months of July and August, river discharge is prohibited when river flows are less than 30 cfs because of very stringent ammonia limitations. The proposed option during July and August is beneficial reuse on approximately 400 acres of poplar trees. A poplar tree plantation allows flexibility for effluent reuse, biosolids reuse, and yard debris reuse. Poplar trees can remove (evapotranspire) water, fertilizers, and organic chemicals from soils faster than seepage rates. Poplars have demonstrated rooting deeper than 6 feet into soil with water removal measured 8 feet below the surface. The roots will not be irrigated with effluent before July, which will prepare the tree roots to hold more water when irrigated during July and August.

A new irrigation pump station would be required. The pump station would apply reclaimed water to approximately 400 acres of poplar trees.

Reuse would require a dosing of chlorine in the effluent to prevent plugging of the irrigation nozzles.

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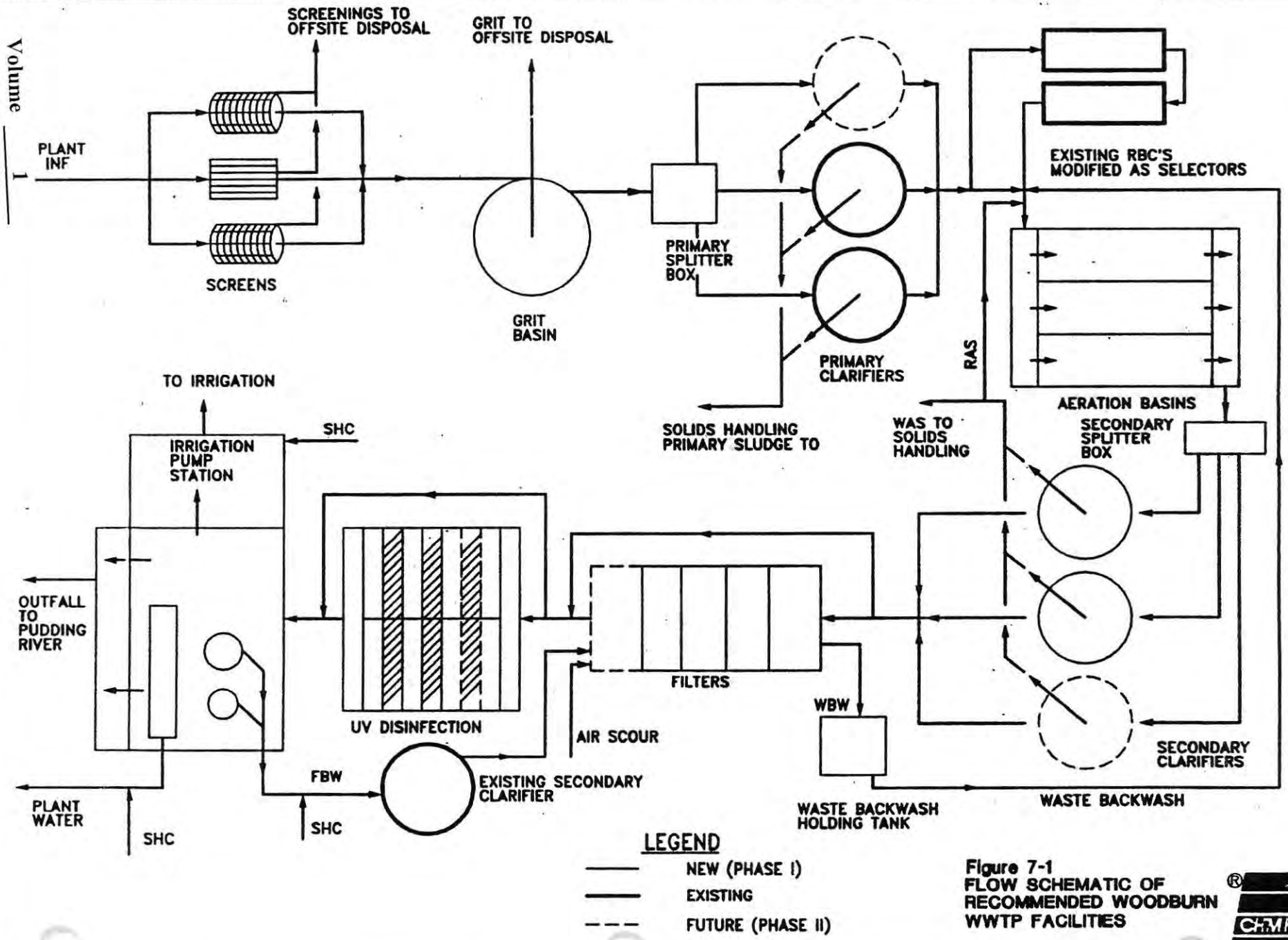

Figure 7-1
FLOW SCHEMATIC OF RECOMMENDED WOODBURN WWTP FACILITIES

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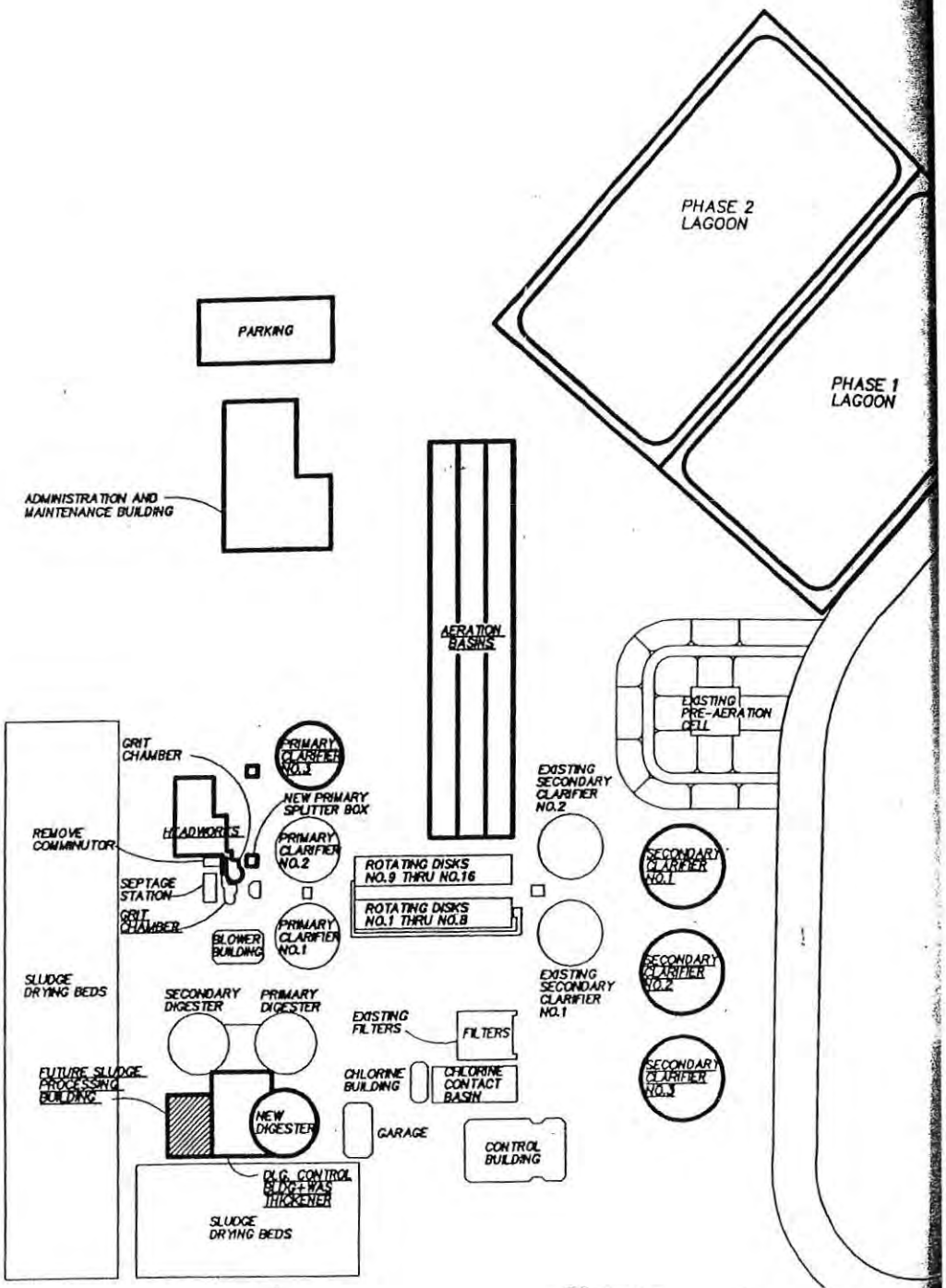

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EXISTING WWTP SITE CONCEPTU

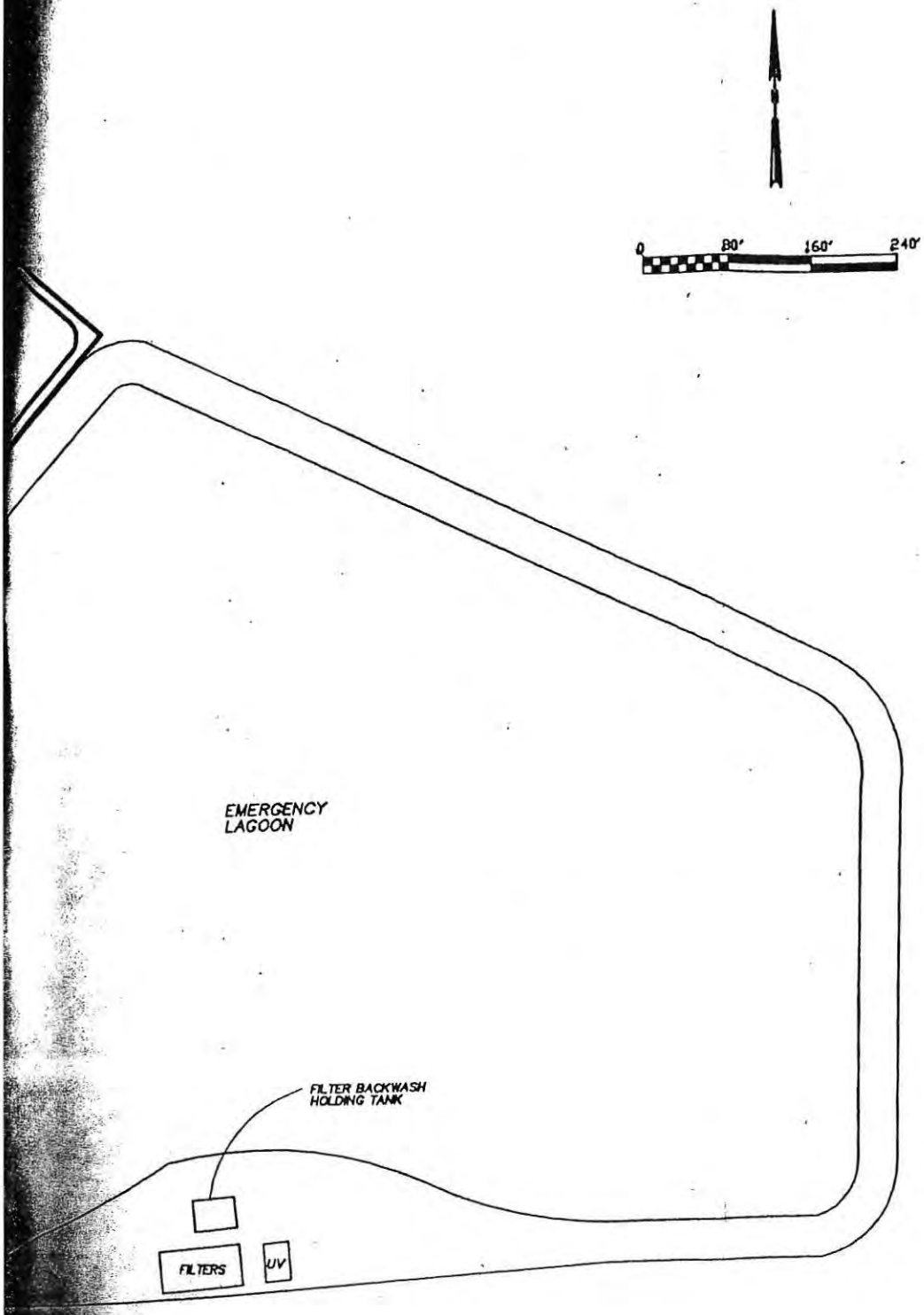

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Figure 7-2
CONCEPTUAL LAYOUT OF
RECOMMENDED WOODBURN
WWTP FACILITIES

CH2M HILL

Figure 7-2
Conceptual Layout of
Recommended
Woodburn WWTP
Facilities

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Because UV disinfection will be implemented to eliminate chlorine residual in the river discharge, a separate chlorination system will be required for reuse. Partial reuse could be accomplished with a packaged chlorination system and a limited storage volume provided by the existing chlorine contact tanks. The use of hypochlorite will eliminate the need for an emergency scrubber system.

A 10-acre poplar tree demonstration project (prototype) is planned in order to collect poplar tree data for 2 years beginning in 1995. The prototype will provide design criteria of the system based on local conditions such as: optimum tree varieties, planting densities, soil amendment requirements, loading rates, and starting with rooted versus non-rooted stock. The demonstration project will provide the data for predesign and design.

Odor Control

Odor control will be provided at the headworks and solids handling facilities. The sludge storage lagoons will have a water cap to contain the odors.

New Administration Building

A new administration building is proposed to replace the existing one. The new building is 8,500 square feet and will be similar to the new administration building constructed at the McMinnville, Oregon, WWTP.

Modification to Existing Building

The existing administration building will be modified to include offices, storage, lunchroom, lockers, toilet, and shower rooms.

Solids

Table 7-2 presents a summary of basic design concepts for the proposed solids management alternative. It can be implemented in two phases in order to minimize the cost of the initial phase of plant expansion and upgrading. The proposed alternative includes WAS thickening, anaerobic digestion, sludge storage in lagoons, and land application on the poplar tree site. The type of sludge application, whether trucked or irrigated, will be determined during predesign. Although most of the facilities outlined in Table 7-2 must be constructed initially, the second WAS thickener and the second cell of the biosolids storage lagoon can be delayed until a later construction phase.

The proposed poplar crop has a relatively high nutrient requirement so the application of biosolids to the site will be beneficial to the crop while minimizing the site requirement.

Estimated Project Costs

Estimated capital and annual operation and maintenance (O&M) costs were developed for the recommended wastewater system improvements summarized above. Cost estimates were developed to reflect 1994 costs.

Capital Costs

Estimated capital costs for the recommended wastewater system improvements are summarized in Table 7-3. The projected timing of expenditures is included under Financial Planning in this section.

Annual Operation and Maintenance Costs

The City operates the existing plant and will continue to operate the upgraded plant in the future. Operation and maintenance costs include the costs for labor, power, chemicals, equipment, and supplies to operate and maintain collection system pipelines and pump stations, treatment WWTP facilities and the biosolids reuse program. The anticipated O&M costs for the Woodburn plant are presented in Table 7-3 in 1994 dollars.

Table 7-2 Woodburn Solids Handling Processes Proposed Design Criteria
Primary Sludge Thickening • Pump Thick (± 4%) from primary clarifiers
WAS Thickening • Type: gravity belt (or centrifuge) • Size: 2.0 meter width • Number: 2 (one redundant) • Design flow rate (8 hr/day basis): 400 gpm
Anaerobic Digestion • Sizing: 20 day detention time at Maximum Month • Volume required: 950,000 gallons • Existing: Two at 338,000 gallons each (one primary, one secondary) • New: One 612,000 gallons
Sludge Storage • Type: Lagoon • Size: 3.5 m gallons • No. Cells: 2 • Sludge Removal: Dredge
Land Application • Site: Poplar tree wastewater land application site

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Table 7-3
Woodburn WWTP Expansion Cost Estimates

Wastewater Treatment Capital Cost in 1994 $		
Item	**Phase 1 Cost**	**Phase 2 Cost**
General conditions	1,940,000	550,000
Site facilities/amenities	821,000	150,000
Headworks	569,550	
Headworks structure	760,000	
Primary flow splitter structure	50,000	
Primary clarifiers		325,000
Aeration basins	2,060,000	825,000
Blower building	540,000	120,000
Secondary flow splitter structure	150,000	
Secondary clarifiers	950,000	578,000
RAS/WAS Pump Station	1,000,000	
Chemical addition	250,000	
Mixed media filters	1,885,000	635,000
UV disinfection	767,000	272,000
Emergency generator	76,000	
Plant water system	250,000	
Effluent flume	75,000	
Outfall	850,000	
Odor control		500,000
Solids handling	3,353,000	750,000
Electrical and controls	1,070,000	300,000
Lab, administration and maintenance building	1,275,000	
Existing admin/lab structure modifications	335,000	
Legal, administration and engineering, @ 23%	4,376,106	1,151,150
Contingency, @ 15%	2,853,982	750,750
Total Capital Cost	***$26,257,000***	***$6,907,000***

Poplar Tree Irrigation Capital Cost in 1994 $		
Poplar Tree Irrigation (incl Legal, Engr, Admin & Cont @ 38%)		
Total Capital Cost	***$3,980,000***	***$975,000***

Collection System Capital Cost in 1994 $		
Collection System (incl Legal, Engr, Admin & Cont @ 38 %)		
Total Capital Cost	***$3,555,000***	***$2,283,000***

Woodburn WWTP Expansion Cost Estimate Summaries

Total Capital Cost (1994 $)	***$33,792,000***	***$10,165,000***
Total Capital Cost (1998 $) (from financial plan)	***$38,354,000***	***$11,892,000***

Phase 2 Cost (2008 $)		***$17,606,000***
Phase 1 and 2 Costs (1998 $)	***$50,246,000***	

Total Cost Without Phasing (1998 $)	***$49,057,000***

Assumes 4.0 % Inflation

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Program Implementation

The wastewater treatment system improvements described previously must be implemented to comply with current effluent discharge requirements and future discharge requirements defined in the SFO. The purpose of this section is to identify activities necessary to implement the recommended improvements, estimate the duration of these activities, define a recommended implementation program to comply with the intent of the DEQ compliance order, and describe implementation issues. A project schedule is shown in Figure 7-3. The implementation steps for the wastewater plant improvements are described in chronological order below.

Facilities Plan Approval and Adoption

This facilities plan and its recommendations should be approved by DEQ and adopted by the City of Woodburn before proceeding with the next steps. It is assumed that approval will be granted by January 1996 in the schedule attached.

The facilities plan approval process must include:

- A dilution waiver for the Pudding River
- Mass load increases for BOD_5 and TSS
- Grit and Secondary Clarification Redundancy and Reliability waiver
- Mixing zone ammonia limit resolution
- Permit drafted that will address new plant operation

Dilution Waiver

A dilution waiver is required for discharge to the Pudding River for BOD_5 and TSS. In the total maximum daily load (TMDL) report (DEQ 1993b), DEQ states that the river has assimilative capacity. The new TMDL-based permit limits reduced oxygen-demanding loads from Woodburn to ensure adequate dilution ratios during low flows and to prevent oxygen sags. A dilution waiver is still required.

Mass Load Increase

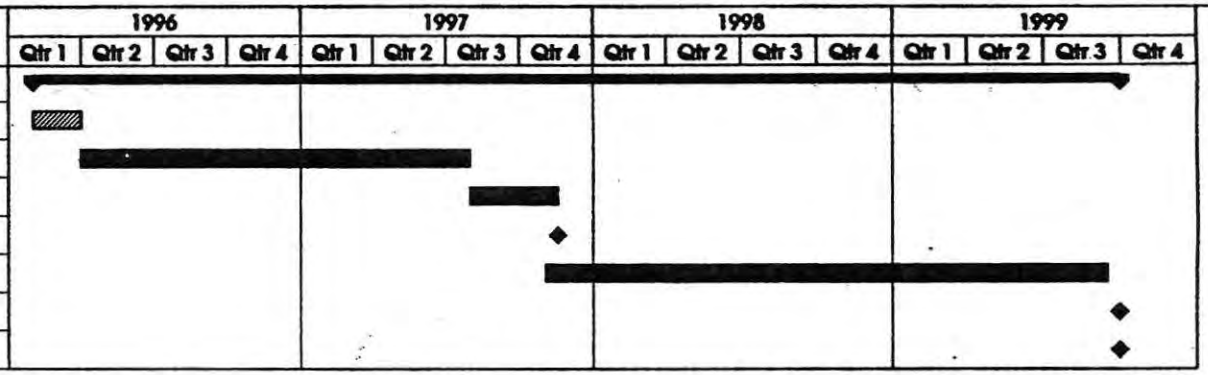
Mass load increases are required to accommodate the increased flow projected for the expansion. The required TMDL-based BOD_5 and TSS effluent concentrations remain at 10 mg/L in summer and 30 mg/L in winter. Mass load increases are requested based on increased flow. The TMDL-based effluent quality requirements were derived to protect the receiving stream at the projected flow.

Redundancy and Reliability

The redundancy and reliability guidelines for grit removal and secondary clarification will not be met. The vortex grit removal unit has no submerged moving parts that would necessitate taking it out of service. Only two secondary clarifiers will be constructed during

FIGURE 7-3
CITY OF WOODBURN

ID	Name
1	A. WOODBURN WWTP DESIGN EXPANSION
2	1. Predesign
3	2. Design Expansion of the Woodburn WWTP
4	3. Bid Phase
5	4. Award Construction
6	5. Construct WWTP Expansion
7	6. Complete Construction
8	7. Compliance with TMDLs



Project: City of Woodburn
Date: 2/20/95

Critical
Noncritical
Progress
Milestone
Summary
Rolled Up

the first phase. The third will be constructed in a future phase. If one of the two units is out of service during peak flows, the redundancy and reliability requirements are not met. However, they can be met at average design flow. A consensus that this is acceptable is requested.

Ammonia Limit Resolution

The SFO states that ammonia limits may be affected by the mixing zone analysis. Resolution is requested before predesign begins.

Draft Permit

Approval of the facilities plan includes obtaining the mass load increases, dilution waivers, and a consensus about redundancy and reliability requirements for the secondary clarifiers and resolution of questions regarding ammonia limits. In addition, a draft permit is requested to define how the new treatment works will be regulated. This is necessary before predesign in order to avoid surprises and costly changes during design.

Treatment Plant Predesign

Once the facilities plan has been approved and adopted, which includes permit negotiation to the draft permit level, the predesign effort can begin. The predesign provides a more detailed engineering development of the facilities presented in the facilities plan recommendation. The predesign must be completed within 4 months after approval of the facilities plan. Predesign typically includes flow diagrams, design criteria, and general layouts of the facilities and piping. The predesign is typically the 10 to 15 percent level of the total design effort. The DEQ has guidelines that define the information that must be in the predesign. The predesign must be approved by DEQ before the final design can begin. The final permit should be negotiated before design begins.

Treatment Plant Final Design

The final design involves the preparation of design drawings, specifications, and contracts that describe the scope of the improvements to be constructed. Designs must comply with industry standards, building codes, safety requirements, permits, and other standards. Designs are typically subject to review and approval by local and state agencies. The DEQ must review and approve the construction contract documents before construction can begin. The final design must be completed within 18 months after approval of the facilities plan.

Construction Bidding

This step involves the activities to solicit bids from interested contractors to construct the project. The bidding process for public projects is regulated by state law. The bidding period includes preparation of bid documents, bid advertisements, design clarifications, bid opening, bid evaluations, and ultimately selecting a construction contractor. For a project of this size, the bid period will typically be 4 to 6 weeks with another 6 weeks to evaluate bids

and award the contract. The SFO requires that the contract be awarded 24 months after approval of the facilities plan.

Treatment Plant Construction

The estimated construction cost of the recommend plant improvements is $30.24 million dollars (1994 dollars). Wastewater treatment plant construction typically includes site excavation, concrete work for the treatment processes, and complex mechanical and electrical equipment. Several months will also be required at the end of construction for startup and operational testing of the treatment processes. The SFO requires that the construction be completed 44 months after approval of the facilities plan, and attain operational level 50 months after approval of the facilities plan.

Financial Plan

A key consideration in the implementation of the wastewater treatment improvements program is the funding source and impact on the City of Woodburn' s wastewater fund and on the City sewer customers. A financial analysis of the facilities plan will be prepared to address these issues. This will be included in Volume II of this facilities plan.

Section 8

Environmental Assessment

Introduction

This section contains the Environmental Assessment for the wastewater treatment plant upgrade, required by the U. S. Environmental Protection Agency (EPA). The outline of the document is complete at this time. However, there is still information that needs to be filled in and included in the document. Most of this information pertains to the poplar tree site, and cannot be completed at the time of this draft. When the site has been selected, this document should be revised and completed.

The following sections clearly will be updated when the poplar tree site is located:

A. 4. Funding

B. 2. b. Water Areas, Wetland, Watersheds and Groundwater Resources

C. 2. c. Agricultural Lands

D. Environmental Effects Associated with the Proposed Action

Additional sections may need updating, depending on the location of the poplar tree reclaimed water irrigation site.

Environmental Assessment Outline

A. Project Identification

1. Name of Applicant: City of Woodburn

   Address: 270 Montgomery Street

   Woodburn, OR 97071

2. Project Number: C-410509

3. Project Title: City of Woodburn Wastewater Treatment Plant and Sewer System Improvements

4. Funding:

   a. Total Estimated Project Cost (1994) $33,792,000

   b. Total Estimated Eligible Project Cost

      1) EPA Share of Eligible Cost $ ~~6,758,400~~

      2) Local Share of Eligible Cost

   c. Planning and Design Allowance (EPA Share) $_____

   d. Total Estimated EPA Share of Cost $_____

   e. Total Estimated Local Share of Project Cost $33,792,000

5. Estimated Annual User Cost

   a. Per Household $ 320

   b. Median Household Income (1980 Census) $_____

B. Description of Proposed Project

1. Project Type:

The City of Woodburn is proposing to construct a sewerage system improvement project. The project consists of an upgrade to the existing wastewater treatment plant.

Improvements to the wastewater treatment plant include a new headworks, an additional primary clarifier, new activated sludge basins with selector technology, new secondary clarifiers, new deep filters, new UV disinfection, new outfall, new

poplar tree irrigation system for July and August, new odor control, a new administration building, and modifications to the existing administration and laboratory building.

The project will address the new Total Maximum Daily Loads (TMDLs) established by the Oregon Department of Environmental Quality (DEQ). The agency determined the Pudding River as a water-quality limited stream because of violations of the instream dissolved oxygen (DO) standard during the summer low flows. The proposed project will be built in phases to accommodate growth. This environmental assessment will cover construction of the whole project. It should be updated when the poplar tree site is located.

2. Specific Project Information

a. Existing Sewerage Facilities

(1) Wastewater Treatment Plant

The City of Woodburn's existing wastewater treatment plant was constructed in 1980. It is located within the city limits on the banks of the Pudding River. The plant provides tertiary treatment with screening and grit collection, primary clarification, rotating biological contactors (RBCs), final clarification, tertiary filtration, chlorination, sludge digestion, and sludge drying beds. The plant was designed for an average dry weather flow of 3.1 million gallons per day (mgd). The treated effluent is discharged into the Pudding River.

Because of increasing organic loading contributions to the treatment plant by the industries, several modifications in process operation have occurred. However, the plant has met the NPDES requirements since its construction. With the new regulatory requirements established by DEQ, the plant needs to be upgraded to meet those standards.

(2) Existing Collection System

The City's existing sanitary wastewater collection system collects wastewater from residences, businesses, industries, and public facilities and conveys the wastewater to the City's Pudding River wastewater treatment plant. For the most part, the existing wastewater collection system was built in the 1950's and conforms to the topography of the Mill Creek basin. Nine pump stations pump wastewater from areas that cannot flow by gravity to the treatment plant, due to the flat topography in the Woodburn area.

Approximately 2,087 acres within the UGB are currently sewered. The total length of municipal sewer, excluding private service laterals,

is approximately 52 miles and pipe sizes range in diameter from 8 to 36 inches.

Two major interceptors comprise the majority of the City's gravity collection system (the following descriptions are paraphrased from the *Woodburn Comprehensive Plan*, February 1989):

- The Mill Creek interceptor follows Mill Creek from the Mill Creek pump station south to Cleveland Street, then west to Settlemier Avenue. It intercepts flow from smaller collector systems both east and west of Mill Creek.

- The Front Street interceptor serves areas west of Front Street by collecting main and lateral lines south to Cleveland Street and west to the Interstate I-5 freeway.

The flat topography adjacent to the streams of the Mill Creek basin requires the use of nine wastewater pump stations. The pump stations serve low lying areas and areas on the extremities of the collection system.

The existing collection system occasionally exceeds its capacity. The Mill Creek pump station and the Mill Creek interceptor are particularly troublesome areas. According to the collection system analysis, peak flow resulting from a 5-year storm event will exceed the capacity of the existing collection system. Pump station and pipeline improvements currently required include:

- Upgrade of the Mill Creek Pump Station

- Hydraulic relief for the Mill Creek Interceptor through the downtown area to Lincoln Street

- Hydraulic relief for the trunkline along Highway 214 and Astor Way serving the north part of town

- Monitoring of four additional pump stations to determine whether they should be upgraded in the immediate future

(3) Infiltration and Inflow (I/I)

An infiltration and inflow study and analysis was performed in June 1993. This analysis identified approximately 2 mgd of I/I that was cost-effective to remove in the Woodburn wastewater collection system. Efforts by the system maintenance staff have focused on removing inflow from three areas:

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- Near Conroy Packing
- Fifth Street near St. Luke's Church
- At the intersection of First Street and Harrison Street

Infiltration and the remaining inflow in other areas are not cost-effective at this point to remove.

An additional analysis was completed as part of the facilities plan (Section 2). This analysis indicates the I/I in the Woodburn collection system is below the EPA target values (120 gpcpd during high groundwater and 275 gpcpd during peak I/I conditions such as a storm event) and warrants no further investigation or removal for their purposes.

3. Proposed Project

The proposed new treatment plant will be located adjacent to the existing facilities, on an area presently leased by the City. The recommended plant configuration includes:

- New headworks
- Maintaining the existing primary treatment facilities
- Activated sludge with selector technology for secondary treatment
- Deep bed filtration
- Ultraviolet radiation
- Pudding River discharge with 2 months of effluent reuse on poplar trees

4. Purpose and Need

Improvements to the City's system are needed to meet new water quality regulations established by EPA and DEQ. These regulations designate wastewater treatment and discharge standards that must be met to protect the Pudding River and other streams that receive treated wastewater. The City's current system is not able to meet these stringent new standards.

In addition to upgrading the existing system to meet regulatory requirements, the improvements will provide for increasing the system's capacity in order to accommodate planned growth in the City.

C. Description of Existing Environment

1. Area Description

The City of Woodburn is located midway between Portland and Salem in Oregon's Willamette Valley. This is the northeastern corner of the French Prairie, which slopes northeast from Salem and varies in elevation from 200 to 170 feet above sea level. The study area drains generally north-northeast through Mill Creek and the Pudding River.

2. Unique or Environmentally Sensitive Areas or Resources

a. Surface Water

Mill Creek is the major drainage course through Woodburn, and drains into the Pudding River 8 miles to the north near Aurora. The Pudding River is the receiving water for the existing treatment plant. Its average monthly flows are as low as 63 cubic feet per second (cfs) in summer, or roughly 40 mgd. Recreational activities in the Pudding River include water contact sports like fishing and boating.

b. Water Areas, Wetlands, Watersheds and Groundwater Resources

Water areas, wetlands, and watersheds have not been identified in Woodburn. However, the DEQ has provided the City with information identifying a sensitive groundwater area underlying the entire City and urban fringe.

The City, according to the Woodburn Comprehensive Plan, has been made aware of the potential impacts to groundwater that underground storage tanks, storm drainage, chemical spills, residential on-site sewage disposal systems, and other similar land uses, can have.

A groundwater monitoring plan has been implemented at the existing site. The 1994 Groundwater Report is provided in Appendix 8.1.

c. Agricultural Lands

The poplar tree irrigation site has not yet been chosen. However, the reuse site will be sited on agricultural land so land use will not change. The existing treatment plant site and MacClaren School land may be the poplar tree irrigation site. Additional land is needed.

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d. Fish and Wildlife

Fish habitats in the Woodburn Planning Area are the Pudding River and Mill Creek. The Pudding River has fish along its entire length, including winter steelhead, coho salmon, fall Chinook, and cutthroat trout. Warm water game fish include large mouth bass, white crappie, bluegill, sunfish, brown bullhead, and channel catfish. A significantly high population of freshwater mussels was noted in a routine survey by DEQ, but variety and number of species present have not been determined.

Mill Creek is known to support cutthroat trout. Both Mill Creek and the Pudding River also have several nongame fish species, including redside shiner, squawfish, coarse-scale sucker, dace, and others.

Wildlife in the planning area consists primarily of small animals such as beavers, opossum, rabbits, and muskrats. Several species of amphibians and reptiles are also found throughout the area (snakes, salamanders, frogs, newts).

Numerous small birds and several game birds, such as pheasant, quail, owls and waterfowls, inhabit the area. These are most commonly found in open space areas which offer some protective vegetation.

3. Population and Land Use

The *Woodburn Comprehensive Plan*, as part of its 1988 Periodic Review, chose to adopt the population projections made by the Center for Population Research at Portland State University. The City's 1990 census was 13,500 (2 percent higher than projected).

In 1986, a Capital Improvements Plan developed by the City's consultant and the Planning Department, estimated a 3.4 percent growth rate for a 30-year planning period. The City assumed those projections to be more conservative and added other population groups to the projections.

The present land use in the City of Woodburn is predominantly residential (56 percent), commercial (13 percent), and industrial (12 percent). The City, at present, consists of almost 2,250 acres of which 1,686 or two thirds are currently in use. According to the City's Comprehensive Plan, approximately 256 acres of the vacant lands are not available for development. However, 595 acres within the present city limits are vacant and available for urban developments. The City in general contains approximately 211 acres currently in use for commercial land. Underdeveloped lands within the City consist of approximately 15 acres. The City currently has 133 acres vacant and available for commercial development.

D. Evaluation of Alternatives

1. No Action Alternative

If this alternative was chosen, the existing treatment plant would not meet new water quality regulations, which would result in permit compliance problems and detrimental effects on fish habitat in the Pudding River.

2. Comparable Alternatives

The following alternatives were presented to and discussed by the Woodburn City Council and the Wastewater Advisory Committee:

a. Alternative I-Upgrade and expand existing plant at existing plant site

b. Alternative II-Existing site co-location with dual treatment process.

c. Alternative III-Existing site and new site; new plant located at new site.

d. Alternative IV-New plant at new site; abandon existing plant and existing site.

e. Tertiary Treatment: Mechanical filtration or wetlands treatment system.

f. Effluent disposal and reuse options: Willamette River, Pudding River, irrigation at agronomic application rates.

g. Effluent disposal during the summer: Option A-Discharge to the Pudding River, with 88-acre storage lagoon for July and August, or Option B-Irrigation at agronomic rates for May through October, with 58-acre storage lagoon and 300 acres irrigable land.

h. I/I correction and cost-effectiveness analysis.

Alternative I-Upgrade and Expand Existing Plant at Existing Plant Site-was selected. For tertiary treatment, mechanical filtration was selected. The effluent disposal options were refined. The treatment plant will discharge to the Pudding River from September through June. During July and August, the treatment plant effluent will be reused to irrigate poplar trees. Because of the higher water uptake rate of poplar trees, it is assumed that a storage lagoon for summer effluent flows will not be necessary.

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E. Environmental Effects Associated with the Proposed Action

1. Direct Environmental Impacts

a. Surface Water Quality

Excavation activities to construct this project could cause temporary negative impacts on surface water quality. Runoff from disturbed soils may temporarily increase the level of suspended solids in the Pudding River. Restrictive and standard construction practices will minimize the adverse effects.

b. Groundwater Quality

Short-term impacts could include releases of hazardous materials (gasoline, solvents) during construction that could infiltrate the groundwater. Biosolids application could result in localized infiltration of nutrients into the groundwater. A safety management of hazardous materials during construction can reduce the risk of groundwater contamination. Nutrient loads to agricultural fields can be monitored to evaluate groundwater levels and water quality. The monitoring program may consist of monitoring wells. Currently, three monitoring wells are active. Additional monitoring wells are anticipated.

c. Air Quality, Noise Pollution and Aesthetics

Short-term construction activities associated with the operation of heavy equipment (engine emissions and dust) will temporarily affect local air quality. The contractor can use dust control techniques (water the roadways) to minimize construction dust. The biosolids stabilization process will not change, so there will be no impact change from current biosolids land application. The treatment plant will continue to use odor control air scrubbing.

Residences adjacent to the plant site may be impacted by the operation of heavy equipment during construction Noise during the construction day is somewhat unavoidable. Construction will not be allowed at night, on weekends, or holidays.

d. Plant/Wildlife Habitat

The new treatment facilities are located on the existing plant site, so no impact is anticipated. The new reuse site will be maintained agricultural so no impacts are anticipated. Temporary disturbances to surrounding wildlife may occur because of construction activities.

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Proper construction practices and timely re-vegetation of the area will minimize the impact on the natural environment.

2. Financing the Project

The project is expected to be financed by bonds and state revolving funds (SRFs).

3. Indirect Environmental Impacts

None identified.

4. Unique and Sensitive Environmental Areas

a. Wetlands

No wetlands are on the existing plant site and none will be effected as a result of the construction of this project.

b. Floodplains

The existing and future treatment facilities are not in the floodplain.

c. Agricultural Lands

The proposed poplar tree irrigation site has not yet been chosen. However, the reuse site will be sited on agricultural land so land use will not change. The site will receive plant effluent for beneficial reuse.

d. Coastal Zone Areas

The eastern boundary of the Oregon Coastal Zone pursuant to the Coastal Zone Management Act of 1972 (PL 92-583) is the crest of the Coast Range. No coastal zone areas will be impacted as a result of this project.

e. Fish and Wildlife

The project will have a direct positive impact on aquatic life in the Pudding River. During periods of summer low flow, plant effluent will not discharge to the Pudding River. During the remainder of the year, the quality of the plant effluent discharged to the Pudding River will be improved.

The new treatment facilities are located on the existing plant site, so no additional areas will be impacted by the plant itself. The new reuse site will be maintained agricultural so no impacts are anticipated.

f. Wild and Scenic Rivers

No federally designated wild and scenic rivers as defined by the Omnibus Oregon Wild and Scenic Rivers Act of 1988, or state-designated scenic waterways as identified by the Oregon Scenic Waterway Act of 1968 are located in the project area.

g. Threatened and Endangered Species

No known threatened or endangered species will be effected by the proposed project.

h. Historic and Cultural Resources

No known historical or archaeological resources will be effected by this project. New construction will be in previously disturbed areas on the plant site so no impact is anticipated.

i. Sole Source Aquifer and Recharge Areas

No sole source aquifers or groundwater recharge areas will be effected as a result of this project, pending location of the poplar tree irrigation site.

j. Wellhead Protection Area

There are no wellhead protection areas designated in the planning area, pending location of the poplar tree irrigation site.

5. Irreversible and Irretrievable Commitment of Resources

The proposed project would commit construction materials, energy use, and labor for construction. These resources are irretrievably committed. The local citizens are committed to support the wastewater treatment system through user charges.

6. Short-Term Use of the Environment versus Maintenance of Long-Term Productivity

Temporary, minor environmental impacts from construction activities will occur. Improvements at the existing treatment plant, construction of a new treatment plant, and improvements to the collection system require site grading and excavation that would destroy vegetation and expose soils to some short-term erosion. Outfall construction would also expose soils. Erosion control measures can be utilized to minimize any impacts.

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Construction will likely cause noise levels to increase and air quality may decrease slightly. The available buffer area at the plant site should reduce the chances for disturbances occurring.

The trade-offs are the long-term benefits of providing necessary treatment to all wastewater. Reduction of pollutants in the plant effluent to the Pudding River will enhance the river's quality, augmenting instream dissolved oxygen levels and protecting fish and wildlife habitat.

F. Steps to Minimize Environmental Impacts

1. Mitigation Measures

Construction impacts will be mitigated/minimized by proper construction practices including:

- Soil runoff control measures
- Appropriate dust control measures
- Minimizing effects due to noise and air pollution, and temporary traffic control
- Scheduling excavation/high-impacting activities to periods of low rainfall.

Project design in the floodplain will consider peak flood elevations to ensure that construction and site location provide minimal impact during a flood event.

2. Policy and Legal Constraints

The policy and legal constraints for this project are included in the following documents:

a. Oregon Land Conservation and Development Land Use goals and guidelines.

b. Woodburn Comprehensive Plan (City of Woodburn 1989).

c. Oregon Administrative Rules, Chapter 340.

d. Clean Water Act.

3. Conformance with State Implementation Plan (SIP)

The proposed project does not include new direct sources of air pollution. The project is in conformance with the SIP for Oregon.

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G. Documentation

1. The following documents were used to prepare this assessment:

   a. *Woodburn Comprehensive Plan*, amended in 1989.

   b. Federal Wild and Scenic Rivers Act.

   c. Facilities Plan Technical Memoranda, 1990-1991.

   d. *Volume I-Treatment, Wastewater Facilities Plan, City of Woodburn*

2. Agencies Contacted:

   a. Department of Environmental Quality.

   b. City of Woodburn.

   c. Woodburn Planning Department.

H. Public Participation

(see Section 9 of Volume I of the facilities plan)

I. Reasons for Concluding There will be No Significant Impact

The proposed project involves upgrade of existing facilities and construction of a new facility. Construction activities will have minor short-term environmental impacts, but they will be minimized.

Appendix 8.1

1994 Summary of Groundwater Monitoring Results for the Woodburn WWTP

Dave Wightman

CH2M HILL
Engineers
Planners
Economists
Scientists

November 30, 1994

OPW27874.B1

Woodburn Wastewater Treatment Facility
2815 Molalla Road
Woodburn, Oregon 97201

Attention: Frank Sinclair, Superintendent

Subject: 1994 Summary of Groundwater Monitoring Results

Dear Mr. Sinclair:

Attached is a technical memorandum containing the 1994 Summary of Groundwater Monitoring Results for the City of Woodburn's Wastewater Treatment Facility. Enclosed are three doubled-sided copies for you to attach to your cover letters to DEQ. Also enclosed are two additional copies for your files.

If you have any questions or concerns, please call me at 235-5000, Extension 4373.

Sincerely,

CH2M HILL

Patty O'Connor

Patty O'Connor
Groundwater Resources

sem/1994covr.ltr

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Portland Office
825 NE Multnomah, Suite 1300
Portland, OR 97232-2146
503 235-5000
Fax No. 503 235-2445

MEMORANDUM

CH2M HILL

TO: Frank Sinclair/Woodburn Wastewater Treatment Facility
David Mann/DEQ

COPIES: Frank Tiwari/Woodburn City Hall
Ken Shump/CH2M HILL
Daria Wightman/CH2M HILL
Jack Ardent/Western Region DEQ Salem Office

FROM: Patty O'Connor/CH2M HILL

DATE: November 30, 1994

SUBJECT: City of Woodburn Wastewater Treatment Facility
1994 Semiannual Sampling Summary of Results

PROJECT: OPW27874.B1

This memorandum presents 1994 semiannual groundwater monitoring results for the City of Woodburn's Wastewater Treatment Facility. The groundwater monitoring program was performed to assess nitrate concentrations in shallow groundwater downgradient of the facility's lagoon, in accordance with the National Pollutant Discharge Elimination System (NPDES) permit requirements.

Summary of Monitoring Program

The 1994 monitoring program consisted of data collection events on April 8 and October 17. Monitoring tasks included water-level elevation measurements and groundwater sample collection from four shallow monitoring wells, one located upgradient, or northwest, of the lagoon (MW-1), and three located downgradient, or southeast, of the lagoon (MW-2, MW-3, and MW-4), as shown in Figure 1. Monitoring activities were consistent with the procedures described in the *Woodburn Wastewater Treatment Facility Lagoon Groundwater Monitoring Report* (CH2M HILL, 1990).

The analytical results were used to evaluate current (1994) and long-term (1990 through 1994) trends in nitrate concentrations at the site. Water level data were used to estimate groundwater elevations and gradients for 1994. These data were compared with previous data to assess variability of shallow groundwater conditions at the site.

02-07-90

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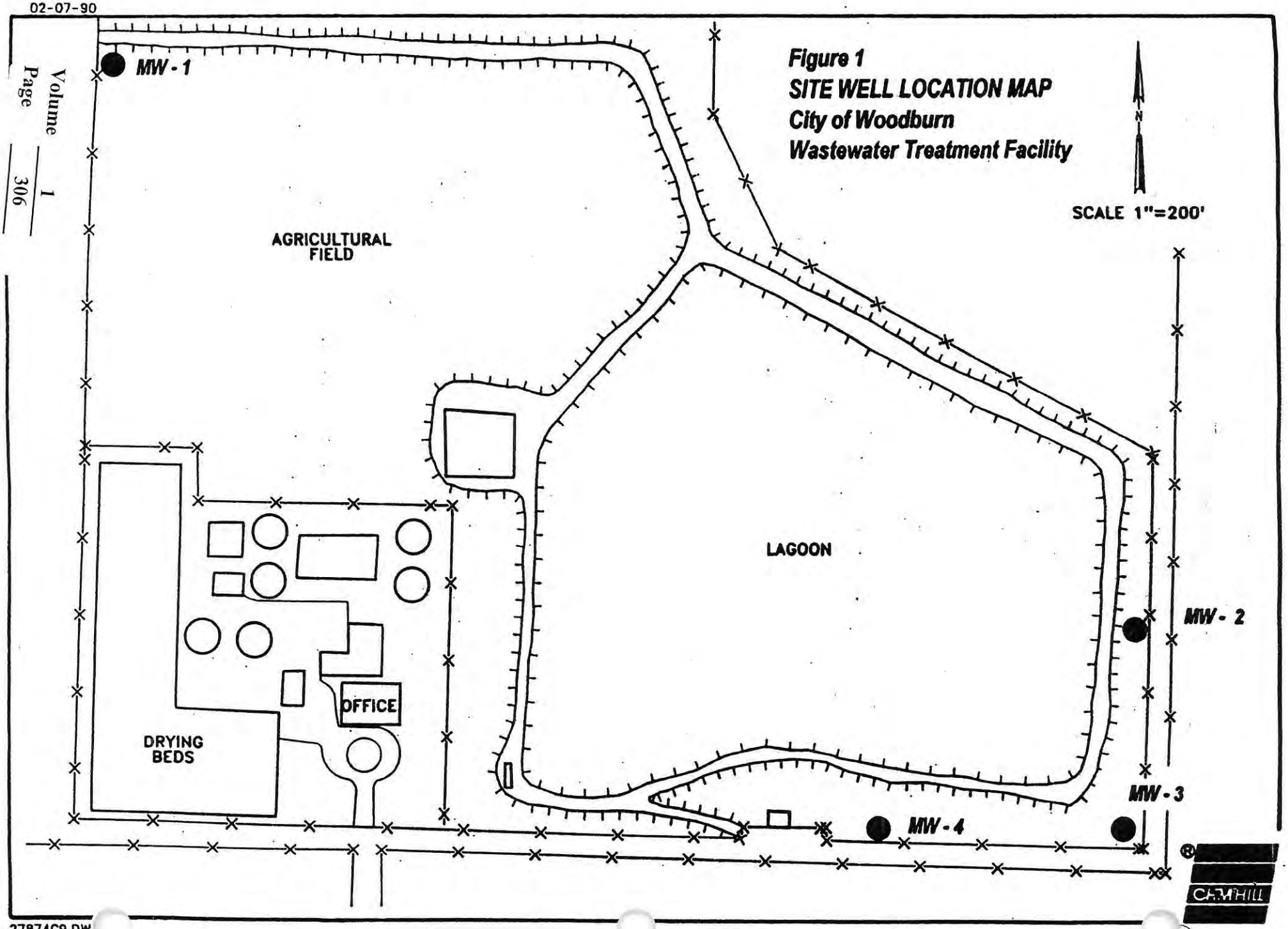


27874G9.DW

In summary, the 1994 data indicate the following:

- Nitrate concentrations in the three wells southeast of the lagoon (MW-2, MW-3, and MW-4) are below the detection limit of 0.10 milligram per liter (mg/L). These concentrations are below the numerical groundwater quality reference levels of 10 mg/L established for drinking water in the state of Oregon (OAR 340-40-080).

- Nitrate concentrations in upgradient well MW-1, located 900 feet northwest of the lagoon, remain elevated at 21.0 mg/L for both April and October 1994 groundwater sampling events.

- In general, the shallow groundwater flow direction at the site is southeast toward the floodplain of the Pudding River. Long-term measurements at site monitoring wells indicate that groundwater levels decline in response to seasonal dry periods or drought years, and rise in response to recharge during the rainy season. Groundwater levels in MW-1 appear to respond to variations in precipitation recharge to a greater degree than groundwater levels in MW-2, -3, and -4, which may suggest that groundwater levels near the lagoon are more stable, possibly because of groundwater mounding near the lagoon. For a limited time during dry periods, groundwater levels in MW-1 occasionally drops below the levels observed in MW-2 and MW-4, indicating the potential for groundwater flow to the northwest. This situation is anticipated to be temporary and localized in the immediate vicinity of the lagoon. The net groundwater flow direction beneath the site is believed to be southeast toward discharge points on the floodplain of the Pudding River.

Discussion of Field Sampling Methods

Groundwater samples were collected from Wells MW-1, MW-2, MW-3, and MW-4 on April 8 and October 17, 1994. The groundwater samples collected during the semiannual sampling rounds were analyzed for nitrate as nitrogen using Environmental Protection Agency (EPA) Method 300.0. Table 1 is a summary of the laboratory analytical results from 1994, including historical data beginning in 1990. Groundwater samples were recorded on chain-of-custody forms and hand delivered to North Creek Analytical Laboratory in Beaverton, Oregon.

Before sampling, monitoring wells were purged a minimum of three well casing volumes using either a polyethylene bailer (April 1994) or a peristaltic pump (October 1994). Prior

Table 1
Nitrate Concentrations in Groundwater (a)
1990 - 1994 Semiannual Monitoring Data
City of Woodburn Wastewater Treatment Facility

Date	MW-1	MW-2	MW-3	MW-4
04/12/90	31.0	0.06	0.24	< 0.03 (c)
07/12/90	15.3	0.04	0.05	< 0.03
07/01/91	30.8	< 0.04	< 0.04	< 0.04
12/19/91	8.89	< 0.03	0.06	< 0.03
08/08/92	24.4	0.43	0.82	< 0.03
12/22/92	25	< 0.10	0.25	< 0.10
05/04/93	27	< 0.10	< 0.10	< 0.10
10/05/93	26	< 0.10	< 0.10	< 0.10
04/08/94	18	< 0.10	< 0.10	< 0.10
04/08/94	21 (b)	–	–	–
10/17/94	21	< 0.10	< 0.10	< 0.10
10/17/94	21 (b)	–	–	–

(a) Analytical method EPA 300.0 - Nitrate as Nitrogen.
Laboratory results reported in mg/L.
(b) Duplicate value for MW-1
(c) "<" Nitrate concentration is less than the detection limit cited.

*** State of Oregon numerical groundwater quality reference levels for Nitrate as Nitrogen is 10 mg/L.

to purging, groundwater levels were measured to calculate the well casing volume and to evaluate static water level conditions. During purging, groundwater pH, temperature, and electrical conductivity were monitored, and groundwater samples were collected when these parameters had stabilized. Samples were collected with either a dedicated polyethylene bailer (April 1994) or dedicated polyethylene tubing from the peristaltic pump (October 1994). Water levels, calculated purge volumes, and parameter measurements were recorded on field sampling forms. Table 2 provides a summary of field parameters compiled from the field sampling forms for the two 1994 sampling events.

Groundwater Levels and Flow

Groundwater Elevations

In general, sitewide groundwater elevations appear to respond to seasonal fluctuations in precipitation. Well MW-1 shows a greater response to precipitation than Wells MW-2, MW-3, and MW-4, which are adjacent to the lagoon. It is probable that groundwater mounding near the lagoon more strongly influences water-level elevations in Wells MW-2, MW-3, and MW-4 than in well MW-1, which is approximately 900 feet northeast of the lagoon.

Water-level fluctuations from 1989 through 1994 are summarized in Table 3 and presented in Figure 2. Water levels declined in each of the four wells during 1991, in response to low precipitation conditions. The maximum decline during this period was 12 feet in Well MW-1, compared with 3 to 4 feet of decline in Wells MW-2, MW-3, and MW-4. The water level in Well MW-1 continued to decline during the dry summer season of 1992, in contrast to Wells MW-2, MW-3, and MW-4, in which water levels rose during the summer of 1992 in response to localized recharge near the lagoon. Groundwater elevations in all four wells rose during both 1992 and 1993 winter seasons.

1994 wet season (April 1994) groundwater levels were generally lower than in previous years (Figure 2). This decline may be related to lagoon levels at the Facility; the lagoon was empty for approximately 3 weeks prior to this groundwater level measurement event. Lower precipitation rates and a lack of lagoon water recharge may have resulted in lower water levels in the downgradient wells. Upgradient well MW-1 static water level was also slightly lower than observed in previous years however, unlike the three downgradient wells, the groundwater level in MW-1 reflected an increasing trend from the previous dry season level (October 1993). An increase in water levels were observed in wells MW-2 (4.18 feet) and MW-4 (5.15 feet) during the dry period (October 1994). Little or no precipitation had occurred in the Woodburn area during this month, therefore the water level increase for these well locations may be attributed to an elevated lagoon Figure 2

Table 2
Field Parameters Summary
1994 Semiannual Monitoring Data
City of Woodburn Wastewater Treatment Facility

Field Parameters	MW-1 (Well Depth: 35 feet)					MW-2 (Well Depth: 30 feet)					MW-3 (Well Depth: 32 feet)						MW-4 (Well Depth:33 feet)				
April 8, 1994 Groundwater Sampling Event																					
Depth to water	12.15					15.54					17.70						15.57				
Target Purge Volume in Gallons (a)	11.2					7.1					9.3						8.5				
Gallons removed during purging	0	3.7	7.4	11.2	AS (b)	0	2.4	4.8	7.2	AS	0.0	2.3	4.6	7.0	9.3	AS	0.0	2.8	5.6	8.5	AS
pH [H+]	6.82	7.08	7.18	7.21	7.22	6.85	6.90	6.99	7.04	7.04	6.70	6.70	6.70	6.80	6.80	6.80	6.90	6.90	6.80	6.80	6.80
Temperature (Degree C)	11.5	12.0	12.0	12.1	12.0	14.8	15.3	15.3	15.2	15.1	13.5	13.3	13.4	13.6	13.3	13.4	15.1	15.2	15.0	15.0	15.0
Electrical Conductivity (umhos/cm)	330	362	375	382	380	435	475	295	298	298	-- (c)	--	--	620	600	600	500	510	510	510	510
October 17, 1994 Groundwater Sampling Event																					
Depth to water	20.76					11.36					19.54						10.42				
Target Purge Volume in Gallons (a)	6.9					9.1					6.1						11.1				
Gallons removed during purging	0	2.3	4.6	6.9	AS	0	3.2	6.4	9.6	AS	0	2.0	4.0	6.0	AS		0	4.2	8.0	12.0	13.0
pH [H+]	6.46	7.38	7.32	7.67	7.73	7.48	7.00	6.92	6.93	6.85	6.91	6.81	6.82	6.80	6.81		7.34	6.91	6.83	6.84	6.87
Temperature (Degree C)	12.3	12.2	12.2	12.4	12.4	13.9	13.1	13.0	13.0	13.0	14.1	14.2	14.2	14.2	14.3		14.8	14.0	14.0	13.9	13.9
Electrical Conductivity (umhos/cm)	--	--	--	--	465	483	600	595	595	595	600	605	600	605	605		550	520	520	520	520

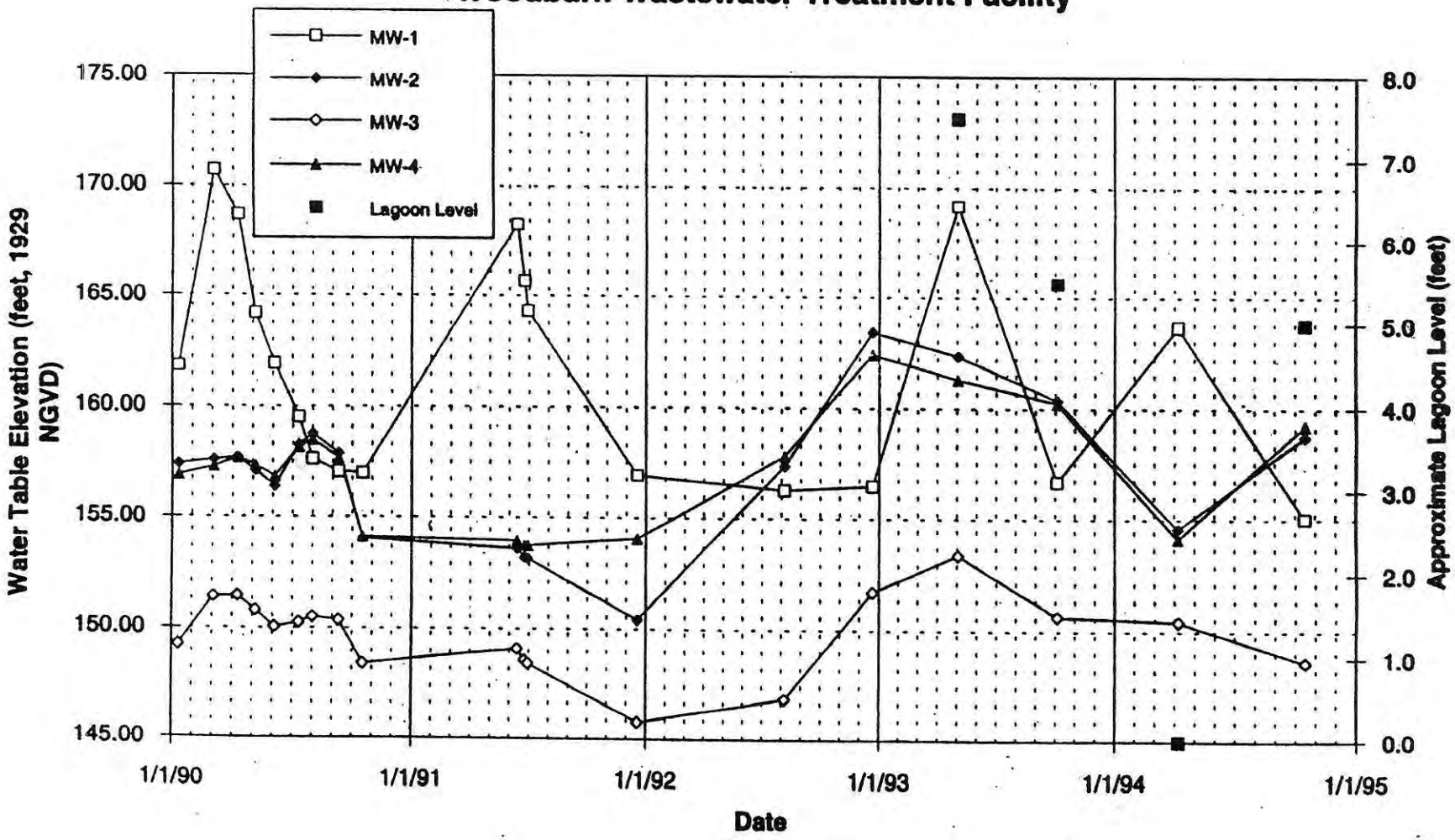
(a) Target Purge volume calculation (gallons): (Water column Height * 0.163 * 3)
(b) AS = After Sample collected.
(c) --- = not measured.

Table 3
Water Level Elevations
Monthly and Semiannual Monitoring Data
City of Woodburn Wastewater Treatment Facility

Date	MW-1 Water Level Elevation (ft)	MW-2 Water Level Elevation (ft)	MW-3 Water Level Elevation (ft)	MW-4 Water Level Elevation (ft)	Approximate Lagoon Level (ft)
MPE (a)	175.80	170.11	168.14	169.68	
11/30/89	156.62	155.29	147.69	155.86	
01/11/90	161.80	157.41	149.22	156.90	
02/09/90	NR	NR	NR	NR	
03/05/90	170.69	157.60	151.38	157.29	
04/10/90	168.67	157.71	151.42	157.70	
05/07/90	164.17	157.11	150.77	157.36	
06/05/90	161.90	156.39	150.04	156.84	
07/12/90	159.50	158.26	150.23	158.19	
08/02/90	157.64	158.77	150.48	158.51	
09/11/90	157.07	157.89	150.34	157.68	
10/18/90	157.00	154.00	148.40	154.07	
06/14/91	168.24	153.56	149.09	153.93	
06/26/91	165.67	153.16	148.57	153.71	
07/01/91	164.30	153.11	148.43	153.71	
12/19/91	156.95	150.41	145.78	154.04	
08/06/92	156.29	157.39	146.85	157.86	
12/22/92	156.49	163.39	151.70	162.39	
05/04/93	169.17	162.33	153.35	161.27	7.5 (b)
10/05/93	156.70	160.36	150.64	160.23	5.5
04/08/94	163.65	154.57	150.44	154.11	0 (c)
10/17/94	155.04	158.75	148.60	159.26	5.0

(a) MPE = Measuring Point Elevation (NGVD, Top of Casing)
(b) 7.5 feet is the approximate Lagoon level maximum.
(c) Lagoon empty for last 3 weeks for maintenance.
Notes:
Water level elevations presented in feet relative to the 1929 National Geodetic Vertical Datum.
Monthly data for February 1990 were not obtained.

Figure 2
Water Table Elevations
Woodburn Wastewater Treatment Facility



level (5.0 feet). Water levels slightly declined at MW-3 (1.84 feet) in contrast to the steeper decline at MW-1 (8.61 feet).

Groundwater Flow

In general, the shallow groundwater flow direction at the site is southeast toward the floodplain of the Pudding River. Long-term measurements at site monitoring wells indicate that groundwater levels decline in response to seasonal dry periods or drought years, and rise in response to recharge during the rainy season. As observed in Figure 2, groundwater levels in MW-1 appear to respond to variations in precipitation recharge to a greater degree than groundwater levels in MW-2, -3, and -4, which may suggest that groundwater levels near the lagoon are more stable, possibly because of groundwater mounding near the lagoon. Localized groundwater mounding in the vicinity of the lagoon may accentuate the appearance or potential for flow reversal because the lagoon may recharge the adjacent downgradient wells, especially MW-2 and MW-4. This temporary artificial rise in water levels at the southeast edge of the lagoon during dry season conditions and the lack of appreciable recharge to MW-1 cause this flow reversal effect. This situation is anticipated to be temporary and localized in the immediate vicinity of the lagoon. The net groundwater flow direction beneath the site is believed to be southeast toward discharge points on the floodplain of the Pudding River.

Discussion of Water Quality

The 1994 semiannual analytical results shown in Table 1 indicate that nitrate concentrations in Wells MW-2, MW-3, and MW-4 were below the detection limit of 0.10 mg/L. Nitrate concentrations in upgradient Well MW-1 were reported to be 21 mg/L in April 1994 and 21 mg/L in October 1994. These levels are consistent with elevated nitrate concentrations reported for this well since the April 1990 groundwater sampling event. In general, nitrate concentrations have declined since April 1990, but have remained at a relatively consistent level during field sampling between 1992 and 1994.

As reported in the January 29, 1991, letter to DEQ, Monitoring Well MW-1 is located at the upgradient property corner of the wastewater treatment facility; therefore, the source of elevated nitrate concentrations is most likely upgradient of the facility. 1994 analytical laboratory reports are presented in the appendix.

Appendix

Groundwater Sampling Results

April 8, 1994

October 17, 1994

PACIFIC
ENVIRONMENTAL
LABORATORY INC.

April 14, 1994

CH2M Hill
825 N.E. Multnomah
Suite 1300
Portland, OR 97232

Attention: Patty O'Conner

RE: JOB # OPE 27874.B1
P.O.#
PROJECT - WOODBURN WWTP

Enclosed are test results for your samples received in this lab on Apr. 08, 1994. For your reference, these analyses have been assigned our PEL # 94-0874.

Solid samples are reported on a dry weight basis except for Oregon DEQ Fuels Methods and where otherwise noted.

Please call if you have any questions.

Respectfully,

Howard Holmes
Project Manager

Howard Boorse
QA Manager

cc: Frank Sinclair
City of Woodburn

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9405 S.W. Nimbus Ave. ■ Beaverton, OR 97005 ■ (503) 644-0660 ■ FAX # (503) 644-2202

PACIFIC ENVIRONMENTAL LABORATORY INC.

Nitrate-Nitrogen per Standard Methods 4500-NO3 D.
Results In mg/L (ppm)

Client: CH2M Hill
Project: WOODBURN WWTP
Received: 04/08/1994

PEL Number: 94-0874
Matrix: water

Sample Name	Analyte	Result	MRL	Date Prepped	Date Analyzed
WBMW1-4/94	Nitrate-N	18	0.10	04/10/94	04/10/94
WBMW2-4/94	Nitrate-N	ND	0.10	04/10/94	04/10/94
WBMW3-4/94	Nitrate-N	ND	0.10	04/10/94	04/10/94
WBMW4-4/94	Nitrate-N	ND	0.10	04/10/94	04/10/94
WBMW11-4/94	Nitrate-N	21	0.10	04/10/94	04/10/94
Method Blank	Nitrate-N	ND	0.10		

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MRL Method Reporting Level
ND None Detected at or above the method reporting level
* See Comment Section at end of report

PACIFIC
ENVIRONMENTAL
LABORATORY INC.

April 14, 1994

CH2M Hill
825 N.E. Multnomah
Suite 1300
Portland OR 97232

Attn: Patty O'Conner

Re: Requested Quality Control Data

Enclosed is the in-house quality control data you requested on PEL project number 94-0874.

Note: Method blank results and Surrogate Recoveries are included in the final report.

QUALITY CONTROL DEFINITIONS

METHOD BLANK RESULTS

A Method Blank is a laboratory-generated sample which assesses the degree to which laboratory operations and procedures cause false-positive analytical results for your sample.

No target parameters were detected in the method blank associated with your sample at the reporting limit levels noted on the data sheets in the final report.

SURROGATE STANDARD

A surrogate standard (i.e., a chemical compound not expected to occur in an environmental sample) is added to each sample, blank, and matrix spike sample just prior to extraction or processing. The recovery of the surrogate standard is used to monitor for unusual matrix effects, gross sample processing errors, etc. Surrogate recovery is evaluated for acceptance by determining whether the measured concentration falls within accepted limits.

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9405 S.W. Nimbus Ave. ■ Beaverton, OR 97005 ■ (503) 644-0660 ■ FAX # (503) 644-2202

PACIFIC
ENVIRONMENTAL
LABORATORY INC.

Accuracy is measured by percent recovery as in:

$$\% \text{ Recovery} = \frac{\text{(Measured Concentration)}}{\text{(Actual Concentration)}} \times 100$$

Precision is measured using duplicate tests by relative percent difference.

$$\text{RPD} = \frac{\text{(Result of Test 1 - Result of Test 2)}}{\text{(Result of Test 1 + Result of Test 2)/2}} \times 100$$

If you should have any questions concerning this report, please contact me.

Sincerely,

Howard Boorse
QA/QC Manager

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9405 S.W. Nimbus Ave. ■ Beaverton, OR 97005 ■ (503) 644-0660 ■ FAX # (503) 644-2202

METHOD BLANK RESULTS FOR NITRATE-NITROGEN PER EPA 300.0
RESULTS IN MG/L (PPM)

Analyte	Method Blank CZ94001a	Detection Limit
Nitrate-N	ND	0.10
Date Prepped:	04/10/94	
Date Analyzed:	04/10/94	

BATCH DUPLICATE RESULTS FOR NITRATE-NITROGEN PER EPA 300.0
RESULTS IN MG/L (PPM)

Analyte	Sample Conc	Dup Conc	RPD	QC Limit RPD
Nitrate-N	21	19	10	20

BATCH MATRIX SPIKE RESULTS FOR NITRATE-NITROGEN PER EPA 300.0
RESULTS IN MG/L (PPM)

Analyte	Spike Added	Sample Conc	MS Conc	MS %Rec	QC Limit %Rec
Nitrate-N(ai)	10	21	28	70	75-125

(ai) Spike recovery is out of control limits.

9405 S.W. Nimbus Ave. ■ Beaverton, OR 97005 ■ (503) 644-0660 ■ FAX # (503) 644-2202

PA IC ENVIRONMENTAL LABORATORY, INC.
9405 S.W. Nimbus Ave.
Beaverton, OR 97005
(503) 644-0660
Fax (503) 644-2202

CHAIN OF CUSTODY RECORD

PEL # 94-0874
Page 1 of 1

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ny: CH2M Hill /PDX
Manager: Patty O'Conner
Name: Woodburn WWTP
Number: OPE 27874.B1

Final Report and faxes to: (if other than Project Manager)

Send Copy of report to:
~~Invoice to: (If other than Project Manager)~~
City of Woodburn, Frank Sinclair
2815 Mollalla Road
Woodburn, OR 97071

P.O. Number:

RUSH: ☐ YES ☒ NO

MENTS:

Sample Requirements: SDWA (drinking water) ☐ NPDES (storm water) ☐ RCRA ☐
Other

IF SAMPLE IS LIQUID & HAS SEDIMENT OR PARTICULATE, SHALL WE:
___ Test Filtrate Only?
___ Mix Sample by Shaking?
___ Test Particulate Only?

IF SAMPLE IS MULTI-LAYER, SHALL WE:
___ Test Each Layer separately?
___ Test only ONE Layer? Which Layer?
___ Mix All Layers by Shaking?

QC Information (check one) ☐ Level 1 ☐ Level 2 ☐ Level 3 (10% Surcharge)

PEL NO.	SAMPLE I.D. (Please print clearly to ensure accurate report.)	DATE	TIME	PRESERV.	MATRIX: SOIL	MATRIX: WATER	MATRIX: OTHER	NUMBER OF CONTAINERS	TPH - HCID STATE:	QUANT. IF DETECTED (✓)	TPH-Gasoline STATE:	TPH - Diesel STATE:	TPH - 418.1 Modified STATE:	8015M - Gas	8015M - Diesel	TPH 418.1	BTEX 602/8020	Halogenated Volatiles 601/8010	Volatiles (circle one) 624/8240/8260/524.2	Semivolatiles 625/8270	PCB's 608/8080	Chlorinated Pesticides 608/8080	TCLP (8) Metals	EPA 300.0 µ, Nitrate As N						REMARKS
	WBMW1-4/94	4/8/94	12:20	NONE		X		1																X							500ml Poly
	WBMW2-4/94	↓	13:20	↓		X		1																X							↓
	WBMW3-4/94	↓	14:15	↓		X		1																X							↓
	WBMW4-4/94	↓	15:10	↓		X		1																X							↓
	WBMW11-4/94	↓	11:00	↓		X		1																X							↓

COLLECTED BY:	RELINQUISHED BY:	COMPANY:	DATE/TIME	RECEIVED BY:	COMPANY:
Mike Abbott	Mike Abbott	CH2M Hill	4/8/94 1645	Judith Reynolds	PEL 4-8-94 1645
COOLER #	RELINQUISHED BY:	COMPANY:	DATE/TIME	RECEIVED BY:	COMPANY:
SEAL #	RELINQUISHED BY:	COMPANY:	DATE/TIME	RECEIVED BY:	COMPANY:

NORTH CREEK ANALYTICAL

18939 120th Avenue N.E., Suite 101 • Bothell, WA 98011-9508 (206) 481-9200 • FAX 485-2992
East 11115 Montgomery, Suite B • Spokane, WA 99206-4776 (509) 924-9200 • FAX 924-9290
9405 S.W. Nimbus Avenue • Beaverton, OR 97008-7132 (503) 643-9200 • FAX 644-2202

October 25, 1994

CH2M Hill
825 N.E. Multnomah
Suite 1300
Portland, OR 97232

Attention: Patty O'Connor

RE: JOB # OPE 27874.B1
P.O.#
PROJECT - WOODBURN WWTP

Enclosed are test results for your samples received in this lab on Oct. 17, 1994. For your reference, these analyses have been assigned our NCA # 94-3100.

Solid samples are reported on a dry weight basis except for Oregon DEQ Fuels Methods and where otherwise noted.

Please call if you have any questions.

Respectfully,

Howard Holmes
Project Manager

cc: Frank Sinclair
City of Woodburn

NORTH CREEK ANALYTICAL
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9405 S.W. Nimbus Avenue • Beaverton, OR 97008-7132 (503) 643-9200 • FAX 644-2202

Nitrate-Nitrogen per EPA 300.0
Results in mg/L (ppm)

Client: CH2M Hill
Project: WOODBURN WWTP
Received: 10/17/1994

NCA Number: 94-3100
Matrix: water

Sample Name	Analyte	Result	MRL	Date Prepped	Date Analyzed
WBMW1-10/17/94	Nitrate-N	21	0.10	10/19/94	10/19/94
WBMW2-10/17/94	Nitrate-N	ND	0.10	10/19/94	10/19/94
WBMW3-10/17/94	Nitrate-N	ND	0.10	10/19/94	10/19/94
WBMW4-10/17/94	Nitrate-N	ND	0.10	10/19/94	10/19/94
WBMW11-10/17/94	Nitrate-N	21	0.10	10/19/94	10/19/94
Method Blank	Nitrate-N	ND	0.10		

MRL Method Reporting Level
ND None Detected at or above the method reporting level
* See Comment Section at end of report

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NORTH CREEK ANALYTICAL

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9405 S.W. Nimbus Avenue • Beaverton, OR 97008-7132 (503) 643-9200 • FAX 644-2202

October 26, 1994

CH2M Hill
825 N.E. Multnomah
Suite 1300
Portland, OR 97232

Attention: Patty O'Connor

Re: Requested Quality Control Data

Enclosed is the in-house quality control data you requested on NCA project number 94-3100.

Note: Method blank results and Surrogate Recoveries are included in the final report.

QUALITY CONTROL DEFINITIONS

METHOD BLANK RESULTS

A Method Blank is a laboratory-generated sample which assesses the degree to which laboratory operations and procedures cause false-positive analytical results for your sample.

No target parameters were detected in the method blank associated with your sample at the reporting limit levels noted on the data sheets in the final report.

SURROGATE STANDARD

A surrogate standard (i.e., a chemical compound not expected to occur in an environmental sample) is added to each sample, blank, and matrix spike sample just prior to extraction or processing. The recovery of the surrogate standard is used to monitor for unusual matrix effects, gross sample processing errors, etc. Surrogate recovery is evaluated for acceptance by determining whether the measured concentration falls within accepted limits.

NORTH CREEK ANALYTICAL

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9405 S.W. Nimbus Avenue • Beaverton, OR 97008-7132 (503) 643-9200 • FAX 644-2202

Accuracy is measured by percent recovery as in:

$$\% \text{ Recovery} = \frac{\text{(Measured Concentration)}}{\text{(Actual Concentration)}} \times 100$$

Precision is measured using duplicate tests by relative percent difference.

$$\text{RPD} = \frac{\text{(Result of Test 1 - Result of Test 2)}}{\text{(Result of Test 1 + Result of Test 2)/2}} \times 100$$

If you should have any questions concerning this report, please contact me.

Sincerely,

Howard Holmes
Project Manager

NORTH CREEK ANALYTICAL
18939 120th Avenue N.E., Suite 101 • Bothell, WA 98011-9508 (206) 481-9200 • FAX 485-2992
East 11115 Montgomery, Suite B • Spokane, WA 99206-4776 (509) 924-9200 • FAX 924-9290
9405 S.W. Nimbus Avenue • Beaverton, OR 97008-7132 (503) 643-9200 • FAX 644-2202

BATCH QUALITY CONTROL RESULTS
Nitrate-Nitrogen per EPA 300.0

Client: CH2M Hill
Project: WOODBURN WWTP

NCA Number: 94-3100
Received: 10/17/1994

METHOD BLANK
Batch # CK94037a
Results in mg/L (ppm)

Compound	Result	MRL
Nitrate-N	ND	0.10
Date Prepped	10/19/1994	
Date Analyzed	10/19/1994	

DUPLICATE
Batch # CK94037a
Results in mg/L (ppm)

Compound	Sample Conc	Dup Conc	RPD	QC Limit RPD
Nitrate-N	21	21	0	20

MATRIX SPIKE
Batch # CK94037a
Results in mg/L (ppm)

Compound	Spike Added	Sample Conc	MS Conc	MS % Rec	QC Limit % Rec
Nitrate-N	30	21	50	97	75-125

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NORTH CREEK ANALYTICAL
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9405 S.W. Nimbus Avenue • Beaverton, OR 97008-7132 (503) 643-9200 • FAX 644-2202

BATCH QUALITY CONTROL RESULTS
Nitrate-Nitrogen per EPA 300.0

Client: CH2M Hill
Project: WOODBURN WWTP

NCA Number: 94-3100
Received: 10/17/1994

LABORATORY CONTROL SAMPLE
Batch # CK94037a
Results in mg/L (ppm)

Compound	True	Found	% Rec	QC Limit % Rec
Nitrate-N	0.20	0.18	90	90-110

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NORTH CREEK ANALYTICAL

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9405 S.W. Nimbus Avenue • Beaverton, OR 97008-7132 (503) 643-9200 • FAX 644-2202

COPY

INVOICE FOR ANALYTICAL SERVICES

CH2M Hill
825 NE Multnomah, #1300
Portland, OR 97232

Attention: Accounts Payable

Project Manager: Patty O'Connor

Project: WOODBURN WWTP

Invoice Number: 24100260

Invoice Date: 10/26/94

#OPE 27874.B1

Water, Received 10/17/94

NCA Sample #: 94-3100

QTY	ANALYSES PERFORMED	UNIT PRICE	NET AMOUNT
5	Nitrate-N	20.00	100.00

Invoice Total $ 100.00

Remit To: North Creek Analytical
18939 - 120th Ave. NE #101
Bothell, WA 98011-9508

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TERMS: NET 30 DAYS
PAST DUE INVOICES ARE SUBJECT TO A 1.5% PER MONTH SERVICE CHARGE

ONMENTAL LABORATORY, INC.
9405 S.W. Nimbus Ave.
Beaverton, OR 97005
(503) 644-0660
Fax (503) 644-2202

CHA... OF CUSTODY RECORD

PEL # 44-3400

Page 1 of 1

Company: CH2M Hill / PDX
Project Manager: Patty O'Connor
Project Name: Woodburn WWTP
Project Number: OPE 27874.B1

Final Report and faxes to: (if other than Project Manager)

Send Copy of Report to:
Invoice to: (if other than Project Manager)
City of Woodburn, Frank Sinclair
2815 Molalla Rd.
Woodburn, OR 97071

P.O. Number:

RUSH: ☐ YES ☒ NO

COMMENTS:

Sample Requirements:
SDWA (drinking water) ☐ NPDES (storm water) ☐ RCRA ☐
Other _____

IF SAMPLE IS LIQUID & HAS SEDIMENT OR PARTICULATE, SHALL WE:
____ Test Filtrate Only?
____ Mix Sample by Shaking?
____ Test Particulate Only?

IF SAMPLE IS MULTI-LAYER, SHALL WE:
____ Test Each Layer separately?
____ Test only ONE Layer? Which Layer?
____ Mix All Layers by Shaking?

QC Information (check one)
☐ Level 1 ☐ Level 2 ☐ Level 3 (10% Surcharge)

PEL NO.	SAMPLE I.D. (Please print clearly to ensure accurate report.)	DATE	TIME	PRESERV.	MATRIX: SOIL	MATRIX: WATER	MATRIX: OTHER	NUMBER OF CONTAINERS	TPH - HCID STATE:	QUANT. IF DETECTED (✓)	TPH-Gasoline STATE:	TPH - Diesel STATE:	TPH - 418.1 Modified STATE:	8015M - Gas	8015M - Diesel	TPH 418.1	BTEX 602/8020	Halogenated Volatiles 601/8010	Volatiles (circle one) 624/8240/8260/524.2	Semivolatiles 625/8270	PCB's 608/8080	Chlorinated Pesticides 608/8080	TCLP (8) Metals	EPA 300.0	Nitrate as N						REMARKS
1	WBMW1-10/17/94	10/17/94	1015	Ø		✓		1																1							
2	WBMW2-10/17/94	10/17/94	1145	Ø		✓																		1							
3	WBMW3-10/17/94	10/17/94	1230	Ø		✓																		1							
4	WBMW4-10/17/94	10/17/94	1320	Ø		✓																		1							
5	WBMW11-10/17/94	10/17/94	0900	Ø		✓		1																1							

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COLLECTED BY:	RELINQUISHED BY:	COMPANY:	DATE/TIME	RECEIVED BY:	COMPANY:
Mike Abbott	Mike Abbott	CH2M Hill	10/17/94 1457	[illegible]	NCA
COOLER #	RELINQUISHED BY:	COMPANY:	DATE/TIME	RECEIVED BY:	COMPANY:
SEAL #	RELINQUISHED BY:	COMPANY:	DATE/TIME	RECEIVED BY:	COMPANY:

NOTE: Samples are disposed of 30 days after report is mailed unless other arrangements are made. Hazardous samples will be returned to client or disposed at client's expense.

Section 9
Pubic Involvement

Introduction

In this section, the public involvement activities for the Woodburn Wastewater Treatment Plant (WWTP) facilities planning are described.

Public Involvement Activities

The first public meeting concerning the Woodburn WWTP facilities plan was held on May 6, 1991. The meeting was held to present the evaluation of several options for the Woodburn wastewater treatment facilities and solicit input on the options. Copies of the mailing list and handouts are attached in Appendix 9.1.

A Woodburn Wastewater Advisory Committee (WAC) was formed in August 1993. The WAC has met monthly to discuss the wastewater issues facing the community and to give staff guidance on the direction the planning should go. The WAC reviewed the planning criteria. The consensus of the group was to plan the wastewater facilities to the year 2020 and to plan for residential and commercial growth of 3.4 percent. An industrial growth rate of 1 percent was agreed to by the WAC by consensus.

The WAC selected Barbara Lucas, a volunteer, to head up the public involvement and publicity.

The WAC also reviewed the Big Picture Alternatives for wastewater treatment and effluent disposal (see Appendix 5.1). The Big Picture Alternatives were again brought to the public in an open house format. The open house, held June 6, 1994, was a huge success, bringing in over 100 people to discuss the wastewater issues and alternatives in Woodburn (handouts are attached in Appendix 9.2). After hearing the public input, the consensus of the WAC was to expand at the existing plant site, which was the least expensive of the Big Picture Alternatives analyzed. The public and WAC also wanted consideration of phasing the selected alternative.

The selected alternative treatment plant components were reviewed by the WAC. The selected alternative received WAC consensus. Phasing of the selected alternative was also favored by the WAC. The effluent disposal option of poplar tree irrigation was highly promoted by the WAC.

A WAC council meeting, which was open to the public, was held on the preliminary rate analysis.

The progress of the facilities planning has been reported in the local newspapers and direct mailings.

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A second open house is planned for March 21, 1995. The selected alternative will be presented along with projected rate increases to cover the next 5-years of operation. A final public hearing is planned to present the final facilities plan which will include comments received from the public open house.

Appendix 9.1
First Public Meeting Mailing List and Handouts

PLANNING FOR WOODBURN'S WASTEWATER TREATMENT FACILITIES

May 1991

INTRODUCTION

The City of Woodburn is preparing a facility plan for its wastewater treatment system. A facility plan defines what steps the City should take to meet its future needs for wastewater treatment. It is the result of a planning process that looks at many possibilities, considers the advantages and disadvantages of each, and identifies the system that can most benefit the community. Once the plan is adopted by the City, it will guide the operation and improvements to the City's treatment system through the year 2020.

Improvements to the city's system are needed to meet new water quality regulations established by the U.S. Environmental Protection Agency (EPA) and the Oregon Department of Environmental Quality (DEQ). These regulations designate wastewater treatment and discharge standards thathat must be met to protect the Pudding River and other bodies of water that receive treated wastewater. The city's current system is not able to meet these stringent new standards.

In addition to upgrading the existing system to meet regulatory requirements, the facility plan will provide for increasing the system's capacity in order to accommodate planned growth in the city. Preparing now for both of these purposes is an efficient and cost-effective planning approach. Additional efficiency is built into the plan by providing for phased construction of the improvements. The plan will enable the city to look ahead to long-term needs through the year 2020, while implementing the improvements only as they are needed.

The City of Woodburn is committed to providing its citizens with a system that is environmentally responsible and cost-effective. Public review and comment is an essential part of the planning process. A number of possible systems are being considered for meeting the City's treatment needs. Optional new technology and mitigation measures can also provide additional advantages, such as reduced odor and noise impacts in the plant vicinity. All of these alternatives have costs and benefits that must be carefully weighed. We welcome your participation in making these decisions.

THE STUDY PROCESS AND SCHEDULE

The City of Woodburn has hired CH2M Hill, an engineering and environmental consulting firm, to help prepare the facility plan. The planning process has the following basic steps:

- Define future wastewater treatment needs, based on population and wastewater projections and on regulatory requirements.
- Identify alternative treatment systems that can meet these needs.
- Evaluate the alternatives on the basis of cost, environmental impacts, ability to construct and operate, and other advantages/disadvantages.

- The cost of this option is estimated at from $26 to $41 million, depending on the amount of industrial flow that is treated. The wetlands option would cost about $2 million more than the filter system, largely because of the cost of land acquisition. The unknown costs associated with groundwater quality regulations could make the cost of the wetlands considerably higher.

  As a point of comparison, the City of McMinnville is improving its facilities at a cost of $27 million, and sewer rates are expected to increase by about $20-25/month after construction is completed.

Option 2: Continued Use of Existing Facility, with New Facility Located Southeast of Existing Plant

This option is essentially the same as Option 1, with the following differences:

- The new facility will be constructed at a separate location, southeast of the existing plant. The new plant will therefore be closer to most industrial wastewater sources, but will require longer pumping distances for excess domestic wastewater from the existing plant.

- No effluent from the new facility will be discharged to the Pudding River. Following secondary treatment, all effluent will be discharged to create a new wetlands area south of the urban growth boundary. The wetlands will discharge either into the Pudding River or into Mill Creek south of the city, enhancing the portion of Mill Creek that flows through the city. Again, the ability to construct wetlands that would meet groundwater quality regulations is unknown at this time.

- The cost of this option is estimated at from $30 to $44 million, depending on the amount of industrial flow that is treated. As in Option 1, costs for the wetlands could be considerably higher.

Comparison of Option 1 and Option 2

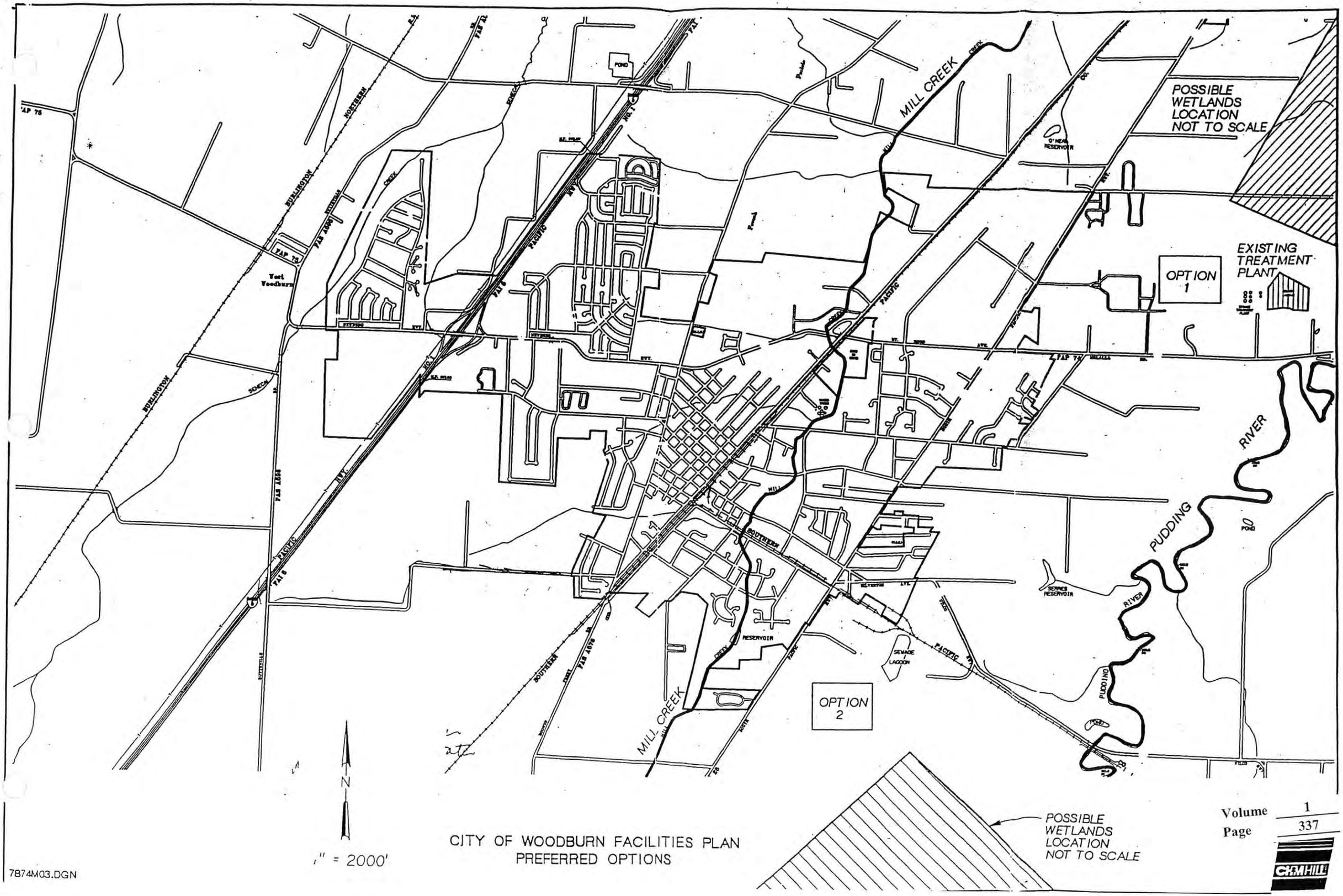
Compared with each other, Options 1 and 2 have the following advantages:

Option 1: Greater operational and maintenance flexibility
Easier land acquisition
Closer to solids handling facilities
Less pumping distance for excess domestic loads

Option 2: Closer to industrial wastewater sources
If effluent is discharged to Mill Creek, enhances the portion of Mill Creek that flows through the city

OTHER OPTIONS CONSIDERED

A number of other alternatives have also been examined, and are considered less advantageous than Preferred Options 1 and 2. These include various combinations of treatment, storage, and discharge systems, and can be grouped under five general categories. These categories and the reasons they compare unfavorably with the preferred options are summarized below.



CITY OF WOODBURN FACILITIES PLAN
PREFERRED OPTIONS

7874M03.DGN

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CH2M HILL

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MEMORANDUM

To: Frank Tiwari, Woodburn Public Works Director

From: Nancy Jerrick, Oman/Jerrick Associates Nancy

Date: April 18, 1991

Subject: Information for May 6 Public Meeting

I am enclosing the following items:

- A copy of the display advertisement that will be run in the Woodburn Independent on April 24 and the Senior Estates newspaper on April 26
- A copy of the project mailing list
- The flyer to be sent/distributed to the mailing list
- A set of mailing labels for the flyer

I will be out of the office Friday, April 19, and Monday, April 22. Please call me on April 23 if you have any questions.

✓cc: Daria Wightman, CH2M Hill

You are invited to a

PUBLIC MEETING

for the

Woodburn Wastewater Treatment Plant Facility Plan

MONDAY, MAY 6, 1991
7:00-9:00 pm
City Council Chambers
270 Montgomery Street, Woodburn, Oregon

The City of Woodburn is developing a long-term plan for its wastewater treatment facilities. Improvements to the existing system are needed to meet new and proposed state and federal water quality regulations and to accommodate planned growth in the City at the same time. Your City is committed to providing its citizens with a system that is environmentally responsible and cost-effective.

The City is evaluating several options that can meet its future wastewater treatment needs. Based on the results of this evaluation, your elected representatives will select the best alternatives that are being considered.

Public review and comment is an important part of the evaluation process. The purpose of this public meeting is to:

- Discuss the alternatives we are considering
- Hear about the issues that are of concern to you and receive your comments about an important community decision.

We invite you to attend this meeting and give us your ideas.
For more information, please call Frank Sinclair, Treatment Plant Superintendent, at 982-5281.

You are invited to a

PUBLIC MEETING

for the

Woodburn Wastewater Treatment Plant Facility Plan

MONDAY, MAY 6, 1991
7:00-9:00 pm
City Council Chambers
270 Montgomery Street, Woodburn, Oregon

The City of Woodburn is developing a long-term plan for its wastewater treatment facilities. Improvements to the existing system are needed to meet new and proposed state and federal water quality regulations and to accommodate planned growth in the City at the same time. Your City is committed to providing its citizens with a system that is environmentally responsible and cost-effective.

The City is evaluating several options that can meet its future wastewater treatment needs. Based on the results of this evaluation, your elected representatives will select the best alternatives that are being considered.

Public review and comment is an important part of the evaluation process. The purpose of this public meeting is to:

- Discuss the alternatives we are considering
- Hear about the issues that are of concern to you and receive your comments about an important community decision.

We invite you to attend this meeting and give us your ideas.
For more information, please call Frank Sinclair, Treatment Plant Superintendent, at 982-5281.

WOODBURN WASTEWATER TREATMENT PLANT FACILITY PLAN

MAILING LIST

April 1991

PROJECT STAFF

Frank Tiwari
Public Works Director
City of Woodburn
270 Montgomery
Woodburn, OR 97071

Daria Wightman
CH2M Hill
2020 S.W. Fourth Avenue
Second Floor
Portland, OR 97201

Nancy Jerrick
Oman/Jerrick Associates
1606 S.E. 34th Avenue
Portland, OR 97214

CITY OF WOODBURN

Steve Goeckritz
Director of Planning
City of Woodburn
270 Montgomery
Woodburn, OR 97071

Woodburn Planning Commission
c/o Steve Goeckritz
Director of Planning
City of Woodburn
270 Montgomery
Woodburn, OR 97071

Woodburn City Council
c/o Mike Quinn
Woodburn City Administrator
270 Montgomery
Woodburn, OR 97071

Dan Glennon
President
Woodburn Area Chamber of Commerce
P.O. Box 194
Woodburn, Oregon 97071

CITY OF WOODBURN (Continued)

Charles Piper
Woodburn Rotary Club
P.O. Box 247
Woodburn, OR 97071

Cliff Zauner
French Prairie Kiwanis Club
c/o KWBY Radio
P.O. Box 158
Woodburn, OR 97071

Mike Padberg
Woodburn Kiwanis Club
11798 Carl Road N.E.
Woodburn, OR 97071

George Tallmadge, President
Senior Estates
1776 Country Club Road
Woodburn, OR 97071

MEDIA

KWBY Radio
P.O. Box 158
Woodburn, OR 97071

Woodburn Independent
601 North First
Woodburn, OR 97071

ODOR COMPLAINTS

Mrs. Heer
12565 Carl Road
Woodburn, OR 97071

Nancy Glatt
2855 Molalla Road
Woodburn, OR 97071

John MacNulty
12664 Carl Road
Woodburn, OR 97071

Fred Kascher
985 Kelowna Ct.
Woodburn, OR 97071

R.L. Koenig
2775 Molalla Road
Woodburn, OR 97071

ODOR COMPLAINTS (Continued)

Herb Koenig
2705 Molalla Road
Woodburn, OR 97071

Sterling Brady
3599 Linda
Woodburn, OR 97071

Marg Lund
2645 Molalla Road
Woodburn, OR 97071

NEIGHBORS

(hand delivered)

SLUDGE COMPLAINTS

(hand delivered)

TRAILER COURT

(hand delivered)

INDUSTRIES

Pacific Rim Products
1660 Silverton Road
Woodburn, OR 97071
Attn: Scott Eagle

Specialty Polymer
2475 Progress Way
Woodburn, OR 97071
Attn: Raymond Southwell

J.M. Smucker Co.
1440 Silverton Road
Woodburn, OR 97071
Attn: Marlin Icenogle

Conroy Packing
Division of Kerr Industries
960 Young Street
Woodburn, OR 97071
Attn: Brian Conroy

Agripac Corporation
North Pacific Highway
Woodburn, OR 97071
Attn: D'Mark Mick

OTHER JURISDICTIONS

City of Molalla
Public Works Department
12424 S. Toliver Road
Molalla, OR 97038
Attn: Jack Dunn

City of Molalla
Public Works Department
12424 S. Toliver Road
Molalla, OR 97038
Attn: Don Kemp

City of Mt. Angel
Public Works Department
P.O. Box 960
Mt. Angel, OR 97362
Attn: Corky Yarbrough

City of Gervais
P.O. Box 348
Gervais, OR 97026

City of Hubbard
P.O. Box 380
Hubbard, OR 97032
Attn: Jerry Ordin

City of Silverton
830 McClaine Street
Silverton, OR 97381
Attn: Richard Barstad

City of Silverton
830 McClaine Street
Silverton, OR 97381
Attn: Steve Yoder

City of Aurora
P.O. Box 100
Aurora, OR 97002
Attn: Dick Johnston

City of St. Paul
P.O. Box 7
St. Paul, OR 97137
Attn: Rod Bowdish

City of Donald
P.O. Box 388
Donald, OR 97020
Attn: Linda Langfeldt, City Recorder

Appendix 9.2
Handouts from June 6, 1994, Open House

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CITY OF WOODBURN

270 Montgomery Street • Woodburn, Oregon 97071 • (503) 982-5222
TDD (503) 982-7433 • FAX (503) 982-5244

May 1994

NOTICE TO ALL SEWER SERVICE CUSTOMERS

In November 1993 the City was notified by the U.S. Environmental Protection Agency (EPA) to develop a plan by November 1994 to meet the more stringent pollution limits that have been set for discharging effluent water from the sewage treatment plant into the Pudding River. The volume of water in the Pudding River during the summer (July & August) is so low the river water can't dilute the sewage effluent being discharged and therefore it cannot maintain the oxygen demand of certain aquatic life, as it can during the winter months. This is the main reason the Pudding River has been classified as a water quality limited stream. Also, besides these environmental improvements the present sewage treatment plant is nearing the end of its 20 year useful life and decisions have to be made on where to go from here.

The Wastewater Advisory Committee, appointed by the governing body, having local citizens and community representatives, has evaluated several different alternatives (see other side) on how to handle these problems and preferred one alternative over the others. No decision will be made by the City Council until your input has been received.

Any changes to the treatment plant or the treatment process will mean better protection to the environment and higher monthly sewer bills. How much higher the bills will be won't be known until late summer. A comparison was done with other cities that are experiencing similar problems and an analysis of monthly costs is as follows:

SEWER RATE INFORMATION FROM COMMUNITIES IMPROVING THEIR SEWERAGE SYSTEMS

CITY	PLANT SIZE (Million Gallons Per Day)	APPROX. SEWERAGE COST	1992 SEWER COST PER UNIT	1994 SEWER COST PER UNIT	2005 PROJECTED IN FACILITIES PLAN
McMinnville	6.0 MGD	$ 37,000,000	$ 18.00	$ 23.20*	$ 43.69
Dallas	2.34 MGD	31,000,000	13.00	17.00	49.19
Woodburn	5.0 MGD	???	14.20	14.20	???

* 30% Increase each year for the next two years has already been approved by Council.

The Wastewater Advisory Committee has scheduled the first of two Open Houses for June 6, 1994 from 4:00 to 8:00 p.m. at Woodburn City Hall in the Council Chambers. This open house is a chance for you to receive information regarding the problems we are facing, review exhibits on the different alternatives and to talk to experts about these problems and the possible solutions. At this time you will be able to give committee members your opinions about the alternative that the committee recommends.

OPEN HOUSE
June 6, 1994
4 PM to 8 PM
Woodburn City Hall

William Mitchell, Chairman
Woodburn Wastewater Advisory Committee

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SEWER RATE PAYERS

The City of Woodburn wants your advice on selecting among Treatment Plant Alternatives to protect our environment and serve our community to the year 2020. These alternatives are expected to be expensive.*

Come to an OPEN HOUSE
June 6, 1994
4 PM to 8 PM
Woodburn City Hall

*Comparative cost options are between $47,000,000 to $86,000,000

* * * * *

A LOS USUARIOS DE LAS CAÑERIAS DE DESAGUE

La ciudad de Woodburn quiere obtener su opinión para escoger entre alternativas, que se espera van a ser muy *caras o costosas, con el propósito de proteger nuestro medio ambiente y servir a nuestra communidad hasta el año 2020.

VENGA A UNA PRESENTACION PUBLICA
Junio 6 de 1994
De 4 a 8 PM
Administración de la Ciudad de Woodburn
(Woodburn City Hall)

*El alcance comparativo está entre $47,000,000 hasta $86,000,000

* * * * *

ПЛАТЕЛЬЩИКИ СТОЧНЫХ ТРУБОВ

Город Вудбурна хочет ваш совет на выборе Алтернативы Перерабочевой Фабрики чтоб охранить нашу окружению и служить нашу обществу в 2020 году. Ожидано что эти алтернативы будут дорогие.*

Приходите на "ОТКРЫТОМ ДОМЕ"
6-го Июня, 1994
4:00-8:00 веч.
В Главном Зале
Город Вудбурна

*Сравнительные стоимости между $47,000,000 до $86,000,000

Planning for Woodburn's Wastewater Treatment Facilities

Introduction

The City of Woodburn is preparing the facilities plan for its wastewater treatment system. A facilities plan defines the steps that the City should take to meet its future needs for wastewater treatment. It is the result of a planning process that looks at many possibilities, considers the advantages and disadvantages of each, and identifies the system that can most benefit the community. Once the plan is adopted by the City, it will guide operations and improvements to the City's wastewater treatment system through the year 2020.

Improvements to the City's system are needed to meet new water quality regulations established by the U.S. Environmental Protection Agency (EPA) and the Oregon Department of Environmental Quality (DEQ) and to protect against citizen lawsuits. These regulations designate wastewater treatment and discharge standards that must be met to protect the Pudding River and other bodies of water that receive treated wastewater. The City's current system will complete its 20-year design life in the year 2000 and it will not be able to meet these stringent new standards or the projected growth requirements of the City.

In addition to upgrading the existing system to meet regulatory requirements, the facilities plan will provide for increasing the system's capacity in order to accommodate planned growth in the City. Preparing now for both of these purposes is an efficient and cost-effective planning approach. The plan will enable the City to look ahead to long-term needs through the year 2020.

The City of Woodburn is committed to providing its citizens with a system that is environmentally responsible and cost-effective. Public review and comment is an essential part of the planning process. A number of possible systems are being considered for meeting the City's treatment needs. Optional new technology and mitigation measures can also provide additional advantages, such as reduced odor and noise impacts in the plant vicinity. All of these alternatives have cost and benefits that must be carefully weighed. We welcome your participation in making these decisions.

The Study Process and Schedule

The City of Woodburn has hired CH2M HILL, an engineering and environmental consulting firm, to help prepare the facilities plan. The planning process has the following basic steps:

1. Define future wastewater treatment needs, based on population and wastewater projections and on regulatory requirements.

2. Identify alternative treatment systems that can meet these needs.

3. Evaluate the alternatives on the basis of cost, environmental impacts, ability to construct and operate, and other advantages/disadvantages.

4. Select the best treatment system for more detailed analysis, and develop a facilities plan.

5. Conduct a financial analysis to determine who pays for the improvements and how.

The planning process is currently in the third step: evaluation of the alternatives. All public comments will be considered in selecting the alternatives that will be carried forward for further analysis. The draft facilities plan is anticipated to be available for public review by November 1994.

Recommendations

The Wastewater Advisory Committee evaluated the treatment plant siting alternatives and the tertiary treatment and effluent disposal options (see Figures 1 through 3). The consensus of the committee was to recommend the following big-picture alternative with two disposal options for further development and analysis:

Issue	Alternative
Treatment Plant Site	Alternative I – Upgrade and Expand Existing Plant
Tertiary Treatment	Mechanical Filtration
Effluent Disposal during the Summer	Option A – Discharge to the Pudding River, with 88-acre storage lagoon for July and August
	Option B – Irrigation at Agronomic Application Rates for May through October, with 58-acre storage lagoon and 300 acres irrigable land
Effluent Disposal during the Winter	Discharge to Pudding River

Treatment Plant Site

Upgrading and expanding the existing plant (Alternative I) was the preferred treatment plant siting alternative because of costs and practicality. The other siting alternatives were shown to have many disadvantages. For instance, adding a second, parallel treatment plant to the same site (Alternative II) would require additional pumping for separate treatment of industrial flows. Retaining the existing plant and building a separate plant at a new site (Alternative III) would require new pumping facilities for sludge and industrial flows, a new force main, and another effluent pipeline. Furthermore, operations and maintenance of two sites would be less efficient than for one. Abandonment of the existing plant and building a new plant at a new site (Alternative IV) would mean losing the value of the existing plant and would entail dismantling the existing facilities and restoring the site to farmland at a significant cost. Moreover, the siting of a new plant would likely face public opposition.

Tertiary Treatment

Mechanical filtration was found to be a better tertiary treatment option in this case than wetland treatment. A wetland treatment system would be approximately 50 percent more expensive because of land costs and the cost of a lining for the wetland (which is required by DEQ) without significant added benefit. Uncertainties regarding long-term permitting for control of the wetlands as a treatment facility and concerns about possible effects on the groundwater (City of Woodburn's main drinking water source) also make this a less attractive option.

Mechanical filtration is a more reliable treatment process and the capacity of a filtration facility can be increased more easily. It is estimated to be the least expensive alternative. The operations staff preferred mechanical filtration.

Effluent Disposal during the Summer

Pudding River and Storage. With mechanical filtration provided, the effluent can be discharged to the Pudding River during May, June, September, and October of the summer period. During July and August of each year, when the effluent ammonia requirements preclude discharge to the Pudding River, the effluent would be stored in an 88-acre lagoon.

Irrigation at Agronomic Application Rates. Agricultural irrigation was selected as another viable alternative for disposal of treatment plant effluent during the summer period. This alternative provides nutrient rich water for irrigation in the area at a time when water is becoming a scarce and valuable resource. A 58-acre storage lagoon would be required to store effluent, assuming roughly 300 acres of land are available for irrigation during July and August. Filtration would be required for disposal of excess wastewater to the Pudding River during the other summer months.

Other Alternatives. The other summer disposal alternatives considered—discharge to the Willamette River and a 5-month-capacity storage lagoon—were seen to be too costly.

Effluent Disposal during the Winter

Because the DEQ effluent requirements for the winter period are less stringent, it will be possible to discharge to the Pudding River with minor modifications to the current treatment processes, upgraded and expanded as necessary to replace aging components and meet new capacity needs.

Costs

Cost opinions were developed based on Environmental Protection Agency (EPA) and industry guidelines. The costs were further adjusted to reflect recent local projects such as wastewater facilities for the City of McMinnville, Oregon. They costs are comparative and not to be used for budgetary purposes.

The present worth cost in 1997 dollars for Option A—Summer Effluent Disposal to Pudding River and Storage was estimated to be $53,300,000. For Option B—Summer Effluent Disposal to Irrigation it was estimated to be $55,850,000. These present worth costs are within 15 percent of each other, and, therefore, are not considered by EPA to be significantly different. The non-cost analyses need to be considered in the selection process. The range of costs for all the alternatives was from $55,300,000 to $92,335,000.

Public Review and Comment

The City of Woodburn would like to hear public comments on the alternatives now under consideration to help decide which should be carried forward for further study. An open house will be held for this purpose on June 6, 1994. A Public Hearing will be held on November 15, 1994, to hear public comment on the draft facilities plan. The public will be notified, and additional information about the project will be available before this meeting.

If you would like more information or have any comments about the facilities plan, please call Frank Sinclair, Woodburn Wastewater Treatment Plant Superintendent, at 982-5281.

Recommended by Wastewater Advisory Committee for further evaluation

ALTERNATIVE I	**Upgrade and expand existing plant at existing plant site**
ALTERNATIVE II	Existing site co-location with dual treatment process
ALTERNATIVE III	Existing site and new site; new plant located at new site
ALTERNATIVE IV	New plant at new site; abandon existing plant and existing site

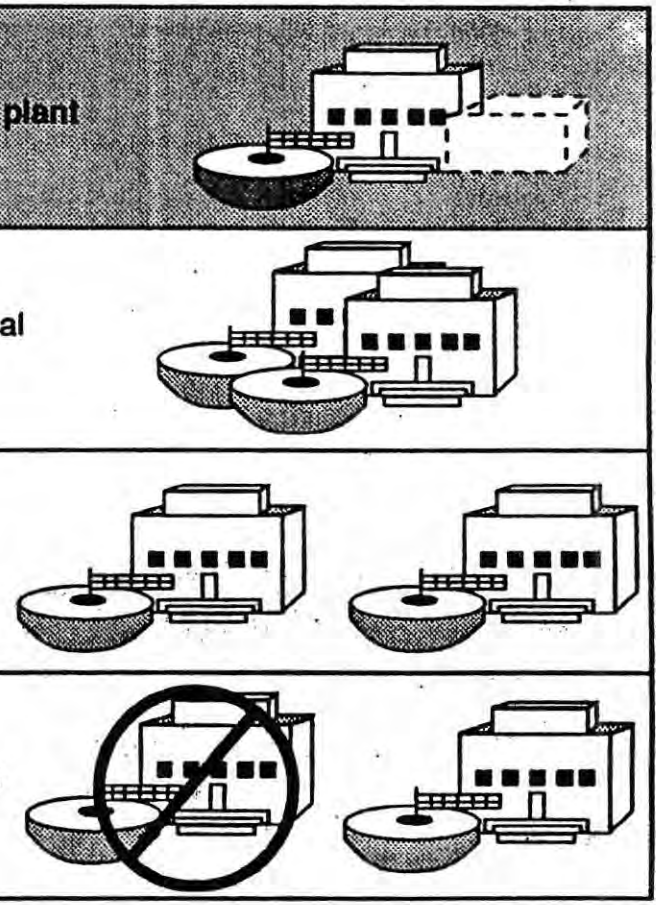


Figure 1
Siting Alternatives
City of Woodburn WWTP
Woodburn, Oregon

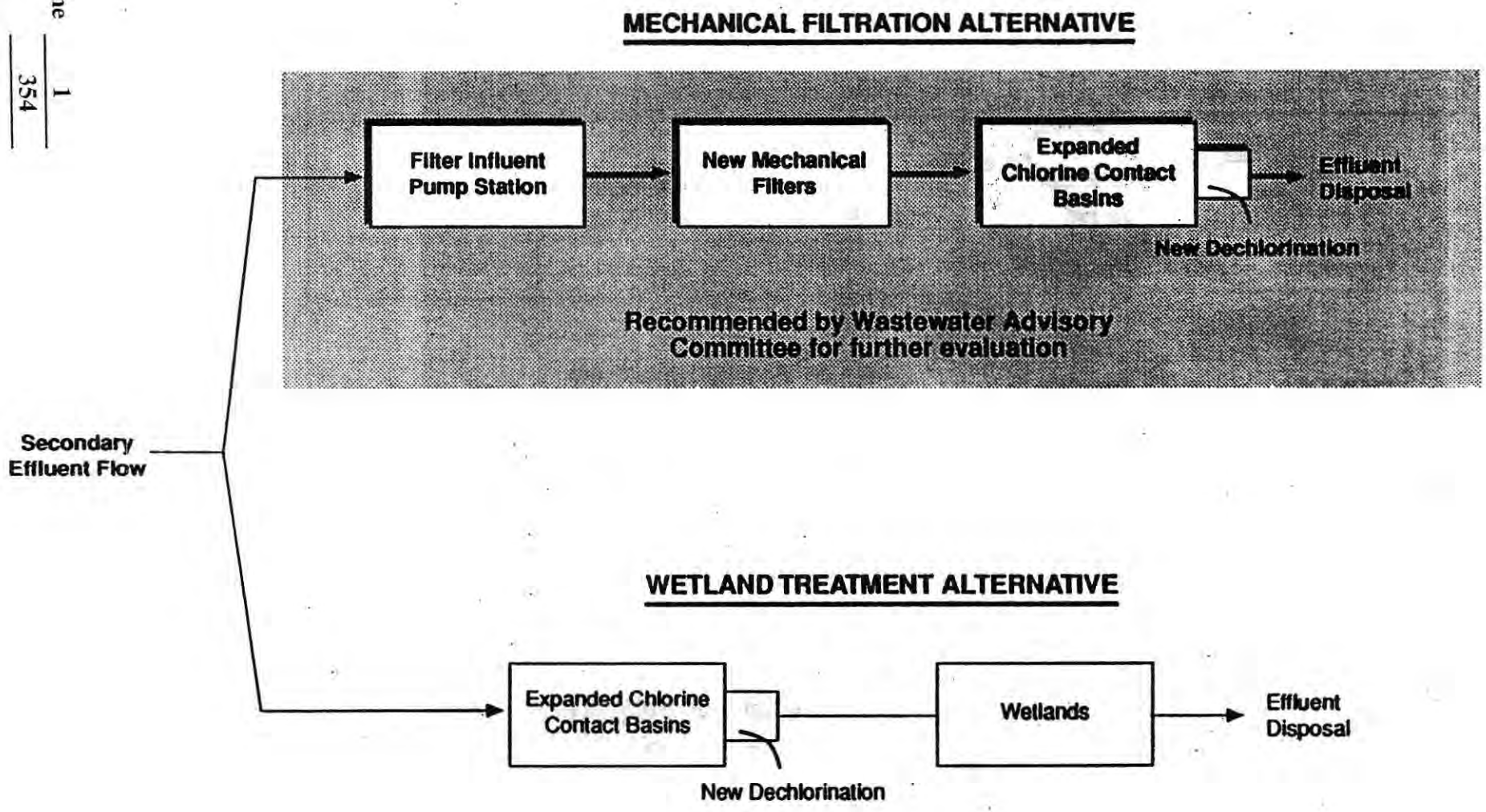


Figure 2
Tertiary Treatment Alternatives
City of Woodburn WWTP
Woodburn, Oregon

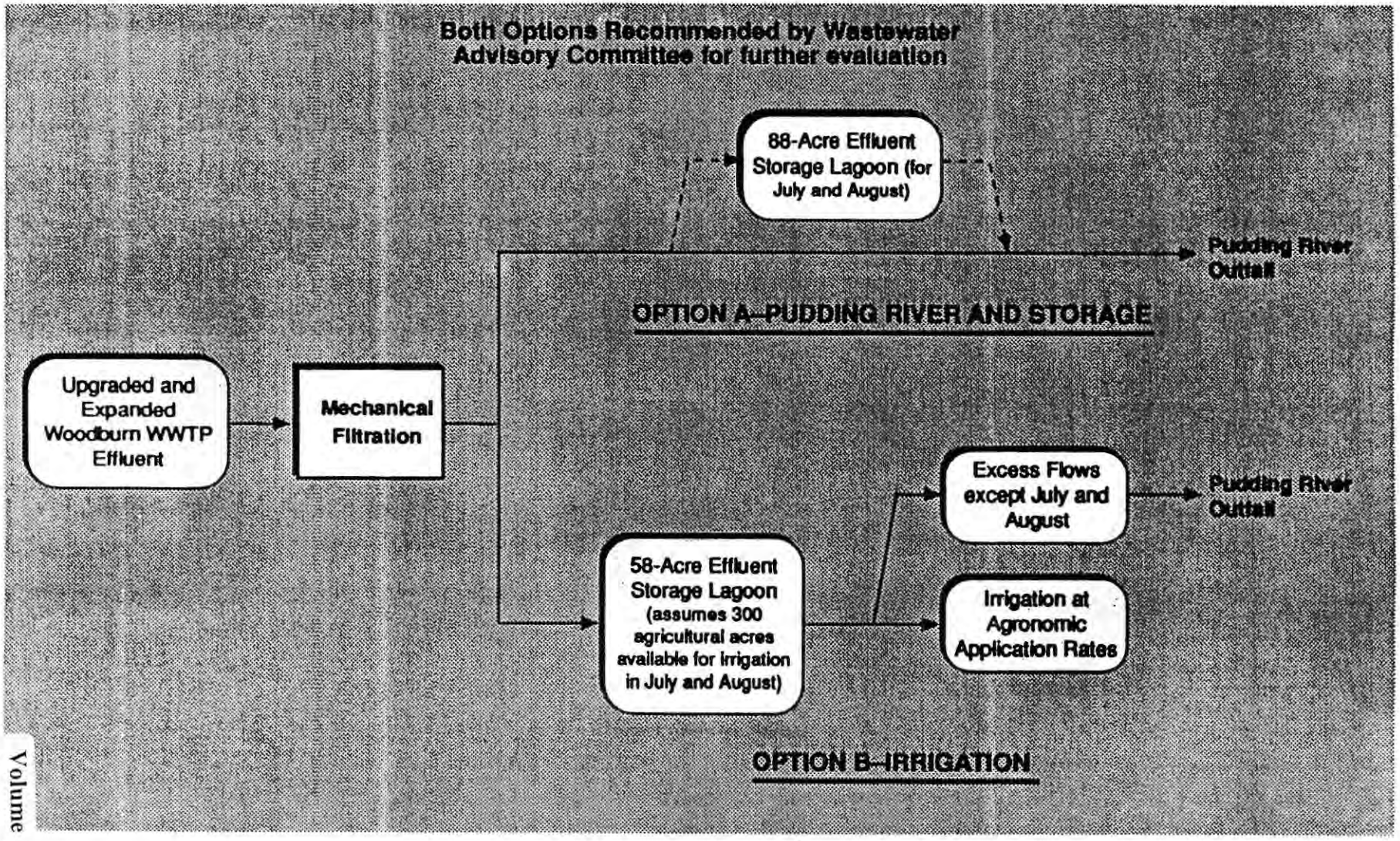
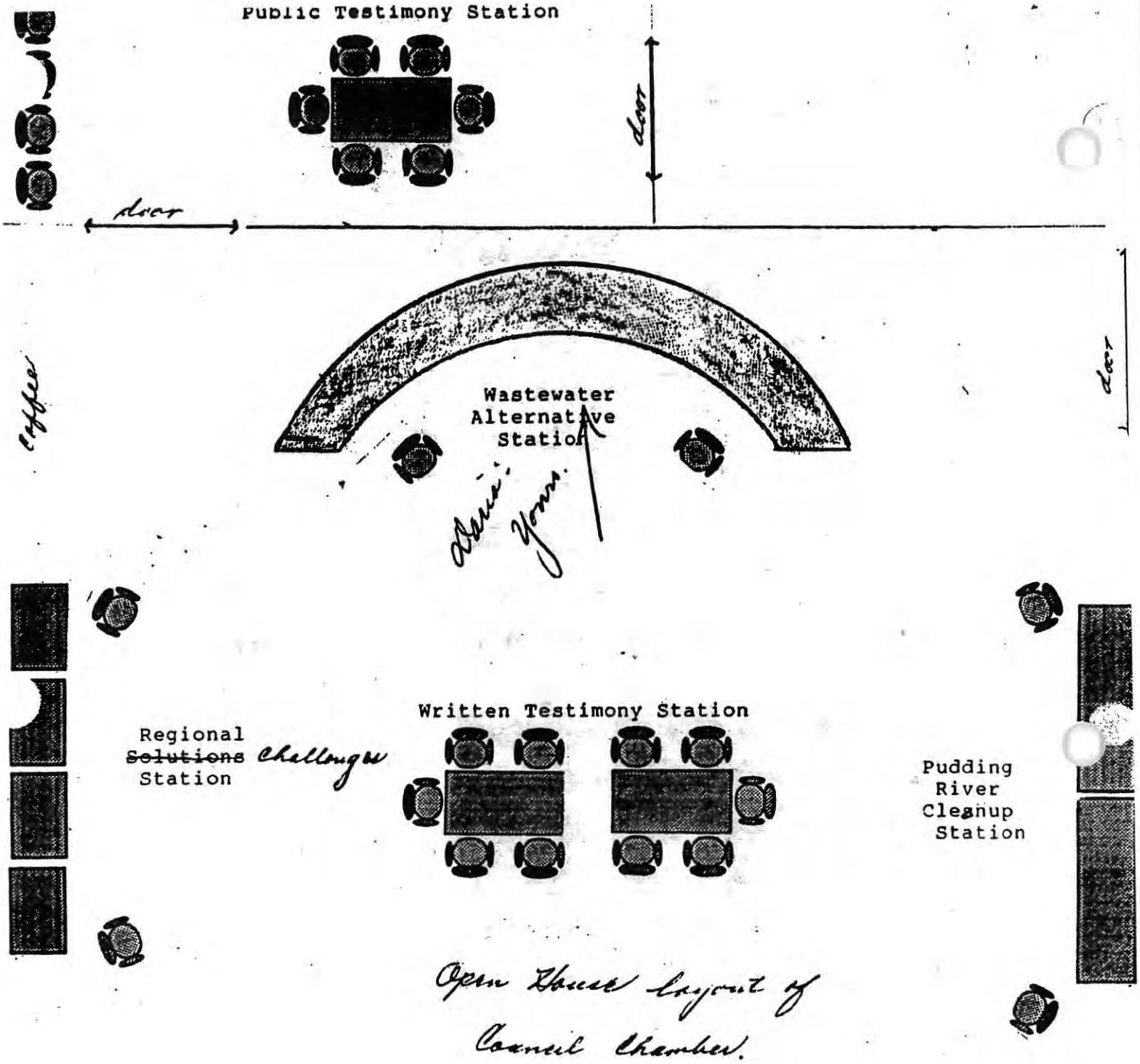
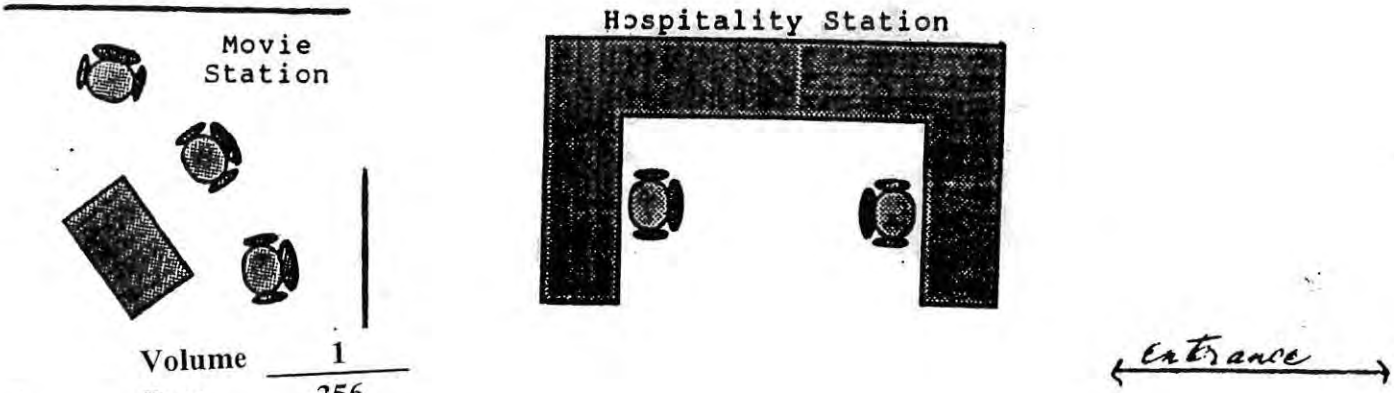


Figure 3
Options for Effluent Disposal During the Summer (May-October)
City of Woodburn WWTP
Woodburn, Oregon


Public Testimony Station
door
door
coffee
door
Wastewater
Alternative
Station
Alice Young
Regional
~~Solutions~~ Challenge
Station
Written Testimony Station
Pudding
River
Cleanup
Station
Open House layout of
Council Chamber.


Movie
Station
Hospitality Station
entrance

WELCOME
to
The Wastewater Advisory Committee OPEN HOUSE
on
WASTEWATER TREATMENT ALTERNATIVES

WHY DO WE NEED TO TREAT WASTEWATER?

Wastewater contains contaminants that could pollute ground water if discharged directly to the river or ground. It is important to note that we draw our drinking water from the ground through wells. The treatment process eliminates many of the contaminants thus reducing the possibility of diseases.

WHY ARE WE HAVING THIS OPEN HOUSE?

Although the committee preferred the alternative that keeps the plant at the present site with modifications and upgrades, they want your ideas regarding:

- wetlands as part of the wastewater treatment system
- piping treated wastewater to the Willamette River
- providing services to other communities for a fee
- treatment facilities that can expand as Woodburn grows
- any other proposals

HOW DOES THIS OPEN HOUSE FORMAT WORK?

The open house format is set up in a series of information stations so you can easily circulate around the room

HOSPITALITY TABLE - Sign in and feel free to circulate

VIDEO - A short video will be shown about how our present treatment system operates.

PUDDING RIVER CLEAN UP - This station will explain why we have to clean up the Pudding River and the estimated costs of cleaning up.

ALTERNATIVE TREATMENT PLANS - At this station the preferred alternative will be explained and the other concepts that were considered will be illustrated.

REGIONAL CHALLENGES - Learn about sewage problems faced by all Pudding River communities, such as Hubbard, Molalla, Mt. Angel and Silverton; problems such as sludge, septic tank waste, leachate and laboratory services.

PUBLIC TESTIMONY - If you would like to make oral comments to committee members sign a public testimony card and take it to the public testimony station.

OR

WRITTEN TESTIMONY - Tables will be available to fill out a written testimony sheet, then drop the sheet in the COMMENTS box at the hospitality table.

∞ ∞ ∞ ∞ ∞ ∞ ∞

YOUR OPINION DOES COUNT - Summaries of the testimonies will be reviewed by the Wastewater Advisory Committee and the City Council. The final decision will not be made without consideration of these summaries.

LATER, summaries of the written and oral testimonies will be printed in the local newspaper. Or you can get a copy by calling Frank Sinclair at the Wastewater Treatment Plant at 981-5281.

Appendix A
Current NPDES Waste Discharge Permit

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Oregon

DEPARTMENT OF
ENVIRONMENTAL
QUALITY

July 31, 1992

Honorable Fred Kyser, Mayor
City of Woodburn
270 Montgomery Street
Woodburn, Oregon 97071

Final Date for Submission of
Written Comments: August 14, 1992

Re: Waste Disposal Permit
File No. 98815
Marion County

Your application for a National Pollutant Discharge Elimination System (NPDES) permit has been reviewed by the Department and a proposed NPDES permit has been drafted. You are invited to review the attached copy and submit any comments you may have in writing prior to the date indicated above. A copy of the permit evaluation report is also enclosed, which describes in more detail the contents of the permit.

A draft Stipulation and Final Order, plus a fact sheet briefly describing Orders, are enclosed. The Department is proposing to issue an Order to you, because we believe that your facility may not be able to comply with the water quality based effluent limits for total residual chlorine. The Order also includes interim limits for total residual chlorine and a compliance schedule to meet the effluent limits in the permit. Note that the compliance schedule included in the Order is based on the adoption of the Total Maximum Daily Load (TMDL) for the Pudding River and approval of the facilities plan.

Some changes have been made in proposed permit since our last meeting. These include the following: 1) rounding mass limits to two significant digits, 2) monitoring 1/week for enterococci bacteria for a one year period only and 3) groundwater monitoring in accordance with your approved plan (this item was inadvertently ommitted in the previous draft). The groundwater monitoring condition requires 2/year monitoring for nitrate-nitrogen, which is consistent with the City's current practice.

811 SW Sixth Avenue
Portland, OR 97204-1390
(503) 229-5696
TDD (503) 229-6993

Although we did receive the FAX from Bob Spetich of CH2M-Hill regarding the design wet weather flow, no changes to the wet weather mass limits are included in the proposed permit. In order to get the higher mass load limits during the winter months, the submittal should include an analysis of the design wet weather flow with supporting documentation.

Other information which will be distributed to the public is enclosed for your review. Your comments on the content of this information will also be appreciated.

At the close of the comment period, and after your comments, if any, have been received, the Department will modify the draft permit as appropriate. The public notice regarding your application will be circulated to interested individuals and organizations. The proposed permit will also be made available to those persons requesting it. A public hearing is scheduled to take public testimony in Woodburn, on September 24, 1992. After the public review and participation period is over, the final NPDES permit will be issued.

If you have any comments or questions, please contact me at 229-5185.

Sincerely,

Rajeev Kapur

Rajeev Kapur
Municipal Wastewater Section
Water Quality Division

Enclosures
cc: Willamette Valley Region, DEQ

APPLICANT REVIEW

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM
WASTE DISCHARGE PERMIT

Department of Environmental Quality
811 S.W. Sixth Avenue, Portland, OR 97204
Telephone: (503) 229-5696

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

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ISSUED TO:

City of Woodburn
270 Montgomery Street
Woodburn, OR 97071

SOURCES COVERED BY THIS PERMIT:

Type of Waste	Outfall Number	Outfall Location
Domestic Sewage	001	R.M. 21.5

Emergency Overflows (Pump Stations) 002 & 003

Lift Station # 2	-	Jansen Way	002	Mill Creek
Lift Station # 6	-	Santiam Drive	003	Storm Sewer

PLANT TYPE AND LOCATION:

Rotating Biological Contactors
with Sand Filters
2815 Molalla Road
Woodburn, OR 97071
Treatment System Class:IV
Collection System Class: III

RECEIVING SYSTEM INFORMATION:

Basin: Willamette
Sub-Basin: Molalla/Pudding
Stream: Pudding River
Hydro Code: 22K-PUDD 21.5 D
County: Marion

EPA REFERENCE NO: OR-002000-1

Issued in response to Application No. 997753 received November 8, 1991.

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

Lydia R. Taylor, Administrator _____ 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 Disposal Limitations not to be Exceeded...	2-3
Schedule B - Minimum Monitoring and Reporting Requirements...	4-7
Schedule C - Compliance Conditions and Schedules.....	8
Schedule D - Special Conditions.....	9-11
Schedule E - Pretreatment Conditions.....	12-14
General Conditions.....	Attached

Unless authorized by another NPDES Permit, each other direct and indirect discharge to public waters is prohibited.

SCHEDULE A

1. **Waste Discharge Limitations not to be Exceeded After Permit Issuance.**

a. Outfall Number 001 (Sewage Treatment Plant Discharge)

(1) May 1 - October 31:

Parameter	Average Effluent Concentrations Monthly	Average Effluent Concentrations Weekly	Mass Load Limits (See Note 1/) Monthly Average lb/day	Mass Load Limits (See Note 1/) Weekly Average lb/day	Mass Load Limits (See Note 1/) Daily Maximum lbs
a. CBOD-5**	10 mg/l	15 mg/l	260	390	520
b. TSS	10 mg/l	15 mg/l	260	390	520

(2) November 1 - April 30: (See note 1/)

Parameter	Average Effluent Concentrations Monthly	Average Effluent Concentrations Weekly	Mass Load Limits (See Note 2/) Monthly Average lb/day	Mass Load Limits (See Note 2/) Weekly Average lb/day	Mass Load Limits (See Note 2/) Daily Maximum lbs
a. CBOD-5**	25 mg/l	40 mg/l	650	980	1300
b. TSS	30 mg/l	45 mg/l	780	1200	1600

** The CBOD-5 concentration limits are considered equivalent to the minimum design criteria for BOD-5 specified in Oregon Administrative Rules 340-41. These limits and CBOD-5 mass limits may be adjusted (up or down) by permit action if more accurate information regarding CBOD-5/BOD-5 equivalency become available.

(3) Other parameters (year-round)

Fecal Coliform Bacteria	Shall not exceed 200/100 ml monthly geometric mean and 400/100 ml weekly geometric mean.
pH	Shall be within the range 6.0 - 9.0.
CBOD and TSS Removal Efficiency	Shall not be less than 85% monthly average.
Total Residual Chlorine (See Note 3/)	0.03 mg/l daily maximum.

(4) Not withstanding the effluent limitations established by this permit, no wastes shall be discharged and no activities shall be conducted which violate Water Quality Standards as adopted in OAR 340-41-445 except in the defined mixing zone:

The allowable mixing zone is that portion of the receiving stream from the point of discharge to 270 feet (90 meters) downstream of the discharge point. In addition, the ZID (Zone of Immediate Dilution) shall not exceed 10 percent of the defined mixing zone in any direction from the point of discharge. (See Note 3/)

b. Emergency Overflow Outfalls 002 and 003:

No wastes shall be discharged from these outfalls and no activities shall be conducted which violate water quality standards as adopted in OAR 340-41-445, unless the cause of the discharge is the result of an upset as defined in general conditions B4 and B6.

1/ Based on average dry weather design flow to the facility equaling 3.1 MGD. The daily maximum limit is suspended on any day when the daily flow to the treatment facility exceeds 6.2 MGD, twice the dry weather design flow.

2/ These limits are based on an average dry weather design flow to the facility equaling 3.1 MGD. However, these limits are interim limits only and are effective until the permittee completes the necessary engineering study to accurately determine the design average wet weather flow pursuant to OAR 340-41-120(9). Upon review and approval of the engineering study and upon request by the permittee, the Department intends to modify this permit and include revised mass load limits and an inflow reduction plan. In addition, the daily maximum limit is suspended on any day when the daily flow to the treatment facility exceeds 6.2 MGD, twice the dry weather design flow.

3/ Once the permittee upgrades treatment facilities, the Department may modify the effluent limit for total residual chlorine and the mixing zone dimensions. The Department also may include effluent limits for ammonia-nitrogen, if necessary.

SCHEDULE B

1. **Minimum Monitoring and Reporting Requirement** (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 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. Such results shall not constitute a violation of 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

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD)	Daily	Meter Reading
Flow Meter Calibration	Semi-annually	Verification
CBOD-5	2/Week	Composite
TSS	2/Week	Composite
pH	3/Week	Grab
TOXICS:		
Metals: (Ag, As, Cd, Cr, Cu, Hg, Ni, Pb, Zn) & Cyanide (CN), measured as total in mg/l (See note 1/)	Semi-annually using 3 consecutive days between Monday and Friday, inclusive	24-hr daily composite (See note 2/)
Total Phenols (See Note 1/)	Semi-annually using 3 consecutive days between Monday and Friday, inclusive	24-hr daily composite (See note 2/)

b. Outfall Number 001 (Treatment Plant Discharge to the Pudding River)

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD)	Daily	Meter Reading
Flow Meter Calibration	Semi-annually	Verification
CBOD-5	2/Week	Composite
TSS	2/Week	Composite
Fecal Coliform	2/Week	Grab
pH	3/Week	Grab
Quantity Chlorine Used	Daily	Measurement
Chlorine Residual	Daily	Grab
Average Percent Removed (CBOD and TSS)	Monthly	Calculation
Nutrients:		
Ammonia - Nitrogen	2/Week	Composite
Nitrite + Nitrate - Nitrogen	2/Week	Composite
Total Phosphorus	Weekly between June 1 & Oct 31	Composite
TOXICS:		
Metals: (Ag, As, Cd, Cr, Cu, Hg, Ni, Pb, Zn) & Cyanide (CN), measured as total in mg/l (See Note 1/)	Semi-annually, using 3 consecutive days between Monday and Friday, inclusive	24-hr daily composite (See note 2/)
Total Phenols (See Note 1/)	Semi-annually, using 3 consecutive days between Monday and Friday, inclusive	24-hr daily composite (See note 2/)
Toxics Removal	Semi-annually	Calculation (See note 3/)
Bioassay of effluent from Outfall 001 (See Note 4/)	Quarterly	Acute and chronic bioassay

c. **Groundwater Monitoring**

Item or Parameter	Minimum Frequency	Type of Sample
Water Level Elevation	2/year	Record
Nitrate - Nitrogen	2/year	Grab

1/ For influent and effluent cyanide and phenol samples, at least eight (8) discrete grab samples shall be collected over the operating day. Each aliquot shall not be less than 100 ml and shall be collected and composited into a larger container which has been preserved with sodium hydroxide for cyanide samples, and sulfuric acid for total phenols samples.

2/ Daily 24-hour composite samples shall be analyzed and reported separately.

3/ Plant removal rates shall be calculated for each 3-day sampling event. Removals shall be calculated by: (1) averaging the three influent concentration values for each parameter collected during the sampling event; (2) averaging the three effluent concentration values for each parameter collected during the sampling event; and (3) using the two average concentrations to calculate the parameter's removal. The removals for each 3-day sampling event as well as monitoring data for each day of sampling shall be reported.

4/ Beginning January, 1994, the permittee shall conduct bioassay testing for a period of one (1) year in accordance with the frequency specified above and the procedure outlined in Schedule D of the permit. 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. Bioassay test results will be required along with the next NPDES Permit renewal application.

d. **Sludge Management**

Item or Parameter	Minimum Frequency	Type of Sample
Sludge analysis including: Total solids (% dry wt.) Volatile solids (% dry wt.) Sludge nitrogen NH_3-N; NO_3-N; & TKN (% dry wt.)	Semi-annually	Composite sample to be representative of the product to be land applied

d. Sludge Management (cont.)

Item or Parameter	Minimum Frequency	Type of Sample
Sludge metals content for Ag, As, Hg, Pb, Zn, Cu, Ni, Cr, & Cd (in mg/kg dry weight) Phosphorus (% dry wt.) Potassium (% dry wt.) pH (standard units)		
% volatile solids reduction accomplished through digestion	Monthly, for anaerobic digesters	Calculation (See Note 5/)
Record of locations where sludge is applied on land (Site location map to be maintained at treatment facility for review upon request by DEQ)	Each Occurrence	Date, quantity & locations where sludge is applied recorded on a site location map

Notes:

5/ Calculation of the % volatile solids reduction for the anaerobic digesters is to be based on comparison of a representative grab sample of total and volatile solids entering and exiting the primary anaerobic digester or entering the primary anaerobic digester and exiting the sludge drying beds.

2. Reporting Procedures

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

State discharge 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.

State discharge monitoring reports (DMRs) 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.

SCHEDULE C

Compliance Schedules and Conditions

1. Within six (6) months after permit issuance date, the permittee shall submit to the Department for review and approval a program and schedule for identifying and reducing inflow. At a minimum, the program shall consist of the following:

   a. Identification of all overflow points and verification that the 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 inflow sources into the sewer system over which the permittee has legal control; and

   d. Within one (1) year of Department approval of the program, the permittee shall begin implementation of the program.

2. Within six (6) months after permit issuance date, the permittee shall submit and implement a contingency plan for the prevention and handling of spills from each emergency overflow point. The spill response plan shall include the posting of high public exposure areas and the issuance of public notifications.

3. 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. All sludge shall be managed in accordance with a sludge management plan approved by the Department of Environmental Quality. No substantial changes shall be made in sludge management activities which significantly differ from operations specified under the approved plan without the prior written approval of the Department.

2. 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 collection and treatment systems supervised by one or more operators who are certified in the class (collection or treatment) and grade level (equal to or greater) that corresponds with the class of the system to be supervised (see 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. No system shall be without supervision (as required by Special Condition 2. a. above) for more than thirty (30) days. During periods when the supervisor is absent (off-site and physically not available), the permittee shall provide an alternate, or in the case of shift operation, designate a shift supervisor. The alternate or shift supervisor shall be certified in the proper class and at no less than one grade level lower than the class of the system to be supervised.

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

   d. The permittee shall notify the Department of Environmental Quality in writing within thirty (30) days of replacement or redesignation of certified operators responsible for supervising system operation (including shifts). The notice shall be filed with the Water Quality Division, Operator Certification Program (see address on page one). This requirement is in addition to the reporting requirements contained under Schedule B of this permit.

e. 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 2.b. above.

3. Bioassay

a. The permittee shall conduct chronic whole effluent toxicity bioassay tests of outfall 001 in accordance with the frequency specified in Schedule B with Ceriodaphnia dubia (water flea), Pimephales promelas (fathead minnow), and Selanastrum capricornutum (green algae).

b. Bioassay tests shall be dual end-point tests in which both acute and chronic end-points can be determined from the results of a single chronic test. The acute end-point (LC50) only applies when significant mortality occurs.

c. Bioassay shall be conducted in accordance with Short-Term Methods for Estimating the Chronic Toxicity of Effluent and Receiving Waters to Freshwater Organisms, EPA/600/4-89/001 and Methods for Measuring the Acute Toxicity of Effluents to Aquatic Organisms, EPA (most current edition). Quality assurance criteria, statistical analyses and data reporting for the bioassays shall be in accordance with the EPA document for chronic testing referenced above.

d. 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.

e. If any acute bioassay test indicates that the effluent sample is toxic at the dilutions determined to occur at the edge of the Zone of Immediate Dilution another toxicity test using the same species and the same methodology shall be conducted within two weeks. If the second test also indicates toxicity, the permittee shall follow the procedure described in section (g) of this permit condition.

f. If a chronic bioassay test indicates that the effluent sample is toxic at the dilutions determined to occur at the edge of the mixing zone, or if there is no dilution data for the edge of the mixing zone and any chronic bioassay test indicates that the effluent is toxic, another toxicity test using the same species and the same methodology shall be conducted within two weeks. If the second test also indicates toxicity, the permittee shall follow the procedure described in section (g) of this permit condition.

g. If, after following the procedure as described in sections (e) or (f) of this permit condition, two consecutive bioassay test results indicate acute and/or chronic toxicity, the permittee shall evaluate the source of the toxicity and submit a plan and time schedule for achieving compliance with the water quality standards for toxicity. Upon approval by the Department, the permittee will implement the plan until compliance has been achieved. Evaluations shall be completed and plans submitted within 6 months unless otherwise approved in writing by the Department.

4. In order to meet summer effluent limits for CBOD and TSS, the permittee has installed a lagoon for additional treatment of primary effluent prior to discharge to the Rotating Biological Contactors and a filtration system for polishing of effluent prior to discharge. These systems are sized to handle summer flows and taking these facilities off-line during the winter months does not constitute a bypass.

SCHEDULE E

Pretreatment Activities

The permittee shall implement the following pretreatment activities:

1. The permittee shall conduct and enforce its Pretreatment Program, as approved by the Department, and comply with the General Pretreatment Regulations (40 CFR Part 403). The permittee shall secure and maintain sufficient resources and qualified personnel to carry out the program implementation procedures described in this permit.

2. The permittee shall adopt all legal authority necessary to fully implement its approved pretreatment program and to comply with all applicable State and Federal pretreatment regulations. The permittee must also establish, where necessary, contracts or agreements with contributing jurisdictions to ensure compliance with pretreatment requirements by industrial users within these jurisdictions. These contracts or agreements shall identify the agency responsible for all implementation and enforcement activities to be performed in the contributing jurisdictions. Regardless of jurisdictional situation, the permittee is responsible for ensuring that all aspects of the pretreatment program are fully implemented and enforced.

3. The permittee shall update its inventory of industrial users at a frequency and diligence adequate to ensure proper identification of industrial users subject to pretreatment standards, but no less than once per year. The permittee shall notify these industrial users of applicable pretreatment standards in accordance with 40 CFR § 403.8(f)(2)(iii).

4. The permittee shall enforce categorical pretreatment standards promulgated pursuant to Section 307(b) and (c) of the Act, prohibited discharge standards as set forth in 40 CFR § 403.5(a) and (b), or local limitations developed by the permittee in accordance with 40 CFR § 403.5(c), whichever are more stringent, or are applicable to nondomestic users discharging wastewater to the collection system. Locally derived discharge limitations shall be defined as pretreatment standards under Section 307(d) of the Act.

   A technical evaluation of the need to revise local limits 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 application, unless the Department requires in writing that it be submitted sooner. Limits development will be in accordance with the procedures established by the Department.

5. The permittee shall issue individual discharge permits to all Significant Industrial Users in a timely manner. The permittee shall also reissue and/or modify permits, where necessary, in a timely manner. Discharge permits must contain, at a minimum, the conditions identified in 40 CFR § 403.8(f)(1)(iii). Unless a more stringent definition has been adopted by the permittee, the definition of Significant Industrial User shall be as stated in 40 CFR § 403.3(t).

6. The permittee shall randomly sample and analyze industrial user effluents at a frequency commensurate with the character, consistency, and volume of the discharge. At a minimum, the permittee shall sample all Significant Industrial Users for all regulated pollutants twice per year, and shall conduct a complete facility inspection once per year. Additionally, at least once every two years the permittee shall evaluate the need for each Significant Industrial User to develop a slug control plan. Where a plan is deemed necessary, it shall conform to the requirements of 40 CFR § 403.8(f)(2)(v).

   Where the permittee elects to conduct all industrial user monitoring in lieu of requiring self-monitoring by the user, the permittee shall gather all information which would otherwise have been submitted by the user. The permittee shall also perform the sampling and analyses in accordance with the protocols established for the user.

   Sample collection and analysis, and the gathering of other compliance data, shall be performed with sufficient care to produce evidence admissible in enforcement proceedings or in judicial actions. Unless specified otherwise by the Director in writing, all sampling and analyses shall be performed in accordance with 40 CFR Part 136.

7. The permittee shall review reports submitted by industrial users and identify all violations of the user's permit or the permittee's local ordinance.

8. The permittee shall investigate all instances of industrial user noncompliance and shall take all necessary steps to return users to compliance. The permittee's enforcement actions shall track its approved Enforcement Response Plan, developed in accordance with 40 CFR § 403.8(f)(5). If the permittee has not developed an approved Enforcement Response Plan, it shall develop and submit a draft to the Department for review within 90 days of the issuance of this permit.

9. The permittee shall publish, at least annually in the largest daily newspaper published in the permittee's service area, a list of all industrial users which, at any time in the previous 12 months, were in Significant Noncompliance with applicable pretreatment requirements. For the purposes of this requirement, an industrial user is in Significant Noncompliance if it meets one or more of the criteria listed in 40 CFR 403.8(f)(2)(vii).

10. The permittee must develop and maintain a data management system designed to track the status of the industrial user inventory, discharge characteristics, and compliance. In accordance with 40 CFR § 403.12(o), the permittee shall retain all records relating to pretreatment program activities for a minimum of three years, and shall make such records available to the Department and USEPA upon request. The permittee shall also provide public access to information considered effluent data under 40 CFR Part 2.

11. The permittee shall submit by March 1 of each year, a report (2 copies) that describes the permittee's pretreatment program during the previous calendar year. The content and format of this report shall be as established by the Department.

12. The permittee shall submit in writing to the Department a statement of the basis for any proposed modification of its approved program and a description of the proposed modification in accordance with 40 CFR § 403.18(b). No substantial program modifications may be implemented by the permittee prior to receiving written authorization from the Department.

P98815W (7-31-92)

NPDES GENERAL CONDITIONS

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) 468.720 and is grounds for enforcement action; for permit termination, suspension, or modification; or for denial of a permit renewal application.

2. Penalties for Violations of Permit Conditions

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, Oregon Law (ORS 468.990) classifies a willful or negligent violation of the terms of a permit or failure to get a permit as a misdemeanor and a person convicted thereof shall be punishable by a fine of not more than $25,000 or by imprisonment for not more than one year, or by both. Each day of violation constitutes a separate offense.

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 which 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.

b. Prohibition of bypass.

(1) Bypass is prohibited unless:

(a) Bypass was necessary to prevent loss of life, personal injury, or severe property damage;

(b) There were no feasible alternatives to the bypass, such as the use of auxiliary treatment facilities, retention of untreated wastes, or maintenance during normal periods of equipment downtime. This condition is not satisfied if adequate backup equipment should have been installed in the exercise of reasonable engineering 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 paragraph c of this section.

(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 paragraph b(1) of this section.

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 Section D, Paragraph D-5.

4. Upset

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

b. Effect of an upset. An upset constitutes an affirmative defense to an action brought for noncompliance with such technology based permit effluent limitations if the requirements of Section B.4.c. of these General Conditions are met. No determination made during administrative review of claims that non-compliance 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; and

(3) The permittee submitted notice of the upset as required in Section D.5., hereof (24-hour notice).

(4) The permittee complied with any remedial measures required under Section 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 an 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; and

(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 Condition B4 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 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 insure 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 by the 15th day of the following month unless specifically approved otherwise in Schedule B of this permit.

6. Additional Monitoring by the Permittee

If the permittee monitors any pollutant more frequently than required by this permit, using test procedures approved under 40 CFR 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 DMR. 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 based on a geometric or log mean.

8. Retention of Records

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 which 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 which may endanger health or the environment. Any information shall be provided orally (by telephone) within 24 hours 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 Accident Response System). A written submission shall also be provided within 5 days of the time the permittee becomes aware of the circumstances. The written submission 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.

e. Public notification steps taken, pursuant to General Condition B-6.

The following shall be included as information which 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 the permit.

c. Violation of maximum daily discharge limitation for any of the pollutants listed by the Director in the 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 non-compliance not reported under Section D4 or D5, 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 which 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 promptly 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 Reports

State law provides that any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit, including monitoring reports or reports of compliance or noncompliance shall, upon conviction be punished by a fine of not more than $1,000 per violation, or by imprisonment for not more than six months per violation, or by both.

SECTION E. DEFINITIONS

1. BOD means five-day biochemical oxygen demand.

2. TSS means total suspended solids (non-filterable residue).

3. Mg/l means milligrams per liter.

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 discreet 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 enterococci bacteria.

Appendix B
Stipulation and Final Order Agreement, September 1992

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APPLICANT REVIEW

BEFORE THE ENVIRONMENTAL QUALITY COMMISSION

OF THE STATE OF OREGON

DEPARTMENT OF ENVIRONMENTAL QUALITY, OF THE STATE OF OREGON, Department, v. CITY OF WOODBURN Respondent.	STIPULATION AND FINAL ORDER No. WQ-MW- MARION COUNTY

WHEREAS:

1. On _____, the Department of Environmental Quality (Department or DEQ) issued National Pollutant Discharge Elimination System (NPDES) Waste Discharge Permit Number _____ (Permit) to the City of Woodburn, (Respondent), pursuant to Oregon Revised Statutes (ORS) 468B.050 and the Federal Water Pollution Control Act Amendments of 197, P.L. 9-500 as amended. The Permit authorizes the Respondent to construct, install, modify or operate wastewater treatment control and disposal facilities (facilities) and discharge adequately treated wastewaters into the Pudding River, waters of the state, in conformance with the requirements, limitations and conditions set forth in the Permit.

2. The Respondent operates a sewage treatment facility that uses chlorine as a disinfecting agent for the treated effluent prior to discharging to public waters.

3. Chlorine and ammonia-nitrogen are toxic substances that can be harmful to aquatic organisms. Discharges of any substance, including chlorine and ammonia-nitrogen, that cause water quality

stream standards violations outside of a designated mixing zone are prohibited by Oregon Administrative Rules (OAR) 340-41-445(2).

4. The water quality standard for dissolved oxygen is violated in the Pudding River. As a result, the Department has classified the Pudding River as a water quality limited stream and is in the process of establishing a Total Maximum Daily Load (TMDL) for dissolved oxygen. A TMDL is the total amount of pollutant that can enter a water body without causing a violation of a water quality standard. The TMDL includes waste load allocations (WLAs) for point sources and load allocations (LAs) for non-point sources and natural background activities.

5. Respondent is required to conduct a mixing zone study as part of the facility planning process to address the TMDL. Until such time as Respondent completes the mixing zone study, final effluent limits for chlorine and ammonia-nitrogen cannot be established. The final effluent limits for chlorine and ammonia-nitrogen will not be effective until the Respondent attains operational level to comply with the TMDL.

6. The Department has included a chlorine effluent limitation in the permit. This limitation is an estimate of the maximum allowable effluent limit which would still meet water quality standards for chlorine.

7. The Department and Respondent recognize that the Respondent may not be able to achieve compliance with the water quality standards for chlorine and ammonia-nitrogen without making necessary improvements in Respondent's sewage treatment facility.

8. The Permit requires the Respondent to conduct bioassay testing on final effluent. The Department and Respondent recognize that whole effluent toxicity due to ammonia-nitrogen and/or chlorine is likely until the Respondent achieves compliance with the water quality standards for chlorine and ammonia-nitrogen.

9. The facility receives high Biochemical Oxygen Demand (BOD) loads from food processing industries during the summer months. Respondent has incorporated an aerated basin and lagoon into the treatment facility in an effort to reduce BOD loading to the secondary treatment facilities. The aerated basin and lagoon are in use for a period of five months during the summer.

10. With the incorporation of the lagoon into the treatment facility, additional solids in the form of algae are encountered. As a result, the Respondent may not be able to consistently meet the effluent limits for Total Suspended Solids (TSS) during the summer.

11. The Permit includes 85 percent removal efficiencies for BOD and TSS in accordance with federal requirements. Respondent may not be able to consistently meet the removal requirements for BOD and TSS in the Permit.

12. The Department and Respondent recognize that the 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 Respondent wish to limit and resolve the future violations referred to in Paragraphs 7, 8, 10 and 11 in advance by this Stipulation and Final Order (SFO).

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

NOW THEREFORE, it is stipulated and agreed that:

14. The Environmental Quality Commission hereby issues a final order:

a. Requiring Respondent to comply with the following schedule:

(1) Twelve (12) months after establishment of the TMDL, the Respondent shall submit a final facilities plan report that evaluates alternatives for complying with the TMDL, percent removal requirements for BOD and TSS, the water quality standards for chlorine and ammonia, and all other applicable water quality standards. The facilities plan shall also include an evaluation of the mixing zone.

(2) Eighteen (18) months after approval of the facilities plan, the Respondent shall submit engineering plans and specifications for construction of necessary improvements.

(3) Twenty-four (24) months after approval of the facilities plan, Respondent shall award construction contracts for completion of necessary improvements. Progress reports are to be submitted to the Department at 6 month intervals from award of bid.

(4) Forty-four (44) months after approval of the facilities plan, Respondent shall complete construction of the necessary improvements.

(5) Fifty (50) months after approval of the facilities plan, Respondent shall attain operational level to comply with the TMDL established for the Pudding River and assure that any toxic impact of the Respondent's discharge of chlorine residual, ammonia-nitrogen or any other toxic substance complies with OAR 340-41-445(2)(p) for toxic substances and OAR 340-41-445(4) for mixing zones.

b. Requiring Respondent to meet the following interim requirements which are effective from Permit issuance date until attainment of operational level to comply with the TMDL:

(1) The chlorine residual concentration shall not exceed 1.5 mg/l on a daily basis.

(2) Total Suspended Solids concentration of 20 mg/l monthly average and 30 mg/l weekly average from May 1 - October 31. (Respondent shall comply with all mass limits in the permit).

(3) Bioassay Tests may be conducted with chlorine and ammonia-nitrogen stripped from the final effluent.

(4) BOD and TSS removal efficiency of 80 percent monthly average.

c. Requiring Respondent to comply with all the terms, schedules and conditions of the Permit except as specified by Paragraph 14.b above, or of any other NPDES waste discharge permit issued to Respondent while this SFO is in effect.

d. Requiring Respondent, upon receipt of a written Penalty Demand notice from the Department, to pay civil penalties of one

hundred dollars ($100) for each day of each violation of Paragraph 14.b and civil penalties of two hundred fifty dollars ($250) for each day of each violation of any other requirement of this SFO.

15. If any event occurs that is beyond Respondent's reasonable control and that causes or may cause a delay or deviation in performance of the requirements of this SFO, Respondent 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 Respondent proposes to carry out such measures. Respondent shall confirm in writing this information within five (5) working days of the onset of the event. It is Respondent'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 Respondent. If Respondent so demonstrates, the Department shall extend times of performance of related activities under this SFO as appropriate. Circumstances or events beyond Respondent'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 shall not be considered circumstances beyond Respondent's control.

16. Respondent 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 this SFO. The Department reserves the right to enforce this SFO through appropriate administrative and judicial proceedings.

17. Regarding the schedule set forth in Paragraph 14.a above, Respondent acknowledges that Respondent is responsible for complying with that schedule regardless of the availability of any federal or state grant monies.

18. The terms of this SFO may be amended by the mutual agreement of the Department and Respondent.

19. Respondent acknowledges that it has actual notice of the contents and requirements of the SFO and that failure to fulfill any of the requirements hereof would constitute a violation of this SFO and subject Respondent to payment of civil penalties pursuant to Paragraph 14.d above.

20. Any stipulated civil penalty imposed pursuant to Paragraph 14.d shall be due upon written demand. Stipulated civil penalties shall be paid by check or money order made payable to the "State Treasurer, State of Oregon" and sent to: Business Office, Department of Environmental Quality, 811 S.W. Sixth Avenue, Portland, OR 97204. Within 21 days of receipt of a "Demand for Payment of Stipulated Civil Penalty" Notice from the Department, Respondent may request a hearing to contest the Demand Notice. At any such hearing, the issue shall be limited to Respondent's compliance or non-compliance with this SFO. The amount of each stipulated civil penalty for each violation and/or day of violation

is established in advance by this SFO and shall not be a contestable issue.

21. Providing Respondent has paid in full all stipulated civil penalties pursuant to Paragraph 20 above, this SFO shall terminate 60 days after respondent demonstrates full compliance with the requirements of the schedule set forth in Paragraph 14.a above.

RESPONDENT

_____ _____
Date (Name)_____
(Title)_____

DEPARTMENT OF ENVIRONMENTAL QUALITY

_____ _____
Date Fred Hansen, Director

FINAL ORDER

IT IS SO ORDERED:

ENVIRONMENTAL QUALITY COMMISSION

_____ _____
Date Fred Hansen, Director
Department of Environmental Quality
Pursuant to OAR 340-11-136(1)

CITY OF WOODBURN

270 Montgomery Street • Woodburn, Oregon 97071 • 982-5222

September 21, 1994

OPW27874.FM

Mr. Jaime Isaza
Project Officer, Water Quality
Western Region
1102 Lincoln, Suite 210
Eugene, Oregon 97401

Dear Mr. Isaza:

Subject: October 1992 SFO Schedule Extension Request

This letter is written as the formal request for a schedule extension to the Stipulated Final Order dated October 1992 corresponding to the current permit. The City of Woodburn requests an extension of 8 months for submission of the Draft Final Woodburn Wastewater Treatment Plant Facilities Plan for approval by the Department of Environmental Quality (DEQ). Instead of November 18, 1994, DEQ would receive the submission July 18, 1995. In the meeting with DEQ on July 7, 1994, a schedule extension was discussed. DEQ requires a written request and substantiation of the request.

Substantiation for the request includes:

1. Incorporating regional issues into the facilities plan

2. Wastewater Advisory Committee (WAC) requests for detailed involvement in development of the facilities plan

3. Desire for additional public involvement through another Open House before the Public Hearing

4. Evaluation of the additional alternatives, Sidestream Elevated Pool Aeration (SEPA) and poplar tree irrigation

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5. Ensuring compliance with the new DEQ facilities plan requirements

6. Negotiation of permit conditions during the facilities planning process

1. Regional Issues

The City of Woodburn, with verbal consent from DEQ, embarked on a regional study to determine the demand for regional laboratory, solids handling, septage receiving, and landfill leacheate disposal. The study has been supported by 13 surrounding communities. Initially, a 2-month delay was expected to allow the results of the study to provide input into the selected alternative planning, but the final product has been delayed by 3 months. It is expected to be presented to the WAC on September 20, 1994.

2. Wastewater Advisory Committee (WAC) Involvement

The WAC has been meeting since August 1993. They have made recommendations on the issues regarding flows and loads, evaluation of alternatives, selected treatment alternative, and effluent disposal. The process is working well. It resulted in a well attended public open house on the alternatives, regional issues, and other environmental issues. The WAC wants to have additional detailed involvement in the facilities plan costs and rates analysis, which was beyond the original concept. In addition, they are still exploring new options they believe may be cost effective, such as SEPA and poplar tree irrigation. This detailed involvement will require additional time.

3. Additional Public Involvement through Another Open House

The scope and cost of the project is greater than anticipated and has required extensive public involvement. During the first phase of the facility planning process prior to final TMDLs being issued, a public meeting was held. After TMDLs were issued a Public Open House was held. The open house, held on the regional issues and "big picture alternatives", was attended by more than 100 people. A second open house is desired to present the selected alternative and the potential impacts on the sewer rates. This additional open house will require more time in the schedule. These two open houses are provided in addition to the mandatory public hearing on the Facility Plan.

4. Additional Alternatives

Two additional alternatives were introduced for investigation after the preferred alternative was selected. The Sidestream Elevated Pool Aeration (SEPA) issue was first introduced at the June 21, 1994, WAC meeting. The Mayor introduced it and he showed how

the SEPA system saved Chicago from having to put in costly treatment plant upgrades. The City then met with DEQ to discuss the feasibility of this alternative and a time extension. In the meeting on July 7, 1994, DEQ agreed that SEPA warranted more exploration and a suitable time extension would be granted. The SEPA issue has delayed the schedule by 3 months already. Poplar tree irrigation was introduced by the wastewater advisory committee as an irrigation alternative they would like to investigate.

The WAC will meet on September 20, 1994, to accept the regional service study report, discuss poplar tree irrigation, and at that time may recommend whether SEPA should be studied further. If there are further studies (these studies took over 2 years in Chicago's application of SEPA), the additional studies would be conducted outside of the facility plan process.

5. DEQ Facilities Plan Requirements

The new version of the DEQ facilities plan requirements were issued on August 9, 1994. These need to be reviewed with DEQ and incorporated into the facilities plan. Of major significance are the flow and load calculations that the remaining work incorporates in the planning process.

6. Negotiation of Permit Conditions

The negotiation of permit conditions has been delayed by the SEPA investigation. Permit negotiations and discussions about related issues were intended to start at the July 7, 1994, DEQ meeting. These negotiations are critical to the development of the selected alternative.

Conclusion

It appears that proper preparation of the Facilities Plan in a period of 12 months for a larger size project, such as Woodburn's, is unrealistic. We believe that the additional time at this stage of the process is absolutely necessary and it will not only make the facilities plan better, it will make the plan more acceptable to the community. A revised schedule showing the project tasks and milestones is attached. Under the revised schedule, DEQ would receive the Draft Final Woodburn Wastewater Treatment Plant Facilities Plan by July 18, 1995. In addition, as recommended by DEQ in the August 30, 1994 meeting, we request that a predesign submittal be added to the schedule, 4 months after facilities plan approval by DEQ.

We propose the SFO be modified as follows:

14a(1) Twenty months after establishment of the TMDL, the Respondent shall submit a final draft facilities plan report that evaluates alternatives for complying with the TMDL, percent removal requirements for BOD and TSS, the water quality standards for chlorine and ammonia, and all other applicable water quality standards. The facility plan shall also include an evaluation of the mixing zone. After receiving written comments from the Department on the final draft facilities plan, the Respondent shall submit a final facility plan within 90 days.

14a(2) Four (4) months after approval of the facilities plan, Respondent shall submit a predesign report. Eighteen (18) months after approval of the facilities plan, the Respondent shall submit engineering plans and specifications for construction of necessary improvements.

We look forward to working with you during the facilities planning process to ensure the preparation of an acceptable submission for your approval.

Sincerely,

Frank Tiwari, P.E.
Public Works Director

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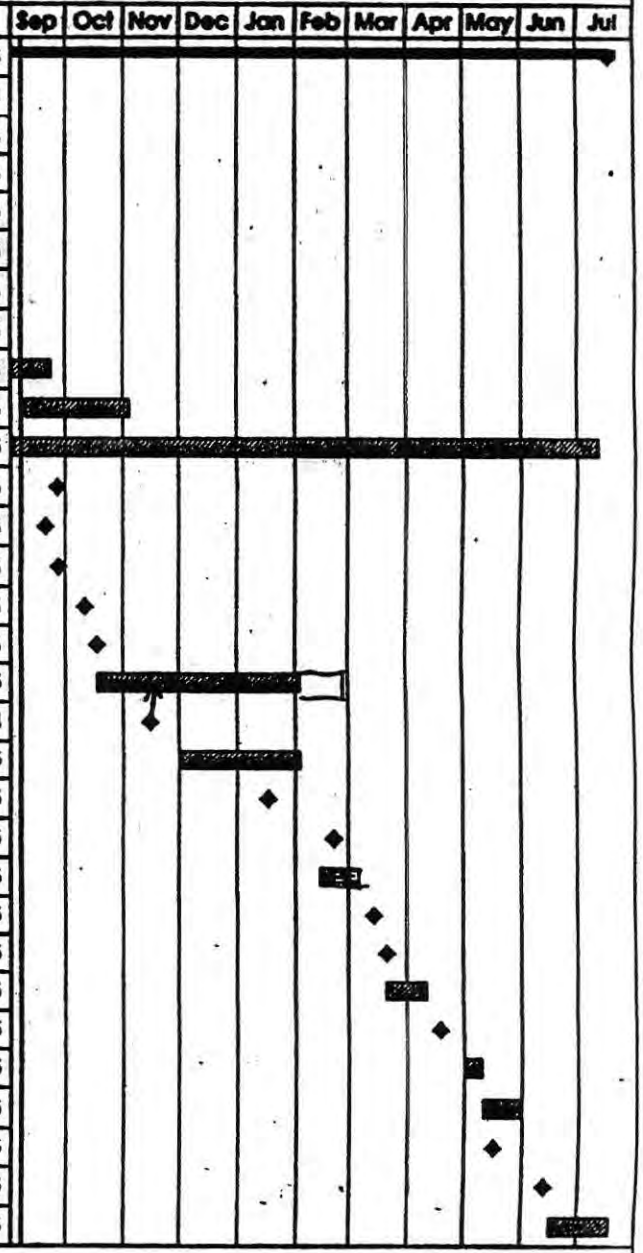
cc: City Council and Mayor
City Administrator
Daria Wightman
Frank Sinclair
Dean Morrison
Mark Hamlin
David Mann

Modified Schedule for Woodburn Facility Plan

ID	Name	Duration	Schedule	Scheduled
1	A. CONTINUE WOODBURN FACILITY PLAN	434d	11/18/93	7/18/95
2	Release of TMDLs by the DEQ(see previous Schedules)	0d	11/18/93	11/18/93
3	SEPA System introduced at WAC meeting	0d	6/21/94	6/21/94
4	Select Alternative (Council Decision)	16d	6/6/94	6/27/94
5	Met with DEQ on Selected Alternative and SEPA	0d	7/7/94	7/7/94
6	SEPA Conceptual Paper to DEQ	0d	8/5/94	8/5/94
7	Develop New flows based on David Mann Memo	14d	8/15/94	9/1/94
8	WAC/DEQ/FW Meeting on SEPA	0d	8/16/94	8/16/94
9	Flows/Loads Meeting with DEQ and Formal Extension Requet	19d	8/30/94	9/23/94
10	Develop Selected Alternative (Liquid, Solids, and Storage)	41d	9/9/94	11/4/94
11	Permit Negotiations with DEQ	227d	8/30/94	7/12/95
12	Rate Philosophy Introduction in Council Meeting	0d	9/26/94	9/26/94
13	WAC Mtg on Regional Service, Tree Irrigation, and Flows and Lc	0d	9/20/94	9/20/94
14	WAC/Council Workshop on Rate Setting Philosophy	0d	9/27/94	9/27/94
15	WAC Meeting on Selected Alternative Issues and Phasing	0d	10/11/94	10/11/94
16	WAC Meeting on Selected Alternative Preiminary Costs (No Bio:	0d	10/18/94	10/18/94
17	Conduct Financial Analysis and Rate Study	79d	10/18/94	2/3/95
18	WAC Meeting on Final Cost Estimates	0d	11/15/94	11/15/94
19	Prepare Draft Facility Plan Document	47d	12/1/94	2/3/95
20	WAC/Council Workshop on Preliminary Rates	0d	1/17/95	1/17/95
21	WAC Meeting on Preliminary Rate Adjustments	0d	2/21/95	2/21/95
22	City and WAC Review Facility Plan (incl rates)	6d	2/14/95	2/21/95
23	Council Workshop to Present Preliminary Plan Document	0d	3/14/95	3/14/95
24	Public Open House	0d	3/21/95	3/21/95
25	Incorporate City, WAC and Open House Comments	16d	3/21/95	4/11/95
26	Council/WAC Workshop on Draft Facility Plan	0d	4/18/95	4/18/95
27	Public Hearing on Draft Facility Plan	7d	5/2/95	5/10/95
28	Incorporate Public Input; Prepare Final Draft Document	15d	5/11/95	5/31/95
29	WAC Reviews Final Draft Facility Plan Comments	0d	5/16/95	5/16/95
30	Council Approves Final Draft Facility Plan	0d	6/12/95	6/12/95
31	Submit Final Draft Facility Plan to DEQ	23d	6/16/95	7/18/95



Project: City of Woodburn
Date: 9/6/94

Critical — Progress — Summary
Noncritical — Milestone — Rolled Up

Appendix C

Pudding River
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Total Maximum Daily Load Program
State of Oregon Department of Environmental Quality
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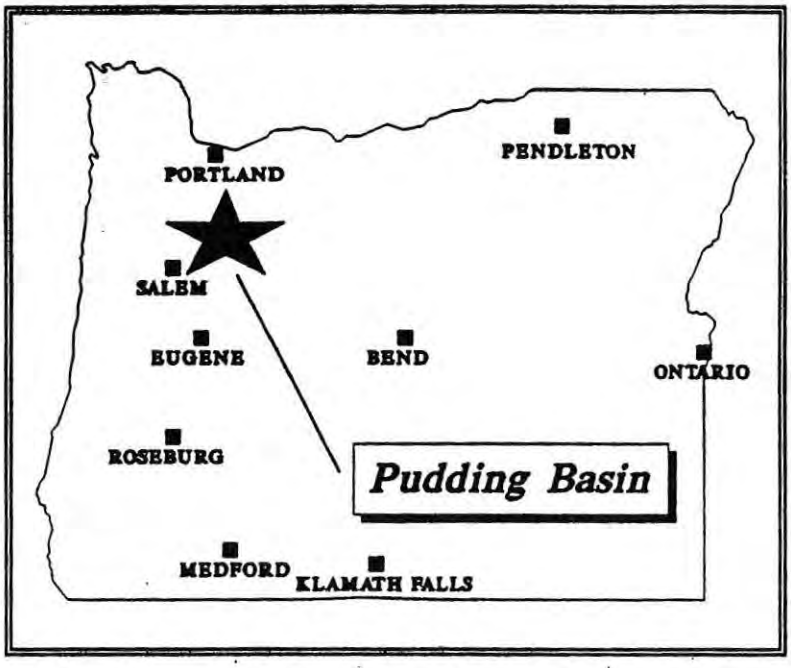
Total Maximum Daily Load Program



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Portland, Oregon 97204

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August 1993

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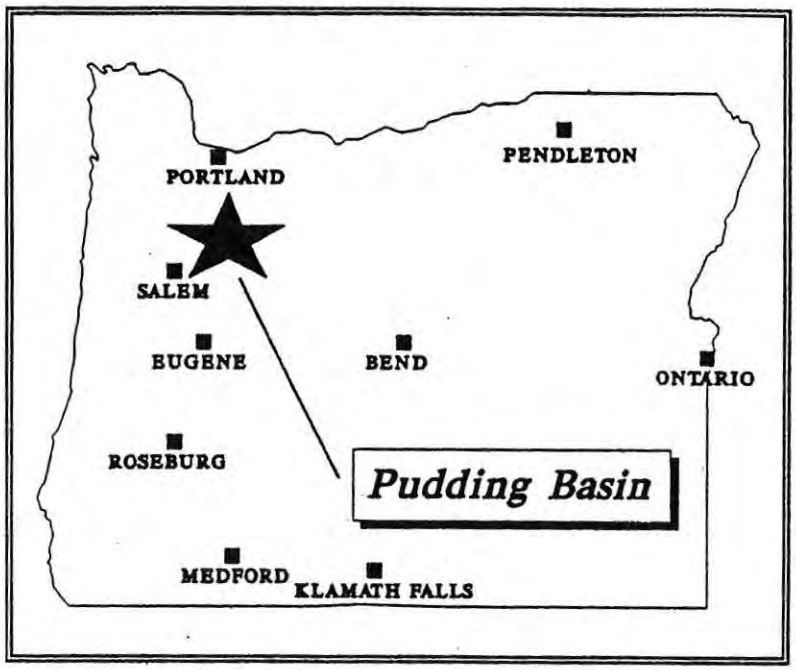
Water Quality Report

Total Maximum Daily Load Program



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Department of Environmental Quality
Standards & Assessments Section
811 Sixth Avenue
Portland, Oregon 97204

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Pudding River

Water Quality Report

Total Maximum Daily Load Program

This report describes the work that the Oregon Department of Environmental Quality (DEQ) has conducted to address water quality concerns in the Pudding River. The assessment is part of the Total Maximum Daily Load (TMDL) process within DEQ's Water Quality Program and reflects the State's water-quality-based approach to water quality problems.

For more information on the State's TMDL Program, contact:

Robert Baumgartner

TMDL Program Manager
Department of Environmental Quality
811 S.W. Sixth Avenue
Portland, OR 97204
(503) 229-5877

For questions regarding Permit Wasteload Allocations, contact:

Dick Nichols

Municipal Projects Manager
Department of Environmental Quality
811 S.W. Sixth Avenue
Portland, OR 97204
(503) 229-5323

To receive additional copies of this or other TMDL reports, contact:

Elizabeth Thomson

Technical Reports Editor
Department of Environmental Quality
811 S.W. Sixth Avenue
Portland, OR 97204
(503) 229-5358

ACKNOWLEDGMENTS:

Neil Mullane ..... **Standards & Assessments Manager**
Bonnie Smith ..... **Layout Design and Typesetting**
Viola Cinotto ..... **Program Support**

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Oregon's Total Maximum Daily Load Program

OVERVIEW

BENEFICIAL USES

The quality of Oregon's streams, lakes, estuaries, and groundwaters is monitored by the Department of Environmental Quality (DEQ). The information collected by DEQ is used to determine whether water quality standards are being violated and, consequently, whether the **beneficial uses** of the waters are being threatened. The beneficial uses include fisheries, aquatic life, drinking water, recreation, shellfish, irrigation, hydroelectric power, and navigation. Specific State and Federal rules are used to determine if violations have occurred: these rules include the *Federal Clean Water Act of 1972*, Oregon's Revised Statutes (ORS), and Oregon's Administrative Rules (OAR Chapter 340).

WATER QUALITY LIMITED STREAMS AND TOTAL MAXIMUM DAILY LOADS

The term **water quality limited** is applied to streams and lakes where required treatment processes are being used but violations of water quality standards occur. With a few exceptions, such as in cases where violations are due to natural causes, the State must establish a **Total Maximum Daily Load**, or **TMDL** for any waterbody designated as water quality limited. A **TMDL** is the total amount of a pollutant (from all sources) that can enter a specific waterbody without violating the water quality standards.

WASTELOAD AND LOAD ALLOCATIONS

The total permissible pollutant load is allocated to point, nonpoint, background, and future sources of pollution. **Wasteload allocations** are portions of the total load that are allotted to point sources of pollution, such as sewage treatment plants or industries. The wasteload allocations are used to establish effluent limits in discharge permits. **Load allocations** are portions of the total load that are attributed to either natural background sources, such as soils, or from nonpoint sources, such as agricultural or forestry activities. Allocations can also be set aside in reserves for future uses.

TMDL PROCESS

The establishment of TMDLs is required by Section 303 of the Clean Water Act. The process of establishing a TMDL includes studying existing data, collecting additional data to answer specific questions, using mathematical models to predict the effects of changes in wasteloads, evaluating alternative strategies for implementation, and holding public hearings and allowing public comment on the TMDL.

PURPOSE OF THIS REPORT

This report provides information on one of the waterbodies in Oregon's TMDL Program. The report includes background information on the drainage basin, the pollution sources, and the applicable water quality standards; a summary of the monitoring data and the technical analyses; and a discussion of the current pollution control strategy.

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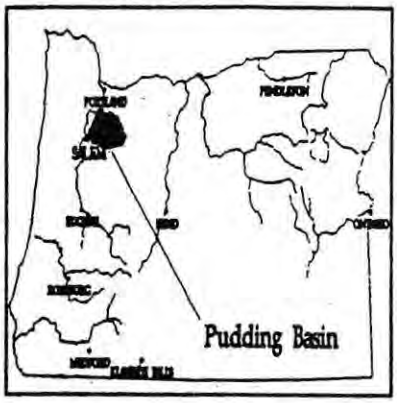
Pudding River

WQ CONCERNS AT A GLANCE:

Water Quality Limited?	*Yes*
Segment Identifiers:	*22K-PUDD*
Parameter of Concern:	*Dissolved Oxygen*
Uses Affected:	*Aquatic Life*
Known Sources:	*Point Sources — POTWs**
	Nonpoint Sources — Agriculture, Forestry



BACKGROUND INFORMATION

The Pudding River is located in northwestern Oregon in the Willamette Valley near Salem. The river originates in the low Waldo Hills and flows sluggishly in a northerly direction for 62 miles. It follows a meandering channel with little slope, flowing past the communities of Silverton, Mt. Angel, and Woodburn. Along the way, many tributaries, such as Butte, Bear, Abiqua, and Silver Creeks flow into the Pudding River. Tne Pudding empties into the Molalla River, which flows into the Willamette River near Wilsonville at river mile 36.

The Pudding River Basin covers 480 square miles and forms roughly the western half of Marion County. Agriculture is the predominant land use in the drainage basin; water from the Pudding River is used primarily for irrigation to maintain the basin's high agricultural productivity. The basin supports a warm-water game fishery and provides recreational opportunities for the residents of Marion County. Steelhead and spring chinook salmon use the Pudding as a migration route to reach tributary streams. Salmon are not known to spawn in the mainstem Pudding River; it is considered a "non-salmonid-producing" stream.

*Publicly-Owned Treatment Works

WATER QUALITY CONCERNS

The Pudding River watershed has been impacted by development since pioneers settled the Willamette Valley in the mid-19th century. Over the past few decades, there have been water quality concerns related to dissolved oxygen, bacteria, nutrients, and habitat degradation due to sedimentation. The Pudding is a naturally turbid stream, most likely due to its flat gradient and the types of soils in the watershed.

Segments of Concern

Segments within the Pudding drainage appear in Oregon's water quality standards as part of the Willamette Basin. Four segments in the Pudding drainage have been identified as water quality limited in Oregon's 1992 Statewide Water Quality Status Assessment Report:

Segment	Name	Boundaries
22K-PUDD	Pudding River	R.M. 0 – 30
22K-PUDD	Pudding River	R.M. 30 – 50
22K-PULI	Little Pudding River	R.M. 0 – 5
22K-ZOLL	Zollner Creek	R.M. 0 – 5

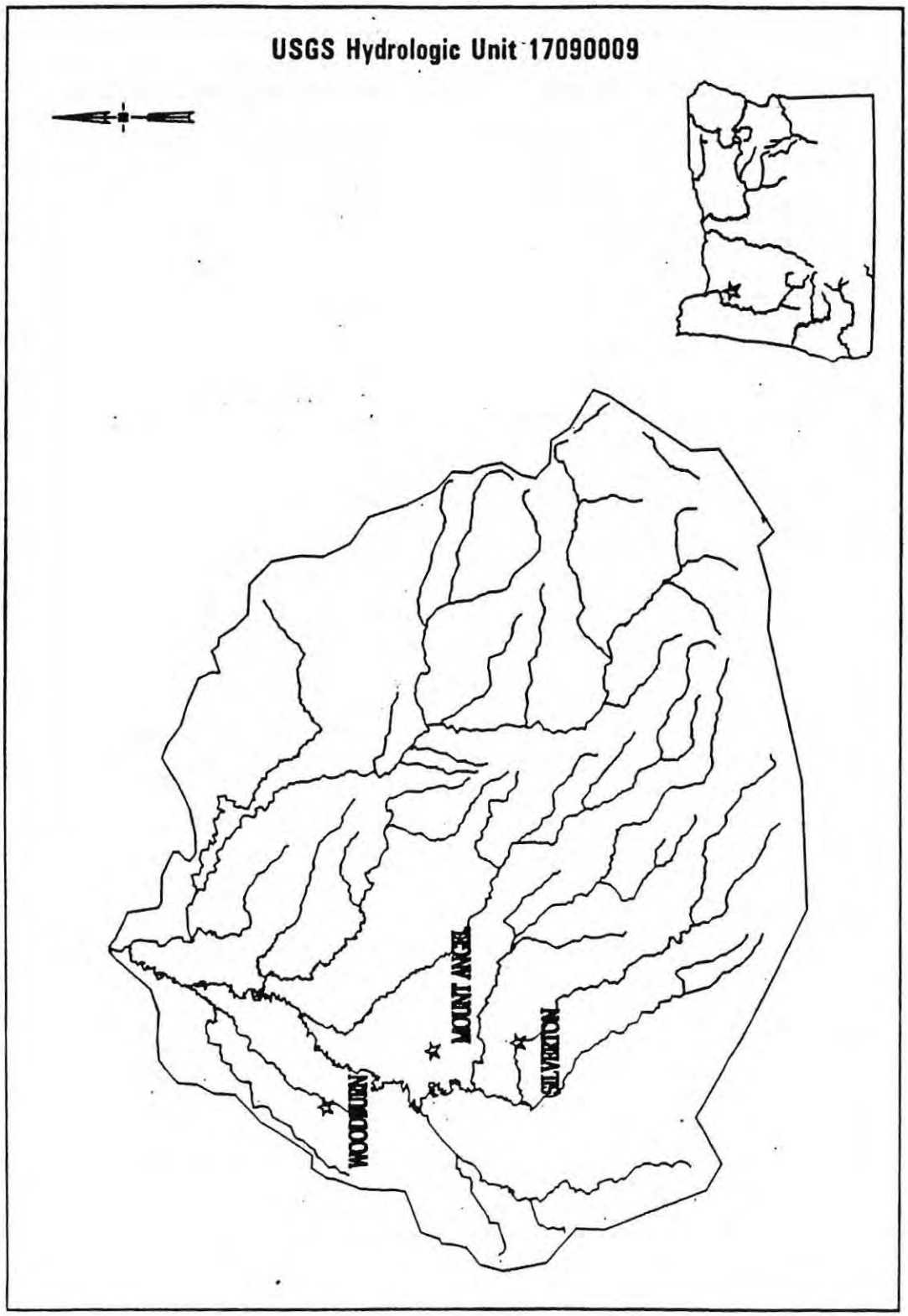


Figure 1. Pudding River Drainage

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Beneficial Uses Affected

The designated beneficial uses of the Pudding River system are identified in Oregon's Administrative Rules (OAR). Uses include water supply, aquatic life, recreation, and aesthetics. As reported in the latest Statewide Water Quality Assessment (the 1992 305(b) Report), the beneficial use found to be most at risk in the Pudding system is aquatic life; it is listed as not supported for river miles 0 to 30 of the Pudding, 0 to 5 of the Little Pudding, and 0 to 5 of Zollner Creek. Water-contact recreation is also listed as not supported. Criteria used to evaluate use support are described in Appendix B.

Applicable Water Quality Standards

A number of water quality parameters, including dissolved oxygen and bacteria, have criteria values which have been adopted as regulatory standards for the Willamette Basin.

Dissolved oxygen is a critical parameter for the protection of aquatic life. The applicable dissolved oxygen criteria for the basin are:

- Salmonid Rearing: 90 percent of saturation.
- Warm-Water Fish (non-salmonid): 6.0 mg/L.

The lower mainstem Pudding River has been identified as a non-salmonid stream; therefore, the 6.0 mg/L criterion applies.

Bacterial water quality standards have been established for protection of water-contact recreation (e.g., swimming). Current standards for bacterial pollution are:

- Water-Contact Recreation:

  33/100 ml (Enterococci).
  200/100 ml (Fecal coliform).

Available Monitoring Data

The Pudding River has been monitored periodically since 1957, with the most extensive monitoring occurring between 1966 and 1975. An early key site was located near Canby; this site was sampled routinely from 1966 to 1975. In Water Year 1980, ambient monitoring was resumed at the Highway 99E Bridge. The purpose was to determine general water quality trends, compliance with the beneficial uses of the river, and any needs for more detailed study. In addition, this site near the mouth of the Pudding gives information on loading to the Willamette River.

Because of concerns related to low dissolved oxygen and high bacterial levels, the Department of Environmental Quality (DEQ) initiated more extensive data collection in 1989. Monitoring sites were located on all major tributaries and on the mainstem. Major point sources were also monitored during intensive sampling efforts. Results of the data collection efforts are presented in Appendix C.

Parameters of Concern

The Pudding River has been identified as water quality limited due to violations of the dissolved oxygen standard. Ambient water quality monitoring indicates that the river experiences periodic low levels of dissolved oxygen during the summer months. During the summer, point source discharges of pollutants have a major influence on water quality in the basin. A total maximum daily load (TMDL) has been prepared to address the dissolved oxygen problem. Fecal coliform levels also exceed standards in some segments. However, the Pudding has not currently been designated for TMDL development to address the bacteria concerns; the effectiveness of other regulatory mechanisms first needs to be evaluated.

POLLUTANT SOURCES

Water quality in the Pudding drainage is affected by both point and nonpoint source discharges. Point sources include several municipal wastewater treatment plants, as well as a food-processing facility. Major nonpoint sources include runoff from both agriculture and forestry activities.

Point Sources

Sewage treatment plants are located at Woodburn, Mt. Angel, Silverton, Molalla, Gervais,

and Hubbard. An additional point source is the Agripac cannery. These sources hold National Pollutant Discharge Elimination System (NPDES) permits. Additional point sources land-irrigate their effluent. Mallorie's Dairy holds a no-discharge Water Pollution Control Facilities (WPCF) permit.

Nonpoint Sources

Nonpoint and background sources can also contribute to water quality problems. Runoff from agricultural land provides a significant load of biochemical oxygen demand, bacteria, ammonia, and organic nitrogen to the Pudding River and its tributaries. Forestry activites in the basin also contribute to nonpoint source loads.

ACTIONS TO DATE

Preliminary TMDLs for the Pudding River were established in 1988. Sources and the public were notified and provided a chance to comment. Based on additional data collection and mathematical modelling, wasteload allocations have been developed for the City of Woodburn and Agripac (see Appendix F).

POLLUTION CONTROL STRATEGY

In February 1993, point-source dischargers were notified of the proposed wasteload allocations and allowed to comment. A public hearing was held to allow for public comment. Final allocations will be issued as permit conditions for the sources, and will be implemented as such. (See Tables 1 and 2 below.) In some instances, compliance with conditions may require construction of new facilities or land application of effluent.

Although nonpoint sources, predominantly agricultural, contribute to the pollutant loads in the Pudding and its tributaries, point sources are believed to account for the majority of the controllable load. It is anticipated that water quality standards will be achieved by implementing the proposed wasteload allocations for point sources. Interagency agreements between DEQ and the Departments of Agriculture and Forestry will be used to promote Best Management Practices designed to reduce nonpoint sources of pollution in the basin and to eliminate impacts on beneficial uses.

APPENDICES

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APPENDIX B — APPLICABLE WATER QUALITY STANDARDS

APPENDIX C — AVAILABLE MONITORING DATA

APPENDIX D — POLLUTANT SOURCE SUMMARY

APPENDIX E — TECHNICAL ANALYSIS AND TMDL DEVELOPMENT

APPENDIX F — PERMIT WASTELOAD ALLOCATIONS

Table 1. Wasteload Allocations (Effluent Limits) for Woodburn & Agripac

Parameter	Daily Maximum (mg/L)	Monthly Average (mg/L)	Minimum Daily Average (mg/L)
Carbonaceous Oxygen Demand ($CBOD_5$)	20	10	—
Total Suspended Solids	20	10	—
Dissolved Oxygen in Effluent	—	—	6.5
Ammonia Nitrogen (NH_3-N)	Based on Streamflow & Month (See Table 2)		

Table 2. Wasteload and Load Allocations — Ammonia-Nitrogen and UBOD

Month	Pudding River Monthly Average Flow (cfs)	Ammonia-Nitrogen (mg/L)		Ultimate Biochemical Oxygen Demand (lb/d)
		Agripac	Woodburn	Nonpoint Sources and Background
June	>150	18	6.8	3,234
	100 – 150	15	5.7	2,156
	50 – 100	8.5	3.1	1,078
	<50	3.2	1.1	647
July and August	>100	10	3.2	2,156
	60 – 100	2.7	1.0	1,294
	30 – 60	1.5	0.50	647
	<60	0.30	0.10	323
September and October	>100	WLAs do not apply.		
	60 – 100	18	6.8	1,294
	30 – 60	9.0	3.3	647
	<30	4.0	1.6	323

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APPENDIX A

EXPANDED BACKGROUND INFORMATION

General

The Pudding River is located in northwestern Oregon in the Willamette Valley near Salem. The topography of the area is generally level with gently rolling hills. The Pudding River is about 62 miles long with numerous tributaries, most notably the Silver, Abiqua, Bear, Little Pudding, and Butte Creeks. The Pudding River itself has 55 miles in typical flat valley drainage (sloping approximately 3 feet per mile) with mixed agricultural and urban land use. The upper 7 miles is more characteristic of foothill drainage with a steeper gradient (79 feet per mile). Overall, the Pudding is a low gradient, sluggish river.

The study area has a temperate marine climate, with cool, wet winters and warm, dry summers. About sixty percent of the annual precipitation occurs between November and February; about ten percent occurs between June and September. Average annual precipitation varies depending on elevation, from a low of 40 inches up to 130 inches. Winter temperatures usually stay above freezing but can drop as low as 10 degrees; summer temperatures normally range from 74 to 82 degrees, but can reach as high as 110.

The drainage area for the Pudding River is 480 square miles. Average monthly flows for the Pudding range from 63 cubic feet per second (cfs) (about 40 million gallons per day) in the summer to about 2,600 cfs in the winter. Streamflow responds to both rainfall and to snowmelt; the snowmelt maintains high flows into late spring. There is a marked decline in streamflow during the summer months, which impacts fisheries, recreation, irrigation, and assimilation of wastes from sewage treatment plants.

Marion County's Comprehensive Plan designates the Pudding River as warm-water habitat. According to the Oregon Department of Fish and Wildlife, many species of fish use the Pudding River year round. These include largemouth bass, sunfish, bullhead catfish, carp, suckers, and sculpins. Seasonal salmonid runs include coho salmon (September to November), steelhead (December to May), chinook salmon (April to July), and cutthroat trout (January to May). The upstream tributaries and the upstream portions of the Pudding are more suited for spawning than the lower Pudding due to their steeper gradients and better oxygenation.

The entire length of the Pudding is zoned for Exclusive Farm Use (EFU) and its land-use designation is Primary Agriculture. The mild climate and wide variety of soils in the region support many different crops including grass seed, beans, corn, wheat, oats, barley, hay, hops, onions, berries, cherries, walnuts, and filberts. Agriculture and associated food-handling and processing plants are major employers in the area. The Nature Conservancy has identified natural areas at the Pudding River Marshland (wildlife/bottom land) and at the confluence of the Pudding River and Silver Creek (wildlife habitat). Recreation areas are located west of Silverton (the Pudding River Picnic Area) and west of Mt. Angel (the Evergreen Golf Club).

The economy of Marion County is largely dependent on farming and forestry. Land use in the county is predominantly agriculture and forests (47 and 43 percent, respectively), with 4 percent urban, 2 percent grazing, and 3 percent parks and conservation. With respect to nonpoint sources, agriculture is the predominant land use impacting the water quality in the Pudding Basin, particularly on the smaller tributaries. Municipal sewage treatment plants and several industries are significant point sources.

Water Quality Concerns

Development has occurred in the Pudding Basin since pioneers settled the Willamette Valley in the

mid-19th century. The Pudding is a naturally turbid stream, most likely due to its flat gradient and the types of soils in the watershed. Over the past few decades, there have been water quality concerns with respect to dissovled oxygen, bacteria and nutrients. Habitat degradation due to sedimentation is an additional concern. The Pudding has been designated as water quality limited due to low dissolved oxygen levels and high levels of bacteria.

APPENDIX B

APPLICABLE WATER QUALITY STANDARDS

The Pudding River has been designated as water quality limited. The parameters of concern are dissolved oxygen and bacteria. Within the State of Oregon, water quality standards are published pursuant to Oregon Revised Statute (ORS) 468.020. Authority to adopt rules, regulations, and standards as are necessary and feasible to protect the environment and health of the citizens of the State is vested with the Environmental Quality Commission. Through the adoption of water quality standards, Oregon has defined the beneficial uses to be protected in each of its drainage basins and the criteria necessary to protect those uses.

Segments of Concern

Segments within the Pudding drainage are included in Oregon's water quality standards as part of the Willamette Basin. Within the Pudding, four segments have been identified as water quality limited in Oregon's Statewide Water Quality Status Assessment Report [1992 305(b) Report]. These segments are:

Segment	Name	Boundaries
22K-PUDD	Pudding River	R.M. 0 – 30
22K-PUDD	Pudding River	R.M. 30 – 50
22K-PULI	Little Pudding River	R.M. 0 – 5
22K-ZOLL	Zollner Creek	R.M. 0 – 5

Ambient water quality monitoring data have shown that the Pudding River as well as portions of several tributaries are water quality limited due to periodic low levels of dissolved oxygen and high levels of bacteria.

Beneficial Uses Affected

Oregon Administrative Rule (OAR) Chapter 340, Division 41, Rule 442, lists the beneficial uses for which water quality will be protected in the Pudding Basin. These are identified in Table B-1. This list of beneficial uses was established by the Oregon Water Resources Commission pursuant to direction given in Oregon Revised Statute (ORS) 536.300. As charged by ORS 468.020, the Oregon Environmental Quality Commission adopted rules and standards that were necessary to protect those recognized beneficial uses. In practice, water quality rules and standards have been set at levels to protect the most sensitive of the uses: aquatic life and human health.

Assessment activities have determined that aquatic life and water-contact recreation are not fully supported in the segments of the Pudding River listed under *"Segments of Concern"*. Aesthetics are listed as partially supported due to elevated levels of nutrients; however, no excessive algal growth has been noticed. Criteria by which supportiveness was evaluated are described in Table B-2.

Applicable Water Quality Standards

A number of water quality parameters have criteria values which have been adopted as regulatory standards for the Pudding Basin. Included are temperature, turbidity (also referred to as total suspended solids or TSS), pH (a measure of acidity), dissolved oxygen, fecal coliform bacteria, and dissolved chemical substances. The primary parameter of concern for the Pudding River is dissolved oxygen. Other parameters of concern are bacteria and nutrients.

Dissolved Oxygen: The Oregon Department of Fish and Wildlife has identified the lower mainstem of the Pudding River as providing passage for warm-water game fish but not providing for salmonid production. The Oregon Water Quality Standard for dissolved oxygen

Table B-1. Beneficial Uses to be Protected in the Willamette Basin

BENEFICIAL USES	
Public Domestic Water Supply[1]	Resident Fish & Aquatic Life
Private Domestic Water Supply[1]	Anadromous Fish Passage
Industrial Water Supply	Salmonid Fish Rearing
Irrigation	Salmonid Fish Spawning
Livestock Watering	Fishing
Boating	Wildlife & Hunting
Water-Contact Recreation	Commercial Navigation & Transportation
Aesthetic Quality	Hydroelectric Power

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

Source:

Oregon Administrative Rules, Ch. 340, Division 41 — DEQ Table 6, Willamette Basin.

Table B-2. Beneficial Use Support Criteria

FISHERIES AND AQUATIC LIFE	
Partially Supported	10% exceedence of basin DO mg/L or DO % Saturation standard.
	10% exceedence of basin pH standard.
Not Supported	25% exceedence of Oregon Administrative Rule (OAR) basin standard for DO mg/L or DO % Saturation.
	25% exceedence of the basin OAR pH standard.
WATER CONTACT	
Partially Supported	10% exceedence of Enterococcus upper-range standard.
Not Supported	25% exceedence of Enterococcus upper-range standard.
AESTHETICS	
Partially Supported	10% exceedence of 15 μg/L chlorophyll *a*.
	25% exceedence of 0.1 mg/L total phosphorus.
Not Supported	25% exceedence of pH standard for basin.

Source:

Oregon's 1992 Water Quality Status Assessment Report (305(b)) Report; pp. B3—6.

in the Pudding River states that: "The dissolved oxygen concentration shall not be less than 6.0 mg/L" [OAR 340-41-445(2)(a)(E)(ii)]. The standard represents the minimum value that the stream should not fall below at any time so that the beneficial uses of aquatic life, fisheries, and salmonid migration will be protected. Because the standard is stated as an absolute value, the total maximum daily load (TMDL) is calculated to attain 6.0 mg/L as a minimum. To account for the fact that dissolved oxygen will vary with the time of day due to the effects of sunlight, measured dissolved oxygen concentrations are reported as daily *averages* so that data are comparable from day to day. To maintain a *minimum* value of 6.0 mg/L, the *average* value will have to be higher to allow for daily variation and still achieve the standard. Diurnal measurements collected in the critical oxygen-sag area of the Pudding (above Agripac) were used to estimate a daily variation of 0.5 mg/L in the dissolved oxygen measurements. To allow for a variation of 0.5 mg/L above or below the measured value, a daily average of 6.5 mg/L must be maintained to achieve a minimum value of 6.0 mg/L.

Bacteria: OAR 340-41-445(2)(e)(A) provides a freshwater limit of "... a log mean of 200 fecal coliform per 100 milliliters based on a minimum of five samples in a 30-day period with no more than ten percent of the samples in that period exceeding 400 per 100 ml."

Nutrients: The basin standard limits pH to the range of 6.5 to 8.5 [OAR 340-41-445(2)(d)(B)]. DEQ's action level for chlorophyll *a* is 15 μg/L. The State also uses EPA's criteria of 0.1 mg/L for total phosphorus.

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APPENDIX C

AVAILABLE MONITORING DATA

Robert Baumgartner, Water Quality, DEQ

The Pudding River has been monitored periodically since 1957, with the most extensive monitoring occurring between 1966 and 1975. An early key site was located near Canby; this site was sampled routinely from 1966 to 1975. In Water Year 1980, ambient monitoring was resumed at the Highway 99E Bridge. The purpose was to determine general water quality trends, compliance with the beneficial uses of the river, and any needs for more detailed study. In addition, this site near the mouth of the Pudding gives information on loading to the Willamette.

Four DEQ monitoring stations are located on the Pudding River for routinely collecting instream water quality data. Ambient data are stored in a computerized database called STORET. The ambient monitoring stations located in the Pudding River are identified in Table C-1. Several intensive water quality surveys were conducted during the summer of 1989. The data were compared to the regulatory standards to determine if violations had occurred. In addition, ambient and intensive data were used in mathematical models to predict water quality impacts during varying conditions, such as changing flow or temperature.

Intensive Surveys

Dissolved Oxygen: Monitoring results are summarized in Table C-2 for several parameters for the summer season of 1989. Summer is the season of primary concern due to the low-flow conditions in the Pudding River during that time. Table C-2 lists the median values (values which fall in the middle of the data set) and the regulatory standard for each parameter.

Diurnal (24-hour cycle) monitoring for dissolved oxygen was conducted for three-day periods during the summer and fall of 1989 using automated monitoring devices left in the stream for the full sampling period. The data provided by the monitors were used to develop equations which allowed data from samples collected at any time of the day to be converted to a *minimum* value for that day. Because of the natural variation in some parameters throughout the course of a day, this conversion allowed data to be more accurately compared to data from other days. On August 15, 1989, the estimated minimum dissolved oxygen value for the area between river mile 23.5 and river mile 17.2 was 5.3 mg/L. Both the observed minimum value and the estimated daily-average value were 5.9 mg/L, which is below the standard.

As can be seen in Table C-2, the standard for dissolved oxygen was violated in the Pudding River. The dissolved oxygen violations observed in 1987 were more frequent and severe than those observed during summer surveys in 1989. Minimum observed values fell to near 5.0 mg/L in the 1987 surveys. Observed violations occurred below the Agripac and Woodburn Sewage Treatment Plant (STP) discharges.

The low dissolved oxygen measurements in the Pudding River usually occurred in the early morning hours. These low readings might be explained in part by daily fluctuations in algal growth and respiration levels, since algal activity and consumption of oxygen is greatest in the morning. It does not appear, however, that the growth of algae in the Pudding River is excessive or usually results in nuisance conditions. Nuisance growth may be prevented by the relatively high levels of suspended solids and turbidity in the Pudding River which limit the amount of light available for growth of algae, or it may be prevented by other natural conditions.

Nitrogenous oxygen demand is the primary factor leading to the observed violations of the oxygen standard in the Pudding River. Organic nitrogen

Table C-1. Ambient Monitoring Stations in the Pudding River

Site Location	River Mile	STORET #
Highway 213	R.M. 49.9	402213
Mt. Angel / Brooks	R.M. 40.7	402560
Highway 211	R.M. 22.9	402317
Highway 99E	R.M. 8.1	402594

Table C-2. Water Quality Summary for the Pudding River Summer 1989

Parameter	Applicable WQ Standard or Criteria	Hwy 99E (R.M. 8.1)	Hwy 211 (R.M. 22.9)	Mt. Angel / Brooks (R.M. 40.7)	Hwy 213 (R.M. 49.9)
Dissolved Oxygen a.m.	6.0	6.5	5.5	6.6	4.7
Dissolved Oxygen p.m.	6.0	8.1	8.4	8.5	8.0
BOD_5	*	1.0	0.8	1.2	1.8
Total Phosphorus	0.1	0.23	0.43	0.09	0.065
Turbidity	**	4.0	6.0	3.5	–
Total Susp. Solids	*	110	90	61	–
Fecal Coliform	200	93	80	195	240
Ammonia (NH_3)	***	0.05	0.07	0.03	0.06
Nitrite-Nitrate Nitrogen (NO_2-NO_3)	*	1.40	1.40	0.55	0.44

- Values are reported as medians.
- BOD_5 represents the five-day biochemical oxygen demand.

LEGEND:

* No applicable standard.

** Standard allows an increase of up to 10% above background.

*** Standard is dependent on pH, temperature, and toxicity; turbidity may impact ambient dissolved oxygen levels.

Units:

- Turbidity as JTU.
- Fecal coliform bacteria as MPN (most probable number of colonies per 100 milliliter).
- All others as mg/L (milligrams per liter).

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and ammonia enter the stream from both point source discharges and nonpoint source runoff. Nitrogenous demands result from the conversion (or nitrification) of organic nitrogen to ammonia (nitrogen plus three hydrogens, NH_3) to nitrite (nitrogen plus two oxygens, NO_2^-) to nitrate (nitrogen plus three oxygens, NO_3^-). The oxygen that becomes associated with the nitrogen is no longer available to fish as dissolved oxygen.

Data collected at the monitoring station at Highway 211, below the Woodburn Sewage Treatment Plant discharge, showed an increase in ammonia and nitrate and a decrease in oxygen, indicating the effect of the STP effluent on the stream. Other low oxygen levels in the upper section of the river, along with relatively high concentrations of BOD, indicate a significant impact from nonpoint sources.

Observed Loads: Point source loads to the Pudding River, as observed during the intensive sampling trip in August 1989, are shown in Table C-3. The observed loads resulted in daily average dissolved oxygen values of 5.9 mg/L during the intensive survey. Flows were above 30 cubic feet per second (cfs) and instream temperatures approached 23 degrees. Observed temperatures in the Pudding have exceeded 27 degrees during July and August in previous years. Minimum streamflows (7Q10) are estimated at 15 cfs. Observed minimum flows during 1989 were less than 20 cfs at Highway 211; minimum dissolved oxygen during 1989 was 5.1 mg/L.

Fecal Coliform Bacteria: The presence of fecal coliform bacteria is commonly used as an indicator of pathogen contamination in surface waters. Elevated levels of fecal coliform bacteria have been observed in the Pudding River. The violations of the standard for fecal coliform bacteria appear to be related to nonpoint sources, particularly in the upper basin (see Table C-2, Highway 213).

Stream Processes — General Information

The dissolved oxygen concentration in a stream results from a balance of processes which consume oxygen and processes which restore oxygen. Fish and other desirable aquatic organisms require a high level of dissolved oxygen to survive. Dissolved oxygen is restored mostly from the atmosphere (reaeration) and from photosynthesis. It is depleted mostly by the activity of bacteria which break down organic matter (particularly by the decay of algae) and by chemical processes such as the conversion of ammonia to nitrate (nitrification).

Pollutant loads are typically described in terms of their biochemical oxygen demand (BOD) or their chemical oxygen demand (COD). The BOD test determines the amount of oxygen required by bacteria to decompose the load of organic matter in a sample of water. The COD test measures the amount of oxygen required to convert both biologically-available and non-biologically-available organic matter to carbon dioxide and water. The BOD test is generally more representative of actual instream conditions. Results can be obtained much more quickly with the COD test, however, which makes it valuable in certain situations such as a waste spill.

Table C-3. Observed Loads to the Pudding River (August 1989)

Source	Flow (cfs)	Observed During Intensive Sampling		
		UCBOD	TSS	Ammonia
Agripac	2.17	130	100	17
Woodburn	2.54	205	200	18
Source	**Flow (cfs)**	**Typical Loads — Estimated**		
		UCBOD	**TSS**	**Ammonia**
Agripac	1.91	300	100	25
Woodburn	2.54	270	200	68

If the pullutant load on a waterbody is light, the replenishment of oxygen can make up for the loss. This is referred to as assimilation. If the load is heavy, oxygen may be depleted to a point where fish cannot survive and aerobic organisms are destroyed. A stream's ability to assimilate waste is largely determined by its concentration of dissolved oxygen. As oxygen is depleted, anaerobic organisms, which can live without oxygen, will take the place of the aerobic organisms, resulting in odors and nuisance conditions. The oxygen-depleted water may travel a considerable distance before natural purification processes can restore the oxygen levels.

Temperature will also influence the dissolved oxygen concentration in a stream. The maximum possible concentration of dissolved oxygen in water (referred to as the saturation level) is largely determined by the water temperature. A stream's ability to process oxygen-demanding loads (its assimilative capacity) is greater at lower temperatures because dissolved oxygen saturation is greater at lower temperatures. This allows an extra reserve during colder weather. Conversely, when temperatures are higher, the stream has a reduced capacity to process wastes..

For example, cold water at 15°C (59°F) can hold up to 10.1 mg/L of dissolved oxygen. After meeting the minimum of 6.0 mg/L required by water quality standards, the stream would have a reserve assimilative capacity of 4.1 mg/L. In contrast, warm water at 24°C (75°F) can hold only 8.4 mg/L of dissolved oxygen, allowing a reserve capacity of only 2.4 mg/L above the minimum standard of 6.0 mg/L. This reduction in assimilative capacity at warmer temperatures and low flows limits the amount of waste which can be tolerated and may prohibit discharge.

Because of the effects of seasonal differences in temperature and streamflow on a stream's assimilative capacity, wasteload limits will be set by month for varying flow and temperature conditions as necessary to meet water quality standards. The summer limits will typically be the most restrictive, with greater discharge allowed during the winter when flows are high and temperatures are low.

APPENDIX D

POLLUTANT SOURCE SUMMARY

Robert Baumgartner, Water Quality, DEQ

Water quality in the Pudding drainage is affected by both point and nonpoint source discharges. Point sources include several municipal wastewater treatment plants, as well as industrial sites including food-processing facilities. Major nonpoint sources include runoff from both agricultural and forestry activities.

Point Sources

The main point sources of pollution in the Pudding River Basin are listed in Table D-1. They include municipal, industrial, and agricultural sources. The point sources which discharge directly to the Pudding River are required to have National Pollutant Discharge Elimination System (NPDES) permits. Within the Pudding Basin, current NPDES permits are issued to the Silverton, Woodburn, Molalla, Mt. Angel, Gervais, and Hubbard sewage treatment plants, and the Agripac cannery. Additional point sources are required to land-irrigate their effluent or may be discharging without a permit. Mallorie's Dairy holds a no-discharge Water Pollution Control Facilities (WPCF) permit.

Municipal Sources: *WOODBURN* — The major point-source discharge to the Pudding River is the City of Woodburn's sewage treatment plant (STP). Dilution of the effluent is provided by the river. The amount of dilution will vary as river flow changes throughout the year but must stay within permit limits.

Permit limits for Woodburn have not been stringent enough to provide for adequate dilution of effluent. Woodburn has been allowed by permit to discharge 3.1 million gallons per day (mgd) or 4.8 cubic feet per second (cfs) during the summer. A seven-day-average low flow of approximately 50 cfs occurs every other year in the Pudding River near Aurora. Based on that flow and Woodburn's discharge volume, a dilution ratio of 10.4 can be calculated. According to the guidelines of the Oregon Water Quality Standards [OAR 340-41-375(1)(c)], a dilution ratio of 15 is required. The existing dilution ratio of 10.4 during critical summer low flows is thus not adequate; under those conditions, the discharge would exceed dilution requirements by fifty percent. New TMDL-based permit limits require reduced oxygen-demanding loads from Woodburn to ensure adequate dilution ratios during low flows and to prevent oxygen sags.

HUBBARD — The City of Hubbard discharges to Mill Creek, a tributary to the Pudding River. Mill Creek enters the Pudding River below the area where water quality violations occur. Establishing TMDLs on the Pudding River should not affect Hubbard's NPDES permit. DEQ has little or no information describing the impact of this discharge on water quality; an intensive mixing-zone survey designed to evaluate permit conditions for the Hubbard STP needs to be conducted.

MOLALLA — The City of Molalla discharges to Bear Creek, a tributary to the Pudding River. DEQ has little or no information on the impact of this discharge on Bear Creek. The City has two options in its discharge permit: discharge to Bear Creek or use the effluent for irrigation water (land apply). Molalla currently land applies its effluent during the critical summer months. There does not appear to be a reason to discontinue land application, and as long as application continues, no wasteload allocation is required for Molalla.

MT. ANGEL — The City of Mt. Angel discharges to a small creek which is a tributary to the Pudding River. The City has elected to discontinue discharging during the summer low-

Table D-1. Point Sources in the Pudding River Basin

Facility	Permitted Discharge Quantity (Summer)	Location (River Mile)	Type of Waste
MUNICIPAL			
Silverton STP	1.0 mgd	Silver Creek to Pudding River	Domestic Sewage
Hubbard STP	0.34 mgd	Mill Creek to Pudding River	Domestic Sewage
Molalla STP	0.79 mgd	Bear Creek to Pudding River at R.M. 10	Domestic Sewage
Woodburn STP	3.1 mgd	Pudding River at R.M. 27	Domestic Sewage
Gervais STP	No Summer Discharge Allowed	Pudding River at R.M. 30.5	Domestic Sewage
Mt. Angel STP	No Summer Discharge Allowed	Pudding River at R.M. 34	Domestic Sewage
INDUSTRIAL			
Agripac, Inc.	2.0 mgd	Pudding River at R.M. 27	Fruit/Vegetable Waste
Avison Lumber	No Discharge	Bear Creek to Pudding River at R.M. 16	Log-Yard Runoff
Mt. Angel Meat	No Discharge	Zollner Creek	Processing Waste
AGRICULTURAL			
Mallorie's Dairy	WPCF Permit; No Discharge Allowed	Pudding River and Silver Creek	Manure, Milk-Processing Waste

flow period. The streamflows under which Mt. Angel may discharge and the accompaning effluent limits are defined in the City's discharge permit.

SILVERTON — The City of Silverton discharges to Silver Creek, a major tributary to the Pudding River. These loads do not appear to influence the dissolved oxygen violations observed below Woodburn. Observed dissolved oxygen values are above the 6.0 mg/L standard identified for the lower river, but have fallen below the 90 percent saturation level required for salmonid waters. The STP is currently operating under a Stipulated and Final Order which identifies discharge limits.

GERVAIS — The City of Gervais is not allowed to discharge during the summer. No wasteload allocation will be given.

Industrial Sources: *AGRIPAC, INC.* — Agripac discharges processed cannery waste to the Pudding River. Agripac's discharge, in combination with the City of Woodburn's discharge, results in violations of water quality standards under current conditions. The TMDLs and wasteload allocations require reductions in the oxygen-demanding loads from Agripac.

MT. ANGEL MEAT — The stream closest to Mt. Angel Meat is Zollner Creek. No discharge from Mt. Angel Meat is allowed; no wasteload allocation will be given.

AVISON LUMBER — Avison Lumber holds a general log-pond permit which does not allow discharge. No wasteload allocation will be given.

Agricultural Sources: *MALLORIE'S DAIRY* —

Mallorie's Dairy has been observed to discharge a high-strength waste stream to the Pudding River. Although these discharges did not occur at the times when field samples were taken (in conjunction with critical low flows), analysis suggests that if a discharge of that type were to occur during a low-flow period, violations of water quality standards would result. Mallorie's Dairy does not have an NPDES permit; the dairy has a WPCF permit which does not allow discharge at any time. The dairy will not be given a wasteload allocation. Assurances must be made that discharges will not occur.

Nonpoint Sources

Interagency agreements between DEQ and the Departments of Agriculture and Forestry will be used to promote Best Management Practices designed to reduce nonpoint source pollution in the Pudding and its tributaries.

Nonpoint-source runoff from agricultural land provides a significant load of biochemical oxygen demand, bacteria, ammonia, and organic nitrogen to the Pudding River and its tributaries. These agricultural loads potentially contribute to water quality problems and impacts on beneficial uses in the tributaries and could contribute to violations of the dissolved oxygen standard in the mainstem of the Pudding River.

A reduction in the amount of nitrogen and other oxygen-demanding materials from nonpoint sources needs to occur not only in the Pudding but also in its tributaries. Loads coming from tributary streams such as Zollner Creek have as much impact as the minor STP discharges on water quality in the Pudding River. Dissolved oxygen violations have been observed in both Zollner Creek and the Little Pudding River although no major point sources are located on those streams.

Additional concerns have been raised by resource agencies and individuals regarding nonpoint sources in the Pudding Basin. The Oregon Department of Fish and Wildlife has stated that sediment in the river is degrading fish habitat. Agricultural interests are concerned with apparent toxicity in the Little Pudding River. Problems due to sediment, toxicity, nutrients, and bacteria should be addressed in the nonpoint source plans for the basin.

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APPENDIX E

TECHNICAL ANALYSIS AND TMDL DEVELOPMENT

Robert Baumgartner, Water Quality, DEQ

Water Quality Modelling

QUAL2E, a steady-state, hydrodynamic model, was used to study the impact of wasteloads on instream water quality and the effects of varying streamflow and weather conditions. The model was used to predict daily average values of dissolved oxygen based on measured (observed) data. In addition to data from monthly monitoring, two detailed data sets were used for the modelling efforts. The data were collected during two intensive surveys which covered the area of the stream from just above the two major discharges to below the area of low dissolved oxygen (referred to as the dissolved oxygen sag). A third survey indicated that dissolved oxygen was not a concern when streamflow was high.

During the surveys, dissolved oxygen was measured along with several parameters which affect the level of oxygen in the stream: biochemical oxygen demand (BOD), nutrients, total suspended solids, ammonia and nitrate (used for determining the rate of nitrification), and temperature. These parameters were measured going downstream (longitudinally). Automated monitors which were left in place in the stream measured dissolved oxygen, temperature, and pH continually for three days and were used to determine the daily variability in dissolved oxygen. Knowing the variability in dissolved oxygen with respect to time made it possible to compute daily averages from the observed dissolved oxygen values. Since dissolved oxygen varies with the time of day, these corrections were necessary for accurate modelling.

Wasteloads and tributary loads were also monitored. Dye tests (time-of-travel tests) were used to estimate velocity as a function of flow. Knowing the velocity, it is possible to convert a change in concentration with distance to a change in concentration over time. In calculating flow-related TMDLs, concentration as a function of time is used to predict concentrations under varying flow conditions.

An initial laboratory measurement for the decay of organic matter (loss of BOD) was used as the starting point for calibrating the model. Decay rates and temperature coefficients were adjusted to fit the observed data for the loss of BOD and ammonia and for the increase in nitrates. The dissolved oxygen sag can be explained as an effect of the input of ammonia and BOD (which depletes oxygen) and reaeration (which replenishes oxygen). Reaeration is modelled as a function of stream velocity, depth, and turbulence using the O'Connor and Dobbins method (1958).

The first set of survey data was used to establish the initial conditions for the model (calibration). The second set of data was used to test whether the model could successfully predict dissolved oxygen under different background conditions (verification). Observed data for parameters such as upstream concentrations, flows, wasteloads, tributary loads, and weather conditions were entered into the model. The values which the model computed for dissolved oxygen were compared to the actual values of dissolved oxygen which were observed during the field survey. The values predicted by the model were found to reasonably match the observed values.

Once an acceptable model was established, the model was used to calculate wasteload and load allocations. There may be several sets of wasteload and load allocations that will achieve water quality standards for the Pudding River. The modelling approach allows alternative scenarios to be evaluated with respect to their

impact on water quality. Different sets of values for streamflow, sunlight, temperature, turbulence, and boundary conditions (upstream loads and tributary loads) were entered into the model. The model calculated the level of dissolved oxygen which should be present under those conditions.

Initial modelling assumed an equitable distribution of wasteload allocations between the major sources (Agripac and Woodburn) and similar permit conditions for efficiency of waste removal. Alternative sets of wasteload allocations for varying flow conditions were entered into the model to determine the resulting levels of dissolved oxygen. Nonpoint source inputs were added to the model to test their effect on dissolved oxygen. The modelling process was repeated until the resulting dissolved oxygen concentrations met water quality standards and satisfied TMDL requirements. A margin of safety was added into the calculations to allow for inherent errors in measurements and modelling.

Parameter Estimation

Hydraulics Estimates: DEQ conducted several "dye studies" to determine the time of travel (TOT) for two sections of the lower portion of the Pudding River. Results varied in terms of accuracy. The dye tests demonstrate that measured velocities are very similar in the lower Pudding (between R.M. 27 and 15) for flows below 60 cfs. Hydraulic barriers, such as the numerous debris dams and the remnant concrete-sill dam (named Falls #1), act to impede velocity and flow during low-flow conditions. Velocity estimates varied both by the subsection for which the estimates were made, by multiple dye drops within the study reach, and by calculation of velocity between sub-reaches. For the set of points a to b to c, the velocity between b and c is calculated by:

$$\frac{[RM\ (a\ to\ c) - RM\ (a\ to\ b)]}{[TOT\ (a\ to\ c) - TOT\ (a\ to\ b)]} * \frac{ft}{mi} * \frac{hours}{second}$$

where TOT equals time of travel in hours. Average flow for a given reach (Q) was calculated as:

$$[\{Q_{upstream} + Q_{downstream}\}/2].$$

Flow estimates were derived from stage-discharge curves empirically developed for several locations on the Pudding River. The variation in flows in Table E-1 is primarily due to the different locations that were sampled during the dye tests.

The stage-discharge curves were developed using three to five representative flow and discharge measurements. For several of the stations, including key tributaries, an adequate number of measurements were not collected. Similarly, flows were not taken at all sampling sites during the dye studies. The hydraulic relationships are difficult to estimate with the available data.

Table E-1. Time-of-Travel Estimates for the Pudding River

Date	Location: River Mile	Flow (cfs)	Velocity (ft/s)	Comments
06/21/89	27 – 6.1	129 – 202		Unreliable
08/01/89	27 – 17.6			
09/26/89	17 – 8.1	22.5 – 49	0.17 – 0.24	Lower Section
10/03/89	27 – 17.5	62.5 – 65	0.28 – 0.46	Multiple Dye Drops

Power Functions: Power functions, developed by Leopold and Maddox, provide an empirical relationship between physical stream factors and streamflow. Several alternative power functions were calculated using different approaches. The resulting data were used in calibrating the model (see Table E-2).

Recognizing that streamflow is the product of cross-sectional area and velocity and that cross-sectional area is the product of width and depth, it can be shown that the sum of the exponents (n+m+f) is 1.0. Plotting the $Log_{base\ 10}$ of Q with the $Log_{base\ 10}$ of the physical stream factors of velocity, depth, and width provides the information for defining the equations. From the plots, the slope provides the power term (n,m,f), and the intercept at Q = 1 provides the remaining term (a,b,c). These relationships apply to free-flowing streams. Impounded reaches in rivers have exponents of m and f equal to zero. It is therefore appropriate to develop site-specific data. The availability of data for empirically developing the power functions is limited, however.

Using all of the available data, the power functions were estimated as:

$V = 0.028\ Q^{0.654}$ Using three dye tests below R.M. 27.

$D = 1.15\ Q^{0.293}$ Using stage-discharge curves near R.M. 27.

$W = 70\ Q^{0.053}$ Width observed at stage sites.

Considerations:

- The stage-discharge curves may not provide an accurate estimate of the depth relationship. The locations used for flow measurements were selected for high-velocity profiles and therefore occurred at free-flowing areas with constrictions, such as bridge crossings.
- The high-flow data for velocity are suspect at best. Very minor meter response was used as the "dye peak." It may not be appropriate to rely on this data to empirically determine the power functions.

The low-flow data appeared to provide a much different relationship than that observed during the high-flow dye study. The low-flow data provided a much flatter response with respect to velocity. The low-flow power functions (for flows less than or equal to 70 cfs) as estimated are:

$V = 0.089\ Q^{0.36}$ Low-flow period.

$D = 0.410\ Q^{0.59}$

$W = 0.750\ Q^{0.05}$

These low-flow power functions are appropriate for the Pudding River for flows at or below 70 cfs between R.M. 27 and 15. The single representative dye test below R.M. 15 resulted in slower velocities than estimated by the above power function. The velocity function was adjusted to predict the observed slower velocities in the lower river by changing the "a" term to 0.05, resulting in $V = 0.05\ Q^{0.36}$.

No dye tests were conducted above R.M. 27. The channel morphology and flow characteristics of the Pudding River do not change dramatically above where the dye tests were

Table E-2. Power-Function Values Used in Calibrating the Model

Equation	Typical Range for the Power Term
Velocity, $V = aQ^n$	0.5 (0.4 to 0.6)
Depth, $D = bQ^m$	0.4 (0.3 to 0.5)
Width, $W = cQ^f$	0.1 (0.0 to 0.2)

conducted. The primary differences are an increase in slope and the influence of several major tributaries which enter above R.M. 27.

Power functions were estimated using Manning's equation. Manning's equation was developed with the data available for the Pudding below R.M. 27. The equation was modified for the increase in slope (0.000405) above R.M. 27 as defined by contours on USGS quadrangle maps. The Manning's equation estimate should provide a representative estimate of the flow relationships. From this modified equation, the power functions defined were:

$V = 0.13\ Q^{0.38}$ Above R.M. 27.

$D = 0.40\ Q^{0.57}$

$W = 60.0\ Q^{0.05}$

Similarly, Manning's equation was used to estimate the power functions for Silver Creek, which receives loads from the Silverton STP. The slope of Silver Creek is 0.004781 ft/mi. The estimated power functions are:

$V = 0.49\ Q^{0.45}$ Silver Creek.

$D = 0.20\ Q^{0.45}$

$W = 12.0\ Q^{0.10}$

The estimates for Silver Creek are very rough and provide only a relative index of the flow relationships. The load from Silver Creek does not appear to greatly influence the substandard section of the Pudding River below R.M. 27. However, for calculating the TMDL, is it necessary to include all major point sources in the basin.

Flow Balance: The flow balance for the Pudding River was empirically developed using observed relationships between monitoring sites, available flow statistics (from USGS), and flows estimated using drainage basin area, stream miles, location in the drainage, and altitude at the reference site.

The site at Highway 211 was used as the initial reference site. Highway 211 is located in the water-quality-limited stream section where most of the water quality violations have been observed. This was also the site where flow was monitored most frequently by DEQ.

Flow at Highway 99 (Aurora) was estimated from the regression equation developed using observed flows at Aurora coincident and dependent on observed flows at Highway 211. Flows for the Pudding River near Mt. Angel and Silver Creek were estimated from historical records. For these regressions, it was assumed that the critical low flows (i.e., 7Q10) occurred coincidentally throughout the basin. From these statistics the estimates for Silver Creek, Butte Creek, and the Upper Pudding were made dependent on observed flows at Aurora (Highway 99).

Estimates for other streams (Butte Creek, Little Pudding River, Zollner Creek, Bear Creek, Abiqua Creek) were made using regressions developed using flows dependent on land area, stream length, location in the basin, and altitude of the reference location for known gauges. Flows for creeks without gauges were extrapolated from these regression equations. Flows for Butte Creek were modified for additional flow that would occur below the gauge site at Monitor.

Permitted point-source flows were calculated as the four-month average that occurred from July to October 1989, reported as monthly averages on the discharge monitoring reports. For Silver Creek, the predicted flow value includes the flow from the sewage treatment plant. All remaining flow estimates are additive. No attempts were made to adjust for irrigation withdrawals.

Input flows were balanced with observed and predicted flows at the three reference locations in the Pudding River. Flows not accounted for were then calculated and termed "overland flow." Overland flow varied from both positive to negative values. Minor flow modifications were proportioned out from the tributary stream estimates to prevent negative overland flow values. It is possible that these negative values are the result of irrigation. However, since the negative values occurred at higher flows, it seems likely they are a result of overestimating ungauged streamflows. This process allows us to identify specific inputs for

desired streamflow statistics.

Sediment Oxygen Demand: Sediment oxygen demands (SOD) are a significant component of the oxygen balance in the Pudding River during summer low-flow conditions. The calibrated SOD rate was 0.25 grams/ft^2-day (0.112 grams/m^2-hr or 2.7 grams/m^2-day).

Table E-3 summarizes other model-derived SOD rates compared to measured rates as discussed in Terry and Morris (1986). Terry and Morris suggest that the indirect method (calibration) may provide a more realistic measurement of oxygen demand than measuring individual points of SOD *in situ*.

Similarly, Whittemore (1986) found a poor correlation (r^2 = 0.58) between field and laboratory measurements: *in situ* measurements were consistently higher at low levels of SOD; the reverse was observed at high levels of SOD. Such errors indicate the need for improved methods for estimating SOD. A summary of rates measured *in situ* by Whittemore (1986) is presented in Table E-4.

EPA suggests that *in situ* methods are more credible than laboratory methods at this time. Ranges for SOD reported in EPA (1985) are shown in Table E-5.

The model-calibration method for estimating SOD is subject to a reasonable range for SOD values. The SOD range estimated for the Pudding River appears to fall within the ranges observed for other streams.

Laboratory Tests for BOD Conversion: Determining which term, or component, of BOD is being referred to in reported BOD measurements can be confusing. The DEQ laboratory

Table E-3. Sediment Oxygen Demand Rates in Other Streams (Reference: Terry & Morris, 1986)

Stream Name	Calibrated Range (mean)	Measured Range (mean) g/m^2-day
Osage Creek	0.5 to 15 (5.9)	0.65 to 0.94 (0.7)
Illinois River	2.4 to 6 (3.8)	0.08 to 1.82 (0.8)
White River	0.7 to 11 (6.7)	1.20 to 6.00 (3.1)
Spring Creek	1.0 to 18 (8.6)	0.66 to 1.58 (0.9)
Muddy Fork	2.8 to 4 (3.3)	0.70 to 3.20 (1.8)

Table E-4. In Situ Rates of Sediment Oxygen Demand (Reference: Whittemore, 1986)

Stream Name	Measured Range (mean) g/m^2-day
Androscoggin River	0.2 to 1.18 (0.74)
Penobscott River	1.1 to 4.15 (3.04)
Presumscott River	1.5 to 6.4 (4.0)

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Table E-5. Ranges of Sediment Oxygen Demand (Reference: EPA, 1985)

River Locations	Measured Range (g/m^2-day)	Comments
Upper Wisconsin	30.022 to 0.91	Sullivan
Eastern U.S.	0.09 to 0.87	NCASI
Four Eastern Rivers below Paper Mills	2.0 to 33 and 0.9 to 14.1	Both ranges from NCASI; different measuring techniques
North Illinois	0.27 to 9.8	Butts and Evans
Eastern Michigan	0.10 to 5.3	Chiaro and Burke
New Jersey	1.1 to 12.8	Hunter, et al.
Sweden	0.3 to 1.4	Edburg and Hofsten
Spring Creek	1.7 to 6.0	McDonnell and Owens
England	1.5 to 9.8	Rolley and Owens
— Streams —	4.6 to 44	James

routinely monitors five-day BOD. Laboratory incubations were used to measure the conversion between $CBOD_5$*, BOD_5, UBOD, and UCBOD. From these relationships, it appears that BOD_5 provides a weak relationship to UCBOD. However, the typical values fall near the default of 66 percent of BOD_5 as UCBOD. Some data were collected for both BOD_5 and UBOD during the August 15 and October 25 (1989) surveys (Table E-6). No effort was made to separate out NBOD as calculated by concentration of ammonia.

Table E-6. BOD Measured During 1989 Monitoring Surveys

Parameter	Edge of Woodburn Mixing Zone	QA for Edge of Woodburn	Upstream of Agripac	Below Agripac
BOD_5	3.2 mg/L	3.0 mg/L	1.0 mg/L	2.0 mg/L
% of UBOD	36%	31%	27%	15%
% of UCBOD	88%	93%	66%	55%
% of $CBOD_{20}$	---	---	---	66%
$CBOD_5$	1.5 mg/L	1.3 mg/L	0.3 mg/L	1.2 mg/L
% of UCBOD	41%	40%	40%	33%
% of UBOD	17%	13%	13%	9%
$NBOD_5$	1.3 mg/L	1.4 mg/L	1.4 mg/L	0.5 mg/L
% of UNBOD	26%	29%	29%	8%
% of UBOD	17%	14%	14%	4%
% of BOD_{20}	---	---	---	5%
UCBOD	3.6 mg/L	3.2 mg/L	1.5 mg/L	3.6 mg/L
$NBOD_{20}$	5.7 mg/L	4.7 mg/L	1.3 mg/L	6.2 mg/L
UBOD	8.8 mg/L	---	3.6 mg/L	13 mg/L
$CBOD_{20}$	---	---	---	3.0 mg/L
BOD_{20}	---	---	---	9.3 mg/L

* BOD_{days} = biochemical oxygen demand, C = carbonaceous, U = ultimate, N = nitrogenous.

Application of Model

Pudding River TMDLs Refined Using QUAL2E: *Allocation Strategies* — Several alternative waste-load allocation strategies were considered during the modelling phase. Preliminary waste-load allocations relied on observed streamflows and temperatures during low-flow conditions and were therefore restricted to a limited number of observed flow and temperature regimes. To estimate TMDLs under other conditions, simulated temperatures for various flow conditions were used. The model was calibrated using data collected during intensive and ambient studies in August 1989.

Atmospheric data as measured near Salem were obtained from the NOAA National Climatological Data Center. Data from the date of sample collection and from the preceding two days were used as input to the model. The data in Table E-7 were used to develop allocations. Median values for barometric pressure were used; other data represents the 20th percentile of average monthly conditions for the last five years.

Observed instream temperatures in the Pudding River exceed 27°C (81°F) during summer low-flow conditions. The warm temperatures and low streamflow result in low assimilative capacity in the Pudding.

Summer is the critical period for allocations in the Pudding River. Allocations for the months of June and September are based on flows. A flow of 25 cfs at Highway 211 (the 14Q2) was used to calculate load allocations for the months of July and August. Although additional flow-based allocations may be developed, the assimilative capacity will not significantly increase even at higher flows.

Hydraulics — QUAL2E allows two methods for describing stream velocity, μ, as a function of streamflow, Q. The options are either Manning's equation or power functions. The power function option sets $\mu = aQ^b$, where a and b are empirically determined constants. Ambient dye tests were used to collect information for evaluating the empirical constants.

The dye tests suggest that for flows between 20 and 60 cfs, stream velocity near the point source discharges is similar (0.35 feet per second). Such a relationship would result in an equation where the "b" term is zero and the "a" term defines stream velocity independent of flow. The alternative model defines velocity as $0.35Q^0$.

The QUAL2E model input files have been modified to have constant stream velocity for the section of the Pudding below the point source discharges where the dye-test data indicated constant velocity below 60 cfs. The input files are only applicable for flows below 60 cfs.

Ammonia Decay — Ammonia decay is usually modelled as first-order decay. As described in U.S. EPA guidance manuals, ammonia decay is often modelled as having multiple steps for first-order decay. Multiple steps were used in the original model based on the observed instream decay rates. The observed and model-

Table E-7. Climate Data — Salem, Oregon

Month	Julian Day	Air Temp. (°F)	Wet Bulb Temp. (°F)	Barometric Pressure (mm Hg)	Wind Speed (mph)	Cloud Cover (Tenths)
June	168	64.0	50.6	29.83	6.5	5.0
July	198	66.8	51.7	28.85	7.1	3.2
August	229	67.8	52.7	29.81	6.5	3.6
September	260	62.7	49.3	29.81	5.5	3.8

Source: NOAA National Climatological Data Center.

calibrated decay rates are higher below the point sources of total Kjeldahl nitrogen in the Pudding and appear to decrease downstream from the sources. The decay rates are modelled as a series of first-order decay rates.

The question of concern for calculating TMDLs is whether the decay rates are a function of concentration. Literature indicates that the decay rate of ammonia may be influenced by several factors, including physical factors and substrate concentrations. For example, shallow streams with large bottom-surface-to-volume ratios have been observed to have high decay rates. Decay rates dependent on substrate concentrations may be explained by a Michaelis-Menton type of kinetics. The growth of bacteria may be dependent on the amount of substrate (food). As the amount of substrate increases, the population growth of bacteria increases. The growth continues until the growth requirements of bacteria are saturated.

If the decay rates are dependent on substrate concentration, then as the TMDLs are implemented and substrate is reduced, the resulting decay rates would be expected to be lower than the previously observed decay rates. The dissolved oxygen depression, and therefore the assimilative capacity, is determined by the combined effects of the rate of demand and the rate of reaeration. Reduced decay rates would alter the assimilative capacity of the Pudding River and would therefore influence the loading capacity and subsequent TMDLs.

If we assume that the decay rate is dependent on substrate, it is necessary to predict the decay rate to determine the TMDLs. To estimate the decay rate as a function of concentration in a particular section of the Pudding, the observed decay rate, $[\ln\{NH_3\}_{t0} - \ln\{NH_3\}_{tt}]/\text{Time}$, was plotted against observed concentration. The plot resulted in a linear equation of $-0.085 + 13.85\{NH_3\}$. Although the ammonia concentration appears in both the axes of the plot, it did provide an indication of the change in decay rate due to initial concentration.

Iterations — Point source allocations were calculated by iteration using QUAL2E. For example, a minimum dissolved oxygen value was calculated for an assumed set of wasteload allocations and a given flow regime. Calculations were repeated using different data for wasteloads and flows until the resulting dissolved oxygen value of 6.5 mg/L was achieved. It is estimated that maintaining a daily *average* of 6.5 mg/L will assure that the daily *minimum* level of dissolved oxygen will remain above the standard.

Loads — In the model, current volumes of waste discharge were used for the major sources:

- Silverton — 1.19 cfs (0.8 mgd)
- Agripac — 2.16 cfs (1.4 mgd)
- Woodburn — 2.54 cfs (1.64 mgd)

The value used for Silverton was its current discharge rather than its permit load. The discharge for Mt. Angel was assumed to be zero to be consistent with its no-discharge permit.

Loads for Agripac and Woodburn were assumed to have equal quantities of TSS, UCBOD, NH_3, and organic nitrogen. TSS was included to form a basis for estimating organic nitrogen loads. For this analysis, it was assumed that the discharged TSS was in the form of cells represented as $C_5H_7O_2N$ (molecular weight of 113). Nitrogen is stoichiometrically 12.39 percent bacterial cells by weight. A discharge of 20 mg/L TSS would yield 2.48 mg/L organic nitrogen.

Nonpoint Sources — The preliminary modelling efforts assumed an overall reduction of 25 percent for ammonia, organic nitrogen, and CBOD from nonpoint sources. The reduction for Zollner Creek was assumed to be 65 percent. The nonpoint source reductions would increase the available supply of dissolved oxygen in the Pudding River above the major point sources and would reduce the amount of oxygen-demanding pollution entering the critical portion of the river. If nonpoint sources are controlled, the assimilative capacity available for the point sources would be increased. If nonpoint sources are ***not*** controlled, then the wasteload allocations for the point sources would need to be reduced.

Initial Modelling Results: *Preliminary Allocations Assuming Equal Effluent, 25 Percent Nonpoint Source Reduction* — Table E-8 presents

Table E-8. Alternative Wasteload Allocations Assuming Equal Effluent and 25 Percent Reduction in Nonpont Source Loads

Month	Flow (cfs) at Hwy 211	Pounds Per Day By Source					
		AGRIPAC			WOODBURN		
		UCBOD	NH_3	TSS	UCBOD	NH_3	TSS
June	280	175	115	230	205	135	275
	200	175	88	230	205	200	275
	50	115	6	115	135	7	135
July	<50	115	4	115	135	4.5	135
August	<50	115	4	115	135	4.5	135
September	25	175	58	230	200	68	270
	60	175	80	230	200	95	270
	100	175	110	230	200	130	270

alternative allocations for achieving the standard of 6.0 mg/L dissolved oxygen. No margin of safety is provided in these allocations, and no allocation is made for future growth and development. A 25 percent reduction in nonpoint source loads is assumed.

Review of the data suggests that very little benefit would occur by increasing the flow ranges during July and August. The 60Q2 estimated for the Pudding River at Highway 211 is approximately 50 cfs. The low-flow allocations result in effluent limits of 0.325 mg/L of ammonia at current discharge levels. Because this limit is not realistically achievable, it is most likely that the major sources would be required to use a "no-discharge" alternative to meet this allocation. The no-discharge period would be expected to extend for two months per year.

Discussion of Other Model Runs with Modified Conditions: *Low-Flow and Maximum Warm Temperature* — Applying the updated hydraulics estimates and the assumption that ammonia decay rates are dependent on ammonia concentration significantly increased the assimilative capacity available for wasteloads during summer low-flow conditions. The model was used to estimate wasteload allocations for differing flow conditions, such as the 30Q2 (monthly average low flow), the 7Q2 (weekly low flow), and the 7Q10 (critical low flow) periods. Conditions for an average July weather pattern and for a "warm" period were also evaluated.

The analysis suggests that during the summer low flows (≤ 30Q2) and warm temperatures (maximum thermal input, July), stream temperatures will approach 25°C (77°F) from below the STP discharges to Aurora. Observed temperatures in the Pudding River immediately above the STP have been observed at 24°C in the afternoon and 22°C in the morning. The temperatures observed at Aurora approach 23.5° to 24°C in the morning and 27°C in the afternoon. The predicted temperatures appear to be reasonable estimates of the critical conditions that may exist during extended warm weather and low-flow conditions.

The analysis also suggests that very little, if any, assimilative capacity will exist at the critical low-flow warm-weather temperatures. Alternatives to application of critical low-flow wasteload allocations could include a no-discharge period for July and August when flows are below 35 cfs and daily averaged stream temperatures are near 25°C.

Artificial Reaeration — Representatives of Agri-

pac requested that DEQ review an alternative which relied on instream artificial reaeration of the Pudding River. The first assumption placed the artificial reaeration upstream of the discharges to provide greater assimilative capacity for the Pudding River. The aeration provided 150 lb/d of oxygen to attain near-saturation. No analysis was conducted to assure that supersaturation of gases other than oxygen would not occur. The second assumption placed an additional aerator of 150 lb/d of oxygen just below the Woodburn STP.

The analysis suggests that the effect of artificial reaeration would not be apparent for long distances below the point of application. Multiple appropriately-placed aerators could offset the oxygen demand placed on the stream. If placed effectively, greater wasteload allocations could be possible.

At this time, no wasteload allocations have been developed for any assumed level of artificial reaeration. A policy evaluation needs to be made to determine if reaeration would be a permissible approach for a point source wasteload allocation.

Nonpoint Source Load Allocations — Nonpoint sources are estimated as both tributary inflow and as overland flow. Analysis suggests that some relief in wasteload allocations may occur through effective nonpoint source controls.

Other Strategies — Because the oxygen-sag curves from Agripac and Woodburn overlap, the actions of one discharger could influence the alternatives available to the other. Although there may be other alternative strategies that would also be equitable, they would require extensive review.

Summary

Modelling efforts have focused on the area of low dissolved oxygen below the two major dischargers (Woodburn and Agripac) for the critical low-flow season extending from June through September. Based on the modelling, a TMDL has been developed which defines the wasteload allocations for point sources, the load allocations for nonpoint sources, and reserves. The analysis used for the final wasteload allocations assumed no significant modification to nonpoint source loads.

Time schedules and strategies for implementing the TMDLs have also been incorporated into the process. Monitoring will be continued to assure compliance and to verify the accuracy of the model. Because of the inherent uncertainties in any modelling effort, it is important that a reasonable margin of safety be included so that permit conditions will not need to be changed drastically in the future, resulting in costly design changes for waste-treatment facilities.

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APPENDIX F

PERMIT WASTELOAD ALLOCATIONS

Approach

The Department of Environmental Quality's 1992 Section 305(b) Report indicates that the Pudding River is water quality limited for dissolved oxygen and, therefore, is not able to fully support aquatic life. DEQ has proposed a total maximum daily load (TMDL) for the Pudding River. The TMDL is intended to limit introduction of oxygen-demanding pollutants into the Pudding River.

The TMDL includes proposed wasteload allocations (WLAs) for the both the City of Woodburn and Agripac, Inc., Plant #3 of Woodburn. These sources both discharge treated effluent into the Pudding River and have been found to be the two principal point sources that contribute to violations of the dissolved oxygen standard in the Pudding River. The wasteload allocations have focused on reduced flow-related loads of ammonia and oxygen-demanding material during the early summer (June) and little or no discharges of ammonia during critical low-flow periods (July and August). As stream temperature decreases in the fall (September), wasteload allocations are increased.

Wasteload Allocations

Agripac: The proposed **WLAs** for Agripac, Inc., Plant #3 are:

- For carbonaceous oxygen demand ($CBOD_5$): Daily maximum not to exceed 20 mg/L; monthly average not to exceed 10 mg/L.
- For total suspended solids (TSS): Daily maximum not to exceed 20 mg/L; monthly average not to exceed 10 mg/L.
- For effluent dissolved oxygen concentration: Daily average not to be less than 6.5 mg/L.
- For ammonia-nitrogen (NH_3-N) — see Table F-1.

Table F-1. Wasteload Allocations for Ammonia-Nitrogen for Agripac

June		
Pudding River Monthly Average Flow (cfs)	**Monthly Average Effluent Ammonia-N Concentration Not to be Exceeded mg/L**	**Daily Maximum Effluent Ammonia-N Concentration Not to be Exceeded mg/L**
>150	18	27
100 — 150	15	23
50 — 100	8.5	13
<50	3.2	4.8

(Continued on next page.)

Table F-1. Wasteload Allocations for Ammonia-Nitrogen for Agripac (Continued)

July and August		
Pudding River Monthly Average Flow (cfs)	**Monthly Average Effluent Ammonia-N Concentration Not to be Exceeded mg/L**	**Daily Maximum Effluent Ammonia-N Concentration Not to be Exceeded mg/L**
>100	10	15
60 – 100	2.7	4.1
30 – 60	1.5	2.3
<30	0.30	0.48
September and October		
Pudding River Monthly Average Flow (cfs)	**Monthly Average Effluent Ammonia-N Concentration Not to be Exceeded mg/L**	**Daily Maximum Effluent Ammonia-N Concentration Not to be Exceeded mg/L**
>100	WLAs do not apply.	
60 – 100	18	27
30 – 60	9.0	14
<30	4.0	6.0

Woodburn: The proposed WLAs for the City of Woodburn are:

- For carbonaceous oxygen demand ($CBOD_5$): Daily maximum not to exceed 20 mg/L; monthly average not to exceed 10 mg/L.
- For total suspended solids (TSS): Daily maximum not to exceed 20 mg/L; monthly average not to exceed 10 mg/L.
- For effluent dissolved oxygen concentration: Daily average not to be less than 6.5 mg/L.
- For ammonia-nitrogen (NH_3-N) (see Table F-2):

Table F-2. Wasteload Allocations for Ammonia-Nitrogen for Woodburn

June		
Pudding River Monthly Average Flow (cfs)	**Monthly Average Effluent Ammonia-N Concentration Not to be Exceeded mg/L**	**Daily Maximum Effluent Ammonia-N Concentration Not to be Exceeded mg/L**
>150	6.8	10.2
100 – 150	5.7	8.6
50 – 100	3.1	4.7
<50	1.1	1.7

(Continued on next page.)

Table F-2. Wasteload Allocations for Ammonia-Nitrogen for Woodburn **(Continued)**

July and August		
Pudding River Monthly Average Flow (cfs)	**Monthly Average Effluent Ammonia-N Concentration Not to be Exceeded mg/L**	**Daily Maximum Effluent Ammonia-N Concentration Not to be Exceeded mg/L**
>100	3.2	4.8
60 – 100	1.0	1.5
30 – 60	0.50	0.75
<30	0.10	0.18
September and October		
Pudding River Monthly Average Flow (cfs)	**Monthly Average Effluent Ammonia-N Concentration Not to be Exceeded mg/L**	**Daily Maximum Effluent Ammonia-N Concentration Not to be Exceeded mg/L**
>100	WLAs do not apply.	
60 – 100	6.8	10.2
30 – 60	3.3	5.0
<30	1.6	2.4

Note:

Wasteload allocations (WLAs) displayed above represent oxygen-demanding mass load limits for specific flow scenarios in the Pudding River. These allocations will be fixed. The actual concentration and mass load limits may be represented differently in the waste discharge permit, however, depending on the specific wastewater control alternatives selected by the City or Agripac. For example, if either of the sources choose to irrigate a portion of their effluent, higher concentration limits may be permitted because effluent discharge flows would be less. Also, the WLAs were derived by distributing the loads based upon an equal percent removal of influent ultimate-oxygen-demand from both sources. The Department considered several other options, but believes this approach is as equitable as any of the alternatives. Load allocations for nonpoint sources are assumed to stay unchanged.

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Appendix D
References

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Executive Summary

A collection system evaluation was conducted to help the Woodburn City staff allocate maintenance resources and plan for capital improvements. The evaluation included a flow monitoring program, a collection system inventory and hydraulic analysis, an infiltration and inflow (I/I) analysis, an I/I reduction and control program, and a 5-year capital improvements plan.

Flow Monitoring Program

A flow monitoring program was conducted by City staff under CH2M HILL's direction. Sanitary sewerage flows were monitored at key locations in the Woodburn collection system to provide data for hydraulic and I/I analyses. Dry- and wet-weather flows were monitored at five collection system locations and at the wastewater treatment plant. The current (1990) instantaneous peak wet-weather flow was estimated at 12.00 mgd.

Collection System Evaluation

The evaluation of Woodburn's collection system included a facilities inventory and a hydraulic analysis of the trunk and interceptor system. The purpose of the collection system inventory was to obtain information that might indicate the extent and possibly the condition of facilities requiring maintenance. The hydraulic analysis was intended to identify portions of the collection system that might not have the capacity to convey current and future peak flows.

The Woodburn collection system is composed of approximately 14.4 miles of trunk and interceptor line and 10 pump stations. The system currently serves about 2,087 acres but will eventually serve an estimated 4,913 acres.

The results of the hydraulic analysis showed that the Mill Creek Pump Station and Pump Station Numbers 1, 2, 3, and 9 may require capacity upgrades. In addition, the Front Street Interceptor through the downtown area to Lincoln Street and the trunkline along Highway 214 and Astor Way serving the northern portion town will require improvement to increase capacity. Additional problems are not expected by 1995, but the problems listed above are expected to get worse. Flow predicted for buildout conditions will surcharge approximately 59 percent of the trunk and interceptor system.

Infiltration and Inflow Analysis

An I/I analysis was conducted by CH2M HILL to determine whether it would be cost-effective to attempt to reduce the I/I rate in the City's sanitary sewerage collection system.

The instantaneous peak I/I rate is approximately 4,222 gallons per acre per day (gpad) in Woodburn. A comparison of the cost of system rehabilitation to the cost of wastewater conveyance and treatment showed that an estimated 2 mgd of I/I could be cost-effectively removed from the collection system.

Infiltration and Inflow Reduction and Control Program

An I/I program was outlined by CH2M HILL to help direct the City's I/I removal efforts. The recommended I/I control program includes flow monitoring, smoke testing, visual inspection of manholes, and some television inspection of pipelines.

5-Year Capital Improvements Plan

Woodburn City staff prepared a 5-year Capital Improvements Plan (CIP) for the sanitary sewerage collection system. The CIP will be conducted in two phases. Phase I will include the Mill Creek Pump Station rehabilitation, and construction of three sanitary sewer projects: (1) the southwest sanitary sewer, (2) the southeast sanitary sewer, and (3) the north sanitary sewer. Construction of a new lift station at the Mill Creek Pump Station site will comprise Phase II of the CIP. The project costs for Phase I and Phase II of the CIP are estimated at $3,555,500 and $2,283,000, respectively.

CH2M HILL

PREPARED FOR: Frank Tiwari/City of Woodburn

PREPARED BY: Jay Holtz/CH2M HILL

COPIES: Dean Morrison/City of Woodburn
Daria Wightman/CH2M HILL

DATE: June 11, 1993

SUBJECT: Flow Monitoring Program

PROJECT: PDX27874.F4

A sanitary sewerage flow monitoring program was conducted by the City of Woodburn and CH2M HILL as part of the Woodburn Wastewater Facilities Plan. City staff installed, operated and removed the flow meters, while CH2M HILL was responsible for processing and interpreting the data. This technical memorandum discusses the goals and objectives of the monitoring program, the status of historical flow records, data collection, data analysis, and monitoring results.

Goals and Objectives

Sanitary sewerage flows were monitored at key locations in the Woodburn collection system to provide data for a computer analysis of hydraulic capacity and for assessing system infiltration and inflow (I/I) rates as a precursor to developing estimates of cost-effective I/I reduction. The results of the hydraulic analysis and the I/I assessment are provided in Technical Memorandums F4-2 and F4-3, respectively.

Collection system flows were monitored during dry-weather conditions to determine the characteristics of sanitary and base flow, while data collected during wet weather were used to estimate existing levels of I/I. Sanitary flow is the total of domestic, commercial, and industrial sewerage contributions. Base flow is commonly defined as the amount of infiltration that is always present in sewerage flow regardless of precipitation or groundwater level, while I/I is the extraneous component of sewerage flow. Infiltration occurs when groundwater leaks into the sanitary sewer through such defects as cracked or broken pipe, defective pipe joints, or dilapidated manholes. Inflow occurs when storm runoff flows directly into the sanitary sewer at storm sewer cross connections, roof and foundation drains, catch basins, and faulty or submerged manholes. I/I rates vary with the duration and intensity of rainfall and the groundwater level.

Historical Flow Records

Until the flow monitoring program was conducted, the City had no record of sewerage flows other than wastewater treatment plant monitoring reports and strip charts, which offer a complete and accurate record of average daily and instantaneous plant effluent flows. These records are considered representative of total system flows.

Data Collection

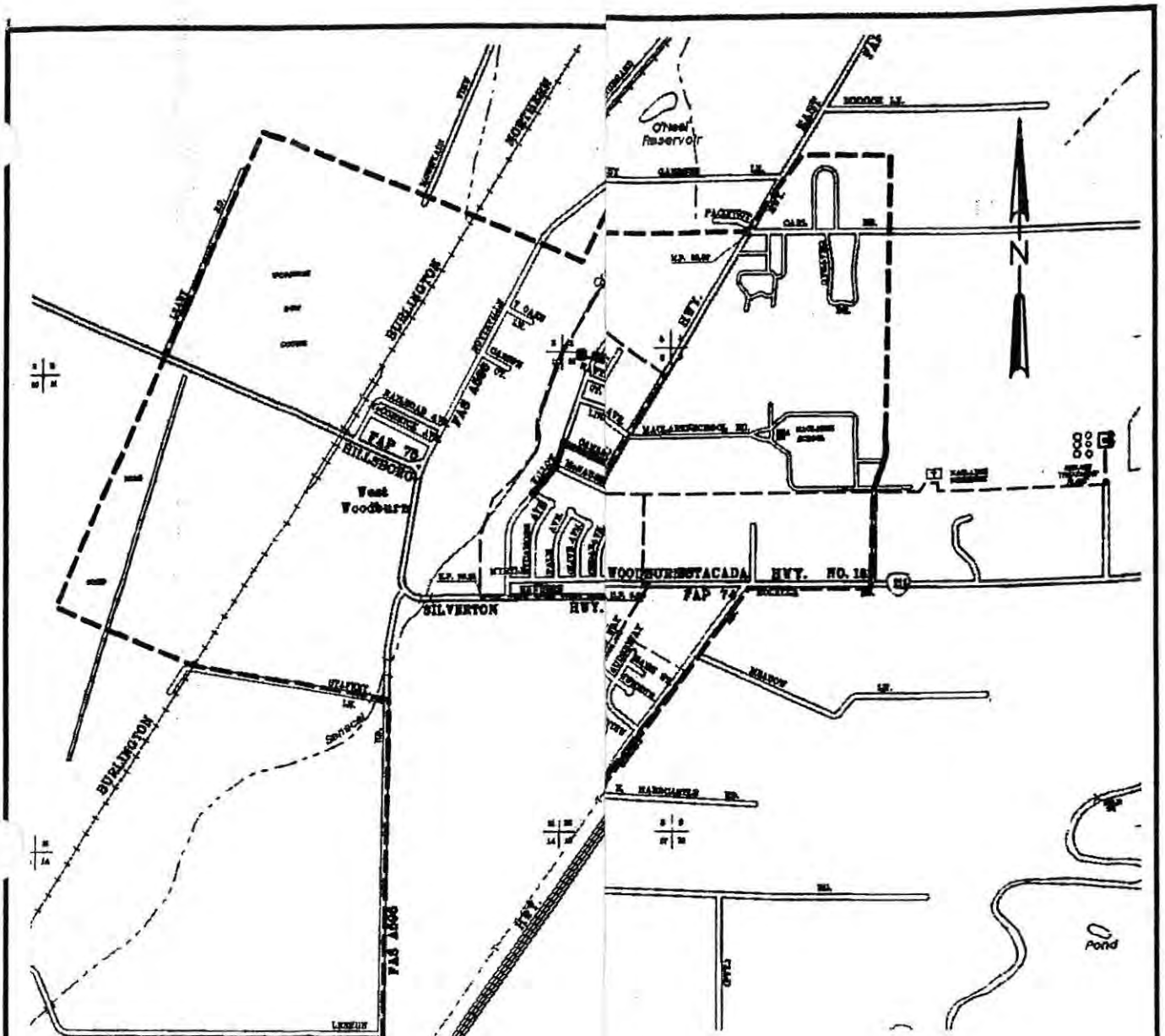
Selection of Flow Monitoring Sites

Sites for the flow meters were selected to roughly divide the collection system into major blocks representing drainage basins or groups of drainage basins. Because meter deployment is costly, the number of monitoring sites used was the minimum determined to be practical given the configuration of the City's collection system. Thus, flow was monitored at five sites in the collection system and at the wastewater treatment plant headworks and effluent line.

Temporary, portable flow meters were placed in manholes at each of the sites listed in Table 1 and shown in Figure 1. Data were also collected from the existing effluent flow meter at the wastewater treatment plant. In addition, a temporary flow meter was installed on the plant influent force main to determine the relationship between influent and effluent flow rates and quantify any dampening of flow rates caused by plant process units. A comparison of the influent and effluent data through March and April 1991 showed that instantaneous effluent flow does not vary significantly from influent flow. Short-term fluctuations in flow entering the plant are dampened slightly by the process units, but effluent flow records proved to be an accurate representation of plant influent and, therefore, total collection system sewerage flows. This determination allows plant effluent flow records to be used in conjunction with the system flow monitoring program data.

Table 1
Flow Monitoring Sites

Meter Location	Manhole ID	Pipe Diameter (inches)
City Compost Dump (at Mill Creek)	MC1-030	36
Young Street (at Mill Creek)	MC1-105	18
Commerce Avenue (at Mill Creek)	MC3-020	16
Fifth Street and Harrison Street	MC3-095	16
Highway 214 (at the Mill Creek Pump Station)	MC7-030	18
Treatment Plant Effluent Line	—	—



LEGEND

- SEWERAGE SERVICE AREA BOUNDARY
- TRUNK AND INTERCEPTOR SYSTEM WITH MANHOLES
- WASTEWATER TREATMENT PLANT
- FORCE MAIN
- PUMP STATION
- FLOW MONITORING DRAINAGE AREA BOUNDARY
- FLOW MONITORING DRAINAGE AREA IDENTIFICATION
- MONITOR LOCATION

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ORING DRAINAGE AREAS
ONITORING SITES
RN FACILITIES PLAN

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Figure 1

The selected flow monitoring sites divide the service area into six drainage areas, identified by the letters A through F in Figure 1 and generally consisting of the following locales:

- Area A: west Woodburn and the area west of I-5
- Area B: north Woodburn
- Area C: central downtown Woodburn
- Area D: south Woodburn
- Area E: southeast Woodburn
- Area F: northeast Woodburn

The service area was divided further into ten drainage basins for the collection system evaluation described in Technical Memorandum F4-2. Table 2 shows the drainage area and drainage basin associated with each flow monitoring site.

Table 2
Flow Monitoring Site Characteristics

Meter Location	Drainage Area	Drainage Basin
City Compost Dump (at Mill Creek)	E	MC-1, MC-4, MC-5
Young Street (at Mill Creek)	D	MC-1, MC-6
Commerce Avenue (at Mill Creek)	C	MC-3
Fifth Street and Harrison Street	A	MC-3, SC-1
Highway 214 (at the Mill Creek Pump Station)	B	MC-7
Treatment Plant Effluent Line	F	MC-2, MC-8, MC-9

Because some drainage areas contribute flow to other drainage areas, the definitions shown in Table 3 were used to determine the net flow from any single area.

Table 3
Flow Monitoring Drainage Area Definitions

Drainage Area	Net Flow
A	Flow from Area A only
B	Flow from Area B only
C	Flow from Area C minus flow from Area A
D	Flow from Area D only
E	Flow from Area E minus flow from Areas C and D
F	Flow from the treatment plant minus flow from Areas B and E

The service characteristics of each flow monitoring drainage area are shown in Table 4.

Table 4 Flow Monitoring Drainage Area Service Characteristics			
Drainage Area	1990 Area Served (acres)	1990 Dwelling Units	1990 Commercial/Industrial Area Served (acres)
A	423	1,418	44.3
B	351	1,605	29.6
C	171	798	46.6
D	242	419	0.0
E	447	1,312	97.9
F	453	455	194.9

Flow Monitoring Equipment

Effluent flow at the wastewater treatment plant is recorded by a Sparling propeller flow meter, which provides both instantaneous and totalized flow records. Marsh McBirney 260A Flo-Tote flow meters were used at each of the five collection system sites. The Flo-Tote meters determine flow from recordings of liquid depth made by a pressure transducer and liquid velocity made by an electromagnetic sensor. Marsh McBirney provides a computer software program that calculates flow from depth and velocity based on the continuity equation:

$$Q = v * A$$

where: Q = Flow (cubic feet per second, or cfs)
v = Velocity of flow (feet per second, or ft/sec)
A = Cross-sectional area of flow (square feet, or ft^2)

The depth measurement from the Flo-Tote meters is generally reliable, but turbulent flow or fouling of the velocity sensor may yield less accurate velocity measurements. When the accuracy of velocity measurements is in question, the quantity of flow may be estimated using liquid depth data and the Manning equation:

$$Q = 1.49/n * r_H^{2/3} * S^{1/2}$$

where: Q = Flow (cfs)
n = Manning roughness coefficient
r_H = Hydraulic radius (feet, or ft)
S = Slope (feet per foot, or ft/ft)

Table 5 lists the identification numbers of the meters used at each site during dry-weather and wet-weather periods. Meter identification numbers may be used when referencing the raw or reduced flow data.

Table 5
Flow Meter Identification Numbers

Meter Location	Meter No. Dry Weather	Meter No. Wet Weather
City Compost Dump (at Mill Creek)	182	778
Young Street (at Mill Creek)	203	612
Commerce Avenue (at Mill Creek)	175	104
Fifth Street and Harrison Street	181	308
Highway 214 (at the Mill Creek Pump Station)	209	119

Flow Monitoring Period

Dry-weather flows were monitored for the 3 weeks beginning August 8, 1990, and ending August 28, 1990. No rainfall occurred during this period, so these flows reflect sanitary and base flow without I/I.

Wet-weather flows were monitored for 5.5 weeks, from January 18, 1991, to February 25, 1991. Daily rainfall during this period varied from 0 to 0.7 inch. The wet-weather flows reflect sanitary flow, base flow, and I/I.

Rainfall Records

The North Willamette Experiment Station near the City of Salem provided a complete record of daily rainfall amounts to the nearest 1/100-inch during the monitoring periods. Daily rainfall included in the appendices was taken from these records.

Data Analysis

Data Quality and Reliability

The following factors can make it difficult to obtain accurate measurements of flow in most situations:

- Condition of the flow meter
- Characteristics of the channel or pipe where the meter is located
- Characteristics of the liquid being monitored
- Site calibration of the flow meter

The permanent effluent flow meter at the wastewater treatment plant is designed, located, and maintained with regular calibration to avoid inaccuracies related to these factors. The portable flow meters may be used under a variety of conditions, however, and are more susceptible to inaccuracy. The portable flow meters must be correctly calibrated at the factory, then carefully field calibrated to suit the conditions at each installation. The voltage of the meter battery must be maintained or the meter will cease recording data. Turbulence can affect both the depth and velocity readings. Grease in wastewater can foul the velocity sensor and affect the velocity readings.

A good method for estimating the quality of the data is to plot each depth record against the corresponding calculated flow for each site. Theoretically, a given liquid depth will result in a single corresponding flow. Thus, a plot of depth versus flow showing a tight band of data points suggests consistent depth and velocity recordings and, in turn, more accurate flow results. A plot showing a dispersed band of data points suggests inconsistent depth and velocity recordings. Most meter installations in the Woodburn monitoring program seemed to yield consistent data, as summarized in Table 6. Plots showing depth versus flow for each site are shown in Appendix A.

Table 6
Apparent Consistency of Flow Data

Meter Location	Dry-Weather Installation	Wet-Weather Installation
City Compost Dump (at Mill Creek)	Fair	Poor
Young Street (at Mill Creek)	Good	Good
Commerce Avenue (at Mill Creek)	Good	Good
Fifth Street and Harrison Street	Poor	Poor
Highway 214 (at the Mill Creek Pump Station)	Good	Good

Flow was calculated with the continuity equation for meters showing good or fair flow data consistency. Because depth readings are generally more dependable than velocity readings, flow was calculated with depth data and the Manning equation for meters showing poor flow data consistency. Wet-weather flow data for the City compost dump site and wet- and dry-weather flow for the Fifth Street and Harrison Street site were calculated using the recorded liquid depth and the Manning equation.

The average daily dry-weather flows were greater than the average daily wet-weather flows at the Young Street and the Fifth and Harrison Street sites. The likely explanation is improper field calibration of the meters. The data cannot be rectified and must be assumed to be unreliable, but are still useful for determining the relative response of flow to rainfall.

Meter 612, which was located at the Young Street site during the wet-weather period, experienced a loss of data for 18 days, apparently because of a low battery.

Data Reduction and Flow Determination

Flow data were processed to reflect average hourly flow and average daily flow at each monitoring site for dry- and wet-weather conditions. The tables containing the hourly data show the date and time, the day of the week, the depth of flow in ft, the velocity of flow in ft/sec, and the flow expressed in cfs. Copies of these tables, which are extensive, are not included herein but are available as computer files. Weekly plots generated from the hourly data for each meter are provided in Appendix B. Tables listing daily flow show the date, daily rainfall in inches (in), and flow expressed in million gallons per day (mgd) and are contained in Appendix C along with plots of daily flow.

Monitoring Results

Historic Flows

Examination of wastewater treatment plant flow records for the years 1988, 1989, and 1990 produced the flow characteristics listed in Table 7. City staff estimates that the peak instantaneous flow rate at the treatment plant may have reached 12 mgd on February 18, 1992, when nearly 2 inches of rainfall were recorded in a 24-hour period.

Table 7
Wastewater Treatment Plant Historic Flow Characteristics

Dry-Weather Flows (May through October)	
Average Daily Flow	1.80 mgd
Maximum Daily Flow	3.56 mgd
Minimum Daily Flow	0.99 mgd
Wet-Weather Flows (November through April)	
Average Daily Flow	2.10 mgd
Maximum Daily Flow	4.66 mgd
Minimum Daily Flow	1.19 mgd

Dry-Weather Flows

The average, maximum, and minimum daily flows recorded during the dry-weather monitoring period are summarized for each meter location, including the wastewater treatment plant, in Table 8.

Table 8 Dry-Weather Period Daily Flow Characteristics			
Meter Location	Average Flow (mgd)	Maximum Flow (mgd)	Minimum Flow (mgd)
City Compost Dump (at Mill Creek)	0.40	0.52	0.30
Young Street (at Mill Creek)	0.19	0.20	0.17
Commerce Avenue (at Mill Creek)	0.25	0.29	0.18
Fifth Street and Harrison Street	0.34	0.37	0.27
Highway 214 (at the Mill Creek Pump Station)	0.18	0.20	0.17
Wastewater Treatment Plant	1.76	2.54	0.93

Wet-Weather Flows

The average, maximum, and minimum daily flows recorded during the wet-weather monitoring period are summarized for each meter location, including the wastewater treatment plant, in Table 9.

Table 9 Wet-Weather Period Daily Flow Characteristics			
Meter Location	Average Flow (mgd)	Maximum Flow (mgd)	Minimum Flow (mgd)
City Compost Dump (at Mill Creek)	1.36	1.64	0.68
Young Street (at Mill Creek)	0.16	0.19	0.13
Commerce Avenue (at Mill Creek)	0.43	0.62	0.32
Fifth Street and Harrison Street	0.28	0.40	0.19
Highway 214 (at the Mill Creek Pump Station)	0.35	0.48	0.27
Wastewater Treatment Plant	1.94	2.78	1.10

Flow Analysis

Historical flow data from the wastewater treatment plant were used to estimate peak system flow rates, while trends in collection system flow data from the portable flow meters were used to identify the origin of peak flow. The following assumptions were made for the analysis:

- Average daily wet-weather flows and instantaneous peak wet-weather flows are taken from historical data for the wastewater treatment plant.

- Sanitary flow for the average daily wet-weather condition and the maximum daily wet-weather condition is equal to the historic dry-weather minimum daily flow.

- Sanitary flow for the peak instantaneous wet-weather condition is equal to the historic dry-weather minimum daily flow times a sanitary peaking factor of 2.4 (as derived from dry-weather flow data).

- Base flow is equal to the difference between the historic dry-weather average daily flow and the historic dry-weather minimum daily flow.

- The ratio of instantaneous peak flow to average daily flow during wet-weather periods is about 1.4 (as derived from wet-weather flow data).

- The population served is 16,219 persons.

- The area served is 2,087 acres.

Current system flow estimates, and the gallon per day contributions per capita (gpcd) and per acre (gpad), are summarized in Table 10.

The relationship between average daily flow and rainfall at each collection system monitoring location was studied to determine relative I/I rates for the flow monitoring drainage areas. The flow from some drainage areas increases significantly in response to rainfall, indicating that more I/I is entering the collection system in these areas than others. The response to rainfall in each area was extrapolated until the total flow from all areas was equal to the instantaneous peak wet-weather I/I flow of 8.81 mgd. The resulting I/I rates are listed in Table 11 in mgd and in gpad as a function of acres served by the sewer system.

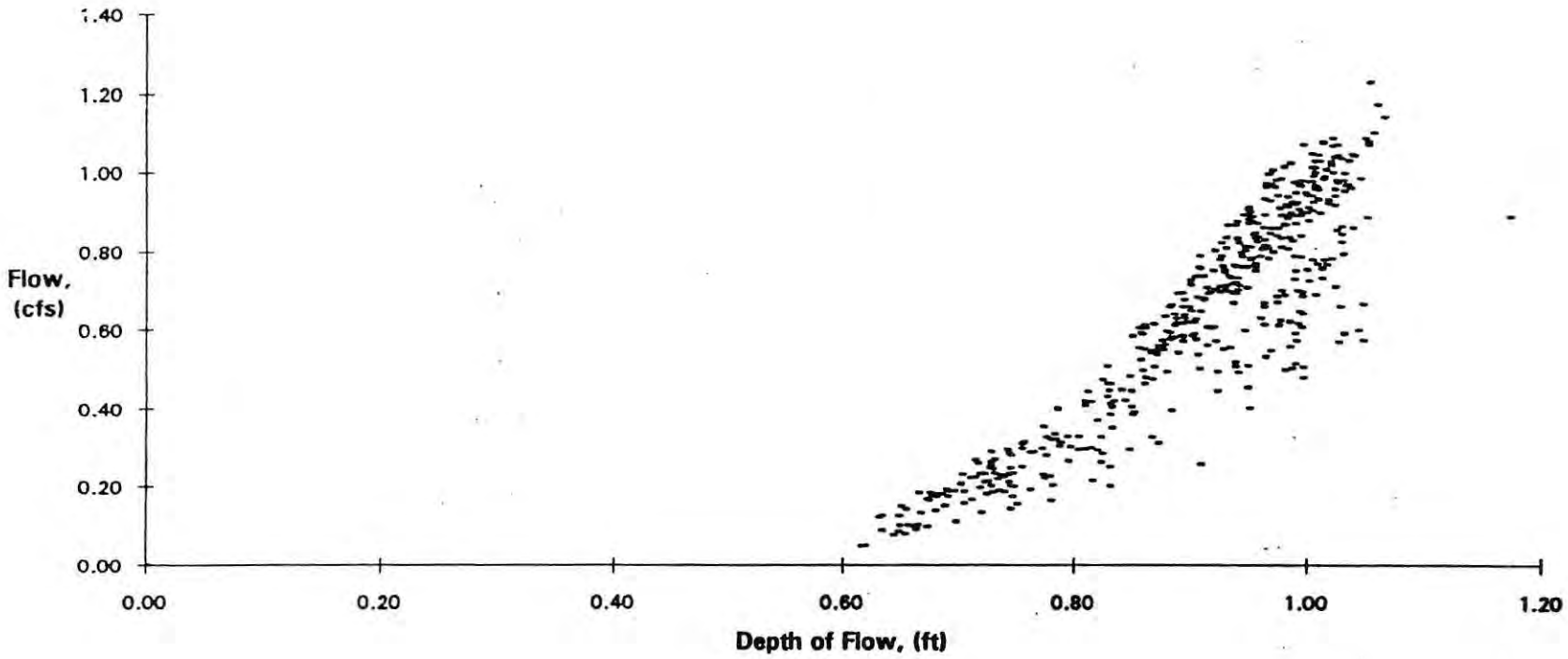
The relative I/I rates for the flow monitoring drainage areas seem reasonable given the characteristics of the collection system facilities in each drainage area. The sewers in the central downtown area of Woodburn are the oldest in the system and exhibit an I/I rate that is almost twice the system average. North Woodburn, which has had some inflow problems in the past, also has a greater than average I/I rate. Most of the sewers in south Woodburn were constructed after 1968. As may be expected, sewers in this area show the lowest I/I rate on a gpad basis.

Table 10 System Wet-Weather Flow Components			
	Average Daily Wet-Weather Flow	**Maximum Daily Wet-Weather Flow**	**Instantaneous Peak Wet-Weather Flow**
Current Flows (mgd)			
Sanitary Flow	0.99	0.99	2.38
Base Flow	0.81	0.81	0.81
I/I	0.30	6.53	8.81
Total Flow	2.10	8.33	12.00
Per Capita Contribution (gpcd)			
Sanitary Flow	61	61	146
Base Flow	50	50	50
I/I	18	403	543
Total Flow	129	514	740
Per Acre Contribution (gpad)			
Sanitary Flow	474	474	1,138
Base Flow	388	388	388
I/I	144	3,130	4,222
Total Flow	1,006	3,992	5,749

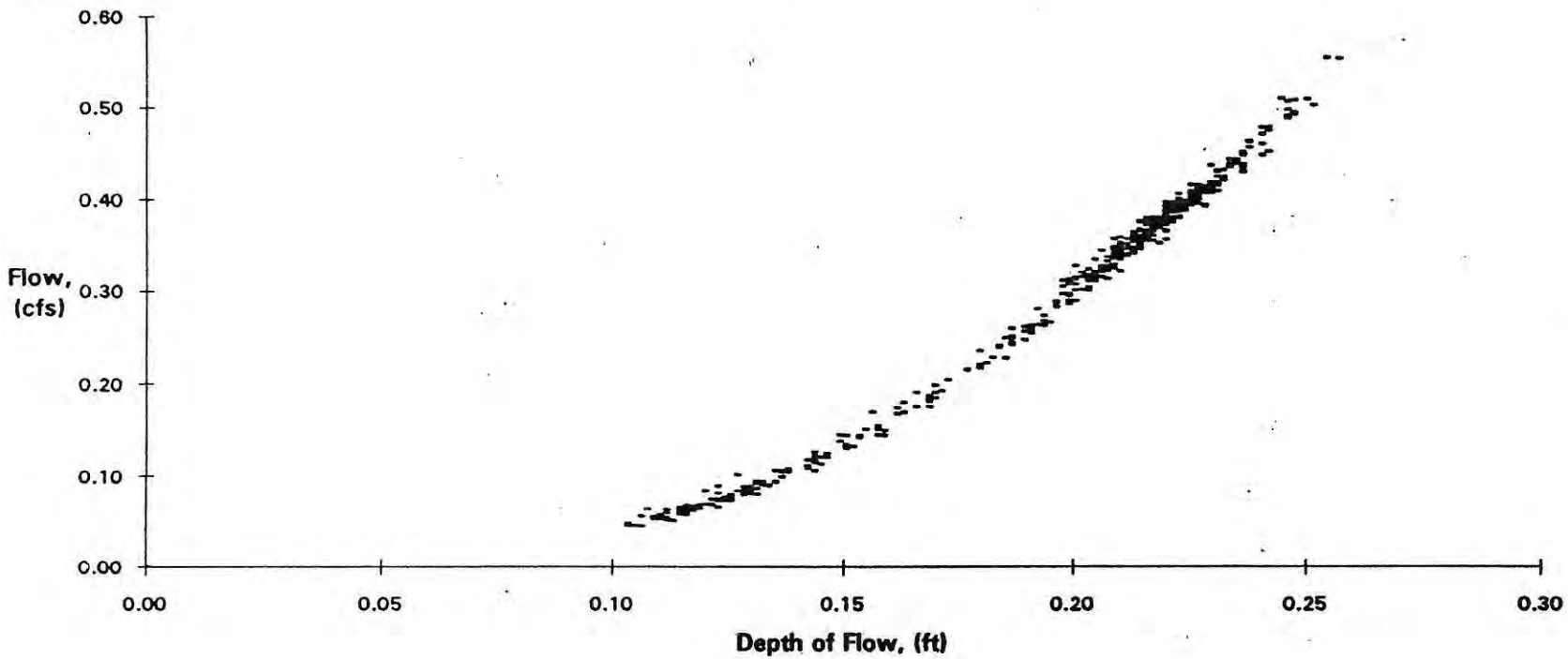
Table 11 I/I Rates for Flow Monitoring Drainage Areas		
Drainage Area	**I/I Rate (mgd)**	**I/I Rate (gpad)**
(A) West Woodburn and West of I-5	1.76	4,155
(B) North Woodburn	1.83	5,207
(C) Central Downtown Woodburn	1.41	8,232
(D) South Woodburn	0.37	1,536
(E) Southeast Woodburn	1.65	3,694
(F) Northeast Woodburn	1.79	3,961
Total	8.81	
Weighted Average		4,222

Appendix A

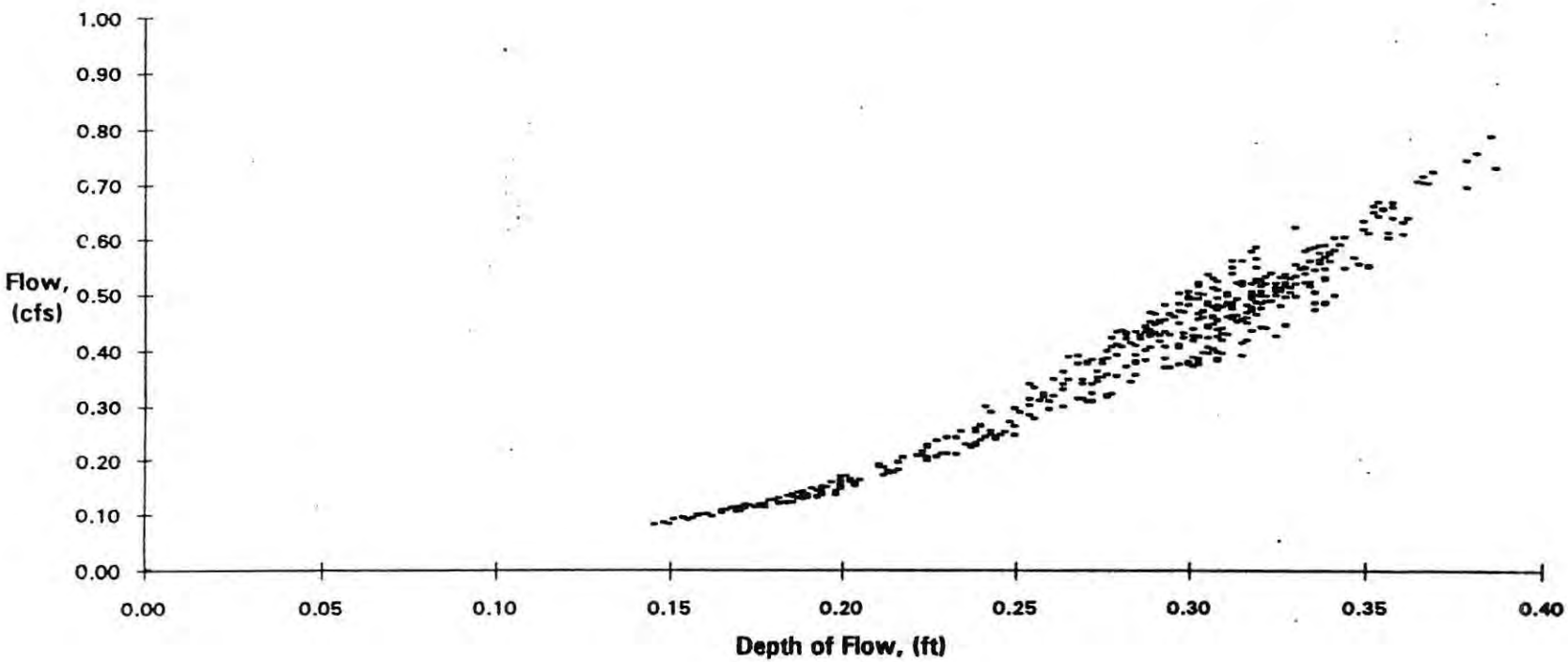
Woodburn Facilities Plan
Dry Weather Flow
Meter 182 Scatter



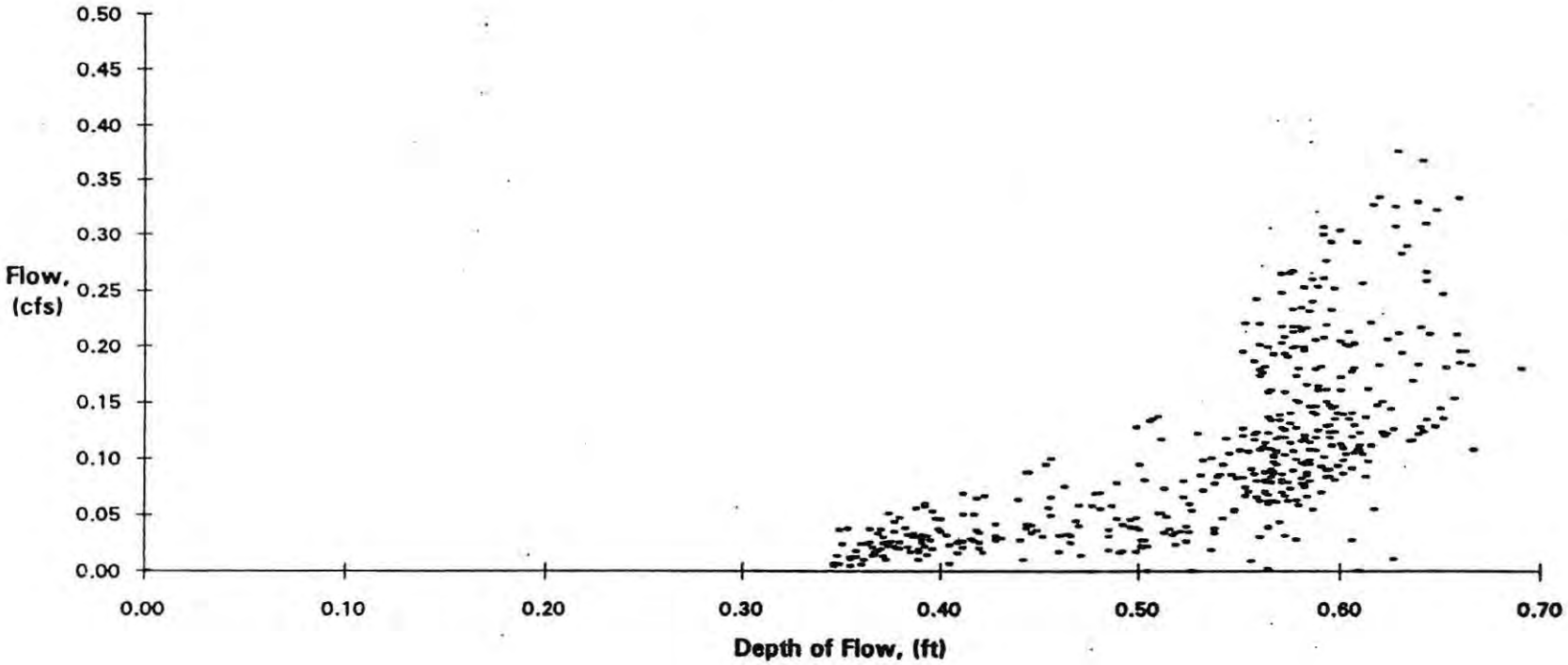
Woodburn Facilities Plan
Dry Weather Flow
Meter 203 Scatter



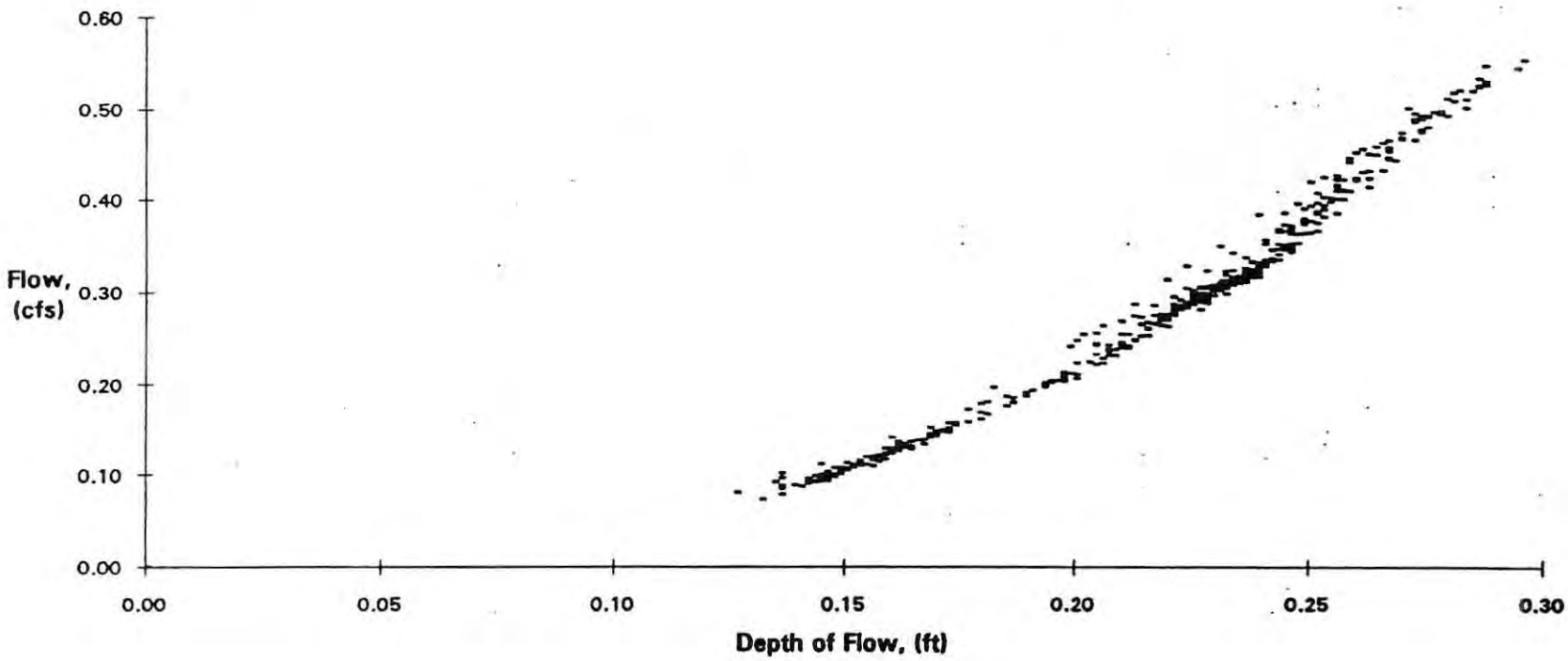
Woodburn Facilities Plan
Dry Weather Flow
Meter 175 Scatter



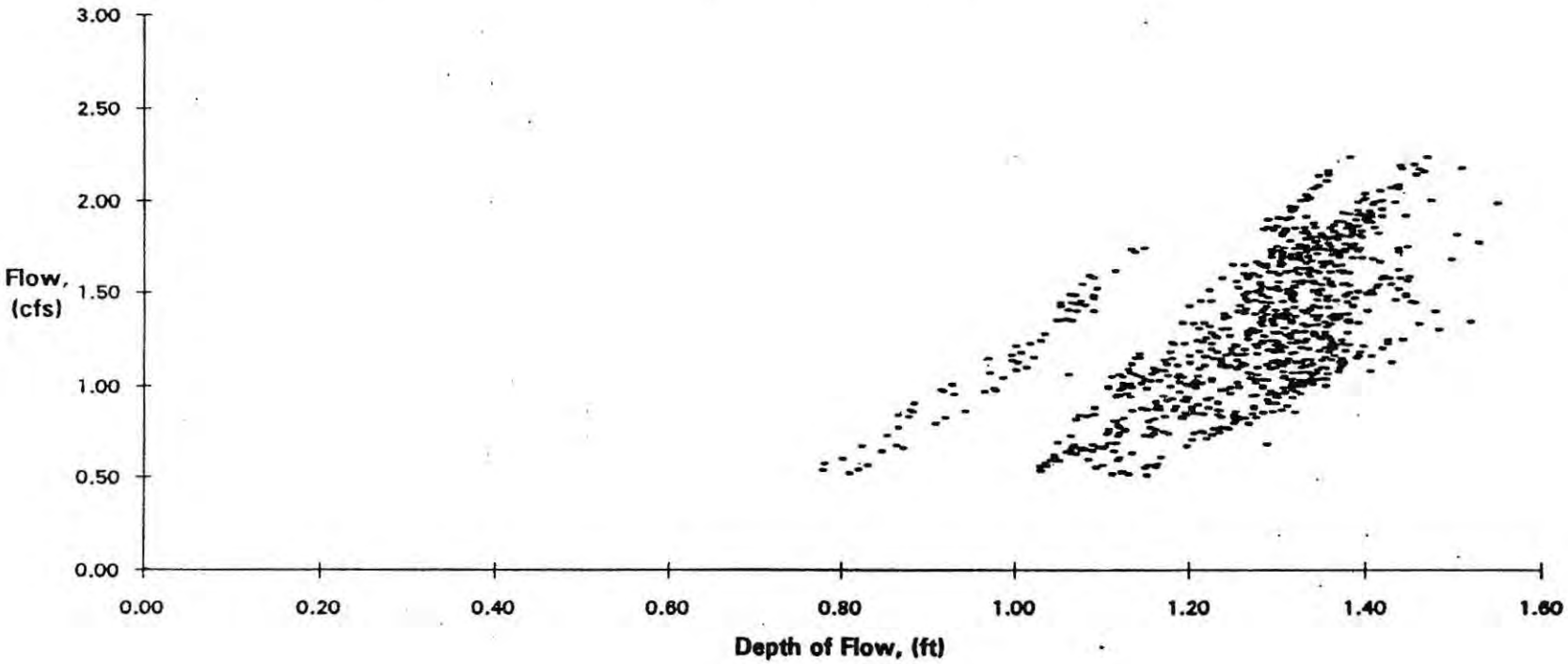
Woodburn Facilities Plan
Dry Weather Flow
Meter 181 Scatter



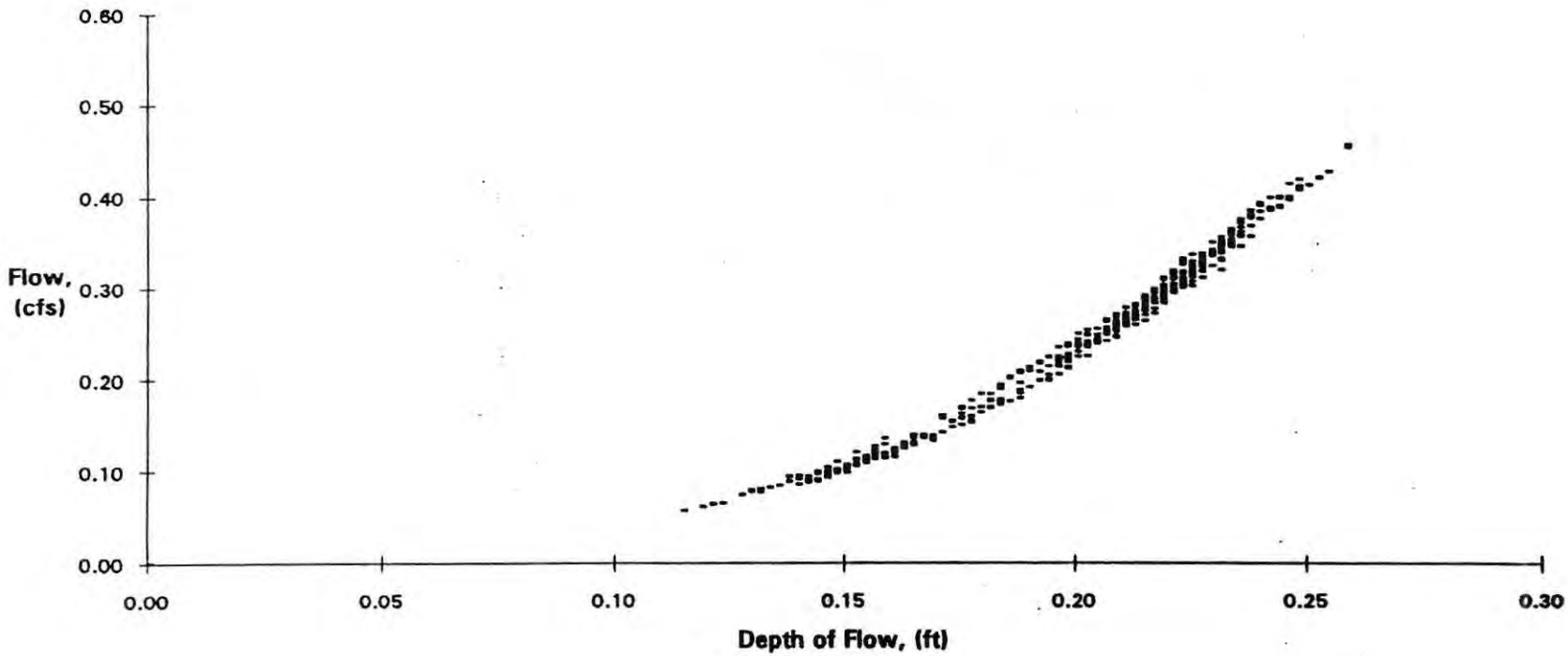
Woodburn Facilities Plan
Dry Weather Flow
Meter 209 Scatter



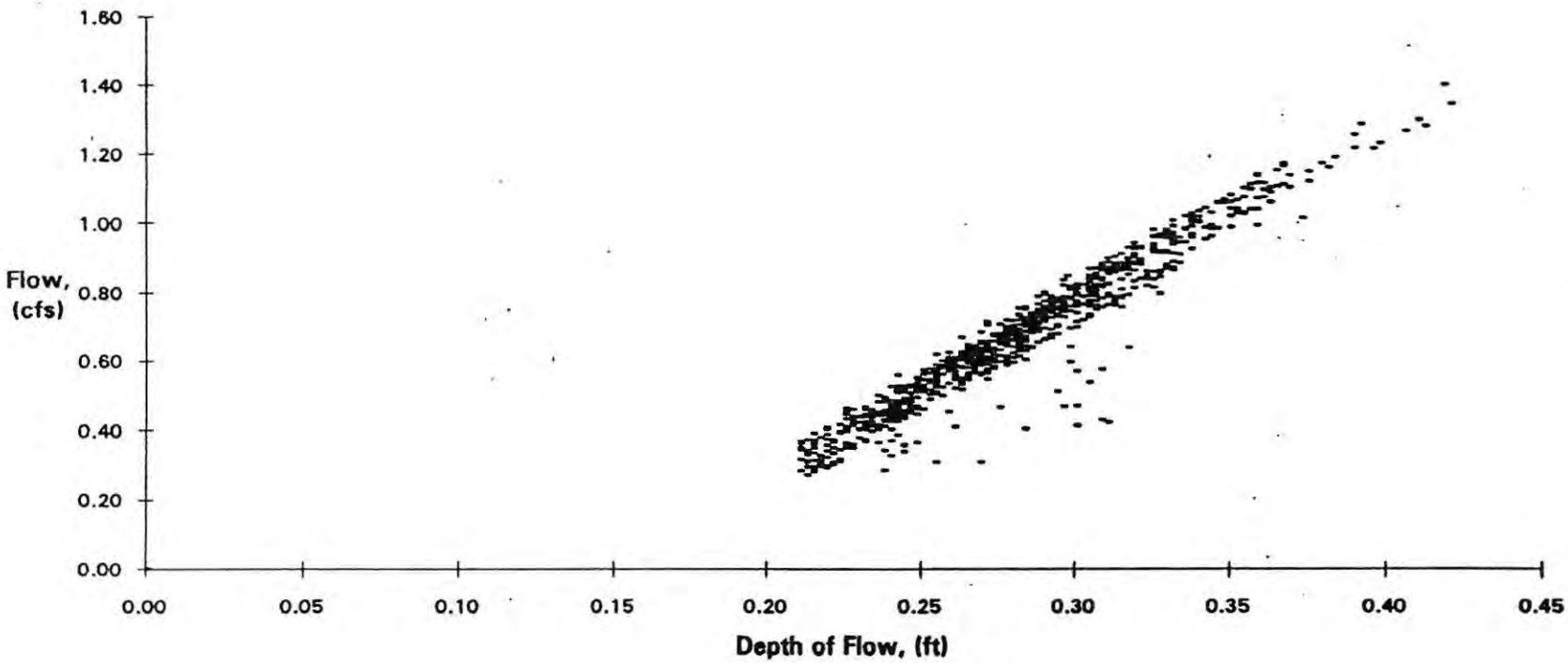
Woodburn Facilities Plan
Wet Weather Flow
Meter 778 Scatter



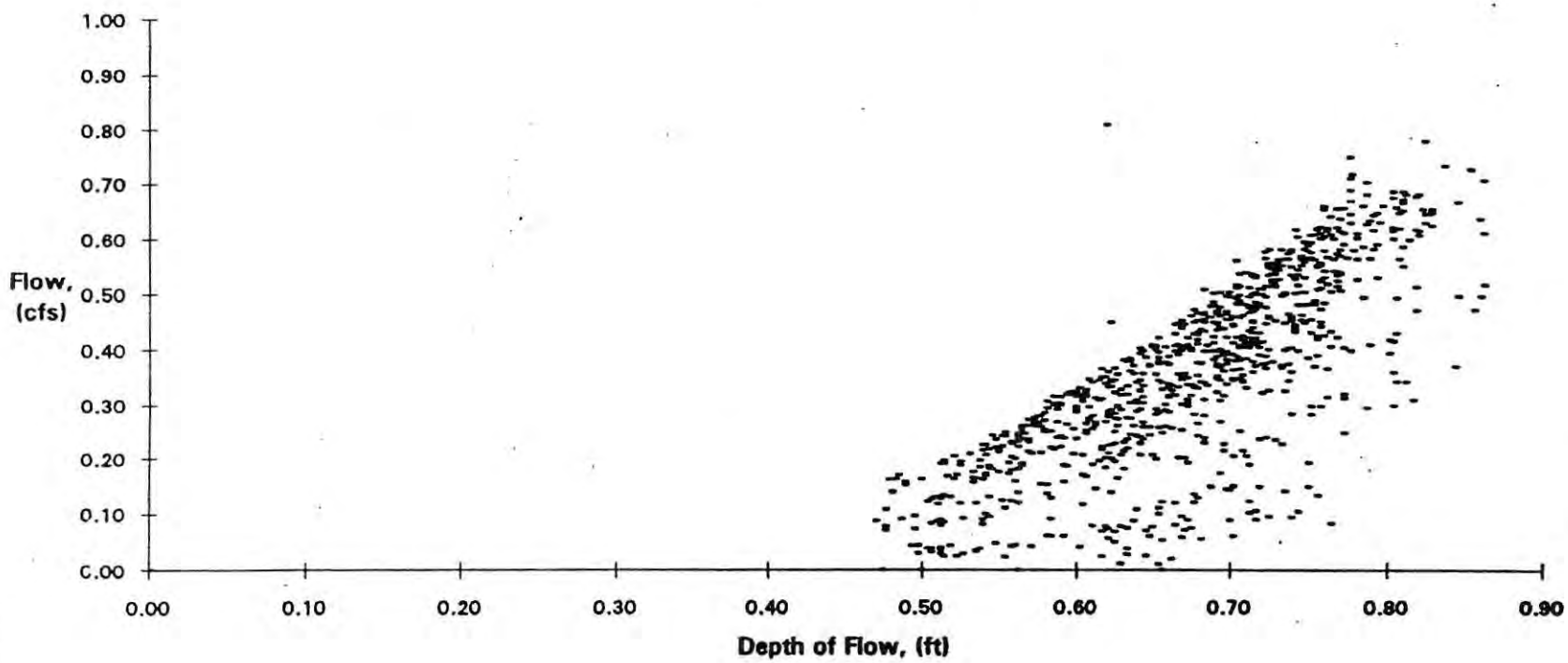
Woodburn Facilities Plan
Wet Weather Flow
Meter 612 Scatter



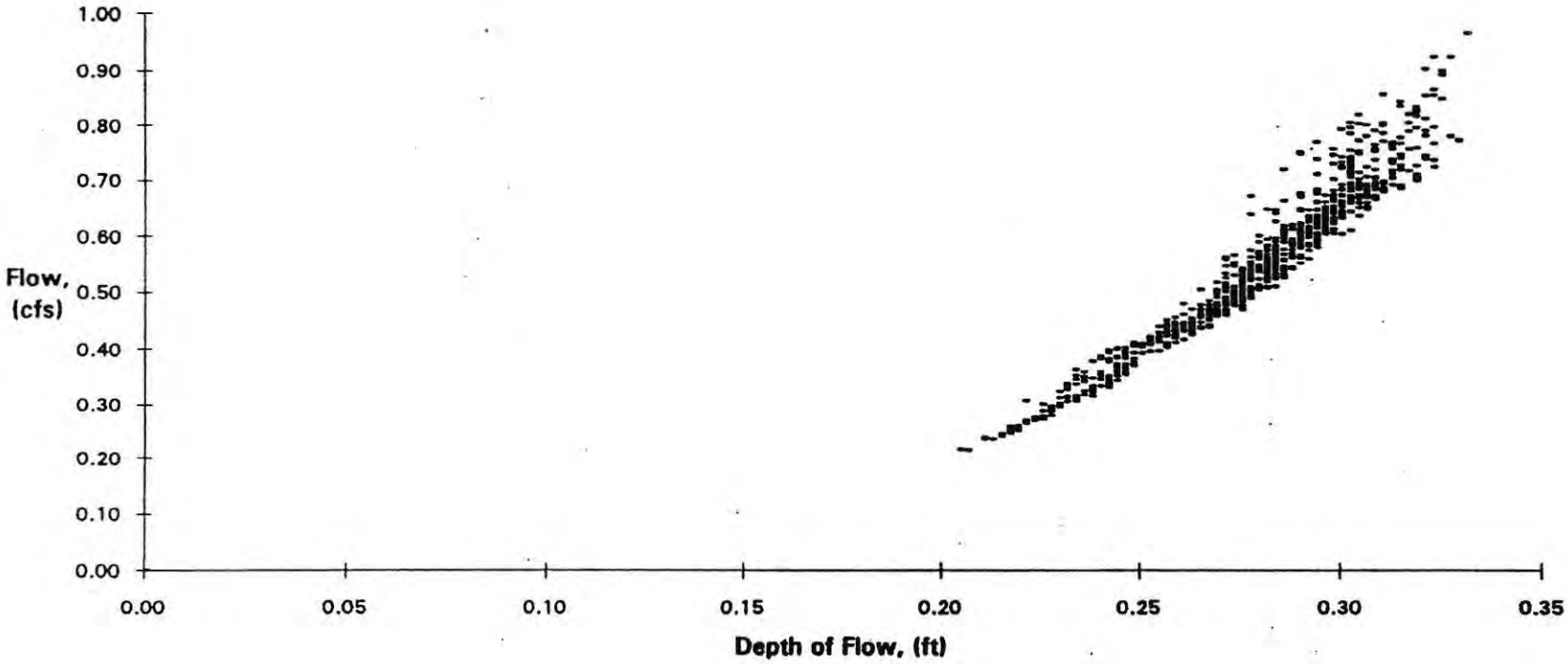
Woodburn Facilities Plan
Wet Weather Flow
Meter 104 Scatter



Woodburn Facilities Plan
Wet Weather Flow
Meter 308 Scatter

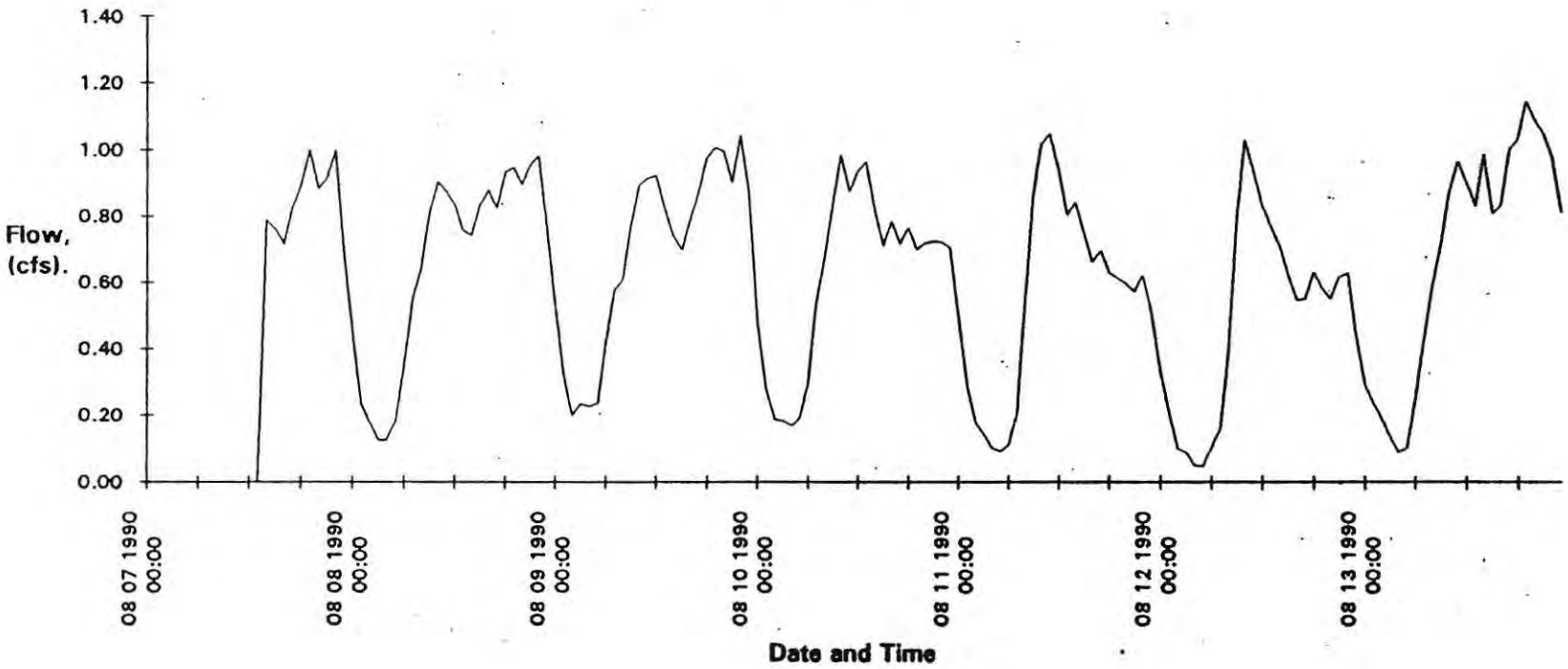


Woodburn Facilities Plan
Wet Weather Flow
Meter 119 Scatter

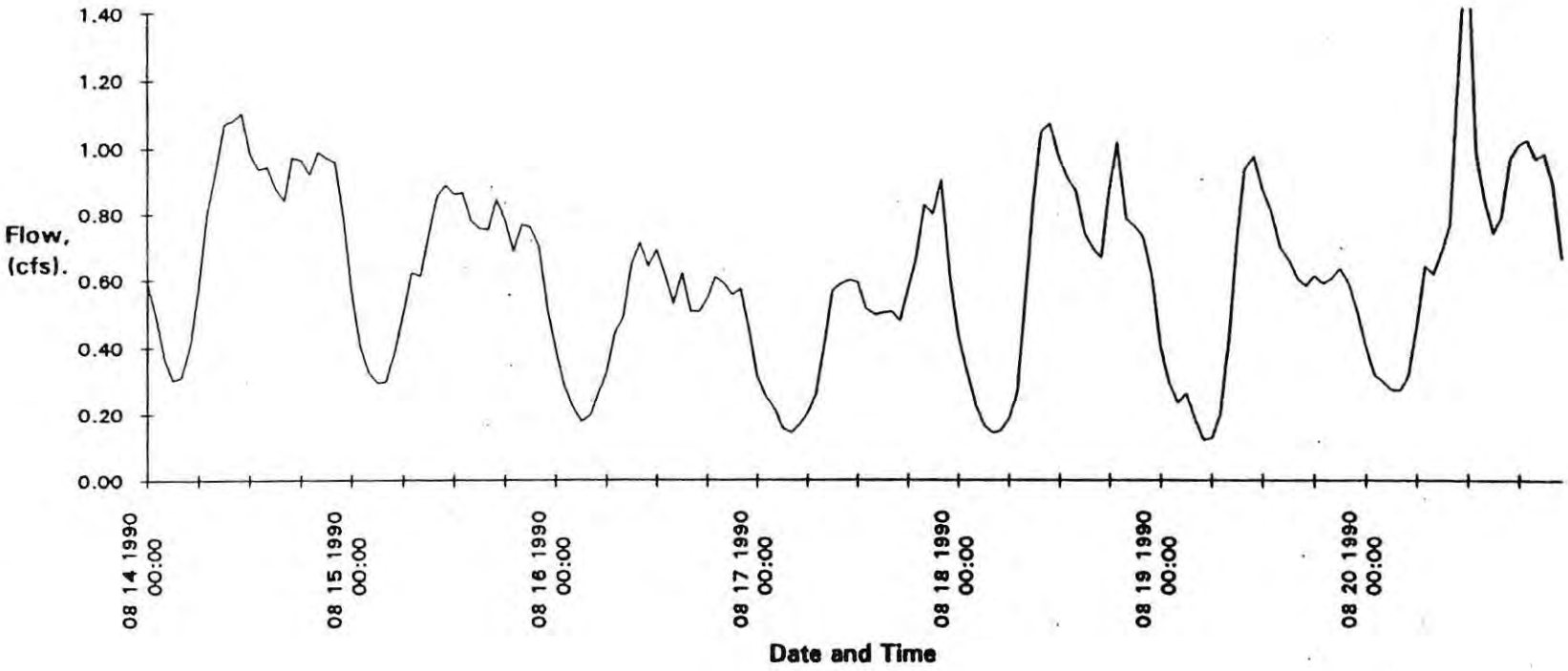


Appendix B

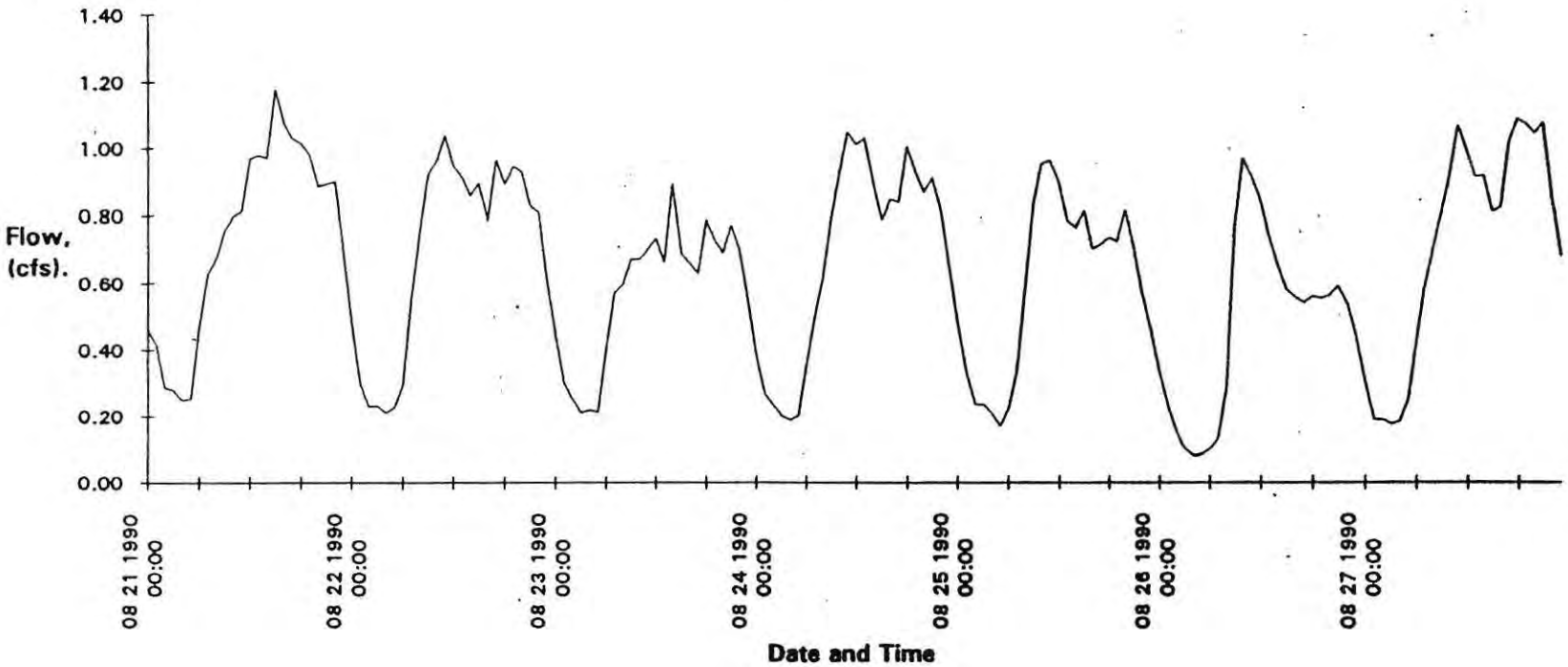
Woodburn Facilities Plan
Dry Weather Flow
Meter 182



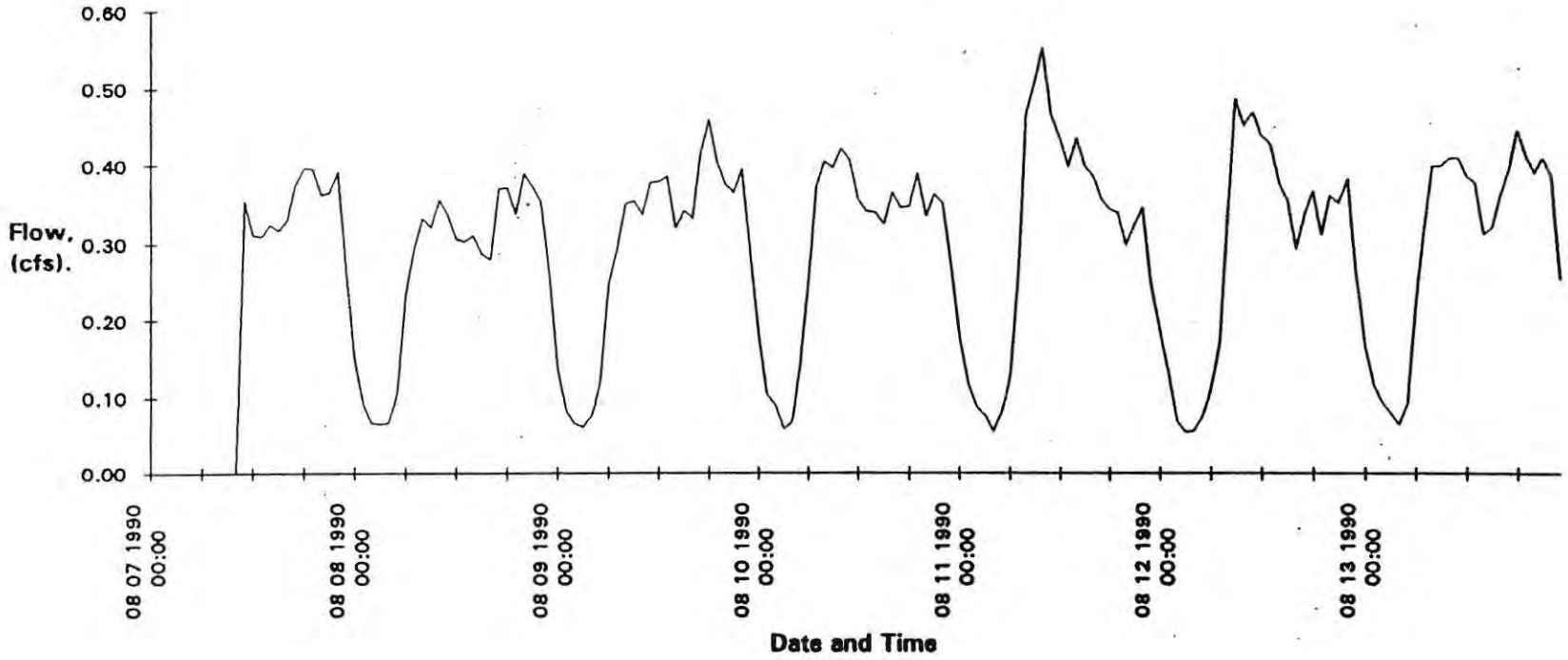
Woodburn Facilities Plan
Dry Weather Flow
Meter 182

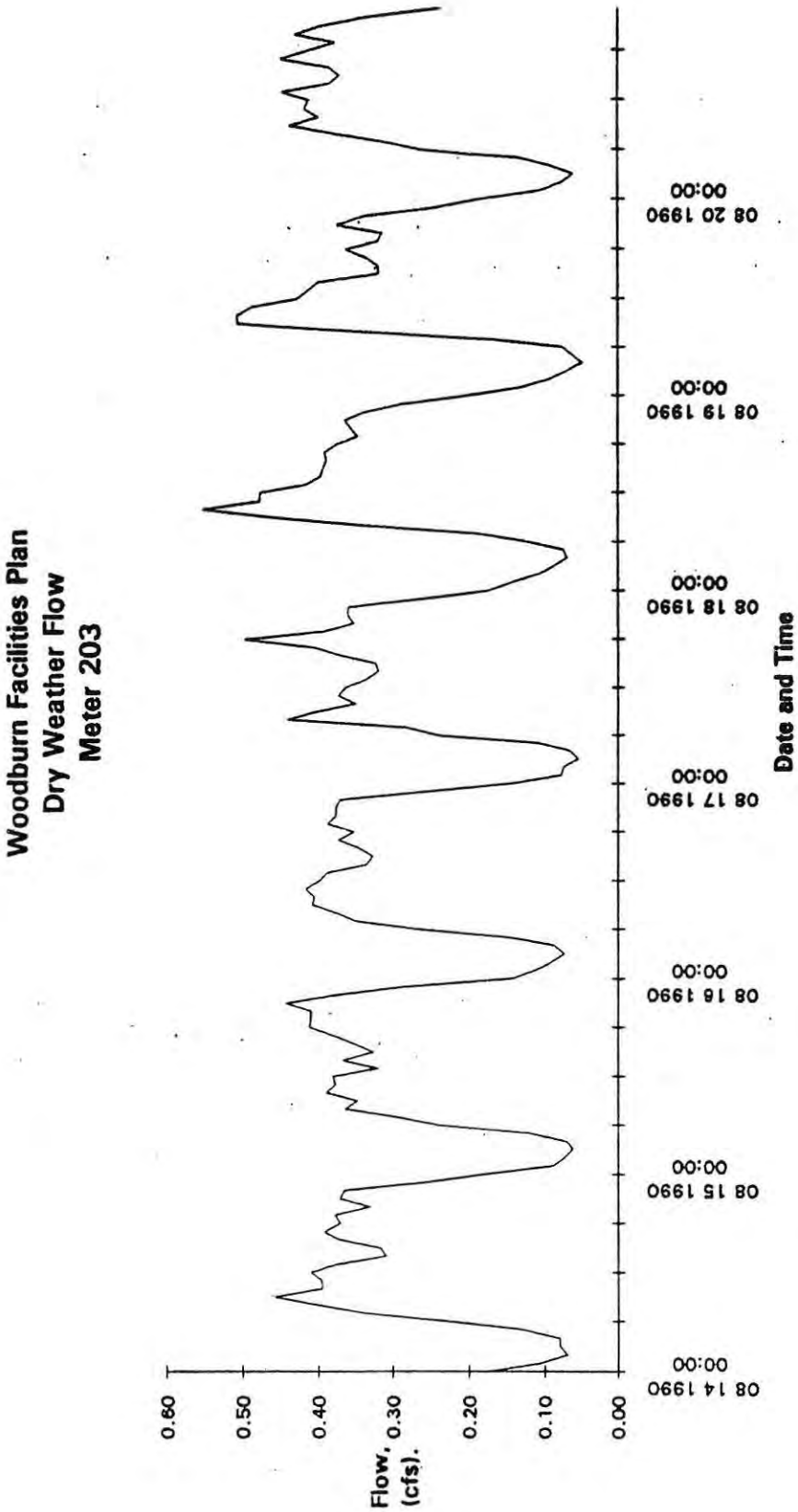


Woodburn Facilities Plan
Dry Weather Flow
Meter 182



Woodburn Facilities Plan
Dry Weather Flow
Meter 203




Woodburn Facilities Plan
Dry Weather Flow
Meter 203
Flow, (cfs).
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0.40
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0.20
0.10
0.00
08 14 1990 00:00
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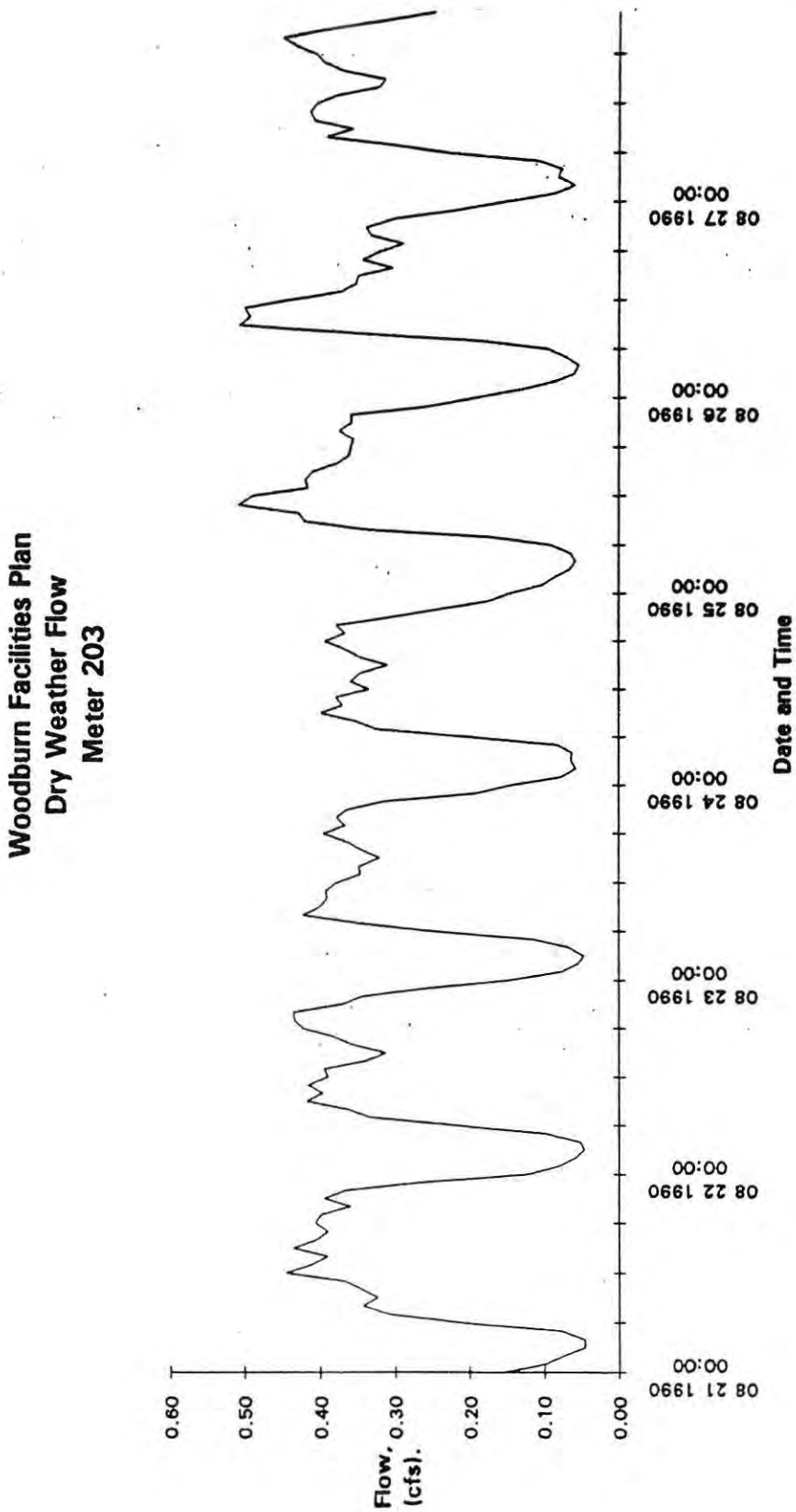
08 18 1990 00:00
08 19 1990 00:00
08 20 1990 00:00
Date and Time


Woodburn Facilities Plan
Dry Weather Flow
Meter 203
Flow, (cfs).
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0.30
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0.10
0.00
08 21 1990 00:00
08 22 1990 00:00
08 23 1990 00:00
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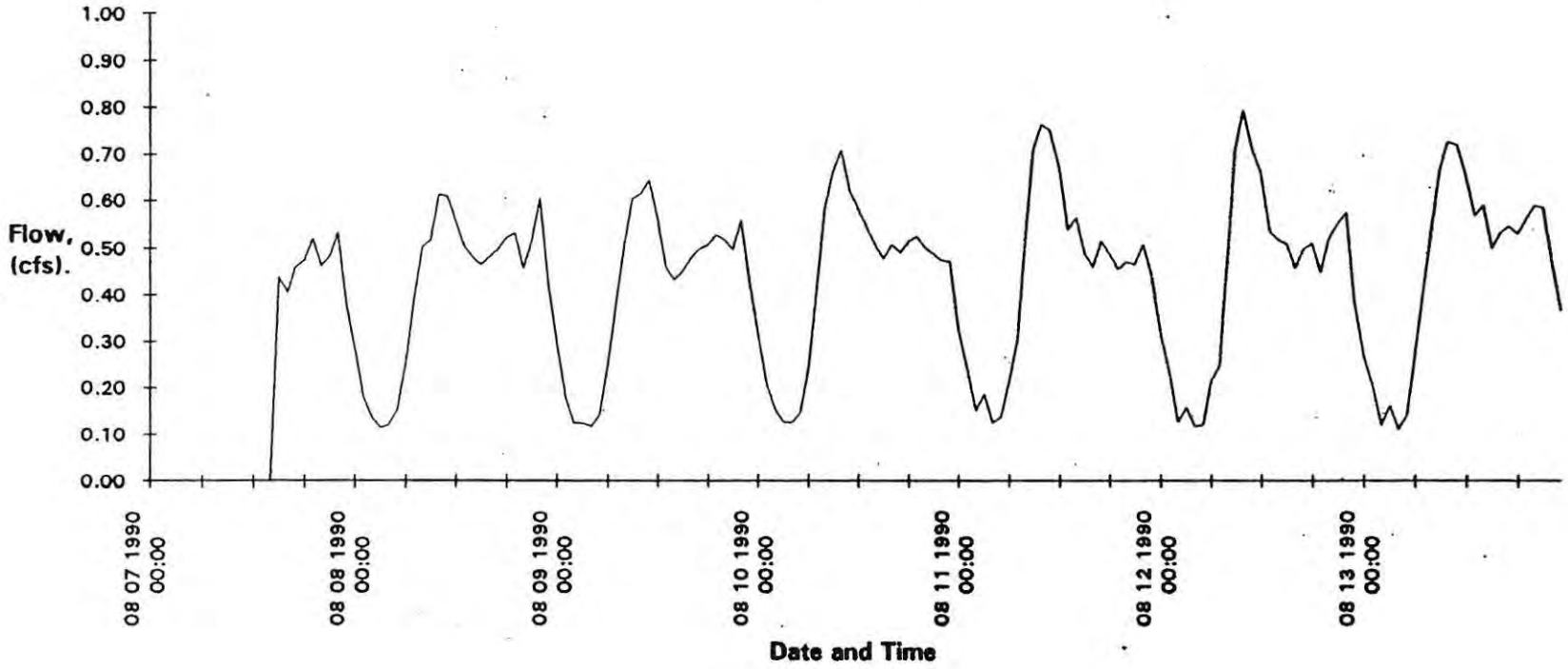
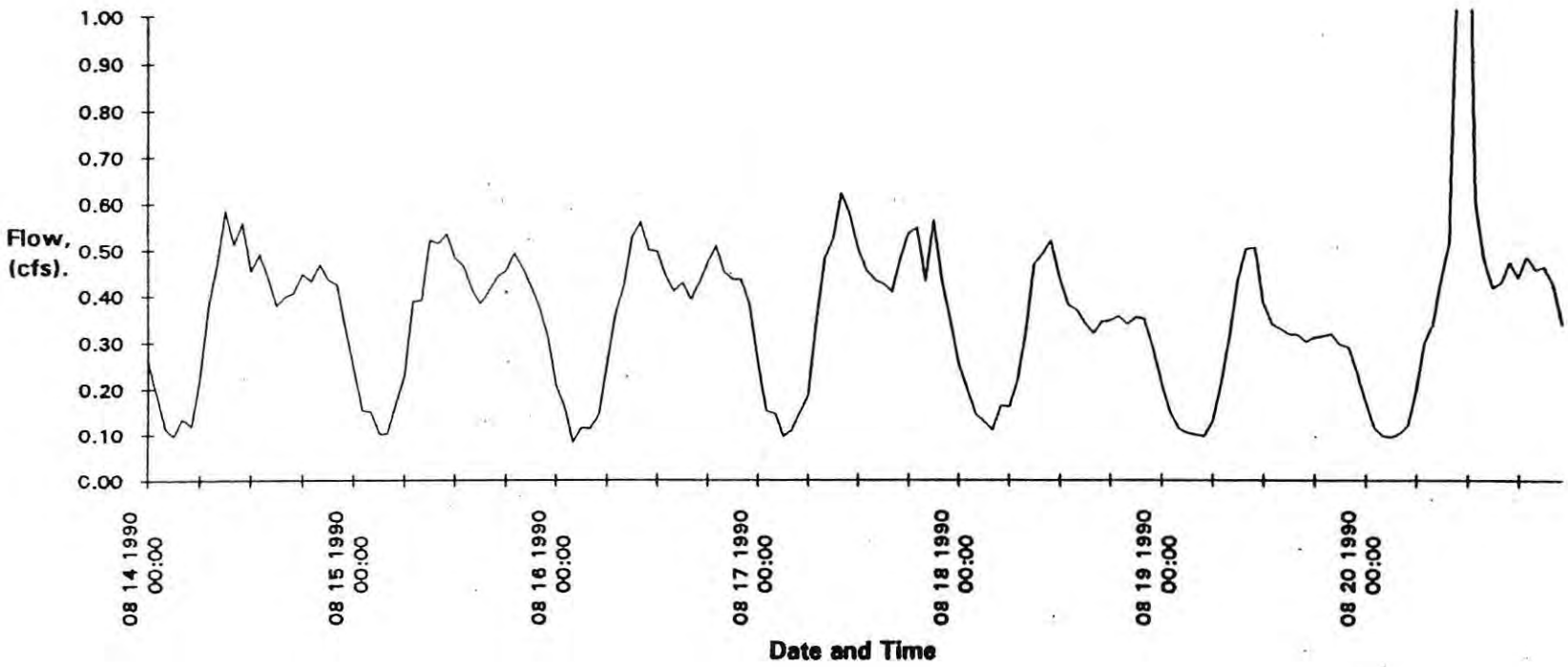
Date and Time

Volume 1
Page 500

Woodburn Facilities Plan
Dry Weather Flow
Meter 175



Woodburn Facilities Plan
Dry Weather Flow
Meter 175



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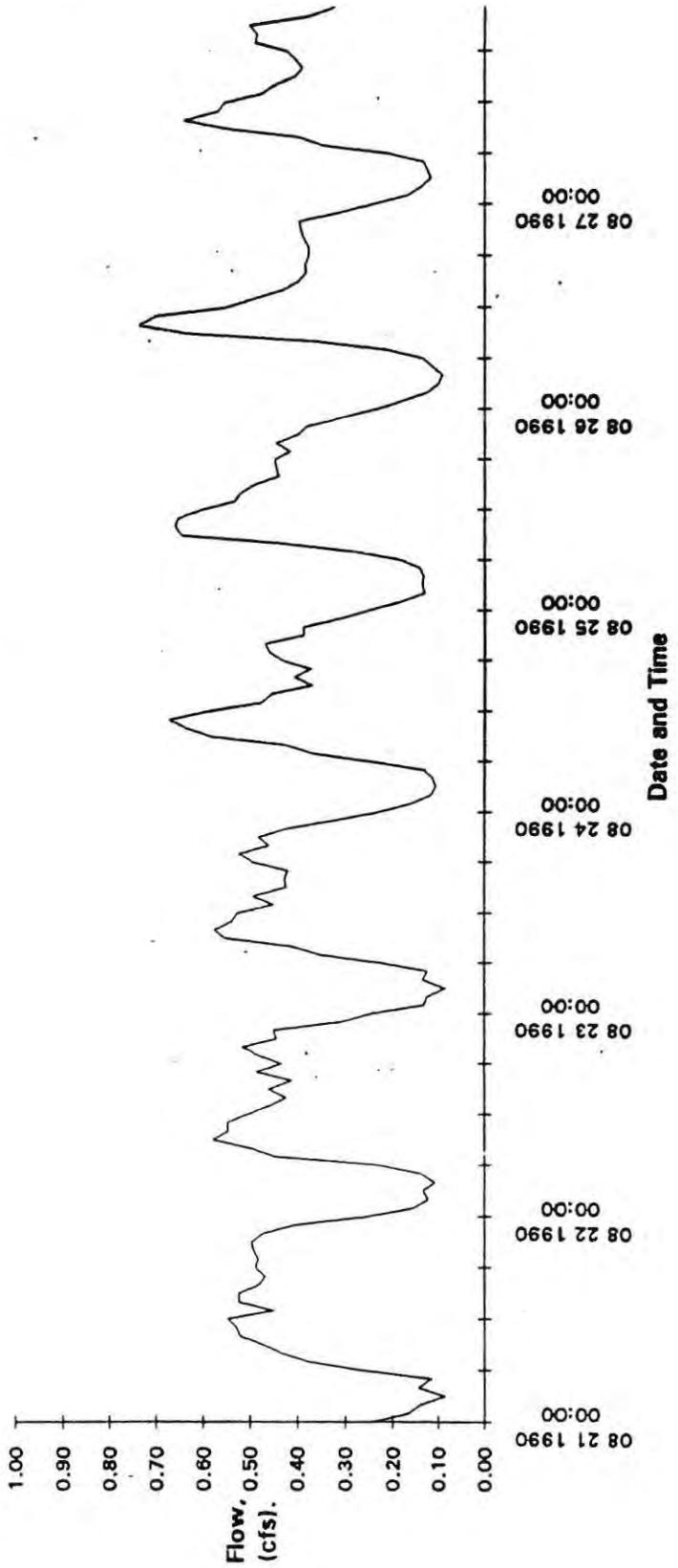
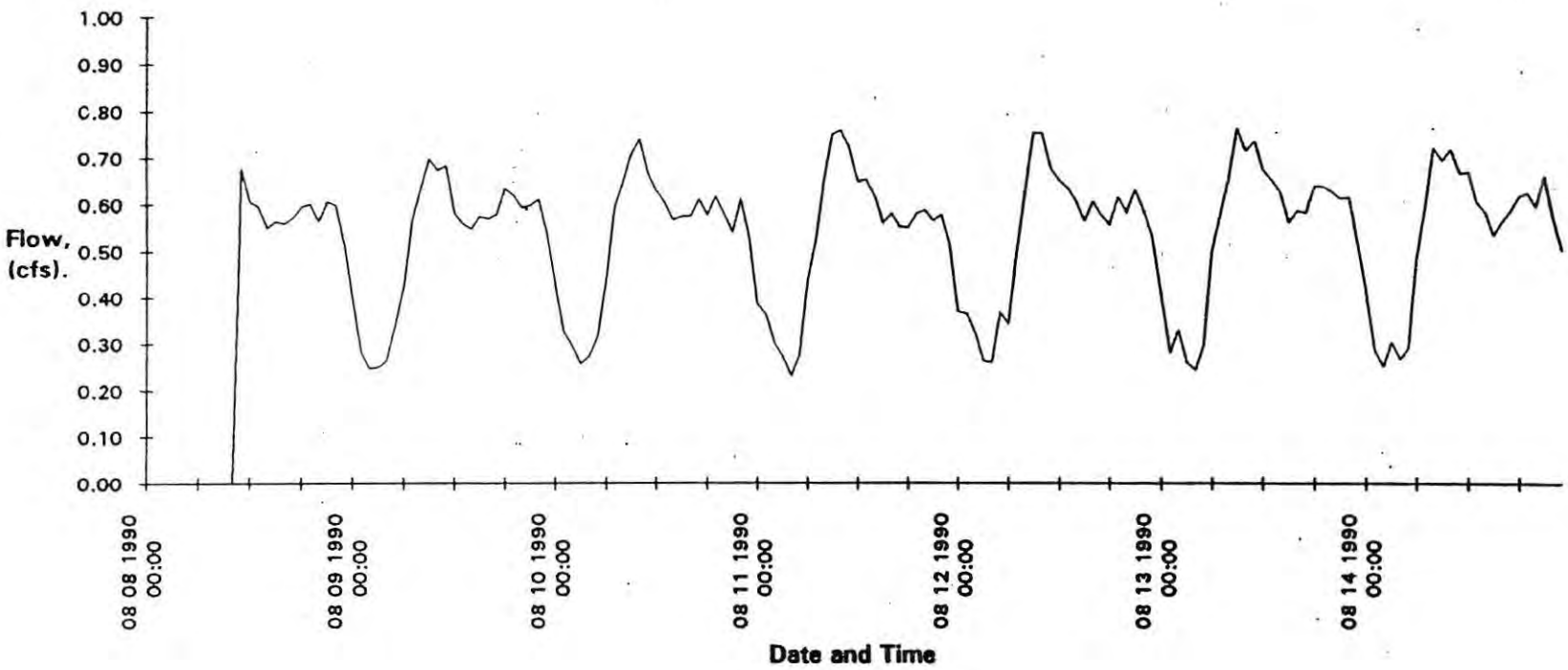
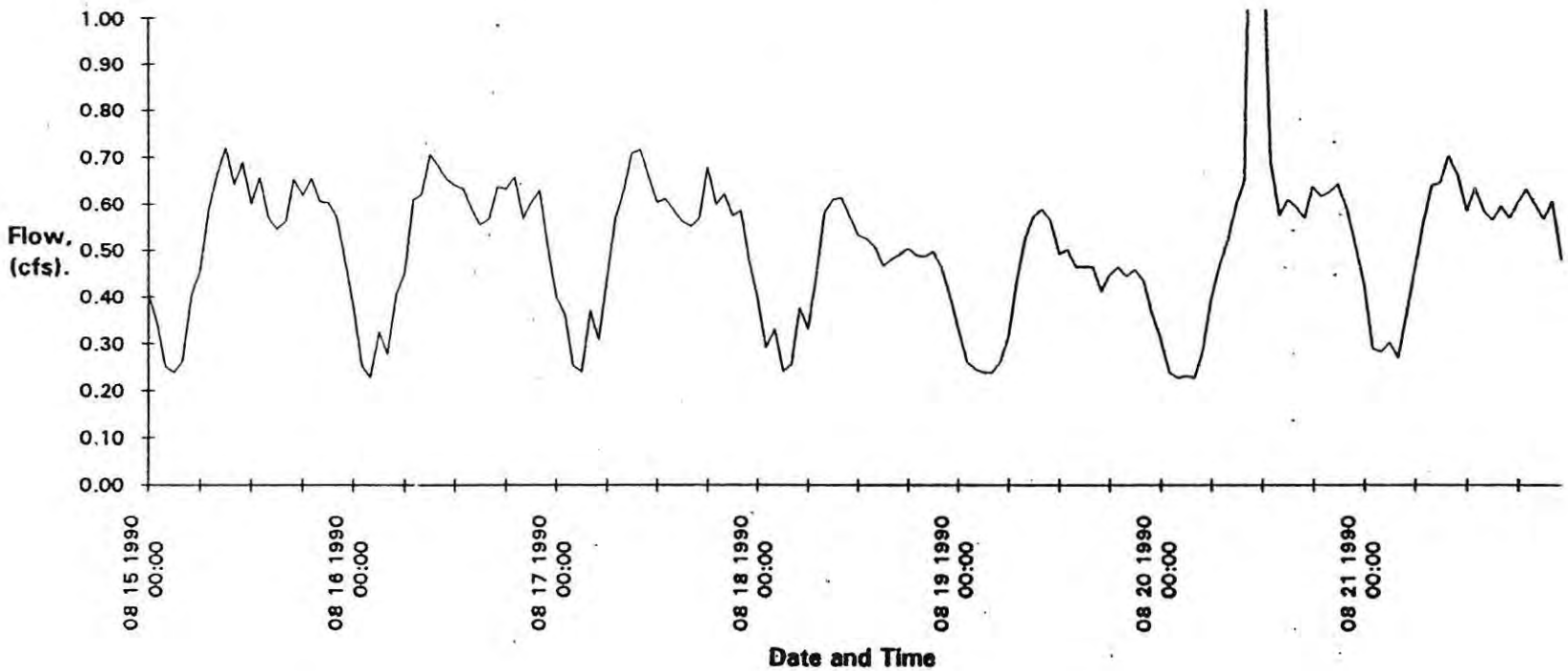


Volume 1
Page 502

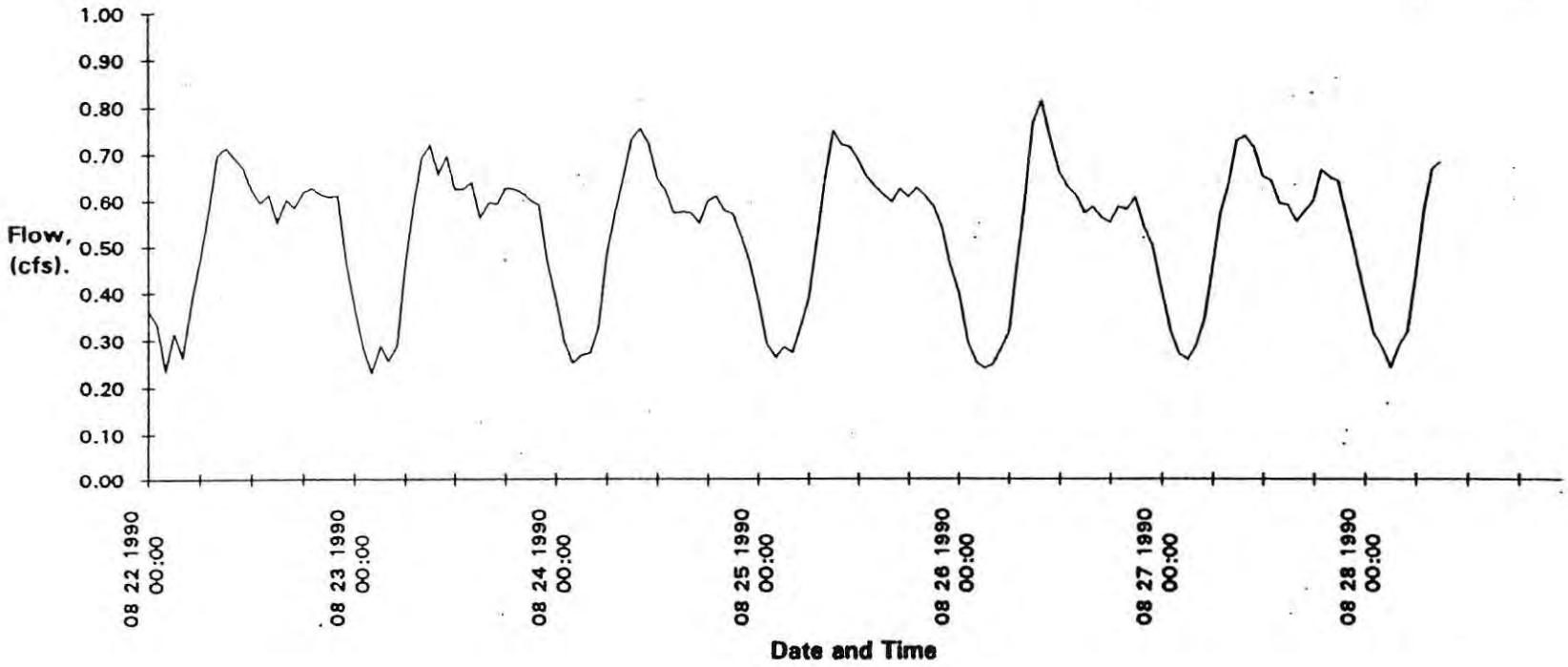
Woodburn Facilities Plan
Dry Weather Flow
Meter 181

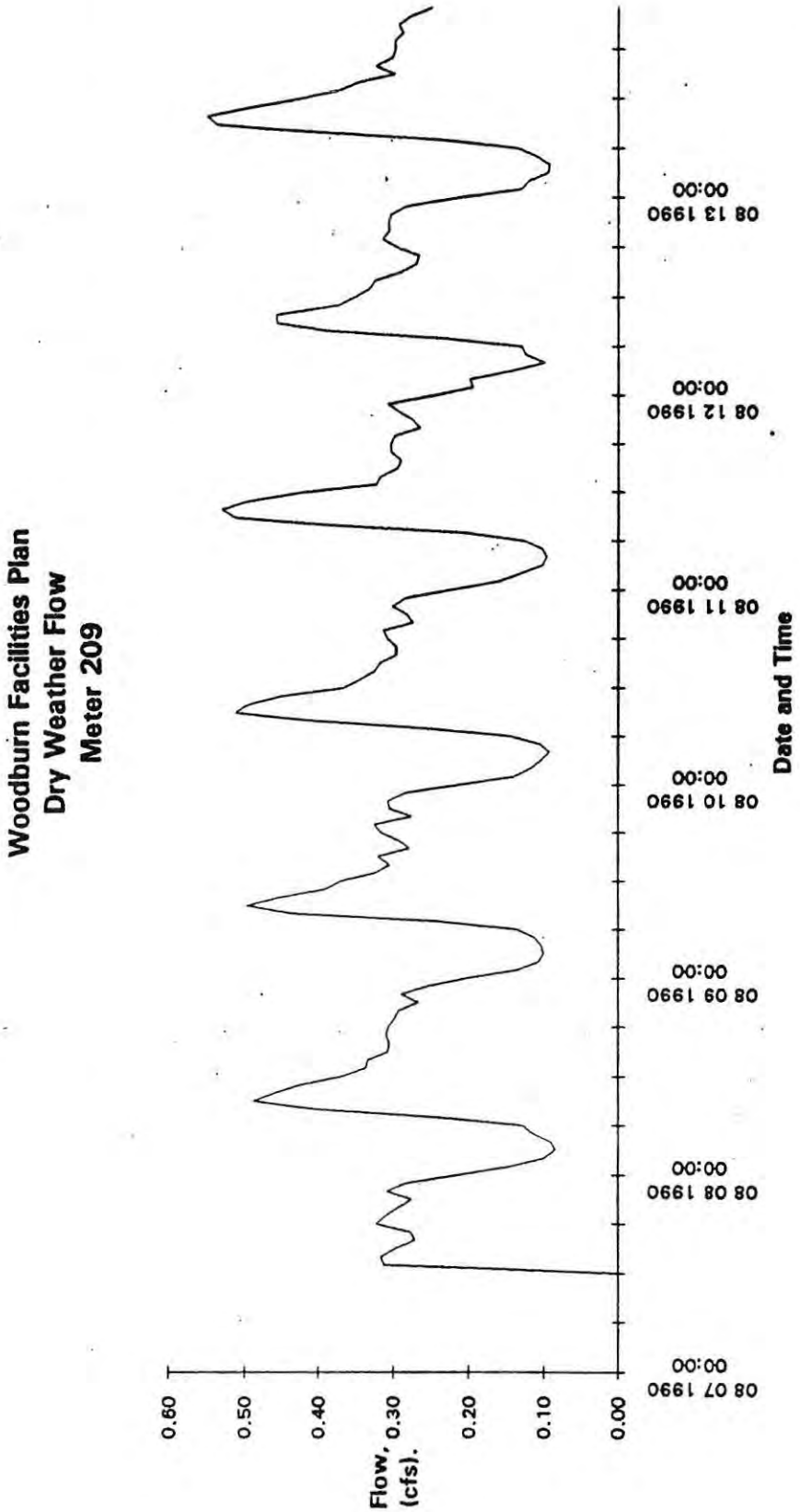


Woodburn Facilities Plan
Dry Weather Flow
Meter 181



Woodburn Facilities Plan
Dry Weather Flow
Meter 181




Woodburn Facilities Plan
Dry Weather Flow
Meter 209
Flow, (cfs).
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0.30
0.20
0.10
0.00
08 07 1990 00:00
08 08 1990 00:00
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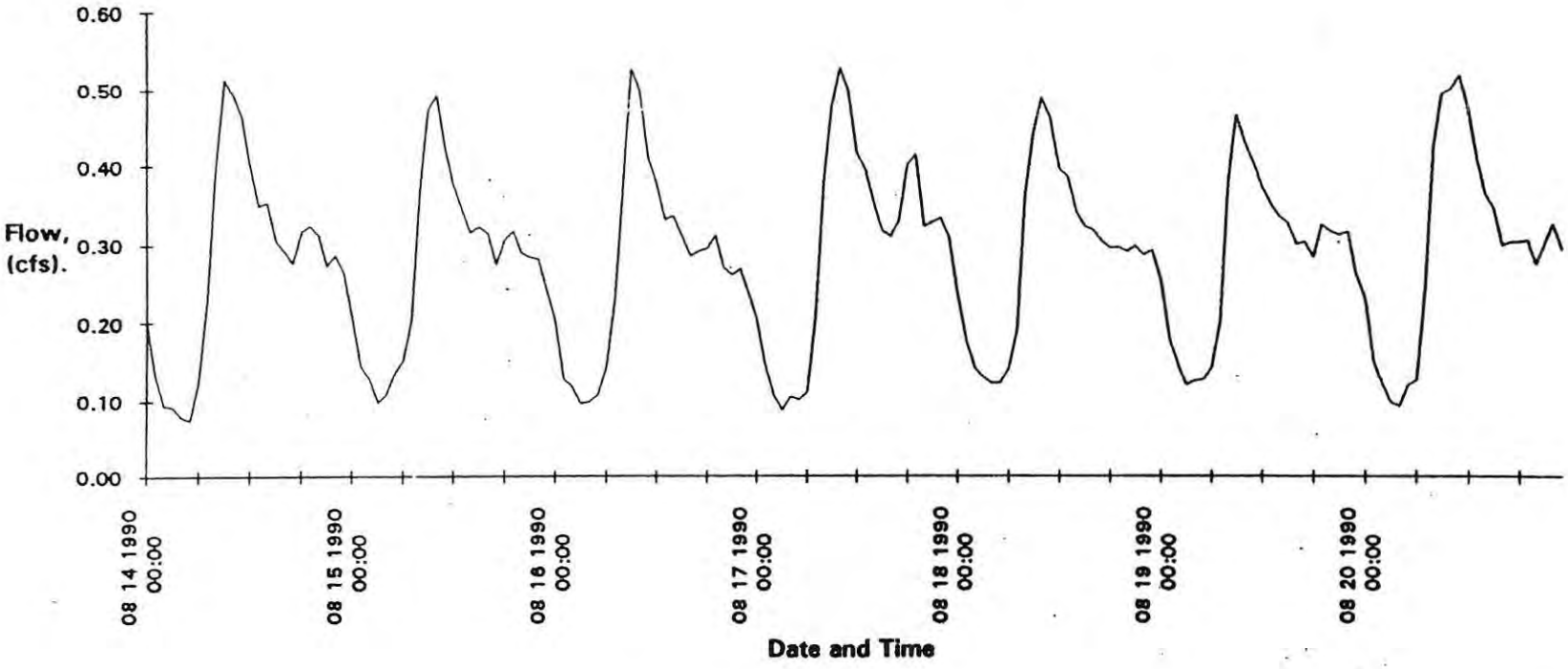
08 11 1990 00:00
08 12 1990 00:00
08 13 1990 00:00
Date and Time

Woodburn Facilities Plan
Dry Weather Flow
Meter 209



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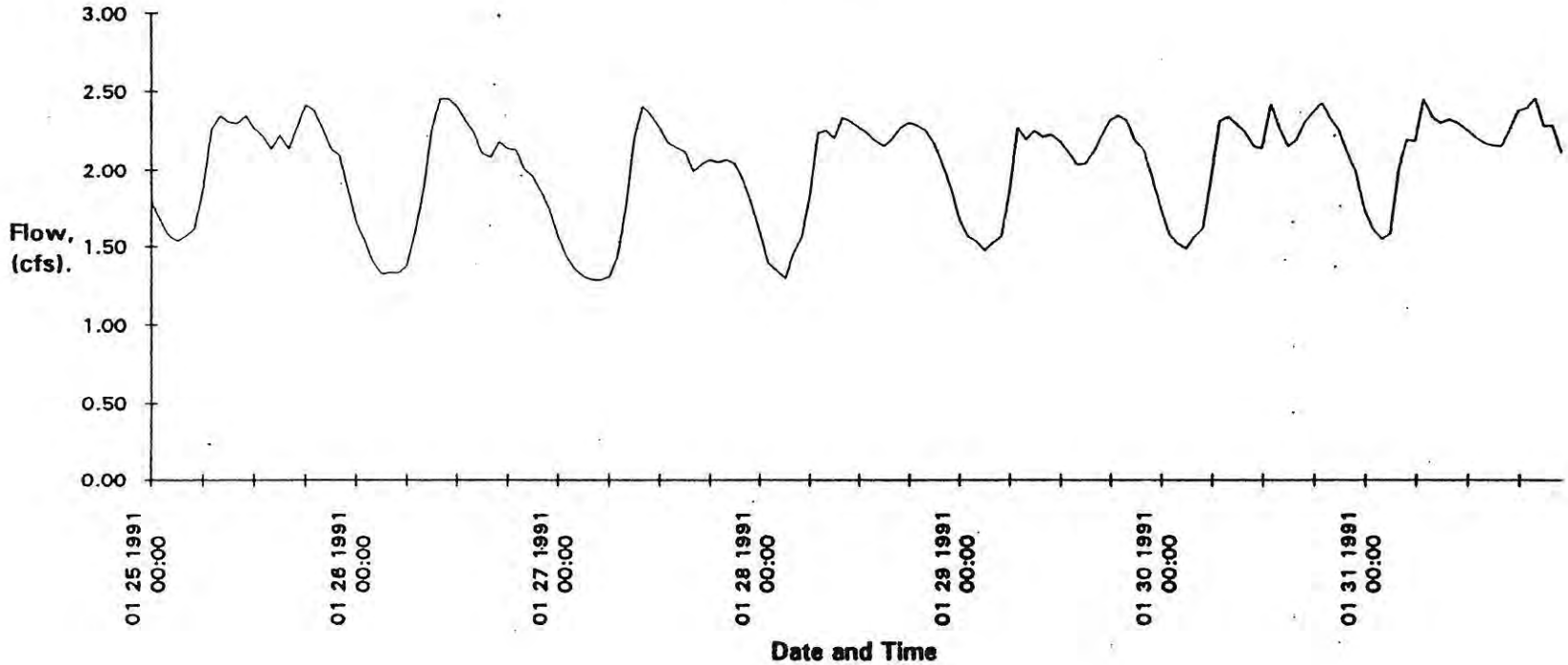
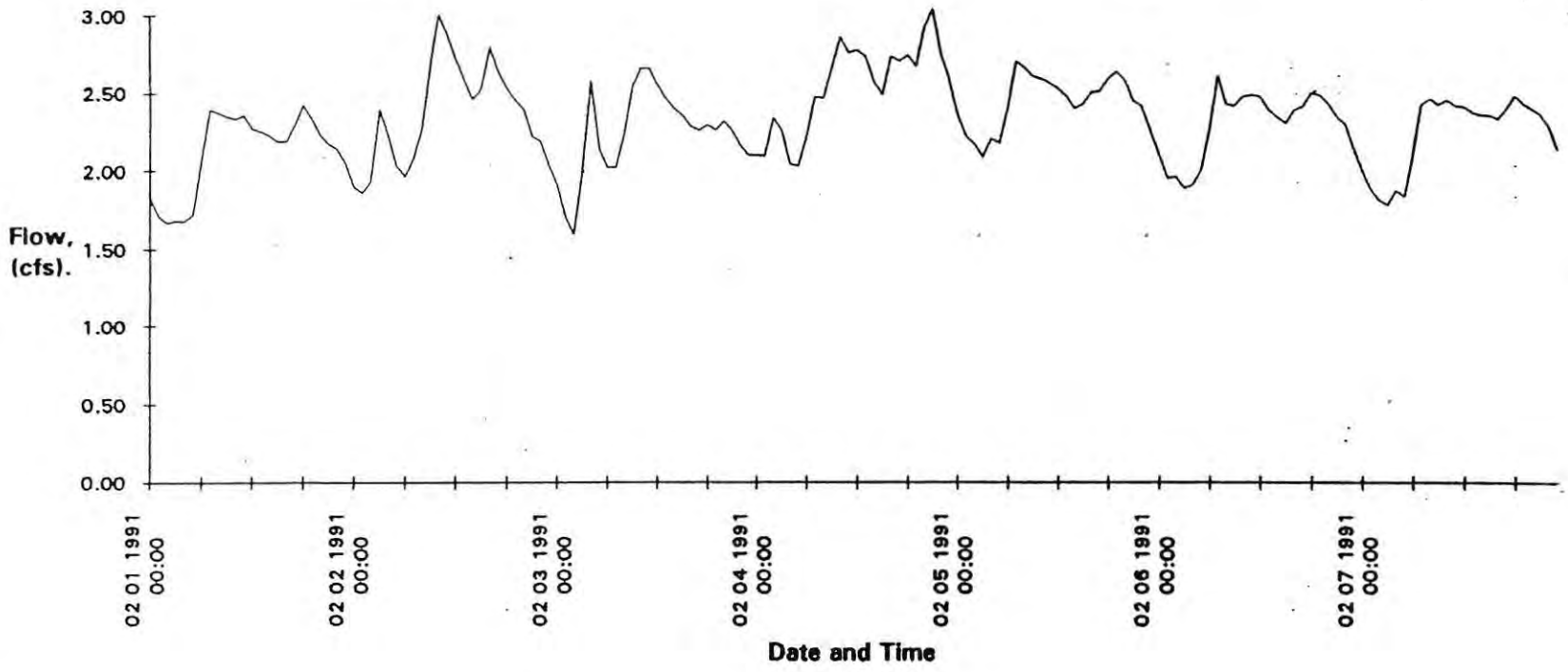
Wet Weather Flow
Meter 778



Woodburn Facilities Plan
Wet Weather Flow
Meter 778



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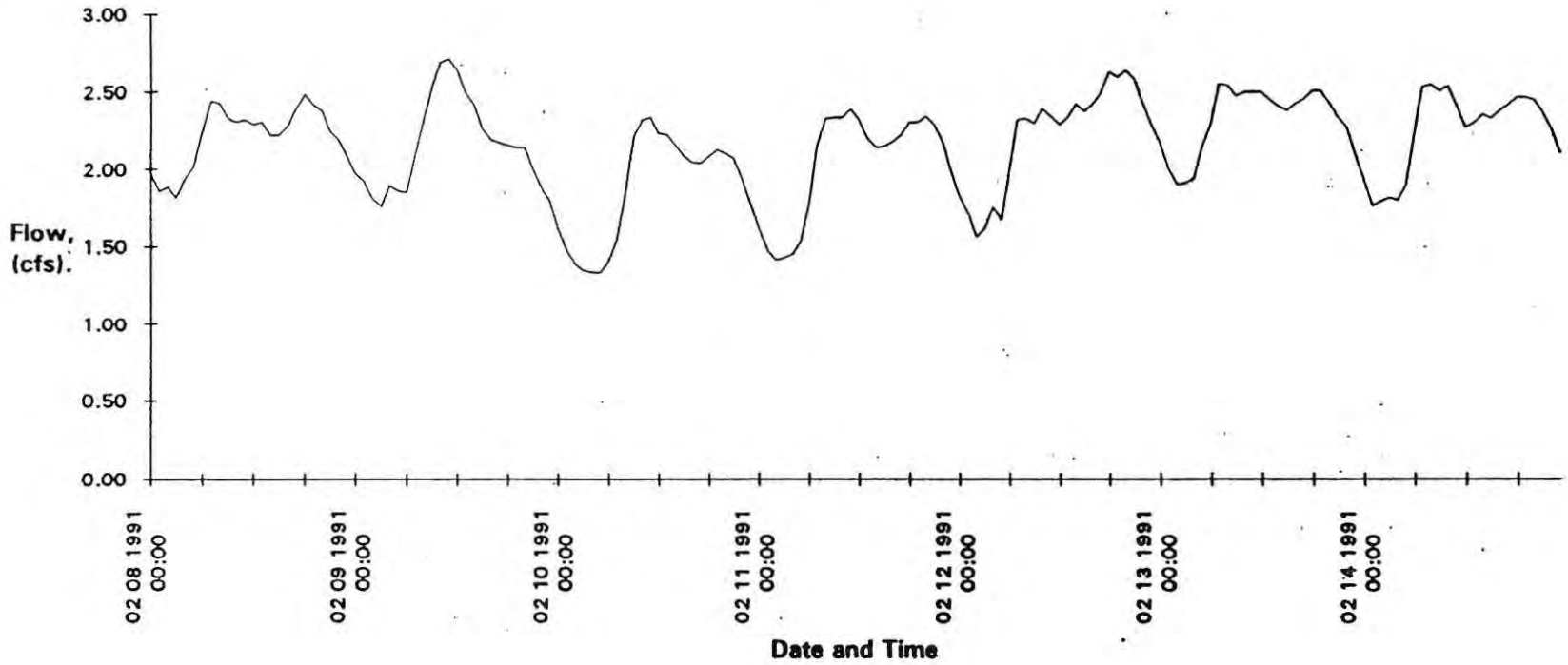
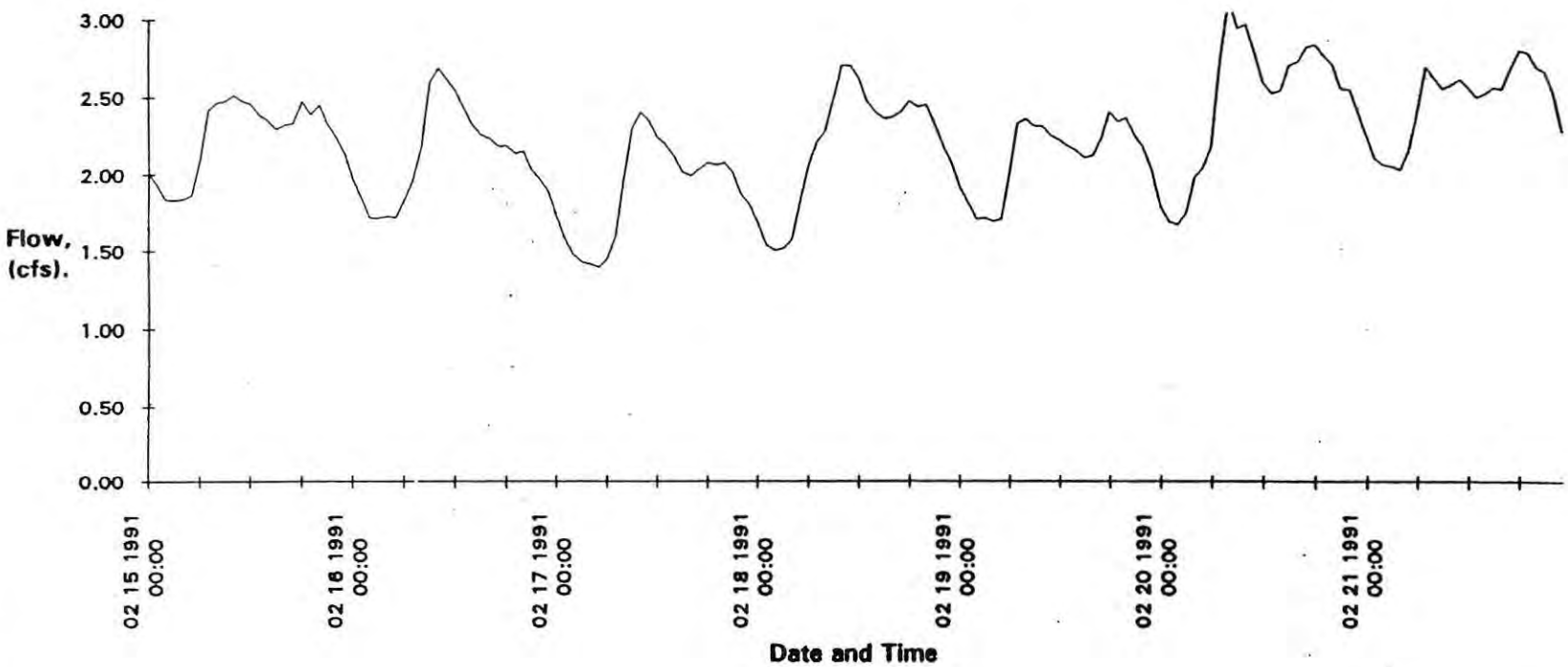
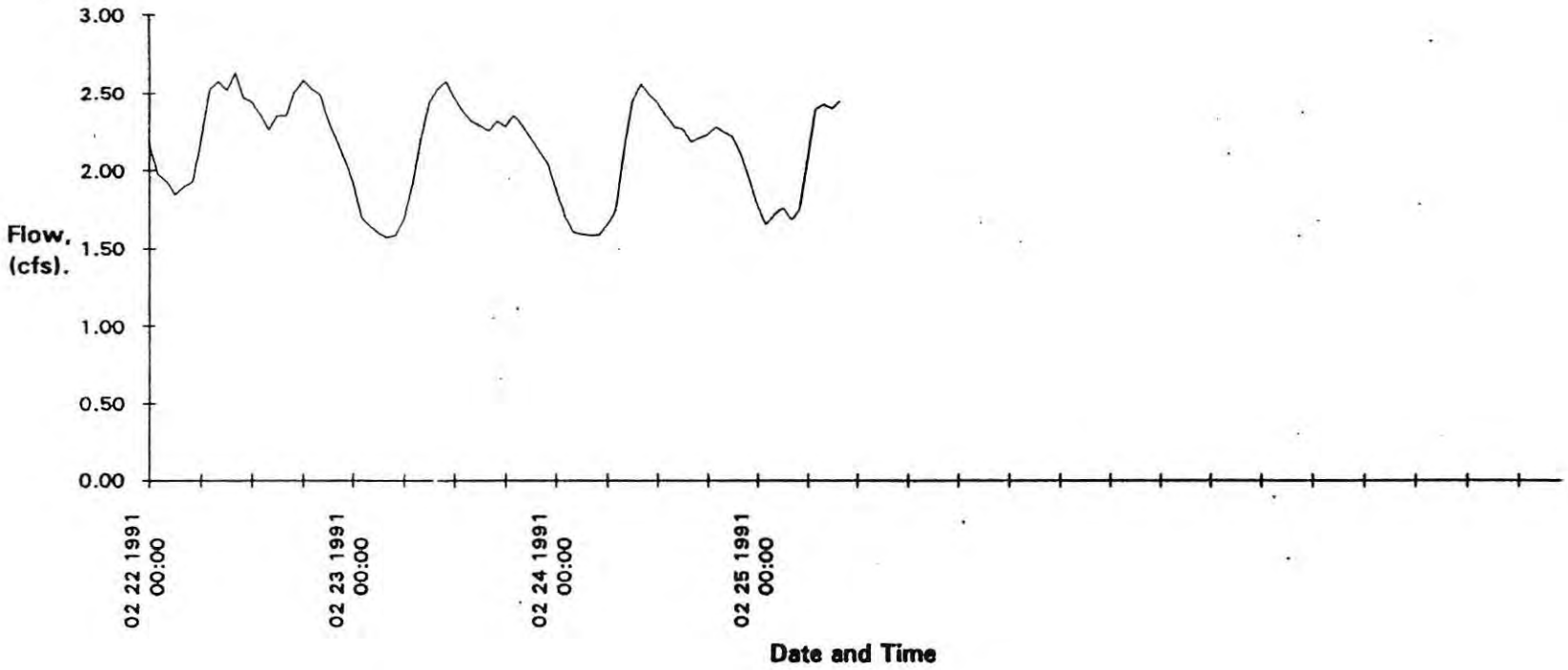
Wet Weather Flow
Meter 778

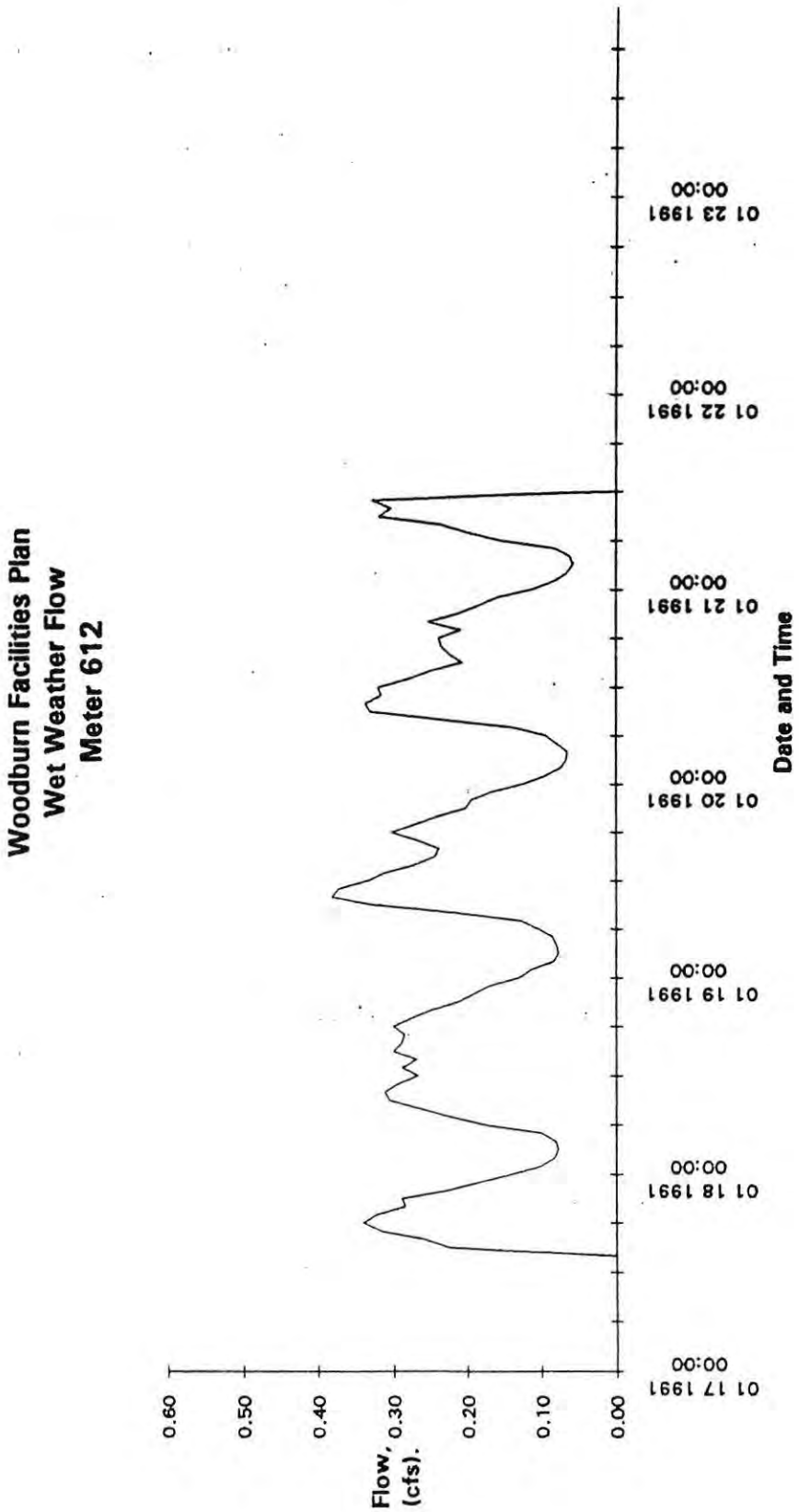


Woodburn Facilities Plan
Wet Weather Flow
Meter 778



Woodburn Facilities Plan
Wet Weather Flow
Meter 778




Woodburn Facilities Plan
Wet Weather Flow
Meter 612
Flow, (cfs).
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0.40
0.30
0.20
0.10
0.00
01 17 1991 00:00
01 18 1991 00:00
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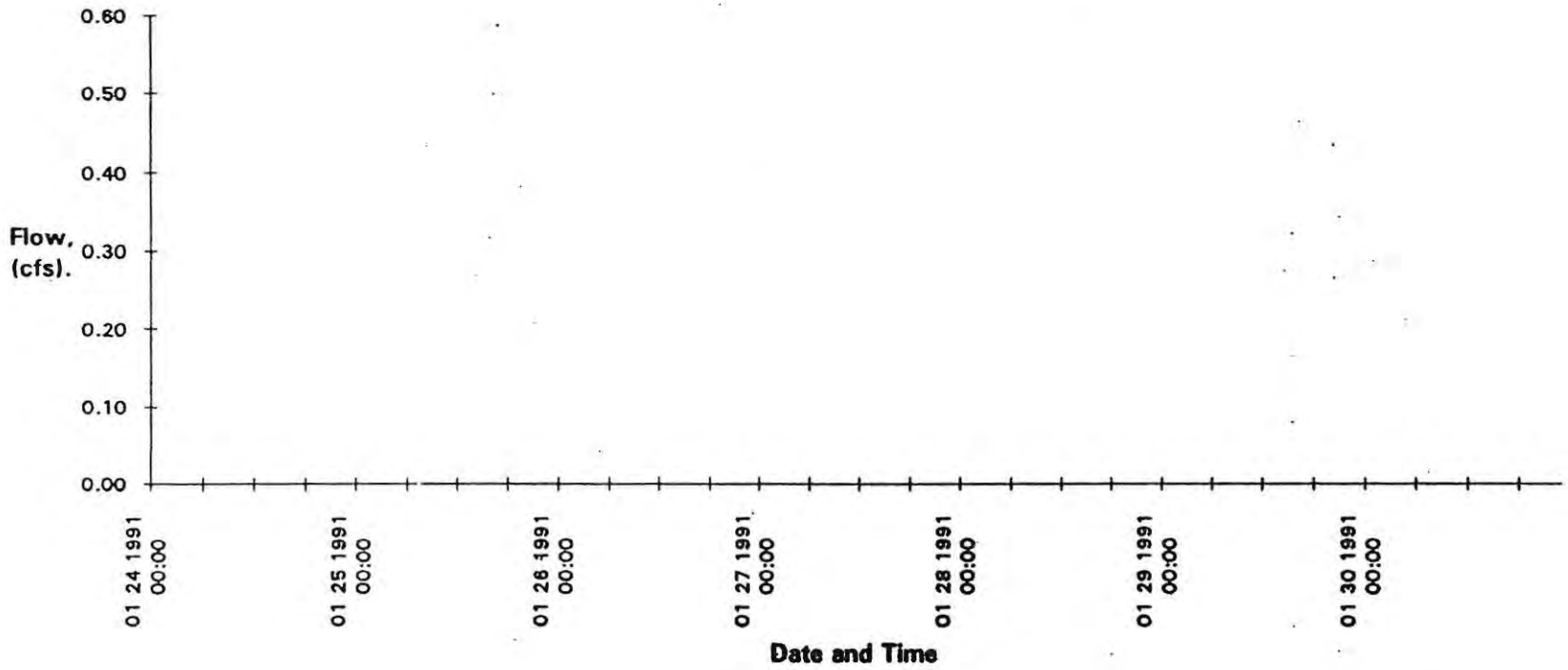
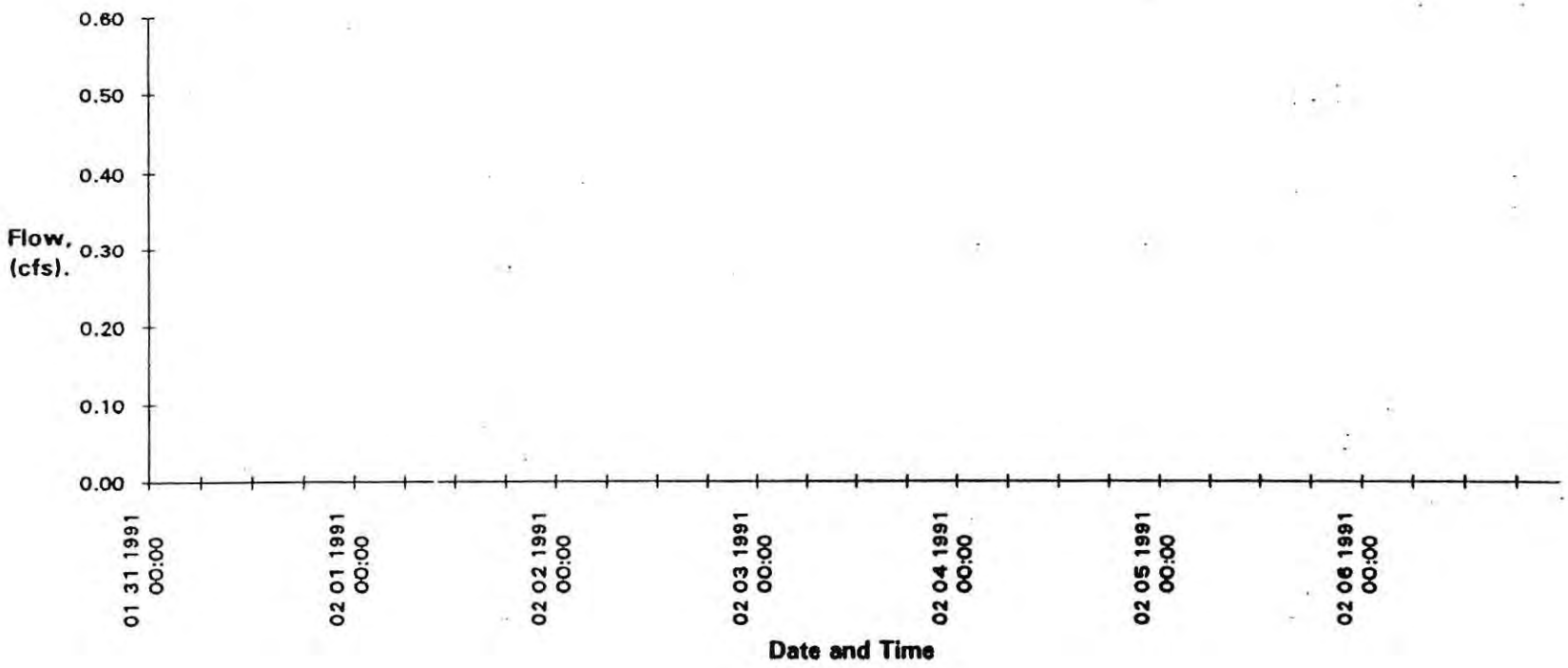
01 23 1991 00:00
Date and Time

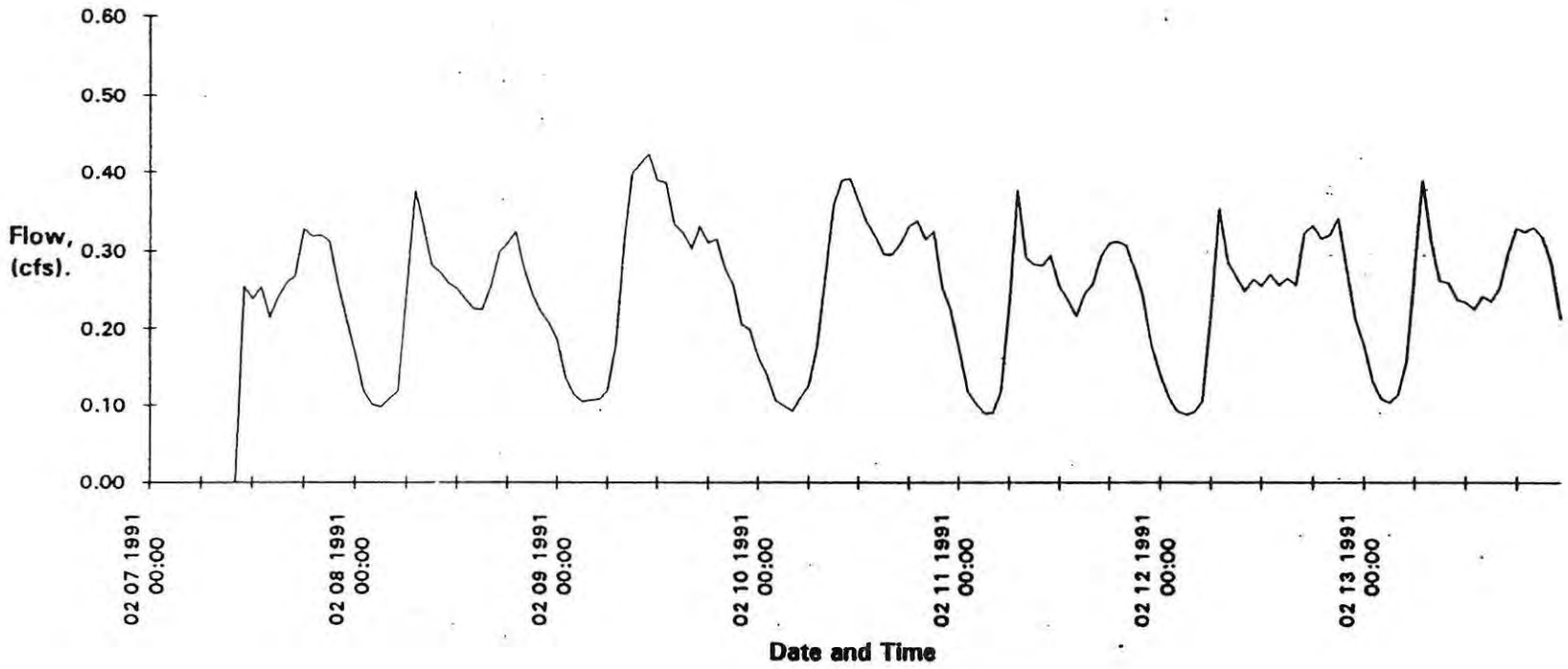
Woodburn Facilities Plan
Wet Weather Flow
Meter 612



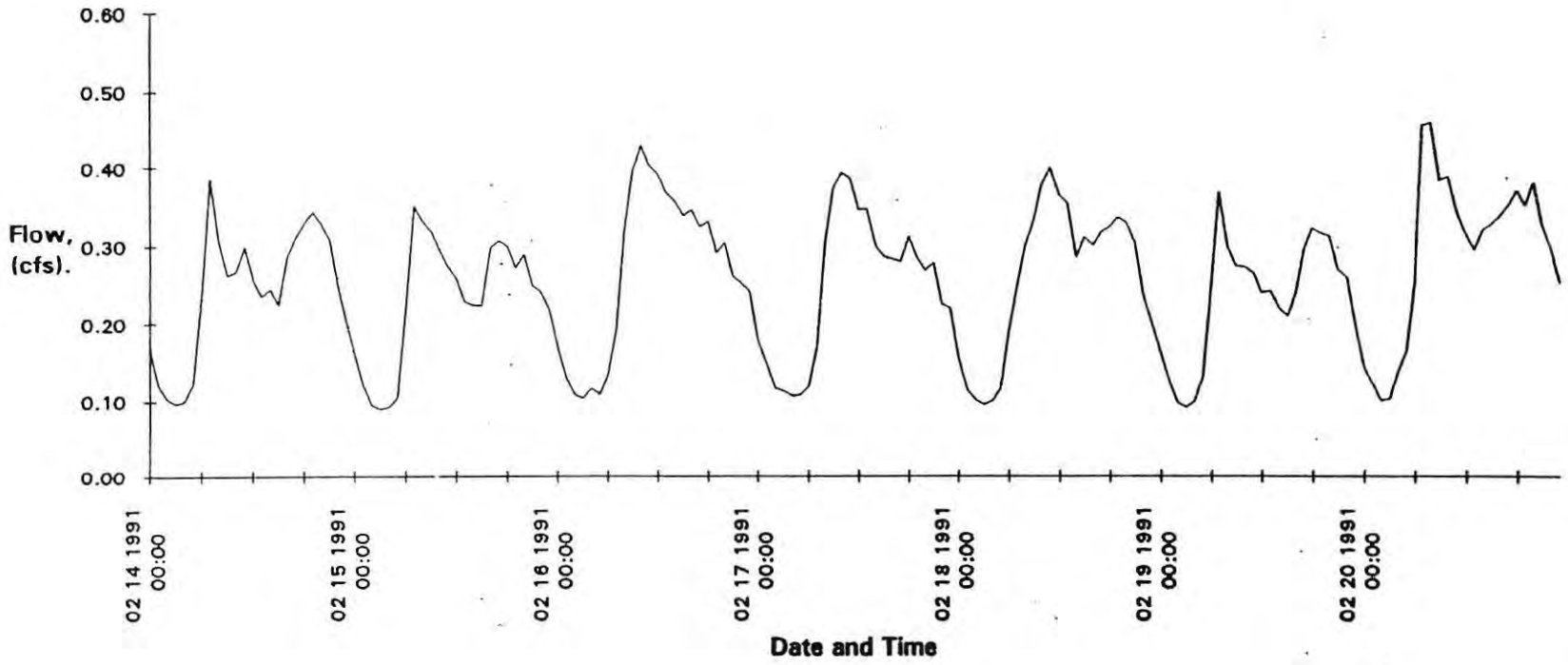
Woodburn Facilities Plan
Wet Weather Flow
Meter 612



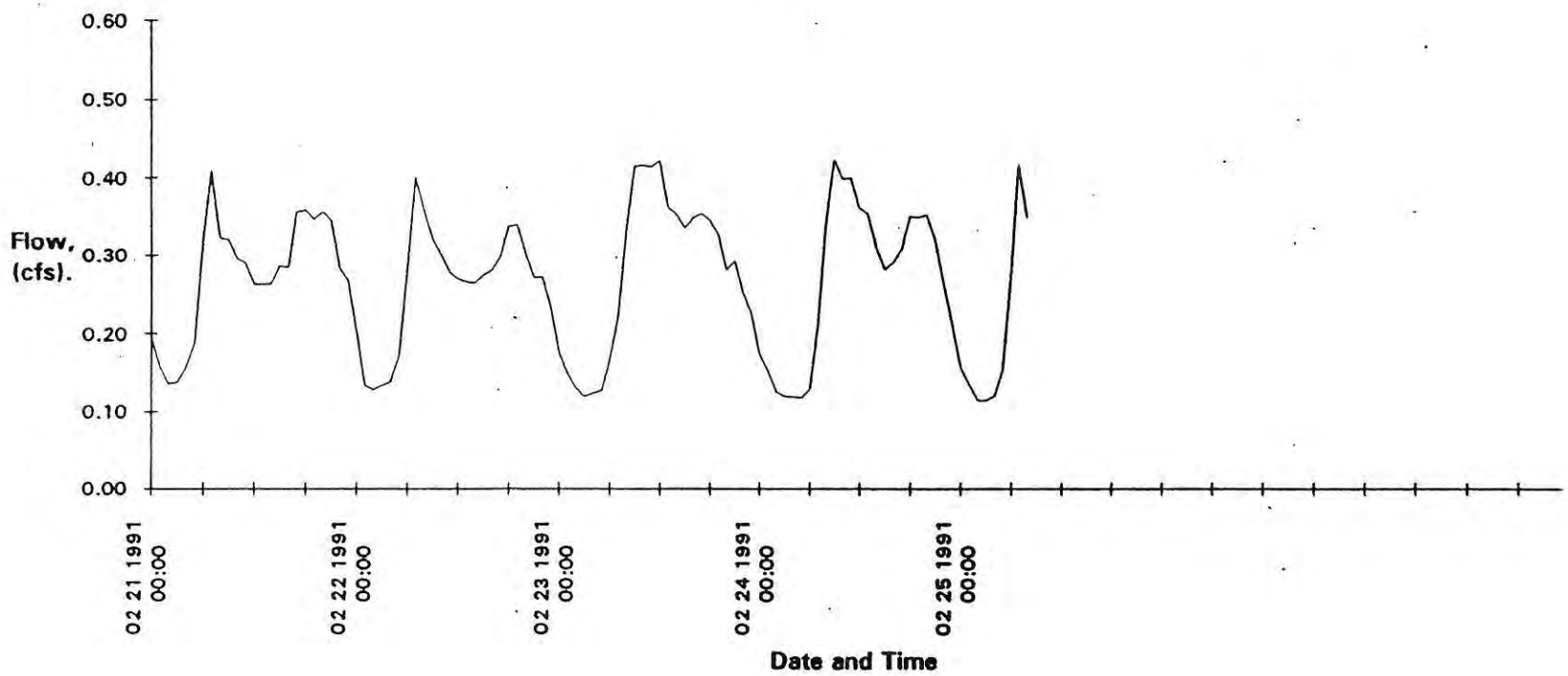
Woodburn Facilities Plan
Wet Weather Flow
Meter 612



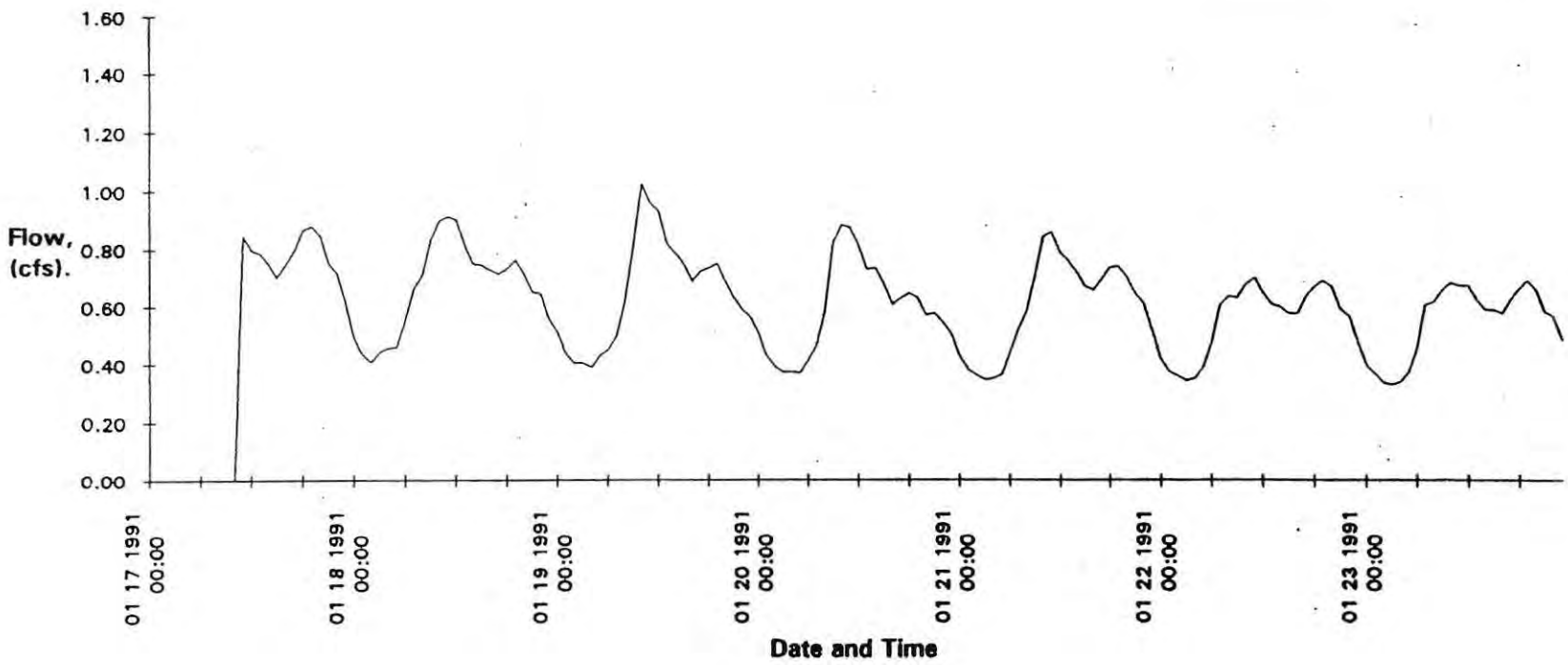
Woodburn Facilities Plan
Wet Weather Flow
Meter 612



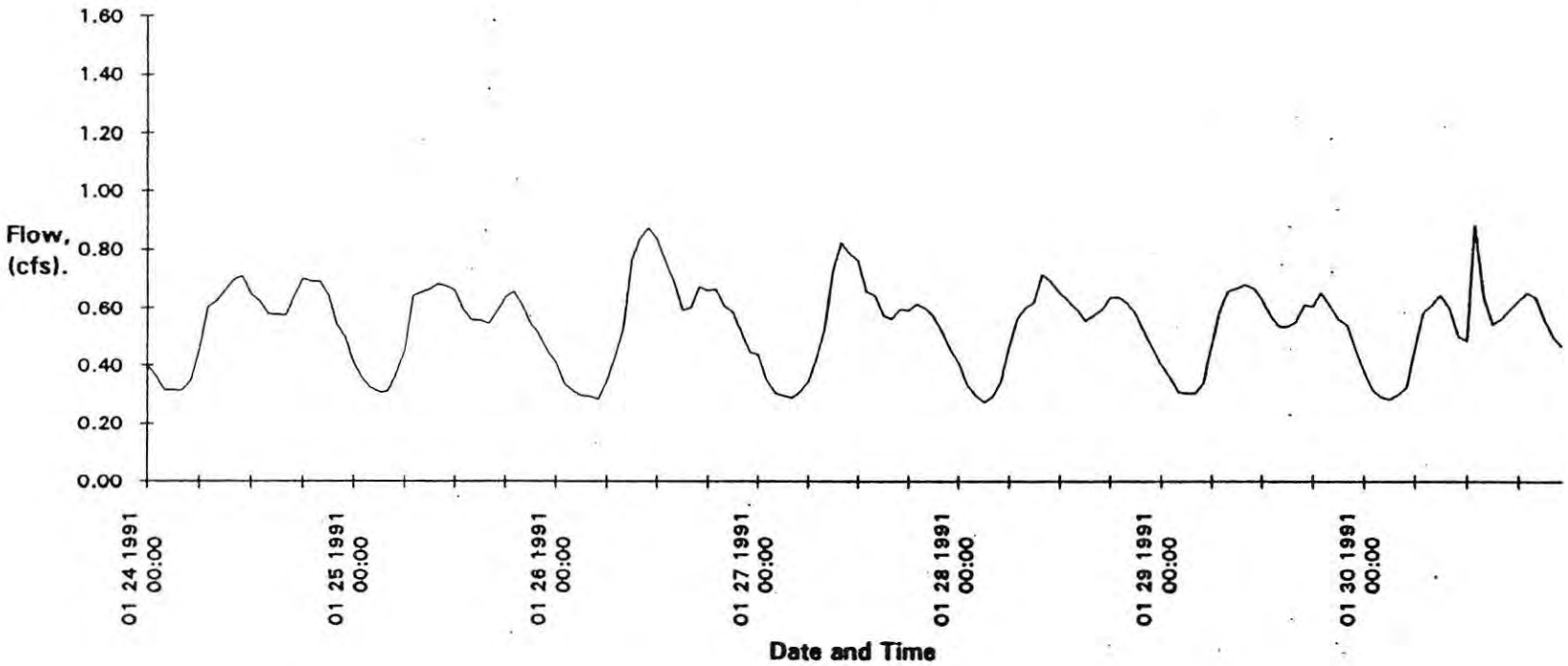
Woodburn Facilities Plan
Wet Weather Flow
Meter 612



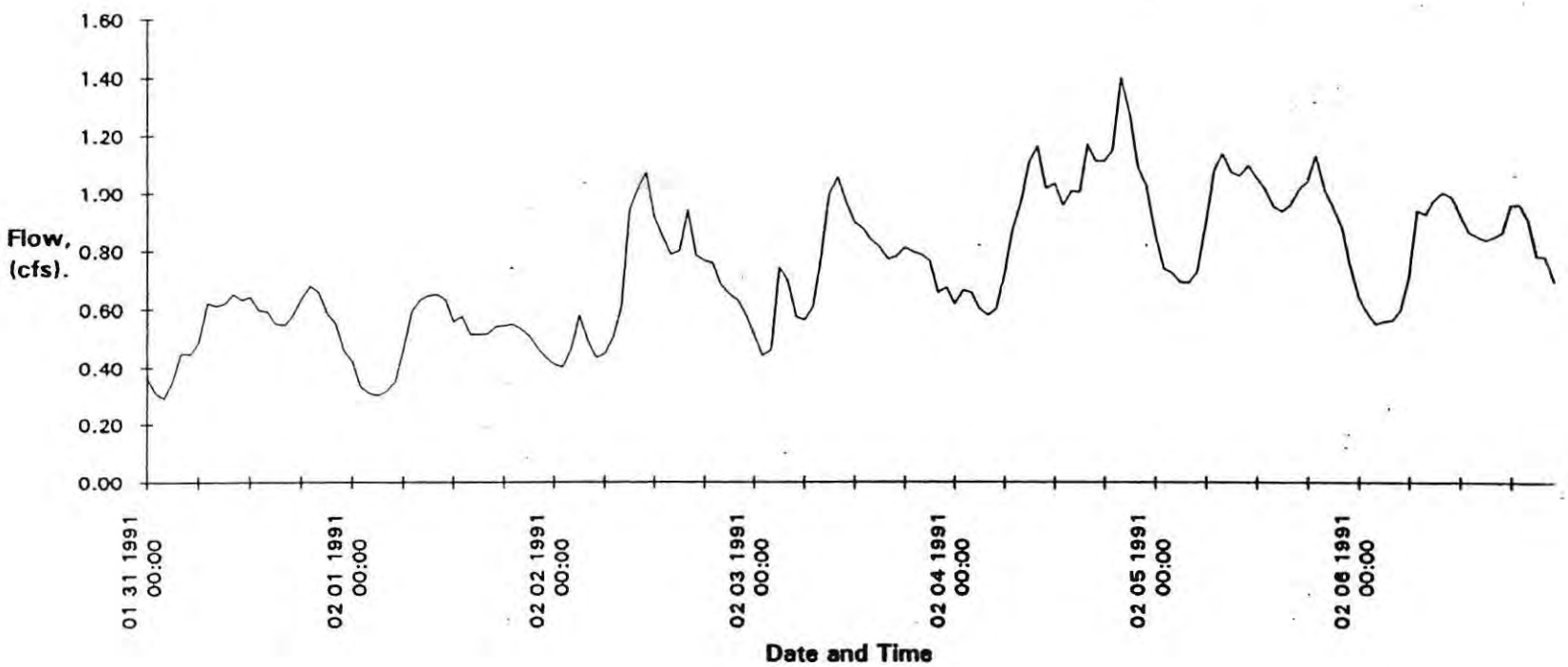
Woodburn Facilities Plan
Wet Weather Flow
Meter 104



Woodburn Facilities Plan
Wet Weather Flow
Meter 104

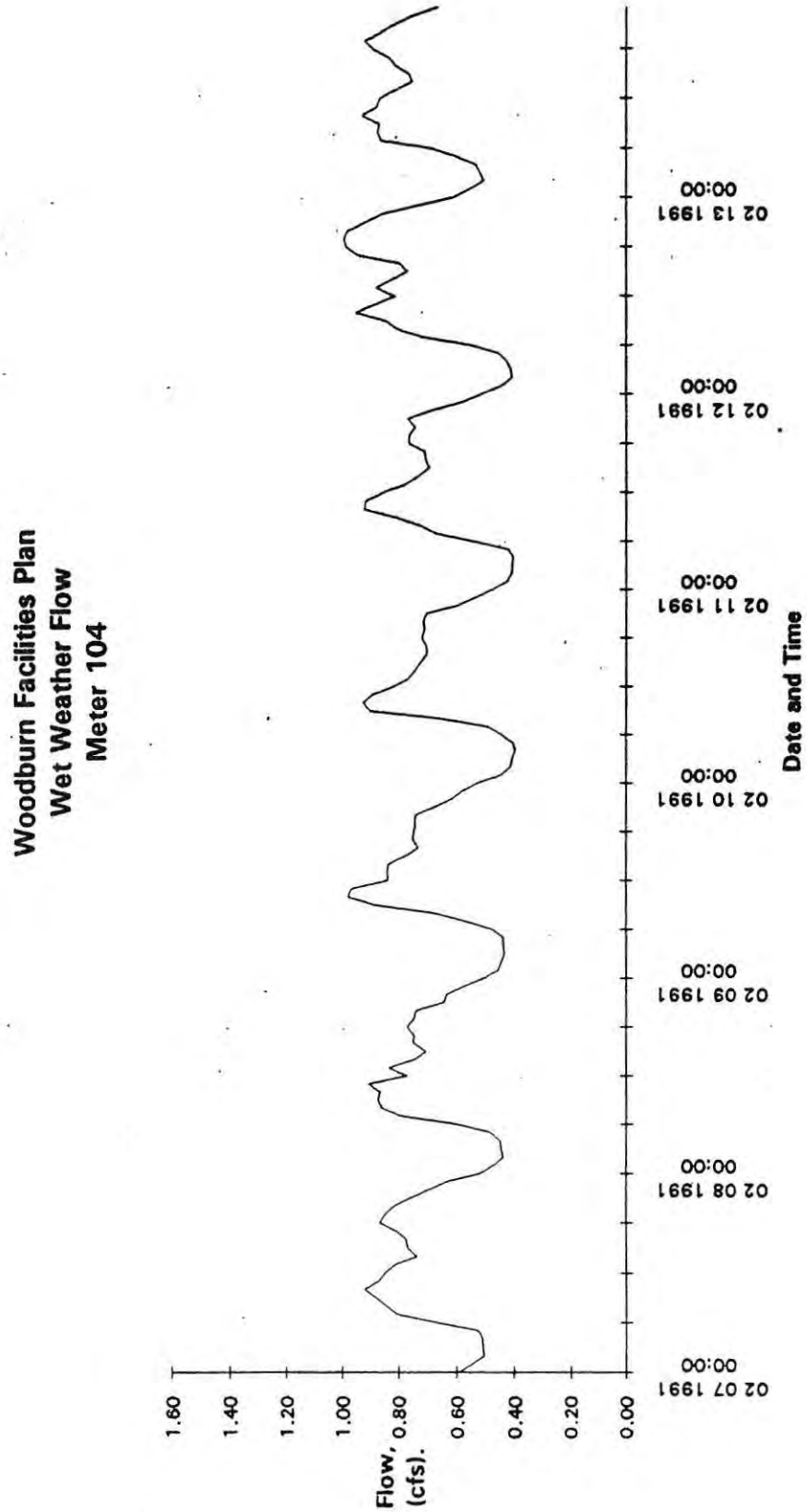


Woodburn Facilities Plan
Wet Weather Flow
Meter 104



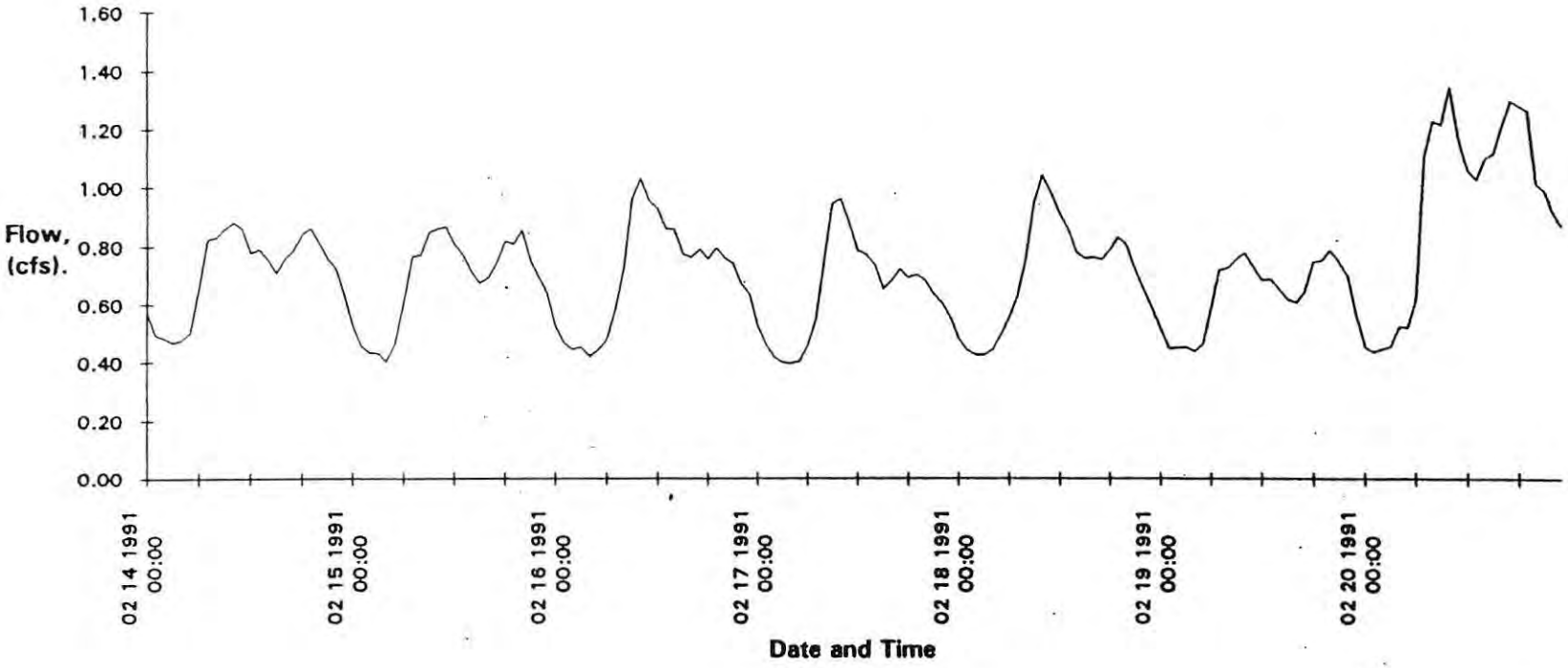
M104_4.XLC

PDX27874.F4 2/15/93

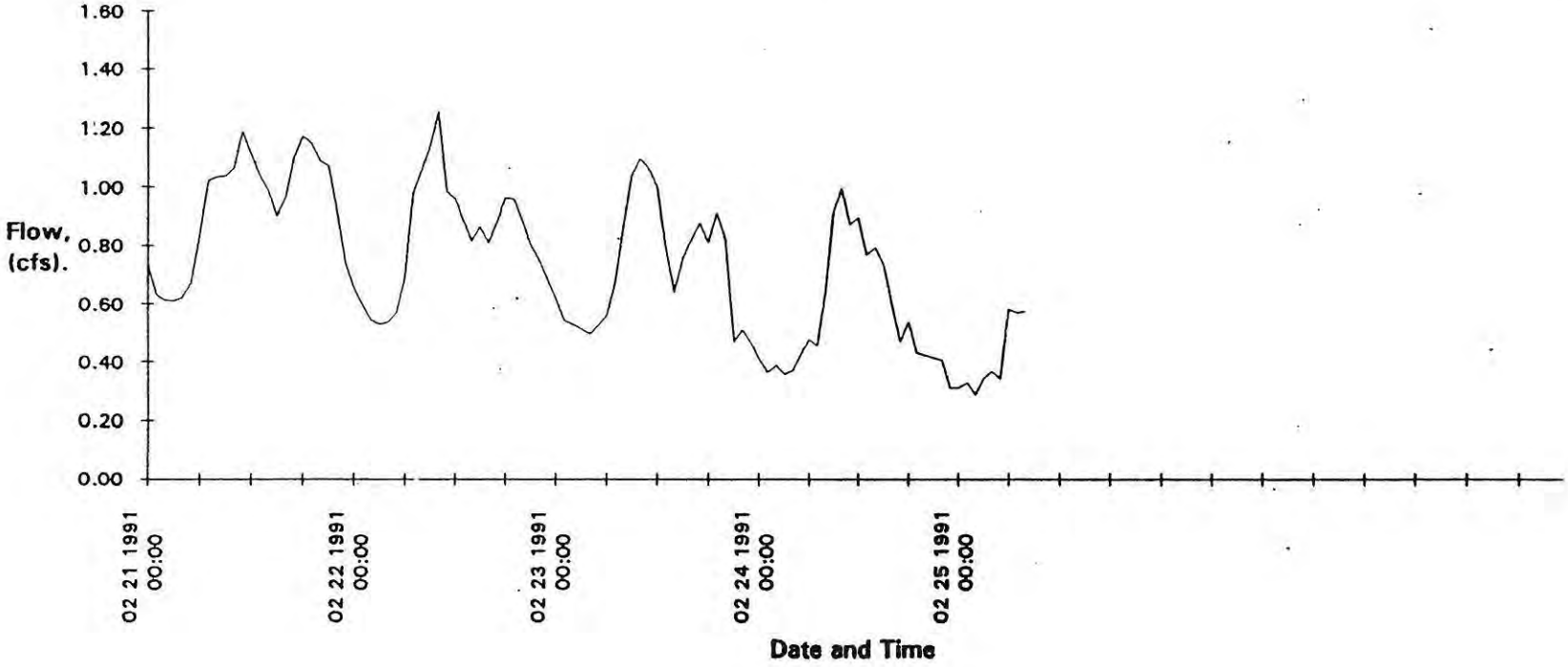


Volume 1
Page 522

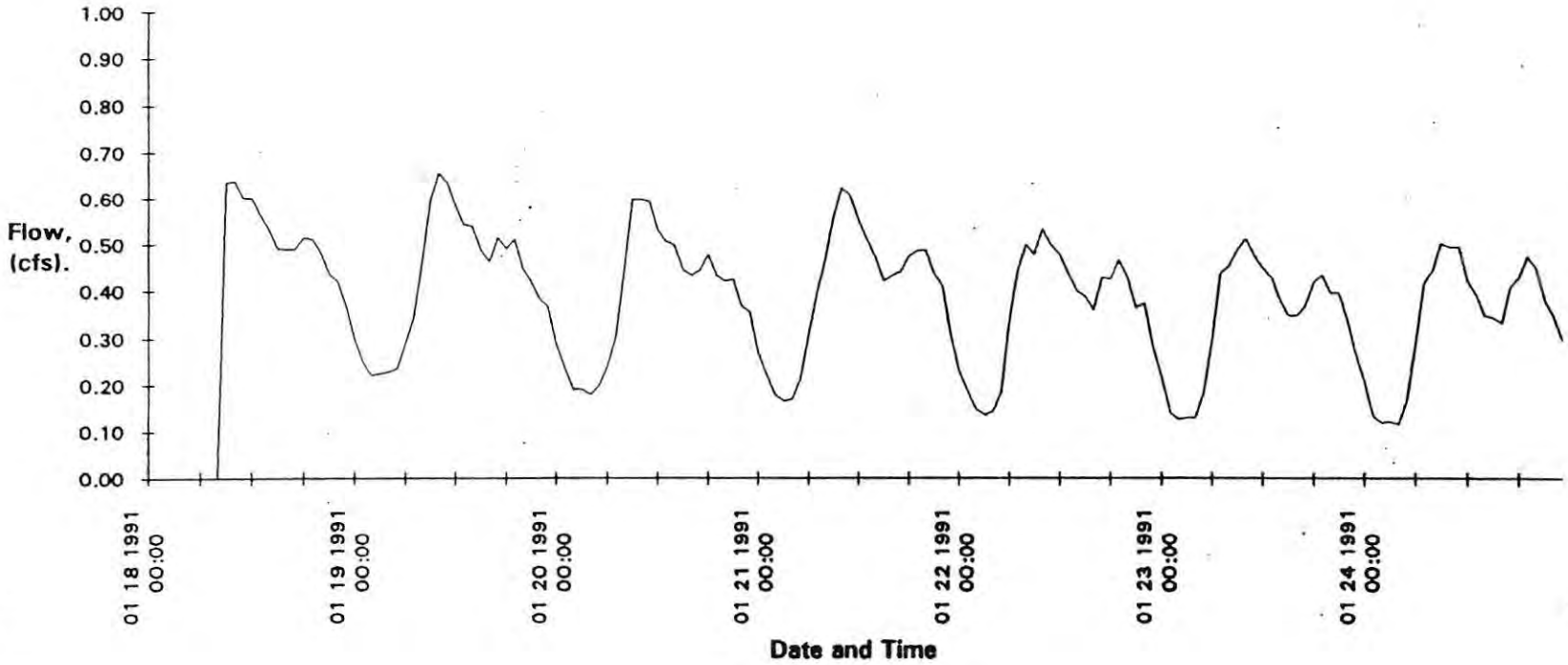
Woodburn Facilities Plan
Wet Weather Flow
Meter 104



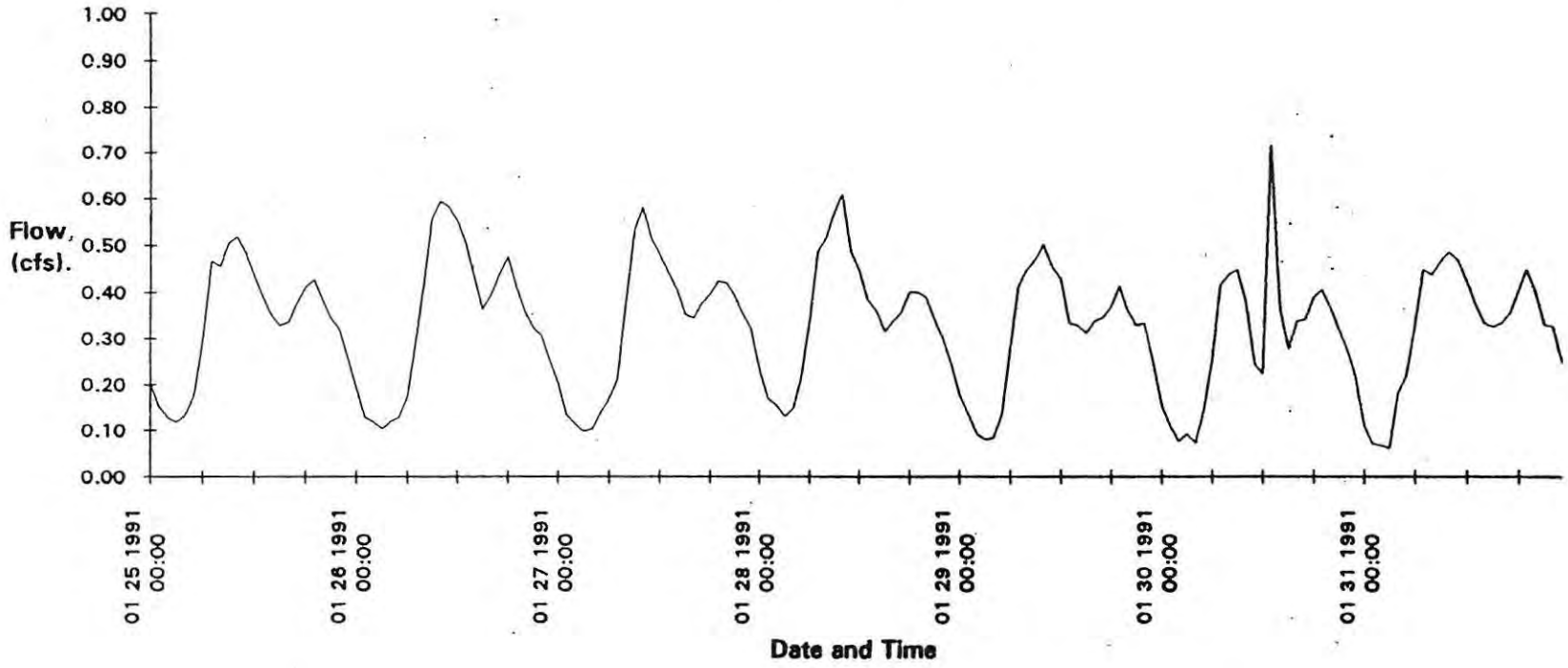
Woodburn Facilities Plan
Wet Weather Flow
Meter 104

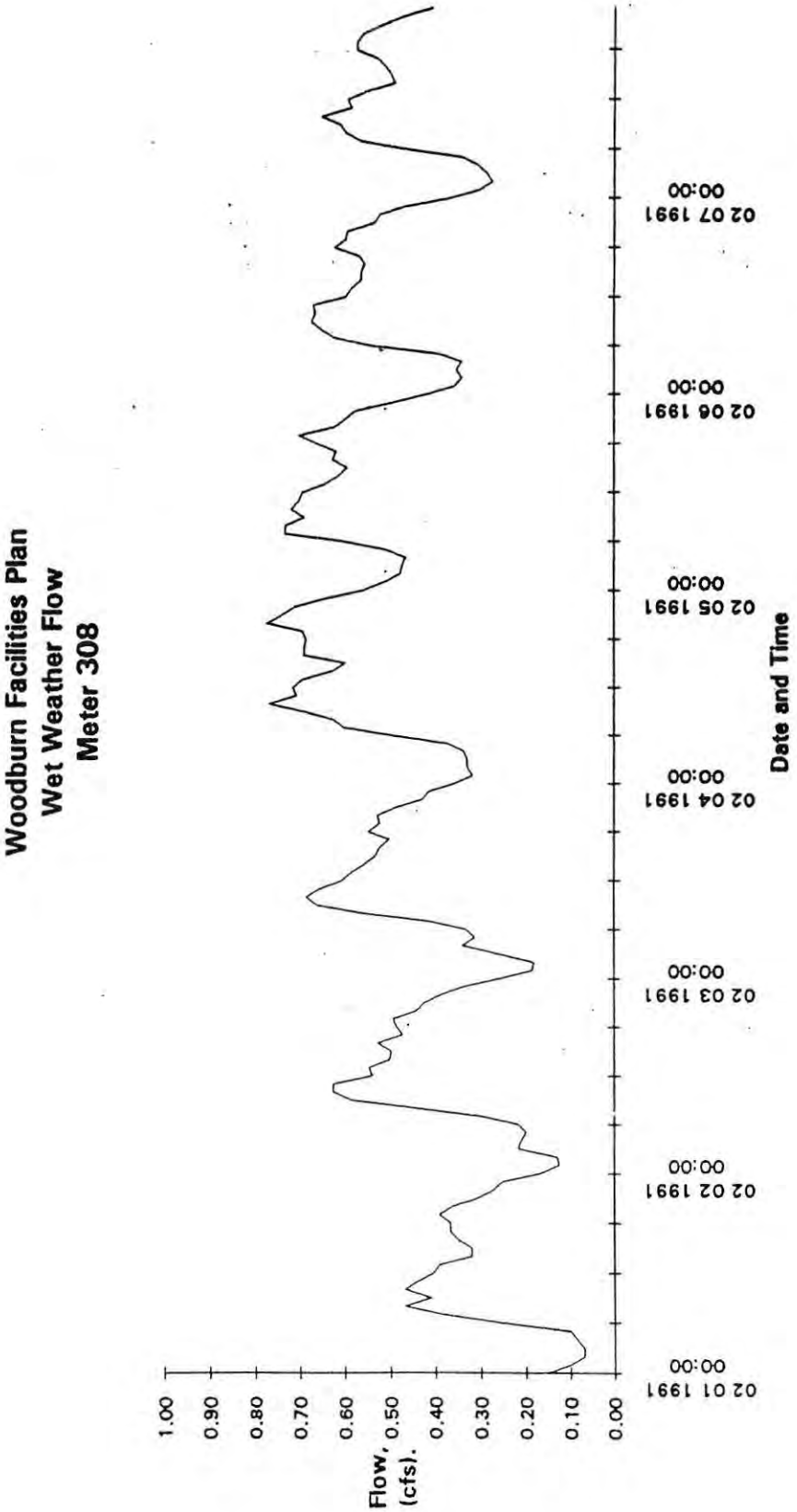


Woodburn Facilities Plan
Wet Weather Flow
Meter 308

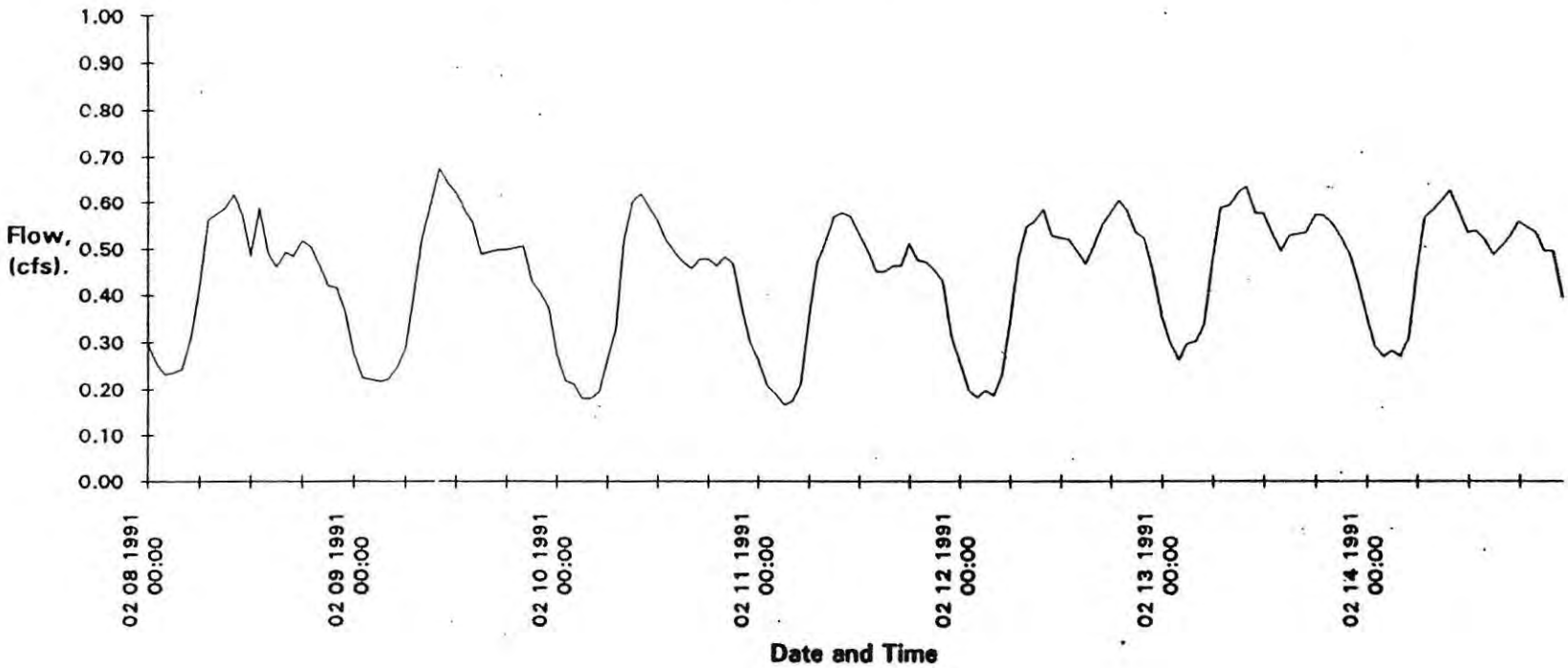


Woodburn Facilities Plan
Wet Weather Flow
Meter 308

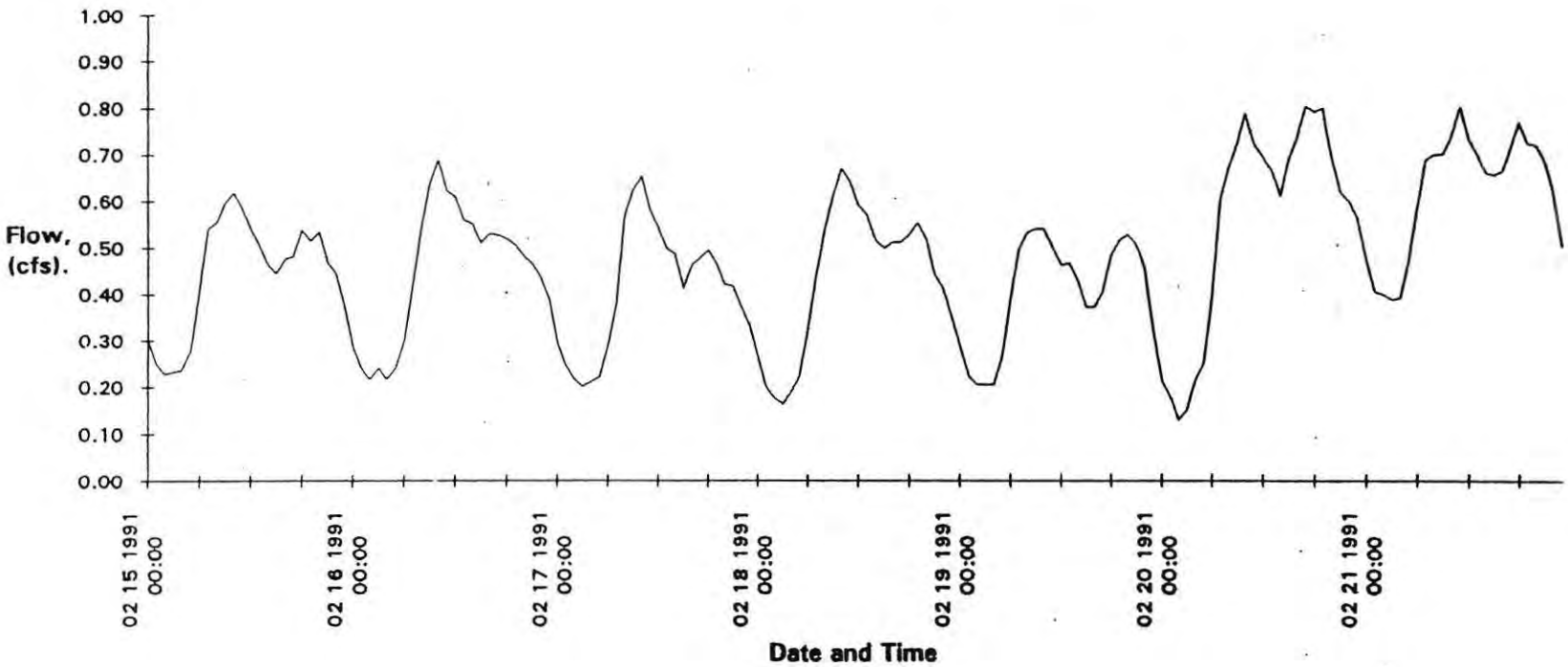



Woodburn Facilities Plan
Wet Weather Flow
Meter 308
Flow, (cfs).
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
02 01 1991 00:00
02 02 1991 00:00
02 03 1991 00:00
02 04 1991 00:00
02 05 1991 00:00
02 06 1991 00:00
02 07 1991 00:00
Date and Time

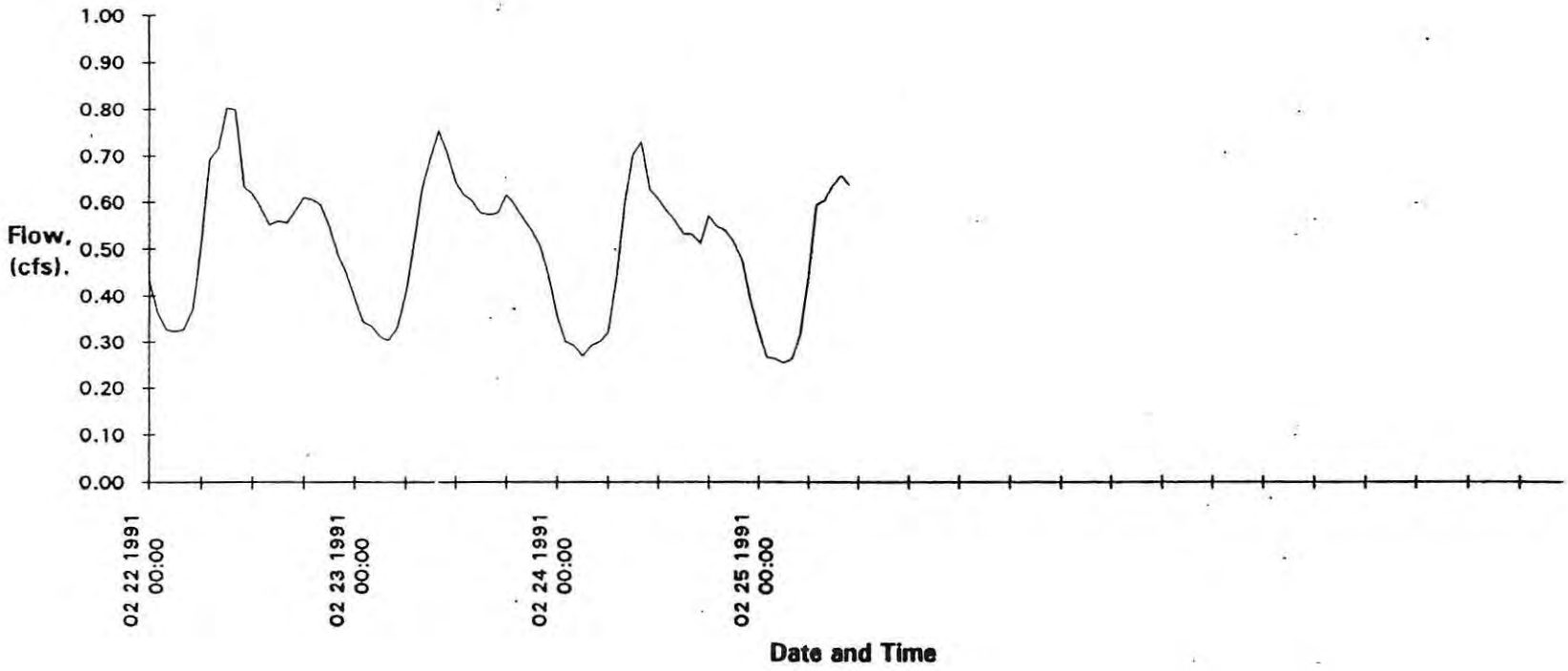
Woodburn Facilities Plan
Wet Weather Flow
Meter 308



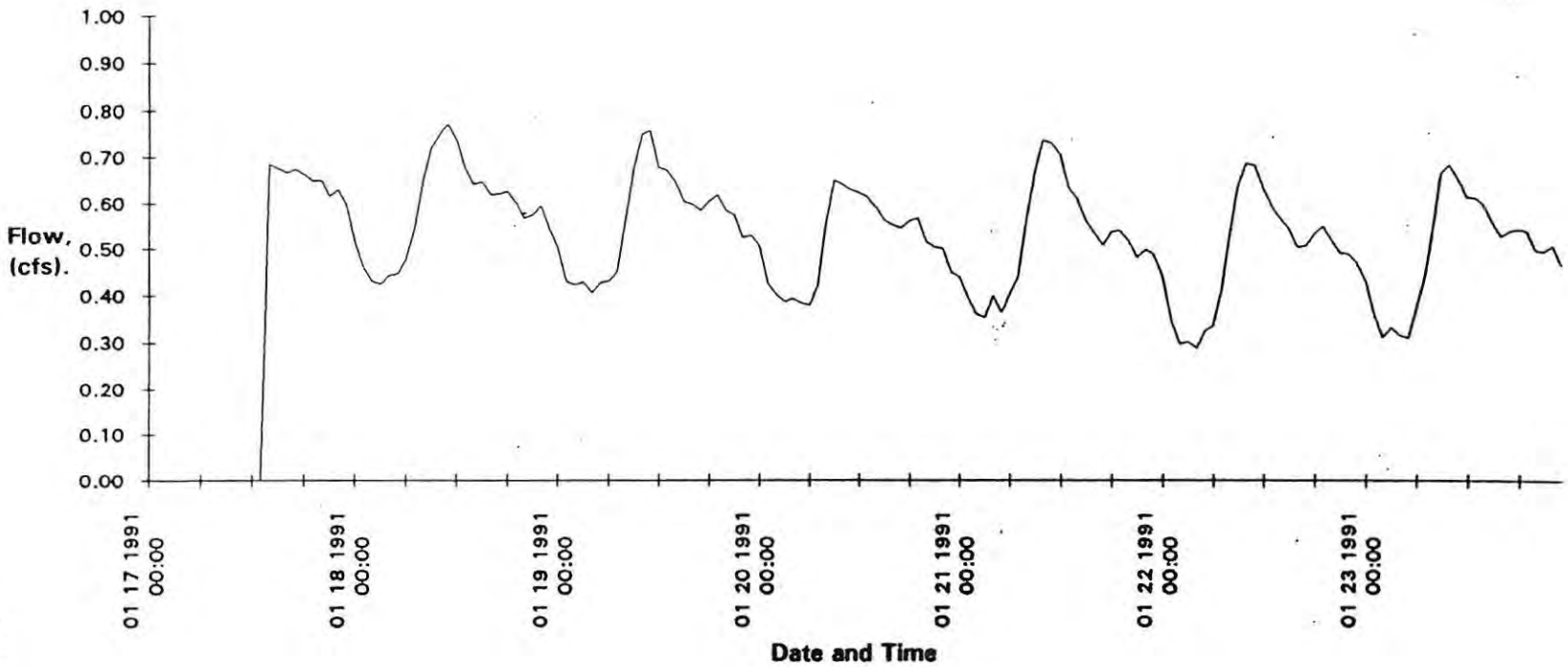
Woodburn Facilities Plan
Wet Weather Flow
Meter 308



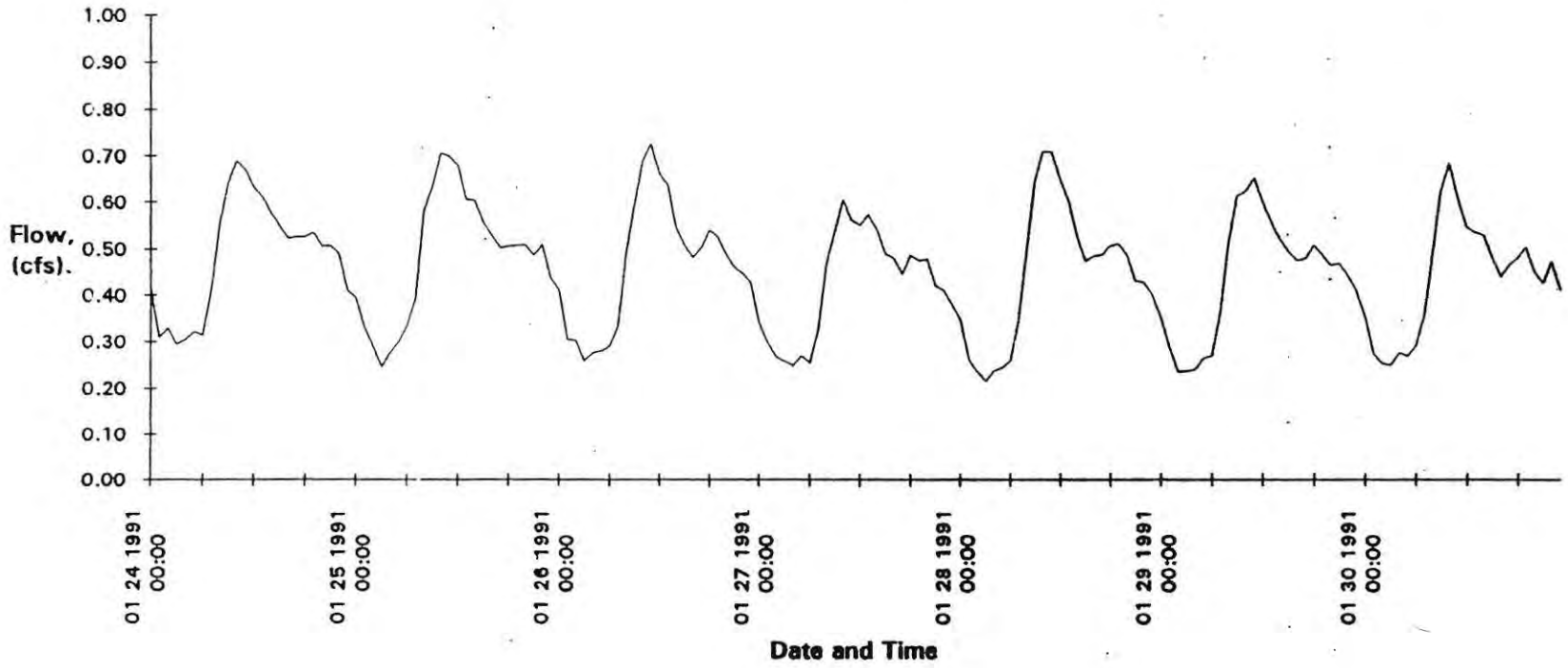
Woodburn Facilities Plan
Wet Weather Flow
Meter 308



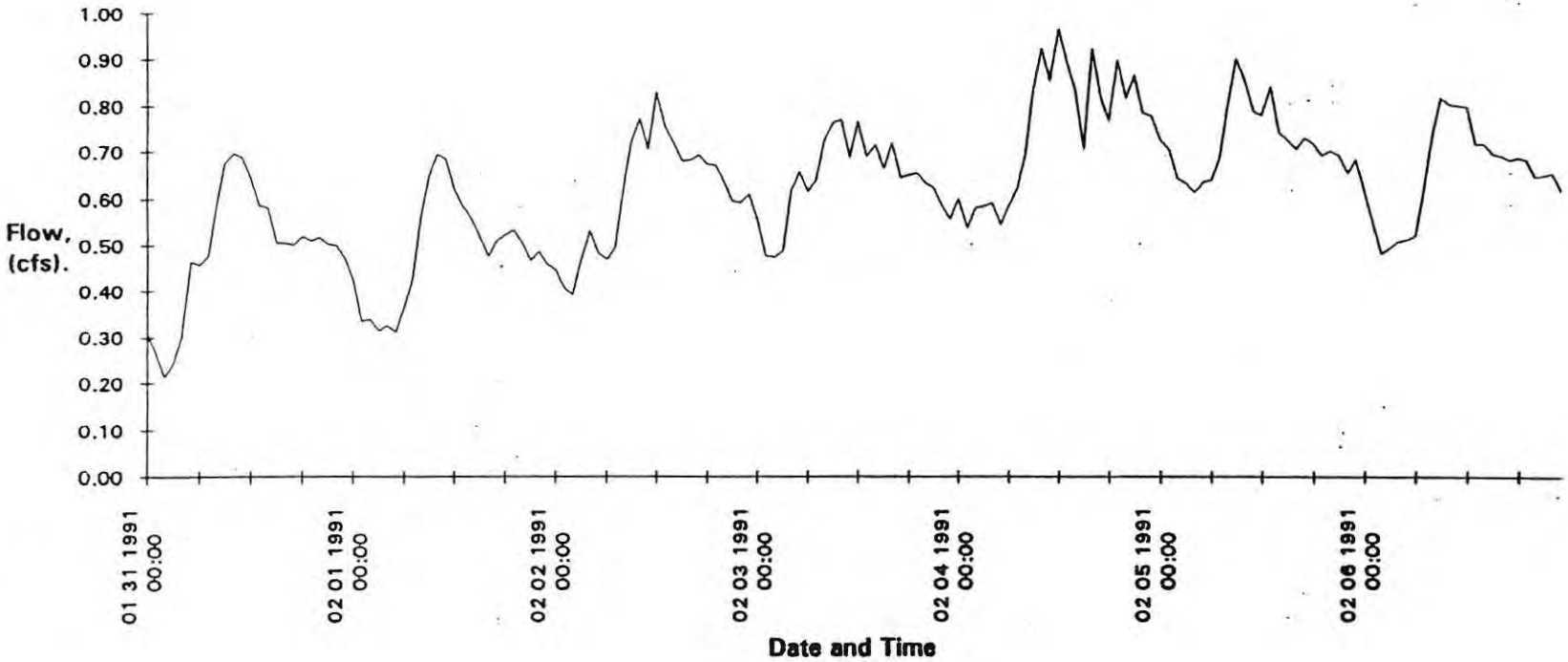
Woodburn Facilities Plan
Wet Weather Flow
Meter 119



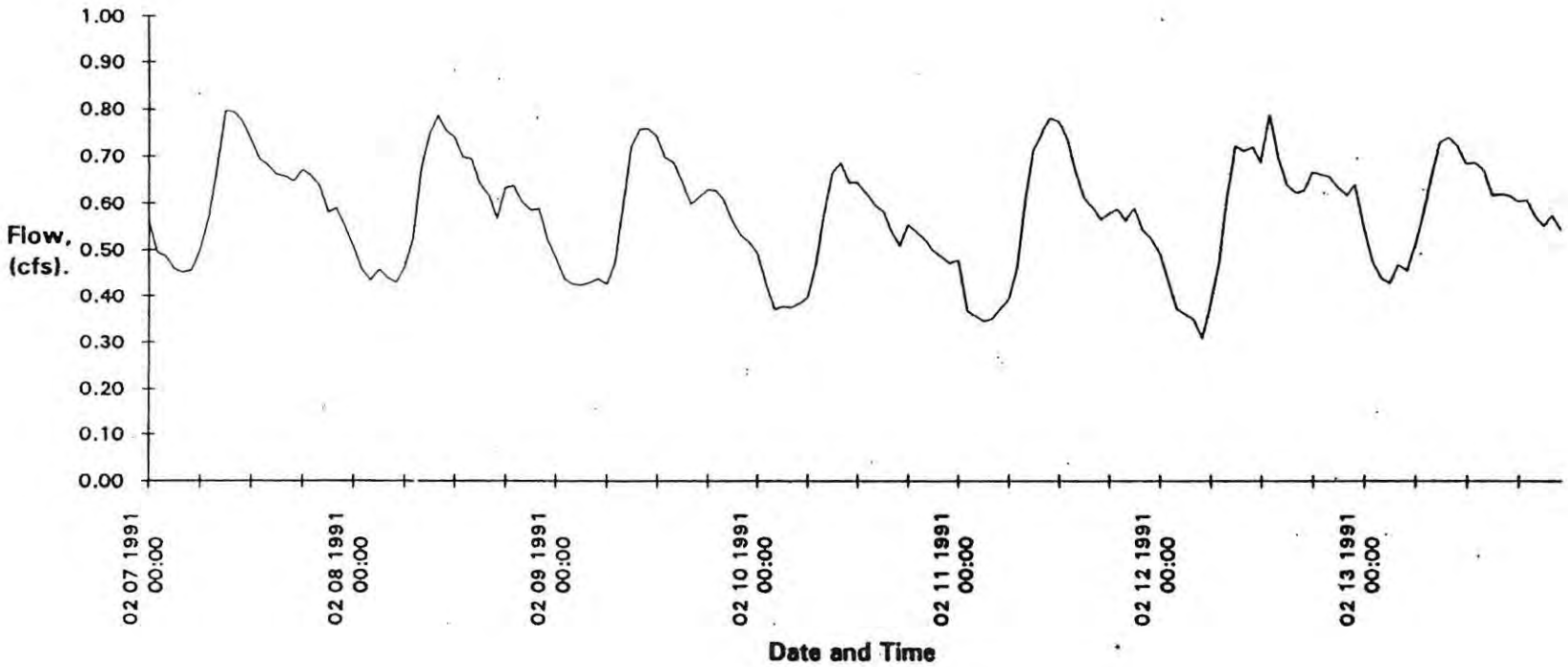
Woodburn Facilities Plan
Wet Weather Flow
Meter 119

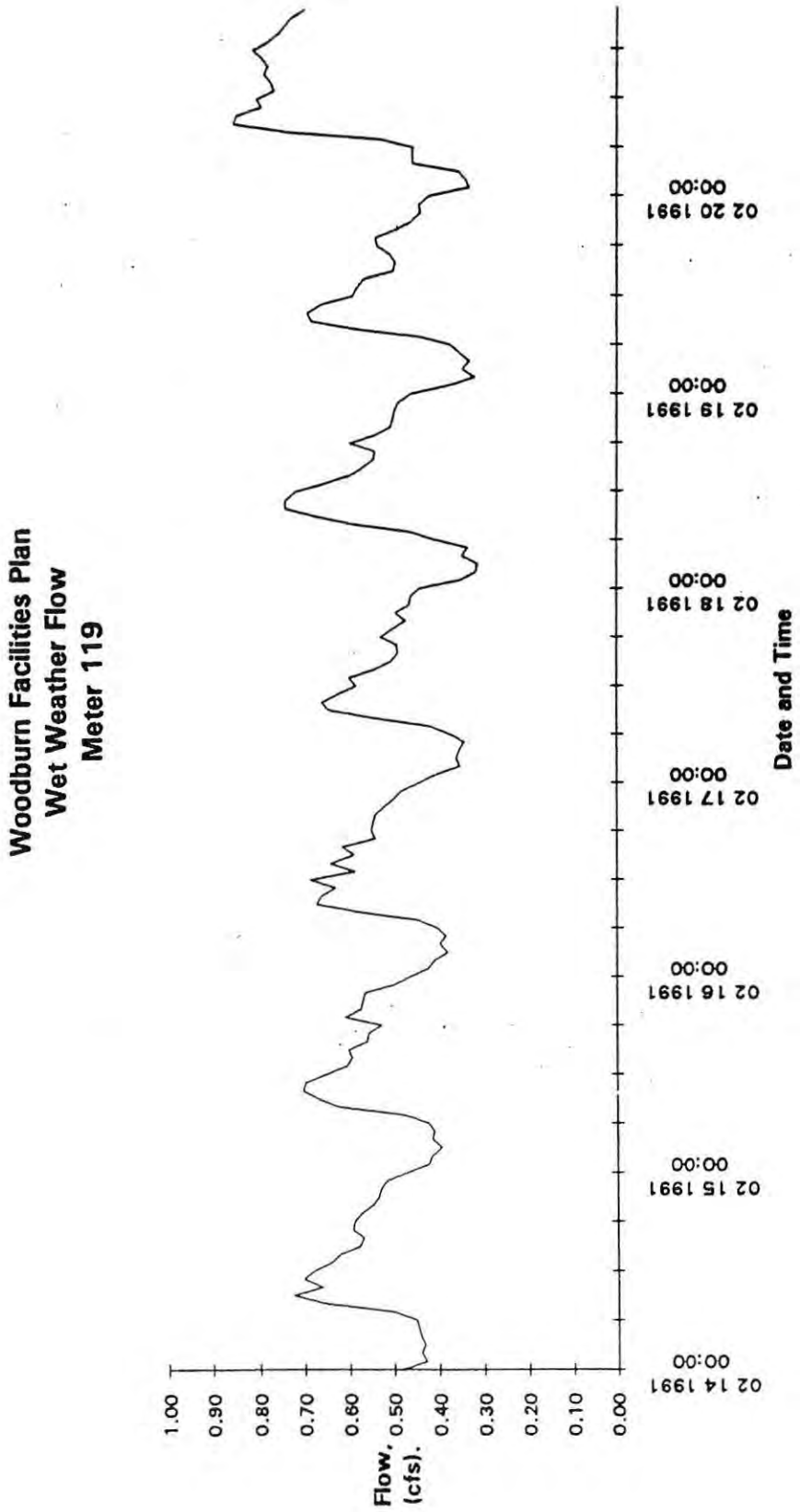


Woodburn Facilities Plan
Wet Weather Flow
Meter 119

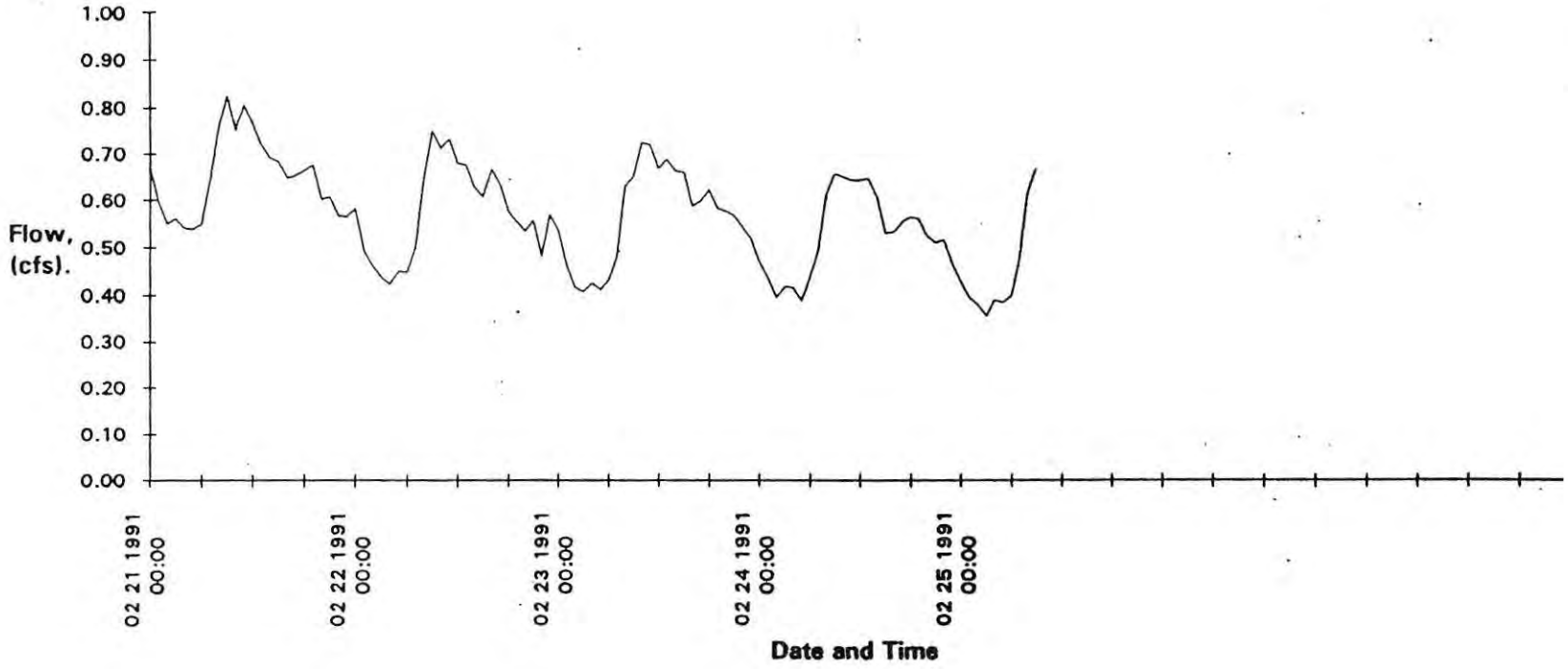


Woodburn Facilities Plan
Wet Weather Flow
Meter 119




Woodburn Facilities Plan
Wet Weather Flow
Meter 119
Flow, (cfs).
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
02 14 1991 00:00
02 15 1991 00:00
02 16 1991 00:00
02 17 1991 00:00
02 18 1991 00:00
02 19 1991 00:00
02 20 1991 00:00
Date and Time

Woodburn Facilities Plan
Wet Weather Flow
Meter 119



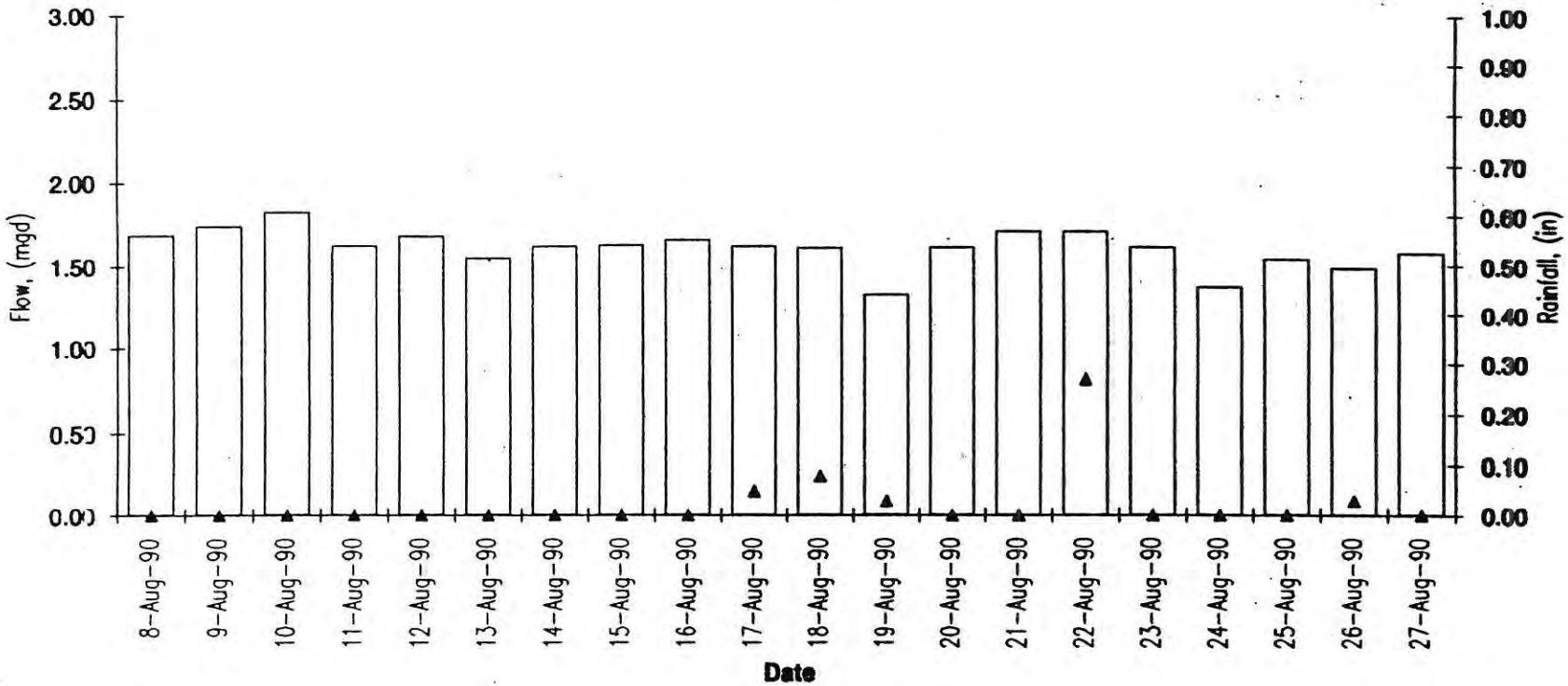
Appendix C

City of Woodburn - - Facilities Plan
Average Daily Flows

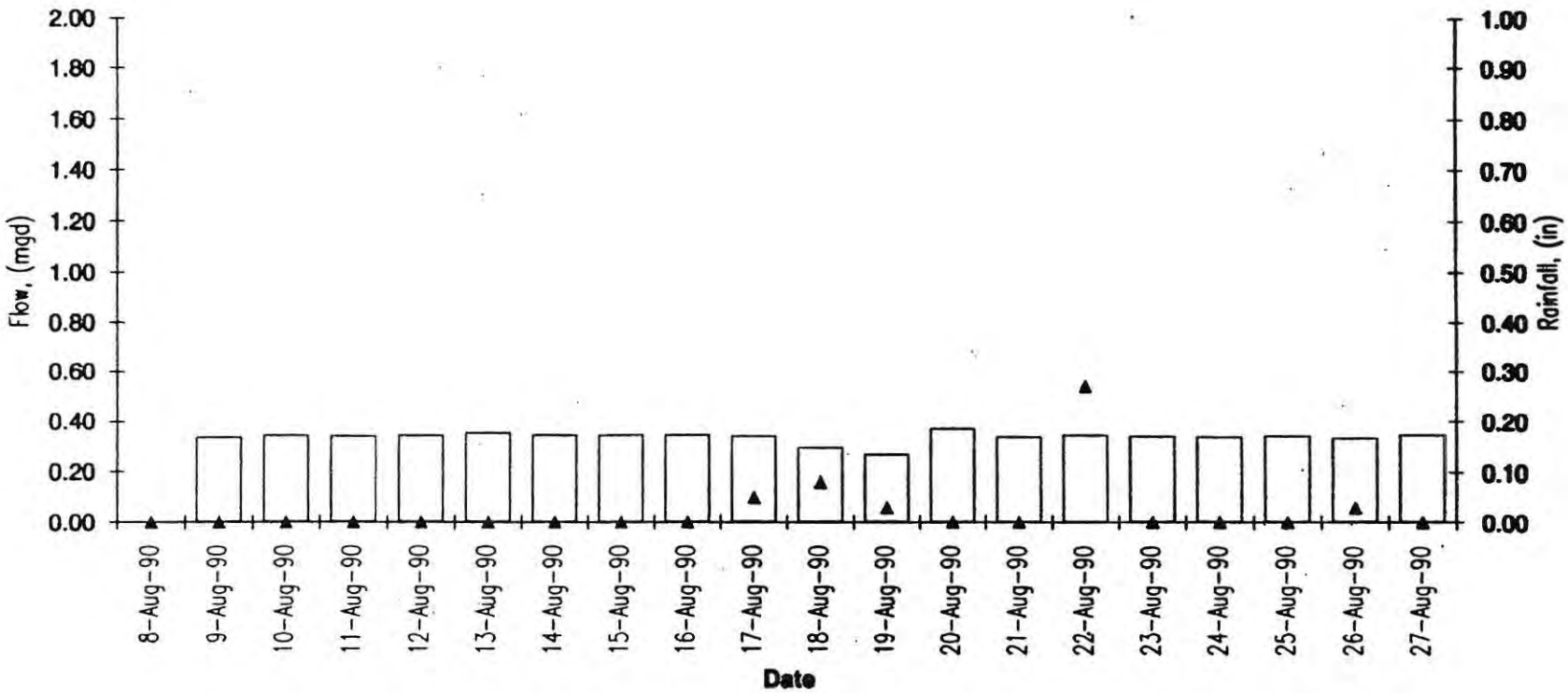
Dry Weather Average Daily Flows

Date	Rainfall (in)	Plant (mgd)	Meter 175 (mgd)	Meter 181 (mgd)	Meter 182 (mgd)	Meter 203 (mgd)	Meter 209 (mgd)
8-Aug-90	0.00	1.69	0.27	--	0.42	0.17	0.18
9-Aug-90	0.00	1.74	0.27	0.34	0.45	0.19	0.18
10-Aug-90	0.00	1.83	0.28	0.34	0.40	0.19	0.18
11-Aug-90	0.00	1.62	0.28	0.34	0.36	0.20	0.18
12-Aug-90	0.00	1.68	0.28	0.34	0.31	0.19	0.18
13-Aug-90	0.00	1.55	0.29	0.35	0.44	0.20	0.18
14-Aug-90	0.00	1.62	0.24	0.34	0.52	0.19	0.18
15-Aug-90	0.00	1.63	0.23	0.34	0.42	0.19	0.18
16-Aug-90	0.00	1.66	0.24	0.34	0.31	0.19	0.18
17-Aug-90	0.05	1.62	0.25	0.34	0.30	0.19	0.19
18-Aug-90	0.08	1.61	0.20	0.29	0.40	0.20	0.18
19-Aug-90	0.03	1.32	0.18	0.27	0.34	0.18	0.18
20-Aug-90	0.00	1.61	0.28	0.37	0.48	0.20	0.19
21-Aug-90	0.00	1.71	0.25	0.34	0.48	0.19	0.20
22-Aug-90	0.27	1.71	0.25	0.34	0.45	0.19	0.19
23-Aug-90	0.00	1.62	0.24	0.34	0.37	0.18	0.18
24-Aug-90	0.00	1.37	0.24	0.34	0.44	0.17	0.17
25-Aug-90	0.00	1.54	0.25	0.34	0.38	0.19	0.17
26-Aug-90	0.03	1.49	0.23	0.33	0.30	0.18	0.17
27-Aug-90	0.00	1.58	0.24	0.35	0.46	0.19	0.19
Average		1.61	0.25	0.34	0.40	0.19	0.18
Maximum		1.83	0.29	0.37	0.52	0.20	0.20
Minimum		1.32	0.18	0.27	0.30	0.17	0.17

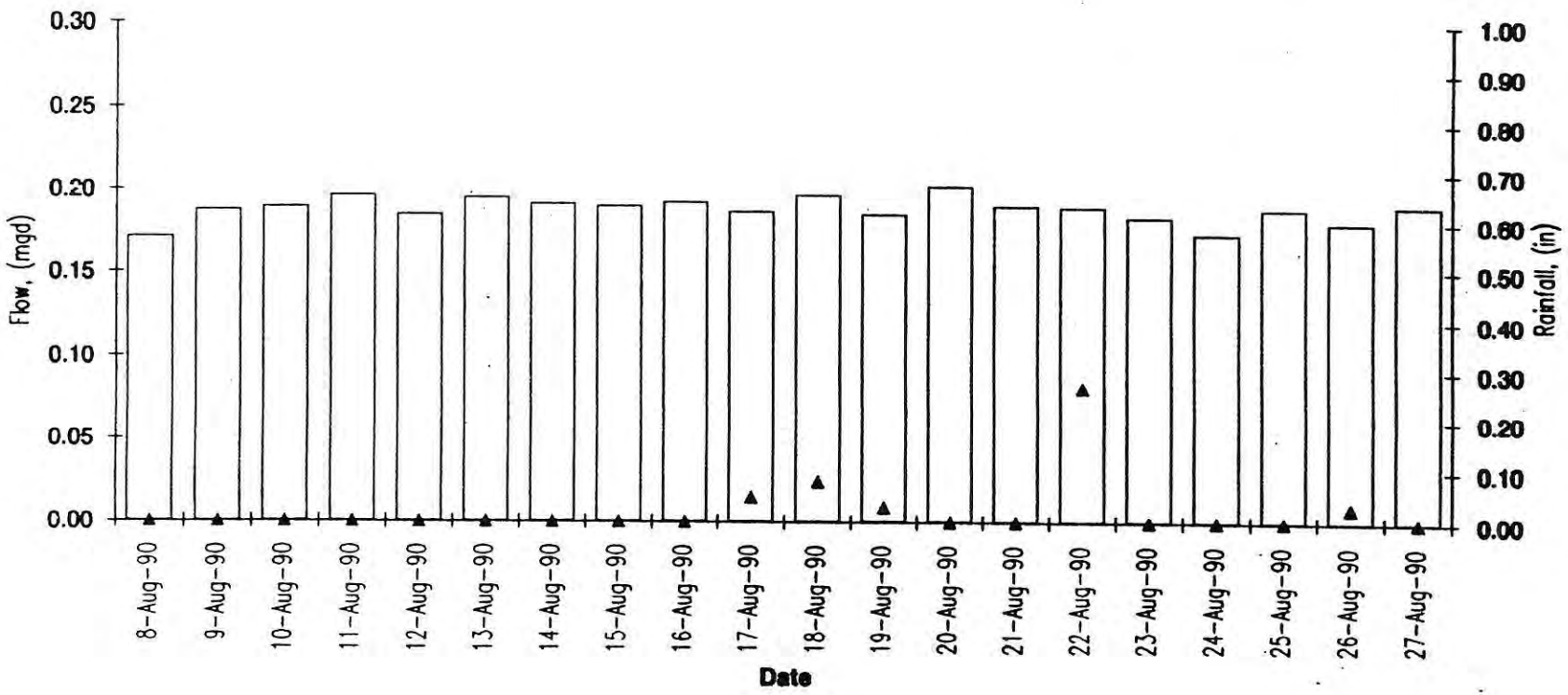
Woodburn Facilities Plan
Dry Weather
Average Daily Flow
Wastewater Treatment Plant



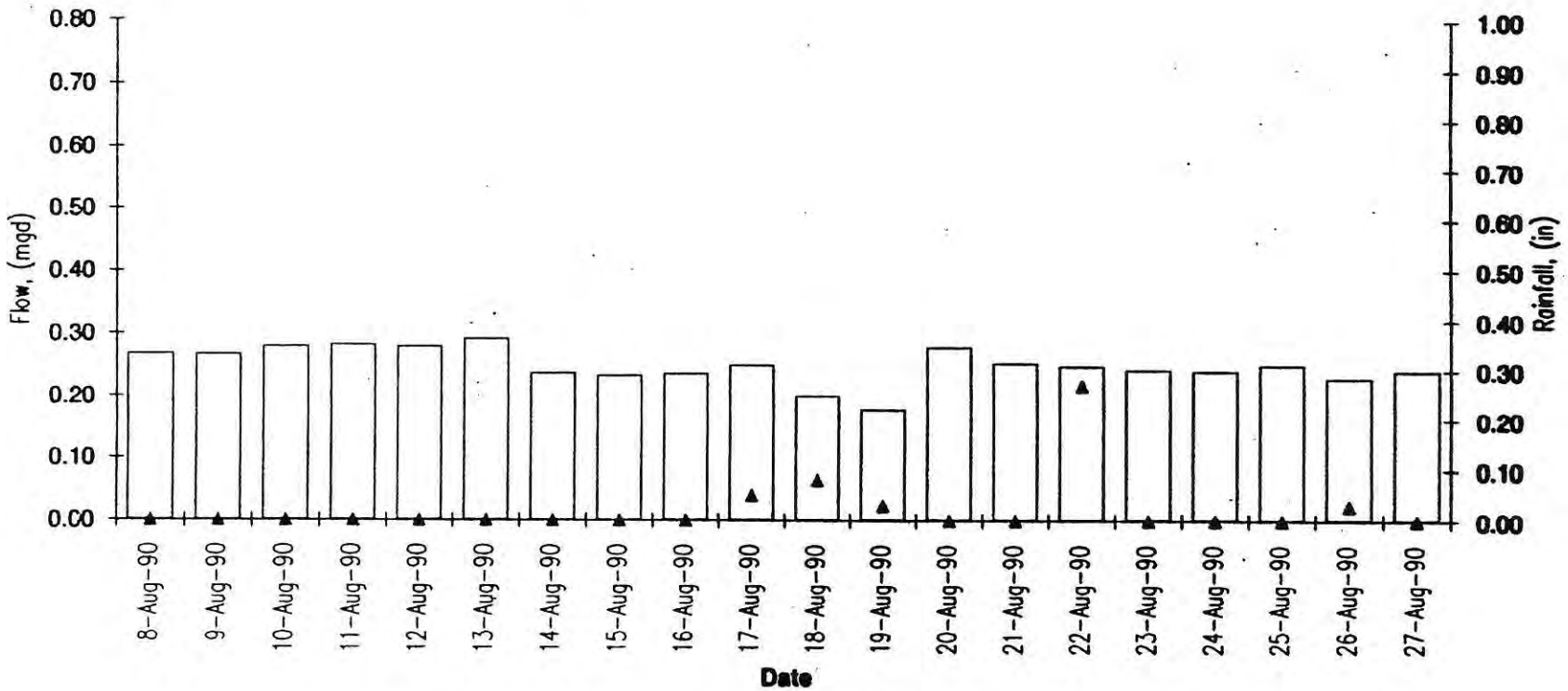
Woodburn Facilities Plan
Dry Weather
Average Daily Flow
Meter 182



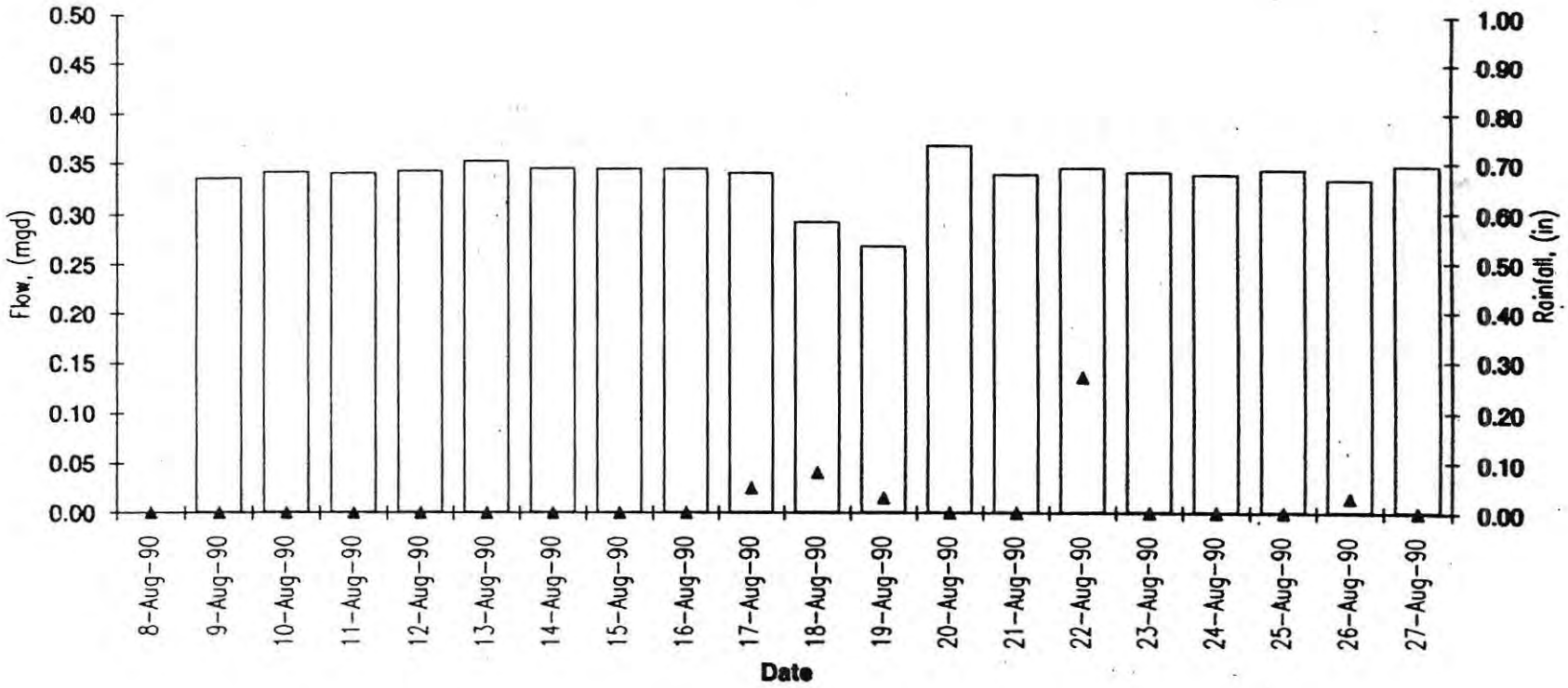
Woodburn Facilities Plan
Dry Weather
Average Daily Flow
Meter 203



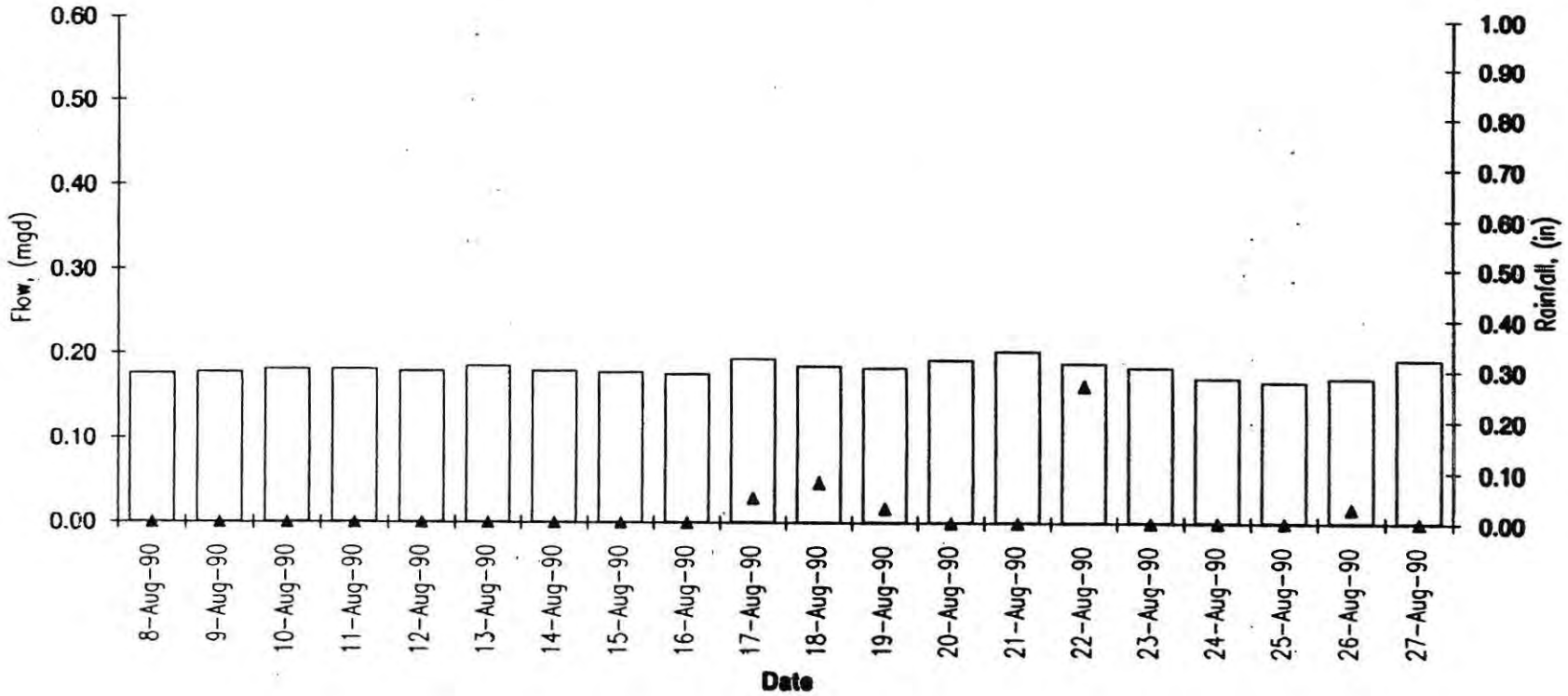
Woodburn Facilities Plan
Dry Weather
Average Daily Flow
Meter 175



Woodburn Facilities Plan
Dry Weather
Average Daily Flow
Meter 181



Woodburn Facilities Plan
Dry Weather
Average Daily Flow
Meter 209

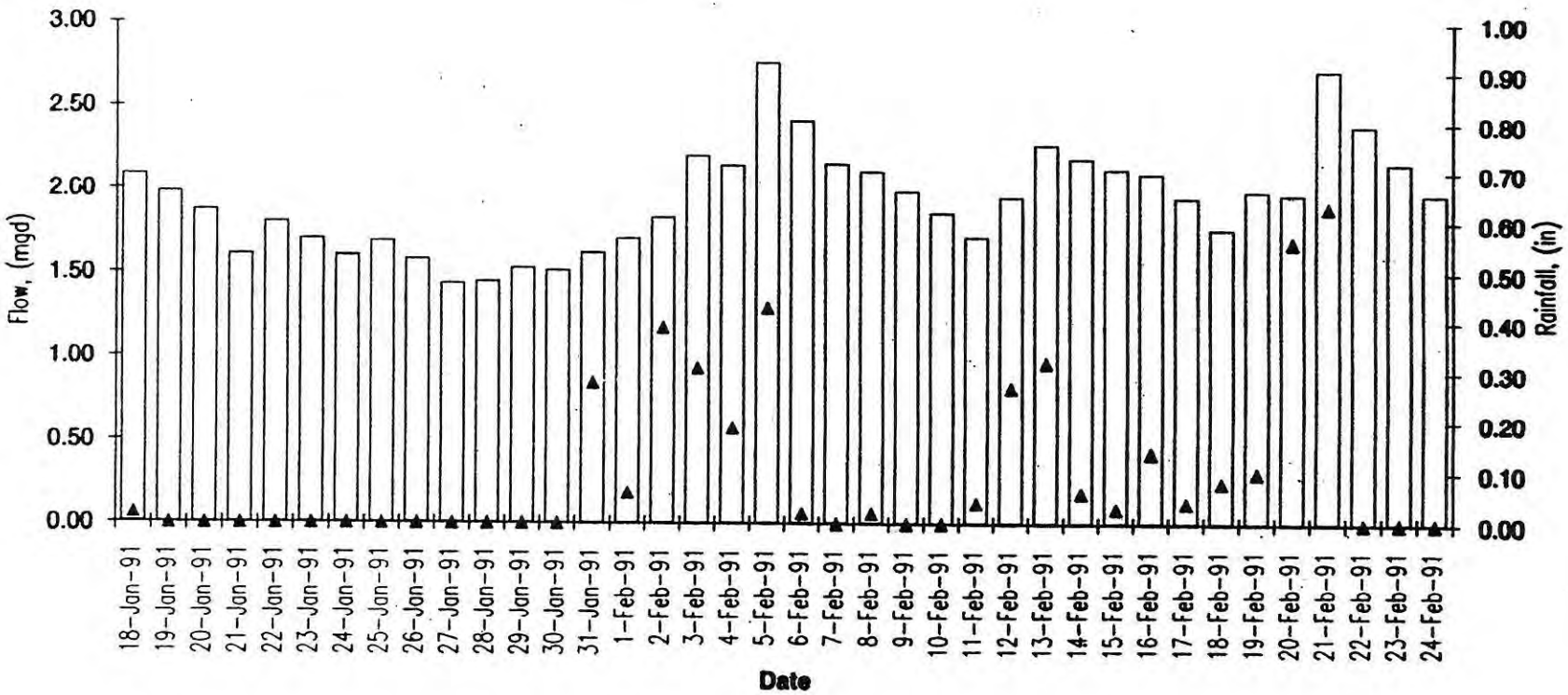


City of Woodburn - - Facilities Plan
Average Daily Flows

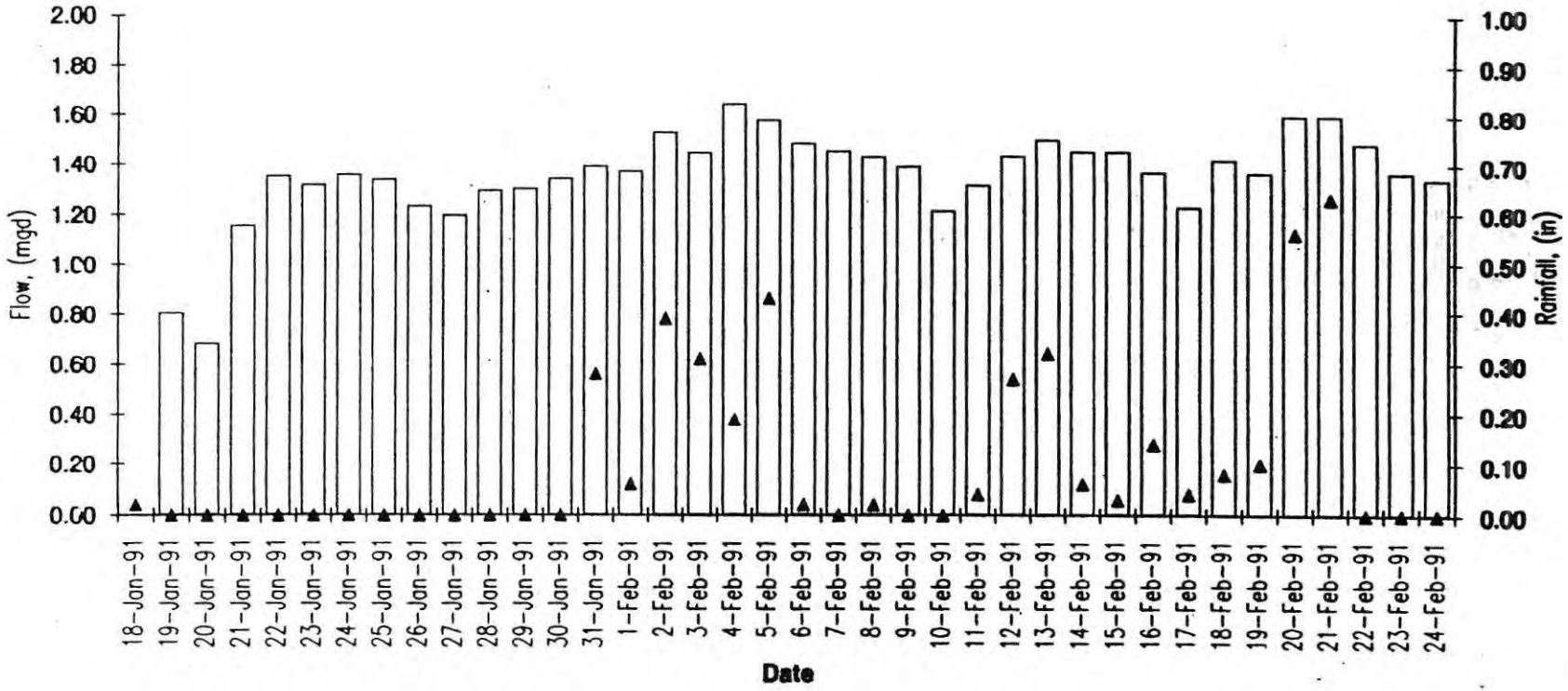
Wet Weather Average Daily Flows

Date	Rainfall (in)	Plant (mgd)	Meter 104 (mgd)	Meter 308 (mgd)	Meter 778 (mgd)	Meter 612 (mgd)	Meter 119 (mgd)
18-Jan-91	0.02	2.09	0.43	--	--	0.14	0.38
19-Jan-91	0.00	1.98	0.42	0.27	0.80	0.14	0.36
20-Jan-91	0.00	1.87	0.38	0.25	0.68	0.13	0.33
21-Jan-91	0.00	1.61	0.39	0.26	1.16	--	0.34
22-Jan-91	0.00	1.80	0.36	0.23	1.35	--	0.31
23-Jan-91	0.00	1.70	0.35	0.22	1.32	--	0.32
24-Jan-91	0.00	1.60	0.36	0.22	1.36	--	0.31
25-Jan-91	0.00	1.69	0.34	0.22	1.34	--	0.31
26-Jan-91	0.00	1.58	0.36	0.22	1.23	--	0.30
27-Jan-91	0.00	1.44	0.34	0.21	1.19	--	0.27
28-Jan-91	0.00	1.45	0.34	0.22	1.29	--	0.29
29-Jan-91	0.00	1.53	0.34	0.20	1.30	--	0.28
30-Jan-91	0.00	1.51	0.33	0.19	1.34	--	0.28
31-Jan-91	0.28	1.62	0.35	0.20	1.39	--	0.32
1-Feb-91	0.06	1.70	0.32	0.19	1.37	--	0.31
2-Feb-91	0.39	1.83	0.44	0.26	1.53	--	0.39
3-Feb-91	0.31	2.20	0.48	0.30	1.45	--	0.41
4-Feb-91	0.19	2.14	0.62	0.38	1.64	--	0.48
5-Feb-91	0.43	2.76	0.61	0.39	1.58	--	0.47
6-Feb-91	0.02	2.41	0.52	0.34	1.48	--	0.42
7-Feb-91	0.00	2.15	0.47	0.31	1.45	--	0.40
8-Feb-91	0.02	2.10	0.44	0.29	1.43	0.15	0.38
9-Feb-91	0.00	1.98	0.44	0.28	1.39	0.17	0.37
10-Feb-91	0.00	1.85	0.41	0.26	1.21	0.16	0.33
11-Feb-91	0.04	1.71	0.43	0.26	1.31	0.15	0.36
12-Feb-91	0.27	1.95	0.48	0.29	1.43	0.15	0.37
13-Feb-91	0.32	2.27	0.49	0.31	1.50	0.16	0.38
14-Feb-91	0.06	2.18	0.46	0.30	1.45	0.16	0.36
15-Feb-91	0.03	2.12	0.44	0.29	1.45	0.15	0.35
16-Feb-91	0.14	2.09	0.45	0.29	1.37	0.17	0.34
17-Feb-91	0.04	1.94	0.41	0.26	1.23	0.16	0.31
18-Feb-91	0.08	1.76	0.45	0.28	1.42	0.17	0.33
19-Feb-91	0.10	1.99	0.41	0.26	1.37	0.15	0.32
20-Feb-91	0.56	1.97	0.61	0.36	1.60	0.19	0.43
21-Feb-91	0.63	2.72	0.60	0.40	1.60	0.18	0.42
22-Feb-91	0.00	2.39	0.53	0.35	1.49	0.17	0.37
23-Feb-91	0.00	2.16	0.47	0.34	1.37	0.18	0.36
24-Feb-91	0.00	1.96	0.36	0.31	1.34	0.17	0.34
Average		1.94	0.43	0.28	1.36	0.16	0.35
Maximum		2.76	0.62	0.40	1.64	0.19	0.48
Minimum		1.44	0.32	0.19	0.68	0.13	0.27

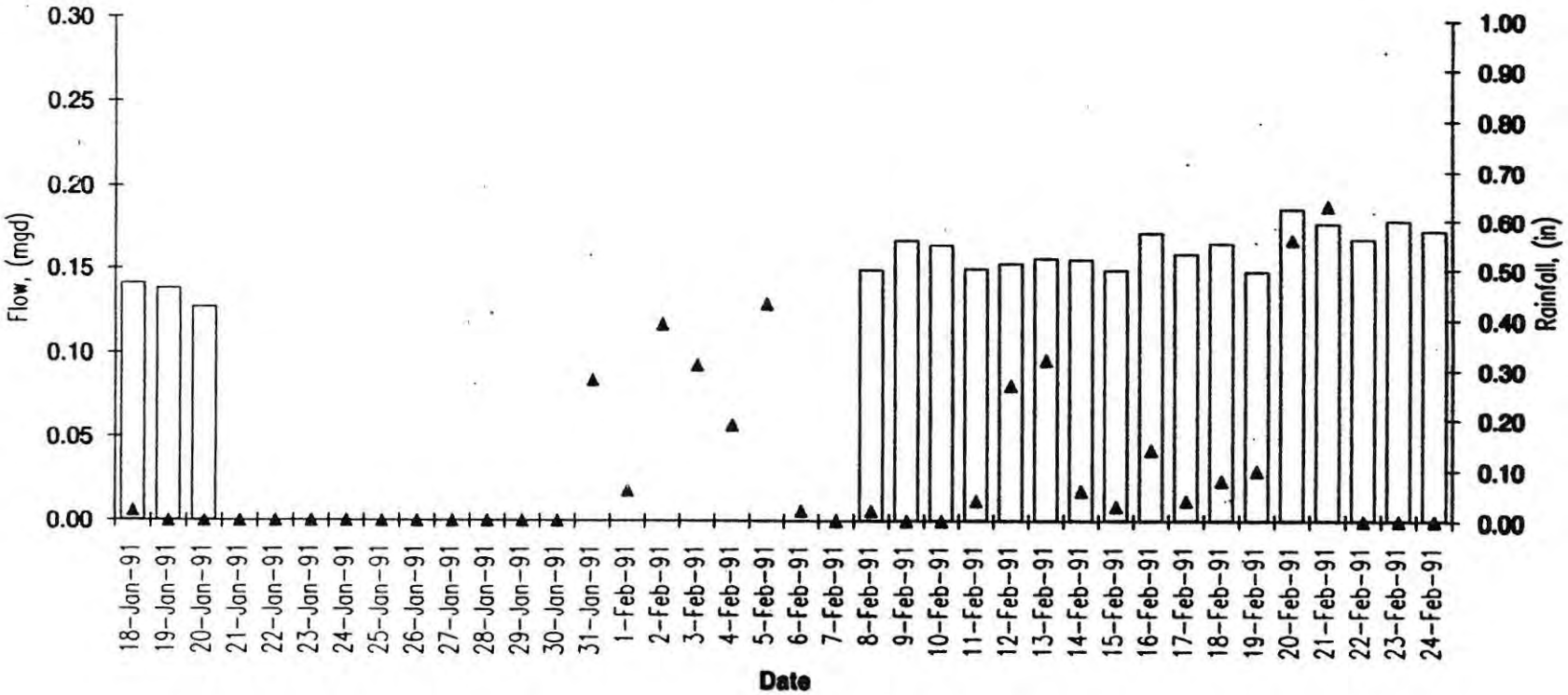
Woodburn Facilities Plan
Wet Weather
Average Daily Flow
Wastewater Treatment Plant



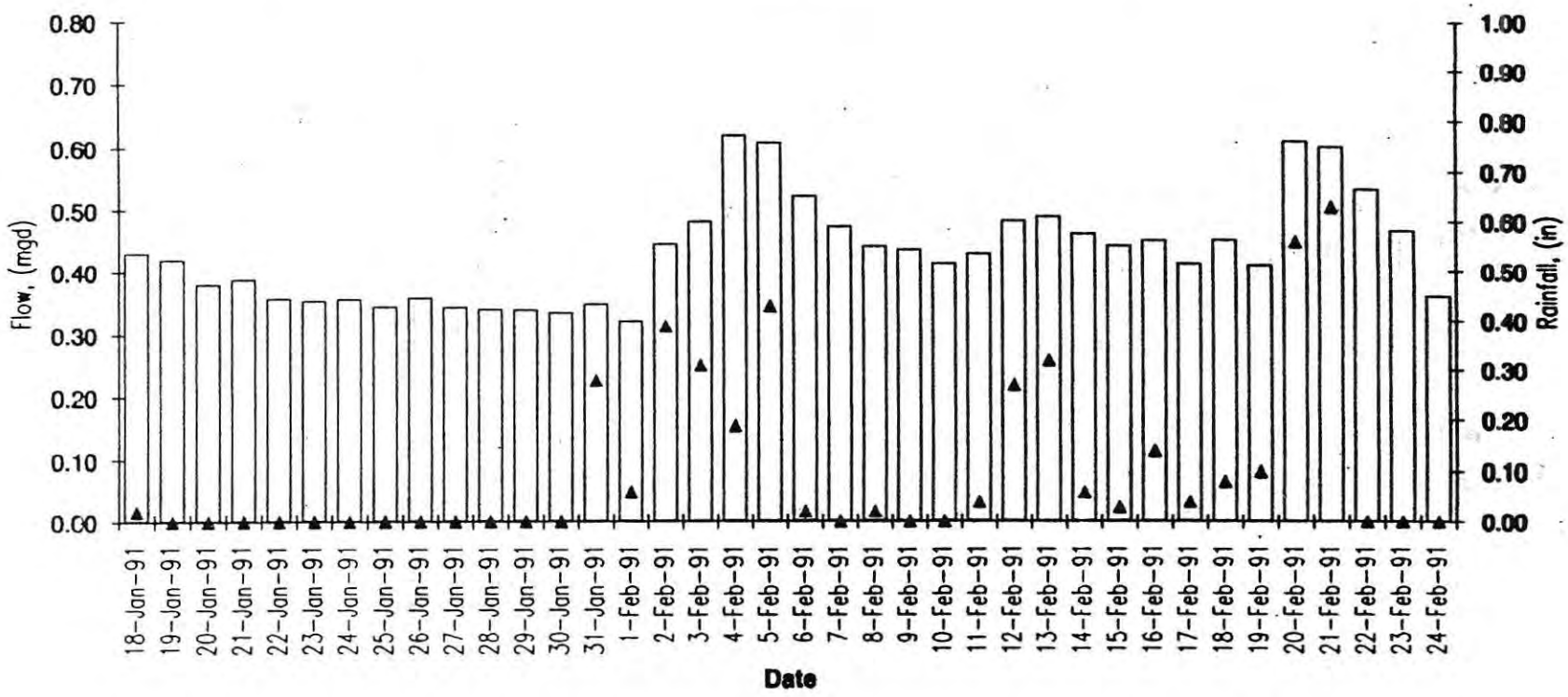
Woodburn Facilities Plan
Wet Weather
Average Daily Flow
Meter 778



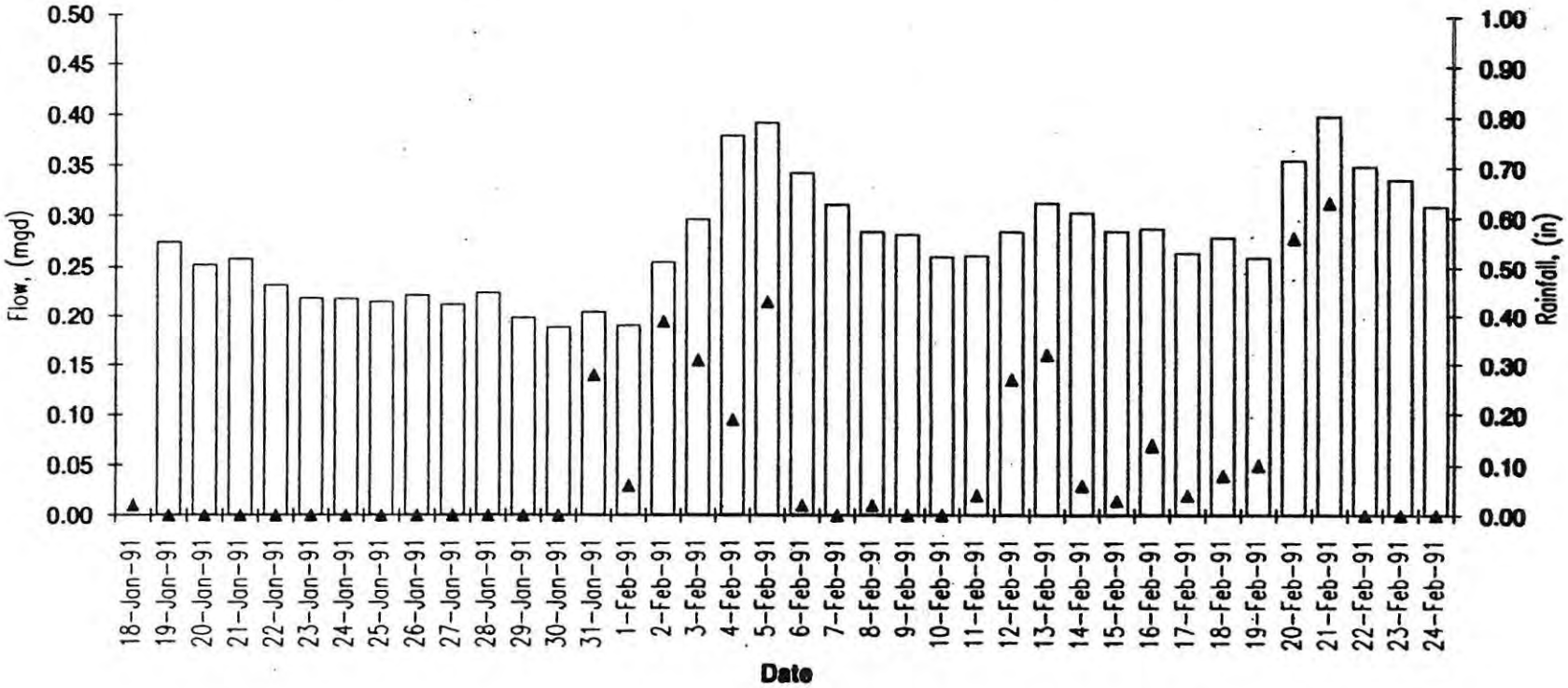
Woodburn Facilities Plan
Wet Weather
Average Daily Flow
Meter 612



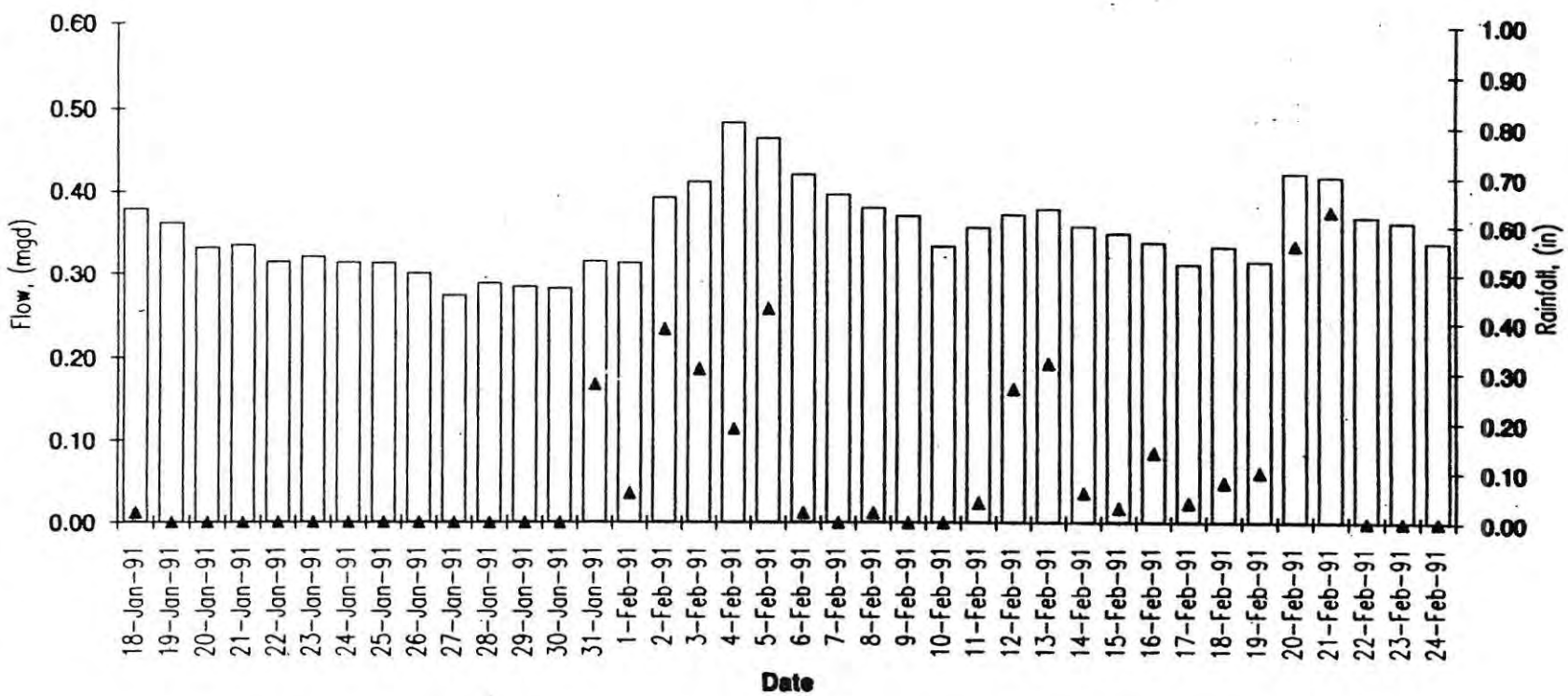
Woodburn Facilities Plan
Wet Weather
Average Daily Flow
Meter 104



Woodburn Facilities Plan
Wet Weather
Average Daily Flow
Meter 308



Woodburn Facilities Plan
Wet Weather
Average Daily Flow
Meter 119



Appendix D

City of Woodburn - - Facilities Plan
Average Daily Flows

Wet Weather Average Daily Flows for Metered Areas

Date	Rainfall (in)	Area A (mgd)	Area B (mgd)	Area C (mgd)	Area D (mgd)	Area E (mgd)	Area F (mgd)
18-Jan-91	0.02	--	0.38	--	0.14	--	--
19-Jan-91	0.00	0.27	0.36	0.15	0.14	0.25	0.82
20-Jan-91	0.00	0.25	0.33	0.13	0.13	0.18	0.86
21-Jan-91	0.00	0.26	0.34	0.13	--	--	0.12
22-Jan-91	0.00	0.23	0.31	0.12	--	--	0.14
23-Jan-91	0.00	0.22	0.32	0.13	--	--	0.06
24-Jan-91	0.00	0.22	0.31	0.14	--	--	0.00
25-Jan-91	0.00	0.22	0.31	0.13	--	--	0.04
26-Jan-91	0.00	0.22	0.30	0.14	--	--	0.05
27-Jan-91	0.00	0.21	0.27	0.13	--	--	0.00
28-Jan-91	0.00	0.22	0.29	0.12	--	--	0.00
29-Jan-91	0.00	0.20	0.28	0.14	--	--	0.00
30-Jan-91	0.00	0.19	0.28	0.14	--	--	0.00
31-Jan-91	0.28	0.20	0.32	0.14	--	--	0.00
1-Feb-91	0.06	0.19	0.31	0.13	--	--	0.02
2-Feb-91	0.39	0.26	0.39	0.19	--	--	0.00
3-Feb-91	0.31	0.30	0.41	0.18	--	--	0.34
4-Feb-91	0.19	0.38	0.48	0.24	--	--	0.01
5-Feb-91	0.43	0.39	0.47	0.21	--	--	0.71
6-Feb-91	0.02	0.34	0.42	0.18	--	--	0.51
7-Feb-91	0.00	0.31	0.40	0.16	--	--	0.30
8-Feb-91	0.02	0.29	0.38	0.16	0.15	0.84	0.29
9-Feb-91	0.00	0.28	0.37	0.15	0.17	0.79	0.22
10-Feb-91	0.00	0.26	0.33	0.15	0.16	0.63	0.31
11-Feb-91	0.04	0.26	0.36	0.17	0.15	0.74	0.04
12-Feb-91	0.27	0.29	0.37	0.20	0.15	0.80	0.14
Feb-91	0.32	0.31	0.38	0.18	0.16	0.85	0.39
14-Feb-91	0.06	0.30	0.36	0.16	0.16	0.84	0.37
15-Feb-91	0.03	0.29	0.35	0.16	0.15	0.86	0.32
16-Feb-91	0.14	0.29	0.34	0.16	0.17	0.75	0.38
17-Feb-91	0.04	0.26	0.31	0.15	0.16	0.65	0.40
18-Feb-91	0.08	0.28	0.33	0.17	0.17	0.80	0.00
19-Feb-91	0.10	0.26	0.32	0.15	0.15	0.81	0.30
20-Feb-91	0.56	0.36	0.43	0.25	0.19	0.81	0.00
21-Feb-91	0.63	0.40	0.42	0.20	0.18	0.82	0.69
22-Feb-91	0.00	0.35	0.37	0.18	0.17	0.79	0.53
23-Feb-91	0.00	0.34	0.36	0.13	0.18	0.72	0.43
24-Feb-91	0.00	0.31	0.34	0.05	0.17	0.81	0.28
Average		0.28	0.35	0.16	0.16	0.72	0.25
Maximum		0.40	0.48	0.25	0.19	0.86	0.86
Minimum		0.19	0.27	0.05	0.13	0.18	0.00

CH2M HILL

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DATE: June 2, 1993

SUBJECT: Collection System Evaluation

PROJECT: PDX27874.F4

As part of the Woodburn Wastewater Facilities Plan, CH2M HILL evaluated the City's sanitary sewerage collection system by conducting a system inventory and a hydraulic capacity analysis. This technical memorandum presents the goals and objectives of the evaluation and discusses the components and results of the collection system inventory and the hydraulic analysis.

Goals and Objectives

The collection system evaluation was conducted to help City staff allocate maintenance resources and plan for capital improvements. The purpose of the collection system inventory was to obtain information that might indicate the extent and possibly the condition of facilities requiring maintenance. The hydraulic analysis was intended to identify portions of the collection system that might not have the capacity to convey the increased flow that will result from future development within the Woodburn sanitary sewerage service area.

The evaluation focuses on the main collectors of the system, trunk and interceptor lines and pump stations. Local collection laterals were not included in the evaluation because they generally serve smaller areas, rarely experience hydraulic overloading, and are not as critical to system capacity as trunk and interceptor lines. Improvements to main collectors and pump stations also require a higher capital investment and, therefore, a higher level of accuracy in fiscal planning.

The hydraulic analysis of the collection system compares existing pipeline and pump station capacities to future sewerage flow estimates to identify parts of the system that may require improvement. Peak system flows were estimated for current (1992), 5-year

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(1997), and buildout development conditions. Facilities lacking adequate capacity to convey current or 5-year peak flows warrant immediate attention, while facilities inadequate for peak flow at buildout must be improved before that development occurs.

Collection System Inventory

The collection system inventory includes a description of the sanitary sewerage service area and drainage basins, as well as an accounting of the trunk and interceptor lines, pump stations, diversions, and overflows that constitute the collection system.

Service Area

The Woodburn sanitary sewerage collection system currently serves 2,087 acres, including most of the area within the City limits. The system will eventually be expanded to serve 4,913 acres, which will include all land within the City's urban growth boundary (UGB) and 1,494 acres outside the UGB. The latter area consists of two large parcels: 312 acres bordering the northeast edge of the UGB and 1,182 acres located along the western edge of the UGB. Figure 1 shows the area considered for service under the collection system evaluation.

Drainage Basins

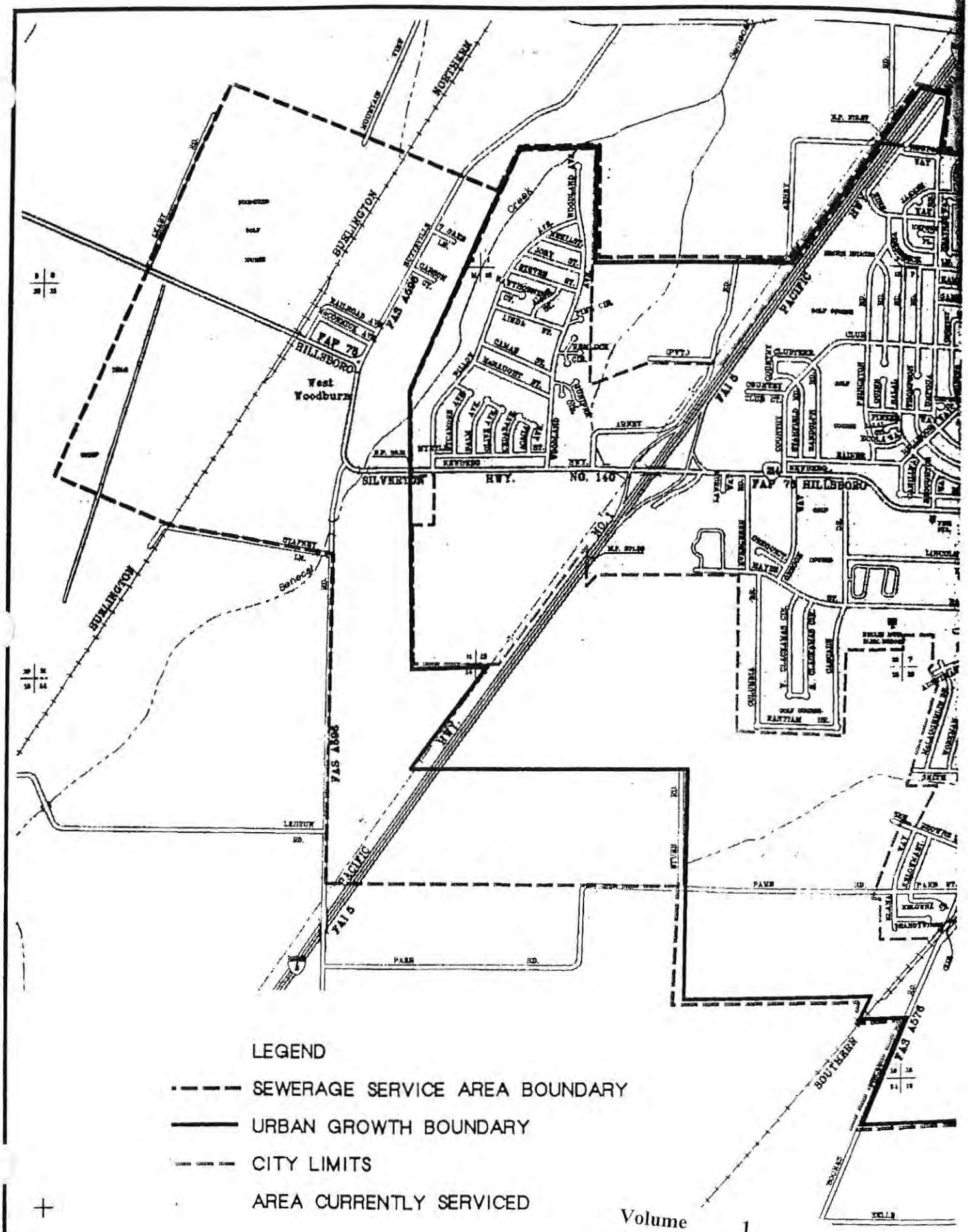
Woodburn has about 40 feet of topographic relief, with Mill Creek and Senecal Creek forming the major drainageways. Mill Creek drains to the northeast through the center of the city, and Senecal Creek drains to the northeast along the western edge of the city. Interstate 5 is the approximate dividing line between the two major drainage areas.

The Woodburn sanitary sewerage service area can be divided into ten distinct, naturally defined drainage basins. Nine of the basins are within the Mill Creek drainage area and have been designated as MC-1 through MC-9. The remaining drainage basin is in the Senecal Creek drainage area and is designated as SC-1. Figure 1 shows the location of each drainage basin, while the total area and estimated area with existing sanitary sewer service are listed for each basin in Table 1.

Trunk lines serving drainage basins MC-1 through MC-6 contribute flow to the Mill Creek interceptor, which runs along Mill Creek and terminates at the Mill Creek pump station. Trunk lines serving drainage basins MC-7 and MC-8 contribute flow directly to the Mill Creek pump station. Flow from drainage basin MC-9 is pumped by a smaller pump station located on Greenville Drive into the Mill Creek pump station force main. Flow from drainage basin SC-1 is pumped across Interstate 5 and into drainage basin MC-3.

Figure 1

Figure 1
Sewerage Service Area



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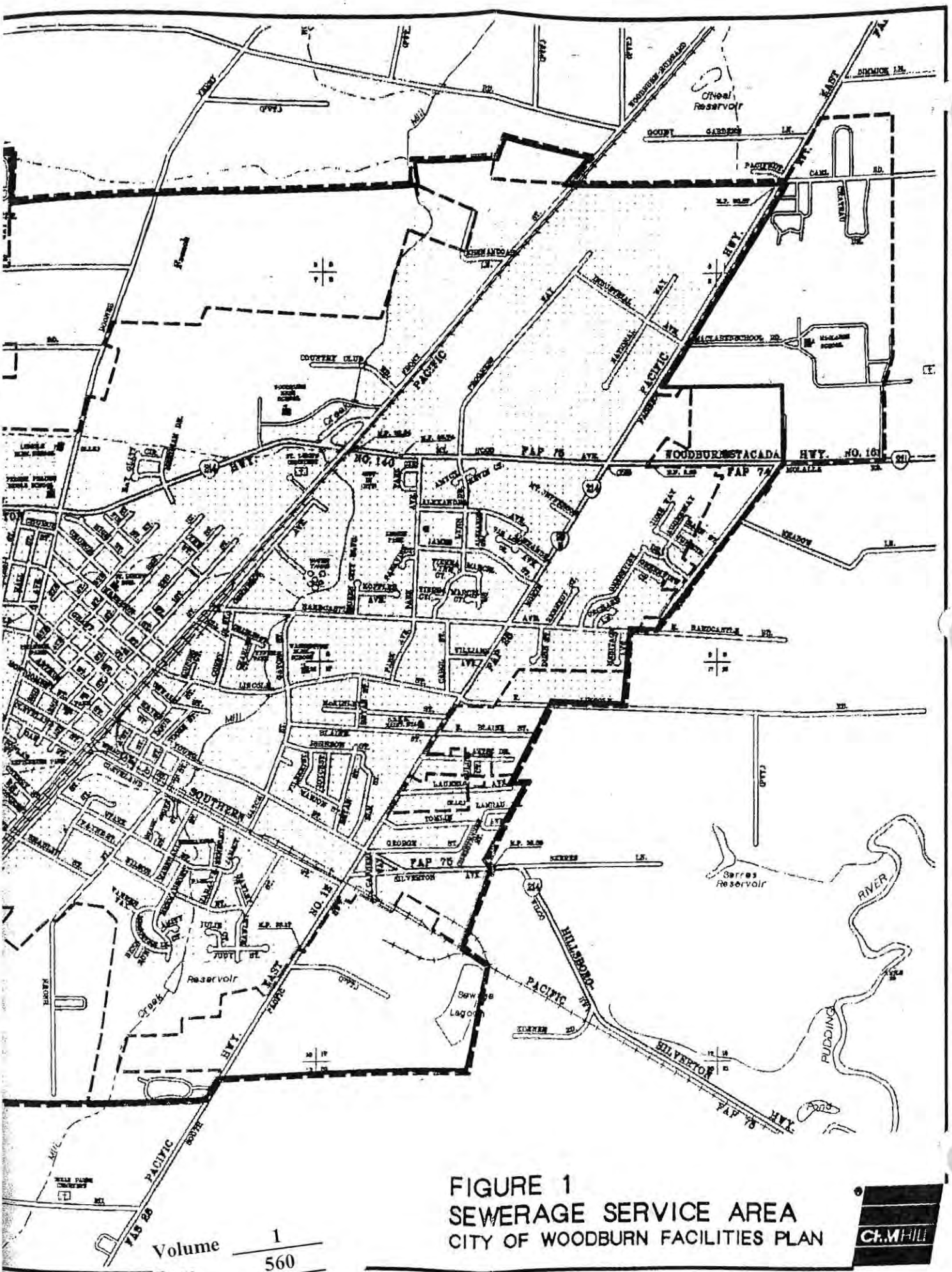


FIGURE 1
SEWERAGE SERVICE AREA
CITY OF WOODBURN FACILITIES PLAN

CH.M HILL

Table 1 Drainage Basins in the Sanitary Sewerage Service Area		
Basin Identification	**Basin Area (acres)**	**Area Served by Sewerage System (acres)**
MC-1	539	171
MC-2	325	293
MC-3	662	422
MC-4	232	196
MC-5	198	161
MC-6	602	161
MC-7	512	351
MC-8	389	80
MC-9	378	80
SC-1	1,095	172
Total	4,913	2,087

Trunk and Interceptor Lines

The trunk and interceptor system was inventoried by CH2M HILL and City staff. CH2M HILL gathered data on the length, diameter, construction material, and year of construction for the pipelines. City staff provided information on trunk and interceptor manhole invert and rim elevations. The trunk and interceptor system is shown in Figure 2. System inventory data are included in Appendix A.

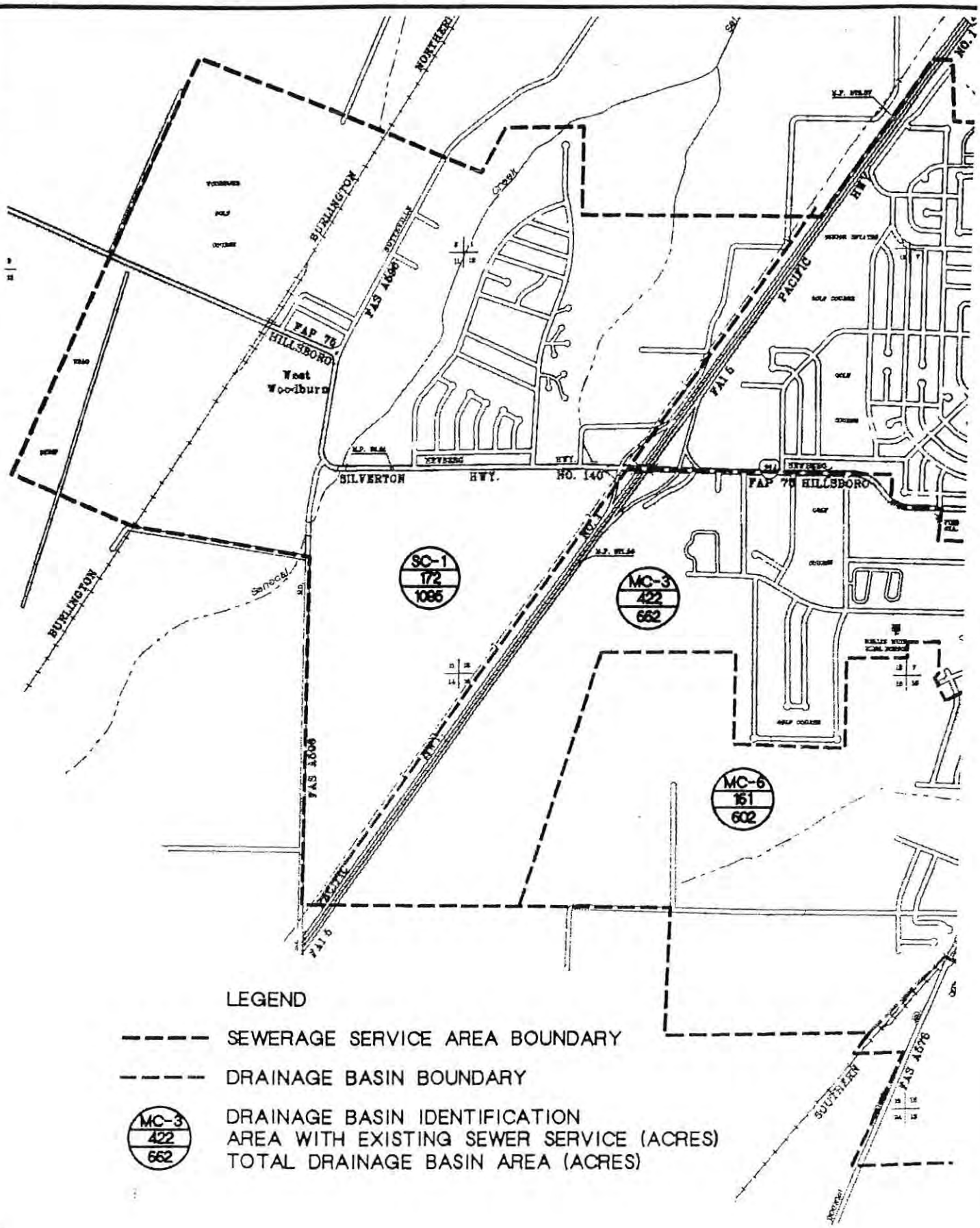
Inventory results indicate approximately 14.4 miles of trunk and interceptor lines handle sanitary sewerage flows for the City. Pipe diameters vary from 8 to 36 inches. Record drawings show that most of the system was constructed between 1954 and 1987. Tables 2, 3, and 4 show the amount of pipe associated with each drainage basin, pipe diameter, and year of construction, respectively.

The sanitary sewerage collection system and storm drainage system are separate in Woodburn. Although a few catch basins and roof and foundation drains may be connected to the sanitary sewerage system, there is no known combined sewer area. Therefore, storm sewers were not included in the collection system inventory.

Pump Stations

The Mill Creek pump station is the City's largest, with a capacity of 8.4 million gallons per day (mgd). The pump station collects wastewater from almost the entire service area and conveys it approximately 2 miles to the wastewater treatment plant. Gravity sewers

Figure 2



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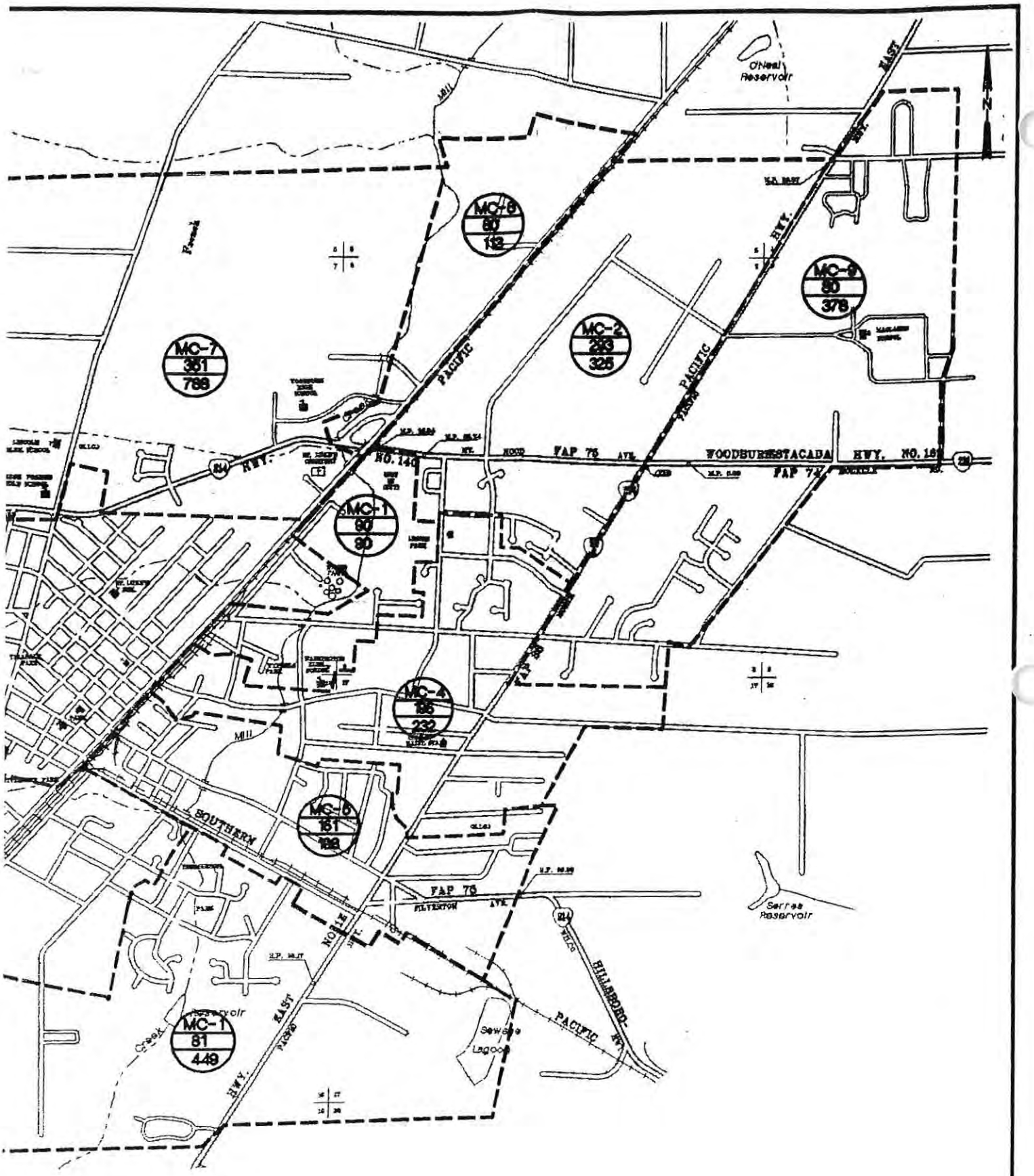

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FIGURE 2
SEWERAGE DRAINAGE BASINS
CITY OF WOODBURN FACILITIES PLAN

CH2M Hill

Table 2 Trunk and Interceptor Line Length Within Each Drainage Basin	
Drainage Basin	**Pipeline Length (ft)**
MC-1	8,463
MC-2	6,114
MC-3	19,743
MC-4	6,758
MC-5	5,251
MC-6	4,096
MC-7	12,836
MC-8	3,467
MC-9	1,252
SC-1	8,047
Total	76,027

Table 3 Trunk and Interceptor Line Length by Pipe Size	
Pipe Diameter (inches)	**Pipe Length (ft)**
36	1,854
30	436
27	1,742
24	1,610
18	9,806
16	3,346
15	6,553
14	1,008
12	16,485
10	27,808
8	5,379
Total	76,027

Table 4 Trunk and Interceptor Line Length by Age	
Year of Construction	**Pipe Length (ft)**
1987	3,467
1985	13,563
1976	855
1975	5,130
1974	2,343
1973	1,313
1970	2,312
1969	2,945
1968	1,101
1966	520
1965	3,381
1964	1,329
1963	3,623
1962	1,477
1961	880
1960	9,499
1955	5,560
1954	4,248
Unknown (Mostly Pre-1954)	12,481
Total	76,027

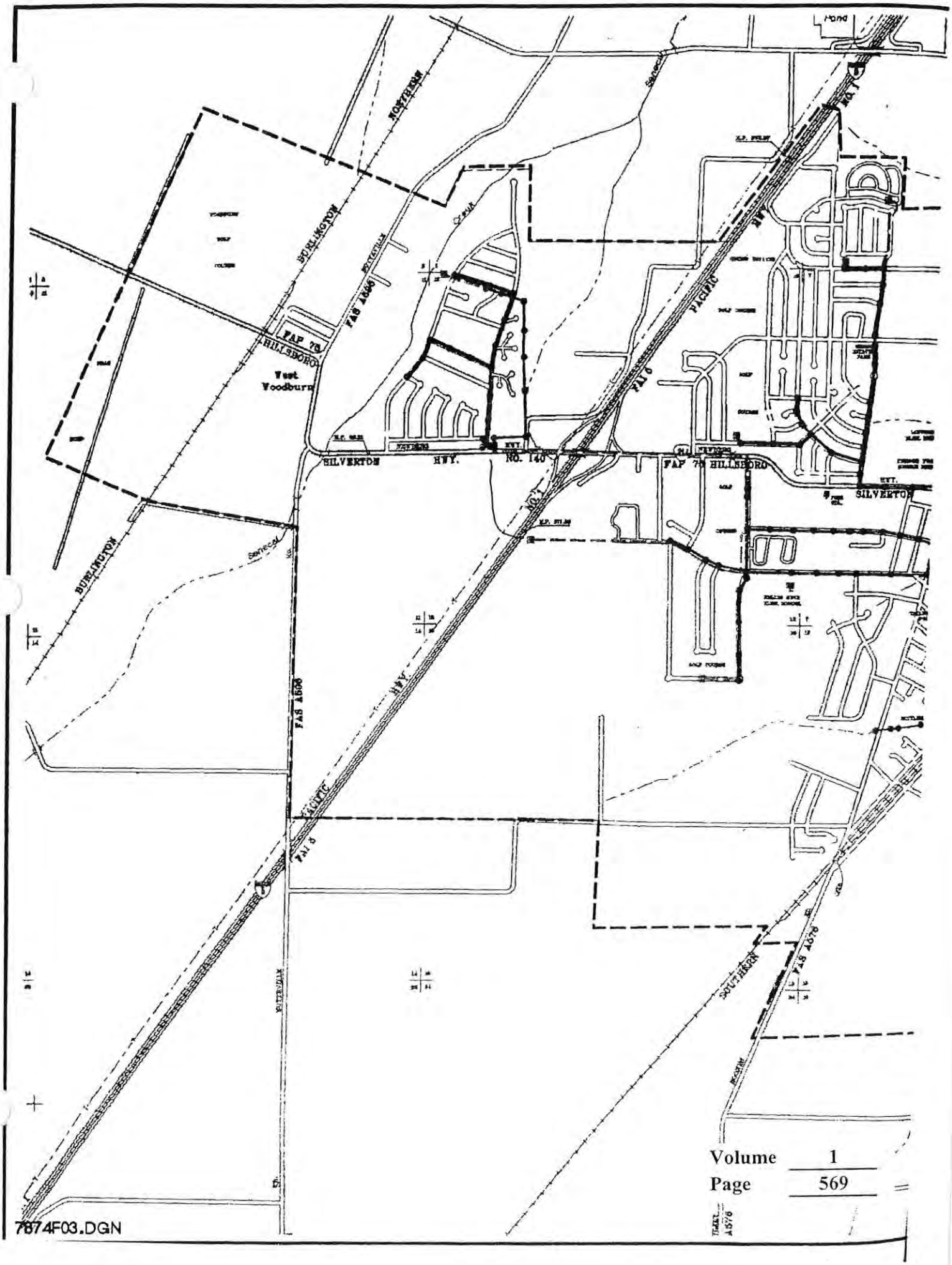
carry most flows to the Mill Creek pump station, but a number of small, localized areas with adverse grades are served by ten smaller pump stations ranging in capacity from 120 to 750 gallons per minute (gpm). Data on all of the City's pump stations are listed in Table 5. Pump station ratings reflect design capacity, and can be considered an approximation of pump flow under normal operational conditions and an average wet well water surface elevation. Pump station locations are shown in Figure 3.

Table 5
Summary Data on Woodburn Collection System Pump Stations

Station Number	Station Location	Year of Construction	Capacity Rating (gpm, except as noted)[a]	Number of Pumps	Notes
1	Vanderbeck Way	1963	150	–	Pneumatic ejector
2	Jansen Way	1968	128	2	2 @ 3 hp
3	Rainer Road	1963	150	–	Pneumatic ejector
4	Woodland Avenue	1967	750	2	1 @ 15 hp, 1 @ 10 hp
5	Willow Avenue	1969	200	2	2 @ 3 hp
6	Santiam Drive	1966	120	2	2 @ 3 hp
7	Greenview Drive	1972	N/A	2	2 @ 20 hp
9	Industrial Avenue	1974	300	2	2 @ 7.5 hp
10	West Woodburn near I-5	1992	450	3	1 @ 215 hp, 2 @ 10 hp
Mill Creek	Highway 214 at Boones Ferry	1980	8.4 mgd	3	2 @ 200 hp, 1 @ 150 hp

[a]Capacity rating reflects one pump out of service.
hp = horsepower.

Figure 3


BURLINGTON
NORTHERN
FAS 1666
FAP 76
HILLSBORO
West Woodburn
SILVERTON HWY. NO. 140
FAP 76 HILLSBORO
SILVERTON
PACIFIC
HWY.
NO. 1
FAS 1666
SOUTHERN
FAS 1676
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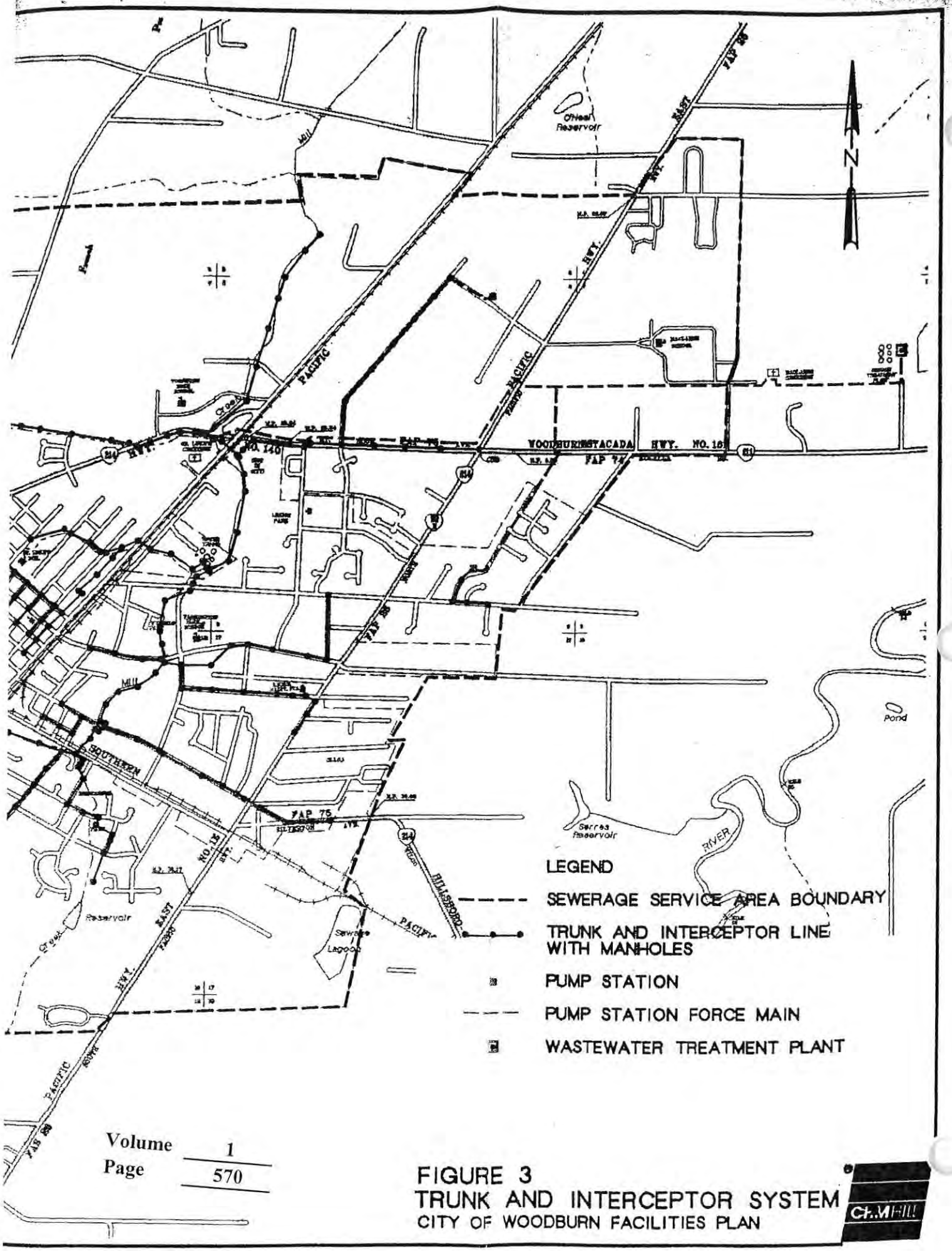

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FIGURE 3
TRUNK AND INTERCEPTOR SYSTEM
CITY OF WOODBURN FACILITIES PLAN

CH2M HILL

Diversions

A diversion is a point at which sewerage flow can go into either of two downstream pipes. A diversion is usually added to a system to help relieve overloading in a particular line or to achieve a balance in flow between two lines. A diversion does not allow flows to exit the collection system. The Woodburn trunk and interceptor system does not include any known diversions.

Overflows

An overflow is a designated point at which raw sewage can leave a collection system. Overflows were once included in nearly every collection system and were considered an acceptable method for providing controlled relief during overload periods. Overflow piping either went directly to a ditch or open waterway or was connected to the storm sewer system. Current concerns about the health and environmental hazards of raw sewage preclude the use of planned sanitary system overflow; new overflows cannot be constructed, and existing overflows must be removed from a system.

The five potential overflow locations within the Woodburn collection system are listed in Table 6 with their current status. If potential overflows are active, they could either release raw sewage to the storm sewer system or, depending on the configuration and hydraulics of the cross connection, allow storm water to enter the sanitary sewerage system. Neither condition is acceptable and, if found, must be corrected.

Table 6
Potential Overflows in the Woodburn Collection System

Location	Status
Conroy Packing	Suspect area, potentially active
First Street at Settlemier Park	Plugged, inactive
Fifth Street at St. Luke's School	Status unknown
By Tennis Court	Plugged, inactive
First and Harrison Streets	Potentially active

Hydraulic Capacity Analysis

A sewerage collection system must have the capacity to handle the simultaneous occurrence of peak sanitary flow and peak infiltration/inflow (I/I). Without adequate capacity, raw sewage may overflow the system and could present health and environmental hazards. A hydraulic analysis of the Woodburn trunk and interceptor system was conducted to determine whether it has the capacity to convey current, 5-year, and buildout peak flows. A computer model was used to calculate pipeline capacities, estimate peak flows, and compare the capacities to the flows to determine portions of the system that are, or will be, undersized and require improvement. The analysis also provided information for preliminary identification of a general location for a new wastewater treatment plant for the Woodburn service area.

Description of Hydraulic Analysis Model

A computer simulation of the collection system operating under current and future peak flow conditions was made using the System Analysis Model (SAM), which was developed by CH2M HILL for hydraulic analysis. Data are input to SAM to characterize the physical features of a collection system and the amount and source of the flows it must convey. For pipeline capacity calculations, system input data are compiled from pipe diameters, pipe lengths, manhole invert elevations, and manhole lid elevations. Information on pipe slope and roughness is also used. Characteristics of special structures, such as pump stations, diversions, or overflows, may be entered in the model. Input data used to estimate system peak flows include the number and location of residential and commercial/industrial customers served by the system, land use characteristics, sanitary contribution rates for each type of customer, sanitary peaking factors, and peak I/I rates. The latter are a function of system condition, rainfall, and groundwater levels.

SAM uses the standard Manning equation to calculate a capacity for each pipeline segment in the system. Flows are estimated at key points in the trunk and interceptor system and reflect the variation of sanitary flow during a typical day, as well as a peak I/I rate that remains constant. The model accumulates flows and uses a mathematical simulation to route them through the system as a function of time. Flow routing allows for optimum use of pipeline capacities, as it reflects the attenuation of peak flow that occurs as sanitary flow enters different parts of the collection system at different times. When flow exceeds the open channel capacity of a pipeline, SAM simulates surcharge and allows the hydraulic gradeline upstream of the pipeline to rise until enough head is generated to force the flow through the pipeline.

SAM output indicates the percent of capacity used by each pipeline segment at the time peak flow occurred. If the capacity of a pipeline is exceeded, the output shows the degree and duration of system surcharge and whether overflow is likely to occur. Through this process, potential system deficiencies may be identified and alternatives for system improvements may be formulated.

Pump stations and trunk and interceptor manholes are identified in SAM input and output data using alphanumeric codes. Figure 4 (in the back of this document) is a large-scale map showing major collection system components and corresponding SAM identification numbers.

Input Data and Criteria

Pipeline Capacity

The capacity of each pipeline segment was calculated by SAM according to the Manning equation for a full pipe. A Manning roughness coefficient of 0.013 was used in the analysis. Pipe diameters, pipe lengths, and manhole invert elevations provide the remainder of the data required to complete the capacity calculations.

Peak Flow

Peak flow for each pipeline segment was calculated using estimates of average daily sanitary flow, sanitary peaking factors, and a peak I/I rate. Sanitary flow generally originates at domestic dwellings, commercial developments, and industrial sites, so the magnitude of the average daily sanitary flow is a function of the area served, the type of development and development densities, and per capita or per acre sanitary contributions. This section discusses the criteria influencing sanitary flow estimates, as well as the peaking factors and peak I/I rates used for the analysis.

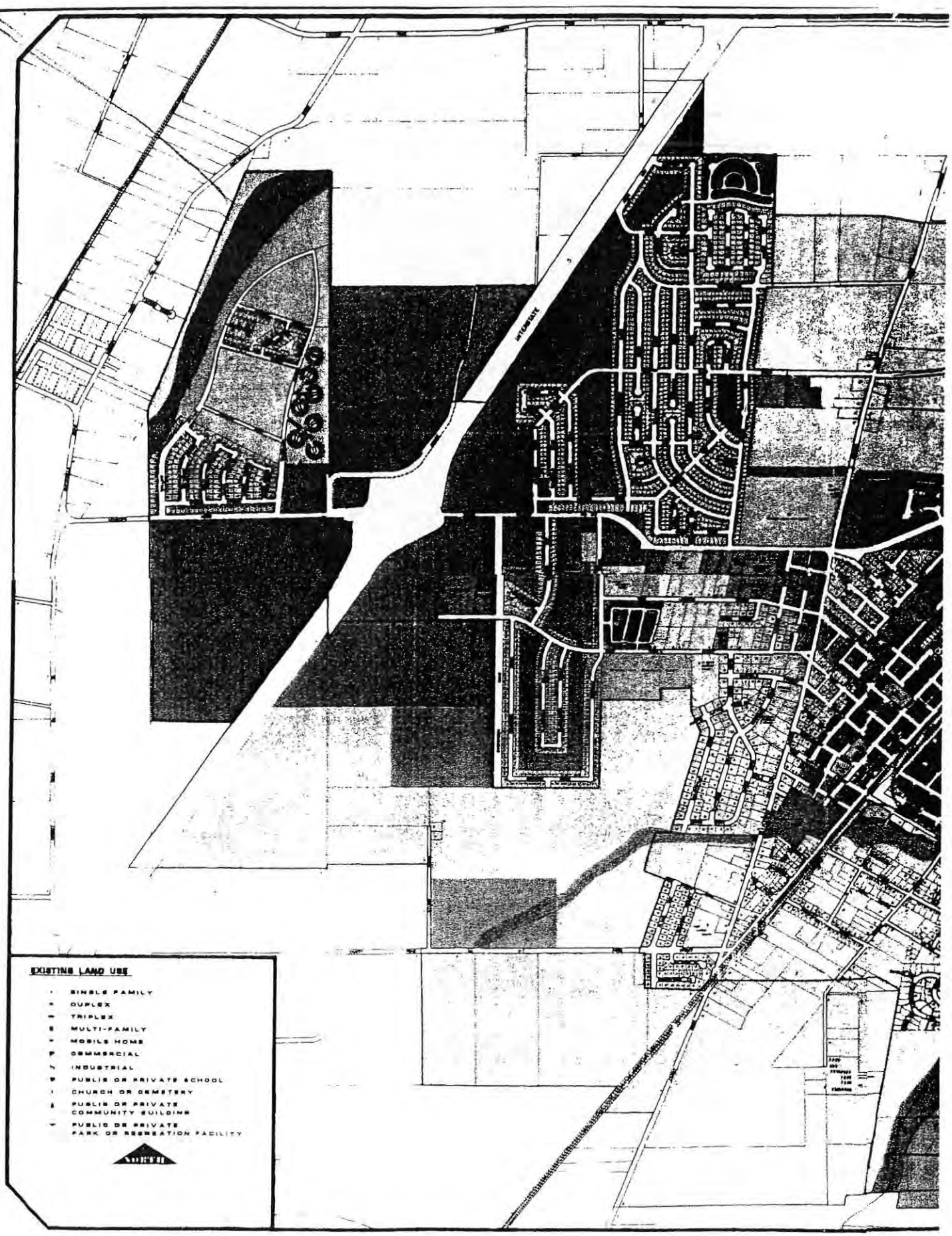
Planning Area. The planning area is the ultimate service area described in the system inventory, or about 4,913 acres. About 40 percent of the planning area currently has sanitary sewerage service; this proportion is expected to increase by 5 percent by 1997.

Each of the ten service area drainage basins was divided into smaller sub-basins for estimating system flows. Each sub-basin represents an area that contributes sanitary flow and I/I to a specific point in the collection system. In most cases, sub-basins are identified by an existing network of lateral lines that connect to the trunk and interceptor system at a single location. Each of the 126 sub-basins used has an average size of about 15 acres and is shown in Figure 5.

Land Use and Development. The number of dwelling units and the amount of commercial and industrial development in Woodburn are guided by the Woodburn Comprehensive Land Use Plan, which was last updated in 1989. The plan designates land for low-density (single-family) residential, high-density (multi-family) residential, commercial, and industrial development. Some land is also designated for public use and open space. Figure 6 is a copy of the City's land use planning map.

The low- and high-density residential designations allow a maximum of 12 and 25 dwelling units per acre, respectively. The density of commercial and industrial development is not specified by the plan, but the areas where commercial and industrial development may occur are explicitly defined.

Figure 5



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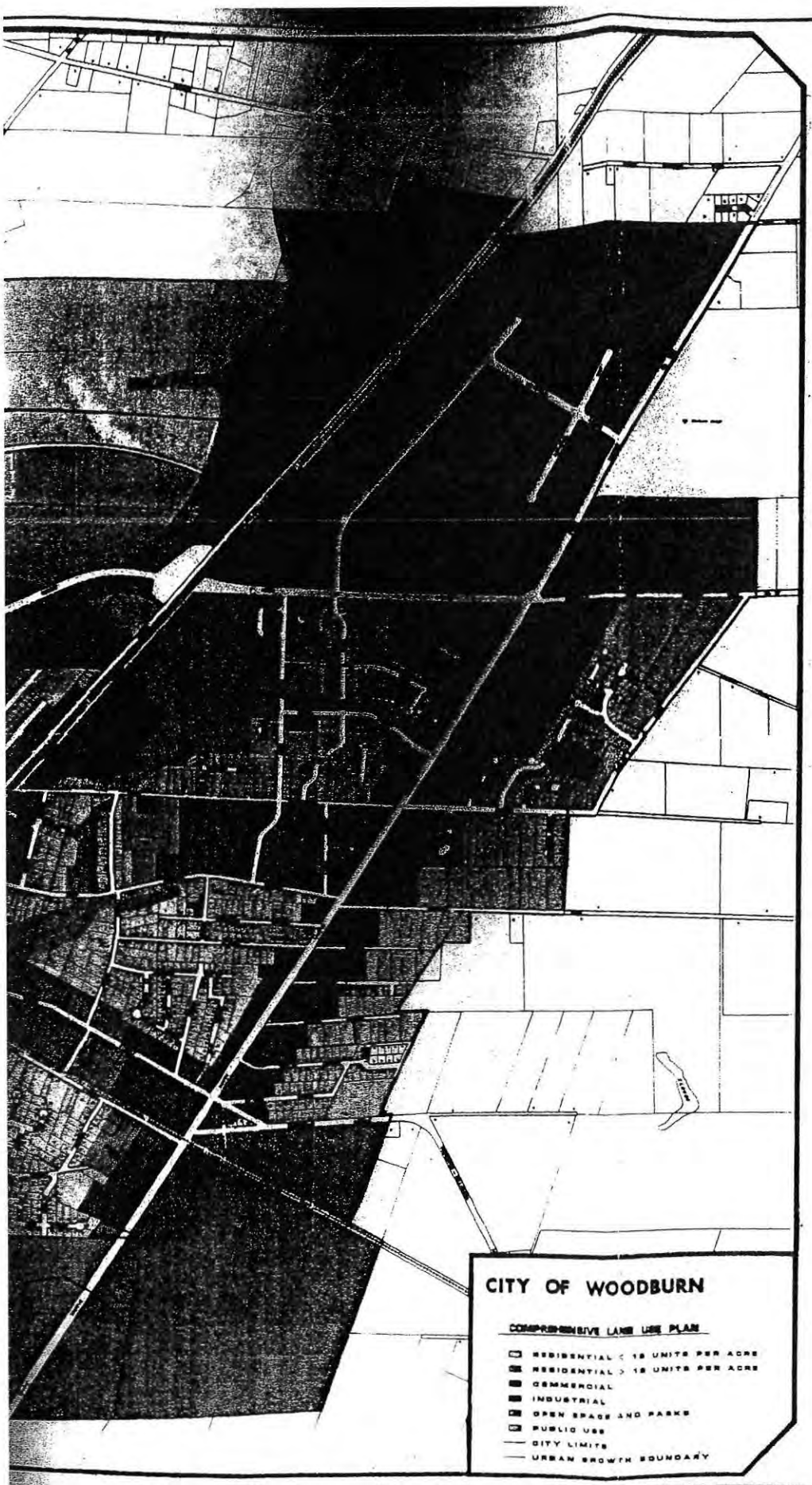

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FIGURE 6
COMPREHENSIVE LAND USE
CITY OF WOODBURN FACILITIES PLAN

CH2MHILL

Figure 6

Areas within the planning area but outside the UGB were not included in the Comprehensive Land Use Plan. These outlying areas will probably be designated for low-density residential development and were so considered for the purpose of this analysis.

According to the Comprehensive Land Use Plan, the planning area includes 3,185 acres for low-density residential development; 327 acres for high-density residential development; and 1,092 acres for commercial and industrial development. The land use inventory for the planning area is summarized by drainage basin in Table 7 and divided by sub-basin in Appendix B.

Table 7
Land Use Inventory

Drainage Basin	Low-Density Residential (acres)	High-Density Residential (acres)	Commercial/ Industrial (acres)	Other (acres)	Total (acres)
MC-1	472	65	212	82	831
MC-2	20	3	331	0	354
MC-3	1,301	67	138	38	1,545
MC-4	132	41	52	13	239
MC-5	60	34	80	8	183
MC-6	516	59	0	33	608
MC-7	239	50	103	90	482
MC-8	257	0	130	24	411
MC-9	60	7	19	4	90
SC-1	127	0	27	17	172
Total	3,185	327	1,092	309	4,913

The level of current residential development was estimated based on an existing population of 16,219 persons. Current commercial and industrial development was estimated from aerial photographs. Development levels for 1997 were based on a projected annual growth rate of about 4 percent. Buildout development levels are dictated by the Comprehensive Land Use Plan. A summary of current, 5-year, and buildout development levels is shown in Table 8.

Table 8 Development Summary			
Type of Development	**Current (1992)**	**5-Year (1997)**	**Buildout**
Low-Density Residential (units)	3,033	3,832	38,225
Low-Density Residential (acres)	978	1,045	3,185
High-Density Residential (units)	2,974	3,758	8,175
High-Density Residential (acres)	204	235	327
Commercial/Industrial (acres)	413	522	1,092

In this analysis, residential development is generally expressed in dwelling units. Each dwelling unit has an average of 2.7 residents, according to 1990 data from the U.S. Census of Population and Housing.

The source of sanitary flow and its entry point to the collection system depend on the distribution of residential dwellings and commercial and industrial development. The distribution of development in Woodburn was determined from the land use planning map and a 1990 aerial photograph. Half of the development increases expected by 1997 were distributed based on building permit information provided by the Woodburn Planning Department. The remaining 5-year development was distributed evenly as in-fill throughout the sub-basins that were not fully built out. The amount of development in each sub-basin is listed for current and buildout conditions in Appendix C and for 5-year conditions in Appendix D.

Sanitary Contributions. The following average sanitary contributions were used in conjunction with land use characteristics to determine the average flow rate from each drainage basin:

- Low-Density Residential — 100 gallons per capita per day (gpcd)
- High-Density Residential — 100 gpcd
- Commercial and Industrial — 3,000 gallons per acre per day (gpad)

The magnitude of the residential sanitary contribution was estimated from the flow data presented in Technical Memorandum F4-1. Based on historical records and published values, a sanitary contribution of 3,000 gpad seems appropriate for the commercial and industrial development typically found in Woodburn.

Peaking Factors. Sanitary sewerage flows vary greatly throughout the typical day. The extreme low flows usually occur between 2 and 6 a.m., and the peak flows occur during the daylight hours. The ratio of sanitary flow at a specific time of day to the average daily sanitary flow is called the sanitary peaking factor.

Because SAM simulates collection system operation over a 24-hour period, a series of sanitary peaking factors is required to calculate the variation in sanitary flow expected throughout the day. A sanitary peaking factor curve that correlates sanitary peaking factor and time of day was generated from actual system flow data. The curve is shown in Figure 7 and incorporates maximum and minimum sanitary peaking factors of 2.4 and 0.1, respectively.

Infiltration and Inflow. I/I is the extraneous component of wastewater flow. Infiltration occurs when groundwater leaks into the sanitary sewerage collection system through such defects as cracked or broken pipe, defective pipe joints, or dilapidated manholes. Inflow occurs when storm runoff flows directly into the sanitary sewerage system at storm sewer cross connections, roof and foundation drains, catch basins, or faulty or submerged manholes. I/I rates vary with the duration and intensity of rainfall and the level of existing groundwater.

Current I/I rates for the Woodburn sanitary sewerage collection system were estimated using actual system flow data. The peak I/I rates presented in Technical Memorandum F4-1 for the six drainage areas were rounded to the nearest 500 gpad for use in SAM. The resulting I/I rates are shown in Table 9. I/I is represented at a constant rate during the 24-hour flow simulation period of the collection system model.

Table 9
Peak I/I Rates in the Woodburn Service Area

Drainage Area	Peak I/I Rate (gpad)
West Woodburn and West of I-5	4,000
North Woodburn	5,000
Central Downtown Woodburn	8,000
South Woodburn	1,500
Southeast Woodburn	3,500
Northeast Woodburn	4,000
Weighted Average	4,000

A peak I/I rate of 1,500 gpad was used in the model for the additional land area served with sanitary sewers under the 5-year and buildout conditions. This rate is reasonable for sewers built using modern pipeline materials and construction techniques.

Modeling Results

Output from SAM was reviewed to identify portions of the collection system that may not have the capacity to convey peak flows under the current, 5-year, or buildout conditions.

Figure 7

An under-capacity pipeline may not inherently pose an operational problem, however. Surcharge or backup of flow upstream of the pipeline may create enough head or pressure to force the flow past the restriction. Operational problems result when the level of surcharge required to produce the required head reaches an elevation where system overflow may occur.

SAM output indicates the potential for system overflow when the hydraulic gradeline or water surface elevation at a manhole location reaches the rim elevation of the manhole. If there are low structures or basements in the area, overflow may actually occur before the water surface elevation reaches the manhole rim elevation.

This section presents the peak system flows for the current, 5-year, and buildout conditions. Corresponding statistics regarding pipeline capacity, surcharge, and overflow are also presented. In some cases, surcharge or overflow is the result of downstream backwater effects rather than a capacity limitation of the particular pipeline segment.

Current (1992) Conditions

SAM input for the current condition reflects an instantaneous peak system flow of 12.9 mgd. Routing effects simulated by the model attenuate this flow to a peak of 12 mgd at the Mill Creek pump station. A breakdown of input flow is shown by drainage basin in Table 10.

Table 10
Current (1992) Peak Flows

Drainage Basin	Peak Sanitary Flow (mgd)	Peak I/I Flow (mgd)	Peak Total Flow (mgd)
MC-1	0.14	0.44	0.58
MC-2	0.66	1.17	1.83
MC-3	1.23	2.37	3.61
MC-4	0.34	0.69	1.03
MC-5	0.56	0.56	1.12
MC-6	0.13	0.24	0.37
MC-7	0.87	1.75	2.62
MC-8	0.05	0.32	0.36
MC-9	0.20	0.32	0.52
SC-1	0.14	0.69	0.83
Total	4.32	8.56	12.87

According to the hydraulic analysis, most pump stations and trunk and interceptor lines in the Woodburn collection system have sufficient capacity to convey current peak flow. The pump stations and pipelines that serve portions of central downtown Woodburn and north Woodburn are the only facilities that require improvement to provide the increased hydraulic capacity necessary to convey current peak flow. Tables 11 and 12 summarize the performance of Woodburn's pump stations and collection system pipelines under current peak flow. Pump stations and pipelines without sufficient capacity to convey peak flow are highlighted in Figure 8. A discussion of the analysis for each drainage basin is presented below. SAM output, which is the basis for the system analysis, is contained in Appendix E.

Drainage Basin MC-1. Sewerage facilities in drainage basin MC-1 include the Mill Creek pump station and most of the Mill Creek interceptor pipeline, which serves south Woodburn. According to the hydraulic analysis, peak flow exceeds the rated capacity of the Mill Creek pump station by an estimated 43 percent. A capacity upgrade of the pump station is currently underway. Peak flow also exceeds the capacity of a 36-inch diameter pipeline located near the City compost dump. The capacity of the line is limited by its slope. No operational problems are expected to occur, as slight surcharge will force the peak flow though the line.

Drainage Basin MC-2. Sewerage facilities in drainage basin MC-2 include pump station 2 and pipelines that serve northeast Woodburn. In the analysis, peak flow exceeded the rated capacity of the pump station, which is located on Industrial Avenue, by an estimated 16 percent. City staff should monitor the operation of pump station 2 during wet-weather conditions to determine whether it will require additional capacity. Peak flow exceeds the capacity of two 10-inch diameter pipelines along Progress Way. The capacity of each line is limited by its slope. No operational problems are expected to occur, as slight surcharge will force the peak flow though the lines.

Drainage Basin MC-3. Sewerage facilities in drainage basin MC-3 include a pump station on Santiam Drive and pipelines that serve central downtown Woodburn. According to the analysis, peak flow exceeds the capacity of 36 pipeline segments in this basin, 28 of which are located on the basin's main trunk line. This line reaches nearly 7,000 feet from the Mill Creek interceptor upstream through downtown Woodburn to Lincoln Street. Line sizes range from 10 to 16 inches in diameter. The City staff has observed excessive surcharge of this line in the past. The line is expected to cause continuing operational problems until its capacity is increased or flow relief is provided. Other lines in the basin along Hayes Street and Cascade Drive will operate under slight surcharge during peak flow but should cause no operational problems.

Drainage Basin MC-4. Sewerage facilities in drainage basin MC-4 include pipelines that serve the downtown area east of Front Street. According to the analysis, the pipelines in this basin have adequate capacity to convey the current peak flow.

Drainage Basin MC-5. Sewerage facilities in drainage basin MC-5 include pipelines serving the south downtown area just north of Cleveland Street. The analysis indicates that these pipelines have adequate capacity to convey the current peak flow.

Table 11 Pump Station Performance, Current (1992) Conditions			
Pump Station	**Capacity Rating (gpm, except as noted)**	**Current Peak Flow (gpm, except as noted)**	**Percent Capacity**
PS-1	150	376	251
PS-2	128	156	122
PS-3	150	167	112
PS-4	750	449	60
PS-5	200	111	56
PS-6	120	92	76
PS-7	N/A	365	N/A
PS-9	300	348	116
PS-10	450	0	0
Mill Creek	8.4 mgd	12.0 mgd	143

Table 12 Trunk and Interceptor Line Performance, Current (1992) Conditions			
Drainage Basin	**Lines Over Capacity**	**Percent of Lines Surcharged**	**Potential Overflow**
MC-1	1	3	No
MC-2	2	35	No
MC-3	36	94	Yes
MC-4	0	0	No
MC-5	0	0	No
MC-6	0	0	No
MC-7	13	50	Yes
MC-8	0	0	No
MC-9	0	0	No
SC-1	0	0	No
Total	52	36	--

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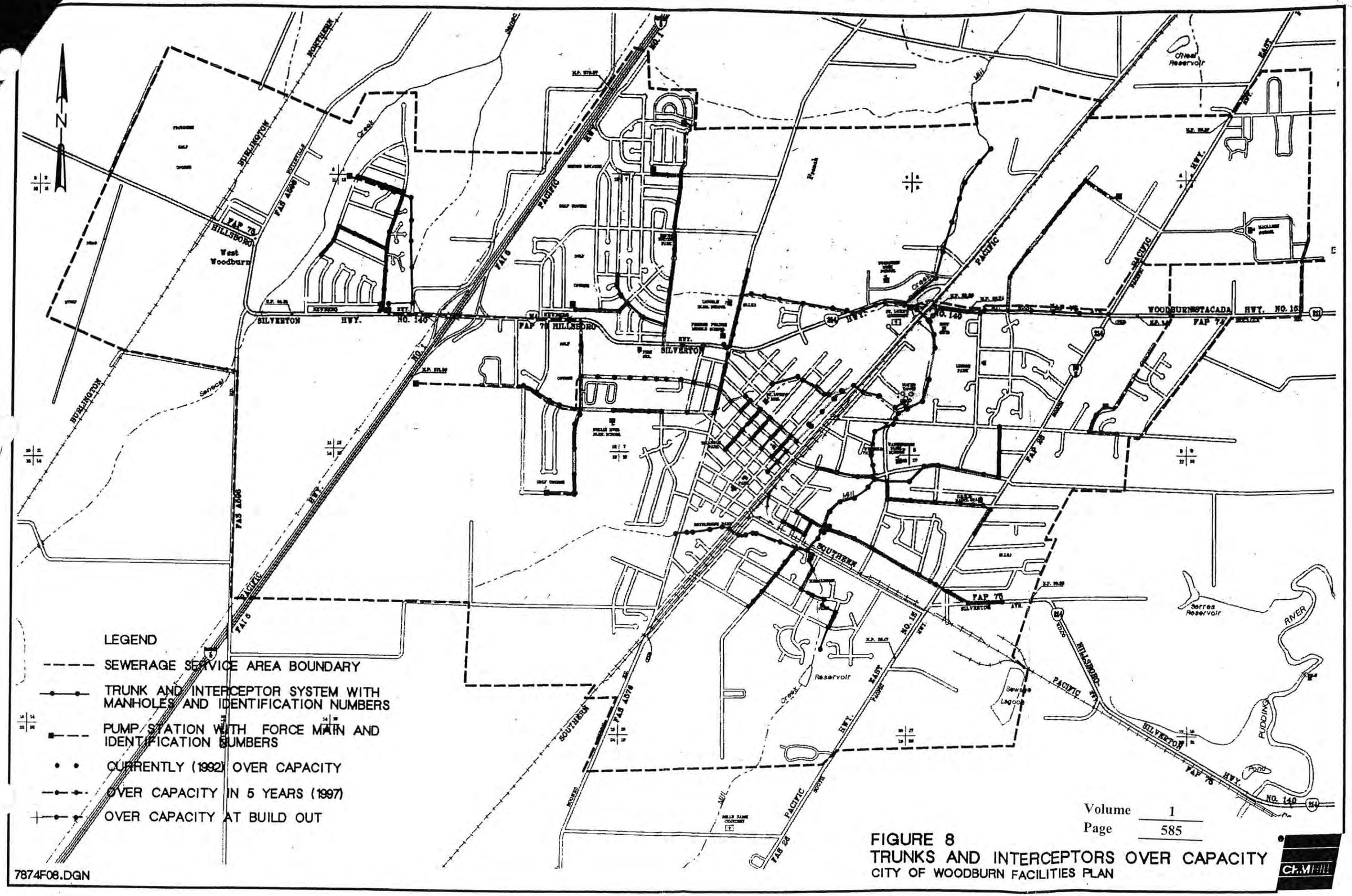

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FIGURE 8
TRUNKS AND INTERCEPTORS OVER CAPACITY
CITY OF WOODBURN FACILITIES PLAN

CH2M HILL

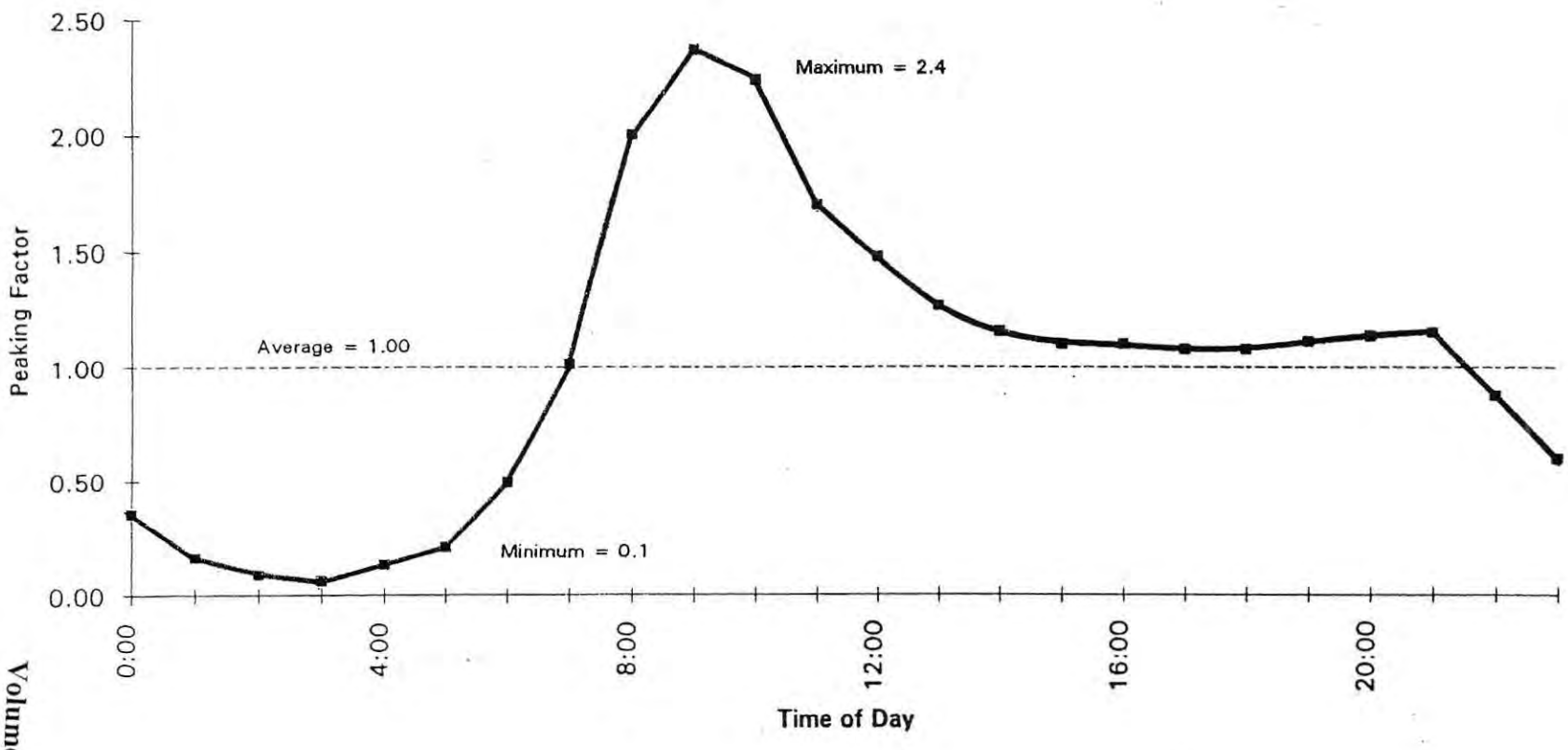

FIGURE 7
SANITARY PEAKING FACTOR CURVE
CITY OF WOODBURN FACILITIES PLAN

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Figure 8

Drainage Basin MC-6. Sewerage facilities in drainage basin MC-6 include the upstream end of the Mill Creek interceptor and other pipelines serving south Woodburn. According to the analysis, the pipelines have adequate capacity to convey the current peak flow.

Drainage Basin MC-7. Sewerage facilities in drainage basin MC-7 include pipelines serving north Woodburn and pump stations 1, 2, and 3, located on Vanderbeck Way, Jansen Way, and Rainer Road, respectively. Peak flows exceed the rated capacities of pump stations 1, 2, and 3 by an estimated 151, 22, and 12 percent, respectively. City staff should monitor the operation of the pump stations (especially pump station 1) during wet-weather conditions to determine whether they will require upgrade. Peak flow exceeds the capacity of a portion of the basin's main trunk line that reaches nearly 4,000 feet along Highway 214 and up Astor Way. This portion of the trunk line is 12 inches in diameter and is expected to cause operational problems until its capacity is increased or flow relief is provided.

Drainage Basin MC-8. Sewerage facilities in drainage basin MC-8 include pipelines that serve northeast Woodburn near Mill Creek. According to the analysis, the pipelines in this basin have adequate capacity to convey the current peak flow.

Drainage Basin MC-9. Sewerage facilities in drainage basin MC-9 include pump station 7 and pipelines serving a small area in east Woodburn. Although the rated capacity of the pump station (located on Greenview Drive) is unknown, it has 20-horsepower pumps that probably provide adequate capacity to convey the expected peak flow. According to the hydraulic analysis, the pipelines have adequate capacity to convey the current peak flow.

Drainage Basin SC-1. Sewerage facilities in drainage basin SC-1 include pump stations 4 and 5 and pipelines serving the area of Woodburn that lies to the west of Interstate 5. According to the analysis, the pump stations and pipelines in this basin have adequate capacity to convey the current peak flow.

5-Year (1997) Conditions

Increased development and a growing service area are expected to increase the peak system flow an additional 1.2 mgd by 1997. SAM input for the 5-year condition reflects a peak system flow of 14.1 mgd. Routing effects simulated by the model attenuate this flow to a peak of 13 mgd at the Mill Creek pump station. Peak sewage flow is shown by drainage basin in Table 13.

According to the hydraulic system analysis, increased flow under the 5-year condition is not expected to cause operational problems in addition to those identified for current conditions, but those problems are expected to get worse. Tables 14 and 15 summarize the performance of pump stations and trunk and interceptor lines under 5-year peak flow conditions.

Table 13 5-Year (1997) Peak Flows			
Drainage Basin	**Peak Sanitary Flow (mgd)**	**Peak I/I Flow (mgd)**	**Peak Total Flow (mgd)**
MC-1	0.39	0.69	1.08
MC-2	0.71	1.18	1.89
MC-3	1.57	1.70	3.27
MC-4	0.41	0.79	1.20
MC-5	0.59	0.64	1.23
MC-6	0.28	0.66	0.94
MC-7	1.04	1.41	2.45
MC-8	0.25	0.33	0.58
MC-9	0.22	0.32	0.54
SC-1	0.24	0.69	0.92
Total	5.68	8.41	14.09

Table 14 Pump Station Performance, 5-Year (1997) Conditions			
Pump Station	**Capacity Rating (gpm, except as noted)**	**Current Peak Flow (gpm, except as noted)**	**Percent Capacity**
PS-1	150	382	255
PS-2	128	152	119
PS-3	150	170	113
PS-4	750	514	69
PS-5	200	44	22
PS-6	120	104	87
PS-7	N/A	374	N/A
PS-9	300	367	122
PS-10	450	0	0
Mill Creek	8.4 mgd	13.0 mgd	155

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Table 15 Trunk and Interceptor Line Performance, 5-Year (1997) Conditions			
Drainage Basin	Lines Over Capacity	Percent of Lines Surcharged	Potential Overflow
MC-1	1	3	No
MC-2	3	35	No
MC-3	36	94	Yes
MC-4	0	0	No
MC-5	0	0	No
MC-6	0	0	No
MC-7	13	50	Yes
MC-8	0	0	No
MC-9	0	0	No
SC-1	0	0	No
Total	27	36	--

Buildout Conditions

SAM input for the buildout condition reflects a peak system flow of 38.7 mgd. Routing effects simulated by the model attenuate this flow to a peak of 35.2 mgd at the Mill Creek pump station. Peak sewage flow is shown by drainage basin in Table 16.

The performance of the collection system trunk and interceptor lines under buildout peak flow is summarized in Table 17. The results of the hydraulic analysis for buildout conditions are intended to serve as a tool for long-range system planning. Because the service life of sanitary sewers is typically 50 years or more, a line should be sized to convey buildout flow when it is replaced. Pump stations and pipelines without sufficient capacity to convey peak flow are highlighted in Figure 8.

The magnitude of peak flow in portions of the trunk and interceptor system may change if certain design criteria change. Flow predictions could also be affected by changes in zoning regulations, sewerage service area, sanitary contributions, peaking factor, or I/I rates. These planning criteria should be monitored and the hydraulic model of the collection system should be updated regularly as part of the City's ongoing capital improvements planning process.

Wastewater Treatment Plant Location

The results of the hydraulic analysis show that location of a second wastewater treatment plant in west or south Woodburn would be most cost-effective with respect to collection

Table 16 Buildout Peak Flows			
Drainage Basin	**Peak Sanitary Flow (mgd)**	**Peak I/I Flow (mgd)**	**Peak Total Flow (mgd)**
MC-1	4.21	1.43	5.63
MC-2	1.33	1.26	2.59
MC-3	8.69	4.06	12.75
MC-4	1.43	0.75	2.18
MC-5	1.03	0.60	1.63
MC-6	3.63	0.91	4.55
MC-7	2.32	1.95	4.27
MC-8	1.93	0.82	2.74
MC-9	0.49	0.33	0.83
SC-1	0.82	0.69	1.51
Total	25.87	12.79	38.6

Table 17 Trunk and Interceptor Line Performance, Buildout Conditions			
Drainage Basin	**Lines Over Capacity**	**Percent of Lines Surcharged**	**Potential Overflow**
MC-1	29	89	Yes
MC-2	11	65	Yes
MC-3	48	97	Yes
MC-4	3	23	No
MC-5	2	14	No
MC-6	7	56	No
MC-7	19	50	Yes
MC-8	6	88	No
MC-9	0	0	No
SC-1	0	0	No
Total	125	59	--

system improvements. Location of a second treatment plant in west Woodburn would relieve flow through the trunk line that serves central downtown Woodburn. This line currently shows capacity limitations and will require improvement to increase hydraulic capacity if the flows are not reduced. Alternatively, location of a second treatment plant in south Woodburn would relieve flow through the Mill Creek interceptor that serves that area. Buildout peak flow will exceed the capacity of this line if flows are not reduced.

Appendix A

City of Woodburn - - Facilities Plan
Modeled Collection System Inventory

Upstream			Downstream		Pipe				Notes
Manhole (#)	Invert (elev.)	Rim (elev.)	Manhole (#)	Invert (elev.)	Length (ft)	Diameter (in)	Material (type)	Age (year)	
PS-MC	---	---	STP	---	11,015	18-FM	---	1980	
MC1-005	137.53	157.5	PS-MC	137.16	32	30	---	1985	B
MC1-010	142.20	159.3	MC1-005	137.53	404	30	---	1985	A, C
MC1-015	142.37	159.3	MC1-010	142.20	12	36	---	1985	A, C
MC1-020	142.55	153.3	MC1-015	142.37	172	36	---	1985	
MC1-025	142.75	153.6	MC1-020	142.55	275	36	---	1985	
MC1-030	142.85	154.1	MC1-025	142.75	709	36	---	1985	
MC1-035	143.61	155.3	MC1-030	142.85	366	36	---	1985	
MC1-040	143.99	156.7	MC1-035	143.61	320	36	---	1985	
MC1-045	144.26	156.0	MC1-040	143.99	170	27	---	1985	
MC1-050	144.54	160.6	MC1-045	144.26	200	27	---	1985	A, C
MC1-055	144.66	165.2	MC1-050	144.54	90	27	---	1985	
MC1-060	145.17	162.0	MC1-055	144.66	301	27	---	1985	A, C
MC1-065	145.88	158.7	MC1-060	145.17	419	27	---	1985	
MC1-070	146.16	160.3	MC1-065	145.88	59	27	---	1985	
MC1-075	146.48	160.8	MC1-070	146.16	275	27	---	1985	A, C
MC1-080	146.74	161.2	MC1-075	146.48	228	27	---	1985	
MC1-085	147.32	159.0	MC1-080	146.74	125	24	---	1985	
MC1-090	148.22	160.5	MC1-085	147.32	226	24	---	1985	
MC1-095	148.66	161.4	MC1-090	148.22	388	24	---	1985	
MC1-100	149.11	161.8	MC1-095	148.66	309	24	---	1985	
MC1-105	149.38	165.4	MC1-100	149.11	344	24	---	1985	
MC1-110	150.43	165.8	MC1-105	149.38	105	24	---	1985	
MC1-115	151.46	164.1	MC1-110	150.43	94	18	---	1985	
MC1-120	153.76	165.9	MC1-115	151.46	359	18	---	1985	A, C
MC1-125	154.30	166.0	MC1-120	153.76	148	15	---	1985	

City of Woodburn - - Facilities Plan
Modeled Collection System Inventory

Upstream			Downstream		Pipe				Notes
Manhole (#)	Invert (elev.)	Rim (elev.)	Manhole (#)	Invert (elev.)	Length (ft)	Diameter (in)	Material (type)	Age (year)	
MC1-130	154.91	164.1	MC1-125	154.30	72	15	---	1985	
MC1-135	158.36	165.5	MC1-130	154.91	68	10	---	1985	
MC1-140	158.96	165.4	MC1-135	158.36	184	8	---	1962	
MC1-145	163.23	175.5	MC1-140	158.96	452	8	---	1962	
MC1-150	164.20	170.0	MC1-145	163.23	244	8	---	1962	
MC1-155	165.51	174.1	MC1-150	164.20	269	8	---	1973	
MC1-160	166.08	170.8	MC1-155	165.51	85	8	---	1973	
MC1-165	166.78	172.2	MC1-160	166.08	240	8	---	1973	
MC1-170	166.91	170.6	MC1-165	166.78	50	8	---	1973	
MC1-175	167.59	172.7	MC1-170	166.91	231	8	---	1973	
MC1-180	169.06	173.3	MC1-175	167.59	438	8	---	1973	
MC2-005	159.55	179.0	MC1-010	142.20	77	18	---	1975	
MC2-010	159.83	180.2	MC2-005	159.55	206	18	---	1975	
MC2-015	160.09	179.8	MC2-010	159.83	339	18	---	1975	A, C
MC2-020	160.38	179.4	MC2-015	160.09	375	18	---	1975	
MC2-025	160.65	179.7	MC2-020	160.38	242	18	---	1975	A, C
MC2-030	161.19	180.0	MC2-025	160.65	497	18	---	1975	
MC2-035	162.14	181.2	MC2-030	161.19	500	18	---	1975	
MC2-040	162.80	181.7	MC2-035	162.14	500	12	---	1975	
MC2-045	163.66	181.7	MC2-040	162.80	515	12	---	1975	
MC2-050	164.61	182.7	MC2-045	163.66	520	12	---	1975	
MC2-105	164.14	181.8	MC2-030	161.19	563	10	---	1974	
MC2-110	166.26	181.1	MC2-105	164.14	280	10	---	1974	
MC2-115	167.62	180.0	MC2-110	166.26	451	10	---	1974	A, C
MC2-120	168.84	180.0	MC2-115	167.62	400	10	---	1974	A, C
MC2-125	169.92	180.0	MC2-120	168.84	349	10	---	1974	A, C

City of Woodburn - - Facilities Plan
Modeled Collection System Inventory

Upstream			Downstream		Pipe				Notes
Manhole (#)	Invert (elev.)	Rim (elev.)	Manhole (#)	Invert (elev.)	Length (ft)	Diameter (in)	Material (type)	Age (year)	
MC2-130	170.81	178.0	MC2-125	169.92	300	10	---	1974	A, C
PS-9	---	---	MC2-130	170.81	782	6-FM	---	1974	
MC3-005	144.49	157.7	MC1-040	143.99	60	18	---	1985	
MC3-010	145.23	159.1	MC3-005	144.49	30	12	---	1985	
MC3-015	153.49	157.3	MC3-010	145.23	132	16	---	1960	
MC3-020	154.51	160.0	MC3-015	153.49	481	16	---	1960	
MC3-025	157.58	166.5	MC3-020	154.51	365	16	---	1960	A, C
MC3-030	158.45	172.9	MC3-025	157.58	104	18	---	1960	
MC3-035	159.50	169.0	MC3-030	158.45	213	14	---	1960	
MC3-040	160.52	169.8	MC3-035	159.50	210	14	AC	1960	
MC3-045	162.82	174.9	MC3-040	160.52	338	12	AC	1960	
MC3-050	165.06	180.0	MC3-045	162.82	313	12	AC	1960	
MC3-055	165.11	180.4	MC3-050	165.06	20	16	AC	1960	
MC3-060	165.84	180.6	MC3-055	165.11	405	16	AC	1960	
MC3-065	166.03	180.2	MC3-060	165.84	156	16	AC	1960	
MC3-070	166.19	180.6	MC3-065	166.03	245	16	AC	1960	
MC3-075	166.29	177.4	MC3-070	166.19	266	15	AC	1960	
MC3-080	166.55	179.1	MC3-075	166.29	249	16	AC	1960	
MC3-085	166.64	179.8	MC3-080	166.55	252	16	AC	1960	
MC3-090	166.93	178.0	MC3-085	166.64	158	16	AC	1960	
MC3-095	167.37	177.9	MC3-090	166.93	283	16	AC	1960	
MC3-100	167.84	180.0	MC3-095	167.37	340	16	AC	1960	
MC3-105	168.80	182.1	MC3-100	167.84	260	10	---	1966	
MC3-110	169.58	183.4	MC3-105	168.80	260	10	---	1966	
MC3-115	170.28	182.3	MC3-110	169.58	258	10	---	1975	
MC3-120	170.98	181.8	MC3-115	170.28	312	10	---	1975	

City of Woodburn - - Facilities Plan
Modeled Collection System Inventory

Upstream			Downstream		Pipe				Notes
Manhole (#)	Invert (elev.)	Rim (elev.)	Manhole (#)	Invert (elev.)	Length (ft)	Diameter (in)	Material (type)	Age (year)	
MC3-125	171.40	182.5	MC3-120	170.98	140	10	---	1975	
MC3-130	172.06	182.5	MC3-125	171.40	289	10	---	1976	
MC3-135	173.13	183.6	MC3-130	172.06	300	10	---	1976	
MC3-140	174.14	183.3	MC3-135	173.13	266	10	---	1976	
MC3-145	175.28	182.7	MC3-140	174.14	260	10	---	1975	
MC3-150	176.84	180.9	MC3-145	175.28	389	10	---	1975	A
MC3-155	178.72	181.9	MC3-150	176.84	470	8	---	---	A, C
MC3-160	179.12	182.8	MC3-155	178.72	100	8	---	---	A
PS-4	---	---	MC3-160	179.12	4,225	8-FM	---	1967	
MC3-205	159.66	175.0	MC3-035	159.50	225	10	---	1954	A
MC3-210	159.71	174.3	MC3-205	159.66	65	10	---	1954	A, C
MC3-215	159.87	174.3	MC3-210	159.71	237	10	---	1954	A, C
MC3-220	160.09	173.5	MC3-215	159.87	307	10	---	1954	
MC3-225	160.52	173.8	MC3-220	160.09	106	10	---	1954	
MC3-230	161.57	175.4	MC3-225	160.52	500	10	---	1954	
MC3-305	170.51	182.5	MC3-060	165.84	260	15	---	---	
MC3-310	171.68	182.6	MC3-305	170.51	260	12	---	---	
MC3-315	174.37	183.2	MC3-310	171.68	265	10	---	---	
MC3-320	171.52	181.1	MC3-065	166.03	260	16	---	---	
MC3-325	172.45	181.9	MC3-320	171.52	260	10	---	---	
MC3-330	173.09	181.0	MC3-070	166.19	260	10	---	---	
MC3-335	174.22	182.3	MC3-330	173.09	260	10	---	---	
MC3-340	171.80	180.0	MC3-075	166.29	265	12	---	---	
MC3-345	173.14	181.3	MC3-340	171.80	260	10	---	---	
MC3-350	172.21	178.9	MC3-080	166.55	260	14	---	---	
MC3-355	173.24	180.0	MC3-350	172.21	260	14	---	---	

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City of Woodburn - - Facilities Plan
Modeled Collection System Inventory

Upstream			Downstream		Pipe				
Manhole (#)	Invert (elev.)	Rim (elev.)	Manhole (#)	Invert (elev.)	Length (ft)	Diameter (in)	Material (type)	Age (year)	Notes
MC3-360	174.32	180.4	MC3-355	173.24	260	12	---	---	
MC3-365	175.85	182.4	MC3-360	174.32	260	10	---	---	
MC3-370	173.26	181.0	MC3-085	166.64	255	10	---	---	
MC3-405	168.65	179.5	MC3-100	167.84	455	12	AC	1960	
MC3-410	169.67	181.6	MC3-405	168.65	530	10	AC	1960	
MC3-415	170.52	182.6	MC3-410	169.67	470	10	AC	1960	
MC3-420	171.30	182.3	MC3-415	170.52	425	10	---	1963	
MC3-425	172.53	184.2	MC3-420	171.30	424	10	---	1963	
MC3-430	173.22	183.9	MC3-425	172.53	220	10	---	1963	
MC3-435	173.49	183.8	MC3-430	173.22	547	10	---	1965	
MC3-440	174.39	183.3	MC3-435	173.49	427	10	---	1965	
MC3-445	174.78	183.6	MC3-440	174.39	225	10	---	1965	
MC3-450	175.28	184.0	MC3-445	174.78	265	10	---	1965	
MC3-455	176.39	183.7	MC3-450	175.28	313	10	---	1965	
PS-10	---	---	MC3-455	176.39	4,440	8-FM	PVC	1992	
MC3-505	174.17	183.0	MC3-435	173.49	85	8	---	1965	
MC3-510	174.73	182.1	MC3-505	174.17	127	8	---	1965	
MC3-515	178.07	183.6	MC3-510	174.73	443	8	---	1965	
MC3-520	178.07	183.6	MC3-515	178.07	500	8	---	1965	
MC3-525	179.46	183.9	MC3-520	178.07	449	8	---	1965	
PS-6	---	---	MC3-525	179.46	535	4-FM	---	1966	
MC3-605	168.48	182.5	MC3-100	167.84	485	12	---	1960	
MC3-610	168.93	182.0	MC3-605	168.48	344	12	---	1960	A, C
MC4-005	156.14	172.2	MC1-080	146.74	316	10	---	1954	
MC4-010	162.66	170.1	MC4-005	156.14	119	10	---	1954	
MC4-015	164.00	175.8	MC4-010	162.66	98	10	---	1954	

City of Woodburn - - Facilities Plan
Modeled Collection System Inventory

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Upstream			Downstream		Pipe				Notes
Manhole (#)	Invert (elev.)	Rim (elev.)	Manhole (#)	Invert (elev.)	Length (ft)	Diameter (in)	Material (type)	Age (year)	
MC4-020	165.45	177.4	MC4-015	164.00	95	10	---	1954	
MC4-025	167.01	179.6	MC4-020	165.45	403	10	---	1954	
MC4-030	168.37	181.3	MC4-025	167.01	402	10	---	1954	
MC4-035	169.83	183.0	MC4-030	168.37	402	10	---	1954	
MC4-040	171.08	182.2	MC4-035	169.83	495	10	---	1954	
MC4-045	172.52	181.2	MC4-040	171.08	478	10	---	1954	
MC4-105	166.47	180.4	MC4-005	156.14	356	10	---	1955	
MC4-110	167.86	180.3	MC4-105	166.47	442	10	---	1955	
MC4-115	169.27	183.0	MC4-110	167.86	445	10	---	1955	
MC4-120	170.41	180.4	MC4-115	169.27	500	10	---	1955	
MC4-125	171.66	181.7	MC4-120	170.41	210	10	---	1955	
MC4-130	172.35	183.4	MC4-125	171.66	275	10	---	1955	
MC4-135	173.08	183.9	MC4-130	172.35	105	10	---	1955	
MC4-140	173.92	183.9	MC4-135	173.08	332	10	CCP	1964	
MC4-145	174.61	184.7	MC4-140	173.92	274	10	CCP	1964	
MC4-205	165.58	171.7	MC1-080	146.74	140	10	---	1970	
MC4-210	166.92	173.8	MC4-205	165.58	267	10	---	1970	
MC4-215	168.42	175.5	MC4-210	166.92	270	10	---	1970	
MC4-220	169.95	177.3	MC4-215	168.42	334	10	---	1970	
MC5-005	155.67	166.0	MC1-105	149.38	25	12	---	1955	
MC5-010	157.03	172.8	MC5-005	155.67	65	14	---	1955	
MC5-015	165.46	175.8	MC5-010	157.03	254	12	---	1955	
MC5-020	166.21	180.4	MC5-015	165.46	367	12	---	1955	
MC5-025	167.10	181.7	MC5-020	166.21	422	12	---	1955	
MC5-030	167.80	182.8	MC5-025	167.10	43	12	---	1955	
MC5-035	168.43	183.0	MC5-030	167.80	501	12	---	1955	

City of Woodburn - - Facilities Plan
Modeled Collection System Inventory

Upstream			Downstream		Pipe				Notes
Manhole (#)	Invert (elev.)	Rim (elev.)	Manhole (#)	Invert (elev.)	Length (ft)	Diameter (in)	Material (type)	Age (year)	
MC5-040	169.31	183.9	MC5-035	168.43	501	12	---	1955	
MC5-045	169.91	184.3	MC5-040	169.31	210	12	---	1955	
MC5-050	170.23	184.6	MC5-045	169.91	119	12	---	1955	
MC5-055	170.96	185.4	MC5-050	170.23	246	12	---	1955	
MC5-060	172.70	184.8	MC5-055	170.96	474	10	---	1955	
MC5-065	174.27	183.8	MC5-060	172.70	500	10	---	1964	
MC5-070	175.18	183.4	MC5-065	174.27	223	10	---	1964	
MC5-105	156.35	164.6	MC1-110	150.43	25	10	---	1970	
MC5-110	165.90	174.5	MC5-105	156.35	205	12	---	1970	
MC5-115	169.51	176.8	MC5-110	165.90	345	10	---	1970	
MC5-205	167.16	178.1	MC5-110	165.90	246	12	---	1970	
MC5-210	169.42	179.4	MC5-205	167.16	238	12	---	1970	
MC5-215	171.71	180.4	MC5-210	169.42	242	10	---	1970	
MC6-005	154.96	167.8	MC1-120	153.76	187	18	---	1985	
MC6-010	156.98	165.7	MC6-005	154.96	406	18	---	1985	
MC6-015	158.06	166.8	MC6-010	156.98	378	18	---	1985	
MC6-020	158.41	167.0	MC6-015	158.06	74	18	---	1985	
MC6-025	158.77	168.5	MC6-020	158.41	214	18	---	1985	
MC6-030	159.16	172.1	MC6-025	158.77	183	18	---	1985	
MC6-035	159.71	173.4	MC6-030	159.16	159	18	---	1985	
MC6-040	160.16	174.0	MC6-035	159.71	178	18	---	1985	
MC6-045	160.85	173.1	MC6-040	160.16	222	18	---	1985	
MC6-050	161.63	173.8	MC6-045	160.85	270	18	---	1985	A, C
MC6-055	161.74	173.8	MC6-050	161.63	40	18	---	1985	A, C
MC6-060	162.62	174.4	MC6-055	161.74	305	18	---	1985	
MC6-105	158.15	168.1	MC6-005	154.96	56	10	---	1968	

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City of Woodburn - - Facilities Plan
Modeled Collection System Inventory

Upstream			Downstream		Pipe				Notes
Manhole (#)	Invert (elev.)	Rim (elev.)	Manhole (#)	Invert (elev.)	Length (ft)	Diameter (in)	Material (type)	Age (year)	
MC6-110	163.58	175.4	MC6-105	158.15	339	10	---	1968	
MC6-115	164.23	176.0	MC6-110	163.58	282	10	---	1968	
MC6-120	165.44	179.2	MC6-115	164.23	424	10	---	1968	
MC6-125	166.14	179.8	MC6-120	165.44	242	10	---	1960	
MC6-130	166.53	180.5	MC6-125	166.14	137	10	---	1985	
MC7-005	134.36	155.4	PS-MC	133.68	33	24	---	1985	B
MC7-010	136.01	155.1	MC7-005	134.36	80	24	---	1985	A, C
MC7-015	140.69	155.1	MC7-010	136.01	226	18	---	1985	A, C
MC7-020	145.40	154.7	MC7-015	140.69	228	18	---	1985	
MC7-025	148.18	158.1	MC7-020	145.40	224	18	---	1985	
MC7-030	151.44	160.6	MC7-025	148.18	311	18	---	1985	
MC7-035	154.07	164.5	MC7-030	151.44	210	18	---	1985	
MC7-040	155.36	165.9	MC7-035	154.07	165	18	---	1985	A, C
MC7-045	157.53	167.3	MC7-040	155.36	276	18	---	1985	
MC7-050	159.73	170.3	MC7-045	157.53	495	18	---	1985	
MC7-055	162.34	172.7	MC7-050	159.73	474	18	---	1985	
MC7-060	163.62	174.0	MC7-055	162.34	257	18	---	1985	
MC7-065	164.73	174.9	MC7-060	163.62	257	18	---	1985	
MC7-070	165.51	178.1	MC7-065	164.73	179	18	---	1985	
MC7-075	166.24	178.4	MC7-070	165.51	215	18	---	1985	
MC7-080	166.96	180.4	MC7-075	166.24	260	18	---	1985	
MC7-085	167.88	181.9	MC7-080	166.96	298	18	---	1985	
MC7-090	168.40	181.2	MC7-085	167.88	213	18	---	1985	
MC7-095	168.65	181.0	MC7-090	168.40	49	18	---	1985	
MC7-100	169.43	181.6	MC7-095	168.65	48	12	---	1960	
MC7-105	169.92	182.0	MC7-100	169.43	478	12	---	1960	

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City of Woodburn - - Facilities Plan
Modeled Collection System Inventory

Upstream			Downstream		Pipe				Notes
Manhole (#)	Invert (elev.)	Rim (elev.)	Manhole (#)	Invert (elev.)	Length (ft)	Diameter (in)	Material (type)	Age (year)	
MC7-110	170.64	182.4	MC7-105	169.92	372	12	---	1960	
MC7-115	171.23	182.8	MC7-110	170.64	372	12	---	1960	
MC7-120	171.46	181.4	MC7-115	171.23	103	12	AC	1960	
MC7-125	171.88	181.8	MC7-120	171.46	177	12	AC	1960	A, C
MC7-130	172.26	181.7	MC7-125	171.88	220	12	AC	1961	
MC7-135	172.57	181.4	MC7-130	172.26	220	12	AC	1961	
MC7-140	172.80	180.6	MC7-135	172.57	225	12	AC	1961	
MC7-145	173.11	181.8	MC7-140	172.80	215	12	AC	1961	
MC7-150	173.80	182.5	MC7-145	173.11	507	12	---	1963	
MC7-155	174.23	182.8	MC7-150	173.80	317	12	---	1963	
MC7-160	174.46	182.3	MC7-155	174.23	185	12	---	1963	
MC7-165	174.96	181.9	MC7-160	174.46	413	12	---	1963	
MC7-170	175.47	182.3	MC7-165	174.96	309	12	---	1963	
MC7-175	175.78	181.4	MC7-170	175.47	220	12	---	1963	
PS-2	---	---	MC7-175	175.78	1,197	4-FM	---	1968	
MC7-180	176.27	181.7	MC7-175	175.78	275	12	---	1963	
MC7-185	176.77	181.5	MC7-180	176.27	328	12	---	1963	
PS-1	---	---	MC7-185	176.77	---	---	---	1963	D
MC7-305	167.61	181.5	MC7-070	165.51	400	10	---	---	
MC7-310	168.81	182.4	MC7-305	167.61	500	10	---	---	
MC7-405	172.87	182.7	MC7-125	171.88	473	10	AC	1960	
MC7-410	173.52	181.1	MC7-405	172.87	420	10	AC	1960	
MC7-415	174.02	180.6	MC7-410	173.52	208	10	AC	1962	
MC7-420	174.58	181.0	MC7-415	174.02	204	10	AC	1962	
MC7-425	174.89	180.7	MC7-420	174.58	185	10	AC	1962	
MC7-505	174.36	181.9	MC7-410	173.52	220	8	---	---	

City of Woodburn - - Facilities Plan
Modeled Collection System Inventory

Upstream			Downstream		Pipe				Notes
Manhole (#)	Invert (elev.)	Rim (elev.)	Manhole (#)	Invert (elev.)	Length (ft)	Diameter (in)	Material (type)	Age (year)	
MC7-510	175.22	182.1	MC7-505	174.36	370	8	---	---	
MC7-515	176.38	182.6	MC7-510	175.22	422	8	---	---	
PS-3	---	---	MC7-515	176.38	---	---	---	1963	D
MC8-005	135.18	154.2	MC7-005	134.36	120	15	---	1987	
MC8-010	136.93	155.7	MC8-005	135.18	500	15	---	1987	
MC8-015	138.03	155.0	MC8-010	136.93	500	15	---	1987	
MC8-020	138.37	154.9	MC8-015	138.03	500	15	---	1987	A, C
MC8-025	138.70	154.8	MC8-020	138.37	500	15	---	1987	
MC8-030	139.40	154.5	MC8-025	138.70	500	15	---	1987	
MC8-035	140.10	154.0	MC8-030	139.40	497	15	---	1987	
MC8-040	140.53	153.5	MC8-035	140.10	350	15	---	1987	
PS-7	---	---	STP	---	3,275	6-FM	---	1972	
MC9-005	162.59	181.3	PS-7	161.68	285	12	---	---	B
MC9-010	163.93	182.6	MC9-005	162.59	420	12	---	---	
MC9-015	165.74	182.4	MC9-010	163.93	269	10	---	---	
MC9-020	166.67	182.1	MC9-015	165.74	278	10	---	---	
SC1-005	160.06	174.4	PS-4	159.89	81	15	---	---	
SC1-010	160.82	176.5	SC1-005	160.06	371	15	---	---	
SC1-015	161.03	177.4	SC1-010	160.82	343	15	---	---	
SC1-020	161.36	177.5	SC1-015	161.03	410	15	---	---	
SC1-025	162.32	177.6	SC1-020	161.36	325	15	---	---	
SC1-030	162.90	179.1	SC1-025	162.32	395	15	---	---	
SC1-035	163.53	179.1	SC1-030	162.90	415	15	---	---	
SC1-040	164.99	180.1	SC1-035	163.53	490	10	---	---	
SC1-045	166.17	176.2	SC1-040	164.99	442	10	---	---	
SC1-105	162.60	179.5	SC1-020	161.36	340	10	---	---	

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City of Woodburn - - Facilities Plan
Modeled Collection System Inventory

Upstream			Downstream		Pipe				Notes
Manhole (#)	Invert (elev.)	Rim (elev.)	Manhole (#)	Invert (elev.)	Length (ft)	Diameter (in)	Material (type)	Age (year)	
SC1-110	163.58	179.3	SC1-105	162.60	340	10	---	---	
SC1-115	164.58	177.3	SC1-110	163.58	340	10	---	---	
SC1-120	165.26	174.9	SC1-115	164.58	251	10	---	---	
SC1-125	166.50	177.2	SC1-120	165.26	370	10	---	---	
SC1-205	161.00	175.6	PS-4	160.63	115	12	---	1969	A, C
SC1-210	162.50	173.4	SC1-205	161.00	460	12	---	1969	A, C
SC1-215	164.46	169.2	SC1-210	162.50	380	12	---	1969	A, C
SC1-220	166.54	169.6	SC1-215	164.46	385	12	---	1969	A, C
SC1-225	168.56	173.9	SC1-220	166.54	375	12	---	1969	A, C
SC1-230	170.59	177.1	SC1-225	168.56	375	12	---	1969	A, C
SC1-235	173.04	178.4	SC1-230	170.59	515	12	---	1969	A, C
SC1-240	174.22	180.3	SC1-235	173.04	340	12	---	1969	A, C
PS-5	---	---	SC1-240	174.22	750	6-FM	---	1969	
SC1-305	160.42	175.0	PS-4	160.24	90	12	---	---	A, B, C
SC1-310	160.62	175.0	SC1-305	160.42	99	12	---	---	A, C

Notes:

A = Upstream Invert Elevation Estimated

B = Downstream Invert Elevation Estimated

C = Upstream Rim Elevation Estimated

D = "Air Pot" type Lift Station

Appendix B

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City of Woodburn - - Facilities Plan
Sub-Basin Land Use

Sub-Basin Number	Current (1990) Area Served					Ultimate (Build-Out) Area Served				
	Single-Family Residential (acres)	Multi-Family Residential (acres)	Commercial/ Industrial (acres)	Open Land (acres)	Total Area (acres)	Single-Family Residential (acres)	Multi-Family Residential (acres)	Commercial/ Industrial (acres)	Open Land (acres)	Total Area (acres)
Basin MC-1										
MC1-030	0.0	0.0	4.6	40.9	45.5	0.0	0.0	4.6	40.9	45.5
MC1-070	28.7	1.5	10.1	4.3	44.6	28.7	1.5	10.1	4.3	44.6
MC1-115	0.0	0.0	0.0	6.8	6.8	0.0	0.0	0.0	6.8	6.8
MC1-145	7.6	0.0	0.0	5.2	12.8	7.6	0.0	0.0	5.2	12.8
MC1-150	15.2	0.0	0.0	8.9	24.1	15.2	0.0	0.0	8.9	24.1
MC1-155	11.6	0.0	0.0	0.2	11.8	11.6	0.0	0.0	0.2	11.8
MC1-170	11.4	0.0	0.0	0.9	12.3	11.4	0.0	0.0	0.9	12.3
MC1-180	11.6	0.0	0.0	1.8	13.4	178.5	63.6	140.4	15.0	397.5
STP	0.0	0.0	0.0	0.0	0.0	218.9	0.0	56.5	0.0	275.4
Basin MC-2										
MC2-025	8.4	3.4	22.1	0.0	33.9	8.4	3.4	22.1	0.0	33.9
MC2-035	11.4	0.0	14.1	0.0	25.5	11.4	0.0	14.1	0.0	25.5
MC2-045	0.0	0.0	22.3	0.0	22.3	0.0	0.0	22.3	0.0	22.3
MC2-050	0.0	0.0	19.1	0.0	19.1	0.0	0.0	43.7	0.0	43.7
MC2-105	0.0	0.0	25.0	0.0	25.0	0.0	0.0	25.0	0.0	25.0
MC2-110	0.0	0.0	13.7	0.0	13.7	0.0	0.0	13.7	0.0	13.7
MC2-115	0.0	0.0	27.1	0.0	27.1	0.0	0.0	27.1	0.0	27.1
MC2-120	0.0	0.0	16.2	0.0	16.2	0.0	0.0	16.2	0.0	16.2
MC2-130	0.0	0.0	24.6	0.0	24.6	0.0	0.0	24.6	0.0	24.6
PS-9	0.0	0.0	86.0	0.0	86.0	0.0	0.0	121.7	0.0	121.7
Basin MC-3										
MC3-020	0.0	0.0	13.9	5.2	19.1	0.0	0.0	13.9	5.2	19.1
MC3-055	1.8	5.9	0.0	0.0	7.7	1.8	5.9	0.0	0.0	7.7
MC3-105	1.8	3.9	0.9	0.0	6.6	1.8	3.9	0.9	0.0	6.6
MC3-110	2.3	3.4	0.0	0.0	5.7	2.3	3.4	0.0	0.0	5.7

City of Woodburn - - Facilities Plan
Sub-Basin Land Use

Sub-Basin Number	Current (1990) Area Served					Ultimate (Build-Out) Area Served				
	Single-Family Residential (acres)	Multi-Family Residential (acres)	Commercial/ Industrial (acres)	Open Land (acres)	Total Area (acres)	Single-Family Residential (acres)	Multi-Family Residential (acres)	Commercial/ Industrial (acres)	Open Land (acres)	Total Area (acres)
MC3-115	4.7	4.0	0.0	0.0	8.7	4.7	4.0	0.0	0.0	8.7
MC3-130	6.9	0.0	0.0	0.0	6.9	6.9	0.0	0.0	0.0	6.9
MC3-135	6.8	0.0	12.6	0.0	19.4	6.8	0.0	12.6	0.0	19.4
MC3-145	1.9	4.3	0.0	0.0	6.2	1.9	4.3	0.0	0.0	6.2
MC3-160	17.1	2.0	24.6	11.4	55.1	17.1	2.0	38.0	11.4	68.5
MC3-205	6.4	3.2	4.5	0.0	14.1	6.4	3.2	4.5	0.0	14.1
MC3-215	10.2	0.0	1.9	0.0	12.1	10.2	0.0	1.9	0.0	12.1
MC3-220	0.0	12.5	0.0	0.0	12.5	0.0	12.5	0.0	0.0	12.5
MC3-230	23.6	5.5	8.0	0.0	37.1	23.6	5.5	8.0	0.0	37.1
MC3-305	0.0	0.0	2.7	0.0	2.7	0.0	0.0	2.7	0.0	2.7
MC3-310	0.0	0.0	2.0	0.0	2.0	0.0	0.0	2.0	0.0	2.0
MC3-315	5.5	0.0	13.2	3.4	22.1	5.5	0.0	13.2	3.4	22.1
MC3-320	0.0	0.0	1.8	0.0	1.8	0.0	0.0	1.8	0.0	1.8
MC3-325	3.2	0.0	10.2	0.0	13.4	3.2	0.0	10.2	0.0	13.4
MC3-330	0.0	0.0	2.5	0.0	2.5	0.0	0.0	2.5	0.0	2.5
MC3-335	0.0	0.0	6.1	0.0	6.1	0.0	0.0	6.1	0.0	6.1
MC3-340	0.0	3.4	0.0	0.0	3.4	0.0	3.4	0.0	0.0	3.4
MC3-345	0.4	0.5	6.4	0.0	7.3	0.4	0.5	6.4	0.0	7.3
MC3-350	0.0	1.8	0.0	0.0	1.8	0.0	1.8	0.0	0.0	1.8
MC3-365	3.2	2.1	0.2	0.0	5.5	3.2	2.1	0.2	0.0	5.5
MC3-370	3.8	3.9	0.0	0.0	7.7	3.8	3.9	0.0	0.0	7.7
MC3-405	21.1	5.3	0.0	0.0	26.4	21.1	5.3	0.0	0.0	26.4
MC3-415	18.0	0.7	0.0	0.0	18.7	18.0	0.7	0.0	0.0	18.7
MC3-430	4.2	3.0	12.0	0.0	19.2	4.2	3.0	12.0	0.0	19.2
MC3-435	2.2	0.5	1.4	0.5	4.6	2.2	0.5	1.4	0.5	4.6
MC3-440	7.7	0.0	0.0	2.5	10.2	7.7	0.0	0.0	2.5	10.2

City of Woodburn - - Facilities Plan
Sub-Basin Land Use

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Sub-Basin Number	Current (1990) Area Served					Ultimate (Build-Out) Area Served				
	Single-Family Residential (acres)	Multi-Family Residential (acres)	Commercial/ Industrial (acres)	Open Land (acres)	Total Area (acres)	Single-Family Residential (acres)	Multi-Family Residential (acres)	Commercial/ Industrial (acres)	Open Land (acres)	Total Area (acres)
MC3-445	5.5	0.0	0.0	3.8	9.3	5.5	0.0	0.0	3.8	9.3
MC3-450	5.3	1.1	0.0	0.0	6.4	5.3	1.1	0.0	0.0	6.4
MC3-455	5.7	0.0	0.0	1.8	7.5	5.7	0.0	0.0	1.8	7.5
MC3-525	7.5	0.0	0.0	3.2	10.7	7.5	0.0	0.0	3.2	10.7
PS-6	15.6	0.0	0.0	6.3	21.9	15.6	0.0	0.0	6.3	21.9
PS-10	0.0	0.0	0.0	0.0	0.0	1108.9	0.0	0.0	0.0	1108.9
Basin MC-4										
MC4-005	2.7	0.0	0.5	3.2	6.4	2.7	0.0	0.5	3.2	6.4
MC4-015	2.9	5.4	0.4	0.9	9.6	2.9	5.4	0.4	0.9	9.6
MC4-020	3.2	4.8	0.0	0.0	8.0	3.2	4.8	0.0	0.0	8.0
MC4-025	1.6	4.5	0.0	0.0	6.1	1.6	4.5	0.0	0.0	6.1
MC4-030	2.2	6.9	0.0	0.0	9.1	2.2	6.9	0.0	0.0	9.1
MC4-035	0.2	1.9	3.6	0.0	5.7	21.6	1.9	12.7	0.0	36.2
MC4-040	0.0	5.9	1.8	0.0	7.7	0.0	5.9	1.8	0.0	7.7
MC4-045	39.7	6.8	12.2	0.0	58.7	39.7	6.8	12.2	0.0	58.7
MC4-105	4.8	0.0	0.0	0.0	4.8	4.8	0.0	0.0	0.0	4.8
MC4-115	17.1	0.0	2.0	0.0	19.1	17.1	0.0	2.0	0.0	19.1
MC4-125	8.4	0.0	5.9	0.0	14.3	8.4	0.0	5.9	0.0	14.3
MC4-140	6.2	0.0	6.1	0.0	12.3	13.0	0.0	10.9	0.0	23.9
MC4-145	6.4	0.0	6.1	0.0	12.5	6.4	0.0	6.1	0.0	12.5
MC4-210	3.4	0.0	0.0	2.3	5.7	3.4	0.0	0.0	2.3	5.7
MC4-215	2.1	5.2	0.0	0.0	7.3	2.1	5.2	0.0	0.0	7.3
MC4-220	2.7	0.0	0.0	6.4	9.1	2.7	0.0	0.0	6.4	9.1
Basin MC-5										
MC5-015	2.0	14.2	0.0	6.6	22.8	2.0	14.2	0.0	6.6	22.8
MC5-020	0.0	6.6	0.0	0.0	6.6	0.0	6.6	0.0	0.0	6.6

City of Woodburn - - Facilities Plan
Sub-Basin Land Use

Sub-Basin Number	Current (1990) Area Served					Ultimate (Build-Out) Area Served				
	Single-Family Residential (acres)	Multi-Family Residential (acres)	Commercial/ Industrial (acres)	Open Land (acres)	Total Area (acres)	Single-Family Residential (acres)	Multi-Family Residential (acres)	Commercial/ Industrial (acres)	Open Land (acres)	Total Area (acres)
MC5-025	5.5	9.1	0.0	0.0	14.6	5.5	9.1	0.0	0.0	14.6
MC5-035	0.0	2.5	4.8	0.0	7.3	0.0	2.5	4.8	0.0	7.3
MC5-040	18.4	0.0	3.0	0.0	21.4	18.4	0.0	3.0	0.0	21.4
MC5-050	4.3	0.0	8.2	0.0	12.5	4.3	0.0	8.2	0.0	12.5
MC5-055	21.0	0.0	17.9	0.0	38.9	21.0	0.0	17.9	0.0	38.9
MC5-070	2.6	0.0	2.0	0.0	4.6	6.7	0.0	19.9	0.0	26.6
MC5-110	0.0	0.7	2.3	0.0	3.0	0.0	0.7	2.3	0.0	3.0
MC5-115	2.3	0.0	14.1	1.6	18.0	2.3	0.0	14.1	1.6	18.0
MC5-215	0.0	1.1	10.1	0.0	11.2	0.0	1.1	10.1	0.0	11.2
Basin MC-6										
MC6-020	8.4	0.0	0.0	5.0	13.4	8.4	0.0	0.0	5.0	13.4
MC6-030	24.1	0.0	0.0	1.4	25.5	24.1	0.0	0.0	1.4	25.5
MC6-060	84.0	0.0	0.0	8.0	92.0	417.8	59.0	0.0	21.2	498.0
MC6-105	2.5	0.0	0.0	4.6	7.1	2.5	0.0	0.0	4.6	7.1
MC6-115	11.6	0.0	0.0	0.9	12.5	11.6	0.0	0.0	0.9	12.5
MC6-130	10.7	0.0	0.0	0.0	10.7	51.7	0.0	0.0	0.0	51.7
Basin MC-7										
MC7-045	0.0	22.8	12.3	8.9	44.0	21.9	22.8	41.6	12.1	98.4
MC7-055	2.0	4.0	14.5	2.5	23.0	2.0	4.0	14.5	2.5	23.0
MC7-075	2.0	0.0	4.6	2.3	8.9	2.0	0.0	4.6	2.3	8.9
MC7-100	0.0	0.7	25.5	3.4	29.6	0.0	0.7	25.5	3.4	29.6
MC7-115	0.0	9.4	2.7	0.0	12.1	0.0	9.4	2.7	0.0	12.1
MC7-120	10.9	0.0	0.0	0.0	10.9	10.9	0.0	0.0	0.0	10.9
MC7-130	11.4	0.0	0.0	0.0	11.4	11.4	0.0	0.0	0.0	11.4
MC7-135	9.3	0.0	0.0	0.0	9.3	9.3	0.0	0.0	0.0	9.3
MC7-140	2.3	0.0	0.0	0.0	2.3	2.3	0.0	0.0	0.0	2.3

City of Woodburn - - Facilities Plan
Sub-Basin Land Use

Sub-Basin Number	Current (1990) Area Served					Ultimate (Build-Out) Area Served				
	Single-Family Residential (acres)	Multi-Family Residential (acres)	Commercial/ Industrial (acres)	Open Land (acres)	Total Area (acres)	Single-Family Residential (acres)	Multi-Family Residential (acres)	Commercial/ Industrial (acres)	Open Land (acres)	Total Area (acres)
MC7-145	14.3	0.0	0.0	1.6	15.9	14.3	0.0	0.0	1.6	15.9
MC7-155	3.9	0.0	0.0	2.5	6.4	3.9	0.0	0.0	2.5	6.4
MC7-165	5.2	0.0	0.0	0.0	5.2	5.2	0.0	0.0	0.0	5.2
MC7-170	3.9	0.0	0.0	0.0	3.9	3.9	0.0	0.0	0.0	3.9
MC7-180	3.6	0.0	0.0	0.0	3.6	3.6	0.0	0.0	0.0	3.6
MC7-310	3.6	0.0	0.0	0.0	3.6	23.2	0.0	2.7	0.0	25.9
MC7-405	4.1	0.0	0.0	0.0	4.1	4.1	0.0	0.0	0.0	4.1
MC7-410	2.0	0.0	0.0	0.0	2.0	2.0	0.0	0.0	0.0	2.0
MC7-415	4.5	0.0	0.0	0.0	4.5	4.5	0.0	0.0	0.0	4.5
MC7-425	17.0	0.0	0.0	0.0	17.0	17.0	0.0	0.0	0.0	17.0
MC7-515	4.3	0.0	0.0	1.8	6.1	4.3	0.0	0.0	1.8	6.1
PS-1	72.1	0.0	0.0	5.6	77.7	72.1	0.0	0.0	51.1	123.2
PS-2	0.0	13.0	0.0	0.0	13.0	0.0	13.0	0.0	0.0	13.0
PS-3	21.2	0.0	11.4	3.6	36.2	21.2	0.0	11.4	12.7	45.3
Basin MC-8										
MC8-020	0.0	0.0	25.1	2.0	27.1	0.0	0.0	25.1	2.0	27.1
MC8-035	0.0	0.0	25.7	5.0	30.7	257.4	0.0	50.6	15.7	323.7
MC8-040	0.0	0.0	16.2	5.9	22.1	0.0	0.0	54.0	5.9	59.9
Basin MC-9										
MC9-010	10.5	7.0	18.9	0.0	36.4	10.5	7.0	18.9	0.0	36.4
MC9-020	14.1	0.0	0.0	0.0	14.1	21.3	0.0	0.0	3.3	24.6
PS-7	28.6	0.0	0.0	0.5	29.1	28.6	0.0	0.0	0.5	29.1
Basin SC-1										
SC1-045	15.0	0.0	0.0	0.0	15.0	15.0	0.0	0.0	0.0	15.0
SC1-110	18.2	0.0	0.0	0.0	18.2	18.2	0.0	0.0	0.0	18.2
SC1-120	10.5	0.0	0.0	0.0	10.5	10.5	0.0	0.0	0.0	10.5

City of Woodburn - - Facilities Plan
Sub-Basin Land Use

Sub-Basin Number	Current (1990) Area Served					Ultimate (Build-Out) Area Served				
	Single-Family Residential (acres)	Multi-Family Residential (acres)	Commercial/ Industrial (acres)	Open Land (acres)	Total Area (acres)	Single-Family Residential (acres)	Multi-Family Residential (acres)	Commercial/ Industrial (acres)	Open Land (acres)	Total Area (acres)
SC1-125	6.3	0.0	0.0	0.0	6.3	6.3	0.0	0.0	0.0	6.3
SC1-215	9.8	0.0	26.9	0.0	36.7	9.8	0.0	26.9	0.0	36.7
SC1-225	7.5	0.0	0.0	0.0	7.5	7.5	0.0	0.0	0.0	7.5
SC1-240	10.2	0.0	0.0	0.0	10.2	10.2	0.0	0.0	0.0	10.2
SC1-310	27.1	0.0	0.0	0.0	27.1	27.1	0.0	0.0	0.0	27.1
PS-5	22.8	0.0	0.0	17.3	40.1	22.8	0.0	0.0	17.3	40.1
Totals	977.5	204.4	694.6	210.9	2087.4	3185.4	327.0	1091.7	309.1	4913.2

Appendix C

City of Woodburn - - Facilities Plan
Sub-Basin Development

Sub-Basin Number	Current (1990) Development						Ultimate (Build-Out) Development		
	Single-Family Residential (units)	Percent Developed (%)	Multi-Family Residential (units)	Percent Developed (%)	Commercial/ Industrial (acres)	Percent Developed (%)	Single-Family Residential (units)	Multi-Family Residential (units)	Commercial/ Industrial (acres)
Basin MC-1									
MC1-030	0.0	0%	0.0	0%	0.0	0%	0.0	0.0	4.6
MC1-070	57.0	17%	31.3	83%	8.0	80%	344.4	37.5	10.1
MC1-115	0.0	0%	0.0	0%	0.0	0%	0.0	0.0	0.0
MC1-145	32.0	35%	0.0	0%	0.0	0%	91.2	0.0	0.0
MC1-150	34.0	19%	0.0	0%	0.0	0%	182.4	0.0	0.0
MC1-155	19.0	14%	0.0	0%	0.0	0%	139.0	0.0	0.0
MC1-170	27.0	20%	0.0	0%	0.0	0%	136.8	0.0	0.0
MC1-180	34.0	2%	0.0	0%	0.0	0%	2,142.0	1,590.0	140.4
STP	0.0	0%	0.0	0%	0.0	0%	2,626.8	0.0	56.5
Basin MC-2									
MC2-025	21.0	21%	81.0	95%	2.7	12%	100.8	85.0	22.1
MC2-035	57.0	42%	0.0	0%	0.9	6%	136.8	0.0	14.1
MC2-045	0.0	0%	0.0	0%	8.2	37%	0.0	0.0	22.3
MC2-050	0.0	0%	0.0	0%	26.8	61%	0.0	0.0	43.7
MC2-105	0.0	0%	0.0	0%	9.2	37%	0.0	0.0	25.0
MC2-110	0.0	0%	0.0	0%	12.6	92%	0.0	0.0	13.7
MC2-115	0.0	0%	0.0	0%	26.6	98%	0.0	0.0	27.1
MC2-120	0.0	0%	0.0	0%	9.9	61%	0.0	0.0	16.2
MC2-130	0.0	0%	0.0	0%	22.6	92%	0.0	0.0	24.6
PS-9	0.0	0%	0.0	0%	44.7	37%	0.0	0.0	121.7
Basin MC-3									
MC3-020	0.0	0%	0.0	0%	0.9	6%	0.0	0.0	13.9
MC3-055	6.0	28%	140.6	95%	0.0	0%	21.6	147.5	0.0
MC3-105	6.0	28%	92.9	95%	0.9	100%	21.6	97.5	0.9
MC3-110	10.0	36%	81.0	95%	0.0	0%	27.6	85.0	0.0

City of Woodburn - - Facilities Plan
Sub-Basin Development

Sub-Basin Number	Current (1990) Development						Ultimate (Build-Out) Development		
	Single-Family Residential (units)	Percent Developed (%)	Multi-Family Residential (units)	Percent Developed (%)	Commercial/ Industrial (acres)	Percent Developed (%)	Single-Family Residential (units)	Multi-Family Residential (units)	Commercial/ Industrial (acres)
MC3-115	17.0	30%	100.0	100%	0.0	0%	56.4	100.0	0.0
MC3-130	3.0	4%	0.0	0%	0.0	0%	82.8	0.0	0.0
MC3-135	5.0	6%	0.0	0%	10.8	86%	81.6	0.0	12.6
MC3-145	2.0	9%	107.5	100%	0.0	0%	22.8	107.5	0.0
MC3-160	50.0	24%	50.0	100%	32.6	86%	205.2	50.0	38.0
MC3-205	10.0	13%	47.7	60%	2.8	61%	76.8	80.0	4.5
MC3-215	45.0	37%	0.0	0%	0.0	0%	122.4	0.0	1.9
MC3-220	0.0	0%	186.2	60%	0.0	0%	0.0	312.5	0.0
MC3-230	66.0	23%	65.5	48%	1.5	18%	283.2	137.5	8.0
MC3-305	0.0	0%	0.0	0%	2.7	100%	0.0	0.0	2.7
MC3-310	0.0	0%	0.0	0%	2.0	100%	0.0	0.0	2.0
MC3-315	8.0	12%	0.0	0%	11.3	86%	66.0	0.0	13.2
MC3-320	0.0	0%	0.0	0%	1.8	100%	0.0	0.0	1.8
MC3-325	7.0	18%	0.0	0%	9.4	92%	38.4	0.0	10.2
MC3-330	0.0	0%	0.0	0%	2.4	98%	0.0	0.0	2.5
MC3-335	0.0	0%	0.0	0%	5.6	92%	0.0	0.0	6.1
MC3-340	0.0	0%	76.0	89%	0.0	0%	0.0	85.0	0.0
MC3-345	0.0	0%	0.0	0%	6.3	98%	4.8	12.5	6.4
MC3-350	0.0	0%	26.8	60%	0.0	0%	0.0	45.0	0.0
MC3-365	6.0	16%	37.5	71%	0.0	0%	38.4	52.5	0.2
MC3-370	12.0	26%	58.1	60%	0.0	0%	45.6	97.5	0.0
MC3-405	45.0	18%	94.7	71%	0.0	0%	253.2	132.5	0.0
MC3-415	25.0	12%	17.5	100%	0.0	0%	216.0	17.5	0.0
MC3-430	5.0	10%	75.0	100%	0.0	0%	50.4	75.0	12.0
MC3-435	10.0	38%	12.5	100%	0.0	0%	26.4	12.5	1.4
MC3-440	45.0	49%	0.0	0%	0.0	0%	92.4	0.0	0.0

City of Woodburn - - Facilities Plan
Sub-Basin Development

Sub-Basin Number	Current (1990) Development						Ultimate (Build-Out) Development		
	Single-Family Residential (units)	Percent Developed (%)	Multi-Family Residential (units)	Percent Developed (%)	Commercial/ Industrial (acres)	Percent Developed (%)	Single-Family Residential (units)	Multi-Family Residential (units)	Commercial/ Industrial (acres)
MC3-445	44.0	67%	0.0	0%	0.0	0%	66.0	0.0	0.0
MC3-450	25.0	39%	27.5	100%	0.0	0%	63.6	27.5	0.0
MC3-455	33.0	48%	0.0	0%	0.0	0%	68.4	0.0	0.0
MC3-525	51.0	57%	0.0	0%	0.0	0%	90.0	0.0	0.0
PS-6	88.0	47%	0.0	0%	0.0	0%	187.2	0.0	0.0
PS-10	0.0	0%	0.0	0%	0.0	0%	13,306.8	0.0	0.0
Basin MC-4									
MC4-005	5.0	15%	0.0	0%	0.3	61%	32.9	0.0	0.5
MC4-015	7.0	20%	8.0	6%	0.4	100%	34.8	135.0	0.4
MC4-020	5.0	13%	7.1	6%	0.0	0%	38.4	120.0	0.0
MC4-025	4.0	21%	33.5	30%	0.0	0%	19.2	112.5	0.0
MC4-030	3.0	11%	20.6	12%	0.0	0%	26.4	172.5	0.0
MC4-035	1.0	0%	0.0	0%	12.7	100%	259.2	47.5	12.7
MC4-040	0.0	0%	35.2	24%	1.8	100%	0.0	147.5	1.8
MC4-045	82.0	17%	141.8	83%	5.2	43%	476.4	170.0	12.2
MC4-105	10.0	17%	0.0	0%	0.0	0%	57.6	0.0	0.0
MC4-115	45.0	22%	0.0	0%	1.6	80%	205.2	0.0	2.0
MC4-125	9.0	9%	0.0	0%	4.3	73%	100.8	0.0	5.9
MC4-140	11.0	7%	0.0	0%	4.0	37%	156.0	0.0	10.9
MC4-145	5.0	7%	0.0	0%	3.0	49%	76.8	0.0	6.1
MC4-210	12.0	29%	0.0	0%	0.0	0%	40.8	0.0	0.0
MC4-215	10.0	40%	15.5	12%	0.0	0%	25.2	130.0	0.0
MC4-220	0.0	0%	0.0	0%	0.0	0%	32.4	0.0	0.0
Basin MC-5									
MC5-015	8.0	33%	211.5	60%	0.0	0%	24.0	355.0	0.0
MC5-020	0.0	0%	98.3	60%	0.0	0%	0.0	165.0	0.0

City of Woodburn - - Facilities Plan
Sub-Basin Development

Sub-Basin Number	Current (1990) Development						Ultimate (Build-Out) Development		
	Single-Family Residential (units)	Percent Developed (%)	Multi-Family Residential (units)	Percent Developed (%)	Commercial/ Industrial (acres)	Percent Developed (%)	Single-Family Residential (units)	Multi-Family Residential (units)	Commercial/ Industrial (acres)
MC5-025	13.0	20%	203.3	89%	0.0	0%	66.0	227.5	0.0
MC5-035	0.0	0%	55.9	89%	0.0	0%	0.0	62.5	4.8
MC5-040	63.0	29%	0.0	0%	1.1	37%	220.8	0.0	3.0
MC5-050	9.0	17%	0.0	0%	4.0	49%	51.6	0.0	8.2
MC5-055	65.0	26%	0.0	0%	11.0	61%	252.0	0.0	17.9
MC5-070	8.0	10%	0.0	0%	18.3	92%	80.4	0.0	19.9
MC5-110	0.0	0%	10.4	60%	1.4	61%	0.0	17.5	2.3
MC5-115	6.0	22%	0.0	0%	12.1	86%	27.6	0.0	14.1
MC5-215	0.0	0%	1.6	6%	8.7	86%	0.0	27.5	10.1
Basin MC-6									
MC6-020	13.0	13%	0.0	0%	0.0	0%	100.8	0.0	0.0
MC6-030	20.0	7%	0.0	0%	0.0	0%	289.2	0.0	0.0
MC6-060	201.0	4%	0.0	0%	0.0	0%	5,013.6	1,475.0	0.0
MC6-105	10.0	33%	0.0	0%	0.0	0%	30.0	0.0	0.0
MC6-115	13.0	9%	0.0	0%	0.0	0%	139.2	0.0	0.0
MC6-130	16.0	3%	0.0	0%	0.0	0%	620.4	0.0	0.0
Basin MC-7									
MC7-045	0.0	0%	0.0	0%	2.5	6%	262.8	570.0	41.6
MC7-055	0.0	0%	11.9	12%	8.9	61%	24.0	100.0	14.5
MC7-075	0.0	0%	0.0	0%	1.7	37%	24.0	0.0	4.6
MC7-100	0.0	0%	0.0	0%	15.6	61%	0.0	17.5	25.5
MC7-115	0.0	0%	210.0	89%	0.8	31%	0.0	235.0	2.7
MC7-120	55.0	42%	0.0	0%	0.0	0%	130.8	0.0	0.0
MC7-130	71.0	52%	0.0	0%	0.0	0%	136.8	0.0	0.0
MC7-135	63.0	56%	0.0	0%	0.0	0%	111.6	0.0	0.0
MC7-140	13.0	47%	0.0	0%	0.0	0%	27.6	0.0	0.0

City of Woodburn - - Facilities Plan
Sub-Basin Development

Sub-Basin Number	Current (1990) Development						Ultimate (Build-Out) Development		
	Single-Family Residential (units)	Percent Developed (%)	Multi-Family Residential (units)	Percent Developed (%)	Commercial/ Industrial (acres)	Percent Developed (%)	Single-Family Residential (units)	Multi-Family Residential (units)	Commercial/ Industrial (acres)
MC7-145	77.0	45%	0.0	0%	0.0	0%	171.6	0.0	0.0
MC7-155	21.0	45%	0.0	0%	0.0	0%	46.8	0.0	0.0
MC7-165	36.0	58%	0.0	0%	0.0	0%	62.4	0.0	0.0
MC7-170	26.0	56%	0.0	0%	0.0	0%	46.8	0.0	0.0
MC7-180	22.0	51%	0.0	0%	0.0	0%	43.2	0.0	0.0
MC7-310	4.0	1%	0.0	0%	0.0	0%	278.4	0.0	2.7
MC7-405	22.0	45%	0.0	0%	0.0	0%	49.2	0.0	0.0
MC7-410	15.0	63%	0.0	0%	0.0	0%	24.0	0.0	0.0
MC7-415	31.0	57%	0.0	0%	0.0	0%	54.0	0.0	0.0
MC7-425	110.0	54%	0.0	0%	0.0	0%	204.0	0.0	0.0
MC7-515	27.0	52%	0.0	0%	0.0	0%	51.6	0.0	0.0
PS-1	338.0	39%	0.0	0%	0.0	0%	865.2	0.0	0.0
PS-2	0.0	0%	325.0	100%	0.0	0%	0.0	325.0	0.0
PS-3	127.0	50%	0.0	0%	0.0	0%	254.4	0.0	11.4
Basin MC-8									
MC8-020	0.0	0%	0.0	0%	6.1	24%	0.0	0.0	25.1
MC8-035	3.0	0%	0.0	0%	6.2	12%	3,088.8	0.0	50.6
MC8-040	0.0	0%	0.0	0%	0.0	0%	0.0	0.0	54.0
Basin MC-9									
MC9-010	20.0	16%	175.0	100%	18.5	98%	126.0	175.0	18.9
MC9-020	32.0	13%	0.0	0%	0.0	0%	255.6	0.0	0.0
PS-7	66.0	19%	0.0	0%	0.0	0%	343.2	0.0	0.0
Basin SC-1									
SC1-045	0.0	0%	0.0	0%	0.0	0%	180.0	0.0	0.0
SC1-110	6.0	3%	0.0	0%	0.0	0%	218.4	0.0	0.0
SC1-120	18.0	14%	0.0	0%	0.0	0%	126.0	0.0	0.0

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City of Woodburn - - Facilities Plan
Sub-Basin Development

Sub-Basin Number	Current (1990) Development						Ultimate (Build-Out) Development		
	Single-Family Residential (units)	Percent Developed (%)	Multi-Family Residential (units)	Percent Developed (%)	Commercial/ Industrial (acres)	Percent Developed (%)	Single-Family Residential (units)	Multi-Family Residential (units)	Commercial/ Industrial (acres)
SC1-125	31.0	41%	0.0	0%	0.0	0%	75.6	0.0	0.0
SC1-215	28.0	24%	0.0	0%	0.0	0%	117.6	0.0	26.9
SC1-225	30.0	33%	0.0	0%	0.0	0%	90.0	0.0	0.0
SC1-240	52.0	42%	0.0	0%	0.0	0%	122.4	0.0	0.0
SC1-310	107.0	33%	0.0	0%	0.0	0%	325.2	0.0	0.0
PS-5	23.0	8%	0.0	0%	0.0	0%	273.6	0.0	0.0
Totals	3,033.0		2,974.0		413.3		38,225.0	8,175.0	1,091.7

Appendix D

City of Woodburn - - Facilities Plan
1997 Sub-Basin Development

Sub-Basin Number	1997 Single-Family Residential			1997 Multi-Family Residential			1997 Commercial/Industrial		
	Planned Development (units)	In-Fill Development (units)	Total Development (units)	Planned Development (units)	In-Fill Development (units)	Total Development (units)	Planned Development (acres)	In-Fill Development (acres)	Total Development (acres)
Basin MC-1									
MC1-030		0	0		0	0		0.2	0.2
MC1-070		2	59		1	32		0.1	8.1
MC1-115		0	0		0	0		0.0	0.0
MC1-145		0	32		0	0		0.0	0.0
MC1-150		1	35		0	0		0.0	0.0
MC1-155		1	20		0	0		0.0	0.0
MC1-170		1	28		0	0		0.0	0.0
MC1-180	109	12	155		185	185		6.0	6.0
STP		15	15		0	0		2.4	2.4
Basin MC-2		0	0		0	0		0.0	0.0
MC2-025		0	21		0	81		0.8	3.5
MC2-035		0	57		0	0		0.6	1.4
MC2-045		0	0		0	0		0.6	8.8
MC2-050		0	0		0	0		0.7	27.5
MC2-105		0	0		0	0		0.7	9.9
MC2-110		0	0		0	0		0.0	12.6
MC2-115		0	0		0	0		0.0	26.6
MC2-120		0	0		0	0		0.3	10.2
MC2-130		0	0		0	0		0.1	22.7
PS-3		0	0		0	0		3.3	48.0
Basin MC-3		0	0		0	0		0.0	0.0
MC3-020		0	0		0	0		0.6	1.4
MC3-055		0	6		1	141		0.0	0.0
MC3-105		0	6		1	93		0.0	0.9
MC3-110		0	10		0	81		0.0	0.0

City of Woodburn - - Facilities Plan
1997 Sub-Basin Development

Sub-Basin Number	1997 Single-Family Residential			1997 Multi-Family Residential			1997 Commercial/Industrial		
	Planned Development (units)	In-Fill Development (units)	Total Development (units)	Planned Development (units)	In-Fill Development (units)	Total Development (units)	Planned Development (acres)	In-Fill Development (acres)	Total Development (acres)
MC3-115		0	17		0	100		0.0	0.0
MC3-130		0	3		0	0		0.0	0.0
MC3-135		0	5		0	0		0.1	10.9
MC3-145		0	2		0	108		0.0	0.0
MC3-160		1	51		0	50		0.2	32.8
MC3-205		0	10		4	51		0.1	2.8
MC3-215		0	45		0	0		0.1	0.1
MC3-220		0	0		15	201		0.0	0.0
MC3-230		1	67		8	74		0.3	1.7
MC3-305		0	0		0	0		0.0	2.7
MC3-310		0	0		0	0		0.0	2.0
MC3-315		0	8		0	0		0.1	11.4
MC3-320		0	0		0	0		0.0	1.8
MC3-325		0	7		0	0		0.0	9.4
MC3-330		0	0		0	0		0.0	2.5
MC3-335		0	0		0	0		0.0	5.6
MC3-340		0	0		1	77		0.0	0.0
MC3-345		0	0		1	1		0.0	6.3
MC3-350		0	0		2	29		0.0	0.0
MC3-365		0	6		2	39		0.0	0.0
MC3-370		0	12		5	63		0.0	0.0
MC3-405		1	46		4	99		0.0	0.0
MC3-415	20	1	46		0	18		0.0	0.0
MC3-430		0	5		0	75		0.5	0.5
MC3-435		0	10		0	13		0.1	0.1
MC3-440		0	45		0	0		0.0	0.0

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City of Woodburn - - Facilities Plan
1997 Sub-Basin Development

Sub-Basin Number	1997 Single-Family Residential			1997 Multi-Family Residential			1997 Commercial/Industrial		
	Planned Development (units)	In-Fill Development (units)	Total Development (units)	Planned Development (units)	In-Fill Development (units)	Total Development (units)	Planned Development (acres)	In-Fill Development (acres)	Total Development (acres)
MC3-445		0	44		0	0		0.0	0.0
MC3-450		0	25		0	28		0.0	0.0
MC3-455		0	33		0	0		0.0	0.0
MC3-525		0	51		0	0		0.0	0.0
PS-6		1	89		0	0	5.0	0.0	5.0
PS-10	37	78	115		0	0	50.0	0.0	50.0
Basin MC-4		0	0		0	0		0.0	0.0
MC4-005		0	5		0	0		0.0	0.3
MC4-015		0	7		15	23		0.0	0.4
MC4-020		0	5		13	20		0.0	0.0
MC4-025		0	4		9	43		0.0	0.0
MC4-030		0	3		18	38		0.0	0.0
MC4-035		2	3		6	6		0.0	12.7
MC4-040		0	0		13	48		0.0	1.8
MC4-045		2	84		3	145		0.3	5.5
MC4-105		0	10		0	0		0.0	0.0
MC4-115		1	46		0	0		0.0	1.6
MC4-125		1	10		0	0		0.1	4.4
MC4-140		1	12		0	0		0.3	4.3
MC4-145		0	5		0	0		0.1	3.1
MC4-210		0	12		0	0		0.0	0.0
MC4-215		0	10		13	29		0.0	0.0
MC4-220		0	0		0	0		0.0	0.0
Basin MC-5		0	0		0	0		0.0	0.0
MC5-015		0	8		17	228		0.0	0.0
MC5-020		0	0		8	106		0.0	0.0

City of Woodburn - - Facilities Plan
1997 Sub-Basin Development

Sub-Basin Number	1997 Single-Family Residential			1997 Multi-Family Residential			1997 Commercial/Industrial		
	Planned Development (units)	In-Fill Development (units)	Total Development (units)	Planned Development (units)	In-Fill Development (units)	Total Development (units)	Planned Development (acres)	In-Fill Development (acres)	Total Development (acres)
MC5-025		0	13		3	206		0.0	0.0
MC5-035		0	0		1	57		0.2	0.2
MC5-040		1	64		0	0		0.1	1.2
MC5-050		0	9		0	0		0.2	4.2
MC5-055		1	66		0	0		0.3	11.3
MC5-070		0	8		0	0		0.1	18.4
MC5-110		0	0		1	11		0.0	1.4
MC5-115		0	6		0	0		0.1	12.2
MC5-215		0	0		3	5		0.1	8.7
Basin MC-6		0	0		0	0		0.0	0.0
MC6-020		1	14		0	0		0.0	0.0
MC6-030		2	22		0	0		0.0	0.0
MC6-060		28	229		171	171		0.0	0.0
MC6-105		0	10		0	0		0.0	0.0
MC6-115		1	14		0	0		0.0	0.0
MC6-130	29	4	49		0	0		0.0	0.0
Basin MC-7		0	0		0	0		0.0	0.0
MC7-045		2	2	135	66	201		1.7	4.2
MC7-055		0	0	45	10	67		0.2	9.1
MC7-075		0	0		0	0		0.1	1.8
MC7-100		0	0		2	2		0.4	16.0
MC7-115		0	0		3	213		0.1	0.9
MC7-120		0	55		0	0		0.0	0.0
MC7-130		0	71		0	0		0.0	0.0
MC7-135		0	63		0	0		0.0	0.0
MC7-140		0	13		0	0		0.0	0.0

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City of Woodburn - - Facilities Plan
1997 Sub-Basin Development

Sub-Basin Number	1997 Single-Family Residential			1997 Multi-Family Residential			1997 Commercial/Industrial		
	Planned Development (units)	In-Fill Development (units)	Total Development (units)	Planned Development (units)	In-Fill Development (units)	Total Development (units)	Planned Development (acres)	In-Fill Development (acres)	Total Development (acres)
MC7-145		1	78		0	0		0.0	0.0
MC7-155		0	21		0	0		0.0	0.0
MC7-165		0	36		0	0		0.0	0.0
MC7-170		0	26		0	0		0.0	0.0
MC7-180		0	22		0	0		0.0	0.0
MC7-310	47	2	53		0	0		0.1	0.1
MC7-405		0	22		0	0		0.0	0.0
MC7-410		0	15		0	0		0.0	0.0
MC7-415		0	31		0	0		0.0	0.0
MC7-425		1	111		0	0		0.0	0.0
MC7-515		0	27		0	0		0.0	0.0
PS-1		3	341		0	0		0.0	0.0
PS-2		0	0		0	325		0.0	0.0
PS-3		1	128		0	0		0.5	0.5
Basin MC-8		0	0		0	0		0.0	0.0
MC8-020		0	0		0	0		0.8	7.0
MC8-035	332	18	353		0	0		1.9	8.1
MC8-040		0	0		0	0		2.3	2.3
Basin MC-9		0	0		0	0		0.0	0.0
MC9-010		1	21		0	175		0.0	18.5
MC9-020	20	1	53		0	0		0.0	0.0
PS-7		2	68		0	0		0.0	0.0
Basin SC-1		0	0		0	0		0.0	0.0
SC1-045		1	1		0	0		0.0	0.0
SC1-110		1	7		0	0		0.0	0.0
SC1-120		1	19		0	0		0.0	0.0

City of Woodburn - - Facilities Plan
1997 Sub-Basin Development

Sub-Basin Number	1997 Single-Family Residential			1997 Multi-Family Residential			1997 Commercial/Industrial		
	Planned Development (units)	In-Fill Development (units)	Total Development (units)	Planned Development (units)	In-Fill Development (units)	Total Development (units)	Planned Development (acres)	In-Fill Development (acres)	Total Development (acres)
SC1-125		0	31		0	0		0.0	0.0
SC1-215		1	29		0	0	25.0	1.1	26.1
SC1-225		0	30		0	0		0.0	0.0
SC1-240		0	52		0	0		0.0	0.0
SC1-310		1	108		0	0		0.0	0.0
PS-5		1	24		0	0		0.0	0.0
Totals	594	205	3,832	180	604	3,758	80.0	29.0	522.3

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Appendix E

City of - ɔurn Facilities Plan
Se 'tary ι er Collection System Analysis
B SC-1
1 Run (PS=FM)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)
PS-4	PS-4	36.0		1.0000	430.871	0.647		0.0	180.14	165.90	(SPECIAL DEVICE AT NODE)			
SC1-005	SC1-005	15.0		0.0021	1.912	0.219	1000	0.3	160.36	174.40	0	6.7	3.2	0.0
SC1-010	SC1-010	15.0		0.0020	1.889	0.219	1000	0.3	161.12	176.50	0	6.7	5.5	0.0
SC1-015	SC1-015	15.0		0.0006	1.033	0.219	1000	0.4	161.44	177.40	0	8.4	6.4	0.0
SC1-020	SC1-020	15.0		0.0008	1.184	0.219	900	0.4	161.75	177.50	0	8.0	10.0	0.0
SC1-025	SC1-025	15.0		0.0030	2.268	0.058	2300	0.1	162.42	177.60	0	3.8	5.8	0.0
SC1-030	SC1-030	15.0		0.0015	1.599	0.058	2300	0.1	163.04	179.10	0	4.3	3.6	0.0
SC1-035	SC1-035	15.0		0.0015	1.626	0.058	2300	0.1	163.67	179.10	0	4.3	0.0	0.0
SC1-040	SC1-040	10.0		0.0030	0.773	0.058	2300	0.2	165.14	180.10	0	3.8	4.1	0.0
SC1-045	SC1-045	10.0		0.0027	0.731	0.058	2300	0.2	166.33	176.20	0	3.9	0.0	0.0
SC1-105	SC1-105	10.0		0.0036	0.855	0.161	900	0.3	162.86	179.50	0	5.3	4.9	0.0
SC1-110	SC1-110	10.0		0.0029	0.760	0.161	900	0.3	163.86	179.30	0	5.6	0.0	0.0
SC1-115	SC1-115	10.0		0.0029	0.768	0.087	900	0.2	164.78	177.30	0	4.4	0.0	0.0
SC1-120	SC1-120	10.0		0.0027	0.737	0.088	900	0.2	165.46	174.90	0	4.5	0.0	0.0
SC1-125	SC1-125	10.0		0.0034	0.819	0.041	900	0.1	166.61	177.20	0	3.2	2.9	0.0
SC1-205	SC1-205	12.0		0.0032	1.305	0.269	900	0.3	161.33	175.60	0	6.6	3.6	0.0
SC1-210	SC1-210	12.0		0.0033	1.314	0.270	900	0.3	162.83	173.40	0	6.6	0.0	0.0
SC1-215	SC1-215	12.0		0.0052	1.653	0.270	900	0.3	164.75	169.20	0	6.1	0.0	0.0
SC1-220	SC1-220	12.0		0.0054	1.692	0.109	900	0.2	166.70	169.60	0	4.3	5.8	0.0
SC1-225	SC1-225	12.0		0.0054	1.689	0.109	900	0.2	168.72	173.90	0	4.3	3.7	0.0
SC1-230	SC1-230	12.0		0.0054	1.693	0.063	900	0.1	170.70	177.10	0	3.5	3.2	0.0
SC1-235	SC1-235	12.0		0.0048	1.587	0.063	900	0.1	173.16	178.40	0	3.6	4.0	0.0
SC1-240	SC1-240	12.0		0.0035	1.356	0.063	900	0.1	174.35	180.30	0	3.8	3.5	0.0
PS-5	PS-5	36.0		1.0000	430.871	0.000		0.0	175.22	153.60	(SPECIAL DEVICE AT NODE)			
SC1-305	SC1-305	12.0		0.0020	1.029	0.160	900	0.3	160.70	175.00	0	6.0	2.9	0.0
SC1-310	SC1-310	12.0		0.0020	1.034	0.161	900	0.3	160.90	175.00	0	6.0	0.0	0.0

City of W burn Facilities Plan
Sanitary S er Collection System Analysis
Basin MC-9
1990 Run (PS=FM)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PC CA
PS-7	PS-7	36.0		1.0000	430.871	0.525		0.0	182.60	161.70	(SPECIAL DEVICE AT NODE)				
MC9-005	MC9-005	12.0		0.0032	1.301	0.379	900	0.4	162.98	181.30	0	7.6	4.2	0.0	29
MC9-010	MC9-010	12.0		0.0032	1.300	0.380	900	0.4	164.32	182.60	0	7.6	7.6	0.0	29
MC9-015	MC9-015	10.0		0.0067	1.161	0.073	900	0.1	165.88	182.40	0	3.5	0.0	0.0	6
MC9-020	MC9-020	10.0		0.0033	0.819	0.073	900	0.2	166.84	182.10	0	4.0	0.0	0.0	8

City of [illegible] burn Facilities Plan
Sanitary S[illegible]er Collection System Analysis
P[illegible]n MC-8
Run (PS=FM)
[illegible] Sanitary and Base Flow = 4.3 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)
MC8-005	MC8-005	15.0		0.0068	3.450	0.366	1000	0.3	135.46	154.20	0	6.5	0.0	0.0
MC8-010	MC8-010	15.0		0.0035	2.469	0.366	1000	0.3	137.27	155.70	0	7.3	7.5	0.0
MC8-015	MC8-015	15.0		0.0022	1.957	0.366	900	0.4	138.42	155.00	0	8.0	0.0	0.0
MC8-020	MC8-020	15.0		0.0007	1.088	0.367	900	0.5	138.89	154.90	0	10.0	0.0	0.0
MC8-025	MC8-025	15.0		0.0007	1.072	0.236	900	0.4	139.12	154.80	0	8.5	0.0	0.0
MC8-030	MC8-030	15.0		0.0014	1.561	0.236	900	0.3	139.75	154.50	0	7.4	0.0	0.0
MC8-035	MC8-035	15.0		0.0014	1.566	0.237	900	0.3	140.45	154.00	0	7.4	0.0	0.0
MC8-040	MC8-040	15.0		0.0012	1.463	0.090	2300	0.2	140.73	153.50	0	5.3	0.0	0.0

City of W burn Facilities Plan
Sanitary .er Collection System Analysis
Basin MC-7
1990 Run (PS=FM)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 12.9 mgd

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PC CA
MC7-005	MC7-005	24.0		0.0206	20.978	2.934	1000	0.5	134.89	155.40	0	11.5	6.2	0.0	13
MC7-010	MC7-010	24.0		0.0206	20.988	2.568	1000	0.5	136.50	155.10	0	10.9	0.0	0.0	12
MC7-015	MC7-015	18.0		0.0207	9.765	2.568	1000	0.6	141.24	155.10	0	10.9	0.0	0.0	26
MC7-020	MC7-020	18.0		0.0207	9.753	2.568	1000	0.6	145.95	154.70	0	10.9	0.0	0.0	26
MC7-025	MC7-025	18.0		0.0124	7.560	2.568	1000	0.6	148.81	158.10	0	12.0	11.6	0.0	33
MC7-030	MC7-030	18.0		0.0105	6.948	2.569	1000	0.7	152.09	160.60	0	12.4	13.0	0.0	36
MC7-035	MC7-035	18.0		0.0125	7.594	2.569	1000	0.6	154.69	164.50	0	12.0	11.1	0.0	33
MC7-040	MC7-040	18.0		0.0078	6.000	2.569	1000	0.7	156.06	165.90	0	13.1	12.9	0.0	42
MC7-045	MC7-045	18.0		0.0079	6.017	2.569	1000	0.7	158.23	167.30	0	13.1	14.2	0.0	42
MC7-050	MC7-050	18.0		0.0044	4.524	2.342	1000	0.8	160.49	170.30	0	14.1	13.3	0.0	51
MC7-055	MC7-055	18.0		0.0055	5.035	2.342	1000	0.7	163.06	172.70	0	13.5	13.7	0.0	46
MC7-060	MC7-060	18.0		0.0050	4.789	2.191	1000	0.7	164.34	174.00	0	13.4	13.4	0.0	45
MC7-065	MC7-065	18.0		0.0043	4.460	2.191	1000	0.7	165.47	174.90	0	13.8	14.3	0.0	49
MC7-070	MC7-070	18.0		0.0044	4.479	2.191	1000	0.7	166.25	178.10	0	13.8	10.6	0.0	48
MC7-075	MC7-075	18.0		0.0034	3.954	2.170	1000	0.8	167.03	178.40	0	14.4	17.0	0.0	54
MC7-080	MC7-080	18.0		0.0028	3.571	2.119	1000	0.8	167.78	180.40	0	14.8	12.2	0.0	59
MC7-085	MC7-085	18.0		0.0031	3.770	2.119	1000	0.8	168.68	181.90	0	14.5	13.2	0.0	56
MC7-090	MC7-090	18.0		0.0024	3.353	2.119	1000	0.8	169.25	181.20	0	15.2	0.0	0.0	63
MC7-095	MC7-095	18.0		0.0051	4.847	2.119	1000	0.7	169.35	181.00	0	13.2	0.0	0.0	43
MC7-100	MC7-100	12.0		0.0162	2.934	2.119	1000	0.6	170.04	181.60	0	10.6	11.2	0.0	72
MC7-105	MC7-105	12.0		0.0010	0.737	1.918	1000	3.8	173.75	182.00	2400	17.2	17.8	14.3	260
MC7-110	MC7-110	12.0		0.0019	1.013	1.918	1000	5.7	176.34	182.40	2400	15.3	17.0	11.5	189
MC7-115	MC7-115	12.0		0.0016	0.917	1.918	1000	7.7	178.92	182.80	2400	15.8	17.0	12.4	209
MC7-120	MC7-120	12.0		0.0022	1.088	1.763	1000	8.1	179.53	181.40	2400	14.4	0.0	10.0	162
MC7-125	MC7-125	12.0		0.0024	1.121	1.687	1000	8.6	180.48	181.80	2400	14.0	14.1	9.3	150
MC7-130	MC7-130	12.0		0.0017	0.957	1.189	1000	8.8	181.06	181.70	2400	13.0	0.0	7.1	124
MC7-135	MC7-135	12.0		0.0014	0.864	1.099	1000	9.0	181.57	181.40	2400	13.1	0.0	7.4	127
MC7-140	MC7-140	12.0		0.0010	0.736	1.025	1000	9.2	182.01	180.60	2400	13.6	0.0	8.5	139
MC7-145	MC7-145	12.0		0.0014	0.874	1.006	1000	9.3	182.42	181.80	2400	12.7	9.8	5.9	115
MC7-150	MC7-150	12.0		0.0014	0.849	0.894	1000	9.4	183.19	182.50	2400	12.2	12.2	4.0	105
MC7-155	MC7-155	12.0		0.0014	0.848	0.894	1000	9.4	183.67	182.80	2400	12.2	13.1	4.1	105
MC7-160	MC7-160	12.0		0.0012	0.812	0.856	900	9.5	183.92	182.30	2400	12.2	0.0	4.0	105
MC7-165	MC7-165	12.0		0.0012	0.801	0.865	900	9.5	184.49	181.90	2400	12.4	0.0	4.7	108
MC7-170	MC7-170	12.0		0.0017	0.935	0.824	900	9.4	184.87	182.30	2100	11.4	12.0	0.0	88
MC7-175	MC7-175	12.0		0.0014	0.864	0.794	900	9.3	185.12	181.40	2100	11.6	0.0	0.0	91
PS-2	PS-2	36.0		1.0000	430.871	0.225		8.3	185.12	168.30	(SPECIAL DEVICE AT NODE)				
MC7-180	MC7-180	12.0		0.0018	0.972	0.570	900	9.0	185.29	181.70	1900	9.8	10.8	0.0	58
MC7-185	MC7-185	12.0		0.0015	0.899	0.542	900	8.7	185.46	181.50	1800	9.9	0.0	0.0	60
PS-1	PS-1	36.0		1.0000	430.871	0.542		7.7	185.46	170.00	(SPECIAL DEVICE AT NODE)				
MC7-305	MC7-305	10.0		0.0053	1.026	0.021	900	0.1	167.67	181.50	0	2.3	2.1	0.0	2
MC7-310	MC7-310	10.0		0.0024	0.693	0.021	900	0.1	168.89	182.40	0	2.7	2.9	0.0	1

City of . ɔurn Facilities Plan
Sanitary S .er Collection System Analysis
r n MC-7
Run (PS=FM)
P k Sanitary and Base Flow = 4.3 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)
MC7-405	MC7-405	10.0		0.0021	0.648	0.502	900	8.2	181.06	182.70	2400	9.1	9.3	0.0
MC7-410	MC7-410	10.0		0.0015	0.557	0.474	900	8.0	181.52	181.10	2400	9.4	0.0	0.0
MC7-415	MC7-415	10.0		0.0024	0.694	0.170	900	7.5	181.55	180.60	2400	5.9	0.0	0.0
MC7-420	MC7-420	10.0		0.0027	0.742	0.136	900	7.0	181.57	181.00	2000	5.3	5.6	0.0
MC7-425	MC7-425	10.0		0.0017	0.579	0.136	900	6.7	181.59	180.70	1900	5.8	0.0	0.0
MC7-505	MC7-505	8.0		0.0038	0.482	0.285	900	7.5	181.81	181.90	2400	6.6	6.6	0.0
MC7-510	MC7-510	8.0		0.0023	0.376	0.286	900	7.1	182.30	182.10	1900	7.2	9.5	0.0
MC7-515	MC7-515	8.0		0.0027	0.409	0.286	900	6.5	182.85	182.60	1700	7.0	8.2	0.0
PS-3	PS-3	36.0		1.0000	430.871	0.241		5.5	182.85	173.10	(SPECIAL DEVICE AT NODE)			

City of W burn Facilities Plan
Sanitary s .er Collection System Analysis
Basin MC-6
1990 Run (PS=FM)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 12.9 mgd

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PC CA
MC6-005	MC6-005	18.0		0.0064	5.436	0.362	900	0.3	155.21	167.80	0	6.5	6.0	0.0	6
MC6-010	MC6-010	18.0		0.0050	4.786	0.300	900	0.2	157.22	165.70	0	6.4	0.0	0.0	6
MC6-015	MC6-015	18.0		0.0029	3.627	0.301	900	0.3	158.35	166.80	0	7.1	7.1	0.0	8
MC6-020	MC6-020	18.0		0.0047	4.667	0.301	900	0.2	158.66	167.00	0	6.4	7.2	0.0	6
MC6-025	MC6-025	18.0		0.0017	2.783	0.276	900	0.3	159.10	168.50	0	7.6	5.8	0.0	9
MC6-030	MC6-030	18.0		0.0021	3.133	0.277	900	0.3	159.46	172.10	0	7.2	4.8	0.0	8
MC6-035	MC6-035	18.0		0.0035	3.991	0.229	900	0.2	159.94	173.40	0	6.2	5.2	0.0	5
MC6-040	MC6-040	18.0		0.0025	3.412	0.229	900	0.3	160.41	174.00	0	6.5	6.2	0.0	6
MC6-045	MC6-045	18.0		0.0031	3.783	0.230	900	0.2	161.09	173.10	0	6.3	0.0	0.0	6
MC6-050	MC6-050	18.0		0.0029	3.647	0.230	900	0.2	161.87	173.80	0	6.4	6.5	0.0	6
MC6-055	MC6-055	18.0		0.0028	3.559	0.230	900	0.2	161.99	173.80	0	6.4	0.0	0.0	6
MC6-060	MC6-060	18.0		0.0029	3.645	0.231	900	0.2	162.86	174.40	0	6.4	6.9	0.0	6
MC6-105	MC6-105	10.0		0.0570	3.378	0.063	900	0.1	158.20	168.10	0	2.2	3.5	0.0	1
MC6-110	MC6-110	10.0		0.0160	1.791	0.046	900	0.1	163.65	175.40	0	2.5	2.4	0.0	2
MC6-115	MC6-115	10.0		0.0023	0.680	0.046	900	0.1	164.37	176.00	0	3.6	3.7	0.0	6
MC6-120	MC6-120	10.0		0.0029	0.756	0.020	900	0.1	165.51	179.20	0	2.6	2.1	0.0	2
MC6-125	MC6-125	10.0		0.0029	0.761	0.020	900	0.1	166.21	179.80	0	2.6	2.7	0.0	2
MC6-130	MC6-130	10.0		0.0028	0.755	0.020	900	0.1	166.60	180.50	0	2.6	2.3	0.0	2

City of _ ɔurn Facilities Plan
Sanitary S .er Collection System Analysis
P ɔ MC-5
Run (PS=FM)
P.. Sanitary and Base Flow = 4.3 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)
MC1-105	MC1-105	36.0		1.0000	430.871	0.908	900	0.0	149.41	165.40	0	3.6	0.0	0.0
MC1-110	MC1-110	36.0		2.0500	616.912	0.203	900	0.0	150.43	165.80	0	1.8	0.0	0.0
MC5-005	MC5-005	12.0		0.2516	11.545	0.908	900	0.2	155.86	166.00	0	4.6	7.2	0.0
MC5-010	MC5-010	14.0		0.0209	5.022	0.908	900	0.4	157.39	172.80	0	7.4	5.5	0.0
MC5-015	MC5-015	12.0		0.0332	4.193	0.909	900	0.3	165.80	175.80	0	6.8	8.2	0.0
MC5-020	MC5-020	12.0		0.0020	1.040	0.730	900	0.6	166.81	180.40	0	10.5	7.5	0.0
MC5-025	MC5-025	12.0		0.0021	1.057	0.660	900	0.6	167.66	181.70	0	10.1	9.4	0.0
MC5-030	MC5-030	12.0		0.0163	2.937	0.506	900	0.3	168.10	182.80	0	6.2	5.7	0.0
MC5-035	MC5-035	12.0		0.0013	0.816	0.509	900	0.6	168.99	183.00	0	10.1	12.5	0.0
MC5-040	MC5-040	12.0		0.0018	0.965	0.458	900	0.5	169.80	183.90	0	9.1	9.0	0.0
MC5-045	MC5-045	12.0		0.0029	1.230	0.348	900	0.4	170.29	184.30	0	7.5	8.1	0.0
MC5-050	MC5-050	12.0		0.0027	1.193	0.348	900	0.4	170.62	184.60	0	7.6	7.7	0.0
MC5-055	MC5-055	12.0		0.0030	1.254	0.285	900	0.3	171.30	185.40	0	6.9	6.8	0.0
MC5-060	MC5-060	10.0		0.0037	0.858	0.080	900	0.2	172.87	184.80	0	4.1	0.0	0.0
MC5-065	MC5-065	10.0		0.0031	0.793	0.081	900	0.2	174.45	183.80	0	4.3	0.0	0.0
MC5-070	MC5-070	10.0		0.0041	0.904	0.082	900	0.2	175.35	183.40	0	4.1	0.0	0.0
MC5-105	MC5-105	10.0		0.2368	6.888	0.203	900	0.1	156.43	164.60	0	2.7	0.0	0.0
MC5-110	MC5-110	12.0		0.0466	4.968	0.203	900	0.1	166.02	174.50	0	3.6	3.6	0.0
MC5-115	MC5-115	10.0		0.0105	1.448	0.110	900	0.2	169.66	176.80	0	3.8	4.1	0.0
MC5-205	MC5-205	12.0		0.0051	1.647	0.070	900	0.1	167.28	178.10	0	3.7	3.0	0.0
MC5-210	MC5-210	12.0		0.0095	2.243	0.070	900	0.1	169.52	179.40	0	3.3	3.6	0.0
MC5-215	MC5-215	10.0		0.0095	1.377	0.070	900	0.1	171.83	180.40	0	3.3	3.8	0.0

City of W burn Facilities Plan
Sanitary . .er Collection System Analysis
Basin MC-4
1990 Run (PS=FM)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 12.9 mgd

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PC CA
MC4-005	MC4-005	10.0		0.0297	2.441	0.921	900	0.4	156.50	172.20	0	6.9	6.7	0.0	37
MC4-010	MC4-010	10.0		0.0548	3.313	0.592	900	0.3	162.91	170.10	0	5.2	0.0	0.0	17
MC4-015	MC4-015	10.0		0.0137	1.655	0.592	900	0.4	164.36	175.80	0	6.8	5.2	0.0	35
MC4-020	MC4-020	10.0		0.0153	1.749	0.551	900	0.3	165.79	177.40	0	6.5	6.4	0.0	31
MC4-025	MC4-025	10.0		0.0039	0.881	0.521	900	0.5	167.46	179.60	0	8.2	7.7	0.0	59
MC4-030	MC4-030	10.0		0.0034	0.823	0.485	900	0.5	168.82	181.30	0	8.2	7.9	0.0	58
MC4-035	MC4-035	10.0		0.0036	0.853	0.443	900	0.4	170.25	183.00	0	7.8	7.6	0.0	51
MC4-040	MC4-040	10.0		0.0025	0.711	0.379	900	0.4	171.51	182.20	0	7.9	0.0	0.0	53
MC4-045	MC4-045	10.0		0.0030	0.777	0.331	900	0.4	172.91	181.20	0	7.3	0.0	0.0	42
MC4-105	MC4-105	10.0		0.0290	2.411	0.307	900	0.2	166.68	180.40	0	4.6	4.8	0.0	12
MC4-110	MC4-110	10.0		0.0031	0.794	0.284	900	0.4	168.22	180.30	0	6.8	0.0	0.0	35
MC4-115	MC4-115	10.0		0.0032	0.797	0.284	900	0.4	169.63	183.00	0	6.8	6.0	0.0	35
MC4-120	MC4-120	10.0		0.0023	0.676	0.193	900	0.3	170.73	180.40	0	6.3	0.0	0.0	28
MC4-125	MC4-125	10.0		0.0060	1.092	0.193	900	0.3	171.91	181.70	0	5.2	5.2	0.0	17
MC4-130	MC4-130	10.0		0.0025	0.709	0.122	900	0.2	172.60	183.40	0	5.2	4.4	0.0	17
MC4-135	MC4-135	10.0		0.0070	1.180	0.123	900	0.2	173.27	183.90	0	4.3	4.6	0.0	10
MC4-140	MC4-140	10.0		0.0025	0.712	0.123	900	0.2	174.17	183.90	0	5.2	0.0	0.0	17
MC4-145	MC4-145	10.0		0.0025	0.710	0.058	900	0.2	174.77	184.70	0	3.9	3.8	0.0	8
MC4-205	MC4-205	10.0		0.1346	5.192	0.095	900	0.1	165.63	171.70	0	2.2	2.5	0.0	1
MC4-210	MC4-210	10.0		0.0050	1.003	0.095	900	0.2	167.10	173.80	0	4.1	3.8	0.0	9
MC4-215	MC4-215	10.0		0.0056	1.055	0.070	900	0.1	168.56	175.50	0	3.6	3.5	0.0	6
MC4-220	MC4-220	10.0		0.0046	0.958	0.032	2300	0.1	170.04	177.30	0	2.8	2.7	0.0	3

City of [illegible] Facilities Plan
Sanitary [illegible] Collection System Analysis
Basin MC-3
1[illegible] Run (PS=FM)
P[illegible] Sanitary and Base Flow = 4.3 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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==========PAGE 9

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	P C
MC3-005	MC3-005	18.0		0.0083	6.195	4.225	900	0.9	145.38	157.70	0	15.6	13.7	0.0	6
MC3-010	MC3-010	12.0		0.0247	3.615	4.225	900	1.3	146.50	159.10	600	12.7	11.3	6.2	11
MC3-015	MC3-015	16.0		0.0626	12.399	4.225	900	0.6	154.05	157.30	0	10.7	0.0	0.0	3
MC3-020	MC3-020	16.0		0.0021	2.283	4.225	900	3.8	158.32	160.00	2400	20.2	16.8	15.1	18
MC3-025	MC3-025	16.0		0.0084	4.546	4.071	900	3.2	160.78	166.50	1400	15.4	13.3	0.0	8
MC3-030	MC3-030	18.0		0.0084	6.206	4.072	900	2.7	161.16	172.90	400	15.4	10.6	0.0	6
MC3-035	MC3-035	14.0		0.0049	2.438	4.072	900	4.6	164.09	169.00	2400	17.0	0.0	12.1	16
MC3-040	MC3-040	14.0		0.0049	2.420	3.258	900	5.4	165.94	169.80	2400	15.7	16.4	9.4	13
MC3-045	MC3-045	12.0		0.0068	1.899	3.258	900	9.9	172.71	174.90	2400	14.7	12.7	10.6	17
MC3-050	MC3-050	12.0		0.0072	1.947	3.258	900	13.9	178.98	180.00	2400	14.6	12.5	10.3	16
MC3-055	MC3-055	16.0		0.0025	2.478	3.258	900	14.0	179.07	180.40	2400	17.7	12.0	10.4	13
MC3-060	MC3-060	16.0		0.0018	2.104	3.124	900	14.8	180.68	180.60	2400	18.6	23.7	12.2	14
MC3-065	MC3-065	16.0		0.0012	1.730	2.857	900	15.2	181.20	180.20	2400	19.3	0.0	13.6	16
MC3-070	MC3-070	16.0		0.0007	1.267	2.692	900	15.7	181.92	180.60	2400	21.2	17.9	16.7	21
MC3-075	MC3-075	15.0		0.0004	0.809	2.592	900	16.7	182.95	177.40	2400	23.2	0.0	20.2	32
MC3-080	MC3-080	16.0		0.0010	1.602	2.450	900	17.0	183.55	179.10	2400	18.8	13.2	12.6	15
MC3-085	MC3-085	16.0		0.0004	0.937	2.359	900	17.5	184.12	179.80	2400	22.6	15.4	18.7	25
MC3-090	MC3-090	16.0		0.0018	2.124	2.299	900	17.5	184.46	178.00	2400	16.5	0.0	6.3	10
MC3-095	MC3-095	16.0		0.0016	1.954	2.299	900	17.7	185.07	177.90	2400	17.0	0.0	8.4	11
MC3-100	MC3-100	16.0		0.0014	1.843	2.299	900	18.0	185.81	180.00	2400	17.4	13.1	9.5	12
MC3-105	MC3-105	10.0		0.0037	0.860	1.486	900	19.9	188.67	182.10	2400	12.3	10.6	8.9	17
MC3-110	MC3-110	10.0		0.0030	0.775	1.410	900	21.7	191.25	183.40	2400	12.5	11.4	9.3	18
MC3-115	MC3-115	10.0		0.0027	0.737	1.341	900	23.3	193.57	182.30	2400	12.5	0.0	9.3	18
MC3-120	MC3-120	10.0		0.0022	0.670	1.253	900	25.0	196.02	181.80	2400	12.6	0.0	9.5	18
MC3-125	MC3-125	10.0		0.0030	0.775	1.253	900	25.7	197.11	182.50	2400	12.0	10.9	8.3	16
MC3-130	MC3-130	10.0		0.0023	0.676	1.253	900	27.3	199.38	182.50	2400	12.6	0.0	9.4	18
MC3-135	MC3-135	10.0		0.0036	0.845	1.226	900	28.5	201.63	183.60	2400	11.5	11.4	7.4	14
MC3-140	MC3-140	10.0		0.0038	0.872	1.108	900	29.1	203.26	183.30	2400	10.9	0.0	6.1	12
MC3-145	MC3-145	10.0		0.0044	0.937	1.108	900	29.6	204.85	182.70	2400	10.6	0.0	5.3	11
MC3-150	MC3-150	10.0		0.0040	0.896	1.030	900	30.1	206.91	180.90	2400	10.5	0.0	4.9	11
MC3-155	MC3-155	8.0		0.0040	0.494	1.030	900	36.4	215.10	181.90	2400	10.5	11.9	8.3	20
MC3-160	MC3-160	8.0		0.0040	0.494	1.030	900	37.7	216.84	182.80	2400	10.5	9.1	8.3	20
MC3-205	MC3-205	10.0		0.0007	0.377	0.814	900	5.2	164.83	175.00	2400	13.3	6.8	10.6	21
MC3-210	MC3-210	10.0		0.0008	0.393	0.667	900	5.3	164.98	174.30	2400	12.2	0.0	8.7	17
MC3-215	MC3-215	10.0		0.0007	0.368	0.667	900	5.6	165.51	174.30	2400	12.5	0.0	9.3	18
MC3-220	MC3-220	10.0		0.0007	0.379	0.549	900	5.9	165.97	173.50	2400	11.5	0.0	7.4	14
MC3-225	MC3-225	10.0		0.0041	0.901	0.364	900	5.5	166.04	173.80	2400	7.1	7.6	0.0	40
MC3-230	MC3-230	10.0		0.0021	0.649	0.365	900	4.8	166.37	175.40	2100	8.1	7.4	0.0	56
MC3-305	MC3-305	15.0		0.0180	5.593	0.267	900	10.2	180.69	182.50	1600	4.8	5.7	0.0	4
MC3-310	MC3-310	12.0		0.0045	1.544	0.238	900	9.0	180.72	182.60	1400	6.0	9.4	0.0	15
MC3-315	MC3-315	10.0		0.0102	1.426	0.218	900	6.4	180.78	183.20	500	4.9	6.6	0.0	15

City of W burn Facilities Plan
S tary . er Collection System Analysis
B 1 MC-3
19.0 Run (PS=FM)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	
MC3-320	MC3-320	16.0		0.0211	7.203	0.166	900	9.7	181.20	181.10	1400	3.9	5.5	0.0	
MC3-325	MC3-325	10.0		0.0036	0.847	0.147	900	8.8	181.23	181.90	1000	5.2	5.3	0.0	
MC3-330	MC3-330	10.0		0.0265	2.306	0.099	900	8.8	181.93	181.00	700	3.1	5.2	0.0	
MC3-335	MC3-335	10.0		0.0043	0.933	0.072	900	7.7	181.94	182.30	500	3.8	3.7	0.0	
MC3-340	MC3-340	12.0		0.0208	3.319	0.142	900	11.2	182.96	180.00	1600	3.7	4.2	0.0	
MC3-345	MC3-345	10.0		0.0052	1.016	0.081	900	9.8	182.96	181.30	1400	3.9	3.9	0.0	
MC3-350	MC3-350	14.0		0.0218	5.122	0.091	900	11.3	183.56	178.90	1600	3.1	0.0	0.0	
MC3-355	MC3-355	14.0		0.0040	2.185	0.066	900	10.3	183.56	180.00	1400	3.8	3.7	0.0	
MC3-360	MC3-360	12.0		0.0042	1.483	0.066	900	9.2	183.56	180.40	600	3.7	4.5	0.0	
MC3-365	MC3-365	10.0		0.0059	1.086	0.066	900	7.7	183.56	182.40	500	3.5	3.3	0.0	
MC3-370	MC3-370	10.0		0.0260	2.281	0.066	900	10.9	184.13	181.00	1600	2.6	3.6	0.0	
MC3-405	MC3-405	12.0		0.0018	0.971	0.818	1000	17.7	186.38	179.50	2400	11.3	0.0	0.0	
MC3-410	MC3-410	10.0		0.0019	0.621	0.656	900	17.8	187.52	181.60	2400	10.2	8.9	3.4	1
MC3-415	MC3-415	10.0		0.0018	0.602	0.656	900	18.0	188.53	182.60	2400	10.3	10.0	4.1	1
MC3-420	MC3-420	10.0		0.0018	0.606	0.560	900	17.9	189.19	182.30	2400	9.7	0.0	0.0	
MC3-425	MC3-425	10.0		0.0029	0.762	0.562	900	17.3	189.86	184.20	2000	8.9	8.2	0.0	
MC3-430	MC3-430	10.0		0.0031	0.793	0.563	900	17.0	190.21	183.90	1900	8.8	0.0	0.0	
MC3-435	MC3-435	10.0		0.0005	0.314	0.447	900	17.3	190.75	183.80	1900	11.4	0.0	7.2	1
MC3-440	MC3-440	10.0		0.0021	0.650	0.218	900	16.5	190.85	183.30	1700	6.6	0.0	0.0	
MC3-445	MC3-445	10.0		0.0017	0.589	0.158	900	16.1	190.88	183.60	1700	6.1	6.4	0.0	
MC3-450	MC3-450	10.0		0.0019	0.615	0.098	900	15.6	190.90	184.00	1700	5.0	5.2	0.0	
MC3-455	MC3-455	10.0		0.0035	0.843	0.048	900	14.5	190.90	183.70	1600	3.4	0.0	0.0	
PS-10	PS-10	36.0		1.0000	430.871	0.000		13.5	190.90	154.00	(SPECIAL DEVICE AT NODE)				
MC3-505	MC3-505	8.0		0.0080	0.698	0.200	900	16.6	190.81	183.00	1800	5.0	0.0	0.0	
MC3-510	MC3-510	8.0		0.0044	0.518	0.201	900	16.2	190.89	182.10	1700	5.6	0.0	0.0	
MC3-515	MC3-515	8.0		0.0075	0.678	0.201	900	13.1	191.19	183.60	1400	5.1	5.9	0.0	
MC3-520	MC3-520	8.0		0.0000	0.035	0.201	900	13.4	191.52	183.60	2400	15.4	0.0	14.4	5
MC3-525	MC3-525	8.0		0.0031	0.433	0.202	900	12.4	191.82	183.90	1400	6.0	8.0	0.0	
PS-6	PS-6	36.0		1.0000	430.871	0.132		11.4	191.82	172.40	(SPECIAL DEVICE AT NODE)				
MC3-605	MC3-605	12.0		0.0013	0.836	0.000	200	17.3	185.81	182.50	2400	0.0	0.0	0.0	
MC3-610	MC3-610	12.0		0.0013	0.832	0.000	0	16.9	185.81	182.00	2400	0.0	0.0	0.0	

City of W burn Facilities Plan
Sanitary s .er Collection System Analysis
Basin MC-2
1990 Run (PS=FM)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 12.9 mgd

03/28/93 11:17

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PC CA
MC2-005	MC2-005	18.0		0.2253	32.211	1.795	1000	0.2	159.77	179.00	0	6.1	6.0	0.0	5
MC2-010	MC2-010	18.0		0.0014	2.502	1.795	1000	0.9	160.75	180.20	0	15.9	12.1	0.0	71
MC2-015	MC2-015	18.0		0.0008	1.879	1.797	900	1.2	161.26	179.80	0	17.7	0.0	0.0	95
MC2-020	MC2-020	18.0		0.0008	1.887	1.805	900	1.2	161.55	179.40	0	17.7	0.0	0.0	95
MC2-025	MC2-025	18.0		0.0011	2.267	1.809	900	1.0	161.64	179.70	0	16.5	16.2	0.0	79
MC2-030	MC2-030	18.0		0.0011	2.237	1.622	900	0.9	162.11	180.00	0	16.0	17.8	0.0	72
MC2-035	MC2-035	18.0		0.0019	2.958	0.421	900	0.4	162.54	181.20	0	8.7	8.3	0.0	14
MC2-040	MC2-040	12.0		0.0013	0.836	0.290	900	0.4	163.22	181.70	0	8.1	8.5	0.0	34
MC2-045	MC2-045	12.0		0.0017	0.941	0.291	900	0.4	164.06	181.70	0	7.7	0.0	0.0	30
MC2-050	MC2-050	12.0		0.0018	0.984	0.173	900	0.3	164.91	182.70	0	6.3	6.2	0.0	17
MC2-105	MC2-105	10.0		0.0052	1.025	1.203	900	2.0	166.18	181.80	500	10.6	11.7	5.2	117
MC2-110	MC2-110	10.0		0.0076	1.232	1.074	900	1.5	167.79	181.10	200	9.5	0.0	0.0	87
MC2-115	MC2-115	10.0		0.0030	0.777	0.978	900	2.3	169.94	180.00	1400	10.9	0.0	6.0	125
MC2-120	MC2-120	10.0		0.0031	0.782	0.776	900	2.3	171.14	180.00	400	10.0	0.0	0.0	99
MC2-125	MC2-125	10.0		0.0031	0.787	0.677	900	2.0	171.94	180.00	200	9.5	0.0	0.0	86
MC2-130	MC2-130	10.0		0.0030	0.771	0.678	900	1.8	172.63	178.00	200	9.5	0.0	0.0	88
PS-9	PS-9	36.0		1.0000	430.871	0.501		0.8	172.63	157.30	(SPECIAL DEVICE AT NODE)				

City of w burn Facilities Plan
Se ary .er Collection System Analysis
B MC-1
1 Run (PS=FM)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 12.9 mgd

03/28/93 11:17

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	P C
PS-MC	PS-MC	36.0		1.0000	430.871	11.965		0.3	182.85	136.60	(SPECIAL DEVICE AT NODE)				
MC1-005	MC1-005	30.0		0.0116	28.492	9.031	1000	1.0	138.54	157.50	0	19.5	9.2	0.0	3
MC1-010	MC1-010	30.0		0.0116	28.488	9.032	1000	1.0	143.21	159.30	0	19.5	23.3	0.0	3
MC1-015	MC1-015	36.0		0.0142	51.284	7.236	1000	0.8	143.21	159.30	0	17.3	0.0	0.0	1
MC1-020	MC1-020	36.0		0.0010	13.939	7.237	1000	1.5	144.08	153.30	0	28.2	0.0	0.0	5
MC1-025	MC1-025	36.0		0.0007	11.620	7.238	1000	1.7	144.43	153.60	0	30.1	27.9	0.0	6
MC1-030	MC1-030	36.0		0.0001	5.117	7.238	1000	3.1	145.96	154.10	1900	41.0	30.3	25.9	14
MC1-035	MC1-035	36.0		0.0021	19.634	7.077	1000	2.4	146.06	155.30	0	24.6	22.5	0.0	3
MC1-040	MC1-040	36.0		0.0012	14.848	7.080	900	2.1	146.06	156.70	0	27.3	21.4	0.0	4
MC1-045	MC1-045	27.0		0.0016	7.973	2.885	1000	1.8	146.06	156.00	0	18.4	0.0	0.0	3
MC1-050	MC1-050	27.0		0.0014	7.486	2.886	1000	1.5	146.06	160.60	0	18.9	11.2	0.0	3
MC1-055	MC1-055	27.0		0.0013	7.306	2.886	1000	1.4	146.06	165.20	0	19.1	9.6	0.0	3
MC1-060	MC1-060	27.0		0.0017	8.235	2.886	1000	1.0	146.12	162.00	0	18.2	0.0	0.0	3
MC1-065	MC1-065	27.0		0.0017	8.236	2.886	1000	1.0	146.83	158.70	0	18.2	0.0	0.0	3
MC1-070	MC1-070	27.0		0.0047	13.783	2.886	1000	0.7	146.90	160.30	0	15.0	10.8	0.0	2
MC1-075	MC1-075	27.0		0.0012	6.825	2.665	1000	1.0	147.48	160.80	0	19.0	17.5	0.0	3
MC1-080	MC1-080	27.0		0.0011	6.756	2.665	1000	1.0	147.75	161.20	0	19.1	17.6	0.0	3
MC1-085	MC1-085	24.0		0.0046	9.955	1.656	1000	0.6	147.90	159.00	0	12.2	0.0	0.0	1
MC1-090	MC1-090	24.0		0.0040	9.222	1.656	1000	0.6	148.83	160.50	0	12.6	11.5	0.0	1
MC1-095	MC1-095	24.0		0.0011	4.921	1.657	900	0.8	149.49	161.40	0	16.0	14.0	0.0	3
MC1-100	MC1-100	24.0		0.0015	5.577	1.661	900	0.8	149.89	161.80	0	15.2	15.6	0.0	2
MC1-105	MC1-105	24.0		0.0008	4.094	1.667	900	0.9	150.29	165.40	0	17.1	10.5	0.0	4
MC1-110	MC1-110	24.0		0.0100	14.614	0.760	900	0.3	150.71	165.80	0	7.9	9.5	0.0	
MC1-115	MC1-115	18.0		0.0110	7.103	0.557	1000	0.3	151.74	164.10	0	6.9	0.0	0.0	
MC1-120	MC1-120	18.0		0.0064	5.431	0.545	900	0.3	154.09	165.90	0	7.6	8.0	0.0	1
MC1-125	MC1-125	15.0		0.0036	2.521	0.184	900	0.2	154.52	166.00	0	5.6	7.7	0.0	
MC1-130	MC1-130	15.0		0.0085	3.841	0.184	900	0.2	155.08	164.10	0	4.8	0.0	0.0	
MC1-135	MC1-135	10.0		0.0507	3.188	0.184	900	0.1	158.49	165.50	0	3.4	4.1	0.0	
MC1-140	MC1-140	8.0		0.0033	0.446	0.184	900	0.3	159.26	165.40	0	5.7	0.0	0.0	4
MC1-145	MC1-145	8.0		0.0094	0.759	0.184	900	0.2	163.47	175.50	0	4.7	4.0	0.0	2
MC1-150	MC1-150	8.0		0.0040	0.492	0.150	900	0.3	164.46	170.00	0	5.1	0.0	0.0	3
MC1-155	MC1-155	8.0		0.0049	0.545	0.095	900	0.2	165.71	174.10	0	4.2	3.4	0.0	1
MC1-160	MC1-160	8.0		0.0067	0.639	0.067	900	0.1	166.23	170.80	0	3.4	0.0	0.0	1
MC1-165	MC1-165	8.0		0.0029	0.422	0.067	900	0.2	166.97	172.20	0	4.0	3.5	0.0	1
MC1-170	MC1-170	8.0		0.0026	0.398	0.067	900	0.2	167.11	170.60	0	4.1	0.0	0.0	1
MC1-175	MC1-175	8.0		0.0029	0.424	0.035	900	0.1	167.72	172.70	0	3.1	2.5	0.0	8
MC1-180	MC1-180	8.0		0.0034	0.452	0.035	900	0.1	169.18	173.30	0	3.1	3.6	0.0	7

City of W burn Facilities Plan
Sanitary . .er Collection System Analysis
Basin SC-1
1997 Run (PS=FM)
Peak Sanitary and Base Flow = 5.7 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 14.3 mgd

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==========PAGE 1

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PC CA
PS-4	PS-4	36.0		1.0000	430.871	0.740		0.0	180.14	165.90	(SPECIAL DEVICE AT NODE)				
SC1-005	SC1-005	15.0		0.0021	1.912	0.221	1000	0.3	160.36	174.40	0	6.7	3.2	0.0	11
SC1-010	SC1-010	15.0		0.0020	1.889	0.221	1000	0.3	161.12	176.50	0	6.7	5.5	0.0	11
SC1-015	SC1-015	15.0		0.0006	1.033	0.221	1000	0.4	161.45	177.40	0	8.4	6.4	0.0	21
SC1-020	SC1-020	15.0		0.0008	1.184	0.222	900	0.4	161.75	177.50	0	8.0	10.0	0.0	18
SC1-025	SC1-025	15.0		0.0030	2.268	0.059	1000	0.1	162.43	177.60	0	3.8	5.8	0.0	2
SC1-030	SC1-030	15.0		0.0015	1.599	0.059	1000	0.1	163.04	179.10	0	4.4	3.6	0.0	3
SC1-035	SC1-035	15.0		0.0015	1.626	0.059	900	0.1	163.67	179.10	0	4.3	0.0	0.0	3
SC1-040	SC1-040	10.0		0.0030	0.773	0.059	900	0.2	165.14	180.10	0	3.8	4.1	0.0	7
SC1-045	SC1-045	10.0		0.0027	0.731	0.059	900	0.2	166.33	176.20	0	3.9	0.0	0.0	8
SC1-105	SC1-105	10.0		0.0036	0.855	0.163	900	0.3	162.86	179.50	0	5.4	4.9	0.0	19
SC1-110	SC1-110	10.0		0.0029	0.760	0.163	900	0.3	163.86	179.30	0	5.6	0.0	0.0	21
SC1-115	SC1-115	10.0		0.0029	0.768	0.088	900	0.2	164.78	177.30	0	4.4	0.0	0.0	11
SC1-120	SC1-120	10.0		0.0027	0.737	0.088	900	0.2	165.46	174.90	0	4.5	0.0	0.0	12
SC1-125	SC1-125	10.0		0.0034	0.819	0.041	900	0.1	166.61	177.20	0	3.2	2.9	0.0	4
SC1-205	SC1-205	12.0		0.0032	1.305	0.359	900	0.4	161.38	175.60	0	7.4	4.0	0.0	27
SC1-210	SC1-210	12.0		0.0033	1.314	0.360	900	0.4	162.88	173.40	0	7.4	0.0	0.0	27
SC1-215	SC1-215	12.0		0.0052	1.653	0.360	900	0.3	164.80	169.20	0	6.8	0.0	0.0	21
SC1-220	SC1-220	12.0		0.0054	1.692	0.109	900	0.2	166.70	169.60	0	4.3	5.9	0.0	6
SC1-225	SC1-225	12.0		0.0054	1.689	0.110	900	0.2	168.73	173.90	0	4.3	3.7	0.0	6
SC1-230	SC1-230	12.0		0.0054	1.693	0.063	900	0.1	170.70	177.10	0	3.5	3.2	0.0	3
SC1-235	SC1-235	12.0		0.0048	1.587	0.063	900	0.1	173.16	178.40	0	3.6	4.0	0.0	4
SC1-240	SC1-240	12.0		0.0035	1.356	0.064	900	0.1	174.35	180.30	0	3.8	3.5	0.0	4
PS-5	PS-5	36.0		1.0000	430.871	0.000		0.0	175.22	153.60	(SPECIAL DEVICE AT NODE)				
SC1-305	SC1-305	12.0		0.0020	1.029	0.161	900	0.3	160.70	175.00	0	6.0	2.9	0.0	1!
SC1-310	SC1-310	12.0		0.0020	1.034	0.162	900	0.3	160.90	175.00	0	6.0	0.0	0.0	1!

City o[illegible] burn Facilities Plan
Sanitary [illegible]er Collection System Analysis
B[illegible]n MC-9
1[illegible] Run (PS=FM)
P[illegible] Sanitary and Base Flow = 5.7 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 14.3 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)
PS-7	PS-7	36.0		1.0000	430.871	0.538		0.0	182.60	161.70	(SPECIAL DEVICE AT NODE)			
MC9-005	MC9-005	12.0		0.0032	1.301	0.389	900	0.4	162.98	181.30	0	7.6	4.3	0.0
MC9-010	MC9-010	12.0		0.0032	1.300	0.391	900	0.4	164.32	182.60	0	7.6	7.7	0.0
MC9-015	MC9-015	10.0		0.0067	1.161	0.084	900	0.1	165.89	182.40	0	3.7	0.0	0.0
MC9-020	MC9-020	10.0		0.0033	0.819	0.084	900	0.2	166.85	182.10	0	4.3	0.0	0.0

City of W burn Facilities Plan
Sanitary . .er Collection System Analysis
Basin MC-8
1997 Run (PS=FM)
Peak Sanitary and Base Flow = 5.7 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 14.3 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PC CA
MC8-005	MC8-005	15.0		0.0068	3.450	0.566	1000	0.4	135.54	154.20	0	7.6	0.0	0.0	16
MC8-010	MC8-010	15.0		0.0035	2.469	0.566	1000	0.4	137.36	155.70	0	8.6	8.9	0.0	22
MC8-015	MC8-015	15.0		0.0022	1.957	0.566	1000	0.5	138.51	155.00	0	9.4	0.0	0.0	28
MC8-020	MC8-020	15.0		0.0007	1.088	0.569	900	0.6	139.01	154.90	0	11.8	0.0	0.0	52
MC8-025	MC8-025	15.0		0.0007	1.072	0.436	900	0.6	139.27	154.80	0	10.7	0.0	0.0	40
MC8-030	MC8-030	15.0		0.0014	1.561	0.439	900	0.5	139.88	154.50	0	9.3	0.0	0.0	28
MC8-035	MC8-035	15.0		0.0014	1.566	0.441	900	0.5	140.58	154.00	0	9.3	0.0	0.0	28
MC8-040	MC8-040	15.0		0.0012	1.463	0.105	900	0.2	140.75	153.50	0	5.6	0.0	0.0	7

City of W burn Facilities Plan
Sanitary . .er Collection System Analysis
P n MC-7
Run (PS=FM)
Peak Sanitary and Base Flow = 5.7 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 14.3 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)
MC7-005	MC7-005	24.0		0.0206	20.978	3.306	1000	0.6	134.93	155.40	0	12.0	6.4	0.0
MC7-010	MC7-010	24.0		0.0206	20.988	2.740	1000	0.5	136.52	155.10	0	11.2	0.0	0.0
MC7-015	MC7-015	18.0		0.0207	9.765	2.740	1000	0.6	141.26	155.10	0	11.2	0.0	0.0
MC7-020	MC7-020	18.0		0.0207	9.753	2.740	1000	0.6	145.97	154.70	0	11.2	0.0	0.0
MC7-025	MC7-025	18.0		0.0124	7.560	2.741	1000	0.6	148.82	158.10	0	12.3	11.8	0.0
MC7-030	MC7-030	18.0		0.0105	6.948	2.741	1000	0.7	152.11	160.60	0	12.7	13.3	0.0
MC7-035	MC7-035	18.0		0.0125	7.594	2.741	1000	0.6	154.71	164.50	0	12.3	11.4	0.0
MC7-040	MC7-040	18.0		0.0078	6.000	2.741	1000	0.7	156.08	165.90	0	13.4	13.2	0.0
MC7-045	MC7-045	18.0		0.0079	6.017	2.741	1000	0.7	158.25	167.30	0	13.4	14.6	0.0
MC7-050	MC7-050	18.0		0.0044	4.524	2.405	1000	0.8	160.50	170.30	0	14.2	13.4	0.0
MC7-055	MC7-055	18.0		0.0055	5.035	2.406	1000	0.7	163.07	172.70	0	13.6	13.9	0.0
MC7-060	MC7-060	18.0		0.0050	4.789	2.230	1000	0.7	164.34	174.00	0	13.5	13.5	0.0
MC7-065	MC7-065	18.0		0.0043	4.460	2.230	1000	0.8	165.48	174.90	0	13.9	14.4	0.0
MC7-070	MC7-070	18.0		0.0044	4.479	2.230	1000	0.7	166.26	178.10	0	13.9	10.6	0.0
MC7-075	MC7-075	18.0		0.0034	3.954	2.186	1000	0.8	167.03	178.40	0	14.4	17.0	0.0
MC7-080	MC7-080	18.0		0.0028	3.571	2.134	1000	0.8	167.78	180.40	0	14.8	12.3	0.0
MC7-085	MC7-085	18.0		0.0031	3.770	2.134	1000	0.8	168.68	181.90	0	14.5	13.3	0.0
MC7-090	MC7-090	18.0		0.0024	3.353	2.134	1000	0.9	169.25	181.20	0	15.2	0.0	0.0
MC7-095	MC7-095	18.0		0.0051	4.847	2.134	1000	0.7	169.36	181.00	0	13.2	0.0	0.0
MC7-100	MC7-100	12.0		0.0162	2.934	2.134	1000	0.6	170.05	181.60	0	10.7	11.2	0.0
MC7-105	MC7-105	12.0		0.0010	0.737	1.930	1000	3.9	173.79	182.00	2400	17.2	17.9	14.4
MC7-110	MC7-110	12.0		0.0019	1.013	1.930	1000	5.8	176.41	182.40	2400	15.3	17.1	11.6
MC7-115	MC7-115	12.0		0.0016	0.917	1.930	1000	7.8	179.02	182.80	2400	15.9	17.1	12.5
MC7-120	MC7-120	12.0		0.0022	1.088	1.772	1000	8.2	179.63	181.40	2400	14.4	0.0	10.1
MC7-125	MC7-125	12.0		0.0024	1.121	1.696	1000	8.7	180.59	181.80	2400	14.0	14.1	9.3
MC7-130	MC7-130	12.0		0.0017	0.957	1.193	1000	8.9	181.18	181.70	2400	13.0	0.0	7.1
MC7-135	MC7-135	12.0		0.0014	0.864	1.102	1000	9.1	181.69	181.40	2400	13.1	0.0	7.4
MC7-140	MC7-140	12.0		0.0010	0.736	1.029	1000	9.3	182.14	180.60	2400	13.6	0.0	8.5
MC7-145	MC7-145	12.0		0.0014	0.874	1.010	1000	9.4	182.55	181.80	2400	12.7	9.8	6.0
MC7-150	MC7-150	12.0		0.0014	0.849	0.897	1000	9.5	183.32	182.50	2400	12.3	12.2	4.1
MC7-155	MC7-155	12.0		0.0014	0.848	0.899	900	9.6	183.80	182.80	2400	12.3	13.1	4.2
MC7-160	MC7-160	12.0		0.0012	0.812	0.861	900	9.6	184.06	182.30	2400	12.3	0.0	4.2
MC7-165	MC7-165	12.0		0.0012	0.801	0.868	900	9.7	184.63	181.90	2400	12.4	0.0	4.7
MC7-170	MC7-170	12.0		0.0017	0.935	0.827	900	9.5	185.02	182.30	2100	11.5	12.0	0.0
MC7-175	MC7-175	12.0		0.0014	0.864	0.797	900	9.5	185.27	181.40	2100	11.6	0.0	0.0
PS-2	PS-2	36.0		1.0000	430.871	0.219		8.5	185.27	168.30	(SPECIAL DEVICE AT NODE)			
MC7-180	MC7-180	12.0		0.0018	0.972	0.579	900	9.2	185.44	181.70	1900	9.9	10.8	0.0
MC7-185	MC7-185	12.0		0.0015	0.899	0.550	900	8.9	185.62	181.50	1800	10.0	0.0	0.0
PS-1	PS-1	36.0		1.0000	430.871	0.550		7.9	185.62	170.00	(SPECIAL DEVICE AT NODE)			
MC7-305	MC7-305	10.0		0.0053	1.026	0.045	900	0.1	167.71	181.50	0	3.1	2.8	0.0
MC7-310	MC7-310	10.0		0.0024	0.693	0.045	900	0.1	168.95	182.40	0	3.6	3.8	0.0

City of ⌊ burn Facilities Plan
Sanitary . ..er Collection System Analysis
Basin MC-7
1997 Run (PS=FM)
Peak Sanitary and Base Flow = 5.7 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 14.3 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT. CAP.
MC7-405	MC7-405	10.0		0.0021	0.648	0.507	900	8.3	181.19	182.70	2400	9.1	9.3	0.0	78.[illegible]
MC7-410	MC7-410	10.0		0.0015	0.557	0.479	900	8.1	181.66	181.10	2400	9.4	0.0	0.0	85.[illegible]
MC7-415	MC7-415	10.0		0.0024	0.694	0.170	900	7.7	181.69	180.60	2400	5.9	0.0	0.0	24.[illegible]
MC7-420	MC7-420	10.0		0.0027	0.742	0.136	900	7.1	181.71	181.00	2000	5.3	5.6	0.0	18.[illegible]
MC7-425	MC7-425	10.0		0.0017	0.579	0.137	900	6.8	181.72	180.70	1900	5.8	0.0	0.0	23.[illegible]
MC7-505	MC7-505	8.0		0.0038	0.482	0.289	900	7.6	181.96	181.90	2400	6.6	6.7	0.0	59.[illegible]
MC7-510	MC7-510	8.0		0.0023	0.376	0.289	900	7.2	182.45	182.10	1900	7.3	9.5	0.0	76.[illegible]
MC7-515	MC7-515	8.0		0.0027	0.409	0.290	900	6.6	183.02	182.60	1700	7.0	8.2	0.0	70.[illegible]
PS-3	PS-3	36.0		1.0000	430.871	0.245		5.6	183.02	173.10	(SPECIAL DEVICE AT NODE)				

City of W burn Facilities Plan
San'tary .er Collection System Analysis
Ba MC-6
19 un (PS=FM)
Peak Sanitary and Base Flow = 5.7 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 14.3 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PI CA
MC6-005	MC6-005	18.0		0.0064	5.436	0.524	900	0.3	155.28	167.80	0	7.5	6.9	0.0	1
MC6-010	MC6-010	18.0		0.0050	4.786	0.440	900	0.3	157.29	165.70	0	7.4	0.0	0.0	1
MC6-015	MC6-015	18.0		0.0029	3.627	0.442	900	0.4	158.43	166.80	0	8.2	8.1	0.0	1
MC6-020	MC6-020	18.0		0.0047	4.667	0.442	900	0.3	158.73	167.00	0	7.4	8.3	0.0	1
MC6-025	MC6-025	18.0		0.0017	2.783	0.417	900	0.4	159.18	168.50	0	8.8	6.8	0.0	1
MC6-030	MC6-030	18.0		0.0021	3.133	0.418	900	0.4	159.55	172.10	0	8.5	5.6	0.0	1
MC6-035	MC6-035	18.0		0.0035	3.991	0.369	900	0.3	160.02	173.40	0	7.4	6.3	0.0	1
MC6-040	MC6-040	18.0		0.0025	3.412	0.370	900	0.3	160.50	174.00	0	7.8	7.4	0.0	1
MC6-045	MC6-045	18.0		0.0031	3.783	0.371	900	0.3	161.17	173.10	0	7.5	0.0	0.0	1
MC6-050	MC6-050	18.0		0.0029	3.647	0.372	900	0.3	161.96	173.80	0	7.6	7.8	0.0	1
MC6-055	MC6-055	18.0		0.0028	3.559	0.372	900	0.3	162.08	173.80	0	7.7	0.0	0.0	1
MC6-060	MC6-060	18.0		0.0029	3.645	0.373	900	0.3	162.95	174.40	0	7.7	8.2	0.0	1
MC6-105	MC6-105	10.0		0.0570	3.378	0.086	900	0.1	158.22	168.10	0	2.5	3.9	0.0	2
MC6-110	MC6-110	10.0		0.0160	1.791	0.069	900	0.1	163.68	175.40	0	2.9	2.8	0.0	3
MC6-115	MC6-115	10.0		0.0023	0.680	0.069	900	0.2	164.41	176.00	0	4.2	4.3	0.0	1
MC6-120	MC6-120	10.0		0.0029	0.756	0.043	900	0.1	165.57	179.20	0	3.4	2.9	0.0	1
MC6-125	MC6-125	10.0		0.0029	0.761	0.043	900	0.1	166.27	179.80	0	3.4	3.5	0.0	1
MC6-130	MC6-130	10.0		0.0028	0.755	0.044	900	0.1	166.66	180.50	0	3.4	3.1	0.0	1

City of W burn Facilities Plan
Sanitary ser Collection System Analysis
Basin MC-5
1997 Run (PS=FM)
Peak Sanitary and Base Flow = 5.7 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 14.3 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT CAP
MC1-105	MC1-105	36.0		1.0000	430.871	0.939	900	0.0	149.41	165.40	0	3.6	0.0	0.0	0.
MC1-110	MC1-110	36.0		2.0500	616.912	0.206	900	0.0	150.43	165.80	0	1.8	0.0	0.0	0.
MC5-005	MC5-005	12.0		0.2516	11.545	0.939	900	0.2	155.86	166.00	0	4.7	7.3	0.0	8.
MC5-010	MC5-010	14.0		0.0209	5.022	0.939	900	0.4	157.39	172.80	0	7.5	5.5	0.0	18.
MC5-015	MC5-015	12.0		0.0332	4.193	0.940	900	0.3	165.80	175.80	0	6.8	8.3	0.0	22.
MC5-020	MC5-020	12.0		0.0020	1.040	0.750	900	0.6	166.82	180.40	0	10.6	7.6	0.0	72.
MC5-025	MC5-025	12.0		0.0021	1.057	0.676	900	0.6	167.67	181.70	0	10.1	9.5	0.0	63.
MC5-030	MC5-030	12.0		0.0163	2.937	0.520	900	0.3	168.10	182.80	0	6.3	5.8	0.0	17.
MC5-035	MC5-035	12.0		0.0013	0.816	0.523	900	0.6	169.00	183.00	0	10.2	12.6	0.0	64.
MC5-040	MC5-040	12.0		0.0018	0.965	0.471	900	0.5	169.80	183.90	0	9.2	9.1	0.0	48.
MC5-045	MC5-045	12.0		0.0029	1.230	0.359	900	0.4	170.30	184.30	0	7.6	8.2	0.0	29.
MC5-050	MC5-050	12.0		0.0027	1.193	0.359	900	0.4	170.62	184.60	0	7.7	7.7	0.0	30.
MC5-055	MC5-055	12.0		0.0030	1.254	0.295	900	0.3	171.31	185.40	0	7.0	6.9	0.0	23.
MC5-060	MC5-060	10.0		0.0037	0.858	0.068	900	0.2	172.88	184.80	0	4.3	0.0	0.0	10.
MC5-065	MC5-065	10.0		0.0031	0.793	0.089	900	0.2	174.46	183.80	0	4.4	0.0	0.0	11.
MC5-070	MC5-070	10.0		0.0041	0.904	0.089	900	0.2	175.36	183.40	0	4.2	0.0	0.0	9.
MC5-105	MC5-105	10.0		0.2368	6.888	0.206	900	0.1	156.43	164.60	0	2.7	0.0	0.0	3.
MC5-110	MC5-110	12.0		0.0466	4.968	0.207	900	0.1	166.02	174.50	0	3.6	3.6	0.0	4.
MC5-115	MC5-115	10.0		0.0105	1.448	0.111	900	0.2	169.66	176.80	0	3.8	4.2	0.0	7.
MC5-205	MC5-205	12.0		0.0051	1.647	0.073	900	0.1	167.29	178.10	0	3.7	3.1	0.0	4.
MC5-210	MC5-210	12.0		0.0095	2.243	0.073	900	0.1	169.52	179.40	0	3.3	3.7	0.0	3.
MC5-215	MC5-215	10.0		0.0095	1.377	0.073	900	0.1	171.83	180.40	0	3.3	3.9	0.0	5.

City of burn Facilities Plan
Sanitary er Collection System Analysis
Ba MC-4
19 un (PS=FM)
Pea sanitary and Base Flow = 5.7 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 14.3 mgd

03/28/93 11:37

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PC CA
MC4-005	MC4-005	10.0		0.0297	2.441	0.979	900	0.4	156.52	172.20	0	7.1	6.9	0.0	40
MC4-010	MC4-010	10.0		0.0548	3.313	0.644	900	0.3	162.92	170.10	0	5.4	0.0	0.0	19
MC4-015	MC4-015	10.0		0.0137	1.655	0.645	900	0.4	164.37	175.80	0	7.0	5.4	0.0	38
MC4-020	MC4-020	10.0		0.0153	1.749	0.595	900	0.3	165.80	177.40	0	6.7	6.6	0.0	34
MC4-025	MC4-025	10.0		0.0039	0.881	0.556	900	0.5	167.48	179.60	0	8.4	7.9	0.0	63
MC4-030	MC4-030	10.0		0.0034	0.823	0.515	900	0.5	168.84	181.30	0	8.4	8.0	0.0	62
MC4-035	MC4-035	10.0		0.0036	0.853	0.462	900	0.4	170.26	183.00	0	7.9	7.7	0.0	54
MC4-040	MC4-040	10.0		0.0025	0.711	0.393	900	0.4	171.52	182.20	0	8.0	0.0	0.0	55
MC4-045	MC4-045	10.0		0.0030	0.777	0.337	900	0.4	172.91	181.20	0	7.3	0.0	0.0	43
MC4-105	MC4-105	10.0		0.0290	2.411	0.313	900	0.2	166.68	180.40	0	4.6	4.9	0.0	12
MC4-110	MC4-110	10.0		0.0031	0.794	0.289	900	0.4	168.22	180.30	0	6.8	0.0	0.0	36
MC4-115	MC4-115	10.0		0.0032	0.797	0.290	900	0.4	169.63	183.00	0	6.8	6.1	0.0	36
MC4-120	MC4-120	10.0		0.0023	0.676	0.198	900	0.3	170.73	180.40	0	6.3	0.0	0.0	29
MC4-125	MC4-125	10.0		0.0060	1.092	0.198	900	0.3	171.91	181.70	0	5.3	5.2	0.0	18
MC4-130	MC4-130	10.0		0.0025	0.709	0.126	900	0.3	172.60	183.40	0	5.2	4.4	0.0	17
MC4-135	MC4-135	10.0		0.0070	1.180	0.126	900	0.2	173.27	183.90	0	4.3	4.6	0.0	10
MC4-140	MC4-140	10.0		0.0025	0.712	0.126	900	0.3	174.17	183.90	0	5.2	0.0	0.0	17
MC4-145	MC4-145	10.0		0.0025	0.710	0.059	900	0.2	174.77	184.70	0	3.9	3.8	0.0	8
MC4-205	MC4-205	10.0		0.1346	5.192	0.103	900	0.1	165.64	171.70	0	2.3	2.6	0.0	1
MC4-210	MC4-210	10.0		0.0050	1.003	0.104	900	0.2	167.11	173.80	0	4.3	3.9	0.0	10
MC4-215	MC4-215	10.0		0.0056	1.055	0.078	900	0.2	168.57	175.50	0	3.8	3.7	0.0	7
MC4-220	MC4-220	10.0		0.0046	0.958	0.032	900	0.1	170.04	177.30	0	2.8	2.7	0.0	3

City of W burn Facilities Plan
Sanitary ; ;er Collection System Analysis
Basin MC-3
1997 Run (PS=FM)
Peak Sanitary and Base Flow = 5.7 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 14.3 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT CAP
MC3-005	MC3-005	18.0		0.0083	6.195	4.405	900	0.9	145.40	157.70	0	15.8	13.9	0.0	71.
MC3-010	MC3-010	12.0		0.0247	3.615	4.405	900	1.4	146.59	159.10	1100	12.9	11.5	6.8	121.
MC3-015	MC3-015	16.0		0.0626	12.399	4.406	900	0.6	154.06	157.30	0	10.9	0.0	0.0	35.
MC3-020	MC3-020	16.0		0.0021	2.283	4.406	900	4.1	158.63	160.00	2400	20.5	17.1	15.6	193.
MC3-025	MC3-025	16.0		0.0084	4.546	4.249	900	3.7	161.31	166.50	1400	15.6	13.6	0.0	93.
MC3-030	MC3-030	18.0		0.0084	6.206	4.250	900	3.3	161.72	172.90	500	15.6	10.7	0.0	68.
MC3-035	MC3-035	14.0		0.0049	2.438	4.250	900	5.4	164.91	169.00	2400	17.2	0.0	12.5	174.
MC3-040	MC3-040	14.0		0.0049	2.420	3.415	900	6.4	166.94	169.80	2400	15.9	16.7	10.0	141.
MC3-045	MC3-045	12.0		0.0068	1.899	3.415	900	11.6	174.38	174.90	2400	15.0	12.9	11.0	179.
MC3-050	MC3-050	12.0		0.0072	1.947	3.415	900	16.2	181.27	180.00	2400	14.8	12.7	10.8	175.
MC3-055	MC3-055	16.0		0.0025	2.478	3.415	900	16.3	181.36	180.40	2400	18.0	12.2	11.1	137.
MC3-060	MC3-060	16.0		0.0018	2.104	3.280	900	17.3	183.14	180.60	2400	18.9	24.1	12.9	155.
MC3-065	MC3-065	16.0		0.0012	1.730	3.012	900	17.7	183.71	180.20	2400	19.7	0.0	14.3	174.
MC3-070	MC3-070	16.0		0.0007	1.267	2.846	900	18.3	184.52	180.60	2400	21.7	18.3	17.4	224.
MC3-075	MC3-075	15.0		0.0004	0.809	2.747	900	19.4	185.67	177.40	2400	23.7	0.0	20.8	339.
MC3-080	MC3-080	16.0		0.0010	1.602	2.603	900	19.8	186.36	179.10	2400	19.2	13.5	13.4	162.
MC3-085	MC3-085	16.0		0.0004	0.937	2.509	900	20.4	187.00	179.80	2400	23.2	15.8	19.4	267.
MC3-090	MC3-090	16.0		0.0018	2.124	2.441	900	20.5	187.39	178.00	2400	16.9	0.0	7.8	114.
MC3-095	MC3-095	16.0		0.0016	1.954	2.441	900	20.7	188.07	177.90	2400	17.4	0.0	9.5	124.
MC3-100	MC3-100	16.0		0.0014	1.843	2.441	900	21.1	188.90	180.00	2400	17.8	13.4	10.5	132.
MC3-105	MC3-105	10.0		0.0037	0.860	1.583	900	23.4	192.15	182.10	2400	12.6	10.9	9.4	184.
MC3-110	MC3-110	10.0		0.0030	0.775	1.507	900	25.5	195.10	183.40	2400	12.8	11.7	9.8	194.
MC3-115	MC3-115	10.0		0.0027	0.737	1.437	900	27.5	197.76	182.30	2400	12.8	0.0	9.8	194.
MC3-120	MC3-120	10.0		0.0022	0.670	1.350	900	29.6	200.60	181.80	2400	13.0	0.0	10.0	201.
MC3-125	MC3-125	10.0		0.0030	0.775	1.350	900	30.5	201.87	182.50	2400	12.3	11.2	8.9	174.
MC3-130	MC3-130	10.0		0.0023	0.676	1.350	900	32.4	204.50	182.50	2400	13.0	0.0	10.0	199.
MC3-135	MC3-135	10.0		0.0036	0.845	1.322	900	34.0	207.11	183.60	2400	11.8	11.8	8.1	156.
MC3-140	MC3-140	10.0		0.0038	0.872	1.203	900	34.9	209.03	183.30	2400	11.3	0.0	7.0	137.
MC3-145	MC3-145	10.0		0.0044	0.937	1.203	900	35.6	210.91	182.70	2400	11.0	0.0	6.2	128.
MC3-150	MC3-150	10.0		0.0040	0.896	1.125	900	36.5	213.37	180.90	2400	10.9	0.0	6.0	125.
MC3-155	MC3-155	8.0		0.0040	0.494	1.125	900	44.4	223.13	181.90	2400	10.9	12.3	8.8	227.
MC3-160	MC3-160	8.0		0.0040	0.494	1.125	900	46.1	225.21	182.80	2400	10.9	9.4	8.8	227.
MC3-205	MC3-205	10.0		0.0007	0.377	0.835	900	6.0	165.69	175.00	2400	13.5	6.8	10.8	221.
MC3-210	MC3-210	10.0		0.0008	0.393	0.685	900	6.1	165.84	174.30	2400	12.3	0.0	9.0	174.
MC3-215	MC3-215	10.0		0.0007	0.368	0.685	900	6.5	166.40	174.30	2400	12.6	0.0	9.5	186.
MC3-220	MC3-220	10.0		0.0007	0.379	0.566	900	6.8	166.89	173.50	2400	11.6	0.0	7.7	149.
MC3-225	MC3-225	10.0		0.0041	0.901	0.372	900	6.4	166.96	173.80	2400	7.2	7.7	0.0	41.
MC3-230	MC3-230	10.0		0.0021	0.649	0.373	900	5.7	167.31	175.40	2100	8.1	7.5	0.0	57.
MC3-305	MC3-305	15.0		0.0180	5.593	0.268	900	12.6	183.15	182.50	1600	4.8	5.7	0.0	4.
MC3-310	MC3-310	12.0		0.0045	1.544	0.239	900	11.5	183.18	182.60	1400	6.0	9.5	0.0	15.
MC3-315	MC3-315	10.0		0.0102	1.426	0.219	900	8.9	183.24	183.20	500	5.0	6.6	0.0	15.

City of w burn Facilities Plan
Se ary s er Collection System Analysis
Bi MC-3
19. Run (PS=FM)
Peak Sanitary and Base Flow = 5.7 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 14.3 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	P C
MC3-320	MC3-320	16.0		0.0211	7.203	0.166	900	12.2	183.72	181.10	1500	3.9	5.5	0.0	
MC3-325	MC3-325	10.0		0.0036	0.847	0.147	900	11.3	183.74	181.90	1400	5.2	5.3	0.0	1
MC3-330	MC3-330	10.0		0.0265	2.306	0.100	900	11.4	184.53	181.00	1400	3.1	5.3	0.0	
MC3-335	MC3-335	10.0		0.0043	0.933	0.072	900	10.3	184.54	182.30	600	3.8	3.7	0.0	
MC3-340	MC3-340	12.0		0.0208	3.319	0.144	900	13.9	185.68	180.00	1600	3.7	4.3	0.0	
MC3-345	MC3-345	10.0		0.0052	1.016	0.082	900	12.6	185.69	181.30	1500	3.9	3.9	0.0	
MC3-350	MC3-350	14.0		0.0218	5.122	0.094	900	14.2	186.36	178.90	1600	3.1	0.0	0.0	
MC3-355	MC3-355	14.0		0.0040	2.185	0.067	900	13.1	186.36	180.00	1500	3.8	3.7	0.0	
MC3-360	MC3-360	12.0		0.0042	1.483	0.067	900	12.0	186.36	180.40	1400	3.8	4.5	0.0	
MC3-365	MC3-365	10.0		0.0059	1.086	0.067	900	10.5	186.37	182.40	600	3.5	3.4	0.0	
MC3-370	MC3-370	10.0		0.0260	2.281	0.068	900	13.8	187.01	181.00	1600	2.7	3.7	0.0	
MC3-405	MC3-405	12.0		0.0018	0.971	0.862	900	20.9	189.54	179.50	2400	11.5	0.0	0.0	8
MC3-410	MC3-410	10.0		0.0019	0.621	0.689	900	21.1	190.79	181.60	2400	10.4	9.1	4.4	11
MC3-415	MC3-415	10.0		0.0018	0.602	0.689	900	21.4	191.91	182.60	2400	10.5	10.2	4.8	11
MC3-420	MC3-420	10.0		0.0018	0.606	0.583	900	21.3	192.63	182.30	2400	9.9	0.0	0.0	9
MC3-425	MC3-425	10.0		0.0029	0.762	0.585	900	20.8	193.35	184.20	2100	9.1	8.3	0.0	7
MC3-430	MC3-430	10.0		0.0031	0.793	0.586	900	20.5	193.73	183.90	1900	8.9	0.0	0.0	7
MC3-435	MC3-435	10.0		0.0005	0.314	0.467	900	20.8	194.32	183.80	1900	11.6	0.0	7.6	14
MC3-440	MC3-440	10.0		0.0021	0.650	0.218	900	20.0	194.43	183.30	1800	6.6	0.0	0.0	3
MC3-445	MC3-445	10.0		0.0017	0.589	0.159	900	19.7	194.45	183.60	1700	6.1	6.4	0.0	2
MC3-450	MC3-450	10.0		0.0019	0.615	0.099	900	19.2	194.47	184.00	1700	5.0	5.3	0.0	1
MC3-455	MC3-455	10.0		0.0035	0.843	0.048	900	18.1	194.47	183.70	1700	3.4	0.0	0.0	
PS-10	PS-10	36.0		1.0000	430.871	0.000		17.1	194.47	154.00	(SPECIAL DEVICE AT NODE)				
MC3-505	MC3-505	8.0		0.0080	0.698	0.219	900	20.2	194.39	183.00	1900	5.2	0.0	0.0	3
MC3-510	MC3-510	8.0		0.0044	0.518	0.219	900	19.8	194.49	182.10	1800	5.8	0.0	0.0	4
MC3-515	MC3-515	8.0		0.0075	0.678	0.219	900	16.8	194.84	183.60	1600	5.2	6.1	0.0	3
MC3-520	MC3-520	8.0		0.0000	0.035	0.219	900	17.2	195.23	183.60	2400	15.9	0.0	14.9	62
MC3-525	MC3-525	8.0		0.0031	0.433	0.220	900	16.1	195.59	183.90	1400	6.2	8.3	0.0	5
PS-6	PS-6	36.0		1.0000	430.871	0.150		15.1	195.59	172.40	(SPECIAL DEVICE AT NODE)				
MC3-605	MC3-605	12.0		0.0013	0.836	0.000	200	20.4	188.90	182.50	2400	0.0	0.0	0.0	
MC3-610	MC3-610	12.0		0.0013	0.832	0.000	0	20.0	188.90	182.00	2400	0.0	0.0	0.0	

City of I 'burn Facilities Plan
Sanitary ‹er Collection System Analysis
Basin MC-2
1997 Run (PS=FM)
Peak Sanitary and Base Flow = 5.7 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 14.3 mgd

03/28/93 11:37

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT CAP
MC2-005	MC2-005	18.0		0.2253	32.211	1.845	1000	0.2	159.78	179.00	0	6.2	6.0	0.0	5.
MC2-010	MC2-010	18.0		0.0014	2.502	1.845	1000	0.9	160.76	180.20	0	16.1	12.2	0.0	73.
MC2-015	MC2-015	18.0		0.0008	1.879	1.846	1000	1.2	161.30	179.80	0	17.9	0.0	0.0	98.
MC2-020	MC2-020	18.0		0.0008	1.887	1.853	900	1.2	161.59	179.40	0	17.9	0.0	0.0	98.
MC2-025	MC2-025	18.0		0.0011	2.267	1.857	900	1.0	161.66	179.70	0	16.7	16.4	0.0	81.
MC2-030	MC2-030	18.0		0.0011	2.237	1.664	900	0.9	162.13	180.00	0	16.1	18.0	0.0	74.
MC2-035	MC2-035	18.0		0.0019	2.958	0.434	900	0.4	162.55	181.20	0	8.8	8.4	0.0	14.
MC2-040	MC2-040	12.0		0.0013	0.836	0.298	900	0.4	163.23	181.70	0	8.1	8.6	0.0	35.
MC2-045	MC2-045	12.0		0.0017	0.941	0.300	900	0.4	164.06	181.70	0	7.8	0.0	0.0	31.
MC2-050	MC2-050	12.0		0.0018	0.984	0.177	900	0.3	164.91	182.70	0	6.3	6.3	0.0	18.
MC2-105	MC2-105	10.0		0.0052	1.025	1.233	900	2.3	166.40	181.80	600	10.7	11.8	5.5	120.
MC2-110	MC2-110	10.0		0.0076	1.232	1.100	900	1.8	168.09	181.10	300	9.6	0.0	0.0	89.
MC2-115	MC2-115	10.0		0.0030	0.777	1.003	900	2.7	170.35	180.00	1500	11.0	0.0	6.3	129.
MC2-120	MC2-120	10.0		0.0031	0.782	0.806	900	2.8	171.65	180.00	500	10.1	0.0	2.7	103.
MC2-125	MC2-125	10.0		0.0031	0.787	0.706	900	2.6	172.52	180.00	300	9.6	0.0	0.0	89.
MC2-130	MC2-130	10.0		0.0030	0.771	0.707	900	2.5	173.27	178.00	300	9.7	0.0	0.0	91.
PS-9	PS-9	36.0		1.0000	430.871	0.529		1.5	173.27	157.30	(SPECIAL DEVICE AT NODE)				

City of W burn Facilities Plan
S itary ser Collection System Analysis
  n MC-1
  / Run (PS=FM)
Peak Sanitary and Base Flow = 5.7 mgd
Peak I/I = 8.6 mgd (Distributed)
Total Peak Flow = 14.3 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)
PS-MC	PS-MC	36.0		1.0000	430.871	13.018		0.3	182.87	136.60	(SPECIAL DEVICE AT NODE)			
MC1-005	MC1-005	30.0		0.0116	28.492	9.712	1000	1.0	138.57	157.50	0	20.0	9.4	0.0
MC1-010	MC1-010	30.0		0.0116	28.488	9.712	1000	1.0	143.24	159.30	0	20.0	24.0	0.0
MC1-015	MC1-015	36.0		0.0142	51.284	7.867	1000	0.9	143.24	159.30	0	17.8	0.0	0.0
MC1-020	MC1-020	36.0		0.0010	13.939	7.868	1000	1.6	144.14	153.30	0	29.1	0.0	0.0
MC1-025	MC1-025	36.0		0.0007	11.620	7.869	1000	1.8	144.52	153.60	0	31.1	28.8	0.0
MC1-030	MC1-030	36.0		0.0001	5.117	7.869	1000	3.1	145.99	154.10	2000	42.3	31.3	28.5
MC1-035	MC1-035	36.0		0.0021	19.634	7.706	1000	2.5	146.11	155.30	0	25.4	23.3	0.0
MC1-040	MC1-040	36.0		0.0012	14.848	7.710	900	2.1	146.11	156.70	0	28.2	22.1	0.0
MC1-045	MC1-045	27.0		0.0016	7.973	3.352	1000	1.9	146.11	156.00	0	19.5	0.0	0.0
MC1-050	MC1-050	27.0		0.0014	7.486	3.352	1000	1.6	146.11	160.60	0	20.0	11.8	0.0
MC1-055	MC1-055	27.0		0.0013	7.306	3.352	1000	1.5	146.11	165.20	0	20.2	10.2	0.0
MC1-060	MC1-060	27.0		0.0017	8.235	3.353	1000	1.0	146.19	162.00	0	19.3	0.0	0.0
MC1-065	MC1-065	27.0		0.0017	8.236	3.353	1000	1.0	146.90	158.70	0	19.3	0.0	0.0
MC1-070	MC1-070	27.0		0.0047	13.783	3.353	1000	0.8	146.96	160.30	0	15.9	11.5	0.0
MC1-075	MC1-075	27.0		0.0012	6.825	3.130	1000	1.1	147.56	160.80	0	20.2	18.5	0.0
MC1-080	MC1-080	27.0		0.0011	6.756	3.130	1000	1.1	147.82	161.20	0	20.2	18.7	0.0
MC1-085	MC1-085	24.0		0.0046	9.955	2.056	1000	0.7	147.97	159.00	0	13.3	0.0	0.0
MC1-090	MC1-090	24.0		0.0040	9.222	2.056	1000	0.7	148.90	160.50	0	13.7	12.4	0.0
MC1-095	MC1-095	24.0		0.0011	4.921	2.057	1000	0.9	149.58	161.40	0	17.3	15.1	0.0
MC1-100	MC1-100	24.0		0.0015	5.577	2.060	900	0.9	149.98	161.80	0	16.5	16.9	0.0
MC1-105	MC1-105	24.0		0.0008	4.094	2.069	900	1.0	150.39	165.40	0	18.6	11.4	0.0
MC1-110	MC1-110	24.0		0.0100	14.614	1.131	900	0.4	150.80	165.80	0	9.2	11.0	0.0
MC1-115	MC1-115	18.0		0.0110	7.103	0.925	900	0.4	151.84	164.10	0	8.4	0.0	0.0
MC1-120	MC1-120	18.0		0.0064	5.431	0.915	900	0.4	154.20	165.90	0	9.2	9.7	0.0
MC1-125	MC1-125	15.0		0.0036	2.521	0.392	900	0.4	154.65	166.00	0	7.5	10.2	0.0
MC1-130	MC1-130	15.0		0.0085	3.841	0.392	900	0.3	155.19	164.10	0	6.4	0.0	0.0
MC1-135	MC1-135	10.0		0.0507	3.188	0.392	900	0.2	158.57	165.50	0	4.6	5.4	0.0
MC1-140	MC1-140	8.0		0.0033	0.446	0.394	900	0.5	159.44	165.40	0	7.6	0.0	0.0
MC1-145	MC1-145	8.0		0.0094	0.759	0.395	900	0.3	163.57	175.50	0	6.3	5.3	0.0
MC1-150	MC1-150	8.0		0.0040	0.492	0.361	900	0.4	164.61	170.00	0	7.1	0.0	0.0
MC1-155	MC1-155	8.0		0.0049	0.545	0.307	900	0.4	165.86	174.10	0	6.5	5.2	0.0
MC1-160	MC1-160	8.0		0.0067	0.639	0.278	900	0.3	166.39	170.80	0	5.9	0.0	0.0
MC1-165	MC1-165	8.0		0.0029	0.422	0.279	900	0.4	167.17	172.20	0	6.9	6.0	0.0
MC1-170	MC1-170	8.0		0.0026	0.398	0.280	900	0.4	167.31	170.60	0	7.0	0.0	0.0
MC1-175	MC1-175	8.0		0.0029	0.424	0.248	900	0.4	167.95	172.70	0	6.5	5.3	0.0
MC1-180	MC1-180	8.0		0.0034	0.452	0.250	900	0.4	169.41	173.30	0	6.4	7.6	0.0

City of W burn Facilities Plan
Sanitary - er Collection System Analysis
Basin SC-1
Ultimate Run (PS=FM)
Peak Sanitary and Base Flow = 25.9 mgd
Peak I/I = 12.8 mgd (Distributed)
Total Peak Flow = 38.7 mgd

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==========PAGE 1

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT CAP
PS-4	PS-4	36.0		1.0000	430.871	1.187		0.0	180.15	165.90	(SPECIAL DEVICE AT NODE)				
SC1-005	SC1-005	15.0		0.0021	1.912	0.464	1000	0.4	160.50	174.40	0	8.8	4.2	0.0	24.
SC1-010	SC1-010	15.0		0.0020	1.889	0.464	1000	0.4	161.27	176.50	0	8.9	7.3	0.0	24.
SC1-015	SC1-015	15.0		0.0006	1.033	0.465	900	0.6	161.62	177.40	0	11.1	8.5	0.0	45.
SC1-020	SC1-020	15.0		0.0008	1.184	0.469	900	0.6	161.92	177.50	0	10.6	13.3	0.0	39.
SC1-025	SC1-025	15.0		0.0030	2.268	0.139	1000	0.2	162.52	177.60	0	5.3	8.1	0.0	6.
SC1-030	SC1-030	15.0		0.0015	1.599	0.140	900	0.3	163.15	179.10	0	6.0	5.0	0.0	8.
SC1-035	SC1-035	15.0		0.0015	1.626	0.142	900	0.3	163.78	179.10	0	6.0	0.0	0.0	8.
SC1-040	SC1-040	10.0		0.0030	0.773	0.143	900	0.3	165.25	180.10	0	5.3	5.7	0.0	18.
SC1-045	SC1-045	10.0		0.0027	0.731	0.143	900	0.3	166.44	176.20	0	5.4	0.0	0.0	19.
SC1-105	SC1-105	10.0		0.0036	0.855	0.332	900	0.4	162.97	179.50	0	7.0	6.4	0.0	38.
SC1-110	SC1-110	10.0		0.0029	0.760	0.333	900	0.4	163.97	179.30	0	7.3	0.0	0.0	43.
SC1-115	SC1-115	10.0		0.0029	0.768	0.159	900	0.3	164.85	177.30	0	5.5	0.0	0.0	20.
SC1-120	SC1-120	10.0		0.0027	0.737	0.160	900	0.3	165.54	174.90	0	5.6	0.0	0.0	21.
SC1-125	SC1-125	10.0		0.0034	0.819	0.062	900	0.2	166.65	177.20	0	3.8	3.4	0.0	7.
SC1-205	SC1-205	12.0		0.0032	1.305	0.465	900	0.4	161.43	175.60	0	8.2	4.4	0.0	35.
SC1-210	SC1-210	12.0		0.0033	1.314	0.467	900	0.4	162.93	173.40	0	8.1	0.0	0.0	35.
SC1-215	SC1-215	12.0		0.0052	1.653	0.468	900	0.4	164.84	169.20	0	7.5	0.0	0.0	28.
SC1-220	SC1-220	12.0		0.0054	1.692	0.169	900	0.2	166.76	169.60	0	5.1	6.9	0.0	10.
SC1-225	SC1-225	12.0		0.0054	1.689	0.170	900	0.2	168.78	173.90	0	5.1	4.4	0.0	10.
SC1-230	SC1-230	12.0		0.0054	1.693	0.096	900	0.2	170.74	177.10	0	4.1	3.7	0.0	5.
SC1-235	SC1-235	12.0		0.0048	1.587	0.096	900	0.2	173.20	178.40	0	4.2	4.7	0.0	6.
SC1-240	SC1-240	12.0		0.0035	1.356	0.097	900	0.2	174.40	180.30	0	4.5	4.1	0.0	7.
PS-5	PS-5	36.0		1.0000	430.871	0.000		0.0	175.22	153.60	(SPECIAL DEVICE AT NODE)				
SC1-305	SC1-305	12.0		0.0020	1.029	0.264	900	0.4	160.78	175.00	0	7.2	3.5	0.0	25.
SC1-310	SC1-310	12.0		0.0020	1.034	0.264	900	0.4	160.98	175.00	0	7.2	0.0	0.0	25.

City of W burn Facilities Plan
Sanitary .er Collection System Analysis
 'n MC-9
 mate Run (PS=FM)
Peak Sanitary and Base Flow = 25.9 mgd
Peak I/I = 12.8 mgd (Distributed)
Total Peak Flow = 38.7 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)
PS-7	PS-7	36.0		1.0000	430.871	0.825		0.0	182.60	161.70	(SPECIAL DEVICE AT NODE)			
MC9-005	MC9-005	12.0		0.0032	1.301	0.547	900	0.5	163.05	181.30	0	8.7	4.9	0.0
MC9-010	MC9-010	12.0		0.0032	1.300	0.549	900	0.5	164.39	182.60	0	8.7	8.7	0.0
MC9-015	MC9-015	10.0		0.0067	1.161	0.192	900	0.2	165.98	182.40	0	5.1	0.0	0.0
MC9-020	MC9-020	10.0		0.0033	0.819	0.192	900	0.3	166.96	182.10	0	5.8	0.0	0.0

City of W urn Facilities Plan
Sanitary S .er Collection System Analysis
Basin MC-8
Ultimate Run (PS=FM)
Peak Sanitary and Base Flow = 25.9 mgd
Peak I/I = 12.8 mgd (Distributed)
Total Peak Flow = 38.7 mgd

03/28/93 11:46

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT CAP
MC8-005	MC8-005	15.0		0.0068	3.450	2.714	900	0.8	135.99	154.20	0	13.7	0.0	0.0	78.
MC8-010	MC8-010	15.0		0.0035	2.469	2.736	900	1.7	138.58	155.70	200	15.6	16.0	6.5	110.
MC8-015	MC8-015	15.0		0.0022	1.957	2.736	900	2.7	140.73	155.00	500	17.0	0.0	10.6	139.
MC8-020	MC8-020	15.0		0.0007	1.088	2.736	900	4.5	142.88	154.90	1900	21.2	0.0	17.5	251.
MC8-025	MC8-025	15.0		0.0007	1.072	2.537	900	6.0	144.73	154.80	1800	20.7	0.0	16.9	236.
MC8-030	MC8-030	15.0		0.0014	1.561	2.537	900	7.2	146.58	154.50	1600	18.0	0.0	12.6	162.
MC8-035	MC8-035	15.0		0.0014	1.566	2.537	900	8.3	148.42	154.00	1600	18.0	0.0	12.5	162.
MC8-040	MC8-040	15.0		0.0012	1.463	0.334	900	7.9	148.44	153.50	1500	8.6	0.0	0.0	22.

City of [illegible]urn Facilities Plan
Se[illegible]tary [illegible]er Collection System Analysis
B[illegible] MC-7
U[illegible]ate Run (PS=FM)
Peak Sanitary and Base Flow = 25.9 mgd
Peak I/I = 12.8 mgd (Distributed)
Total Peak Flow = 38.7 mgd

03/28/93 11:46

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	P C
MC7-005	MC7-005	24.0		0.0206	20.978	6.883	900	0.8	135.18	155.40	0	15.8	8.5	0.0	3
MC7-010	MC7-010	24.0		0.0206	20.988	4.172	900	0.6	136.65	155.10	0	13.1	0.0	0.0	1
MC7-015	MC7-015	18.0		0.0207	9.765	4.175	900	0.7	141.39	155.10	0	13.1	0.0	0.0	4
MC7-020	MC7-020	18.0		0.0207	9.753	4.178	900	0.7	146.10	154.70	0	13.1	0.0	0.0	4
MC7-025	MC7-025	18.0		0.0124	7.560	4.181	900	0.8	148.97	158.10	0	14.4	13.9	0.0	5
MC7-030	MC7-030	18.0		0.0105	6.948	4.186	900	0.8	152.27	160.60	0	14.9	15.6	0.0	6
MC7-035	MC7-035	18.0		0.0125	7.594	4.189	900	0.8	154.86	164.50	0	14.4	13.4	0.0	5
MC7-040	MC7-040	18.0		0.0078	6.000	4.192	900	0.9	156.26	165.90	0	15.7	15.5	0.0	6
MC7-045	MC7-045	18.0		0.0079	6.017	4.197	900	0.9	158.43	167.30	0	15.7	17.1	0.0	6
MC7-050	MC7-050	18.0		0.0044	4.524	3.360	900	0.9	160.67	170.30	0	16.1	15.2	0.0	7
MC7-055	MC7-055	18.0		0.0055	5.035	3.368	900	0.9	163.22	172.70	0	15.5	15.7	0.0	6
MC7-060	MC7-060	18.0		0.0050	4.789	3.146	900	0.9	164.49	174.00	0	15.4	15.3	0.0	6
MC7-065	MC7-065	18.0		0.0043	4.460	3.150	900	0.9	165.64	174.90	0	15.8	16.4	0.0	7
MC7-070	MC7-070	18.0		0.0044	4.479	3.154	900	0.9	166.41	178.10	0	15.8	12.1	0.0	7
MC7-075	MC7-075	18.0		0.0034	3.954	2.967	900	0.9	167.18	178.40	0	16.2	19.1	0.0	7
MC7-080	MC7-080	18.0		0.0028	3.571	2.899	900	1.0	167.96	180.40	0	16.6	13.7	0.0	8
MC7-085	MC7-085	18.0		0.0031	3.770	2.904	900	1.0	168.84	181.90	0	16.3	14.9	0.0	7
MC7-090	MC7-090	18.0		0.0024	3.353	2.909	900	1.1	169.46	181.20	0	17.1	0.0	0.0	8
MC7-095	MC7-095	18.0		0.0051	4.847	2.909	900	0.8	169.47	181.00	0	14.9	0.0	0.0	6
MC7-100	MC7-100	12.0		0.0162	2.934	2.910	900	0.8	170.25	181.60	0	12.0	12.6	0.0	9
MC7-105	MC7-105	12.0		0.0010	0.737	2.662	900	6.9	176.83	182.00	2400	19.4	20.2	17.2	36
MC7-110	MC7-110	12.0		0.0019	1.013	2.662	900	11.2	181.81	182.40	2400	17.2	19.3	14.4	26
MC7-115	MC7-115	12.0		0.0016	0.917	2.662	900	15.6	186.79	182.80	2400	17.9	19.3	15.3	29
MC7-120	MC7-120	12.0		0.0022	1.088	2.483	900	16.5	187.98	181.40	2400	16.4	0.0	13.2	22
MC7-125	MC7-125	12.0		0.0024	1.121	2.370	900	18.0	189.86	181.80	2400	15.9	16.0	12.5	21
MC7-130	MC7-130	12.0		0.0017	0.957	1.682	900	18.8	191.04	181.70	2400	14.8	0.0	10.8	17
MC7-135	MC7-135	12.0		0.0014	0.864	1.559	900	19.5	192.05	181.40	2400	15.0	0.0	11.1	18
MC7-140	MC7-140	12.0		0.0010	0.736	1.461	900	20.2	192.95	180.60	2400	15.5	0.0	11.9	19
MC7-145	MC7-145	12.0		0.0014	0.874	1.435	900	20.7	193.79	181.80	2400	14.5	11.2	10.2	16
MC7-150	MC7-150	12.0		0.0014	0.849	1.276	900	21.5	195.35	182.50	2400	14.0	13.9	9.3	15
MC7-155	MC7-155	12.0		0.0014	0.848	1.276	900	22.1	196.32	182.80	2400	14.0	15.0	9.3	15
MC7-160	MC7-160	12.0		0.0012	0.812	1.222	900	22.4	196.85	182.30	2400	14.0	0.0	9.3	15
MC7-165	MC7-165	12.0		0.0012	0.801	1.222	900	23.1	198.01	181.90	2400	14.1	0.0	9.4	15
MC7-170	MC7-170	12.0		0.0017	0.935	1.167	900	23.3	198.80	182.30	2400	13.0	13.6	7.1	12
MC7-175	MC7-175	12.0		0.0014	0.864	1.125	900	23.5	199.33	181.40	2400	13.3	0.0	7.7	13
PS-2	PS-2	36.0		1.0000	430.871	0.474		22.5	199.33	168.30	(SPECIAL DEVICE AT NODE)				
MC7-180	MC7-180	12.0		0.0018	0.972	0.651	900	23.3	199.55	181.70	2400	10.3	11.3	0.0	6
MC7-185	MC7-185	12.0		0.0015	0.899	0.613	900	23.0	199.78	181.50	2400	10.4	0.0	0.0	6
PS-1	PS-1	36.0		1.0000	430.871	0.613		22.0	199.78	170.00	(SPECIAL DEVICE AT NODE)				
MC7-305	MC7-305	10.0		0.0053	1.026	0.192	900	0.3	167.87	181.50	0	5.3	4.9	0.0	1
MC7-310	MC7-310	10.0		0.0024	0.693	0.193	900	0.3	169.13	182.40	0	6.2	6.5	0.0	2

City of W urn Facilities Plan
Sanitary S er Collection System Analysis
Basin MC-7
Ultimate Run (PS=FM)
Peak Sanitary and Base Flow = 25.9 mgd
Peak I/I = 12.8 mgd (Distributed)
Total Peak Flow = 38.7 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT CAP
MC7-405	MC7-405	10.0		0.0021	0.648	0.696	900	18.1	191.01	182.70	2400	10.3	10.5	3.8	107.
MC7-410	MC7-410	10.0		0.0015	0.557	0.653	900	18.4	191.90	181.10	2400	10.6	0.0	5.2	117.
MC7-415	MC7-415	10.0		0.0024	0.694	0.225	900	17.9	191.95	180.60	2400	6.6	0.0	0.0	32.
MC7-420	MC7-420	10.0		0.0027	0.742	0.180	900	17.4	191.99	181.00	2400	5.9	6.3	0.0	24.
MC7-425	MC7-425	10.0		0.0017	0.579	0.181	900	17.1	192.02	180.70	2400	6.5	0.0	0.0	31.
MC7-505	MC7-505	8.0		0.0038	0.482	0.406	900	18.1	192.49	181.90	2400	7.5	7.6	0.0	84.
MC7-510	MC7-510	8.0		0.0023	0.376	0.408	900	18.3	193.50	182.10	2400	8.2	10.8	3.1	108.
MC7-515	MC7-515	8.0		0.0027	0.409	0.412	900	18.3	194.68	182.60	2200	8.0	9.4	1.1	100.
PS-3	PS-3	36.0		1.0000	430.871	0.355		17.3	194.68	173.10	(SPECIAL DEVICE AT NODE)				

City of W burn Facilities Plan
Sar ry S er Collection System Analysis
Ba MC-6
Ult te Run (PS=FM)
Peak Sanitary and Base Flow = 25.9 mgd
Peak I/I = 12.8 mgd (Distributed)
Total Peak Flow = 38.7 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PC CA
MC6-005	MC6-005	18.0		0.0064	5.436	4.433	900	1.0	155.96	167.80	0	16.7	15.3	0.0	81
MC6-010	MC6-010	18.0		0.0050	4.786	3.968	900	1.0	158.00	165.70	0	16.8	0.0	0.0	82
MC6-015	MC6-015	18.0		0.0029	3.627	3.994	900	1.7	159.79	166.80	200	18.7	18.6	7.6	110
MC6-020	MC6-020	18.0		0.0047	4.667	3.996	900	1.6	160.05	167.00	200	17.0	18.9	0.0	85
MC6-025	MC6-025	18.0		0.0017	2.783	3.929	900	2.0	160.77	168.50	500	20.5	15.7	12.9	141
MC6-030	MC6-030	18.0		0.0021	3.133	3.929	900	2.2	161.38	172.10	400	19.6	12.9	10.8	125
MC6-035	MC6-035	18.0		0.0035	3.991	3.758	900	2.2	161.87	173.40	300	17.6	15.0	0.0	94
MC6-040	MC6-040	18.0		0.0025	3.412	3.769	900	2.3	162.42	174.00	300	18.7	17.7	7.7	110
MC6-045	MC6-045	18.0		0.0031	3.783	3.776	900	2.3	163.10	173.10	200	18.0	0.0	0.0	95
MC6-050	MC6-050	18.0		0.0029	3.647	3.797	900	2.3	163.95	173.80	200	18.3	18.7	5.4	104
MC6-055	MC6-055	18.0		0.0028	3.559	3.800	900	2.3	164.08	173.80	200	18.5	0.0	6.6	106
MC6-060	MC6-060	18.0		0.0029	3.645	3.823	900	2.4	165.04	174.40	200	18.3	19.7	5.8	104
MC6-105	MC6-105	10.0		0.0570	3.378	0.477	900	0.2	158.37	168.10	0	4.8	7.5	0.0	14
MC6-110	MC6-110	10.0		0.0160	1.791	0.451	900	0.3	163.88	175.40	0	6.0	5.6	0.0	25
MC6-115	MC6-115	10.0		0.0023	0.680	0.453	900	0.5	164.72	176.00	0	8.6	8.7	0.0	66
MC6-120	MC6-120	10.0		0.0029	0.756	0.370	900	0.4	165.85	179.20	0	7.6	6.4	0.0	48
MC6-125	MC6-125	10.0		0.0029	0.761	0.371	900	0.4	166.55	179.80	0	7.6	7.9	0.0	48
MC6-130	MC6-130	10.0		0.0028	0.755	0.371	900	0.4	166.94	180.50	0	7.7	6.9	0.0	49

City of W burn Facilities Plan
Sanitary S er Collection System Analysis
Basin MC-5
Ultimate Run (PS=FM)
Peak Sanitary and Base Flow = 25.9 mgd
Peak I/I = 12.8 mgd (Distributed)
Total Peak Flow = 38.7 mgd

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT CAP
MC1-105	MC1-105	36.0		1.0000	430.871	1.349	900	0.0	149.42	165.40	0	4.1	0.0	0.0	0.
MC1-110	MC1-110	36.0		2.0500	616.912	0.244	900	0.0	150.44	165.80	0	1.9	0.0	0.0	0.
MC5-005	MC5-005	12.0		0.2516	11.545	1.349	900	0.2	155.91	166.00	0	5.4	8.3	0.0	11.
MC5-010	MC5-010	14.0		0.0209	5.022	1.349	900	0.4	157.46	172.80	0	8.6	6.3	0.0	26.
MC5-015	MC5-015	12.0		0.0332	4.193	1.350	900	0.4	165.87	175.80	0	7.8	9.5	0.0	32.
MC5-020	MC5-020	12.0		0.0020	1.040	1.104	900	1.1	167.31	180.40	200	12.3	8.7	4.2	106.
MC5-025	MC5-025	12.0		0.0021	1.057	1.005	900	1.0	168.11	181.70	100	11.8	11.0	0.0	95.
MC5-030	MC5-030	12.0		0.0163	2.937	0.818	1000	0.4	168.18	182.80	0	7.4	6.8	0.0	27.
MC5-035	MC5-035	12.0		0.0013	0.816	0.833	900	1.0	169.46	183.00	200	12.1	15.0	2.8	102.
MC5-040	MC5-040	12.0		0.0018	0.965	0.765	900	0.7	170.01	183.90	0	11.0	11.0	0.0	79.
MC5-045	MC5-045	12.0		0.0029	1.230	0.573	900	0.5	170.39	184.30	0	9.0	9.7	0.0	46.
MC5-050	MC5-050	12.0		0.0027	1.193	0.574	900	0.5	170.72	184.60	0	9.1	9.2	0.0	48.
MC5-055	MC5-055	12.0		0.0030	1.254	0.476	900	0.4	171.40	185.40	0	8.3	8.2	0.0	38.
MC5-060	MC5-060	10.0		0.0037	0.858	0.159	900	0.3	172.96	184.80	0	5.3	0.0	0.0	18.
MC5-065	MC5-065	10.0		0.0031	0.793	0.160	900	0.3	174.54	183.80	0	5.5	0.0	0.0	20.
MC5-070	MC5-070	10.0		0.0041	0.904	0.160	900	0.3	175.43	183.40	0	5.2	0.0	0.0	17.
MC5-105	MC5-105	10.0		0.2368	6.888	0.244	900	0.1	156.44	164.60	0	2.9	0.0	0.0	3.
MC5-110	MC5-110	12.0		0.0466	4.968	0.244	900	0.1	166.04	174.50	0	3.9	3.8	0.0	4.
MC5-115	MC5-115	10.0		0.0105	1.448	0.128	900	0.2	169.68	176.80	0	4.0	4.4	0.0	8.
MC5-205	MC5-205	12.0		0.0051	1.647	0.087	900	0.1	167.30	178.10	0	4.0	3.3	0.0	5.
MC5-210	MC5-210	12.0		0.0095	2.243	0.087	900	0.1	169.54	179.40	0	3.6	3.9	0.0	3.
MC5-215	MC5-215	10.0		0.0095	1.377	0.088	900	0.1	171.85	180.40	0	3.6	4.2	0.0	6.

City of w burn Facilities Plan
S tary S er Collection System Analysis
B MC-4
U mate Run (PS=FM)
Peak Sanitary and Base Flow = 25.9 mgd
Peak I/I = 12.8 mgd (Distributed)
Total Peak Flow = 38.7 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)
MC4-005	MC4-005	10.0		0.0297	2.441	1.955	900	0.6	156.69	172.20	0	9.2	8.9	0.0
MC4-010	MC4-010	10.0		0.0548	3.313	1.320	900	0.4	163.03	170.10	0	7.1	0.0	0.0
MC4-015	MC4-015	10.0		0.0137	1.655	1.321	900	0.5	164.55	175.80	0	9.2	7.0	0.0
MC4-020	MC4-020	10.0		0.0153	1.749	1.208	900	0.5	165.95	177.40	0	8.7	8.5	0.0
MC4-025	MC4-025	10.0		0.0039	0.881	1.107	900	1.7	168.75	179.60	400	10.9	10.2	6.0
MC4-030	MC4-030	10.0		0.0034	0.823	1.025	900	2.5	170.86	181.30	400	10.9	10.4	5.9
MC4-035	MC4-035	10.0		0.0036	0.853	0.907	900	2.7	172.51	183.00	300	10.2	9.9	3.5
MC4-040	MC4-040	10.0		0.0025	0.711	0.656	900	2.5	173.57	182.20	300	9.7	0.0	0.0
MC4-045	MC4-045	10.0		0.0030	0.777	0.556	900	1.8	174.31	181.20	200	8.8	0.0	0.0
MC4-105	MC4-105	10.0		0.0290	2.411	0.599	900	0.3	166.77	180.40	0	5.9	6.2	0.0
MC4-110	MC4-110	10.0		0.0031	0.794	0.555	900	0.5	168.36	180.30	0	8.7	0.0	0.0
MC4-115	MC4-115	10.0		0.0032	0.797	0.558	900	0.5	169.77	183.00	0	8.7	7.7	0.0
MC4-120	MC4-120	10.0		0.0023	0.676	0.391	900	0.4	170.86	180.40	0	8.1	0.0	0.0
MC4-125	MC4-125	10.0		0.0060	1.092	0.392	900	0.4	172.02	181.70	0	6.8	6.8	0.0
MC4-130	MC4-130	10.0		0.0025	0.709	0.272	900	0.4	172.72	183.40	0	7.0	5.9	0.0
MC4-135	MC4-135	10.0		0.0070	1.180	0.273	900	0.3	173.37	183.90	0	5.8	6.2	0.0
MC4-140	MC4-140	10.0		0.0025	0.712	0.274	900	0.4	174.29	183.90	0	7.0	0.0	0.0
MC4-145	MC4-145	10.0		0.0025	0.710	0.103	900	0.2	174.84	184.70	0	4.9	4.7	0.0
MC4-205	MC4-205	10.0		0.1346	5.192	0.185	900	0.1	165.67	171.70	0	2.9	3.2	0.0
MC4-210	MC4-210	10.0		0.0050	1.003	0.185	900	0.3	167.18	173.80	0	5.3	4.9	0.0
MC4-215	MC4-215	10.0		0.0056	1.055	0.147	900	0.2	168.64	175.50	0	4.8	4.7	0.0
MC4-220	MC4-220	10.0		0.0046	0.958	0.048	900	0.1	170.06	177.30	0	3.2	3.1	0.0

City of W burn Facilities Plan
Sanitary s er Collection System Analysis
Basin MC-3
Ultimate Run (PS=FM)
Peak Sanitary and Base Flow = 25.9 mgd
Peak I/I = 12.8 mgd (Distributed)
Total Peak Flow = 38.7 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT. CAP.
MC3-005	MC3-005	18.0		0.0083	6.195	13.918	900	3.5	148.02	157.70	1800	24.4	21.4	19.6	224.6
MC3-010	MC3-010	12.0		0.0247	3.615	13.918	900	13.8	158.99	159.10	2400	19.9	17.7	17.8	385.0
MC3-015	MC3-015	16.0		0.0626	12.399	13.918	900	15.9	169.39	157.30	1400	16.7	0.0	7.3	112.2
MC3-020	MC3-020	16.0		0.0021	2.283	13.918	900	52.8	207.32	160.00	2400	31.5	26.3	29.5	609.7
MC3-025	MC3-025	16.0		0.0084	4.546	13.713	900	77.7	235.25	166.50	2400	24.2	21.0	20.8	301.6
MC3-030	MC3-030	18.0		0.0084	6.206	13.713	900	81.1	239.50	172.90	2400	24.2	16.7	19.3	220.9
MC3-035	MC3-035	14.0		0.0049	2.438	13.713	900	113.2	272.73	169.00	2400	26.8	0.0	24.9	562.5
MC3-040	MC3-040	14.0		0.0049	2.420	12.584	900	139.8	300.33	169.80	2400	26.0	27.2	24.0	520.1
MC3-045	MC3-045	12.0		0.0068	1.899	12.584	900	238.6	401.37	174.90	2400	24.4	21.0	22.9	662.8
MC3-050	MC3-050	12.0		0.0072	1.947	12.584	900	329.9	494.95	180.00	2400	24.2	20.7	22.7	646.3
MC3-055	MC3-055	16.0		0.0025	2.478	12.584	900	331.1	496.24	180.40	2400	29.4	19.9	27.1	507.7
MC3-060	MC3-060	16.0		0.0018	2.104	12.440	900	355.9	521.75	180.60	2400	31.2	39.7	29.1	591.1
MC3-065	MC3-065	16.0		0.0012	1.730	12.139	900	365.1	531.10	180.20	2400	33.2	0.0	31.4	701.7
MC3-070	MC3-070	16.0		0.0007	1.267	11.956	900	379.2	545.35	180.60	2400	37.1	31.3	35.6	943.8
MC3-075	MC3-075	15.0		0.0004	0.809	11.854	900	400.5	566.82	177.40	2400	41.1	0.0	40.0	1465.1
MC3-080	MC3-080	16.0		0.0010	1.602	11.699	900	414.1	580.69	179.10	2400	33.7	23.7	31.9	730.4
MC3-085	MC3-085	16.0		0.0004	0.937	11.577	900	427.8	594.44	179.80	2400	41.1	28.0	39.8	1235.9
MC3-090	MC3-090	16.0		0.0018	2.124	11.477	900	436.0	602.91	178.00	2400	30.1	0.0	27.9	540.4
MC3-095	MC3-095	16.0		0.0016	1.954	11.477	900	450.7	618.08	177.90	2400	31.1	0.0	29.0	587.2
MC3-100	MC3-100	16.0		0.0014	1.843	11.477	900	468.5	636.31	180.00	2400	31.8	24.0	29.8	622.7
MC3-105	MC3-105	10.0		0.0037	0.860	2.266	900	474.2	642.98	182.10	2400	14.4	12.4	12.0	263.5
MC3-110	MC3-110	10.0		0.0030	0.775	2.181	900	479.6	649.15	183.40	2400	14.7	13.4	12.5	281.3
MC3-115	MC3-115	10.0		0.0027	0.737	2.102	900	484.6	654.84	182.30	2400	14.8	0.0	12.6	285.0
MC3-120	MC3-120	10.0		0.0022	0.670	1.995	900	490.1	661.04	181.80	2400	15.1	0.0	12.9	297.4
MC3-125	MC3-125	10.0		0.0030	0.775	1.995	900	492.4	663.82	182.50	2400	14.3	13.0	11.9	257.3
MC3-130	MC3-130	10.0		0.0023	0.676	1.995	900	497.5	669.56	182.50	2400	15.0	0.0	12.8	294.5
MC3-135	MC3-135	10.0		0.0036	0.845	1.930	900	502.0	675.14	183.60	2400	13.6	13.6	11.0	228.3
MC3-140	MC3-140	10.0		0.0038	0.872	1.769	900	505.2	679.30	183.30	2400	13.0	0.0	10.1	202.8
MC3-145	MC3-145	10.0		0.0044	0.937	1.769	900	508.1	683.36	182.70	2400	12.7	0.0	9.6	188.7
MC3-150	MC3-150	10.0		0.0040	0.896	1.682	900	512.0	688.85	180.90	2400	12.7	0.0	9.5	187.6
MC3-155	MC3-155	8.0		0.0040	0.494	1.682	900	531.9	710.66	181.90	2400	12.7	14.3	11.1	340.6
MC3-160	MC3-160	8.0		0.0040	0.494	1.682	900	536.2	715.30	182.80	2400	12.7	10.9	11.1	340.6
MC3-205	MC3-205	10.0		0.0007	0.377	1.129	900	114.5	274.17	175.00	2400	15.1	7.6	12.9	299.1
MC3-210	MC3-210	10.0		0.0008	0.393	0.929	900	114.7	274.45	174.30	2400	13.8	0.0	11.2	236.6
MC3-215	MC3-215	10.0		0.0007	0.368	0.929	900	115.6	275.47	174.30	2400	14.2	0.0	11.7	252.5
MC3-220	MC3-220	10.0		0.0007	0.379	0.767	900	116.3	276.37	173.50	2400	13.0	0.0	10.1	202.5
MC3-225	MC3-225	10.0		0.0041	0.901	0.523	900	116.0	276.51	173.80	2400	8.2	8.7	0.0	57.9
MC3-230	MC3-230	10.0		0.0021	0.649	0.525	900	115.6	277.20	175.40	2400	9.2	8.5	0.0	80.9
MC3-305	MC3-305	15.0		0.0180	5.593	0.301	900	351.2	521.76	182.50	2400	5.0	5.9	0.0	5.3
MC3-310	MC3-310	12.0		0.0045	1.544	0.272	900	350.1	521.80	182.60	2400	6.3	9.9	0.0	17.6
MC3-315	MC3-315	10.0		0.0102	1.426	0.253	900	347.5	521.88	183.20	2400	5.2	6.9	0.0	17.7

City of [illegible] burn Facilities Plan
Sanitary [illegible]wer Collection System Analysis
E[illegible]n MC-3
L[illegible]mate Run (PS=FM)
Peak Sanitary and Base Flow = 25.9 mgd
Peak I/I = 12.8 mgd (Distributed)
Total Peak Flow = 38.7 mgd

03/28/93 11:46

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	
MC3-320	MC3-320	16.0		0.0211	7.203	0.183	900	359.6	531.10	181.10	2400	4.0	5.7	0.0	
MC3-325	MC3-325	10.0		0.0036	0.847	0.164	900	358.7	531.14	181.90	2400	5.4	5.6	0.0	
MC3-330	MC3-330	10.0		0.0265	2.306	0.101	900	372.3	545.37	181.00	2400	3.1	5.3	0.0	
MC3-335	MC3-335	10.0		0.0043	0.933	0.073	900	371.2	545.38	182.30	2400	3.9	3.8	0.0	
MC3-340	MC3-340	12.0		0.0208	3.319	0.155	900	395.0	566.83	180.00	2400	3.8	4.4	0.0	
MC3-345	MC3-345	10.0		0.0052	1.016	0.089	900	393.7	566.84	181.30	2400	4.0	4.0	0.0	
MC3-350	MC3-350	14.0		0.0218	5.122	0.122	900	408.5	580.70	178.90	2400	3.5	0.0	0.0	
MC3-355	MC3-355	14.0		0.0040	2.185	0.089	900	407.5	580.70	180.00	2400	4.2	4.2	0.0	
MC3-360	MC3-360	12.0		0.0042	1.483	0.089	900	406.4	580.70	180.40	2400	4.2	5.0	0.0	
MC3-365	MC3-365	10.0		0.0059	1.086	0.089	900	404.9	580.71	182.40	2400	3.9	3.7	0.0	
MC3-370	MC3-370	10.0		0.0260	2.281	0.100	900	421.2	594.45	181.00	2400	3.1	4.3	0.0	
MC3-405	MC3-405	12.0		0.0018	0.971	9.211	900	540.5	709.19	179.50	2400	27.9	0.0	26.8	9[illegible]
MC3-410	MC3-410	10.0		0.0019	0.621	8.925	900	750.3	919.93	181.60	2400	27.2	23.7	26.4	14[illegible]
MC3-415	MC3-415	10.0		0.0018	0.602	8.925	900	936.3	1106.82	182.60	2400	27.5	26.7	26.8	14[illegible]
MC3-420	MC3-420	10.0		0.0018	0.606	8.737	900	1097.5	1268.76	182.30	2400	27.2	0.0	26.5	14[illegible]
MC3-425	MC3-425	10.0		0.0029	0.762	8.737	900	1257.8	1430.33	184.20	2400	25.0	23.0	24.1	11[illegible]
MC3-430	MC3-430	10.0		0.0031	0.793	8.737	900	1340.9	1514.16	183.90	2400	24.6	0.0	23.7	11[illegible]
MC3-435	MC3-435	10.0		0.0005	0.314	8.557	900	1540.6	1714.11	183.80	2400	34.5	0.0	34.0	27[illegible]
MC3-440	MC3-440	10.0		0.0021	0.650	8.250	900	1684.8	1859.18	183.30	2400	25.9	0.0	25.2	12[illegible]
MC3-445	MC3-445	10.0		0.0017	0.589	8.167	900	1759.3	1934.10	183.60	2400	26.8	28.2	26.1	13[illegible]
MC3-450	MC3-450	10.0		0.0019	0.615	8.097	900	1845.5	2020.83	184.00	2400	26.3	27.4	25.5	131
MC3-455	MC3-455	10.0		0.0035	0.843	8.028	900	1945.1	2121.53	183.70	2400	23.3	0.0	22.3	95
PS-10	PS-10	36.0		1.0000	430.871	7.964		1944.1	2121.53	154.00	(SPECIAL DEVICE AT NODE)				
MC3-505	MC3-505	8.0		0.0080	0.698	0.265	900	1540.0	1714.20	183.00	2400	5.6	0.0	0.0	3
MC3-510	MC3-510	8.0		0.0044	0.518	0.265	900	1539.6	1714.35	182.10	2400	6.2	0.0	0.0	5
MC3-515	MC3-515	8.0		0.0075	0.678	0.266	900	1536.8	1714.87	183.60	2400	5.6	6.5	0.0	3
MC3-520	MC3-520	8.0		0.0000	0.035	0.266	900	1537.4	1715.45	183.60	2400	17.1	0.0	16.3	76
MC3-525	MC3-525	8.0		0.0031	0.433	0.267	900	1536.5	1715.97	183.90	2400	6.7	8.9	0.0	6
PS-6	PS-6	36.0		1.0000	430.871	0.179		1535.5	1715.97	172.40	(SPECIAL DEVICE AT NODE)				
MC3-605	MC3-605	12.0		0.0013	0.836	0.000	200	467.8	636.31	182.50	2400	0.0	0.0	0.0	
MC3-610	MC3-610	12.0		0.0013	0.832	0.000	0	467.4	636.31	182.00	2400	0.0	0.0	0.0	

City of W burn Facilities Plan
Sanitary S..er Collection System Analysis
Basin MC-2
Ultimate Run (PS=FM)
Peak Sanitary and Base Flow = 25.9 mgd
Peak I/I = 12.8 mgd (Distributed)
Total Peak Flow = 38.7 mgd

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT CAP
MC2-005	MC2-005	18.0		0.2253	32.211	2.534	1000	0.3	159.83	179.00	0	6.9	6.8	0.0	7.
MC2-010	MC2-010	18.0		0.0014	2.502	2.543	900	1.5	161.34	180.20	200	18.1	13.8	3.9	101.
MC2-015	MC2-015	18.0		0.0008	1.879	2.543	900	1.7	161.82	179.80	1200	20.2	0.0	12.2	135.
MC2-020	MC2-020	18.0		0.0008	1.887	2.543	900	2.0	162.35	179.40	1200	20.1	0.0	12.1	134.
MC2-025	MC2-025	18.0		0.0011	2.267	2.543	900	2.0	162.69	179.70	600	18.8	18.4	8.2	112.
MC2-030	MC2-030	18.0		0.0011	2.237	2.280	900	2.1	163.25	180.00	400	18.1	20.2	4.1	101.
MC2-035	MC2-035	18.0		0.0019	2.958	0.652	900	1.2	163.29	181.20	0	10.2	9.8	0.0	22.
MC2-040	MC2-040	12.0		0.0013	0.836	0.437	900	0.7	163.47	181.70	0	9.4	9.9	0.0	52.
MC2-045	MC2-045	12.0		0.0017	0.941	0.439	900	0.5	164.14	181.70	0	9.0	0.0	0.0	46.
MC2-050	MC2-050	12.0		0.0018	0.984	0.272	900	0.4	164.99	182.70	0	7.4	7.3	0.0	27.
MC2-105	MC2-105	10.0		0.0052	1.025	1.632	900	6.6	170.73	181.80	1700	11.9	13.1	8.2	159.
MC2-110	MC2-110	10.0		0.0076	1.232	1.446	900	7.4	173.65	181.10	1500	10.6	0.0	5.2	117.
MC2-115	MC2-115	10.0		0.0030	0.777	1.345	900	10.1	177.72	180.00	1800	12.3	0.0	8.9	173.
MC2-120	MC2-120	10.0		0.0031	0.782	1.139	900	11.5	180.32	180.00	1700	11.5	0.0	7.5	145.
MC2-125	MC2-125	10.0		0.0031	0.787	1.017	900	12.2	182.12	180.00	1600	11.0	0.0	6.3	129.
MC2-130	MC2-130	10.0		0.0030	0.771	1.017	900	12.9	183.67	178.00	1600	11.1	0.0	6.5	131.
PS-9	PS-9	36.0		1.0000	430.871	0.833		11.9	183.67	157.30	(SPECIAL DEVICE AT NODE)				

City of ... ourn Facilities Plan
Sanitary S..er Collection System Analysis
B... MC-1
U... ate Run (PS=FM)
Pea... Sanitary and Base Flow = 25.9 mgd
Peak I/I = 12.8 mgd (Distributed)
Total Peak Flow = 38.7 mgd

03/28/93 11:46

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	P C
PS-MC	PS-MC	36.0		1.0000	430.871	35.161		0.6	183.16	136.60	(SPECIAL DEVICE AT NODE)				
MC1-005	MC1-005	30.0		0.0116	28.492	28.282	900	2.0	139.57	157.50	0	29.9	14.0	0.0	9
MC1-010	MC1-010	30.0		0.0116	28.488	28.324	900	2.0	144.24	159.30	0	29.9	35.8	0.0	9
MC1-015	MC1-015	36.0		0.0142	51.284	25.799	900	1.9	144.24	159.30	0	27.8	0.0	0.0	5
MC1-020	MC1-020	36.0		0.0010	13.939	25.799	900	3.4	145.99	153.30	1600	45.4	0.0	33.9	18
MC1-025	MC1-025	36.0		0.0007	11.620	25.799	900	4.2	146.98	153.60	1700	48.6	45.0	38.8	22
MC1-030	MC1-030	36.0		0.0001	5.117	25.799	900	6.7	149.52	154.10	2400	66.0	48.8	60.8	50
MC1-035	MC1-035	36.0		0.0021	19.634	25.622	900	7.2	150.82	155.30	1700	39.8	36.5	23.1	13
MC1-040	MC1-040	36.0		0.0012	14.848	25.622	900	8.0	151.95	156.70	1700	44.2	34.6	31.9	17
MC1-045	MC1-045	27.0		0.0016	7.973	11.704	900	8.3	152.53	156.00	1800	31.2	0.0	20.3	14
MC1-050	MC1-050	27.0		0.0014	7.486	11.704	900	8.7	153.21	160.60	1700	31.9	18.9	21.8	15
MC1-055	MC1-055	27.0		0.0013	7.306	11.704	900	8.9	153.52	165.20	1700	32.2	16.3	22.3	16
MC1-060	MC1-060	27.0		0.0017	8.235	11.704	900	9.4	154.55	162.00	1700	30.8	0.0	19.5	14
MC1-065	MC1-065	27.0		0.0017	8.236	11.704	900	10.1	155.99	158.70	1600	30.8	0.0	19.5	14
MC1-070	MC1-070	27.0		0.0047	13.783	11.708	900	10.0	156.19	160.30	1600	25.4	18.3	0.0	8
MC1-075	MC1-075	27.0		0.0012	6.825	11.336	900	10.6	157.07	160.80	1600	32.7	30.0	23.1	16
MC1-080	MC1-080	27.0		0.0011	6.756	11.336	900	11.1	157.80	161.20	1600	32.8	30.2	23.3	16
MC1-085	MC1-085	24.0		0.0046	9.955	9.207	900	11.0	158.30	159.00	1600	23.3	0.0	0.0	9
MC1-090	MC1-090	24.0		0.0040	9.222	9.237	900	11.0	159.20	160.50	1400	24.0	21.8	2.2	10
MC1-095	MC1-095	24.0		0.0011	4.921	9.237	900	12.1	160.75	161.40	1600	30.4	26.6	22.8	18
MC1-100	MC1-100	24.0		0.0015	5.577	9.237	900	12.9	161.99	161.80	1400	29.0	29.6	20.5	16
MC1-105	MC1-105	24.0		0.0008	4.094	9.237	900	14.0	163.36	165.40	1700	32.6	20.0	26.1	22
MC1-110	MC1-110	24.0		0.0100	14.614	7.892	900	13.2	163.67	165.80	1000	19.1	22.8	0.0	5
MC1-115	MC1-115	18.0		0.0110	7.103	7.655	900	13.4	164.86	164.10	900	18.5	0.0	6.9	10
MC1-120	MC1-120	18.0		0.0064	5.431	7.642	900	15.7	169.42	165.90	600	20.5	21.4	12.9	14
MC1-125	MC1-125	15.0		0.0036	2.521	3.215	900	16.0	170.29	166.00	600	16.4	22.5	9.3	12
MC1-130	MC1-130	15.0		0.0085	3.841	3.216	900	15.8	170.72	164.10	600	14.0	0.0	0.0	8
MC1-135	MC1-135	10.0		0.0507	3.188	3.218	900	15.9	174.24	165.50	500	10.0	11.9	1.7	10
MC1-140	MC1-140	8.0		0.0033	0.446	3.218	900	46.5	205.51	165.40	2400	16.8	0.0	15.9	72
MC1-145	MC1-145	8.0		0.0094	0.759	3.218	900	119.1	282.32	175.50	2400	13.8	11.7	12.4	42
MC1-150	MC1-150	8.0		0.0040	0.492	3.156	900	158.0	322.19	170.00	2400	16.1	0.0	15.1	64
MC1-155	MC1-155	8.0		0.0049	0.545	3.030	900	197.2	362.73	174.10	2400	15.2	12.3	14.1	55
MC1-160	MC1-160	8.0		0.0067	0.639	2.945	900	208.7	374.83	170.80	2400	14.2	0.0	12.9	46
MC1-165	MC1-165	8.0		0.0029	0.422	2.945	900	242.2	408.99	172.20	2400	16.6	14.6	15.7	69
MC1-170	MC1-170	8.0		0.0026	0.398	2.945	900	249.2	416.11	170.60	2400	16.9	0.0	16.0	73
MC1-175	MC1-175	8.0		0.0029	0.424	2.861	900	279.5	447.13	172.70	2400	16.4	13.3	15.4	67
MC1-180	MC1-180	8.0		0.0034	0.452	2.861	900	336.9	505.97	173.30	2400	16.0	18.9	15.0	63

City of W burn Facilities Plan
Sanitary s..er Collection System Analysis
Basin SC-1
1990 Run (PS=FM) (I/I=50%)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 4.3 mgd (Distributed)
Total Peak Flow = 12.9 mgd

03/28/93 12:06

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT. CAP
PS-4	PS-4	36.0		1.0000	430.871	0.401		0.0	180.13	165.90	(SPECIAL DEVICE AT NODE)				
SC1-005	SC1-005	15.0		0.0021	1.912	0.128	1000	0.2	160.27	174.40	0	5.4	2.6	0.0	6.
SC1-010	SC1-010	15.0		0.0020	1.889	0.128	1000	0.2	161.03	176.50	0	5.5	4.5	0.0	6.
SC1-015	SC1-015	15.0		0.0006	1.033	0.128	1000	0.3	161.34	177.40	0	6.9	5.2	0.0	12.
SC1-020	SC1-020	15.0		0.0008	1.184	0.128	900	0.3	161.65	177.50	0	6.5	8.2	0.0	10.
SC1-025	SC1-025	15.0		0.0030	2.268	0.032	2300	0.1	162.38	177.60	0	3.0	4.7	0.0	1.
SC1-030	SC1-030	15.0		0.0015	1.599	0.032	2300	0.1	162.99	179.10	0	3.5	2.9	0.0	2.
SC1-035	SC1-035	15.0		0.0015	1.626	0.032	200	0.1	163.62	179.10	0	3.5	0.0	0.0	1.
SC1-040	SC1-040	10.0		0.0030	0.773	0.032	200	0.1	165.09	180.10	0	3.0	3.3	0.0	4.
SC1-045	SC1-045	10.0		0.0027	0.731	0.032	2300	0.1	166.28	176.20	0	3.1	0.0	0.0	4.
SC1-105	SC1-105	10.0		0.0036	0.855	0.096	900	0.2	162.80	179.50	0	4.4	4.0	0.0	11.
SC1-110	SC1-110	10.0		0.0029	0.760	0.096	900	0.2	163.79	179.30	0	4.6	0.0	0.0	12.
SC1-115	SC1-115	10.0		0.0029	0.768	0.055	900	0.1	164.73	177.30	0	3.7	0.0	0.0	7.
SC1-120	SC1-120	10.0		0.0027	0.737	0.055	900	0.2	165.41	174.90	0	3.8	0.0	0.0	7.
SC1-125	SC1-125	10.0		0.0034	0.819	0.028	900	0.1	166.59	177.20	0	2.8	2.5	0.0	3.
SC1-205	SC1-205	12.0		0.0032	1.305	0.166	1000	0.3	161.25	175.60	0	5.5	3.0	0.0	12.
SC1-210	SC1-210	12.0		0.0033	1.314	0.166	900	0.3	162.75	173.40	0	5.5	0.0	0.0	12.
SC1-215	SC1-215	12.0		0.0052	1.653	0.167	900	0.2	164.68	169.20	0	5.1	0.0	0.0	10.
SC1-220	SC1-220	12.0		0.0054	1.692	0.076	900	0.1	166.67	169.60	0	3.8	5.1	0.0	4.
SC1-225	SC1-225	12.0		0.0054	1.689	0.077	900	0.1	168.69	173.90	0	3.8	3.3	0.0	4.
SC1-230	SC1-230	12.0		0.0054	1.693	0.043	900	0.1	170.67	177.10	0	3.0	2.8	0.0	2.
SC1-235	SC1-235	12.0		0.0048	1.587	0.044	900	0.1	173.13	178.40	0	3.1	3.5	0.0	2.
SC1-240	SC1-240	12.0		0.0035	1.356	0.044	900	0.1	174.32	180.30	0	3.3	3.0	0.0	3.
PS-5	PS-5	36.0		1.0000	430.871	0.000		0.0	175.22	153.60	(SPECIAL DEVICE AT NODE)				
SC1-305	SC1-305	12.0		0.0020	1.029	0.109	900	0.2	160.65	175.00	0	5.2	2.5	0.0	10.
SC1-310	SC1-310	12.0		0.0020	1.034	0.109	900	0.2	160.84	175.00	0	5.2	0.0	0.0	10.

City of [illegible]burn Facilities Plan
San[illegible]tary [illegible]er Collection System Analysis
Ba[illegible] MC-9
19[illegible] un (PS=FM) (I/I=50%)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 4.3 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PC CA
PS-7	PS-7	36.0		1.0000	430.871	0.370		0.0	182.59	161.70	(SPECIAL DEVICE AT NODE)				
MC9-005	MC9-005	12.0		0.0032	1.301	0.282	900	0.3	162.93	181.30	0	6.8	3.8	0.0	21
MC9-010	MC9-010	12.0		0.0032	1.300	0.283	900	0.3	164.27	182.60	0	6.8	6.8	0.0	21
MC9-015	MC9-015	10.0		0.0067	1.161	0.047	900	0.1	165.84	182.40	0	3.0	0.0	0.0	4
MC9-020	MC9-020	10.0		0.0033	0.819	0.047	900	0.1	166.80	182.10	0	3.4	0.0	0.0	5

City of W urn Facilities Plan
Sanitary S er Collection System Analysis
Basin MC-8
1990 Run (PS=FM) (I/I=50%)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 4.3 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT. CAP.
MC8-005	MC8-005	15.0		0.0068	3.450	0.211	1000	0.2	135.38	154.20	0	5.3	0.0	0.0	6.1
MC8-010	MC8-010	15.0		0.0035	2.469	0.211	1000	0.2	137.18	155.70	0	6.0	6.1	0.0	8.5
MC8-015	MC8-015	15.0		0.0022	1.957	0.211	1000	0.3	138.32	155.00	0	6.5	0.0	0.0	10.7
MC8-020	MC8-020	15.0		0.0007	1.088	0.212	900	0.4	138.77	154.90	0	8.1	0.0	0.0	19.4
MC8-025	MC8-025	15.0		0.0007	1.072	0.132	900	0.3	139.01	154.80	0	6.8	0.0	0.0	12.3
MC8-030	MC8-030	15.0		0.0014	1.561	0.133	900	0.2	139.65	154.50	0	6.0	0.0	0.0	8.5
MC8-035	MC8-035	15.0		0.0014	1.566	0.133	900	0.2	140.35	154.00	0	6.0	0.0	0.0	8.5
MC8-040	MC8-040	15.0		0.0012	1.463	0.045	2300	0.1	140.65	153.50	0	4.1	0.0	0.0	3.(

City of W burn Facilities Plan
Sa ary S er Collection System Analysis
Ba MC-7
1990 Run (PS=FM) (I/I=50%)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 4.3 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	P C
MC7-005	MC7-005	24.0		0.0206	20.978	1.945	1000	0.4	134.78	155.40	0	9.8	5.3	0.0	
MC7-010	MC7-010	24.0		0.0206	20.988	1.735	1000	0.4	136.40	155.10	0	9.4	0.0	0.0	
MC7-015	MC7-015	18.0		0.0207	9.765	1.735	1000	0.5	141.14	155.10	0	9.4	0.0	0.0	1
MC7-020	MC7-020	18.0		0.0207	9.753	1.735	1000	0.5	145.85	154.70	0	9.4	0.0	0.0	1
MC7-025	MC7-025	18.0		0.0124	7.560	1.735	1000	0.5	148.70	158.10	0	10.4	10.0	0.0	2
MC7-030	MC7-030	18.0		0.0105	6.948	1.736	1000	0.5	151.98	160.60	0	10.7	11.2	0.0	2
MC7-035	MC7-035	18.0		0.0125	7.594	1.736	1000	0.5	154.59	164.50	0	10.4	9.6	0.0	2
MC7-040	MC7-040	18.0		0.0078	6.000	1.736	1000	0.6	155.94	165.90	0	11.3	11.1	0.0	2
MC7-045	MC7-045	18.0		0.0079	6.017	1.737	1000	0.6	158.11	167.30	0	11.3	12.3	0.0	2
MC7-050	MC7-050	18.0		0.0044	4.524	1.619	1000	0.6	160.37	170.30	0	12.2	11.6	0.0	3
MC7-055	MC7-055	18.0		0.0055	5.035	1.619	1000	0.6	162.95	172.70	0	11.8	12.0	0.0	3
MC7-060	MC7-060	18.0		0.0050	4.789	1.526	1000	0.6	164.23	174.00	0	11.7	11.7	0.0	3
MC7-065	MC7-065	18.0		0.0043	4.460	1.527	1000	0.6	165.36	174.90	0	12.0	12.5	0.0	3
MC7-070	MC7-070	18.0		0.0044	4.479	1.527	1000	0.6	166.14	178.10	0	12.0	9.2	0.0	3
MC7-075	MC7-075	18.0		0.0034	3.954	1.512	1000	0.7	166.90	178.40	0	12.6	14.8	0.0	3
MC7-080	MC7-080	18.0		0.0028	3.571	1.481	1000	0.7	167.65	180.40	0	12.9	10.7	0.0	4
MC7-085	MC7-085	18.0		0.0031	3.770	1.481	1000	0.7	168.55	181.90	0	12.7	11.6	0.0	3
MC7-090	MC7-090	18.0		0.0024	3.353	1.481	1000	0.7	169.11	181.20	0	13.3	0.0	0.0	4
MC7-095	MC7-095	18.0		0.0051	4.847	1.481	1000	0.6	169.24	181.00	0	11.5	0.0	0.0	3
MC7-100	MC7-100	12.0		0.0162	2.934	1.481	1000	0.5	169.93	181.60	0	9.3	9.8	0.0	5
MC7-105	MC7-105	12.0		0.0010	0.737	1.351	1000	2.2	172.08	182.00	1900	15.1	15.6	11.2	18
MC7-110	MC7-110	12.0		0.0019	1.013	1.351	1000	2.7	173.36	182.40	1600	13.4	14.9	8.0	13
MC7-115	MC7-115	12.0		0.0016	0.917	1.351	1000	3.4	174.64	182.80	1600	13.9	14.9	9.1	14
MC7-120	MC7-120	12.0		0.0022	1.088	1.222	1000	3.5	174.93	181.40	1600	12.5	0.0	5.5	11
MC7-125	MC7-125	12.0		0.0024	1.121	1.171	1000	3.5	175.39	181.80	1400	12.2	12.3	3.7	10
MC7-130	MC7-130	12.0		0.0017	0.957	0.834	1000	3.4	175.68	181.70	1000	11.4	0.0	0.0	8
MC7-135	MC7-135	12.0		0.0014	0.864	0.770	1000	3.4	175.93	181.40	600	11.5	0.0	0.0	8
MC7-140	MC7-140	12.0		0.0010	0.736	0.717	1000	3.3	176.15	180.60	600	11.9	0.0	0.0	9
MC7-145	MC7-145	12.0		0.0014	0.874	0.705	900	3.2	176.35	181.80	500	11.1	8.6	0.0	8
MC7-150	MC7-150	12.0		0.0014	0.849	0.634	900	2.9	176.73	182.50	500	10.8	10.7	0.0	7
MC7-155	MC7-155	12.0		0.0014	0.848	0.637	900	2.7	176.97	182.80	400	10.8	11.5	0.0	7
MC7-160	MC7-160	12.0		0.0012	0.812	0.610	900	2.6	177.09	182.30	400	10.8	0.0	0.0	7
MC7-165	MC7-165	12.0		0.0012	0.801	0.613	900	2.4	177.37	181.90	300	10.9	0.0	0.0	7
MC7-170	MC7-170	12.0		0.0017	0.935	0.585	900	2.1	177.56	182.30	200	10.1	10.5	0.0	6
MC7-175	MC7-175	12.0		0.0014	0.864	0.562	900	1.9	177.69	181.40	200	10.2	0.0	0.0	6
PS-2	PS-2	36.0		1.0000	430.871	0.192		0.9	177.69	168.30	(SPECIAL DEVICE AT NODE)				
MC7-180	MC7-180	12.0		0.0018	0.972	0.370	900	1.5	177.76	181.70	200	8.4	9.2	0.0	3
MC7-185	MC7-185	12.0		0.0015	0.899	0.348	900	1.1	177.84	181.50	200	8.4	0.0	0.0	3
PS-1	PS-1	36.0		1.0000	430.871	0.348		0.1	177.84	170.00	(SPECIAL DEVICE AT NODE)				
MC7-305	MC7-305	10.0		0.0053	1.026	0.015	900	0.0	167.65	181.50	0	2.0	1.9	0.0	
MC7-310	MC7-310	10.0		0.0024	0.693	0.015	900	0.1	168.87	182.40	0	2.4	2.5	0.0	

City of W ɔurn Facilities Plan
Sanitary s .er Collection System Analysis
Basin MC-7
1990 Run (PS=FM) (I/I=50%)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 4.3 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT. CAP.
MC7-405	MC7-405	10.0		0.0021	0.648	0.341	900	2.8	175.66	182.70	500	7.9	8.0	0.0	52.(
MC7-410	MC7-410	10.0		0.0015	0.557	0.319	900	2.3	175.87	181.10	400	8.1	0.0	0.0	57.:
MC7-415	MC7-415	10.0		0.0024	0.694	0.124	900	1.9	175.88	180.60	300	5.2	0.0	0.0	17.!
MC7-420	MC7-420	10.0		0.0027	0.742	0.097	900	1.3	175.89	181.00	200	4.7	5.0	0.0	13.(
MC7-425	MC7-425	10.0		0.0017	0.579	0.097	900	1.0	175.90	180.70	200	5.1	0.0	0.0	16.(
MC7-505	MC7-505	8.0		0.0038	0.482	0.182	900	1.6	175.98	181.90	200	5.5	5.6	0.0	37.(
MC7-510	MC7-510	8.0		0.0023	0.376	0.182	900	1.0	176.18	182.10	200	6.1	8.0	0.0	48.(
MC7-515	MC7-515	8.0		0.0027	0.409	0.183	900	0.3	176.70	182.60	0	5.9	6.9	0.0	44.(
PS-3	PS-3	36.0		1.0000	430.871	0.151		0.0	177.38	173.10	(SPECIAL DEVICE AT NODE)				

City of W burn Facilities Plan
San ry S er Collection System Analysis
Bas IC-6
1990 Run (PS=FM) (I/I=50%)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 4.3 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PC CA
MC6-005	MC6-005	18.0		0.0064	5.436	0.252	1000	0.2	155.16	167.80	0	5.7	5.2	0.0	4
MC6-010	MC6-010	18.0		0.0050	4.786	0.209	1000	0.2	157.17	165.70	0	5.6	0.0	0.0	4
MC6-015	MC6-015	18.0		0.0029	3.627	0.210	900	0.2	158.29	166.80	0	6.2	6.2	0.0	5
MC6-020	MC6-020	18.0		0.0047	4.667	0.210	900	0.2	158.60	167.00	0	5.6	6.3	0.0	4
MC6-025	MC6-025	18.0		0.0017	2.783	0.192	900	0.3	159.03	168.50	0	6.6	5.1	0.0	6
MC6-030	MC6-030	18.0		0.0021	3.133	0.192	900	0.2	159.40	172.10	0	6.3	4.2	0.0	6
MC6-035	MC6-035	18.0		0.0035	3.991	0.164	900	0.2	159.89	173.40	0	5.4	4.6	0.0	4
MC6-040	MC6-040	18.0		0.0025	3.412	0.164	900	0.2	160.36	174.00	0	5.8	5.5	0.0	4
MC6-045	MC6-045	18.0		0.0031	3.783	0.165	900	0.2	161.04	173.10	0	5.6	0.0	0.0	4
MC6-050	MC6-050	18.0		0.0029	3.647	0.166	900	0.2	161.82	173.80	0	5.6	5.8	0.0	4
MC6-055	MC6-055	18.0		0.0028	3.559	0.166	900	0.2	161.94	173.80	0	5.7	0.0	0.0	4
MC6-060	MC6-060	18.0		0.0029	3.645	0.166	900	0.2	162.81	174.40	0	5.7	6.1	0.0	4
MC6-105	MC6-105	10.0		0.0570	3.378	0.044	900	0.0	158.19	168.10	0	2.0	3.1	0.0	1
MC6-110	MC6-110	10.0		0.0160	1.791	0.033	900	0.1	163.63	175.40	0	2.2	2.1	0.0	1
MC6-115	MC6-115	10.0		0.0023	0.680	0.033	900	0.1	164.34	176.00	0	3.2	3.3	0.0	4
MC6-120	MC6-120	10.0		0.0029	0.756	0.014	900	0.1	165.49	179.20	0	2.2	1.9	0.0	1
MC6-125	MC6-125	10.0		0.0029	0.761	0.014	900	0.1	166.19	179.80	0	2.2	2.3	0.0	1
MC6-130	MC6-130	10.0		0.0028	0.755	0.014	900	0.1	166.58	180.50	0	2.2	2.0	0.0	1

City of W burn Facilities Plan
Sanitary s er Collection System Analysis
Basin MC-5
1990 Run (PS=FM) (I/I=50%)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 4.3 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT CAP
MC1-105	MC1-105	36.0		1.0000	430.871	0.687	900	0.0	149.40	165.40	0	3.2	0.0	0.0	0.
MC1-110	MC1-110	36.0		2.0500	616.912	0.145	900	0.0	150.43	165.80	0	1.6	0.0	0.0	0.
MC5-005	MC5-005	12.0		0.2516	11.545	0.687	900	0.2	155.83	166.00	0	4.2	6.5	0.0	5.
MC5-010	MC5-010	14.0		0.0209	5.022	0.687	900	0.3	157.34	172.80	0	6.6	4.9	0.0	13.
MC5-015	MC5-015	12.0		0.0332	4.193	0.688	900	0.3	165.75	175.80	0	6.1	7.4	0.0	16.
MC5-020	MC5-020	12.0		0.0020	1.040	0.548	900	0.5	166.72	180.40	0	9.4	6.7	0.0	52.
MC5-025	MC5-025	12.0		0.0021	1.057	0.491	900	0.5	167.58	181.70	0	9.0	8.4	0.0	46.
MC5-030	MC5-030	12.0		0.0163	2.937	0.363	900	0.2	168.05	182.80	0	5.5	5.0	0.0	12.
MC5-035	MC5-035	12.0		0.0013	0.816	0.366	900	0.5	168.90	183.00	0	8.9	11.0	0.0	44.
MC5-040	MC5-040	12.0		0.0018	0.965	0.329	900	0.4	169.73	183.90	0	8.0	8.0	0.0	34.
MC5-045	MC5-045	12.0		0.0029	1.230	0.257	900	0.3	170.24	184.30	0	6.7	7.2	0.0	20.
MC5-050	MC5-050	12.0		0.0027	1.193	0.257	900	0.3	170.56	184.60	0	6.8	6.8	0.0	21.
MC5-055	MC5-055	12.0		0.0030	1.254	0.214	900	0.3	171.26	185.40	0	6.2	6.1	0.0	17.
MC5-060	MC5-060	10.0		0.0037	0.858	0.074	900	0.2	172.87	184.80	0	4.0	0.0	0.0	8.
MC5-065	MC5-065	10.0		0.0031	0.793	0.075	900	0.2	174.45	183.80	0	4.1	0.0	0.0	9.
MC5-070	MC5-070	10.0		0.0041	0.904	0.075	900	0.2	175.34	183.40	0	3.9	0.0	0.0	8.
MC5-105	MC5-105	10.0		0.2368	6.888	0.145	900	0.1	156.41	164.60	0	2.3	0.0	0.0	2.
MC5-110	MC5-110	12.0		0.0466	4.968	0.145	900	0.1	165.99	174.50	0	3.2	3.2	0.0	2.
MC5-115	MC5-115	10.0		0.0105	1.448	0.078	900	0.1	169.63	176.80	0	3.3	3.6	0.0	5.
MC5-205	MC5-205	12.0		0.0051	1.647	0.051	900	0.1	167.26	178.10	0	3.3	2.7	0.0	3.
MC5-210	MC5-210	12.0		0.0095	2.243	0.051	900	0.1	169.50	179.40	0	2.9	3.2	0.0	2.
MC5-215	MC5-215	10.0		0.0095	1.377	0.051	900	0.1	171.80	180.40	0	2.9	3.4	0.0	3.

City of w urn Facilities Plan
Sanitary S er Collection System Analysis
Ba MC-4
19 un (PS=FM) (I/I=50%)
Pe. Sanitary and Base Flow = 4.3 mgd
Peak I/I = 4.3 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	P(C/
MC4-005	MC4-005	10.0		0.0297	2.441	0.642	900	0.3	156.45	172.20	0	6.1	5.9	0.0	2(
MC4-010	MC4-010	10.0		0.0548	3.313	0.423	900	0.2	162.87	170.10	0	4.6	0.0	0.0	1:
MC4-015	MC4-015	10.0		0.0137	1.655	0.423	900	0.3	164.30	175.80	0	6.0	4.6	0.0	2!
MC4-020	MC4-020	10.0		0.0153	1.749	0.395	900	0.3	165.74	177.40	0	5.7	5.6	0.0	2:
MC4-025	MC4-025	10.0		0.0039	0.881	0.378	900	0.4	167.40	179.60	0	7.3	6.8	0.0	4:
MC4-030	MC4-030	10.0		0.0034	0.823	0.349	900	0.4	168.76	181.30	0	7.2	7.0	0.0	4:
MC4-035	MC4-035	10.0		0.0036	0.853	0.320	900	0.4	170.19	183.00	0	6.9	6.7	0.0	3:
MC4-040	MC4-040	10.0		0.0025	0.711	0.263	900	0.4	171.44	182.20	0	6.9	0.0	0.0	3(
MC4-045	MC4-045	10.0		0.0030	0.777	0.228	900	0.3	172.84	181.20	0	6.3	0.0	0.0	2!
MC4-105	MC4-105	10.0		0.0290	2.411	0.203	900	0.2	166.63	180.40	0	4.0	4.1	0.0	8
MC4-110	MC4-110	10.0		0.0031	0.794	0.186	900	0.3	168.15	180.30	0	5.8	0.0	0.0	2:
MC4-115	MC4-115	10.0		0.0032	0.797	0.187	900	0.3	169.56	183.00	0	5.8	5.1	0.0	2:
MC4-120	MC4-120	10.0		0.0023	0.676	0.129	900	0.3	170.67	180.40	0	5.4	0.0	0.0	1!
MC4-125	MC4-125	10.0		0.0060	1.092	0.129	900	0.2	171.86	181.70	0	4.5	4.5	0.0	1:
MC4-130	MC4-130	10.0		0.0025	0.709	0.084	900	0.2	172.55	183.40	0	4.5	3.8	0.0	1:
MC4-135	MC4-135	10.0		0.0070	1.180	0.084	900	0.1	173.23	183.90	0	3.7	4.0	0.0	7
MC4-140	MC4-140	10.0		0.0025	0.712	0.084	900	0.2	174.12	183.90	0	4.5	0.0	0.0	1:
MC4-145	MC4-145	10.0		0.0025	0.710	0.039	900	0.1	174.73	184.70	0	3.4	3.3	0.0	5
MC4-205	MC4-205	10.0		0.1346	5.192	0.063	900	0.0	165.62	171.70	0	1.9	2.1	0.0	1
MC4-210	MC4-210	10.0		0.0050	1.003	0.063	900	0.1	167.05	173.80	0	3.5	3.3	0.0	6
MC4-215	MC4-215	10.0		0.0056	1.055	0.044	900	0.1	168.52	175.50	0	3.0	3.0	0.0	4
MC4-220	MC4-220	10.0		0.0046	0.958	0.019	2300	0.1	170.01	177.30	0	2.3	2.2	0.0	2

City of W burn Facilities Plan
Sanitary S er Collection System Analysis
Basin MC-3
1990 Run (PS=FM) (I/I=50%)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 4.3 mgd (Distributed)
Total Peak Flow = 12.9 mgd

03/28/93 12:06

PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT. CAP.
MC3-005	MC3-005	18.0		0.0083	6.195	2.794	1000	0.7	145.20	157.70	0	13.4	11.7	0.0	45.1
MC3-010	MC3-010	12.0		0.0247	3.615	2.794	1000	0.6	145.87	159.10	0	10.9	9.7	0.0	77.3
MC3-015	MC3-015	16.0		0.0626	12.399	2.795	1000	0.5	153.95	157.30	0	9.2	0.0	0.0	22.5
MC3-020	MC3-020	16.0		0.0021	2.283	2.795	1000	1.8	156.36	160.00	600	17.3	14.4	9.1	122.4
MC3-025	MC3-025	16.0		0.0084	4.546	2.714	1000	0.7	158.31	166.50	0	13.2	11.5	0.0	59.7
MC3-030	MC3-030	18.0		0.0084	6.206	2.715	1000	0.7	159.15	172.90	0	13.2	9.1	0.0	43.7
MC3-035	MC3-035	14.0		0.0049	2.438	2.715	1000	1.4	160.92	169.00	300	14.6	0.0	6.2	111.3
MC3-040	MC3-040	14.0		0.0049	2.420	2.201	1000	1.2	161.77	169.80	200	13.5	14.1	0.0	90.9
MC3-045	MC3-045	12.0		0.0068	1.899	2.201	1000	2.0	164.86	174.90	400	12.7	10.9	6.0	115.9
MC3-050	MC3-050	12.0		0.0072	1.947	2.201	1000	2.7	167.72	180.00	300	12.6	10.8	5.6	113.0
MC3-055	MC3-055	16.0		0.0025	2.478	2.201	1000	2.7	167.76	180.40	300	15.3	10.4	0.0	88.8
MC3-060	MC3-060	16.0		0.0018	2.104	2.111	1000	2.7	168.50	180.60	300	16.0	20.4	1.8	100.3
MC3-065	MC3-065	16.0		0.0012	1.730	1.943	1000	2.7	168.74	180.20	300	16.7	0.0	7.3	112.3
MC3-070	MC3-070	16.0		0.0007	1.267	1.838	1000	2.9	169.07	180.60	1500	18.4	15.5	11.9	145.0
MC3-075	MC3-075	15.0		0.0004	0.809	1.772	1000	3.3	169.55	177.40	2400	20.1	0.0	16.0	219.0
MC3-080	MC3-080	16.0		0.0010	1.602	1.672	1000	3.3	169.84	179.10	1500	16.3	11.4	4.9	104.3
MC3-085	MC3-085	16.0		0.0004	0.937	1.608	1000	3.5	170.10	179.80	1800	19.6	13.3	14.1	171.6
MC3-090	MC3-090	16.0		0.0018	2.124	1.557	1000	3.3	170.26	178.00	700	14.2	0.0	0.0	73.3
MC3-095	MC3-095	16.0		0.0016	1.954	1.557	1000	3.2	170.54	177.90	400	14.7	0.0	0.0	79.6
MC3-100	MC3-100	16.0		0.0014	1.843	1.564	900	3.0	170.87	180.00	200	15.0	11.4	0.0	84.8
MC3-105	MC3-105	10.0		0.0037	0.860	1.018	900	3.4	172.18	182.10	400	10.7	9.2	5.3	118.3
MC3-110	MC3-110	10.0		0.0030	0.775	0.955	900	3.8	173.34	183.40	500	10.8	9.8	5.8	123.1
MC3-115	MC3-115	10.0		0.0027	0.737	0.899	900	4.1	174.36	182.30	500	10.8	0.0	5.7	121.8
MC3-120	MC3-120	10.0		0.0022	0.670	0.824	900	4.4	175.40	181.80	500	10.8	0.0	5.8	122.8
MC3-125	MC3-125	10.0		0.0030	0.775	0.826	900	4.5	175.87	182.50	500	10.2	9.3	3.6	106.5
MC3-130	MC3-130	10.0		0.0023	0.676	0.826	900	4.8	176.84	182.50	500	10.8	0.0	5.7	122.1
MC3-135	MC3-135	10.0		0.0036	0.845	0.814	900	4.7	177.81	183.60	400	9.9	9.8	0.0	96.2
MC3-140	MC3-140	10.0		0.0038	0.872	0.736	900	4.4	178.51	183.30	400	9.4	0.0	0.0	84.3
MC3-145	MC3-145	10.0		0.0044	0.937	0.737	900	3.9	179.19	182.70	200	9.1	0.0	0.0	78.6
MC3-150	MC3-150	10.0		0.0040	0.896	0.674	900	3.2	180.06	180.90	200	9.0	0.0	0.0	75.1
MC3-155	MC3-155	8.0		0.0040	0.494	0.674	900	4.8	183.56	181.90	1600	9.0	10.1	5.5	136.5
MC3-160	MC3-160	8.0		0.0040	0.494	0.674	900	5.2	184.31	182.80	1600	9.0	7.7	5.5	136.5
MC3-205	MC3-205	10.0		0.0007	0.377	0.521	900	1.6	161.22	175.00	1600	11.3	5.7	7.0	137.9
MC3-210	MC3-210	10.0		0.0008	0.393	0.426	900	1.6	161.27	174.30	1500	10.3	0.0	4.0	108.5
MC3-215	MC3-215	10.0		0.0007	0.368	0.426	900	1.6	161.48	174.30	700	10.6	0.0	5.0	115.9
MC3-220	MC3-220	10.0		0.0007	0.379	0.355	900	1.6	161.67	173.50	500	9.8	0.0	0.0	93.7
MC3-225	MC3-225	10.0		0.0041	0.901	0.215	900	1.2	161.70	173.80	300	5.8	6.3	0.0	23.9
MC3-230	MC3-230	10.0		0.0021	0.649	0.216	900	0.3	161.91	175.40	0	6.6	6.1	0.0	33.3
MC3-305	MC3-305	15.0		0.0180	5.593	0.170	900	0.1	170.63	182.50	0	4.0	4.8	0.0	3.0
MC3-310	MC3-310	12.0		0.0045	1.544	0.148	900	0.2	171.89	182.60	0	5.0	7.9	0.0	9.5
MC3-315	MC3-315	10.0		0.0102	1.426	0.134	900	0.2	174.55	183.20	0	4.1	5.5	0.0	9.4

City of W burn Facilities Plan
Sanitary er Collection System Analysis
Basin MC-3
19 un (PS=FM) (I/I=50%)
Pe anitary and Base Flow = 4.3 mgd
Peak I/I = 4.3 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCI CAI
MC3-320	MC3-320	16.0		0.0211	7.203	0.107	900	0.1	171.59	181.10	0	3.3	4.6	0.0	1.
MC3-325	MC3-325	10.0		0.0036	0.847	0.095	900	0.2	172.64	181.90	0	4.4	4.5	0.0	11.
MC3-330	MC3-330	10.0		0.0265	2.306	0.067	900	0.1	173.17	181.00	0	2.7	4.5	0.0	2.
MC3-335	MC3-335	10.0		0.0043	0.933	0.046	900	0.1	174.33	182.30	0	3.2	3.1	0.0	4.
MC3-340	MC3-340	12.0		0.0208	3.319	0.103	900	0.1	171.90	180.00	0	3.3	3.8	0.0	3.
MC3-345	MC3-345	10.0		0.0052	1.016	0.055	900	0.1	173.26	181.30	0	3.3	3.4	0.0	5.
MC3-350	MC3-350	14.0		0.0218	5.122	0.065	900	0.1	172.26	178.90	0	2.7	0.0	0.0	1.
MC3-355	MC3-355	14.0		0.0040	2.185	0.046	900	0.1	173.32	180.00	0	3.3	3.3	0.0	2.
MC3-360	MC3-360	12.0		0.0042	1.483	0.046	900	0.1	174.42	180.40	0	3.3	3.9	0.0	3.
MC3-365	MC3-365	10.0		0.0059	1.086	0.046	900	0.1	175.95	182.40	0	3.1	2.9	0.0	4.
MC3-370	MC3-370	10.0		0.0260	2.281	0.053	900	0.1	173.32	181.00	0	2.4	3.4	0.0	2.
MC3-405	MC3-405	12.0		0.0018	0.971	0.553	1000	2.5	171.13	179.50	200	9.7	0.0	0.0	56.
MC3-410	MC3-410	10.0		0.0019	0.621	0.439	1000	2.0	171.64	181.60	200	8.8	7.7	0.0	70.
MC3-415	MC3-415	10.0		0.0018	0.602	0.439	1000	1.6	172.09	182.60	200	8.9	8.6	0.0	72.
MC3-420	MC3-420	10.0		0.0018	0.606	0.381	1000	1.1	172.40	182.30	200	8.4	0.0	0.0	62.
MC3-425	MC3-425	10.0		0.0029	0.762	0.382	900	0.4	172.95	184.20	0	7.7	7.1	0.0	50.
MC3-430	MC3-430	10.0		0.0031	0.793	0.383	900	0.4	173.63	183.90	0	7.6	0.0	0.0	48.
MC3-435	MC3-435	10.0		0.0005	0.314	0.311	900	0.7	174.17	183.80	0	10.0	0.0	0.0	98.
MC3-440	MC3-440	10.0		0.0021	0.650	0.153	900	0.3	174.68	183.30	0	5.8	0.0	0.0	23.
MC3-445	MC3-445	10.0		0.0017	0.589	0.113	900	0.3	175.04	183.60	0	5.4	5.6	0.0	19.
MC3-450	MC3-450	10.0		0.0019	0.615	0.073	900	0.2	175.48	184.00	0	4.5	4.7	0.0	11.
MC3-455	MC3-455	10.0		0.0035	0.843	0.035	900	0.1	176.49	183.70	0	3.0	0.0	0.0	4.
PS-10	PS-10	36.0		1.0000	430.871	0.000		0.0	177.39	154.00	(SPECIAL DEVICE AT NODE)				
MC3-505	MC3-505	8.0		0.0080	0.698	0.136	900	0.2	174.38	183.00	0	4.3	0.0	0.0	19.4
MC3-510	MC3-510	8.0		0.0044	0.518	0.136	900	0.2	174.98	182.10	0	4.8	0.0	0.0	26.2
MC3-515	MC3-515	8.0		0.0075	0.678	0.136	900	0.2	178.29	183.60	0	4.4	5.1	0.0	20.1
MC3-520	MC3-520	8.0		0.0000	0.035	0.136	900	0.8	178.89	183.60	2400	13.3	0.0	11.9	390.5
MC3-525	MC3-525	8.0		0.0031	0.433	0.137	900	0.3	179.73	183.90	0	5.2	6.9	0.0	31.6
PS-6	PS-6	36.0		1.0000	430.871	0.087		0.0	180.46	172.40	(SPECIAL DEVICE AT NODE)				
MC3-605	MC3-605	12.0		0.0013	0.836	0.000	200	2.4	170.87	182.50	200	0.0	0.0	0.0	0.0
MC3-610	MC3-610	12.0		0.0013	0.832	0.000	0	1.9	170.87	182.00	200	0.0	0.0	0.0	0.0

City of W burn Facilities Plan
Sanitary S. er Collection System Analysis
Basin MC-2
1990 Run (PS=FM) (I/I=50%)
Peak Sanitary and Base Flow = 4.3 mgd
Peak I/I = 4.3 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	PCT. CAP.
MC2-005	MC2-005	18.0		0.2253	32.211	1.228	1000	0.2	159.72	179.00	0	5.3	5.2	0.0	3.81
MC2-010	MC2-010	18.0		0.0014	2.502	1.228	1000	0.7	160.57	180.20	0	13.8	10.5	0.0	49.08
MC2-015	MC2-015	18.0		0.0008	1.879	1.228	1000	0.9	160.95	179.80	0	15.3	0.0	0.0	65.35
MC2-020	MC2-020	18.0		0.0008	1.887	1.230	900	0.9	161.24	179.40	0	15.3	0.0	0.0	65.20
MC2-025	MC2-025	18.0		0.0011	2.267	1.234	900	0.8	161.43	179.70	0	14.3	14.1	0.0	54.44
MC2-030	MC2-030	18.0		0.0011	2.237	1.112	900	0.7	161.94	180.00	0	13.8	15.5	0.0	49.69
MC2-035	MC2-035	18.0		0.0019	2.958	0.285	900	0.3	162.46	181.20	0	7.5	7.2	0.0	9.64
MC2-040	MC2-040	12.0		0.0013	0.836	0.205	900	0.4	163.16	181.70	0	7.1	7.5	0.0	24.54
MC2-045	MC2-045	12.0		0.0017	0.941	0.207	900	0.3	164.00	181.70	0	6.8	0.0	0.0	22.00
MC2-050	MC2-050	12.0		0.0018	0.984	0.134	900	0.3	164.87	182.70	0	5.7	5.6	0.0	13.62
MC2-105	MC2-105	10.0		0.0052	1.025	0.832	900	0.6	164.70	181.80	0	9.2	10.1	0.0	81.20
MC2-110	MC2-110	10.0		0.0076	1.232	0.749	900	0.5	166.72	181.10	0	8.3	0.0	0.0	60.80
MC2-115	MC2-115	10.0		0.0030	0.777	0.681	900	0.6	168.21	180.00	0	9.5	0.0	0.0	87.64
MC2-120	MC2-120	10.0		0.0031	0.782	0.530	900	0.5	169.33	180.00	0	8.6	0.0	0.0	67.84
MC2-125	MC2-125	10.0		0.0031	0.787	0.464	900	0.5	170.37	180.00	0	8.2	0.0	0.0	58.95
MC2-130	MC2-130	10.0		0.0030	0.771	0.465	900	0.5	171.27	178.00	0	8.3	0.0	0.0	60.34
PS-9	PS-9	36.0		1.0000	430.871	0.333		0.0	171.82	157.30	(SPECIAL DEVICE AT NODE)				

City of W burn Facilities Plan
Sanitary S er Collection System Analysis
Ba MC-1
19 un (PS=FM) (I/I=50%)
Pe Sanitary and Base Flow = 4.3 mgd
Peak I/I = 4.3 mgd (Distributed)
Total Peak Flow = 12.9 mgd

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PEAK FLOWS AND HYDRAULIC GRADE LINES

NODE	DISCH. PIPE	SIZE (IN.)	SHAPE CODE	SLOPE	CAPACITY (MGD)	PEAK Q AT NODE (MGD)	TIME (MIL)	MAX DEPTH (FT.)	MAX HGL (FT.)	LID ELEV (FT.)	SURCH. DURATION (HHMM)	REQ. DIA. AT PIPE SLOPE(IN.)	REQ. DIA. AT GROUND SLOPE(IN.)	OVFLW PIPE REQ. DIA. AT PIPE SLOPE(IN.)	P(C/
PS-MC	PS-MC	36.0		1.0000	430.871	8.057		0.2	182.77	136.60	(SPECIAL DEVICE AT NODE)				
MC1-005	MC1-005	30.0		0.0116	28.492	6.112	1000	0.8	138.36	157.50	0	16.8	7.9	0.0	21
MC1-010	MC1-010	30.0		0.0116	28.488	6.114	1000	0.8	143.03	159.30	0	16.8	20.1	0.0	21
MC1-015	MC1-015	36.0		0.0142	51.284	4.886	1000	0.7	143.03	159.30	0	14.9	0.0	0.0	9
MC1-020	MC1-020	36.0		0.0010	13.939	4.888	1000	1.3	143.82	153.30	0	24.3	0.0	0.0	35
MC1-025	MC1-025	36.0		0.0007	11.620	4.892	1000	1.4	144.13	153.60	0	26.0	24.1	0.0	42
MC1-030	MC1-030	36.0		0.0001	5.117	4.931	1000	2.4	145.21	154.10	0	35.5	26.3	0.0	96
MC1-035	MC1-035	36.0		0.0021	19.634	4.849	1000	1.6	145.21	155.30	0	21.3	19.6	0.0	24
MC1-040	MC1-040	36.0		0.0012	14.848	4.851	1000	1.2	145.22	156.70	0	23.7	18.5	0.0	32
MC1-045	MC1-045	27.0		0.0016	7.973	2.057	1000	1.0	145.22	156.00	0	16.2	0.0	0.0	25
MC1-050	MC1-050	27.0		0.0014	7.486	2.058	1000	0.8	145.39	160.60	0	16.6	9.8	0.0	27
MC1-055	MC1-055	27.0		0.0013	7.306	2.058	1000	0.9	145.52	165.20	0	16.8	8.5	0.0	28
MC1-060	MC1-060	27.0		0.0017	8.235	2.058	1000	0.8	145.98	162.00	0	16.1	0.0	0.0	24
MC1-065	MC1-065	27.0		0.0017	8.236	2.059	1000	0.8	146.69	158.70	0	16.1	0.0	0.0	25
MC1-070	MC1-070	27.0		0.0047	13.783	2.059	1000	0.6	146.78	160.30	0	13.2	9.5	0.0	14
MC1-075	MC1-075	27.0		0.0012	6.825	1.915	1000	0.9	147.34	160.80	0	16.8	15.4	0.0	28
MC1-080	MC1-080	27.0		0.0011	6.756	1.915	1000	0.9	147.60	161.20	0	16.8	15.5	0.0	28
MC1-085	MC1-085	24.0		0.0046	9.955	1.216	1000	0.5	147.81	159.00	0	10.9	0.0	0.0	12
MC1-090	MC1-090	24.0		0.0040	9.222	1.216	1000	0.5	148.73	160.50	0	11.2	10.2	0.0	13
MC1-095	MC1-095	24.0		0.0011	4.921	1.216	1000	0.7	149.38	161.40	0	14.2	12.4	0.0	24
MC1-100	MC1-100	24.0		0.0015	5.577	1.219	900	0.7	149.78	161.80	0	13.6	13.9	0.0	21
MC1-105	MC1-105	24.0		0.0008	4.094	1.225	900	0.8	150.17	165.40	0	15.3	9.4	0.0	29
MC1-110	MC1-110	24.0		0.0100	14.614	0.539	900	0.2	150.65	165.80	0	7.0	8.3	0.0	3
MC1-115	MC1-115	18.0		0.0110	7.103	0.396	1000	0.2	151.68	164.10	0	6.1	0.0	0.0	5
MC1-120	MC1-120	18.0		0.0064	5.431	0.389	1000	0.3	154.02	165.90	0	6.7	7.0	0.0	7
MC1-125	MC1-125	15.0		0.0036	2.521	0.138	900	0.2	154.48	166.00	0	5.0	6.9	0.0	5
MC1-130	MC1-130	15.0		0.0085	3.841	0.138	900	0.1	155.05	164.10	0	4.3	0.0	0.0	3
MC1-135	MC1-135	10.0		0.0507	3.188	0.138	900	0.1	158.46	165.50	0	3.1	3.7	0.0	4
MC1-140	MC1-140	8.0		0.0033	0.446	0.139	900	0.3	159.23	165.40	0	5.2	0.0	0.0	31
MC1-145	MC1-145	8.0		0.0094	0.759	0.139	900	0.2	163.43	175.50	0	4.2	3.6	0.0	18
MC1-150	MC1-150	8.0		0.0040	0.492	0.111	900	0.2	164.43	170.00	0	4.6	0.0	0.0	22
MC1-155	MC1-155	8.0		0.0049	0.545	0.076	900	0.2	165.69	174.10	0	3.8	3.1	0.0	13
MC1-160	MC1-160	8.0		0.0067	0.639	0.054	900	0.1	166.21	170.80	0	3.2	0.0	0.0	8
MC1-165	MC1-165	8.0		0.0029	0.422	0.054	900	0.2	166.95	172.20	0	3.7	3.3	0.0	12
MC1-170	MC1-170	8.0		0.0026	0.398	0.054	900	0.2	167.08	170.60	0	3.8	0.0	0.0	13
MC1-175	MC1-175	8.0		0.0029	0.424	0.029	900	0.1	167.70	172.70	0	2.9	2.4	0.0	6
MC1-180	MC1-180	8.0		0.0034	0.452	0.029	900	0.1	169.17	173.30	0	2.9	3.4	0.0	6

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CH2M HILL

PREPARED FOR: Frank Tiwari/City of Woodburn

PREPARED BY: Jay Holtz/CH2M HILL

COPIES: Dean Morrison/City of Woodburn
Daria Wightman/CH2M HILL

DATE: June 2, 1993

SUBJECT: Infiltration and Inflow Analysis

PROJECT: PDX27874.F4

An infiltration and inflow (I/I) analysis was conducted by CH2M HILL to determine the potential cost-effectiveness of reducing I/I rates in the City of Woodburn's sanitary sewerage collection system. This technical memorandum presents the goals and objectives of the I/I analysis, current I/I rates, and an assessment of whether I/I reduction would be cost-effective.

Goals and Objectives

Collection systems designed to convey wastewater inevitably also convey a certain quantity of extraneous flow known as I/I, which can originate as groundwater or storm-water runoff. Sources of direct inflow of storm water into the collection system include connected catch basins or roof drains and submerged manhole covers. Sources that allow infiltration of storm water or groundwater into the collection system include broken pipes or defective pipe joints, foundation drains connected to the system, and defective service connections.

The entry of groundwater and storm-water runoff into the sanitary sewerage collection system increases the costs of operation and can lead to overloaded pipelines and pump stations. The result may be overflow of raw sewage into streets, nearby creeks, or other bodies of water. In addition to the health hazards created by raw sewage overflows, excessive I/I is an indicator of a leaking and deteriorating collection system.

The presence of I/I reduces collection system carrying capacity and treatment plant process performance capabilities. As a result, I/I increases the need for relief sewer lines, pump stations, and treatment units. Operation and maintenance (O&M) costs for pumping, energy, and chemical consumption at treatment facilities are also increased by excessive flow.

The objective of the I/I analysis was to identify portions of the system with excessive I/I flows and to develop and compare rough cost estimates for I/I reduction, conveyance, and treatment. The results of the analysis may be used to help determine and schedule collection system improvements to remove or decrease I/I and to prioritize system maintenance efforts.

Existing I/I Rates

Existing I/I rates for the Woodburn sanitary sewerage collection system were estimated from wet- and dry-weather flow data gathered by the City of Woodburn staff and analyzed by CH2M HILL. A description of the flow monitoring program is presented in Technical Memorandum F4-1. Analysis of the data resulted in the peak wet-weather flow components shown in Table 1 and estimated drainage area I/I rates shown in Table 2. Infiltration could not be reliably distinguished from inflow because hourly rainfall data were not available for the monitoring periods.

Table 1
Wet-Weather Flow Components

	Average Daily Wet-Weather Flow (mgd)	Maximum Daily Wet-Weather Flow (mgd)	Instantaneous Peak Wet-Weather Flow (mgd)
Sanitary Flow	0.99	0.99	2.38
Base Flow	0.81	0.81	0.81
I/I	0.30	6.53	8.81
Total Flow	2.10	8.33	12.00

mgd = million gallons per day.

Table 2
Drainage Area I/I Rates

Drainage Area	Peak I/I Rate (mgd)	Peak I/I Rate (gpad)
West Woodburn and West of I-5	1.76	4,155
North Woodburn	1.83	5,207
Central Downtown Woodburn	1.41	8,232
South Woodburn	0.37	1,536
Southeast Woodburn	1.65	3,694
Northeast Woodburn	1.79	3,961
Total	8.81	
Weighted Average		4,222

gpad = gallons per acre per day.

The flow data show that the maximum daily wet-weather flow is about four times greater than the average daily wet-weather flow and the instantaneous peak wet-weather flow is about six times the average daily wet-weather flow. The collection system in the central downtown area of Woodburn exhibits an I/I rate that is almost twice the system average, and the system in north Woodburn also shows a greater than average I/I rate.

Cost-Effectiveness Analysis

A cost-effectiveness analysis was conducted to determine whether I/I could be economically eliminated from the Woodburn collection system. The analysis compares the estimated costs of I/I reduction and continued conveyance and treatment, with the result that the least expensive combination of rehabilitation, conveyance, and treatment represents the cost-effective solution to I/I management.

I/I Reduction Costs

Reduction of extraneous flow requires identification of actual I/I sources and subsequent sewer system rehabilitation as justified. A sewer system evaluation survey (SSES) is usually conducted to identify I/I sources. The SSES and sewer system rehabilitation techniques are described below with estimates of their associated costs.

Sewer System Evaluation Survey

The SSES is a systematic survey of the collection system undertaken to establish the type, location, and flow rate of specific sources of I/I. The estimated cost of eliminating or reducing each source and the subsequent identification of sources of excessive I/I are also determined as part of the SSES. SSES efforts may be guided by the results of an I/I analysis.

Basic SSES costs may range from $0.65 to $0.90 per linear foot of pipeline. The cost includes smoke testing, visual inspections, limited cleaning and television inspections, recommendations for rehabilitation, and a 35 percent contingency.

Only the trunk and interceptor lines were inventoried as part of the Woodburn collection system evaluation. However, if it is assumed that trunk and lateral lines make up about 20 percent of the system on a linear foot basis, the cost of an SSES would be approximately $245,000 for the entire Woodburn service area.

Source Removal and Collection System Rehabilitation

Sources of excessive inflow identified by the SSES should be removed from the collection system. Inflow sources may include catch basin connections, area drains, abandoned service connections, foundation drains, roof drain connections, and submerged manhole

covers. Inflow source removal is generally very effective for reducing total I/I if the collection system in the area is otherwise in good condition.

If excessive infiltration is the result of dilapidated and leaking manholes, pipelines, or service laterals, these facilities should be rehabilitated. Leaking pipe joints, broken or cracked pipe, and leaky service laterals and manholes may be sealed, grouted, lined, or replaced. The amount of infiltration removed by such rehabilitation measures varies because of groundwater migration and other factors.

Collection system rehabilitation costs were estimated for four levels of I/I reduction: zero, 25, 50, and 75 percent. A 100 percent reduction is considered nearly impossible to achieve without complete renovation of the collection system. Table 3 shows the estimated extent of source removal, manhole and pipeline grouting and sealing, system replacement, and service lateral replacement required to achieve each level of I/I reduction.

Table 3
System Rehabilitation Required for I/I Reduction

I/I Reduction (%)	Source Removal (%)	Grout and Seal (%)	System Replacement (%)	Service Replacement (%)
0	0	0	0	0
25	100	25	10	10
50	100	50	25	25
75	100	50	50	50

Unit costs were estimated for each system rehabilitation method. The cost of source removal was estimated at $2,500 per source. The number of sources was approximated based on pipeline lengths and existing I/I rates. The cost of grouting and sealing was estimated at $4.25 per linear foot, and it was assumed that one in four joints would require grouting and pressure testing. System replacement costs were estimated from U.S. Environmental Protection Agency (EPA) wastewater conveyance construction cost data at $125 per linear foot. The cost of service lateral replacement was estimated at $1,200 per service connection. All costs include a 35 percent contingency. The resulting costs for the various levels of I/I reduction are shown in Table 4.

Conveyance and Treatment Costs

If I/I is not removed from the collection system, it will be conveyed to the wastewater treatment plant and combined with other process flows. Conveyance and treatment costs were estimated for the same four levels of I/I reduction discussed above and include capital costs for wastewater conveyance and treatment, present worth costs for system O&M,

and a 35 percent contingency. Present worth costs are estimated for a 20-year period and a 6-percent interest rate.

Table 4
Estimated Costs for Source Removal and Collection System Rehabilitation

I/I Reduction (%)	Total Peak Flow (mgd)	Source Removal Cost ($)	Grout and Seal Cost ($)	System Replacement Cost ($)	Service Replacement Cost ($)	Total Cost ($)
0	12.00	0	0	0	0	0
25	9.80	95,000	335,000	3,940,000	720,000	5,090,000
50	7.60	95,000	670,000	9,840,000	1,800,000	12,405,000
75	5.39	95,000	670,000	19,690,000	3,605,000	24,060,000

Pipelines and Pump Stations

The collection system was evaluated with lower I/I rates to determine whether the resulting decrease in hydraulic loading would eliminate or reduce the need for system improvements described in Technical Memorandum F4-2. The analysis showed that a 50 percent reduction in I/I would delay the expansion of the pump stations on Rainer Road and Industrial Avenue but would not eliminate the need for other system improvements. Pipelines in the MC-3 and MC-7 drainage basins and the pump station on Jansen Way would still require improvement. However, some cost savings may result from the use of smaller relief lines and pumps to handle the reduced flow.

Pipeline and pump station capital and O&M costs were estimated from EPA cost curves. Table 5 shows the estimated costs for conveyance for the various levels of I/I reduction.

Table 5
Estimated Costs for Conveyance

I/I Reduction (%)	Total Peak Flow (mgd)	Pumping Capital Cost ($)	Pipeline Capital Cost ($)	Conveyance O&M Cost ($)	Total Cost ($)
0	12.00	55,000	415,000	35,000	505,000
25	9.80	10,000	310,000	25,000	345,000
50	7.60	5,000	210,000	20,000	235,000
75	5.39	0	105,000	10,000	115,000

Wastewater Treatment Plant

Capital and O&M cost estimates for treatment were based on the incremental cost to treat combined sanitary and I/I flows under the four I/I reduction scenarios. Capital costs include expansion of the hydraulic capacity of the Woodburn wastewater treatment plant. O&M costs include labor, materials, supplies, power, and chemicals for those process units affected by flow. Table 6 shows the estimated costs for treatment for the various levels of I/I reduction.

Table 6
Estimated Costs for Treatment

I/I Reduction (%)	Total Peak Flow (mgd)	Capital Cost ($)	O&M Cost ($)	Total Cost ($)
0	12.00	16,720,000	140,000	16,860,000
25	9.80	10,520,000	120,000	10,640,000
50	7.60	5,120,000	90,000	5,210,000
75	5.39	0	60,000	60,000

Cost-Effective I/I Management

The cost-effective solution to I/I management in the Woodburn collection system is to remove approximately 2 mgd of I/I and continue to convey and treat the remainder. Table 7 summarizes the costs of I/I reduction and conveyance and treatment. As the table shows, the least expensive option on the basis of total cost is 25 percent I/I reduction. Figure 1 graphically illustrates the cost relationships shown in the table.

Table 7
Summary of Estimated Costs for I/I Management

I/I Reduction (%)	Total Peak Flow (mgd)	I/I Reduction Cost ($)	Conveyance and Treatment Cost ($)	Total Cost ($)
0	12.00	0	17,365,000	17,365,000
25	9.80	5,335,000	10,985,000	16,320,000
50	7.60	12,650,000	5,445,000	18,095,000
75	5.39	24,305,000	175,000	24,480,000

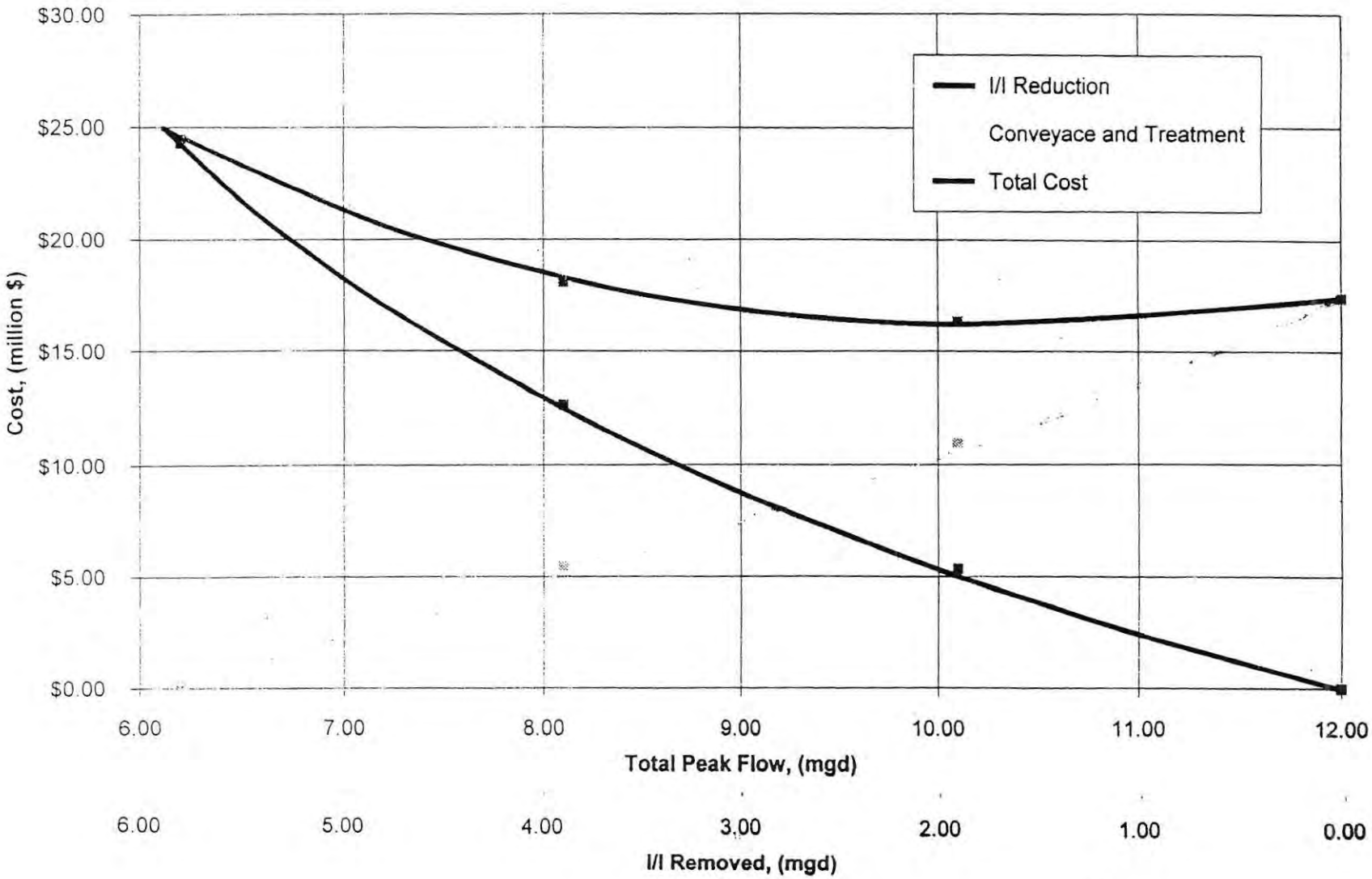

FIGURE 1
COST EFFECTIVE I/I REDUCTION
CITY OF WOODBURN FACILITIES PLAN

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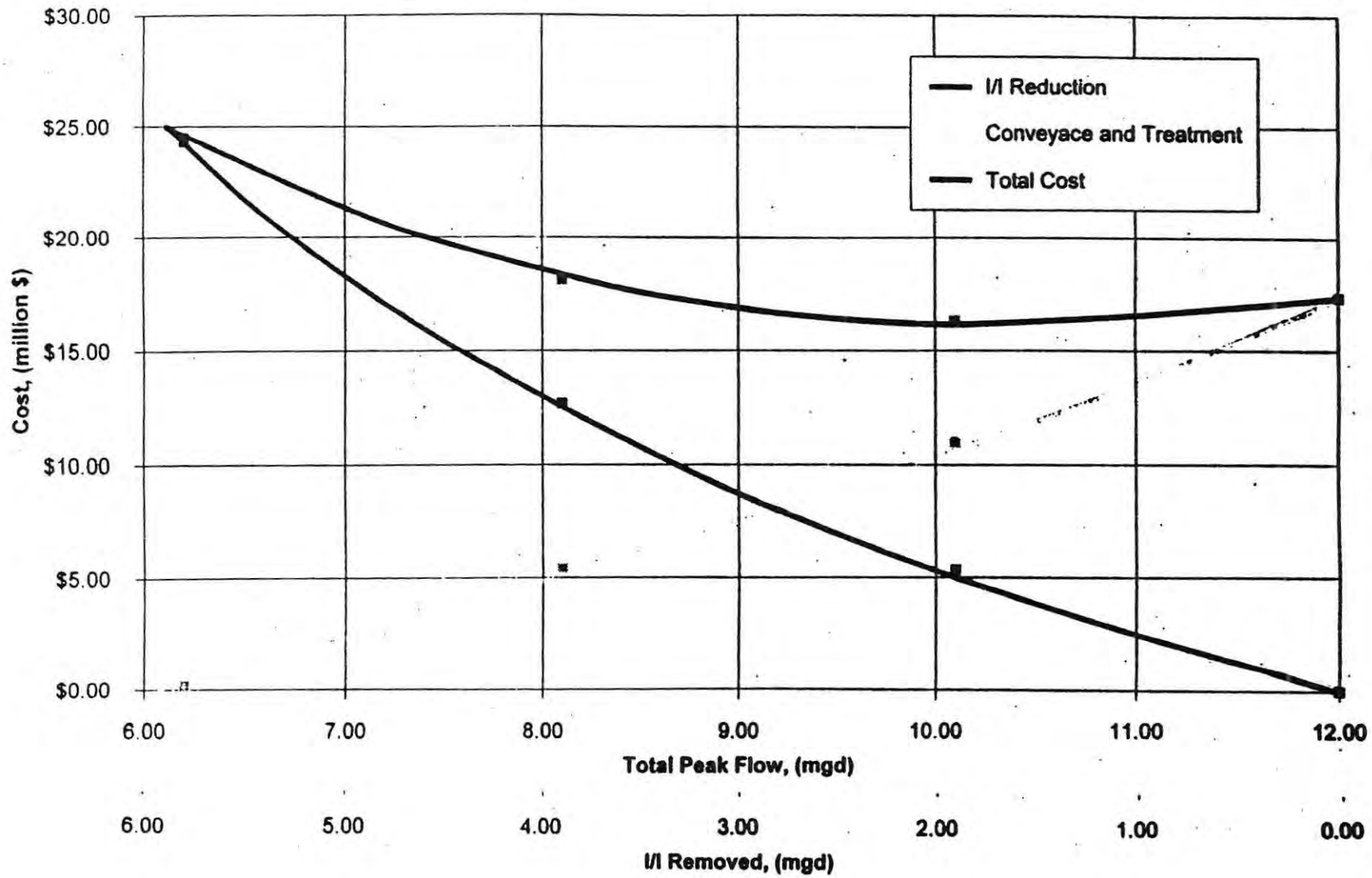

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FIGURE 1
COST EFFECTIVE I/I REDUCTION
CITY OF WOODBURN FACILITIES PLAN

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With a 25 percent reduction in I/I, the total current peak flow will decrease from 12 to 10 mgd. The cost associated with I/I reduction will be $5,335,000 and the cost of conveyance and treatment will be $10,985,000.

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PREPARED BY: Jay Holtz/CH2M HILL

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DATE: June 2, 1993

SUBJECT: Infiltration and Inflow Reduction and Control Program

PROJECT: PDX27874.F4

An infiltration and inflow (I/I) reduction and control program has been outlined by CH2M HILL to help direct the efforts of the City of Woodburn sewerage system maintenance staff. This technical memorandum discusses goals and objectives, current practices, and recommendations for I/I reduction and control.

Goals and Objectives

The results of the I/I analysis presented in Technical Memorandum F4-3 indicate that a 2-million-gallon-per-day (mgd) reduction in the current peak I/I rate may be cost-effective. Interviews with the sewer system maintenance staff and the results of the flow monitoring program presented in Technical Memorandum F4-1 support the conclusions of the I/I analysis. City staff should focus I/I reduction efforts on discrete areas of the system and make an effort to control I/I within newly constructed lines, so that the system can continue to operate efficiently into the future.

The maintenance staff believes that most I/I in the Woodburn system originates at privately owned sewer connections. Most of the publicly owned portion of the collection system is thought to be in good condition and relatively watertight, as evidenced by staff experience and existing television inspection logs. The few publicly owned pipelines in the collection system that may be admitting very high rates of infiltration serve First Street, Montgomery Street, Arthur Street, and Lincoln Street. These pipelines are older and known to be in poorer condition than the rest of the system. The results of the flow monitoring program show that the peak I/I rate from the central downtown area of Woodburn where these pipelines are located is almost twice the system average.

System maintenance staff also mentions the following locations where a sanitary/storm sewer cross connection may be active and admitting very high rates of inflow:

- Near Conroy Packing
- Fifth Street near St. Luke's Church
- At the intersection of First Street and Harrison Street

No catch basins are known to be connected to the sanitary sewerage system. Although a few roof and foundation drains are probably connected, there is no known area where roof and foundation drains are predominately connected to the system.

Current I/I Reduction and Control Practices

The City of Woodburn has no formal program for sanitary sewerage system I/I reduction and control. To date, system maintenance staff has not recognized a need for such a program, because operational problems have been the result of grease buildup or other stoppages rather than hydraulic overloading caused by I/I.

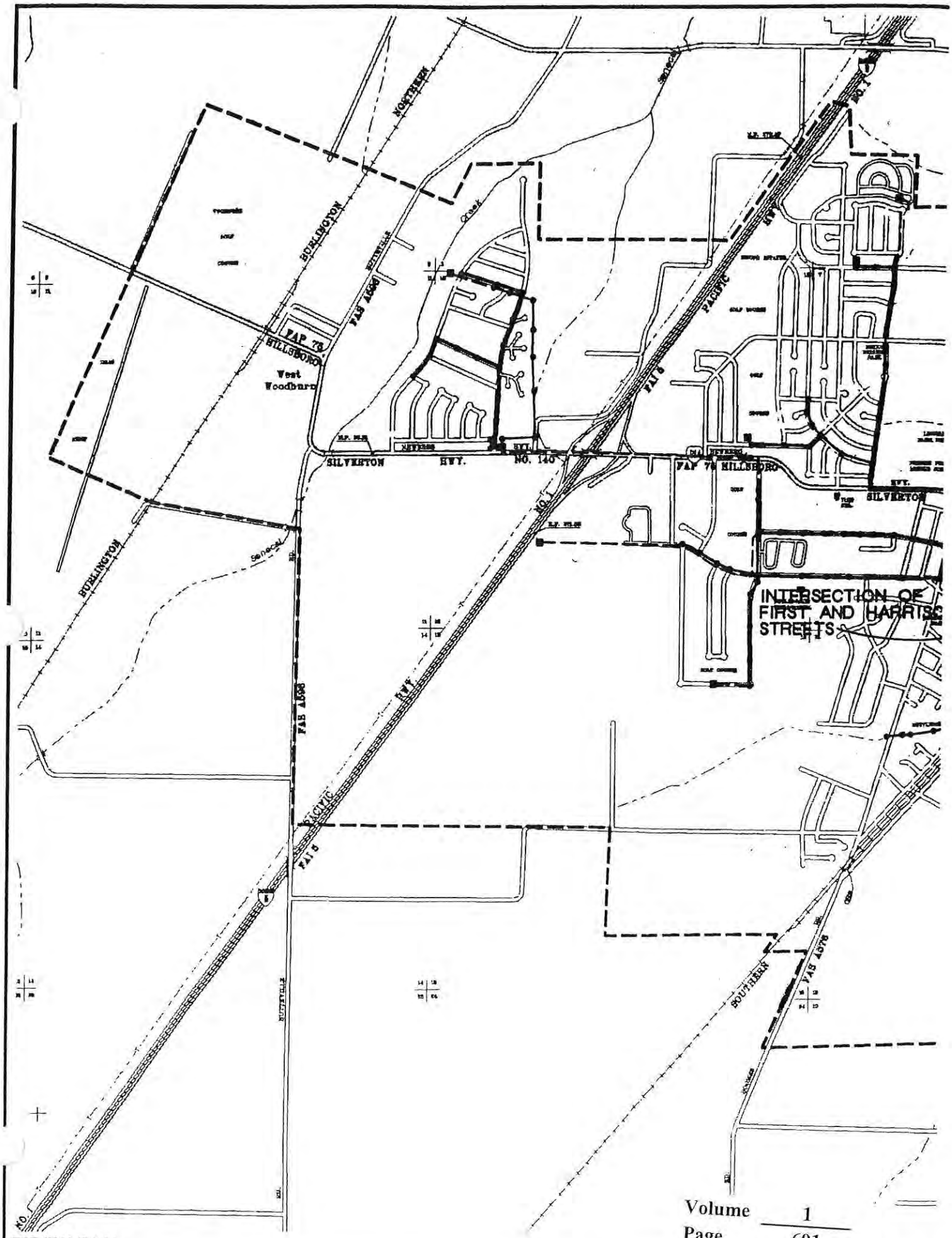
Flow monitoring, smoke testing, visual inspection of manholes, and television inspection of pipelines have only been conducted in response to operational problems. The results of these investigations have been documented and filed at the City maintenance shops. Approximately 25 percent of the publicly owned sewer lines in the City system have been inspected using closed circuit television. Most of these pipelines are smaller-diameter laterals rather than trunk and interceptor lines.

Recommended I/I Reduction and Control Program

An I/I reduction and control program should be conducted to verify flow rates, identify sources, and determine rehabilitation techniques for the pipelines and possible sanitary/storm sewer cross connections listed earlier and shown in Figure 1. I/I rates may be verified by further flow monitoring. Smoke testing, visual inspection of manholes, and television inspection of pipelines may be used to identify I/I sources and help determine the best method for system rehabilitation. A diligent maintenance program and specific regulations governing the quality and workmanship of newly constructed pipelines will help control I/I in the future.

Flow Monitoring

Wet-weather flow measurement at key locations in the collection system will help determine pipeline I/I rates. Flow meters may be placed upstream and downstream of the area in question to provide data for differential isolation of flow. The flow meters must be carefully installed and calibrated so that the net flow between them can be accurately calculated. Flow measurements gathered before and after pipeline rehabilitation will provide data to determine the effectiveness of rehabilitation techniques and the success of I/I reduction projects.



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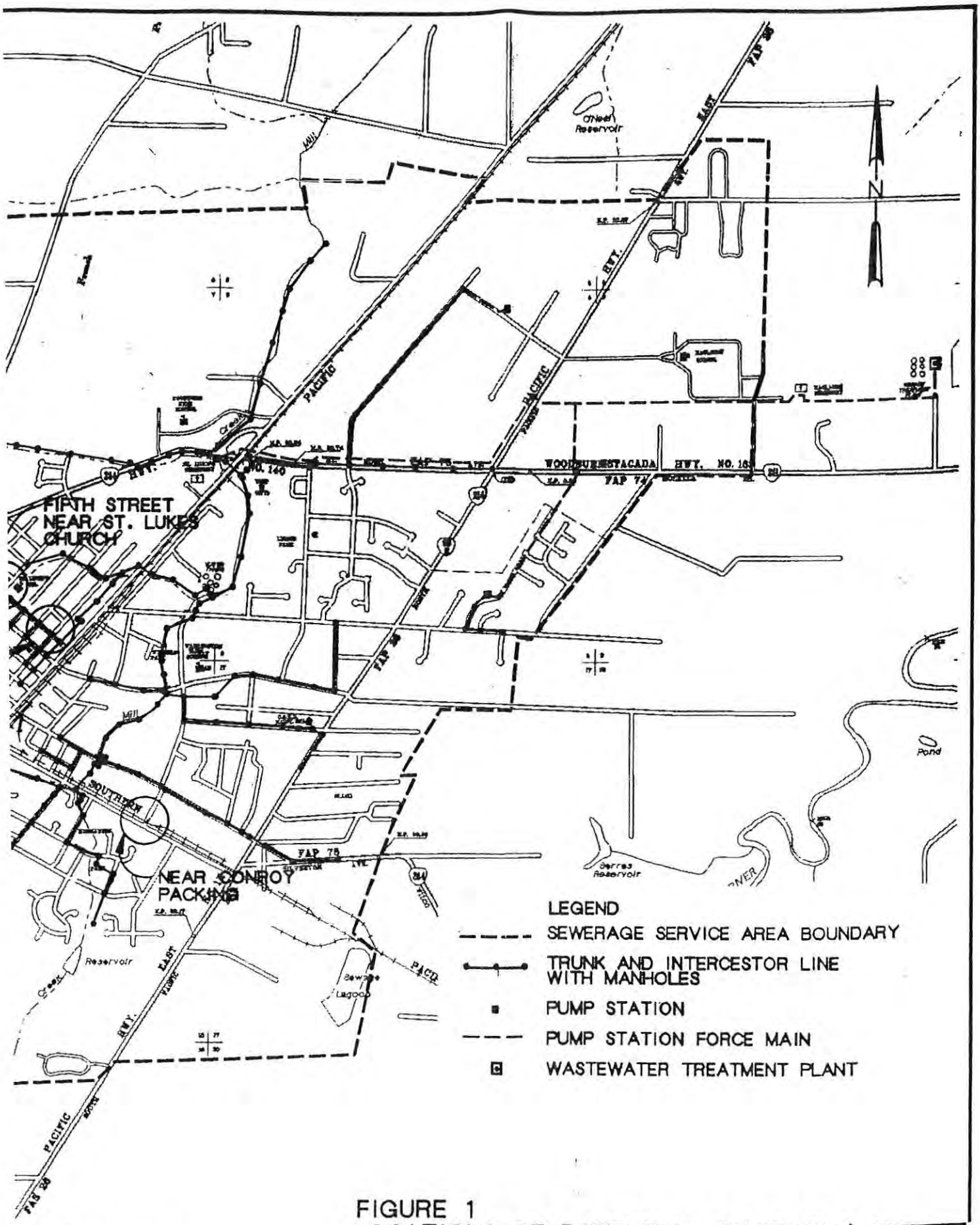

FIGURE 1
LOCATIONS OF POTENTIAL SANITARY/
STORM SEWER CROSS CONNECTIONS
CITY OF WOODBURN FACILITIES PLAN

CH2M HILL

Figure 1

Additional flow monitoring should be conducted for the pipelines that serve the areas along First Street, Montgomery Street, Arthur Street, and Lincoln Street. When I/I rates have been determined for each pipeline, the cost-effectiveness of rehabilitation may, in turn, be calculated. In cases where I/I reduction is economically justified, television inspection should be conducted to determine the appropriate pipeline rehabilitation technique.

Smoke Testing

Smoke testing is a relatively inexpensive and quick method of detecting I/I sources in sewer systems. This method may be best used to identify inflow sources such as sanitary/storm sewer cross connections or connected roof drains. These sources are often the least expensive to repair on a basis of dollars per gallon of I/I removed.

Smoke testing may be used to determine whether the three possible sanitary/storm sewer cross connections are, in fact, active. Smoke testing should be conducted in a systematic manner throughout the area near each suspected cross connection. The City staff may choose to confirm affirmative smoke testing results using dye testing or another method of detection before reconstruction begins. Smoke will sometimes pass between deteriorated pipelines even though no actual cross connection exists.

Visual Inspection of Manholes

Defective manholes can allow a substantial amount of infiltration. Visual inspection of manholes and pipelines in the sewerage system provides information on the degree of I/I and the general condition of the manhole. Visual inspection is quick, inexpensive, and effective if conducted in a systematic manner.

The condition of manholes should be visually inspected along First Street, Montgomery Street, Arthur Street, and Lincoln Street. Manholes should be inspected for evidence of sanitary/storm sewer cross connection near Conroy Packing, at Fifth Street near St. Luke's Church, and at the intersection of First Street and Harrison Street.

Television Inspection of Pipelines

Sewer lines may be inspected using closed-circuit television equipment. A small television camera and high-intensity light are pulled through a length of sewer line, producing videotaped images that provide an inspection record. Unless the City chooses to invest in this equipment, a contractor may have to be retained to perform the work.

Sewer lines should be cleaned before they are inspected using television equipment. Cleaning removes debris that can damage the recording equipment and removes grit and grease deposits from pipe walls, permitting defects to be detected more easily.

Pipelines along First Street, Montgomery Street, Arthur Street, and Lincoln Street should be inspected with television equipment if the results of the flow monitoring and smoke

testing justify such action. Television inspection will provide data to help determine the appropriate pipeline rehabilitation technique. Television inspection may also help determine whether sanitary/storm sewer cross connections exist near Conroy Packing, at Fifth Street near St. Luke's Church, or at the intersection of First Street and Harrison Street.

I/I Control and System Maintenance

Perhaps the best method for controlling I/I is to take preventive measures during the initial planning and construction of new sewer lines. The following measures are appropriate:

- Low allowable leakage limit specifications for acceptance of newly constructed pipeline
- Careful inspection during pipeline construction to check for quality workmanship
- Appropriate testing for field acceptance of newly constructed pipeline
- Requirements for proof of watertightness for pipe products, joint types, and appurtenances
- Use of local regulation and control to provide proper service lateral connections

New and old pipelines require continuing attention to maintain structural integrity and prevent gradual increases in I/I. Many problems can be solved more economically if they are addressed in early stages. Diligent and thorough maintenance may prevent a major rebuilding of the sewer system.

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DATE: January 25, 1995

SUBJECT: 5-Year Capital Improvements Plan

PROJECT: OPW27874.F4

Woodburn City staff prepared a 5-year Capital Improvements Plan (CIP) for the sanitary sewerage collection system based, in part, on the results of the collection system evaluation and infiltration and inflow (I/I) analysis work conducted by CH2M HILL in 1993. This technical memorandum summarizes the findings of the collection system evaluation and I/I control program and presents the City's 5-year CIP.

Collection System Evaluation

A computer analysis of the collection system was conducted as part of the collection system evaluation. According to the results of the analysis, the current (1992) peak flow resulting from the 5-year storm event is expected to exceed pump station and pipeline capacities and cause operational problems at the following locations:

- Mill Creek Pump Station
- Pump Station Number 1 near Vanderbeck Way
- Pump Station Number 2 near Jansen Road
- Pump Station Number 3 near Rainer Road
- Pump Station Number 9 near Industrial Avenue
- Front Street Interceptor through the downtown area to Lincoln Street
- Trunkline along Highway 214 and Astor Way serving the northern portion town

Field observations by collection system operation and maintenance personnel support the findings of the model. Hydraulic overloading has been physically observed at the Mill Creek

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Pump Station and in the Front Street Interceptor and the trunkline along Highway 214 and Astor Way during periods of heavy rain.

The wastewater facilities plan provided the following recommendations for collection system improvements:

- Upgrade the Mill Creek Pump Station
- Provide hydraulic relief for the Front Street Interceptor through the downtown area to Lincoln Street
- Provide hydraulic relief for the trunkline along Highway 214 and Astor Way serving the north part of town
- Monitor Pump Stations Number 1, 2, 3, and 9 to determine whether they should be upgraded in the immediate future

5-Year Capital Improvements Plan

The City staff developed a 5-year CIP based on the recommendations presented in the wastewater facilities plan. The CIP will be conducted in two phases. Phase 1 will include the Mill Creek Pump Station rehabilitation, and construction of three sanitary sewer projects: (1) the southwest sanitary sewer, (2) the southeast sanitary sewer and, (3) the north sanitary sewer. Each of these projects is described below. Construction of a new lift station at the Mill Creek Pump Station site will comprise Phase 2 of the CIP. This project is also described below. The project costs for Phase 1 and Phase 2 of the CIP are estimated at $3,555,500 and $2,283,000, respectively. The total project cost for the CIP is expected to be about $5,838,500. Table 1 provides a summary of the CIP. Figure 1 shows the approximate location of each project.

Table 1
5-Year Capital Improvements Plan (CIP) Summary

Description	Estimated Project Cost
Mill Creek Pump Station Rehabilitation	$2,529,000
Southwest Sanitary Sewer	$153,700
Southeast Sanitary Sewer	$617,100
North Sanitary Sewer	$255,700
Total Phase 1	**$3,555,500**
New Lift Station at the Mill Creek Site	$2,283,000
Total Phase 2	**$2,283,000**
Total CIP Cost	**$5,838,500**

Mill Creek Pump Station Rehabilitation

The Mill Creek Pump Station rehabilitation project includes pump and electrical system modifications at the Mill Creek Pump Station. The project will also include the addition of a 20-inch diameter force main and construction of an overflow retention basin. The new force main will act in parallel to the existing force main but will be constructed along a different alignment. The alignment of the new force main will parallel Front Street to the northeast for about 3,400 feet before heading directly east to the wastewater treatment plant. The total project cost for the Mill Creek Pump Station rehabilitation is expected to be about $2,529,000 including engineering and contingencies.

Southwest Sanitary Sewer

A gravity sewerline will be constructed from Santiam Drive to the Settlemier trunk to eliminate the need for Pump Station Number 6. The new gravity line will include about 2,800 feet of 18-inch diameter pipeline and 500 feet of 15-inch diameter pipeline. The total project cost for the southwest sanitary sewer is expected to be about $153,700 including engineering and contingencies.

Southeast Sanitary Sewer

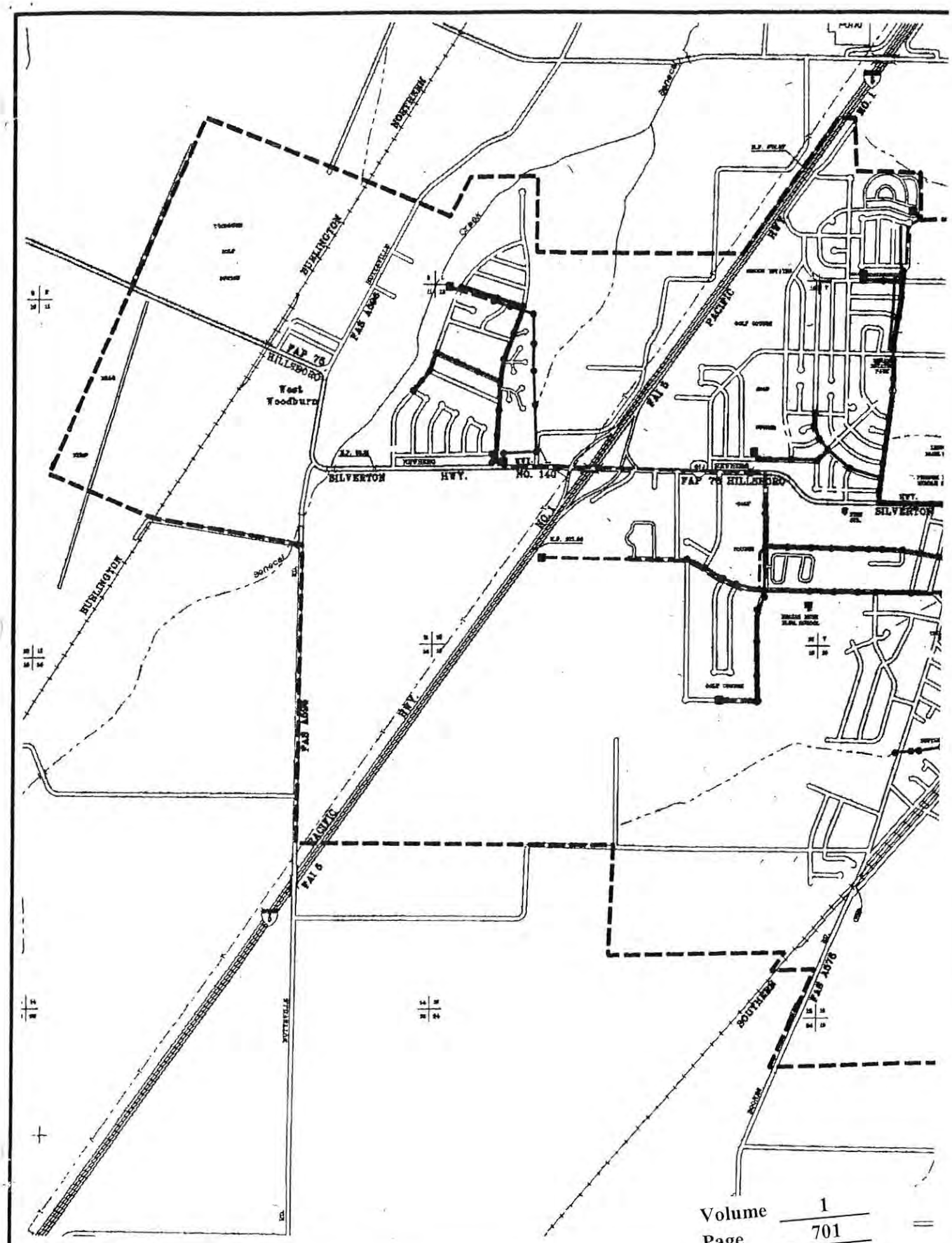
The southeast sanitary sewer project will include construction of a new lift station, force main, and gravity pipeline. The facilities will be constructed along Brown Street to serve the southernmost portion of the service area. The force main will be 8-inches in diameter and the gravity pipeline will vary in diameter from 20 to 8 inches. The total project cost for the southeast sanitary sewer is expected to be about $617,100 including engineering and contingencies.

North Sanitary Sewer

The north sanitary sewer will be constructed to relieve hydraulic overloading in the existing collection system facilities that serve the northern portion of town near Highway 214 and Astor Way. A gravity option and three pumping options were considered for this project. City staff selected one of the pumping options for construction. This option includes about 1,800 feet of 14-inch diameter gravity sewer along Jansen Way and Vanderbeck Road, a lift station near the intersection of Vanderbeck Road and Astor Way, and 1,100 feet of 8-inch diameter force main along Astor Way to Country Club Drive. The total project cost for the north sanitary sewer is expected to be about $255,700 including engineering and contingencies.

New Lift Station at the Mill Creek Site

A new lift station would be constructed about 1,400 feet north of the existing Mill Creek Pump Station. The new station would provide relief for the Mill Creek Pump Station during periods of heavy flow. The new lift station would share the 20-inch diameter force main



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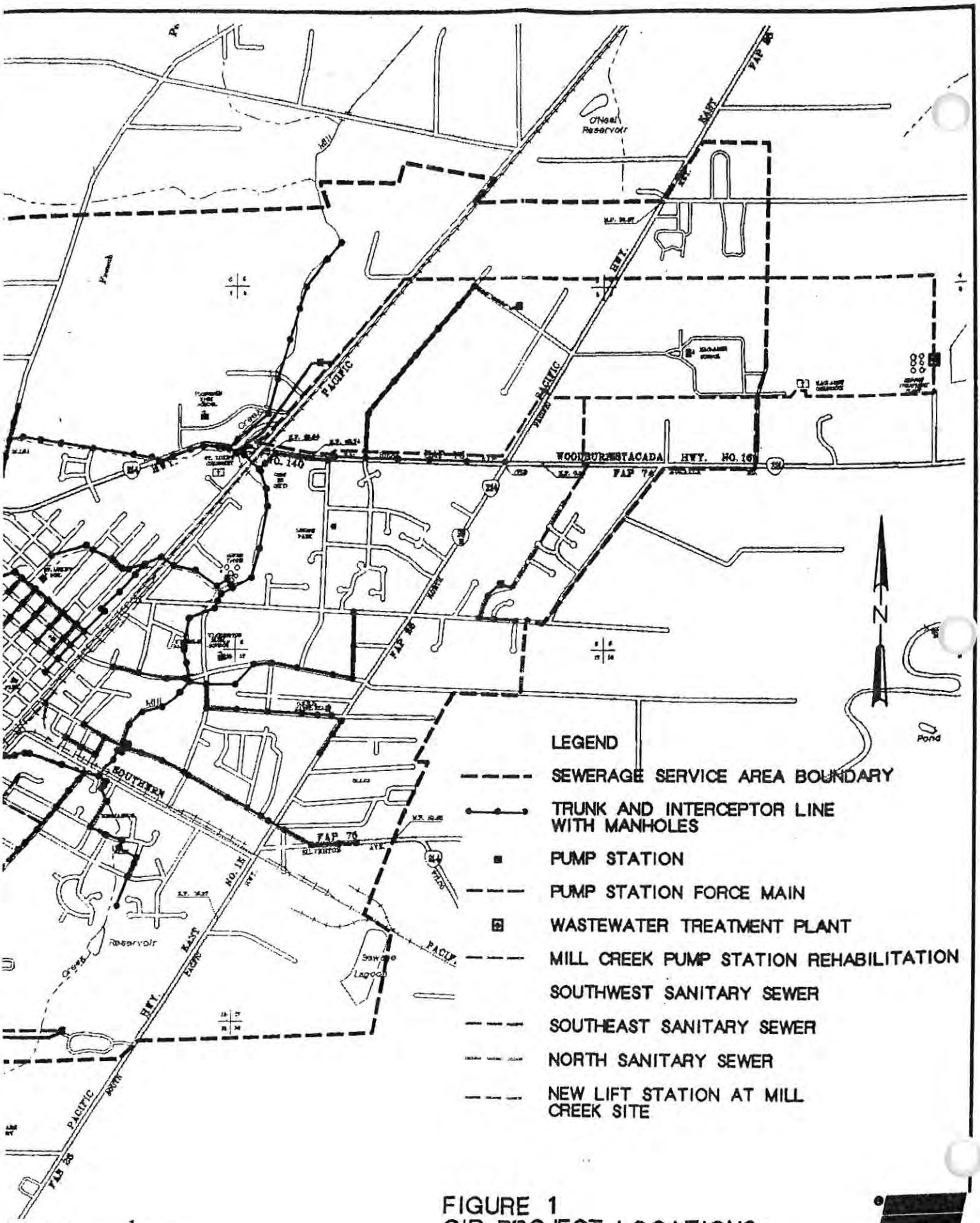
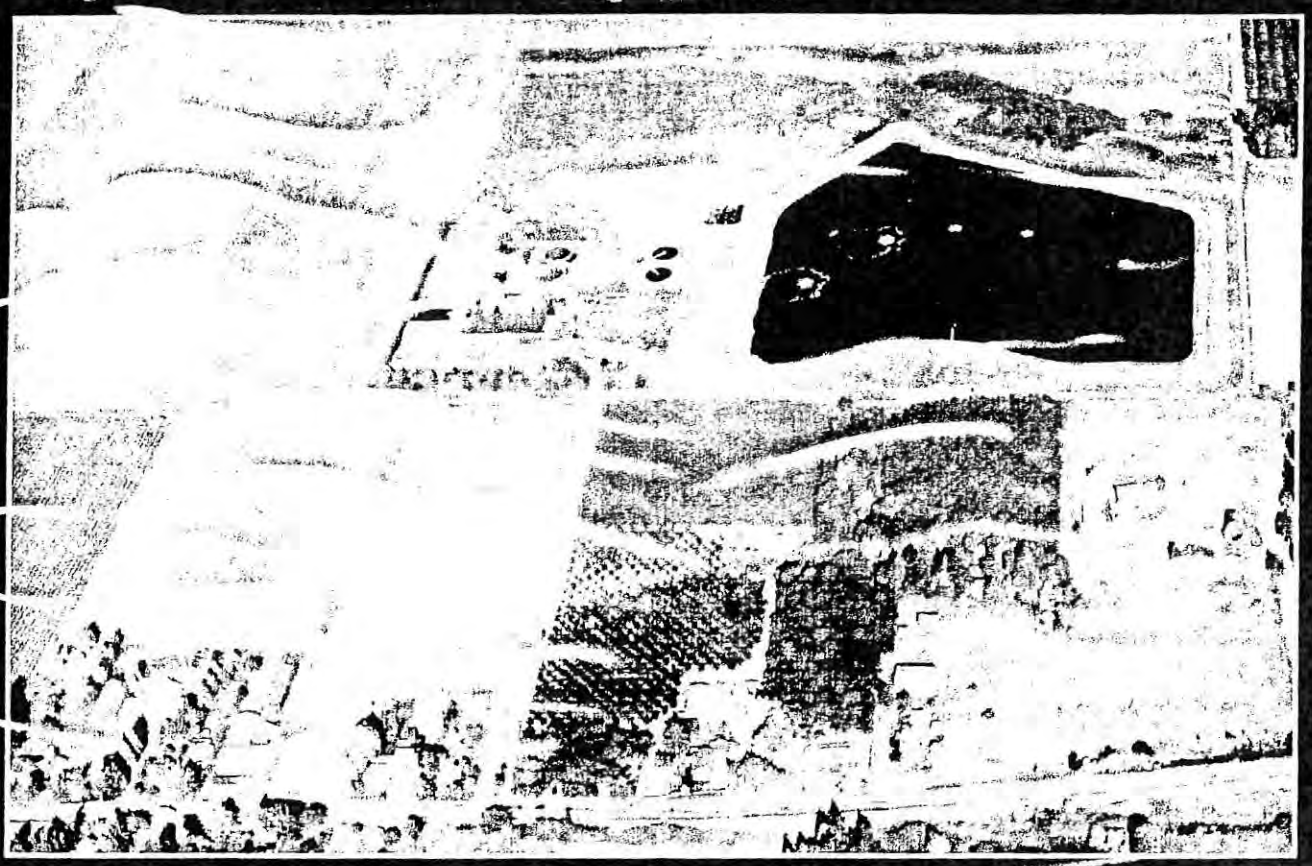


FIGURE 1
CIP PROJECT LOCATIONS
CITY OF WOODBURN FACILITIES PLAN

CH2M HILL

WASTEWATER FACILITIES PLAN

Volume III–Wastewater Rate and System Development Charge Study



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WASTEWATER FACILITIES PLAN

Volume III–Wastewater Rate and System Development Charge Study

February 1995

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Section 1
Introduction

Authorization and Purpose

The City of Woodburn authorized CH2M HILL to conduct a wastewater system and cost-of-service wastewater rate and system development charge (SDC) analysis. The purpose of the study was to develop wastewater rates and SDCs to recover anticipated system costs.

This technical report presents the results of the wastewater rate and SDC analysis for the City of Woodburn. It presents projected cost-of-service rates for the 5-year period of fiscal year (FY) 1995/96 through FY1999/2000, as well as proposed SDCs for the City's wastewater system.

Scope

The scope of the study involved the following major tasks:

- Review the City's current rate practices, technical issues relating to alternative rate practices, and system financing.
- Conduct a cost-of-service analysis to determine the cost of providing wastewater treatment service to each of the City's customer classes.
- Develop alternative rate designs to recover system costs.
- Develop system development charges for recovery of costs associated with capacity provided for future system growth.
- Prepare a final report presenting study assumptions, methodology, and findings.
- Meet with a Citizens Advisory Group to discuss the rate and SDC study methodologies and results of the analyses.

Background

The City of Woodburn's wastewater service area has been expanding over the years and moderate growth is expected to continue for several more years. The strength characteristics of the wastewater treatment plant's influent have changed and, coupled with increasing environmental effluent standards, have resulted in the need for treatment plant improvements.

The first phase of these capital improvements is planned to be completed within the 5-year study period of this report.

Report Organization

Section 2 provides a general overview of the methodology that was used to calculate the cost-of-service wastewater rates presented in this report. The calculation of system revenue requirements and net revenue requirements from rates is discussed in Section 3. Section 4 summarizes customer class user characteristics and the basis for projections of user flows and loadings for FY1995/96 through FY1999/2000. The allocation of system costs to wastewater parameters and then to customer classes is presented in Section 5. Section 6 presents proposed rate structure alternatives for wastewater service. The chapter includes a discussion of the impacts of the proposed rate alternatives on typical user bills. Section 7 summarizes the legal requirements relating to the calculation of SDCs and the methodology used to calculate these charges for the City of Woodburn. Section 8 presents a summary and the recommendations of the study.

Acknowledgments

CH2M HILL gratefully acknowledges the assistance we received from the City of Woodburn. In particular, we would like to express our appreciation to Frank Tiwari, Frank Sinclair, and Dean Morrison for their extra efforts in this regard.

Section 2

General Overview of the Sewer Rate Determination Process

Introduction

This chapter outlines the major steps taken to generate cost-of-service wastewater system rates and charges for the City of Woodburn's wastewater system. Figures 2-1 and 2-2 illustrate the basic steps involved in this process. They include:

- Estimation of annual wastewater system revenue requirements
- Determination of revenue requirements (costs) that must be recovered from rates and/or user charges
- Allocation of costs to loading parameters, including flow, biochemical oxygen demand (BOD), and total suspended solids (TSS)
- Estimation of annual wastewater system proportionate user or user class sewage loadings
- Allocation of user charge revenue requirements to users or user classes
- Computation of total revenue requirements by user or user class
- Development and design of alternative rate schedules

System Revenue Requirements

The first element of information required for a wastewater system rate study is an estimate of system revenue requirements. For this study, the cash basis method of determining system revenue requirements was used. Under the cash basis, system revenue requirements consist of operation and maintenance (O&M) expenses, debt service, and any capital outlays funded directly from rates. Table 2-1 summarizes the calculation of system revenue requirements.

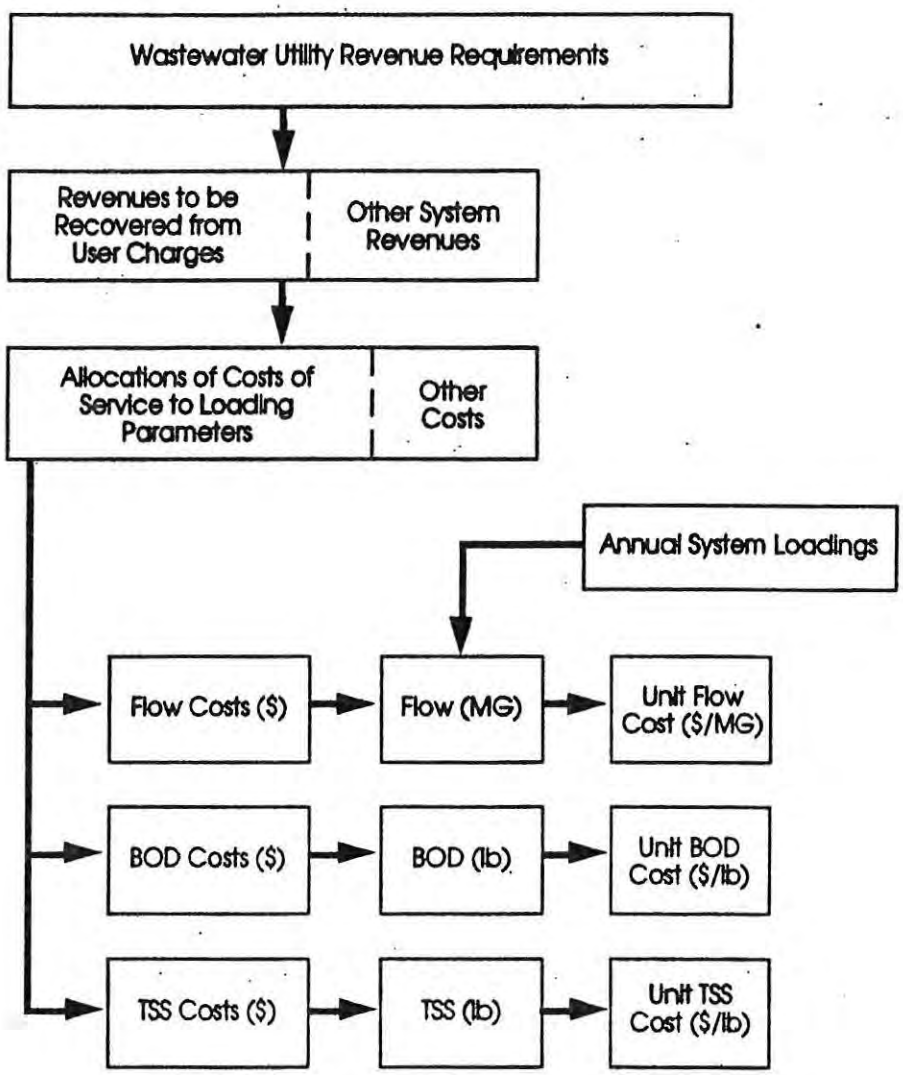

Figure 2-1
Schematic Development of Wastewater Rates

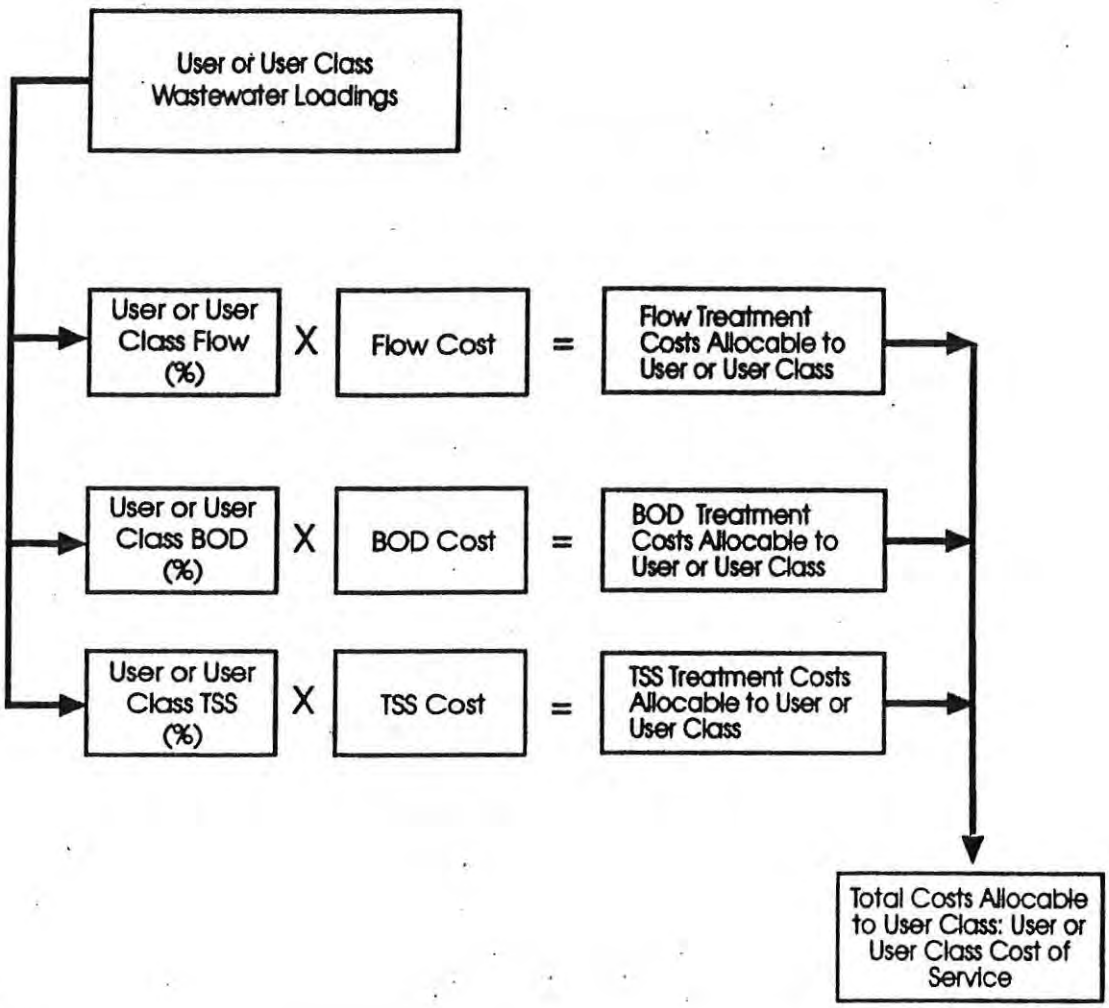

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Figure 2-2
Allocation of Costs to User or User Class

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Table 2-1 System Revenue Requirements ($/year)
Operation and maintenance costs + Debt service + Capital outlays = System revenue requirements

Net Revenue Requirements

The portion of annual system revenue requirements to be recovered through rates depends on a wastewater utility's financing policy and other sources of income. As illustrated in Figure 2-1 and outlined in Table 2-2, other system revenues (for example, interest income) are used to offset a portion of system revenue requirements.

Table 2-2 Net Revenue Requirements ($/year)
System revenue requirements - Other system revenues = Net revenue requirements

Allocation of Costs to Wastewater Loading Parameters

A wastewater utility's O&M costs and capital assets are usually divided into unit processes, such as collection, treatment, billing services, and general and administrative expenditures. An analysis of the wastewater system's treatment processes and design engineering judgment are used to separately allocate the system's unit process costs to wastewater loading parameters. This results in estimates of annual system flow costs, BOD treatment costs, and TSS treatment costs.

Annual Wastewater System Loadings

An analysis of past water consumption records, when used in conjunction with estimates of the strengths of user (or user class) wastewater flows and/or sampling results, can be used to estimate annual wastewater system loadings by customer class. For this study, flow is

measured in gallons or hundred cubic feet (hcf), while BOD and TSS loadings are measured in pounds. Infiltration and inflow (I/I) was estimated for the wastewater system and subtracted from total system flow estimates.

Distribution of Costs to System Users

Estimates of flow and pounds of BOD and TSS treated can be used in conjunction with the cost allocations to the wastewater parameters to determine the costs of treatment by loading parameter for each customer class. For example, that portion of a system's annual costs determined to be associated with treatment of TSS, when multiplied by each customer class' annual proportional contribution of TSS to the system, yields the costs of TSS that are applicable to each class. Using this method in the distribution of system costs results in an equitable distribution of the costs to the appropriate users or user classes.

Customer Class Revenue Requirements

Total revenue requirements by customer class are calculated by totaling the costs allocated to a given customer class. To these costs must be added other system costs, which are not directly allocable to users based on their respective wastewater loadings (for example, costs associated with billings).

One typical method used to allocate these other system costs to customer classes is to compute a uniform annual charge per connection to the system. For example, billing costs generally do not vary substantially from customer to customer; therefore, these costs can be allocated to users in the form of a uniform charge per connection or charge per bill. The end result of this process is an equitable distribution of system revenue requirements that are recoverable from each user or user class through wastewater rates and charges.

Development of a Rate Structure

A wastewater rate for a user or user class can be computed in one of several ways. Typical rate structures in use include a flat charge per connection per billing period, a flow-based charge set in dollars per hcf of water use or metered wastewater flow, or a combination of flat and flow-based rates. Under the flow-based wastewater rate, it is assumed that metered water use is either an accurate indicator of a user's wastewater flows or can be adjusted to be an accurate indicator. Surcharges for higher strength wastewater are used in many communities.

Minimum service charges per billing period can be computed based on billing costs and initial sewage flows. Customer costs or billing costs are directly correlated with the number of customer accounts, and these costs can be allocated to users as a uniform charge per account.

An initial amount of sewage discharge per billing period can also be included in the minimum service charge.

Section 3
System Revenue Requirements

Introduction

System revenue requirements (system costs) must first be determined before wastewater system rates and charges can be developed. The purposes of this chapter are to:

- Determine total wastewater system revenue requirements
- Determine the portion of these total revenue requirements that will be recovered through wastewater rates

For this analysis, system revenue requirements have been developed under the cash basis of accounting. Under the cash basis, system revenue requirements are the summation of system cash outlays (expenditures) and other system financial commitments (reserve requirements, for example) that the system must meet through its operating revenues and other revenue sources. For this analysis, system revenue requirements consist of the following:

- System O&M expenses, including funding of a reserve fund
- Annual debt service requirements
- Capital outlays not financed from other revenue sources

System revenue requirements have been projected for a 5-year period, FY1995/96 through FY1999/2000. Nonrate revenues and other sources of funding, such as transfers from the SDC account, interest income, and planned uses of wastewater fund reserves, are deducted from system revenue requirements to determine the amount of revenue that must be generated through the wastewater rates.

Operation and Maintenance Costs

Table 3-1 shows the City of Woodburn's projected O&M costs for FY1995/96 through FY1999/00. All system O&M costs were projected to increase at an annual escalation rate of 3 or 4 percent throughout the study period from FY1994/95 budgeted expenditure levels. An additional two employees are assumed to be hired in FY1999/00, one year before the wastewater treatment plant improvements have been completed.

Total system O&M costs are projected to increase from $1,720,000 in FY1995/96 to nearly $1,789,400 by FY1999/00. O&M costs are higher in FY1995/96 due to the funding of a new O&M contingency fund equal to one month's worth of total annual O&M costs. O&M costs decrease in FY1998/99 because a current bond is paid off the year before and O&M costs increase again in FY1999/00 due to the two additional employees added to the system.

Table 3–1
CITY OF WOODBURN
WASTEWATER RATE MODEL
PROJECTED OPERATION AND MAINTENANCE COSTS – rounded

ITEM	BUDGETED 1994/95	PROJECTED 1995/96	1996/97	1997/98	1998/99	1999/00
Annual Escalation Rate	N.A.	3.00%	3.00%	4.00%	4.00%	4.00%
Treatment:						
Personnel Services	$511,800	$527,200	$543,000	$564,700	$587,300	$746,500
Materials and Services	302,900	312,000	321,400	334,200	347,600	361,500
Capital Outlays	13,900	14,300	14,700	15,300	15,900	16,500
Subtotal Treatment Costs	$828,600	$853,500	$879,100	$914,200	$950,800	$1,124,500
Collection:						
Personnel Services	$92,400	$95,200	$98,000	$101,900	$106,000	$110,200
Materials and Services	11,700	12,100	12,400	12,900	13,400	14,000
Capital Outlays	66,000	68,000	70,000	72,800	75,700	78,800
Subtotal Collection Costs	$170,100	$175,300	$180,400	$187,600	$195,100	$203,000
Miscellaneous:						
Personnel Services	$22,800	$23,500	$24,200	$25,200	$26,200	$27,200
Transfer/Contingency/Unappropriated	348,400	327,900	337,800	351,300	365,400	380,000
Current Debt:						
Bonded Debt Payment	162,738	160,450	162,690	164,320	0	0
Loan Debt Payment	47,043	47,043	47,043	47,043	47,043	47,043
Subtotal Miscellaneous Costs	$581,000	$558,900	$571,700	$587,900	$438,600	$454,200
O&M Contingency	$0	$132,300	$3,600	$4,900	$0	$7,700
Total O&M Costs	$1,579,700	$1,720,000	$1,634,800	$1,694,600	$1,584,500	$1,789,400

Capital Costs

System capital costs consist of debt service requirements and capital outlays not financed from other sources.

Capital Improvements

Table 3-2 summarizes the wastewater system's planned capital improvement expenditures for FY1995/96 through FY1999/00. Major capital expenditures include construction of the wastewater treatment plant expansion and improvements, and construction of sewer lines to provide additional wastewater service to developing areas. Collection system improvements primarily consist of major trunk lines that are currently undersized for the current or projected wastewater volumes they need to handle. Total capital outlays are projected to range from zero in FY1995/96 to a high of $21.6 million in FY1998/99, and then back down to $12.9 million in FY 1999/00.

Debt Service

The wastewater system's outstanding long-term debts include a bond that will be retired in FY1997/98 and annual expenses for a long-term loan of $47,043. A new revenue bond will be issued in January 1997 for expansion and improvements of the wastewater treatment plant. New bond proceeds are assumed to equal $38,354,000 and will be used for capital improvements. Annual debt service is expected to be about $3,885,300 for 20 years.

Total Revenue Requirements

The summation of O&M costs, revenue-financed capital costs, and debt service costs equals total system revenue requirements, which are summarized in Table 3-3. Total revenue requirements amount to $1,657,600 in FY1995/96 and are projected to increase to $4,522,300 in FY1999/00.

Nonrate Revenues

Total nonrate revenues amount to $96,800 in FY1994/95 but then range from a low of $62,400 in FY1995/96 to a high of $255,600 in FY1996/97. Nonrate revenues are subtracted from total system revenue requirements to determine the amount of revenue that needs to be recovered through the wastewater rates. These nonrate revenues include interest on investments and other operating revenues. In addition, SDC revenues used to offset debt service costs are another nonrate revenue source.

Table 3-2
CITY OF WOODBURN
WASTEWATER SYSTEM DEVELOPMENT CHARGE MODEL
CAPITAL IMPROVEMENT PLAN NET OF CONTRIBUTIONS– INFLATED

PROJECT	1995/96	1996/97	1997/98	1998/99	1999/00	5-YEAR TOTAL	Percent Allocated to New Users	Total CIP Allocated to New Users	Annual Escalation Rate
Collection System:									
Mill Creek Lift Station & Force Main	$0	$0	$827,424	$1,934,766	$0	$2,762,190	69%	$1,905,911	4.00%
South–West Sewer	0	0	166,169	0	0	166,169	0%	0	4.00%
South–East Sewer	0	0	$0	$0	$721,766	721,766	100%	721,766	4.00%
North Sewer – Phase I	0	0	0	68,266	0	68,266	25%	17,067	4.00%
North Sewer – Phase II	0	0	0	269,019	0	269,019	50%	134,510	4.00%
Subtotal	$0	$0	$993,593	$2,272,051	$721,766	$3,987,410		$2,779,254	
Allocation to New Users								70%	
Treatment System:									
Headworks	$0	$0	$0	$1,360,579	$140,383	1,500,962	79%	1,185,760	4.00%
Aeration Basins	0	0	0	2,572,002	366,751	2,938,753	31%	911,013	4.00%
Secondary Clarifiers	0	0	0	1,130,488	111,137	1,241,625	38%	471,818	4.00%
RAS/WAS PS	0	0	0	749,910	389,952	1,139,862	32%	364,756	4.00%
Filters	0	0	0	1,575,934	858,676	2,434,610	65%	1,582,497	4.00%
UV Disinfection	0	0	0	690,756	178,895	869,651	54%	469,612	4.00%
Control/Lab Building	0	0	0	717,101	1,137,687	1,854,788	55%	1,020,133	4.00%
Digestion	0	0	0	1,754,788	608,326	2,363,114	100%	2,363,114	4.00%
Thickening/Storage	0	0	0	1,073,964	372,307	1,446,271	40%	578,508	4.00%
Popular Tree Irrigation	0	0	0	2,657,169	610,484	3,267,653	52%	1,699,180	4.00%
Outfall	0	0	0	478,067	497,190	975,257	100%	975,257	4.00%
Site Related Miscellaneous	0	0	0	3,110,924	1,773,974	4,884,898	59%	2,882,090	4.00%
Engineering	0	0	2,643,873	1,478,814	1,281,639	5,604,326	59%	3,306,552	4.00%
Contingency	0	0	0	0	3,844,916	3,844,916	59%	2,268,500	4.00%
Subtotal	$0	$0	$2,643,873	$19,350,496	$12,172,317	$34,366,686		$20,078,790	
Allocation to New Users								58%	
TOTAL WASTEWATER CIP	$0	$0	$3,637,466	$21,622,547	$12,894,083	$38,354,096		$22,858,044	
Allocation to New Users	$0	$0	$2,248,806	$12,572,185	$8,037,052	$22,858,045		60%	

Table 3–3
CITY OF WOODBURN
WASTEWATER RATE MODEL
NET REVENUE REQUIREMENTS FROM RATES

ITEM	BUDGETED 1994/95	PROJECTED 1995/96	1996/97	1997/98	1998/99	1999/00	95/96 – 99/00 TOTAL
Revenue Requirements:							
Operation & Maintenance Costs	$1,579,700	$1,720,000	$1,634,800	$1,694,600	$1,584,500	$1,789,400	$8,423,300
Capital Costs – Revenue Financed	0	0	0	0	0	0	$0
Debt Service Costs	0	0	1,942,700	3,885,300	3,885,300	3,885,300	$13,598,600
Total Revenue Requirements	$1,579,700	$1,720,000	$3,577,500	$5,579,900	$5,469,800	$5,674,700	$22,021,900
Less:							
Non–Rate Revenues	$96,800	$62,400	$255,600	$220,400	$193,300	$164,000	$895,700
SDC Revenue for Debt Service	0	0	1,756,400	919,900	937,800	988,400	$4,602,500
Net Revenue Requirements	$1,482,900	$1,657,600	$1,565,500	$4,439,600	$4,338,700	$4,522,300	$16,523,700

Net Revenue Requirements from Rates

Deduction of the nonrate revenues from total system revenue requirements results in the amount of revenue that needs to be generated through rates to cover system costs in each year. System net revenue requirements from rates and are shown at the bottom of Table 3-3 and amount to $1,657,600 in FY1995/96, and then increase to $4,439,600 by FY1997/98, and $4,522,300 in FY1999/00.

Net revenues from rates total $16,523,700 for the 5-year ratesetting period. These net revenue requirements will be allocated to the customer classes in Section 5.

Section 4

Wastewater Characteristics

Introduction

This chapter contains projections of wastewater accounts and annual wastewater flows and loadings by customer class. This information will be used in Section 5 as a basis to distribute system revenue requirements from rates to the customer classes, and will be used in Sections 6 and 7 to develop wastewater rates and SDCs.

Customer Classes

The City of Woodburn currently classifies its wastewater system customers into four separate customer classes: single-family residential, multifamily residential, commercial, and industrial. Single-family residential customers are billed a monthly flat rate, while multifamily residential customers are billed the same monthly flat rate per equivalent dwelling unit (EDU). Commercial customers are billed a base minimum service charge plus a volume rate per cubic foot of water use over the initial 6 hcf of demand. Industrial customers are charged based on measured discharges of flow, BOD and TSS.

Accounts

The City of Woodburn is projected to serve 4,879 wastewater customers in FY1995/96. Of these accounts, 4,430 (or 90.8 percent) are residential customers, 446 (9.1 percent) are commercial accounts and 3 (0.06 percent) are industrial customers.

Population growth within the areas currently being served by the wastewater system is projected at a 3.4 percent compound annual rate, in accordance with the population growth projections by service area presented in the draft Wastewater Facility Plan prepared by CH2M HILL. Table 4-1 lists the anticipated number of new connections by customer class to Woodburn's wastewater system. Table 4-2 presents the projected total number of wastewater system accounts to be served by the City for FY1995/96 through FY1999/00. The total number of accounts served by the wastewater system is projected to increase from 4,879 in FY1995/96 to 5,542 in FY1999/00.

Wastewater Flows

The projected annual wastewater flows by customer class for FY1994/95 through FY1999/00 are shown in Table 4-3. Residential customers are projected to discharge approximately

Table 4–1
CITY OF WOODBURN
WASTEWATER RATE MODEL
PROJECTED NEW SYSTEM CONNECTIONS

CUSTOMER CLASS	1995/96	1996/97	1997/98	1998/99	1999/00
Residential	134	137	142	146	151
Commercial	20	21	21	21	24
Industrial	0	0	0	0	0
Total	154	158	163	167	175

Table 4–2
CITY OF WOODBURN
WASTEWATER RATE MODEL
PROJECTED SYSTEM CUSTOMERS

CUSTOMER CLASS	1995/96	1996/97	1997/98	1998/99	1999/00
Residential (Number of Meters)	4,430	4,567	4,709	4,855	5,006
Commercial	446	467	488	509	533
Industrial	3	3	3	3	3
Total	4,879	5,037	5,200	5,367	5,542
Percent Change	N.A.	3.24%	3.24%	3.21%	3.26%
Number of Bills @12 Bills per Year	58,548	60,444	62,400	64,404	66,504
Multifamily Res. Units @10 Units/Meter	1,260	1,300	1,340	1,380	1,420
Total Residential Units	5,564	5,737	5,915	6,097	6,284

Table 4-3
CITY OF WOODBURN
WASTEWATER RATE MODEL
PROJECTED FLOWS BY CUSTOMER CLASS – hcf

CUSTOMER CLASS	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00
Residential	455,000	469,100	483,700	498,800	514,300	530,300
Commercial	210,300	220,300	230,300	240,900	251,400	262,900
Industrial	75,000	75,700	76,500	77,200	78,000	78,800
Total	740,300	765,100	790,500	816,900	843,700	872,000
Residential	61%	61%	61%	61%	61%	61%
Commercial	29%	29%	29%	29%	30%	30%
Industrial	10%	10%	10%	10%	9%	9%
Total	100%	100%	100%	100%	100%	100%

455,000 hcf (61 percent of total flows) of wastewater into the treatment system in FY1994/95. Commercial customers will discharge 210,300 hcf (29 percent) and industrial customers will discharge 75,000 hcf (10 percent) in FY1994/95. Total user wastewater flows are projected to be about 740,300 hcf in FY1994/95, and to increase to about 872,000 hcf in FY1999/00.

It was determined that the amount of infiltration and inflow (I/I) collected and treated by the wastewater treatment system is not an excessive problem in Woodburn, and therefore it was not addressed separately in this study.

Wastewater Strengths

Table 4-4 lists the projections of pounds of BOD discharged by each customer class for FY1994/95 through FY1999/00, respectively. The residential class is projected to discharge 616,500 pounds of BOD in FY1994/95 (41 percent of the total). The commercial class discharges the most BOD at 678,200 pounds (47 percent) while the industrial class discharges 184,300 pounds (12 percent). Total pounds of BOD discharged into the wastewater system are nearly 1,479,000 pounds in FY1994/95 and 1,698,300 in FY1999/00.

Table 4-5 shows the projections of TSS discharged by each customer class from FY1994/95 through FY1999/00. Residential class TSS discharges are about 616,500 pounds of TSS in FY1994/95 (61 percent of the total). Commercial class TSS discharges are nearly 340,200 pounds (33 percent) while industrial class discharges are estimated to be 66,100 (6 percent). Total TSS discharged to the wastewater system are about 1,022,800 pounds in FY1994/95 and increase to 1,183,000 in 1999/00.

Table 4–4
CITY OF WOODBURN
WASTEWATER RATE MODEL
PROJECTED BOD BY CUSTOMER CLASS – lbs

CUSTOMER CLASS	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00
Residential	616,500	635,800	655,500	675,900	697,200	718,700
Commercial	678,200	701,500	726,000	755,200	784,400	785,800
Industrial	184,300	186,200	188,000	189,800	192,000	193,800
Total	1,479,000	1,523,500	1,569,500	1,620,900	1,673,600	1,698,300
Residential	41%	41%	41%	41%	41%	42%
Commercial	47%	47%	47%	47%	47%	46%
Industrial	12%	12%	12%	12%	12%	12%
Total	100%	100%	100%	100%	100%	100%

Table 4–5
CITY OF WOODBURN
WASTEWATER RATE MODEL
PROJECTED TSS BY CUSTOMER CLASS – lbs

CUSTOMER CLASS	1994/95	1995/96	1996/97	1997/98	1998/99	1999/00
Residential	616,500	635,800	655,500	675,900	697,200	718,700
Commercial	340,200	352,600	365,700	382,200	398,900	394,900
Industrial	66,100	66,400	67,200	67,900	68,600	69,400
Total	1,022,800	1,054,800	1,088,400	1,126,000	1,164,700	1,183,000
Residential	61%	61%	61%	60%	60%	61%
Commercial	33%	33%	33%	34%	34%	33%
Industrial	6%	6%	6%	6%	6%	6%
Total	100%	100%	100%	100%	100%	100%

Table 5–1
CITY OF WOODBURN
WASTEWATER RATE MODEL
PLANT INVESTMENT ALLOCATION PERCENTAGES

ITEM	FLOW	BOD	TSS	CUSTOMER	TOTAL
Treatment	57%	32%	11%		100%
Collection & Lift Station	100%				100%
General (a)	69%	23%	8%		100%

Notes:
(a) Weighted average of other categories.

Table 5-2 shows the allocation of the wastewater system's noncontributed depreciated plant investment to the wastewater unit processes. Based on this analysis, 69 percent of the system's total capital investment is allocated to flow, 23 percent is allocated to BOD, 8 percent is allocated to TSS, and zero percent is allocated to the customer service function. These allocation percentages are applied to the annual net capital costs to derive the allocations to the wastewater parameters for these costs. The allocations of annual net capital costs are shown in Table 5-3. A negative number means that nonrate revenues and SDCs available for debt service are greater than the capital-related costs for that fiscal year.

Operation and Maintenance Cost Allocations

Annual system O&M costs for FY1994/95 through FY1999/00 are listed by major unit process in Table 5-4. Before being allocated to the customer classes, the O&M costs are first separated into unit processes, and then allocated to the wastewater parameters. The costs associated with each wastewater parameter are then distributed to the customer classes, based on their discharge of those parameters.

The O&M costs are then allocated to the wastewater parameters by the allocation percentages shown in Table 5-5. The annual costs associated with the treatment are allocated 25 percent to flow, 65 percent to BOD, and 10 percent to TSS. Collection and lift station unit processes are allocated 100 percent to flow, while billing service costs are allocated 100 percent to customers. General and administration costs are allocated to the parameters based on the weighted average of the other categories. Table 5-6 summarizes the allocation of O&M costs to the unit processes and to the wastewater parameters for FY1995/96 through FY1999/00. As shown in Table 5-6, 35 percent of the system's total O&M costs in FY1995/96 are allocable to flow, 51 percent to BOD, 8 percent to TSS, and 6 percent to the customer cost function.

Cost Allocations to Customer Classes

The capital and O&M costs are allocated to the customer classes by multiplying the capital and O&M annual costs associated with each wastewater parameter and customer cost function by each customer class's projected discharge of each wastewater parameter and number of customers, respectively. Table 5-7 lists the capital cost allocations to customer classes, and Table 5-8 shows the O&M cost allocations to customer classes. Table 5-9 presents the resulting total cost-of-service allocations to each customer class. As shown in Table 5-9, $872,200, or 53 percent, of system net revenue requirements from rates in FY1995/96 are allocable to residential users, $618,500 (37 percent) to commercial users, and $166,900 (10 percent) to industrial customers.

Table 5–2
CITY OF WOODBURN
WASTEWATER RATE MODEL
ALLOCATION OF PLANT INVESTMENT

ITEM	TOTAL	FLOW	BOD	TSS	CUSTOMER
Treatment	$1,314,974	$749,500	$420,800	$144,600	$0
Collection & Lift Station	505,584	505,600	0	0	0
General	0	0	0	0	0
	---	---	---	---	---
Total	$1,820,600	$1,255,100	$420,800	$144,600	$0
Percent	100%	69%	23%	8%	0%

Table 5–3
CITY OF WOODBURN
WASTEWATER RATE MODEL
ALLOCATION OF NET CAPITAL COSTS (a)

FISCAL YEAR	TOTAL	FLOW	BOD	TSS	CUSTOMER
1994/95	($96,800)	($66,700)	($22,400)	($7,700)	$0
1995/96	(62,400)	(43,000)	(14,400)	(5,000)	0
1996/97	(69,300)	(47,800)	(16,000)	(5,500)	0
1997/98	2,745,000	1,892,400	634,400	218,000	0
1998/99	2,754,200	1,898,700	636,500	218,700	0
1999/00	2,732,900	1,884,100	631,600	217,000	0

Notes:
(a) Revenue financed capital costs plus debt service costs less SDCs available for debt service less non–rate revenue.

Table 5–4
CITY OF WOODBURN
WASTEWATER RATE MODEL
O&M COSTS BY MAJOR UNIT PROCESSES – rounded

ITEM	BUDGETED 1994/95	PROJECTED 1995/96	1996/97	1997/98	1998/99	1999/00
Treatment	$828,600	$853,500	$879,100	$914,200	$950,800	$1,124,500
Collection & Lift Station	170,100	175,300	180,400	187,600	195,100	203,000
Billing Services	70,000	72,100	74,300	77,200	80,300	83,500
General & Administration	511,000	619,100	501,000	515,600	358,300	378,400
Total	$1,579,700	$1,720,000	$1,634,800	$1,694,600	$1,584,500	$1,789,400

Table 5–5
CITY OF WOODBURN
WASTEWATER RATE MODEL
O&M ALLOCATION PERCENTAGES

ITEM	FLOW	BOD	TSS	CUSTOMER	TOTAL
Treatment	25%	65%	10%		100%
Collection & Lift Station	100%				100%
Billing Services				100%	100%
General & Administration (a)	35%	51%	8%	6%	100%

Notes:
(a) Weighted average of other categories.

Table 5–6
CITY OF WOODBURN
WASTEWATER RATE MODEL
ALLOCATION OF O&M COSTS TO PARAMETERS

ITEM	FLOW	BOD	TSS	CUSTOMER	TOTAL
1995/96					
Treatment	$213,400	$554,800	$85,400	$0	$853,600
Collection & Lift Station	175,300	0	0	0	175,300
Billing Services	0	0	0	72,100	72,100
General & Administration	216,700	315,700	49,500	37,100	619,000
Total	$605,400	$870,500	$134,900	$109,200	$1,720,000
1996/97					
Treatment	$219,800	$571,400	$87,900	$0	$879,100
Collection & Lift Station	180,400	0	0	0	180,400
Billing Services	0	0	0	74,300	74,300
General & Administration	175,400	255,500	40,100	30,100	501,100
Total	$575,600	$826,900	$128,000	$104,400	$1,634,900
1997/98					
Treatment	$228,600	$594,200	$91,400	$0	$914,200
Collection & Lift Station	187,600	0	0	0	187,600
Billing Services	0	0	0	77,200	77,200
General & Administration	180,500	263,000	41,200	30,900	515,600
Total	$596,700	$857,200	$132,600	$108,100	$1,694,600
1998/99					
Treatment	$237,700	$618,000	$95,100	$0	$950,800
Collection & Lift Station	195,100	0	0	0	195,100
Billing Services	0	0	0	80,300	80,300
General & Administration	125,400	182,700	28,700	21,500	358,300
Total	$558,200	$800,700	$123,800	$101,800	$1,584,500
1999/00					
Treatment	$281,100	$730,900	$112,500	$0	$1,124,500
Collection & Lift Station	203,000	0	0	0	203,000
Billing Services	0	0	0	83,500	83,500
General & Administration	132,400	193,000	30,300	22,700	378,400
Total	$616,500	$923,900	$142,800	$106,200	$1,789,400

Table 5-7
CITY OF WOODBURN
WASTEWATER RATE MODEL
ALLOCATION OF NET CAPITAL COSTS TO CUSTOMER CLASSES

CUSTOMER CLASS	1995/96	1996/97	1997/98	1998/99	1999/00	TOTAL
Residential	($35,200)	($39,200)	$1,545,300	$1,550,400	$1,547,000	$4,568,300
Commercial	(21,000)	(23,200)	921,100	943,200	927,300	2,747,400
Industrial	(6,300)	(7,000)	278,400	260,400	258,400	783,900
Total	($62,500)	($69,400)	$2,744,800	$2,754,000	$2,732,700	$8,099,600

Table 5–8
CITY OF WOODBURN
WASTEWATER RATE MODEL
ALLOCATION OF O&M COSTS TO CUSTOMER CLASSES – SUMMARY

CUSTOMER CLASS	1995/96	1996/97	1997/98	1998/99	1999/00	TOTAL
Residential	$907,400	$862,700	$893,000	$835,300	$947,400	$4,445,800
Commercial	639,500	607,500	631,200	595,500	667,100	3,140,800
Industrial	173,200	164,600	170,700	153,800	175,100	837,400
Total	$1,720,100	$1,634,800	$1,694,900	$1,584,600	$1,789,600	$8,424,000

Table 5–9
CITY OF WOODBURN
WASTEWATER RATE MODEL
ALLOCATION OF TOTAL COSTS OF SERVICE TO CUSTOMER CLASSES

CUSTOMER CLASS	1995/96	1996/97	1997/98	1998/99	1999/00	TOTAL
Residential	$872,200	$823,500	$2,438,300	$2,385,700	$2,494,400	$9,014,100
Commercial	618,500	584,300	1,552,300	1,538,700	1,594,400	5,888,200
Industrial	166,900	157,600	449,100	414,200	433,500	1,621,300
Total	$1,657,600	$1,565,400	$4,439,700	$4,338,600	$4,522,300	$16,523,600

Section 6
Wastewater Rates

Introduction

Section 5 contained an analysis of the cost of providing service to each of the wastewater system's customer classes. The purpose of this chapter is to develop alternative rate schedules that recover those costs allocable to each customer class from the users composing each class. In addition, this chapter presents an analysis of the impact that the proposed rates would have on the monthly bills of some typical users. The projected revenues under the proposed rates are also presented.

Existing Rates

The City's current rates are shown in Table 6-1. The City of Woodburn currently charges its residential customers a flat rate of $14.20 per monthly billing period. The system's multifamily residential customers are charged the same $14.20 per EDU. Commercial customers pay a minimum service charge of $14.85 plus an additional volume charge of $2.20 per hcf. Industrial customers currently have a seasonal rate structure based on actual discharges of flow, BOD, and TSS. Industrial customers are charged separate unit rates for flow, BOD and TSS discharges. If an industrial customer's discharges are so low that the calculated charge is less than $367.60, then the minimum charge of $367.60 is billed. The higher summer BOD and TSS rates are due to higher summer loadings and the associated higher costs to treat those loadings.

New Rates

Three alternative wastewater rate structures were considered in the analysis: flat rates, volume rates, and volume rates plus minimum service charge. In all three cases, rate structures were designed as an average rate for the entire 5-year study period. That is, based on the projections contained in this report, no additional rate adjustments would be required for the 5-year ratesetting period. These 5-year average rates were then adjusted an additional 5 percent for the anticipated decrease in sewage demand due to the higher rates and the proposed change from a flat rate to a volume rate plus minimum service charge rate structure. The proposed wastewater rates are shown in Table 6-2.

Flat Rate

A flat rate was calculated for the residential class by dividing the 5-year total residential class's cost of service by the 5-year total projected number of residential units (one unit equals one single-family residential dwelling or one apartment in an apartment complex). The calculation results in a flat rate charge of $26.10 per month per unit. Commercial and

Table 6–1
CITY OF WOODBURN
WASTEWATER RATE MODEL
CURRENT WASTEWATER RATES

CUSTOMER CLASS	MINIMUM CHARGE	VOLUME CHARGE	Industrial Class VOLUME CHARGE	BOD CHARGE	TSS CHARGE
		$/hcf	$/mil gals	$/1,000 lbs	$/1,000 lbs
Residential (per unit)	$14.20	N.A.	N.A.	N.A.	N.A.
Commercial (a)	$14.85	$2.20	N.A.	N.A.	N.A.
Industrial – Jun, Jul, Aug, Sep (b)	$367.60	N.A.	$725.00	$344.44	$386.64
Industrial – Winter Months (b)	$367.60	N.A.	$725.00	$278.70	$322.20

Notes:
hcf = 100 cubic feet
(a) The minimum charge for the Commercial Class includes 6 hundred cubic feet (hcf) of flow.
(b) The minimum charge for the Industrial Class is charged only if the calculated bill is less than the minimum amount.

industrial customers always pay based on a monthly volume rate basis. The wastewater flat rate is shown in Table 6-2.

Volume Rate

The volume rates were calculated for each customer class by dividing the 5-year total class's cost of service by the 5-year total projected sewage demand. The residential class's sewage demand is based on the average winter water use for each customer and is held constant for the entire year. This calculation results in a charge of $3.81 per hcf for residential customers, $5.13 for commercial customers, and $4.41 for industrial customers. The wastewater volume rates are shown in Table 6-2.

Volume Rates Plus Minimum Service Charge

The volume rates plus minimum service charge were calculated for each customer class over the 5-year study period. The minimum charge includes customer costs and the initial sewage discharge in a billing period (5 hcf for residential, 6 hcf for commercial, and 10 hcf for industrial users). A single weighted volume charge was calculated for all customer class's initial discharge to calculate the minimum charge. The individual customer class's volume rate is added to the minimum service charge for all sewage demand greater than the initial discharge. This volume rate varies between the customer classes based on each class's cost of service and sewer demand.

The industrial class's volume charge has been disaggregated into volume, BOD, and TSS charges. The industrial users' actual discharges of flow, BOD, and TSS are known and can therefore be charged separately. The volume charge for the residential and commercial users could also be disaggregated but individual user BOD and TSS loadings are not available and therefore their charges are based on flow discharges only. The wastewater volume rates plus minimum service charge are shown in Table 6-2.

Impact On Typical Bills

Table 6-3 indicates the impact that the proposed FY1995/96 through FY1999/00 rates would have on the bills of some typical users. In all cases, except extremely low discharges, new proposed bills will be significantly greater than current bills. This is because of the capital improvements at the treatment plant. As shown, a residential customer discharging 10 hcf would pay $14.20 under current rates as compared to $26.10 under the proposed flat rate, $38.10 under proposed volume rates, and $37.14 under proposed volume rates plus minimum service charge.

TABLE 6–2
CITY OF WOODBURN
WASTEWATER RATE MODEL
PROPOSED COST OF SERVICE WASTEWATER RATES

CUSTOMER CLASS	FLAT RATE	VOLUME RATE	Volume Rate plus Minimum Service Charge				
			MINIMUM CHARGE	MINIMUM VOLUME (a)	VOLUME RATE	BOD RATE (b)	TSS RATE (c)
	$/Bill	$/hcf		hcf	$/hcf	$/lb	$/lb
Residential (per unit)	$26.10	$3.81	$19.19	5	$3.59	N.A.	N.A.
Commercial	N.A. (d)	$5.13	$22.69	6	$5.08	N.A.	N.A.
Industrial	N.A. (d)	$4.41	$51.90	10	$2.18	$0.82	$0.24

Notes:
hcf = 100 cubic feet
(a) The minimum charge includes an initial sewage discharge volume per billing period.
(b) The industrial class minimum charge includes an initial BOD discharge of 25 lbs per month.
(c) The industrial class minimum charge includes an initial TSS discharge of 9 lbs per month.
(d) The Commercial and Industrial classes always pay on a monthly volume rate basis.

Table 6–3
CITY OF WOODBURN
WASTEWATER RATE MODEL
TYPICAL MONTHLY WASTEWATER BILL COMPARISONS

CUSTOMER CLASS	Monthly Use (hcf)	Typical Volume Rate Bill	Typical Vol + Min Charge Bill	Typical Flat Rate Bill	Typical Current Bill
Residential (per unit)	5	$19.05	$19.19	$26.10	$14.20
Residential (per unit)	10	$38.10	$37.14	$26.10	$14.20
Residential (per unit)	15	$57.15	$55.09	$26.10	$14.20
Residential (per unit)	25	$95.25	$90.99	$26.10	$14.20
Residential (per unit)	35	$133.35	$126.89	$26.10	$14.20
Commercial	40	$205.20	$195.41	(a)	$89.65
Commercial	70	$359.10	$347.81	(a)	$155.65
Commercial	135	$692.55	$678.01	(a)	$298.65
Commercial	200	$1,026.00	$1,008.21	(a)	$441.65
Commercial	275	$1,410.75	$1,389.21	(a)	$606.65
Industrial (25 lbs BOD/9 lbs TSS) (b)	10	$44.10	$51.90	(a)	$367.60
Industrial (250 lbs BOD/90 lbs TSS) (b)	100	$441.00	$452.04	(a)	$367.60
Industrial (750 lbs BOD/270 lbs TSS) (b)	300	$1,323.00	$1,341.24	(a)	$367.60
Industrial (1750 lbs BOD/630 lbs TSS)	700	$3,087.00	$3,119.64	(a)	$691.22
Industrial (2500 lbs BOD/900 lbs TSS)	1,000	$4,410.00	$4,453.44	(a)	$987.46

Notes:
(a) Commercial and Industrial users are charged based on volumes and not on a flat rate basis.
(b) Industrial typical current bill equals the minimum charge.

Projected Revenues

Table 6-4 compares the customer class revenue requirements (cost of service) with projected revenues under the proposed flat rate. The new rates generate sufficient revenues to cover system net revenue requirements for the 5-year study period. However, the proposed rates generate excess rate revenues in the first 2 years of the study period while rate revenues show a shortfall for the latter years. The excess revenues collected in the first 2 years of the study period should be held in a rate stabilization account until they are needed in the latter years. This is caused by the fact that the rates are an average over the 5-year period.

Table 6-5 shows the projected revenues collected from the three customer classes under the proposed volume rates. Total revenues collected throughout the 5-year ratesetting period are sufficient for total 5-year net revenue requirements. The excess revenues collected in the first 2 years are needed to offset the shortfall in revenues collected in the last 3 years of the study period.

Table 6-6 shows the net revenue requirements and the customer class projected revenues collected under the proposed volume rates plus minimum service charge. Total revenues collected for the 5-year period are sufficient for total 5-year net revenue requirements. Again, the excess revenues collected in the first 2 years of the study period should be held in a rate stabilization account until they are needed in the latter years.

Table 6-7 shows the projected customer class rate revenues collected under current rates. Note that the rate revenues are not sufficient to meet the total 5-year system net revenue requirements. Table 6-7 demonstrates the need for increased wastewater rates in FY1995/96.

Rate Implementation

The proposed rates developed in this report are for the 5-year period starting July 1, 1995 and ending June 30, 2000. If new rates are not implemented at this time then revisions to the proposed rates will be needed and, in fact, the proposed rates would increase if implementation is delayed one year until July 1, 1996. The increase would be caused by additional O&M costs from the new treatment plant improvement which becomes operational in FY2000/01, inflation, and an additional annual debt service payment. Assuming no other changes, total net revenue requirements for the 5-year period of FY1996/97 through 2000/01 increase approximately $2,895,000, or 17.5 percent above the net revenue requirements of this report. Subsequently, the proposed rates would have to increase to recover this increase in net revenue requirements.

Table 6–4
CITY OF WOODBURN
WASTEWATER RATE MODEL
PROJECTED ANNUAL RATE REVENUES – RESIDENTIAL FLAT RATE

CUSTOMER CLASS	1995/96	1996/97	1997/98	1998/99	1999/00	TOTAL
Residential	$1,742,600	$1,796,800	$1,852,600	$1,909,600	$1,968,100	$9,269,700
Commercial (Vol + Min Charge)	1,077,400	1,126,300	1,178,200	1,229,500	1,285,700	5,897,100
Industrial (Vol + Min Charge)	333,900	337,400	340,500	344,200	347,700	1,703,700
Total Rate Revenue (Current Demand)	$3,153,900	$3,260,500	$3,371,300	$3,483,300	$3,601,500	$16,870,500
Total Rate Revenue (Adjusted Demand)	$3,089,500	$3,193,770	$3,302,105	$3,411,640	$3,527,180	$16,524,200
Net Revenue Requirements	$1,657,600	$1,565,500	$4,439,600	$4,338,700	$4,522,300	$16,523,700
Rate Revenue Excess (Shortfall)	$1,431,900	$1,628,300	($1,137,500)	($927,100)	($995,100)	$500

Table 6-5
CITY OF WOODBURN
WASTEWATER RATE MODEL
PROJECTED ANNUAL RATE REVENUES – VOLUME RATES

CUSTOMER CLASS	1995/96	1996/97	1997/98	1998/99	1999/00	TOTAL
Residential	$1,787,271	$1,842,897	$1,900,428	$1,959,483	$2,020,443	$9,510,500
Commercial	1,130,139	1,181,439	1,235,817	1,289,682	1,348,677	6,185,800
Industrial	333,837	337,365	340,452	343,980	347,508	1,703,100
Total Rate Revenue (Current Demand)	$3,251,247	$3,361,701	$3,476,697	$3,593,145	$3,716,628	$17,399,400
Total Rate Revenue (Adjusted Demand)	$3,088,685	$3,193,616	$3,302,862	$3,413,488	$3,530,797	$16,529,400
Net Revenue Requirements	$1,657,600	$1,565,500	$4,439,600	$4,338,700	$4,522,300	$16,523,700
Rate Revenue Excess (Shortfall)	$1,431,100	$1,628,100	($1,136,700)	($925,200)	($991,500)	$5,700

Table 6–6
CITY OF WOODBURN
WASTEWATER RATE MODEL
PROJECTED ANNUAL RATE REVENUES – VOLUME RATES PLUS MINIMUM SERVICE CHARGE

CUSTOMER CLASS	1995/96	1996/97	1997/98	1998/99	1999/00	TOTAL
Residential	$1,766,900	$1,821,800	$1,878,700	$1,937,000	$1,997,300	$9,401,700
Commercial	1,077,400	1,126,300	1,178,200	1,229,500	1,285,700	5,897,100
Industrial	333,900	337,400	340,500	344,200	347,700	1,703,700
Total Rate Revenue (Current Demand)	$3,178,200	$3,285,500	$3,397,400	$3,510,700	$3,630,700	$17,002,500
Total Rate Revenue (Adjusted Demand)	$3,089,520	$3,193,735	$3,302,375	$3,412,390	$3,528,870	$16,526,900
Net Revenue Requirements	$1,657,600	$1,565,500	$4,439,600	$4,338,700	$4,522,300	$16,523,700
Adjusted Rate Revenue Excess (Shortfall)	$1,431,900	$1,628,200	($1,137,200)	($926,300)	($993,400)	$3,200

Table 6-7
CITY OF WOODBURN
WASTEWATER RATE MODEL
PROJECTED ANNUAL RATE REVENUES – CURRENT RATES

CUSTOMER CLASS	1995/96	1996/97	1997/98	1998/99	1999/00	TOTAL
Residential	$948,100	$977,600	$1,007,900	$1,038,900	$1,070,800	$5,043,300
Commercial	493,500	515,900	539,700	563,200	589,000	2,701,300
Industrial	131,000	132,300	133,600	134,900	136,200	668,000
Total Rate Revenue	$1,572,600	$1,625,800	$1,681,200	$1,737,000	$1,796,000	$8,412,600
Net Revenue Requirements	$1,657,600	$1,565,500	$4,439,600	$4,338,700	$4,522,300	$16,523,700
Rate Revenues Excess (Shortfall)	($85,000)	$60,300	($2,758,400)	($2,601,700)	($2,726,300)	($8,111,100)

Section 7

System Development Charges

Introduction

One of the principal sources of revenue for financing wastewater system expansions is a one-time initial charge for connection to the wastewater system. This charge is generally referred to as a connection fee, impact fee, capital contribution fee, and/or SDC. In some communities, these fees are designed to recover only the costs incurred by the utility to install the connection and, if needed, to extend the sewer line to serve the new user. For other communities, these fees also include recovery of all or a portion of the capital investment made by the wastewater system to provide sufficient capacity in the system to serve these new users.

The City of Woodburn currently assesses an SDC that is intended to reimburse the current users for the costs they have incurred to provide capacity in the system to serve future users. The City will begin construction on a treatment plant expansion and improvement program that will change the unit costs for the capacity provided in the system to serve future users. In this chapter, the underlying assumptions and calculations of revised wastewater SDCs for the City of Woodburn are discussed. Charges for installation of the connection or extension of a sewer line should be assessed in addition to the SDCs calculated in this report.

System Development Charges: An Overview

Capital improvements to provide new capacity in a wastewater system must generally be constructed in large increments; therefore, system expansions are often constructed years in advance of when the added capacity will be fully utilized. As a result, current system users are often forced to pay rates that are partially used to pay for a portion of the system to serve future users. SDCs, designed to recover the wastewater system's investment in this extra capacity, are often assessed, either to avoid charging existing users for these extra capacity costs or to partially compensate the existing users for the costs they have previously incurred to provide this capacity.

Revenues generated through the assessment of SDCs are generally used to directly offset the costs of a system expansion or to repay any debt issued to finance the system expansion. The revenues may also be held to offset the costs of future system expansions. Use of these revenues to offset these debt service costs reduces the amount of revenue that needs to be generated through the wastewater rates assessed to existing users. In this way, the SDC revenues can be used either to directly finance plant expansions or to reimburse existing users (through lower rates) for the costs they have incurred to provide capacity for new users.

SDCs are typically assessed when a new user or developer connects to the sewer, when a new development permit is issued, or when a user changes the usage of his or her property. There

are a number of alternative SDC structures: a simple charge per connection, charges that vary with the number of fixture units (sewer drains), new charges per equivalent residential unit, and charges that vary with the water meter size for the new connection (that is, 5/8- by 3/4-inch equivalent meters). The City currently assesses SDCs on an equivalent dwelling unit basis for all users.

Legislative Considerations

In 1989, the State of Oregon enacted legislation that applies to any SDCs in effect on or after July 1, 1991. The bill authorizes local governments to define and assess SDCs and places limits on the ways revenues generated through SDCs can be used.

An SDC, as defined in this bill, is an amount charged to a new user at the time of connection to the wastewater system in excess of the cost of inspecting and installing the connection. The SDC does not include fees assessed or collected through local improvement districts. The legislation further breaks down the SDC into an improvement fee or a reimbursement fee. The improvement fee is a fee for the cost of capital expansions to be constructed. The reimbursement fee is a fee for the costs of capital improvements already constructed or under construction. The SDC may consist of an improvement fee, a reimbursement fee, or a combination of the two.

The legislation specifies that the reimbursement fee must be established by an ordinance or resolution that sets forth the methodology that will be used to calculate the charge. It further specifies that the methodology must consider the cost of existing facilities, prior contributions by existing users, value of unused capacity, ratemaking principles employed to finance the capital improvements, and other relevant factors. The objective of the methodology must be that future system users contribute no more than an equitable share of the cost of existing facilities.

The improvement fee methodology must also be specified in an ordinance or resolution that considers the cost of projected capital improvements needed to increase the capacity of the system. The legislation further requires that a credit be provided for the construction of qualified public improvements (contributions). A qualified public improvement is one required as a condition of residential development approval, identified in the system's capital improvement program, and not located on or contiguous to the property being developed.

Revenues generated through the reimbursement fees must be spent only on capital improvements to the system or repayment of debt on those improvements. Revenues generated through the improvement fees are dedicated to capacity-increasing capital improvements or repayment of debt on capacity-increasing capital improvements. An increase in capacity is established if the improvements increase the level of performance or service provided by existing facilities or provide new facilities. The portion of such improvements funded by improvement fees must be related to current or projected development.

Other provisions of the legislation require that:

- A local government implementing an SDC must develop a capital improvement program (CIP) or comparable plan that lists the improvements that may be funded with improvement fee revenues and the estimated timing and cost for each improvement.

- SDC revenues must be deposited into dedicated accounts, and local government must provide an annual accounting of revenues and expenditures.

- Local government must provide for an administrative appeal procedure whereby a citizen or other interested party may challenge an expenditure of SDC revenues.

- No legal action challenging the methodology used to calculate SDCs may be filed after 60 days from enactment of or revision to the SDC.

- The provisions of the legislation are invalidated if these provisions are construed to impair the local government's bond obligations or the ability of the local government to issue new bonds or other financing.

Overview Of Methodology

The SDCs calculated in this study were prepared in accordance with CH2M HILL's understanding of ORS223.297-314. The SDCs developed for this analysis consist of a reimbursement fee, an improvement fee, and a combined fee.

Reimbursement Fee

The methodology used in this analysis to calculate the reimbursement fee involves dividing the system's noncontributed depreciated original cost plant investment in the collection system and wastewater treatment plant by their respective capacities. The resulting investment per mgd of capacity is then converted into an SDC that is based on a charge per EDU for single-family residential connections. The resulting SDCs are based on the estimated requirements for capacity associated with new users' potential demands on the system.

The reimbursement SDCs calculated in this study result in charges that are equal to the new user's proportionate share of the depreciated investment by the utility in plant and equipment to serve new users. The steps taken in this study to calculate these SDCs, as shown in Table 7-1, are as follows:

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Table 7–1
CITY OF WOODBURN
WASTEWATER SYSTEM DEVELOPMENT CHARGE MODEL
REIMBURSEMENT FEE

ITEM	Collection System	Treatment Plant			Total Components
		FLOW	BOD	TSS	
Current Fixed Asset Value	$505,584	$578,589	$710,086	$26,299	$1,820,558
Less: Current Outstanding Debt Principal (a)	162,459	185,918	228,172	8,451	$585,000
Plus: Current Cash Reserves (a)	0	0	0	0	$0
Net System Investment	$343,125	$392,671	$481,914	$17,848	$1,235,558
Limiting ADW Capacity (mgd or lbs/day)	1.78	1.78	3,876	2,673	
Less: ADWF Infiltration/Inflow @23% (mgd)	0.41	0.41	N.A.	N.A.	
Existing User Capacity (mgd or lbs/day)	1.37	1.37	3,876	2,673	
Net Investment per mgd or lbs/day	$250,456	$286,621	$124	$7	
Net Investment per gpd	$0.2505	$0.2866			
Winter Average EDU Sewer Demand (gpd) (b)	175	175	0.56	0.56	
Reimbursement SDC per EDU	$44	$50	$69	$4	$167
Flow Reimbursement SDC ($/gal/day)	$0.54				
BOD Reimbursement SDC ($/lb/day)	$123.21				
TSS Reimbursement SDC ($/lb/day)	$7.14				

Notes:
(a) Total distributed to components using same proportions of current fixed assets.
(b) Single–Family Residential winter water use equals 175 gallons per unit.
Single–Family Residential loadings equals .2 lbs pcpd * 2.78 people per unit.

- Determine historical system cost as measured by the noncontributed depreciated original cost of the system's investment in the collection system, and in the treatment plant, less outstanding debt on these improvements, to derive the net system investment in these improvements.

- Divide investment in collection system and treatment plant facilities by estimated system capacity of the system components in mgd to derive charges per unit of capacity.

- Multiply the resulting total charge per unit of capacity by the estimated capacity requirements of a typical single-family residential user or equivalent dwelling unit (EDU). The charges for multifamily residential, commercial, and industrial users are based on their sewage demand compared to an EDU's sewage demand.

These steps result in a reimbursement SDC of $167.00 per new EDU that distributes system capital costs equitably between current and future users. In addition, these charges will equitably recover those costs allocable to future users in proportion to their estimated potential demands for wastewater service.

Improvement Fee

The improvement fee methodology used in this analysis to calculate the SDCs is to divide the expansion capital project costs from the CIP by the new capacity being added to the wastewater system. The resulting unit cost of new capacity per mgd is then converted into an SDC that is based on a charge per EDU. The resulting SDCs are based on the estimated requirements for capacity associated with new users' potential demands on the system and the unit costs per mgd of additional capacity.

The improvement SDCs calculated in this study result in charges that are equal to the utility's costs to provide additional capacity for new users. The steps taken in this study to calculate these SDCs, as shown in Table 7-2, are as follows:

- Determine the value of CIP expansion projects to be allocated to new users (see Table 3-2).

- Divide the CIP expansion project costs by the additional wastewater system capacity being added by the CIP expansion projects.

- Multiply the resulting charge per unit of capacity by the estimated capacity requirements of an EDU or typical single-family residential user. The charges for other user classes are based on their sewage demand compared to the sewage demand of an EDU.

Table 7–2
CITY OF WOODBURN
WASTEWATER SYSTEM DEVELOPMENT CHARGE MODEL
IMPROVEMENT FEE

ITEM	Collection System	Treatment Plant			Total Components
		FLOW	BOD	TSS	
CIP Value Allocated to New Users	$2,779,254	$12,047,274	$5,421,273	$2,610,243	$22,858,044
Additional ADW System Capacity (mgd)	1.55	1.55	3,342	4,007	
Less: ADWF Infiltration/Inflow @23% (mgd)	0.36	0.36	N.A.	N.A.	
Projected New User Capacity (mgd or lbs/day)	1.19	1.19	3,342	4,007	
Unit Cost of New Capacity per mgd	$2,335,508	$10,123,760	$1,622	$651	
Unit Cost of New Capacity per gpd or lbs/day	$2.34	$10.12			
Winter Average EDU Sewer Demand (gpd) (a)	175	175	0.56	0.56	
Improvement SDC per EDU	$410	$1,771	$908	$365	$3,454
Debt Service Credit per EDU (b)	$77	$328	$167	$71	$644
Net Improvement SDC per EDU	$333	$1,443	$741	$294	$2,810
Flow Improvement SDC ($/gal/day)	$10.15				
BOD Improvement SDC ($/lb/day)	$1,323.21				
TSS Improvement SDC ($/lb/day)	$525.00				

Notes:
(a) Single–Family Residential winter water use equals 175 gallons per unit.
Single–Family Residential loadings equals .2 lbs pcpd * 2.78 people per unit.
(b) Total distributed to components using same proportions as the SDC.

- Subtract a debt service credit, if any, for the expansion-related CIP costs that will be debt financed. This is needed to ensure that new customers are not double-charged through debt service payments included in the rates.

These steps result in an improvement SDC of $2,810.00 that distributes additional system capacity costs equitably to future users. In addition, these charges will equitably recover those expansion capital costs allocated to future uses in proportion to their estimated potential demands for water service.

Combined Fee

The combined fee methodology for calculating SDCs is the addition of the reimbursement fee and the improvement fee. The resulting combined SDC, as shown in Table 7-3, is based on the estimated requirements for capacity associated with new users, the potential demands on the system, and the unit investment per mgd of existing system capacity plus the unit costs per mgd of additional system capacity. The combined SDC for a typical single-family connection is $2,977.

Proposed System Development Charges

SDCs are calculated on an EDU basis for all users. An EDU was estimated to demand 175 gallons per day (gpd) of flow and .56 pounds per day (lb/day) of BOD and TSS.

Table 7-3 includes a breakdown of the combined SDC into its component parts of flow, BOD, and TSS. To determine the combined SDC for a new commercial user with estimated discharges of 100 gpd flow, 20 lbs/day BOD, and 10 lbs/day TSS, calculate the combined SDC as follows: (100 x $10.69) + (20 x $1,446.42) + (10 x $532.14) = $1,069 + $28,928.40 + $5,321.40 = $35,318.80.

Recommendations

We recommend that the City implement the combined SDC shown in Table 7-3. This method is easy to administer, is simple to understand, provides a reasonable approximation of the relative sewage flows of various new customers, and does not require the City to monitor changes in other user activities. Wastewater SDCs should not be charged to meters used exclusively for irrigation or exclusively to provide water used in a product that does not enter the sewer system.

We also recommend that the City:

- Establish or continue to maintain an account for the SDC revenues and an accounting system to track the receipt and expenditure of these funds.

Table 7–3
CITY OF WOODBURN
WASTEWATER SYSTEM DEVELOPMENT CHARGE MODEL
COMBINED SDCs

Reimbursement SDC per EDU	$167
Improvement SDC per EDU	$2,810
Combined SDC per EDU	$2,977
Flow Combined SDC ($/gal/day)	$10.69
BOD Combined SDC ($/lb/day)	$1,446.42
TSS Combined SDC ($/lb/day)	$532.14

Notes:
Current Sewer SDC = $2,832 per EDU.
One EDU = 175 gpd of Flow and .56 lbs/day of BOD & TSS.

- Establish or continue to maintain an administrative procedure whereby a citizen or other interested party may challenge an expenditure of SDC revenues.

- Identify in the City's capital improvement program which capital improvements may be financed from revenues generated through SDCs.

- Incorporate the SDC calculation methodology used in this report into the SDC ordinance.

Section 8
Summary and Recommendations

Rates

This study uses a cost-of-service analysis to determine the actual cost involved in serving each of the customer classes. The results are based on the available data and current estimates of future costs for the 5-year ratesetting period of FY 1995/96 through FY 1999/2000. These results show that average wastewater rates need to be increased in FY1995/96 to meet system financial requirements. This increase is the result of increases for some costs, some shifting of costs among classes in order to achieve better equity for ratepayers, and needed capital improvements. If the City adopts a volume-based rate structure CH2M HILL recommends that the summer season bills for the residential customer class be calculated based on the average winter month water use. This approach will avoid charging these users for significant quantities of water that do not enter the sewer system, such as lawn watering and car washing.

We recommend that the City of Woodburn adopt wastewater rates that reflect the cost of service and are consistent with the financial requirements of the system. This will result in rates that are adequate to meet the system's revenue needs and also maximize rate equity for its users.

System Development Charges

A reimbursement SDC, an improvement SDC, and a combined SDC were calculated for this study. CH2M HILL recommends that the City of Woodburn adopt the combined sewer SDC for new customers connecting to the wastewater system. These SDCs are equitable and charge new users their appropriate share of the investment the City has or plans to have in the wastewater system.

Recommendations

The wastewater rates presented in this report are for the 5-year ratesetting period of FY 1995/96 through FY 1999/2000. We recommend that the proposed rates become effective July 1, 1995. We also recommend that wastewater rates be reviewed annually to help maintain adequate revenues and equitable rates.

We suggest that the City continue to monitor the strengths of the large industrial users. Such monitoring will aid the City in evaluating any changes or trends in usage for these customers and in determining the required adjustments to their rates.

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Executive Summary

Introduction

This report contains an analysis of the City of Woodburn's wastewater facilities, capital and operating costs, revenues, and wastewater user characteristics. This analysis has been used to develop cost-of-service wastewater rate alternatives for the City. In addition, wastewater system development charges (SDCs) have been developed that are in accordance with our understanding of the State of Oregon statute relating to the implementation and assessment of SDCs.

Existing Rates

The City of Woodburn currently charges its single-family residential customers a monthly flat rate of $14.20 The system's multifamily residential customers pay the same $14.20 flat rate per equivalent dwelling unit (EDU). All commercial customers pay a minimum sewer charge plus a volume charge per hundred cubic feet (hcf) of water consumption. The minimum commercial service charge includes 6 hcf of initial flow per billing period. These commercial charges are a monthly minimum charge of $14.85 plus a volume charge of $2.20 per hcf over the initial 6 hcf. The City's current rates are shown in Table ES-1.

The city also has three industrial customers who are charged the greater of a monthly flat rate of $367.60 or $725.00 per million gallons plus $278.70 per 1,000 pounds of biochemical oxygen demand (BOD) plus $332.20 per 1,000 pounds of total suspended solids (TSS). To account for greater system loadings during the four summer months of June, July, August, and September, the industrial BOD charge increases to $344.44 per 1,000 pounds and the industrial TSS charge increases to $386.64 per 1,000 pounds.

Rate Structure

Three rate structure alternatives were developed for this study: flat rates, volume rates, and volume rates plus minimum service charge. The proposed flat rates are designed for residential customers only. Commercial customers pay based on their discharge flows and industrial customers pay based on their flows and strength loadings. The proposed volume rates vary between customary classes because of their different costs of service and are based on the total costs and sewer demands over the 5–year study period. The proposed volume rates plus minimum service charge recover customer costs and the initial sewage discharge in the base minimum charge. Flow-related costs are included as volume rates. The proposed volume rate alternatives would charge all residential users a volume charge based on their winter water use, which is held constant for the entire year. The proposed volume rate alternatives would charge all commercial and industrial users according to their monthly water use.

Table ES–1
CITY OF WOODBURN
WASTEWATER RATE MODEL
CURRENT WASTEWATER RATES

CUSTOMER CLASS	MINIMUM CHARGE	VOLUME CHARGE	Industrial Class VOLUME CHARGE	BOD CHARGE	TSS CHARGE
		$/hcf	$/mil gals	$/1,000 lbs	$/1,000 lbs
Residential (per unit)	$14.20	N.A.	N.A.	N.A.	N.A.
Commercial (a)	$14.85	$2.20	N.A.	N.A.	N.A.
Industrial – Jun, Jul, Aug, Sep (b)	$367.60	N.A.	$725.00	$344.44	$386.64
Industrial – Winter Months (b)	$367.60	N.A.	$725.00	$278.70	$322.20

Notes:
hcf = 100 cubic feet
(a) The minimum charge for the Commercial Class includes 6 hundred cubic feet (hcf) of flow.
(b) The minimum charge for the Industrial Class is charged only if the calculated bill is less than the minimum amount.

The proposed volume-based rate structures are more fair and equitable compared to flat rates since they are based on actual use. All customers will pay more compared to current bills, and customers with large sewage discharges will have to pay significantly more under the proposed volume rate alternatives.

Proposed Rates

The proposed rate structure alternatives are for the period of fiscal year (FY)1995/96 through FY1999/2000. As shown in Table ES-2, the proposed residential flat rate would be $26.10 per EDU. Under the proposed volume rates, rates range from $3.81 per hcf for residential customers to $5.13 per hcf for commercial customers. Industrial customers will pay $4.41 per hcf. These rates are average rates for the entire 5–year study period. That is, based on the projections contained in this report, no additional rate adjustments would be required for 5 years.

All users under volume rates plus minimum service charge would pay a base charge per monthly billing period which includes the initial sewage discharge in the billing period. The minimum service charge is $19.19 per month for residential customers and includes 5 hcf of volume, while commercial customers' minimum charge is $22.69, which includes 6 hcf of volume discharge. The industrial customers' minimum service charge is $51.90 per month and includes the initial discharges of 10 hcf volume, 25 pounds BOD, and 9 pounds TSS. The additional volume and strength charges, which vary between the customer classes, are added to the base charge for sewage discharges greater than the initial discharge allowed for each customer class.

Impact On Typical Bills

Table ES-3 shows the impact that the proposed rate structures would have on the bills of some typical users. The table shows the "current" wastewater rates that are in effect now and the proposed rate alternatives from this report. Under all of the new rate alternatives, all customers will pay more than the current rates except small industrial users. A residential customer with a monthly discharge of 10 hcf would have a current bill of $14.20 compared to a volume rate bill of $38.10, a volume rate plus minimum charge bill of $37.14, or a flat rate bill of $26.10.

Rate Implementation

The proposed rates developed in this report are for the 5-year period starting July 1, 1995 and ending June 30, 2000. If new rates are not implemented until July 1, 1996, then the proposed rates will have to increase to recover a 17.5 percent increase in net revenue requirements for FY1996/97 through 2000/01.

TABLE ES–2
CITY OF WOODBURN
WASTEWATER RATE MODEL
PROPOSED COST OF SERVICE WASTEWATER RATES

			Volume Rate plus Minimum Service Charge				
CUSTOMER CLASS	FLAT RATE	VOLUME RATE	MINIMUM CHARGE	MINIMUM VOLUME (a)	VOLUME RATE	BOD RATE (b)	TSS RATE (c)
	$/Bill	$/hcf		hcf	$/hcf	$/lb	$/lb
Residential (per unit)	$26.10	$3.81	$19.19	5	$3.59	N.A.	N.A.
Commercial	N.A. (d)	$5.13	$22.69	6	$5.08	N.A.	N.A.
Industrial	N.A. (d)	$4.41	$51.90	10	$2.18	$0.82	$0.24

Notes:
hcf = 100 cubic feet
(a) The minimum charge includes an initial sewage discharge volume per billing period.
(b) The industrial class minimum charge includes an initial BOD discharge of 25 lbs per month.
(c) The industrial class minimum charge includes an initial TSS discharge of 9 lbs per month.
(d) The Commercial and Industrial classes always pay on a monthly volume rate basis.

Table ES–3
CITY OF WOODBURN
WASTEWATER RATE MODEL
TYPICAL MONTHLY WASTEWATER BILL COMPARISONS

CUSTOMER CLASS	Monthly Use (hcf)	Typical Volume Rate Bill	Typical Vol + Min Charge Bill	Typical Flat Rate Bill	Typical Current Bill
Residential (per unit)	5	$19.05	$19.19	$26.10	$14.20
Residential (per unit)	10	$38.10	$37.14	$26.10	$14.20
Residential (per unit)	15	$57.15	$55.09	$26.10	$14.20
Residential (per unit)	25	$95.25	$90.99	$26.10	$14.20
Residential (per unit)	35	$133.35	$126.89	$26.10	$14.20
Commercial	40	$205.20	$195.41	(a)	$89.65
Commercial	70	$359.10	$347.81	(a)	$155.65
Commercial	135	$692.55	$678.01	(a)	$298.65
Commercial	200	$1,026.00	$1,008.21	(a)	$441.65
Commercial	275	$1,410.75	$1,389.21	(a)	$606.65
Industrial (25 lbs BOD/9 lbs TSS) (b)	10	$44.10	$51.90	(a)	$367.60
Industrial (250 lbs BOD/90 lbs TSS) (b)	100	$441.00	$452.04	(a)	$367.60
Industrial (750 lbs BOD/270 lbs TSS) (b)	300	$1,323.00	$1,341.24	(a)	$367.60
Industrial (1750 lbs BOD/630 lbs TSS)	700	$3,087.00	$3,119.64	(a)	$691.22
Industrial (2500 lbs BOD/900 lbs TSS)	1,000	$4,410.00	$4,453.44	(a)	$987.46

Notes:
(a) Commercial and Industrial users are charged based on volumes and not on a flat rate basis.
(b) Industrial typical current bill equals the minimum charge.

System Development Charges

As part of this study, an analysis was prepared of the cost of providing capacity in the wastewater system to serve future connections. In addition, charges were calculated to recover these costs from new connections. The SDC charges to a new, single-family residential equivalent dwelling unit (EDU) customer would increase from $2,832 under the current fee schedule to $2,977 under the proposed fee schedule. The fee structure for new commercial and industrial users is based on the estimated contributions of flow, BOD, and TSS to the system in gallons per day (gpd) or pounds per day (lbs/day). The SDCs are shown in Table ES-4.

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Table ES–4
CITY OF WOODBURN
WASTEWATER SYSTEM DEVELOPMENT CHARGE MODEL
COMBINED SDCs

Reimbursement SDC per EDU	$167
Improvement SDC per EDU	$2,810
Combined SDC per EDU	$2,977
Flow Combined SDC ($/gal/day)	$10.69
BOD Combined SDC ($/lb/day)	$1,446.42
TSS Combined SDC ($/lb/day)	$532.14

Notes:
Current Sewer SDC = $2,832 per EDU.
One EDU = 175 gpd of Flow and .56 lbs/day of BOD & TSS.

Recommendations

CH2M HILL recommends that the City of Woodburn:

- Adopt the proposed volume rates plus minimum service charge rate structure shown in Table ES-2.
- Adopt the proposed combined SDCs shown in Table ES-4.
- Implement these new rates and SDCs on July 1, 1995.

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FLAUGHER & TACK
Members—Wastewater Advisory Committee
Woodburn, OR 97071
1352 Astor Way 2197 Camellia Way
Tel. 982-3719 Tel. 981-6712

July 31, 1995

Mr. Scott Burlingham, Chairman
Wastewater Advisory Committee
City of Woodburn, OR 97071

Dear Scott:

You have no doubt been made aware that the Council, at its meeting on July 24th, voted to keep our Committee active until it decides otherwise.

We believe that the Mayor and Council may be interested in two specific areas of additional involvement by the Committee:

1. Input from, and our input to, activities of the Pudding River Basin Water Resources Development Association. Their plans and time schedules could be of critical financial interest to the City of Woodburn. Further, their activities may contribute to added attention to "watershed management", a concept gaining recognition at the other levels of government.

2. Continued monitoring of the process of converting the Woodburn Facility Plan into functioning "hardware".

We are of the opinion that the Mayor and Council may wish to call us into a special session to define their interests in, and expectations from, our continued involvement.

Sincerely yours,

Preston B. Tack

Gilbert E. Flaugher

Copies to: Mayor Nancy Kirksey
Councilors Jennings
Pugh
Chadwick

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FLAUGHER & TACK
Members—Wastewater Advisory Committee
Woodburn, OR 97071
1352 Astor Way Tel. 982-3719
2197 Camellia Way Tel. 981-6712

July 31, 1995

Mr. Scott Burlingham, Chairman
Wastewater Advisory Committee
City of Woodburn, OR 97071

Dear Scott:

Based on the probability of continued meetings of the Committee, we have the following suggestions and/or requests:

1. That any minutes presented to the Committee follow the intent of the motion approved by the Committee at its October 11, 1994 meeting. This was to provide a special summary section of all actions taken, and votes thereon, by the Committee. The intent was to provide a section in the minutes which would be the official voting record and would obviate the necessity of going through the minutes to find such information.

2. That staff ensure that the request, adopted in our March 9, 1995 meeting, for staff to "...carefully identify any and all changes to the three volume Wastewater Facilities Plan..." , has been followed. A further definition of this procedure was adopted by the Committee and consists of items C1 & C2 of our letter to you of March 27, 1995 and was applicable to the material distributed at the last meeting.

3. That we review the applicability of Ordinance 1498 to the design phase of the Wastewater Facilities Plan.

Sincerely yours,

Preston B. Tack

Preston B. Tack

Gilbert E. Flaugher

Gilbert E. Flaugher

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WASTEWATER ADVISORY COMMITTEE
WORKSHOP w/ CITY COUNCIL
JUNE 20, 1995 7:00 PM
WOODBURN CITY COUNCIL CHAMBERS

1) **Roll call**

2) **Chairman's comments**

Committee nearing completion of assignment including rate recommendation

3) **Presentations and/or questions and answers:**

a. Poplar tree test plot project status report by Staff

b. Table that compares residential, commercial, and laundromat sampling results by staff

Note: One additional laundromat sampling result relating to BOD will be completed on June 19 and it will be provided in the meeting of June 20

c. Presentation of changes to draft by CH2M Hill/Staff

(Note: Discussion is expected to be brief since public input did not require significant changes to initial draft)

Attachment: 1) Modification to Vol. III Executive Summary to include poplar tree revenue use and also to include November implementation rate table
Note: July vs. November ES4 rate table is part of the summary. Please note that this rate table provides same amount of revenues without cross subsidy whether implementation starts in July or November of 1995.

4) **Approval of Minutes:**

Attachments:

a. Minutes of April 18, 1995

5) **Committee discussion/ tabled items**

a. Explanation of current contracting process for professional services that does not require bidding and for construction contracts that require bidding by Staff

b. May 12, 1995 letter from Preston Tack and Gil Flaugher to Scott Burlingham

6) Adjournment

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AGENWW06.20

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TABLE SHOWING RESIDENTIAL COMMERCIAL and LAUNDROMAT SAMPLING RESULTS (ALL RESULTS in MG\L)

	TBOD				TSS			
DATE	RES. TBOD	COM. TBOD	LAUN. FRONT TBOD	LAUN99 TBOD	RES TSS	COM. TSS	LAUN FRONT TSS	LAUN 99 TSS
3-2-95		169				126		
3-9-95	180 CF	180 CF		[110] CF*	270 CF	120 CF		[150] CF*
3-9-95	222 WWT	207 WWT		[109] WWT*	254 WWT	175 WWT		[142]WWT*
3-10-95	144 CF	156 CF			240 CF	100 CF		
3-10-95	202 WWT	147 WWT			252 WWT	71 WWT		
3-15-95				353				262
3-16-95				229				254
3-17-95				260				274
3-28-95	142				218			
4-4-95		310				218		
4-[illegible]-95	121				141			
4-[illegible]-95			220				122	
4-21-95			210				50	
5-2-95		356				300		
5-9-95		301				218		
5-16-95	158				222			
5-30-95		482				320		
6-14-95							284	760
AVE	167	256	215	281	228	183	152	578
AVERAGE LAUNDROMAT DISCHARGE CONCENTRATIONS			TBOD 254				TSS 287	

* These 99E Laundry samples were diluted because Coffiey Laboratories failed to remove flush water from the sample container. The BOD and TSS split shows a high degree of laboratory accuracy but an extremely low degree of precision. The analysis results were treated as outliers and not used in calculating averages.

CF: Coffey Laboratories Inc.

WWT: City of Woodburn Wastewater Treatment Laboratory

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CH2M HILL
Engineers
Planners
Economists
Scientists

Portland Office

June 15, 1995

OPW27874.R1

Mr. Frank Tiwari
Director of Public Works
City of Woodburn
270 Montgomery Street
Woodburn, Or 97071

Subject: Revised Executive Summary for the Wastewater Rate & SDC Study

Dear Mr. Tiwari:

The revised Executive Summary for the Wastewater Rate and System Facilities Charge Study is being sent to you for adjusting prior changes and to provide modified tables for changing the rate implementation date from July 1, 1995 to November 1, 1995. It should be noted that the modified rate tables provide the same amount of revenue for the rate-setting period as provided by the July 1st rate implementation date and there is no cross-subsidies between residential, commercial, and industrial users.

Sincerely,

CH2M HILL

Robert J. Tomlinson

Robert J. Tomlinson
Economist

c: Daria Wightman, CH2M HILL

Enclosures

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Serving Oregon and Southwest Washington from two locations:
Portland Office *825 N.E. Multnomah, Suite 1300, Portland, OR 97232-2146* *503.235.5000* *503.235.2445 FAX*
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Executive Summary

Introduction

This report contains an analysis of the City of Woodburn's wastewater facilities, capital and operating costs, revenues, and wastewater user characteristics. This analysis has been used to develop cost-of-service wastewater rate alternatives for the City. In addition, wastewater system development charges (SDCs) have been developed that are in accordance with our understanding of the State of Oregon statute relating to the implementation and assessment of SDCs.

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The city also has three industrial customers who are charged the greater of a monthly flat rate of $367.60 or $725.00 per million gallons plus $278.70 per 1,000 pounds of biochemical oxygen demand (BOD) plus $332.20 per 1,000 pounds of total suspended solids (TSS). To account for greater system loadings during the four summer months of June, July, August, and September, the industrial BOD charge increases to $344.44 per 1,000 pounds and the industrial TSS charge increases to $386.64 per 1,000 pounds.

Rate Structure

Three rate structure alternatives were developed for this study: flat rates, volume rates, and volume rates plus minimum service charge. The proposed flat rates are designed for residential customers only. Commercial customers pay based on their discharge flows and industrial customers pay based on their flows and strength loadings. The proposed volume rates vary between ~~customary~~ customer classes because of their different costs of service and are based on the total costs and sewer demands over the 5-year study period. The proposed volume rates plus minimum service charge recover customer costs and the initial sewage discharge in the base minimum charge. Flow-related costs are included as volume rates. The proposed volume rate alternatives would charge all residential users a volume charge based on their winter water use, which is held constant for the entire year. The proposed volume rate alternatives would charge all commercial and industrial users according to their monthly water use.

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Table ES–1
CITY OF WOODBURN
WASTEWATER RATE MODEL
CURRENT WASTEWATER RATES

CUSTOMER CLASS	MINIMUM CHARGE	VOLUME CHARGE	Industrial Class VOLUME CHARGE	BOD CHARGE	TSS CHARGE
		$/hcf	$/mil gals	$/1,000 lbs	$/1,000 lbs
Residential (per unit)	$14.20	N.A.	N.A.	N.A.	N.A.
Commercial (a)	$14.85	$2.20	N.A.	N.A.	N.A.
Industrial – Jun, Jul, Aug, Sep (b)	$367.60	N.A.	$725.00	$344.44	$386.64
Industrial – Winter Months (b)	$367.60	N.A.	$725.00	$278.70	$322.20

Notes:
hcf = 100 cubic feet
(a) The minimum charge for the Commercial Class includes 6 hundred cubic feet (hcf) of flow.
(b) The minimum charge for the Industrial Class is charged only if the calculated bill is less than the minimum amount.

The proposed volume-based rate structures are more fair and equitable compared to flat rates since they are based on actual use. All customers will pay more compared to current bills, and customers with large sewage discharges will have to pay significantly more under the proposed volume rate alternatives.

Proposed Rates

The proposed rate structure alternatives are for the period of fiscal year (FY)1995/96 through FY1999/2000. As shown in Table ES-2, the proposed residential flat rate would be $26.10 per EDU. Under the proposed volume rates, rates range from $3.81 per hcf for residential customers to $5.13 per hcf for commercial customers. Industrial customers will pay $4.41 per hcf. These rates are average rates for the entire 5-year study period. That is, based on the projections contained in this report, no additional rate adjustments would be required for 5 years.

All users under volume rates plus minimum service charge would pay a base charge per monthly billing period which includes the initial sewage discharge in the billing period. The minimum service charge is $19.19 per month for residential customers and includes 5 hcf of volume, while commercial customers' minimum charge is ~~$22.69~~ $24.27, which includes 6 hcf of volume discharge. The industrial customers' minimum service charge is $51.90 per month and includes the initial discharges of 10 hcf volume, 25 pounds BOD, and 9 pounds TSS. The additional volume and strength charges, which vary between the customer classes, are added to the base charge for sewage discharges greater than the initial discharge allowed for each customer class.

Impact On Typical Bills

Table ES-3 shows the impact that the proposed rate structures would have on the bills of some typical users. The table shows the "current" wastewater rates that are in effect now and the proposed rate alternatives from this report. Under all of the new rate alternatives, all customers will pay more than the current rates except small industrial users. A residential customer with a monthly discharge of 10 hcf would have a current bill of $14.20 compared to a volume rate bill of $38.10, a volume rate plus minimum charge bill of $37.14, or a flat rate bill of $26.10.

Rate Implementation

The proposed rates developed in this report are for the 5-year period starting July 1, 1995, and ending June 30, 2000. If new rates are not implemented ~~until~~ on July 1, 1996~~5~~, then the proposed rates will have to increase since current rates do not recover sufficient revenues for FY1995/96. ~~to recover a 17.5 percent increase in net revenue requirements for FY1996/97 through 2000/01~~. The City is considering a delay in rate implementation until November 1, 1995. Table ES-4 shows proposed rates for a November 1, 1995, implementation date and

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Table ES–2
CITY OF WOODBURN
WASTEWATER RATE MODEL
PROPOSED COST OF SERVICE WASTEWATER RATES

			Volume Rate plus Minimum Service Charge				
CUSTOMER CLASS	FLAT RATE	VOLUME RATE	MINIMUM CHARGE	MINIMUM VOLUME (a)	VOLUME RATE	BOD RATE (b)	TSS RATE (c)
	$/Bill	$/hcf		hcf	$/hcf	$/lb	$/lb
Residential (per unit)	$26.10	$3.81	$19.19	5	$3.59	N.A.	N.A.
Commercial	N.A. (d)	$5.13	***$24.27***	6	$5.08	N.A.	N.A.
Industrial	N.A. (d)	$4.41	$51.90	10	$2.18	$0.82	$0.24

Notes:
hcf = 100 cubic feet
(a) The minimum charge includes an initial sewage discharge volume per billing period.
(b) The industrial class minimum charge includes an initial BOD discharge of 25 lbs per month.
(c) The industrial class minimum charge includes an initial TSS discharge of 9 lbs per month.
(d) The Commercial and Industrial classes always pay on a monthly volume rate basis.

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Table ES–3
CITY OF WOODBURN
WASTEWATER RATE MODEL
TYPICAL MONTHLY WASTEWATER BILL COMPARISONS

CUSTOMER CLASS	Monthly Use (hcf)	Typical Volume Rate Bill	Typical Vol + Min Charge Bill	Typical Flat Rate Bill	Typical Current Bill
Residential (per unit)	5	$19.05	$19.19	$26.10	$14.20
Residential (per unit)	10	$38.10	$37.14	$26.10	$14.20
Residential (per unit)	15	$57.15	$55.09	$26.10	$14.20
Residential (per unit)	25	$95.25	$90.99	$26.10	$14.20
Residential (per unit)	35	$133.35	$126.89	$26.10	$14.20
Commercial	*6*	***$30.78***	***$24.27***	(a)	$14.85
Commercial	*10*	***$51.30***	***$44.59***	(a)	$23.65
Commercial	*25*	***$128.25***	***$120.79***	(a)	$56.65
Commercial	*50*	***$256.50***	***$247.79***	(a)	$111.65
Commercial	*100*	***$513.00***	***$501.79***	(a)	$221.65
Industrial (25 lbs BOD/9 lbs TSS) (b)	10	$44.10	$51.90	(a)	$367.60
Industrial (250 lbs BOD/90 lbs TSS) (b)	100	$441.00	$452.04	(a)	$367.60
Industrial (750 lbs BOD/270 lbs TSS) (b)	300	$1,323.00	$1,341.24	(a)	$367.60
Industrial (1750 lbs BOD/630 lbs TSS)	700	$3,087.00	$3,119.64	(a)	$691.22
Industrial (2500 lbs BOD/900 lbs TSS)	1,000	$4,410.00	$4,453.44	(a)	$987.46

Notes:
(a) Commercial and Industrial users are charged based on volumes and not on a flat rate basis.
(b) Industrial typical current bill equals the minimum charge.

Revised 14–Jun–95

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Table ES–4
CITY OF WOODBURN
WASTEWATER RATE MODEL
PROPOSED WASTEWATER RATES
JULY 1, 1995 vs NOVEMBER 1, 1995 IMPLEMENTATION DATE

CUSTOMER CLASS	FLAT RATE $/Bill		Volume Rate plus Minimum Service Charge: MINIMUM CHARGE	MINIMUM VOLUME (a) hcf	VOLUME RATE $/hcf	BOD RATE (b) $/lb	TSS RATE (c) $/lb
JULY 1, 1995 RATE IMPLEMENTATION DATE							
Residential (per unit)	$26.10	<-- or -->	$19.19	5	$3.59	N.A.	N.A.
Commercial	N.A.		***$24.27***	6	$5.08	N.A.	N.A.
Industrial	N.A.		$51.90	10	$2.18	$0.82	$0.24
NOVEMBER 1, 1995 RATE IMPLEMENTATION DATE							
Residential (per unit)	$27.48	<-- or -->	$20.20	5	$3.59	N.A.	N.A.
Commercial	N.A.		$25.54	6	$5.49	N.A.	N.A.
Industrial	N.A.		$54.62	10	$2.27	$0.85	$0.25

Revised 14–Jun–95

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the proposed rates for a July 1, 1995, implementation date as discussed above. The minimum charge will increase 5.25 percent for each customer class with the same minimum volumes if rate implementation is delayed until November 1, 1995. The proposed residential volume rate will remain unchanged while the commercial users' volume rate will increase 8.07 percent to $5.49 per hcf. The industrial users' volume rate will increase 4.13 percent to $2.27 per hcf; the BOD rate will increase 3.66 percent to $0.85 per pound; and the TSS rate will increase 4.17 percent to $0.25 per pound.

Table ES-5 shows the impact that the proposed November 1, 1995, rate structures will have on the bills of some typical users. In all cases, these bills will be greater than the bills based on a July 1, 1995, rate implementation date as shown in Table ES-3.

System Development Charges

As part of this study, an analysis was prepared of the cost of providing capacity in the wastewater system to serve future connections. In addition, charges were calculated to recover these costs from new connections. The SDC charges to a new, single-family residential equivalent dwelling unit (EDU) customer would increase from $2,832 under the current fee schedule to $2,977 under the proposed fee schedule. The fee structure for new commercial and industrial users is based on the estimated contributions of flow, BOD, and TSS to the system in gallons per day (gpd) or pounds per day (lbs/day). The SDCs are shown in Table ES-46.

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Table ES–5
CITY OF WOODBURN
WASTEWATER RATE MODEL
TYPICAL MONTHLY WASTEWATER BILL COMPARISONS
NOVEMBER 1, 1995 RATE IMPLEMENTATION

CUSTOMER CLASS	Monthly Use (hcf)	Typical Vol + Min Charge Bill	Typical Flat Rate Bill	Typical Current Bill
Residential (per unit)	5	$20.20	$27.48	$14.20
Residential (per unit)	10	$38.15	$27.48	$14.20
Residential (per unit)	15	$56.10	$27.48	$14.20
Residential (per unit)	25	$92.00	$27.48	$14.20
Residential (per unit)	35	$127.90	$27.48	$14.20
Commercial	6	$25.54	(a)	$14.85
Commercial	10	$47.50	(a)	$23.65
Commercial	25	$129.85	(a)	$56.65
Commercial	50	$267.10	(a)	$111.65
Commercial	100	$541.60	(a)	$221.65
Industrial (25 lbs BOD/9 lbs TSS) (b)	10	$54.62	(a)	$367.60
Industrial (250 lbs BOD/90 lbs TSS) (b)	100	$470.42	(a)	$367.60
Industrial (750 lbs BOD/270 lbs TSS) (b)	300	$1,394.42	(a)	$367.60
Industrial (1750 lbs BOD/630 lbs TSS)	700	$3,242.42	(a)	$691.22
Industrial (2500 lbs BOD/900 lbs TSS)	1,000	$4,628.42	(a)	$987.46

Notes:
(a) Commercial and Industrial users are charged based on volumes and not on a flat rate basis.
(b) Industrial typical current bill equals the minimum charge.

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Table ES–6
CITY OF WOODBURN
WASTEWATER SYSTEM DEVELOPMENT CHARGE MODEL
COMBINED SDCs

Reimbursement SDC per EDU	$167
Improvement SDC per EDU	$2,810
Combined SDC per EDU	$2,977
Flow Combined SDC ($/gal/day)	$10.69
BOD Combined SDC ($/lb/day)	$1,446.42
TSS Combined SDC ($/lb/day)	$532.14

Notes:
Current Sewer SDC = $2,832 per EDU.
One EDU = 175 gpd of Flow and .56 lbs/day of BOD & TSS.

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Recommendations

CH2M HILL recommends that the City of Woodburn:

- Adopt the proposed volume rates plus minimum service charge rate structure shown in Table ES-2 if new rates are implemented on July 1, 1995.
- Adopt the proposed volume rates plus minimum service charge rate structure shown in Table ES-4 if new rates are implemented on November 1, 1995.
- Adopt the Wastewater Advisory Committee's recommendation that revenues generated by the poplar tree plantation harvest be used for rate adjustments.
- Adopt the proposed combined SDCs shown in Table ES-4.
- ~~Implement these new rates and SDCs on July 1, 1995.~~

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WASTEWATER ADVISORY COMMITTEE
MINUTES OF APRIL 18, 1995

WORKSHOP w/ CITY COUNCIL
CITY COUNCIL CHAMBERS

Members Present:	Scott Burlingham, Chairman	Council Members Present:	Richard Jennings
	Preston Tack		Mary Chadwick
	Gil Flaugher		Don Hagenauer
	Rich Barstad		Nancy Kirksey
	Walt Lawson		
	JoAnn Bjelland		
	G.S. (Frank) Tiwari	Public Information:	Barbara Lucas
	Forest Mills	Staff:	Frank Sinclair
			Dean Morrison
Members Absent:	Marv Shelby		Randy Rohman
			Chris Childs
Consultants:	Daria Wightman, CH2M Hill		
	Bob Tomlinson, CH2M Hill		

Chairman Scott Burlingham opened the Wastewater Advisory Committee meeting at 7:05 p.m.

Rich Barstad/ Gil Flaugher motioned and seconded approval of minutes, as corrected, of March 9, 1995. Motion passed unanimously.

Rich Barstad/ Walt Lawson motioned and seconded approval of minutes of March 20, 1995. Motion passed unanimously.

Chairman Burlingham asked committee members to review public input and discuss items of concern. Considerable time was spent discussing points raised by a resident who wrote that he had seen no factual necessity for a $50 million treatment plant, and suggested that Woodburn and other cities challenge federal and state unfunded mandates.

Councillor Dick Jennings cautioned Woodburn residents that the unfunded mandate legislation before Congress doesn't do anything for present unfunded mandates; it will only affect those done in the future. We will have to pay the full cost of our plant. Other Councilors stressed that Woodburn does not have the resources to challenge the federal government.

In the discussion about the need for the plant, staff reiterated the points it has made many times before about the present plant reaching the end of its useful life, being too small to serve future growth, and unable to meet stricter federal requirements. In fact, present federal requirements are not met by the existing plant, and to avoid fines the city is

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following DEQ's stipulated final order and has agreed to build a plant which will meet federal pollution limits by the year 2000.

The members were provided with page 2 of volume 1 Executive Summary and were requested to replace the old page from the Executive Summary given to them in the last meeting. Members were informed that only one sentence relating the need for land conservation and development (LCDC) was added and it is shown in gray shade. In addition, tables and charts were added. Daria Wightman revised the original executive summary because the committee had asked for this addition prior to presentation to council.

Frank informed the committee that a letter had been received from an environmental group asking the city for information on sewage services that are provided by the city outside the existing city limits such as McLaren School. He informed the committee that, based on the council's agreement, city has been serving Agripac's domestic waste for over 30 years and McLaren's waste for about fifteen years. Although the city's proposed plan includes the service area outside the urban growth boundary (UGB), the intent is to protect the city's ground water. No sewer extension will be made outside the urban growth boundary without approval of LCDC.

Preston Tack/ Gil Flaugher motioned and seconded providing high school and city library with copies of Wastewater Facilities Plan if they are willing to keep one set and use the process that is used to update this plan will be used for high school and library copies also. Motion passed unanimously.

There was a comment made by Preston Tack followed by a brief discussion on current cost estimates that could vary 50 percent higher or 30 percent lower than the projected cost.

Preston Tack/ Gil Flaugher motioned and seconded that separate bids be requested on further engineering and/or the construction bid documents of all or any of the main categories such as treatment plant, collection system and poplar tree plantation, with the purpose being to take advantage of competitive bidding and lowering the cost in any of these categories.

Rich Barstad commented that it can be difficult to work with an engineer through a certain planning process and then switch engineers. It is time consuming to bring the new engineer up to speed. If you are unhappy with the present engineer because of certain circumstances like quality of work or not finishing on time, that is a definite problem.

Frank Tiwari commented that the law requires we use the bidding process on public works contracts where construction is involved, but this is not required for professional services where decision making takes place and expert knowledge is needed. Frank suggested we just follow whatever the law is and leave the decision to the governing body.

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Preston Tack commented that he could see this approach for state of the art technology, but he has seen no particular improvement or breakthrough in the wastewater plant area in the last 25 years. Frank Tiwari suggested that technology in wastewater is continuously evolving, and the use of poplar tree plantations and selector technology are very recent approaches to effluent treatment that City of Woodburn plans to utilize.

Chris Childs expressed concern that loss of continuity could increase the price of this project by millions because of different approaches. Staff and city council deserve flexibility to proceed. In Chris' estimation, for the committee to dictate how we go about selecting our engineering firm may be going far afield.

Councilor Jennings said that too much time has been used to discuss committee items and asked that council members be given an opportunity to discuss the items on the main agenda.

Walt Lawson/ Preston Tack motioned and seconded tabling the item until the next meeting. Motion passed unanimously.

The Pudding River Clean Up video was shown. Positive comments were made about the video by council members and members of the committee. Engineers advised that the comments from the public at the Wastewater Open House do not require changes to the facilities plan. Rates may be adjusted, according to council's desires, to generate the needed revenues and meet the requirements of residents and other customers.

Discussion of rates ensued with a suggestion to institute a flat rate for residential, which would be $26.10, taking the place of the rates based on usage. The proposed rates are based on a minimum charge of $19.19 for 500 cubic feet per month, plus $3.59 for each additional 100 cubic feet. Cubic footage would be based on the average monthly water use during the winter months of November, December, and January.

The proposed rates for commercial are based on a minimum of $24.27 for the first 600 cubic feet per month, then $5.08 for each additional 100 cubic feet over 600. Commercial rates, like residential, would be based on the average monthly water use during the winter months of November, December, and January.

There are only three industrial sewer users at present: Smuckers, Conroy Packing, and Specialty Polymers. Rates for these three firms are based on metered sewage flow, plus charges for loads of biochemical oxygen demand (BOD) material and total suspended solids (TSS). Councillor Dick Jennings said that he and other councillors had been discussing rates with their constituents and among themselves. He and two other councillors, Figley and Sifuentez, were in favor of a flat rate of $26.10 every month for residential customers. He was willing to pay more each month for his sewer service so that people with large families would not have to pay so much. Dick Jennings shared a suggestion from three of city council members that industrial and commercial be combined into one

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classification. Another option could be a division of commercial into high, medium, and low loading classifications.

Staff members from the city and the consulting firm pointed out that to establish high, medium, or low categories for load, the sewage of business firms would have to be sampled periodically. This would be expensive for the firms. Jennings suggested that testing may be necessary initially but there should be no need for perodic testing because loadings from businesses would not vary, and another option for professionals will be to use the values in technical books.

The committee agreed that the facility plan volumes dealing with the treatment plant and collection system do not need any changes, but some modification of the rate structure volume could be considered because of input from the council and the public. Discussion followed that any change in the rate classes should have no bearing on domestic rates, and the present fair costs for distribution among domestic, commercial, and industrial uses will be kept intact.

Scott Burlingham said that at the May 8 public hearing it could be brought out that the committee and council are still working on rate structure. The ultimate result will be the system will pay for itself. Dick Jennings agreed that, even if some adjustments are made, council will adopt a plan that will generate revenues shown in the study.

Frank Tiwari announced that there would be a public hearing at the May 8, 1995 city council meeting to review all three sections of the Woodburn Wastewater Treatment Facility Plan: collection, treatment, and rates. A workshop with the city council will be scheduled after that hearing.

Preston Tack/ Gil Flaugher moved to adjourn. Meeting adjourned at 9:39 p.m.

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RECEIVED MAY 16 1995

FLAUGHER & TACK
Members—Wastewater Advisory Committee
Woodburn, OR 97071
1552 Astor Way Tel. 982-5719
2197 Camellia Way Tel. 981-6712

May 12, 1995

Mr. Scott Burlingham, Chairman
Wastewater Advisory Committee
City of Woodburn, OR 97071

Dear Scott:

The Committee has recently reviewed, and forwarded to the Council, the three volume Wastewater Facilities Plan(WFP). Volume 1 is on the treatment works. Volume 2 is on the collection system. Volume 3 is on the financing and user rate structure. Months of work by CH2M/Hill, consultation with the committee, and two well publicized open houses, have gone into their preparation and it seems appropriate to assume that this WFP will be approved by DEQ.

The time and effort that have gone into the preparation of these documents, and their DEQ approval, must surely provide well defined scopes for the final design and preparation of bid document phases of this project.

There are at least three elements of the Woodburn WPF that justify consideration for open bidding on the final designs and preparation of bid documents:

1. Wastewater Treatment Plant
2. Poplar Tree Plantation
3. Collection System

Why open up the final design and bid document phase to competitive bidding on any or all of the elements above?

A. Giving this work to one consulting firm would freeze out other capable engineering firms, large and small.

B. Potential for innovative alternate design proposals which might contribute to lower construction and/or operating costs would be minimized.

C. Potential for final design cost savings through separate competitive bids on any or all of the elements will never be known unless called for.

D. Current estimates for engineering are included in a category entitled, "Legal,Admin.,Engng.." which, for Phases 1 & 2, come to a little over $3.7million, 16.7% of the entire project estimated cost. But this figure is subject to a plus 50%($11million) or minus 35%($4.8million) range of accuracy. It seems reasonable to assume that final design and specifications will be a sizable chunk of these figures. Isn't it worth assuring that the rate payers won't be paying too much for this part of the costs?

E. The same philosophy would apply to construction bids. Contractors could bid on all of the work, showing costs for leach element, or any one element. Here again, large and small, general land specialized, would have an opportunity to bid.

The prime objective of this proposal is to assure that, while quality workmanship is insisted upon, it will be at the lowest immediate and ultimate construction and operating costs to Woodburn taxpayers.

S' rely yours,

Preston B. Tack

Preston B. Tack

Gilbert E. Flaugher

Gilbert E. Flaugher

wacwdbaV1/winworks/acost06.ltr

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Woodburn Transportation System Plan

Woodburn, Oregon

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June 1996

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Woodburn Transportation System Plan

Woodburn, Oregon

Prepared for:
City of Woodburn

Prepared by:
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610 SW Alder, Suite 700
Portland, Oregon 97205
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Project No.: 1374

June 1996

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Section 1

Executive Summary

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1.0 Executive Summary

Using a combination of funds from the Oregon Department of Transportation (ODOT) Transportation & Growth Management Program and the City of Woodburn, the City in 1994-95 sponsored the development of an updated Woodburn Transportation System Plan. The plan has a planning horizon to the year 2015, and represents for the first time a comprehensive, multi-modal transportation plan to guide transportation investment in the City over the next 20 years.

As an input into the plan, an assessment of existing conditions was conducted, as well as an assessment of highway and transit system alternatives, and potential financing mechanisms. To aid the alternatives analysis and plan development, an updated year 2015 traffic forecasting model was developed, using year 2015 population and employment projections.

The highway system alternatives analysis, in addition to a no-build alternative, focused on an assessment of three I-5 access alternatives:

1. develop a split diamond interchange;
2. develop a second interchange at Butteville Road; and
3. improve the existing interchange, including an option to convert to a partial cloverleaf configuration.

Included in all three "build" alternatives was the development of the South Arterial from Highway 214 on the west to Highway 99E on the east, as well as improvements to other minor arterials and collectors on the City street system.

The alternatives analysis included an assessment of different intracity and intercity bus service configurations to serve Woodburn. Improvements to the existing single fixed-route, one-way bus service in the city were assessed, including making the route two-way, increasing the service frequency, and/or establishing separate east and west side bus routes. Intercity shuttle bus service transportation plan options from Woodburn to Portland and Salem were also examined.

Finally, alternate pedestrian and bicycle facility and Transportation Demand Management (TDM) strategies were evaluated to further reduce vehicle trip marking in Woodburn.

Transportation Plan

The final Transportation System Plan includes recommended facilities, standards, and improvements for the following modes of transportation:

1. roadway;
2. transit;
3. pedestrian;
4. bicycle;
5. golf cart; and
6. rail.

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Roadway

There are four critical elements to the roadway plan. The first is the designation of an I-5 interchange improvement, the specific configuration to be identified in a future Refinement Plan. The second is the development of a South Arterial facility between Highways 219 and 99E. The third is the reclassification of the collector street system into two categories: service collector and access street. Several streets are identified for minor widening to provide bike lanes and sidewalk treatments in the future. The fourth is the identification of potential local street connections within and between neighborhood areas.

Transit

The transit plan proposes initially converting the existing intercity bus route to two-way operation, with possible improvement in service frequency over time to 30 minutes, at least during weekday peak periods. Intercity shuttle bus service to downtown Portland (with an intermediate stop at the Tualatin park-n-ride), and to downtown Salem is also proposed. In the longer-term, a downtown transportation center should be considered if an intercity passenger rail stop is developed, and/or there is sufficient transit demand to warrant both east and west side routes. The existing paratransit and taxi services in the City are recommended to be maintained as well.

A more detailed Transit Development Program study is recommended to detail specific operating strategies and associated maintenance and capital requirements and funding sources.

Pedestrian

The pedestrian facility plan identifies providing sidewalks on all arterial and collector streets in Woodburn in the future. The plan also shows the development of an off-street pathways system along the Mill Creek and Goose Creek corridors.

Bicycle

The bicycle facility plan calls for an expansion of the existing system of bike lanes on City streets into a comprehensive system of bike routes and lanes. All arterials and service collectors are identified bike routes, with bike lanes desired where physically possible. The off-street pathways plan would include bike trails either combined or separated from pedestrian trails.

Golf Carts

To improve accessibility of golf cart users to central Woodburn, certain streets between Senior Estates and downtown Woodburn would be designated for golf cart use, using widened shoulder areas which would also be used by bicycles. Golf carts would also be allowed on certain segments of the off-street pathways system, combined with bicycle use where allowed.

Rail

The rail plan recognizes the continued presence of the Southern Pacific Railroad through Woodburn, including spur tracks serving local businesses. All existing at-grade railroad/high-

way crossings are proposed to be maintained. In the longer-term, improvements to trackage and crossings could occur if intercity passenger rail service were implemented, with a possible stop in Woodburn.

A 20-year program for transportation improvements in Woodburn is an integral part of the TSP. Improvements needed in the 0-5, 5-10, 10-20, and 20+ year time frame have been identified. The current estimated cost of all identified improvements (in 1995 dollars) is about $62 million. Over $20 million of this is in the construction of an I-5 interchange improvement and the South Arterial. Potential funding sources include federal funding through the Intermodal Surface Transportation Efficiency Act (ISTEA), state gas and lottery funds, and local gas tax, traffic impact fee, bonding revenues, developer participation and other funding sources that may be developed during the effective time of the plan.

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Section 2

Introduction

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2.0 Introduction

In April 1991, the Land Conservation and Development Commission (LCDC), with the concurrence of the Oregon Department of Transportation (ODOT), adopted the Transportation Planning Rule (TPR), OAR 660 Division 12. The TPR requires local jurisdictions to prepare and adopt a Transportation System Plan (TSP) by May 1996.

The Transportation Planning Rule requires ODOT, Metropolitan Planning Organizations, and each county and city to prepare and adopt a transportation system plan (TSP) and implementing regulations (OAR 660-12-015). For a city with the population of the City of Woodburn, the TSP must include:

1. determination of transportation needs,
2. road plan for arterials and collectors,
3. public transportation plan,
4. bicycle and pedestrian plan,
5. air, rail, water and pipeline transportation plan,
6. policies and land use regulations for implementing the TSP as provided in OAR 660-12-045,
7. transportation financing plan, and
8. ordinances to assure coordinated planning of transportation facilities.

With a partial grant from the Oregon Department of Transportation, the City of Woodburn initiated its transportation system plan in November of 1994. As part of this process the City's Transportation Task Force (which previously existed), City Staff, ODOT, and Marion County provided guidance and review of preliminary products in the development of this plan. In addition three public open houses were held. The purpose of these open houses was to ensure that the recommended transportation system plan would reflect the local needs of the citizens of Woodburn.

The development of the Woodburn Transportation System Plan began with the Transportation Task Force, Woodburn, ODOT, and Marion County staff working together to develop the transportation system plan goals and objectives which are presented in **Section 3**.

The transportation analysis began in **Section 4** with an assessment of existing traffic conditions and operations in the city of Woodburn. In this section of the document, among other issues, existing average daily traffic, traffic operations at key intersections, transit, pedestrian and bicycle routes are summarized.

To determine transportation needs in the future, a travel demand model was developed which forecasted year 2015 weekday PM peak hour traffic volumes. In **Section 5** these forecast volumes were analysed on alternative roadway transportation system improvements. The results of this analysis indicated which alternatives would provide for the best traffic operations in the future.

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Section 6 presents options for improving the public transportation system in Woodburn, including both intra city ind inter city bus service improvements.

Section 7 presents options for improving pedestrian and bicycle facilities in Woodburn, as well as Transportation Demand Management strategies.

Section 8 presents the results of the access management analysis performed on Highway 214 between Woodland Road and Settlemier Avenue, and on Highway 99E between Lincoln Street and the south city limits. In this analysis, public road, traffic signal and private access spacing was analyzed to determine the need for consolidating access in order to develop safer, more efficient State highways through Woodburn.

The next step, as summarized in **Section 9**, involved the development of the individual elements of the Woodburn Transportation System Plan. Included are the highway plan, transit plan, pedestrian/bicycle facility plans, golf cart facility plan, and a TDM plan.

In **Section 10** the transportation finance plan is developed. This plan identifies alternatives available to the City of Woodburn to fund transportation system improvements which would provide adequate capacity and access for all users of the Woodburn transportation system.

In **Section 11**, land use ordinances necessary to facilitate implementation of the Transportation System Plan are summarized.

This document concludes with **Section 12** which lists the requirements and recommendations of the Oregon Transportation Planning Rule (OAR 660 Division 12) and outlines how the Woodburn Transportation System Plan complies with the TPR.

This project is partially funded by a grant from the Transportation and Growth Management (TGM) Program, a joint program of the Oregon Department of Transportation and the Oregon Department of Land Conservation and Development. TGM grants rely on federal Intermodal Surface Transportation Efficiency Act and Oregon Lottery Funds. The contents of this document do not necessarily reflect views or policies of the State of Oregon.

Section 3

Goals and Policies

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3.0 Goals and Policies

The Woodburn Transportation Task Force in concert with City of Woodburn staff developed five principal goals (and associated policies) which guided the development and implementation of the Transportation System Plan. These goals are identified below:

Goal 1

Develop a multi-modal transportation system that avoids or reduces a reliance upon one form of transportation, and minimizes energy consumption and air quality impacts.

Policies

1. Develop an expanded intracity bus transit system which will provide added service and route coverage to improve the mobility and accessibility of the transportation disadvantaged, and to attract traditional auto users to use the system.
2. Develop a plan for providing travel options between Woodburn and Portland and/or Salem, including intercity bus service and potential bus/carpool park-n-ride facilities.
3. Develop a plan for accommodating golf cart travel in Senior Estates, Tukwila, and other new golf course residential communities and for connectivity between residential and commercial areas in the Woodburn area.
4. Develop a bikeway system which will provide routes and facilities to allow bicyclists to travel from residential areas to schools, parks, places of employment and commercial areas. Off-street facilities in City greenway/park areas will be identified. Insure all new collector and arterial streets are constructed with bike lanes.
5. Identify sidewalk and off-street pathway improvements to improve pedestrian mobility within neighborhoods and between residential areas and schools, parks, places of employment and commercial areas. Insure all new collector and arterial streets are constructed with sidewalks.

Goal 2

Develop a street system which will handle projected year 2015 traffic demands in the Woodburn area, and interconnects residential areas with employment centers, schools, parks, churches and regional transportation facilities.

Policies

1. Develop an updated roadway functional classification plan for the Woodburn area, that reflects the desired function of different roadways, and is consistent with current federal guidelines for the designation of major streets in an urban area.

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2. Develop a strategy for providing improved access to I-5 from the Woodburn area, through either improvements to the existing Highway 214 interchange and/or a new interchange in the Woodburn vicinity (with supporting local roadway improvements). This strategy will be developed following a refinement study as outlined in the Transportation Planning Rule.
3. Develop a strategy for improving Highways 219/214, 211, and 99E through Woodburn, including added travel lanes, signalization, and access management.
4. Identify new east-west and north-south collector/minor arterial streets within the City to relieve traffic demands on Highways 219/214, 211, and 99E.
5. Develop updated street design standards for arterials, collectors, and local streets.
6. Identify a final strategy for paving current unimproved streets in the City.
7. Identify the need for added public parking provisions in Woodburn, including park and ride, as well as a plan to support increased carpooling and transit use in the future.
8. Develop a capital improvement program that will fulfill the transportation goals established by the community.

Goal 3

Develop transportation improvements that will improve overall traffic safety in the Woodburn area.

Policies

1. Develop access management strategies for Highways 219/214, 211 and 99E through Woodburn, particularly focusing on the section of Highway 214 between I-5 and Cascade Drive, and Highway 99E south of Lincoln Avenue.
2. Develop a plan for improving pedestrian and bicycle safety for travel to/from local schools.
3. Identify street/railroad crossings in need of improvement, and those that should be closed or relocated.
4. Develop a plan for designated truck routes through the City, and a plan to handle truck and rail hazardous cargoes.

Goal 4

Develop a set of reliable funding sources that can be applied to fund future transportation improvements in the Woodburn area.

Policies

1. Evaluate the feasibility of the full range of funding mechansims for transportation improvements.

2. Evaluate the feasibility of instituting an added City gas tax for transportation improvements.
3. Identify a traffic impact fee structure associated with new developments in the Woodburn area to fund transportation improvements.

Goal 5

Develop amendments to City land use standards and ordinances to reduce travel demand and promote use of modes of transportation other than the automobile.

Policies

1. Identify changes in the Woodburn Zoning Ordinance to encourage implementation of Transportation Demand Management (TDM) strategies by local businesses. TDM strategies should include bicycle and carpool parking provisions, and allowable overall parking reductions for employer institution of TDM strategies, including transit fare subsidies, carpool matching programs, and flexible work hours.
2. Identify changes in the Woodburn Zoning ordinance to encourage transit and pedestrian-oriented development. This includes proper building orientation to improve access for transit users and patrons, direct pedestrian connections, and bus stop provisions where appropriate.
3. Identify changes in the Woodburn Subdivision Standards to encourage neo-traditional development patterns and adequate local street standards to accommodate all modes of transportation.
4. Adopt traffic impact analysis guidelines to be used by the City and developers to identify the impact of new development on street system improvement needs.

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Section 4

Existing Conditions

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4.0 Existing Conditions

4.1 Introduction

This section of the Woodburn Transportation System Plan provides a summary of existing transportation conditions within the Woodburn Urban Growth Boundary. The section is divided into two categories: transportation facilities, and traffic operations. The transportation facilities section summarizes the existing:

- street functional classification system;
- pedestrian and bicycle facilities;
- rail service/roadway grade crossings;
- air transport facilities; and
- water transport facilities.

In the existing traffic operations section, the principal issues are:

- intersection traffic control and lane configuration;
- recent 24-hour traffic counts;
- intersection PM peak hour traffic volumes;
- intersection PM peak hour traffic level of service; and
- high accident locations.

4.2 Transportation Facilities

4.2.1 Functional Classification

The 1985 Woodburn Transportation Plan identifies five categories of road facilities. Those categories are:

- freeway,
- principal arterial,
- minor arterial,
- collector, and
- local street.

The adopted functional classification plan is shown in Figure 1, and the roadway standards are summarized in Table 1.

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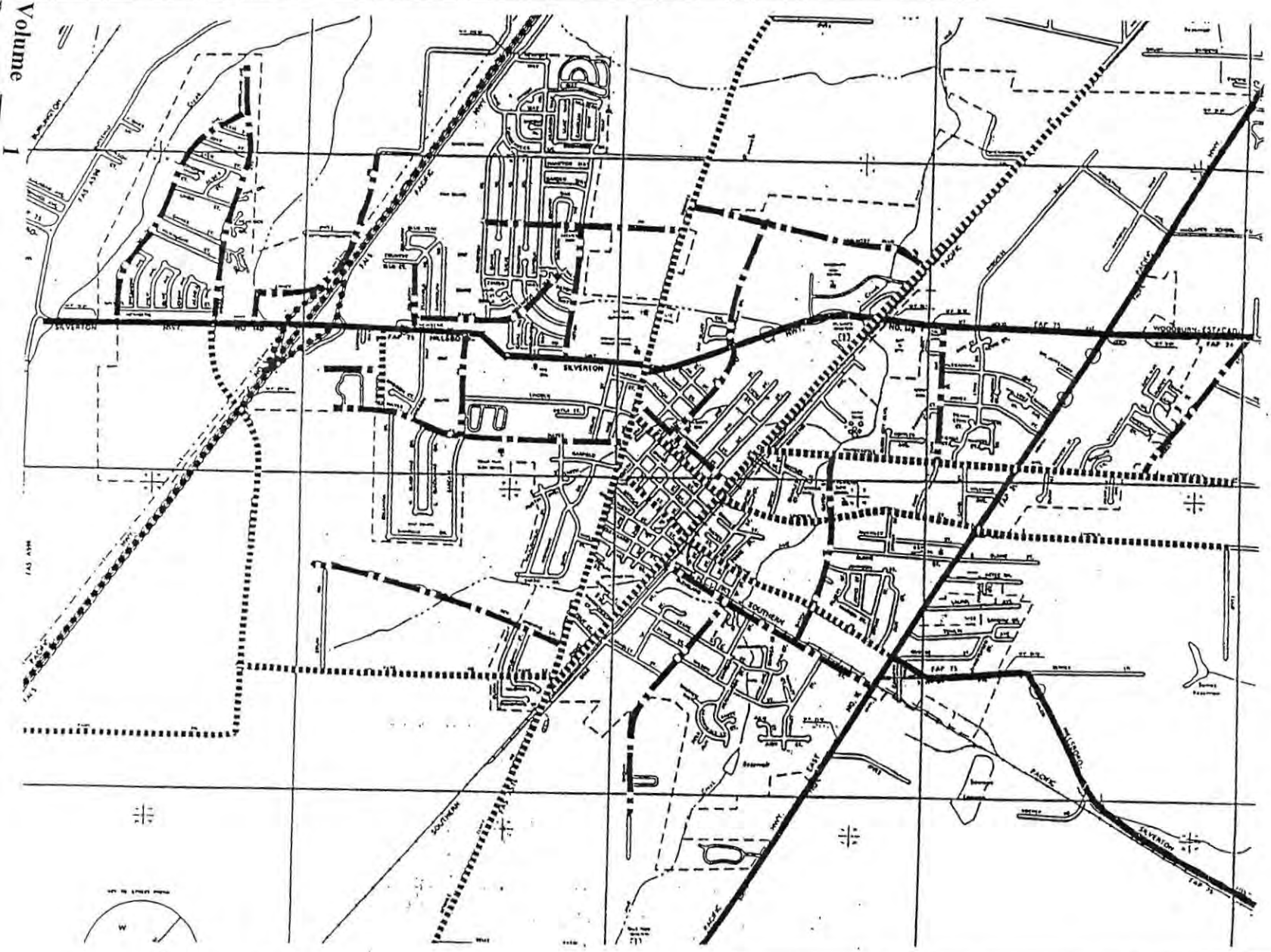

LEGEND

- FREEWAY
- MAJOR ARTERIAL
- MINOR ARTERIAL
- COLLECTOR

1985 STREET FUNCTIONAL CLASSIFICATION PLAN

CITY OF WOODBURN
TRANSPORTATION SYSTEM PLAN
JUNE 1996

FIGURE 1

Table 1

Existing Street Functional Classification Standards

Functional Classfication	R/W (ft)	Pavement (ft)	Lanes	Volume (AST)	Miles within UGB
Freeway	120+	12/lane	4-8	>30,000	2.2
Principal Arterial	80-120	12/lane	2-5	>10,000	6.3
Minor Arterial	80-100	36-44	2-4	3,000-10,000	9.8
Collector	60	34-44	2	2,000-5,000	9.3
Local	60	34-36	2	<2,000	27.0 (incl.cul-de-sac)
Cul-De-Sac	50-60	34-36			
Cul;-De-Sac (Turnaround)	45.6 Radius	40 Radius			

4.2.2 Pedestrian and Bicycle Facilities

Figure 2 shows the location of pedestrian and bicycle facilities, and their relationship to major activity centers. Overall, the existing pedestrian and bicycle facility system is deficient due to lack of connectivity and incompleteness (particularly the overall lack of off-street pathways and bike routes).

The majority of the sidewalks in Woodburn are provided on local streets. With the exception of Senior Estates (north and south of Highway 214) sidewalks are provided in most of the residential areas and also in downtown Woodburn. Similarly, sidewalks are provided on the principal and minor arterials in Woodburn, although in many locations on only one side of the street. There are also sidewalks on Highway 214 between Interstate 5 and Meridian Drive, and from Progress Way through Highway 99E. In addition, there are sidewalks on Highway 99E from the northern city limits south to Lincoln Street. Finally, there are sidewalks on Settlemier Avenue from Garfield Street to Parr Road. There are very few off-street pathways.

Figure 2 also shows the location of existing bike routes within the City of Woodburn. As shown in this figure, there are three designated bike routes within the City:

- Highway 214 between Settlemier Avenue and Highway 99E;
- Highway 99E between the northern City limits, and Lincoln Road; and
- West Hayes Street from Nellie Muir School to Settlemier Avenue.

There are no current off-street bicycle facilities in Woodburn.

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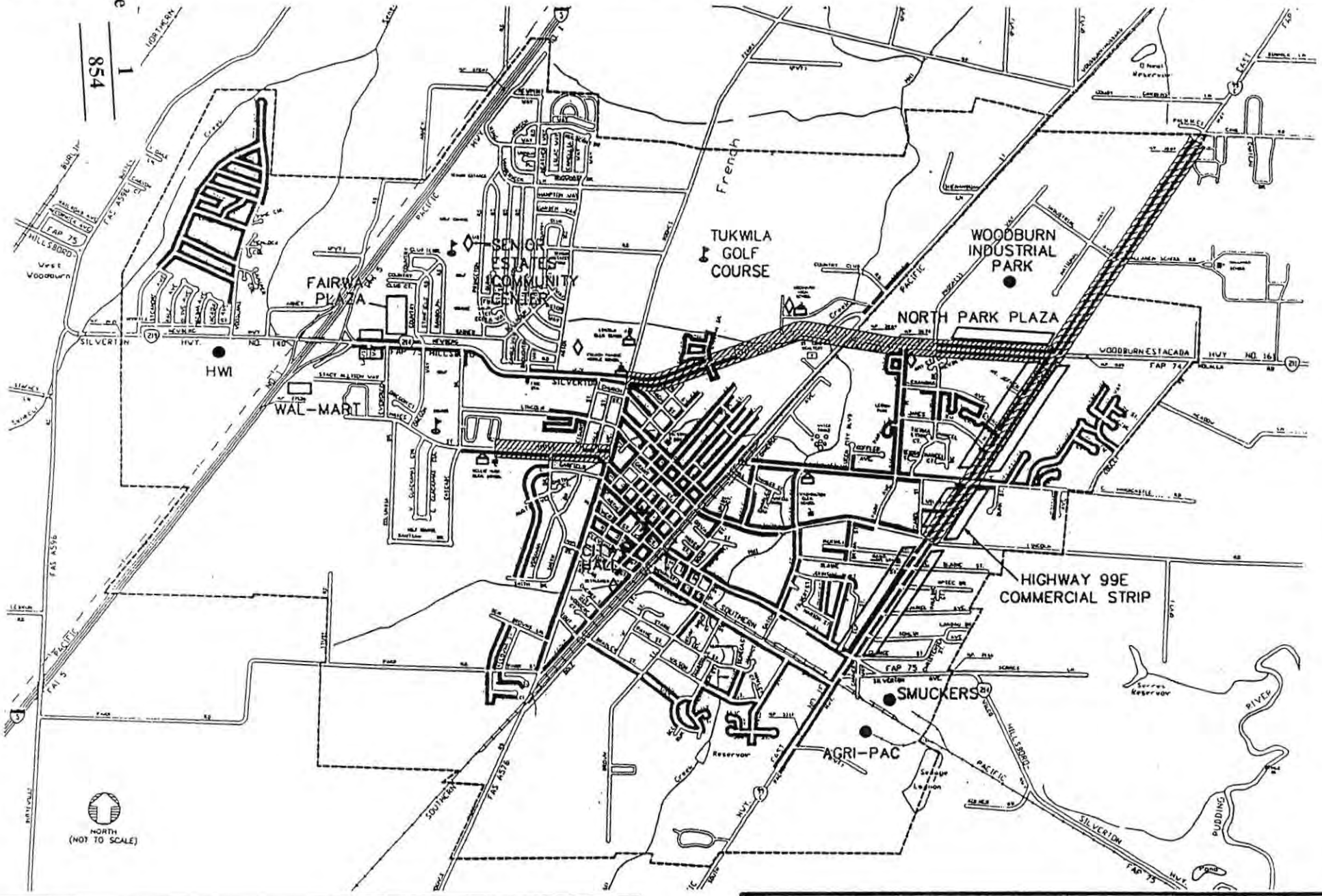

LEGEND

- WALKS WITH CURBS
- BIKE ROUTES
- ◊ = RECREATION
- ⌂ = SCHOOL
- ▭ = RETAIL
- ⚑ = GOLF COURSE
- ☆ = DOWNTOWN
- ● = EMPLOYMENT NODES
- O = GOVERNMENT CENTER

EXISTING PEDESTRIAN AND BICYCLE FACILITIES

CITY OF WOODBURN
TRANSPORTATION SYSTEM PLAN

JUNE 1996

FIGURE 2

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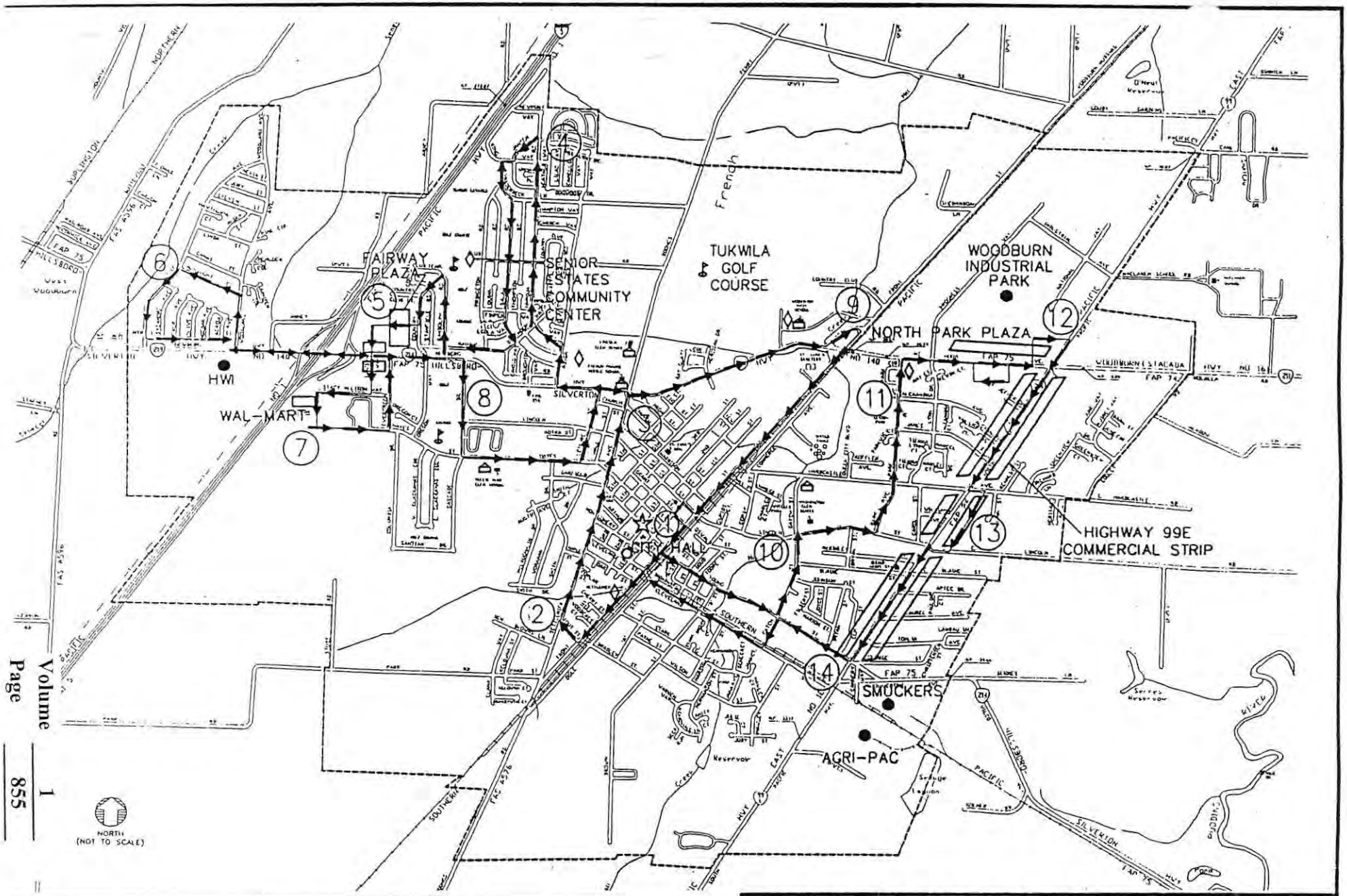


LEGEND

→ → DIRECTION OF TRAVEL

(1) SCHEDULE TIME POINTS

◊ = RECREATION
⌂ = SCHOOL
▭ = RETAIL
⚑ = GOLF COURSE
☆ = DOWNTOWN
● = EMPLOYMENT NODES
○ = GOVERNMENT CENTER

EXISTING TRANSIT SERVICE (FIXED-ROUTE)

CITY OF WOODBURN TRANSPORTATION SYSTEM PLAN

JUNE 1996

FIGURE 3

1373FBUS

4.2.3 Woodburn Transit and Paratransit System

The City of Woodburn operates two transit systems: one is a single, fixed route system called the *Woodburn Transit System*; the other, the *Woodburn Paratransit System,* is a dial-a-ride system.

The *Woodburn Transit System* operates from 9:00 AM to 5:00 PM Monday through Friday[1]. Figure 3 shows the 15.3 mile fixed-route of the *Woodburn Transit System.* As shown in the figure, there are thirteen scheduled time points on this system. That is, there are thirteen locations where patrons can expect the bus to be at scheduled times. These locations are identified in Figure 3. Besides these locations, the bus will stop anywhere that it is safe to pick up or drop off passengers. In 1994, the adult fare was 75 cents, and children under the age of six could ride for free when accompanied by an adult.

As shown in Figure 4, the peak transit patronage for the *Woodburn Transit System* occurred in fiscal year (FY) 1985 (July 1 - June 30), most likely due to the introduction of Saturday service. From FY 1986 to FY 1992 patronage declined. Since 1992 transit patronage has been increasing. In 1994, approximately 28,000 people rode the *Woodburn Transit System.*

There are three part time drivers for the *Woodburn Transit System*, and the City owns two buses. The primary bus is a 1993 El Dorado 22-passenger bus which is lift equipped. The backup bus, which operates when the primary bus needs maintenance, is a 1987 Champion 23-passenger bus which is also lift equipped. In fiscal year 1993-1994, the City budgeted $130,525 for total operating expenses.

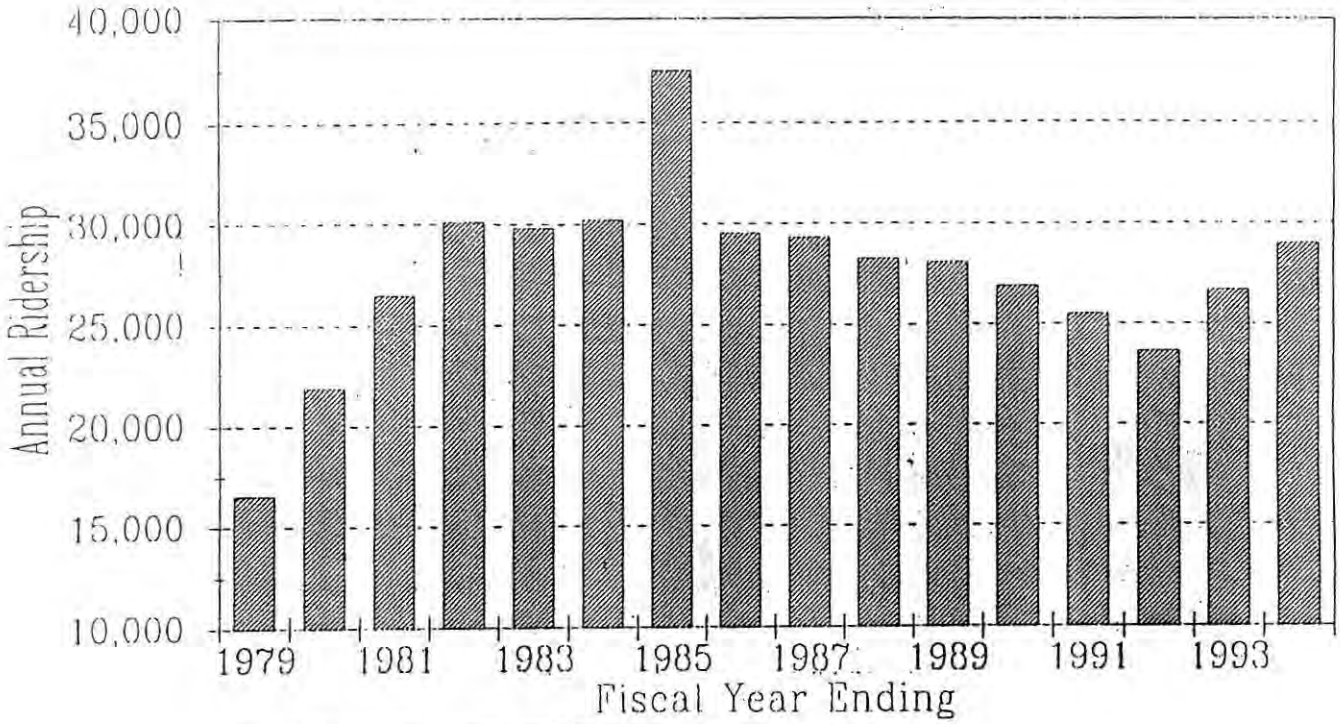
The *Woodburn Paratransit System* provides transportation for disabled persons who can not utilize the fixed route system. The City owns one van which operates Tuesday, Thursday and Friday from 9:00 AM to 5:00 PM. The service provides door-to-door transportation; reservations must be made 24 hours in advance. There is a $2 charge for each round trip.

As shown in Figure 5, transit patronage on the *Woodburn Paratransit System* has increased since FY 1992. In FY 1994, approximately 3,400 people used the *Woodburn Paratransit System.*

The City also sponsors a free volunteer driver service to transport elderly and disabled citizens from their homes to medical appointments in Woodburn, Salem, and Portland. The volunteers use their own vehicles. The City provides support through municipal tax dollars, personnel, office space and other materials. Donations are a main source of program funding.

In addition to the City-operated paratransit service, the privately-owned Woodburn Taxi company operates within a 100 mile radius of Woodburn. This service, which is seven years old, carries about 500 passengers per month, comprised primarily of Senior Citizens and the Hispanic population in the City. A lot of the trips are out-of-town, including trips to Portland Airport. One vehicle is used to provide the service. The operating hours for the service are 7 AM to 6 PM Monday through Saturday. The fare is an initial $2.40 at the start of a trip, plus $1.50 per mile thereafter. If the service is maintained over the long-term, the owners are contemplating expanding the service into the Canby area.

1 Saturday service was provided between 1985 and 1990.

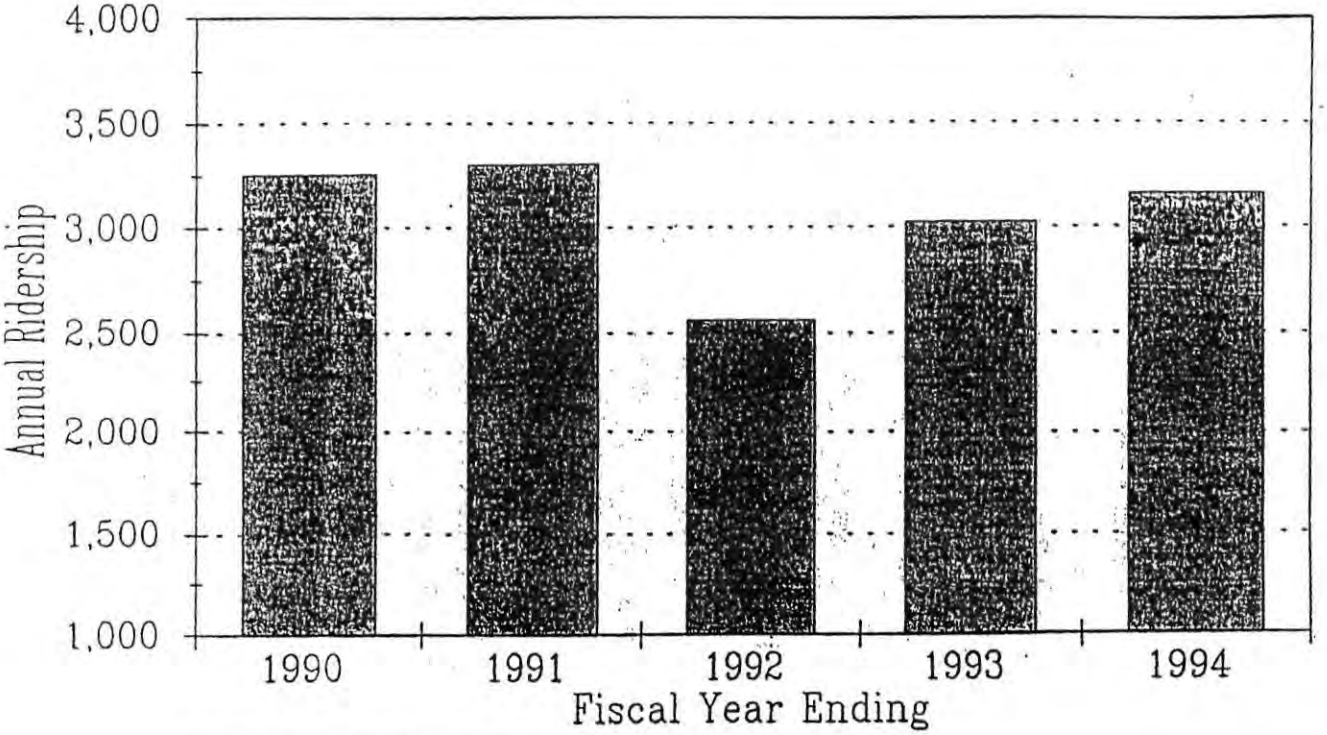


*Fixed Route Service. Saturday Service was included 1985-1990

WOODBURN TRANSIT SYSTEM
RIDERSHIP TRENDS (FY JULY 1-JUNE 30)

CITY OF WOODBURN
TRANSPORTATION SYSTEM PLAN
JUNE 1996

FIGURE
4



* Door - to - Door service; provided three days per week.

1373F05

WOODBURN PARATRANSIT SYSTEM
RIDERSHIP TRENDS (FY JULY 1–JUNE 30)

CITY OF WOODBURN
TRANSPORTATION SYSTEM PLAN

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FIGURE 5

4.2.4 Rail Service/Roadway Grade Crossings

Figure 6 shows that there are nine at-grade railway crossings of the Southern Pacific Line in Woodburn. Freight trains run principally on the tracks along Front Street, with local businesses being served by the spur lines along Cleveland Street. There are no passenger train stops in the City.

In 1994, freight trains traveled through the City of Woodburn on average 25 times per day. The maximum allowable speed of freight trains within Woodburn is 45 miles per hour. Outside of the city limits trains can travel up to 70 miles per hour[2].

4.2.5 Air Transport Facilities

There are no air transport facilities within the Woodburn Urban Growth Boundary.

4.2.6 Pipeline Transport Facilities

There are no pipline transport facilities within the Woodburn Urban Growth Boundary.

4.2.7 Water Transport Facilities

There are no water transport facilities within the Woodburn Urban Growth Boundary.

4.3 Traffic Operations

4.3.1 Intersection Traffic Control and Lane Channelization

Figure 7 shows the existing intersection traffic control and lane channelization for the major intersections within the City of Woodburn. As shown in this map, of the eleven major intersections, eight are controlled with a traffic signal. Only the intersections of Highway 214/Northbound I-5 Ramps, Parr Road/Settlemier Avenue and Front Street/Settlemier Avenue are controlled with stop signs.

4.3.2 24-Hour Traffic Counts

Figure 8 shows the recent 24-hour traffic counts in Woodburn[3]. This traffic is called average daily traffic (ADT). As shown in this map, ADT in the City of Woodburn is highest on Interstate 5, Highway 214 east of I-5, and Highway 214/99E south of Highway 211. Traffic volumes are also relatively high on Highway 214 from I-5 west to Willow Avenue, Settlemier Avenue from Highway 214 south to West Hayes Street, and on Young Street from Highway 214/99E to Front Street.

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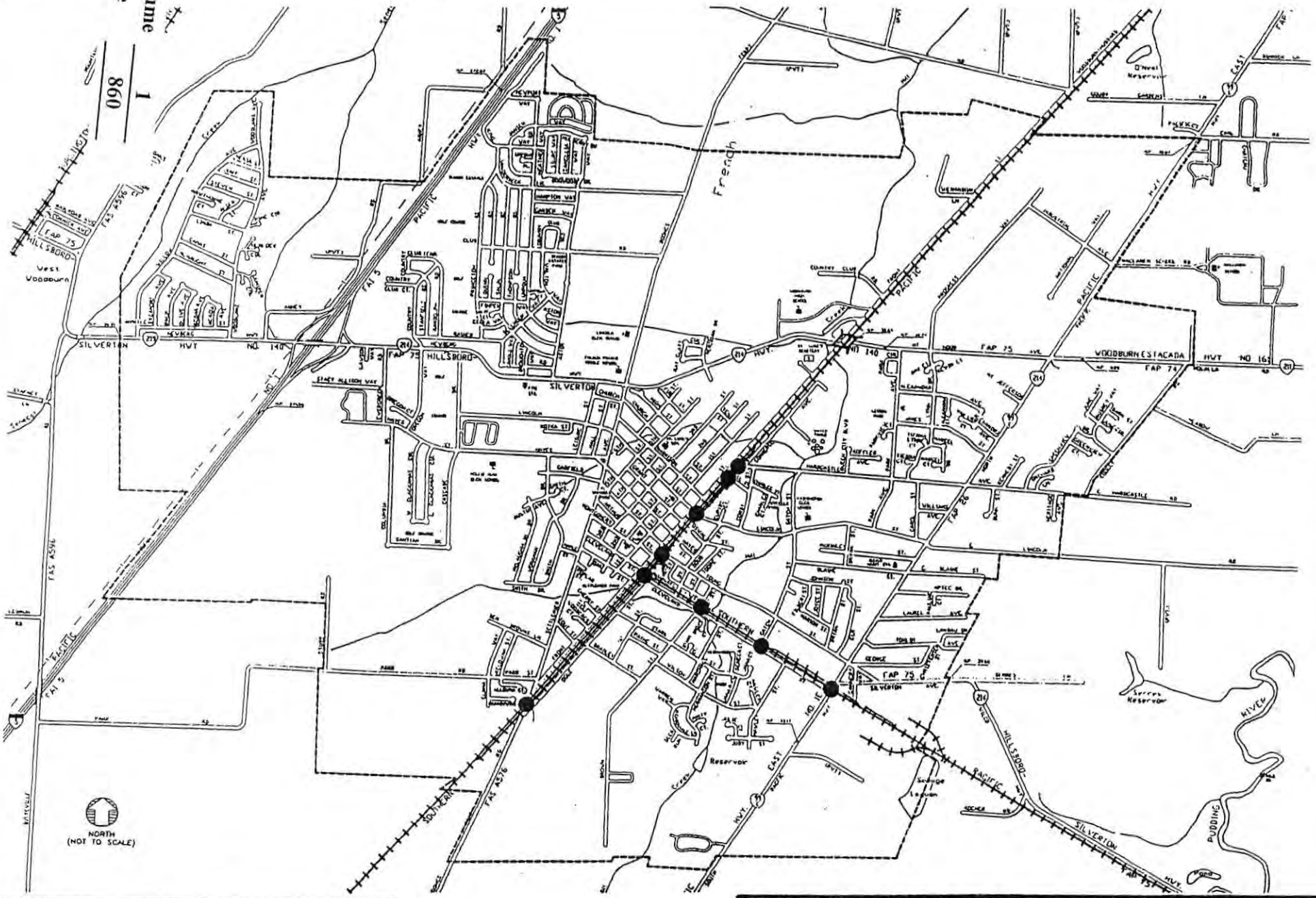
2 Source: Southern Pacific Railroad

3 Source: City of Woodburn, 1991



LEGEND

- EXISTING RAIL LINES
- RAILROAD AT-GRADE CROSSING
- RAILROAD GRADE SEPARATION

EXISTING RAIL SERVICE ROADWAY GRADE CROSSINGS

CITY OF WOODBURN
TRANSPORTATION SYSTEM PLAN

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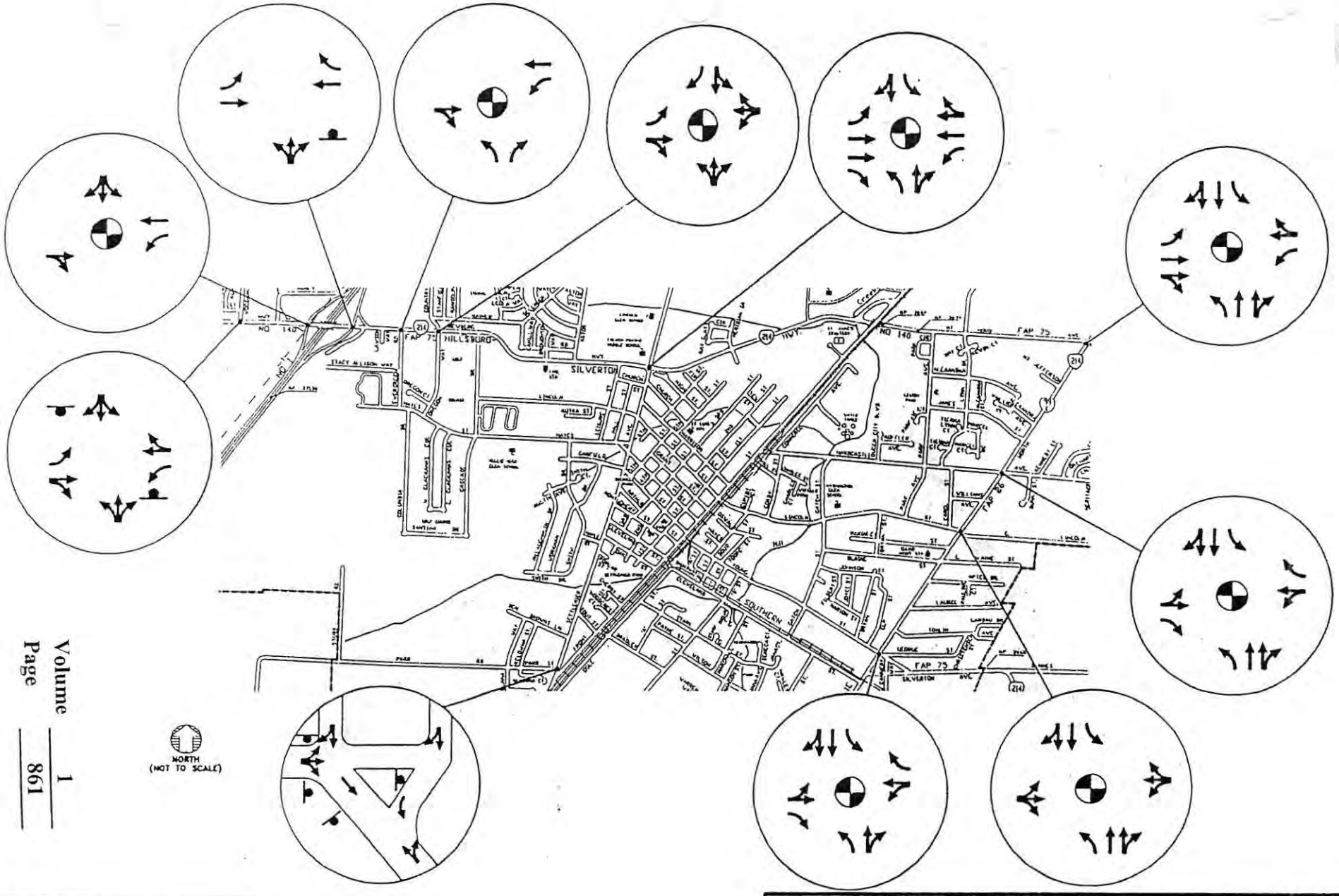
FIGURE 6

1373FRA



LEGEND

TRAFFIC SIGNAL

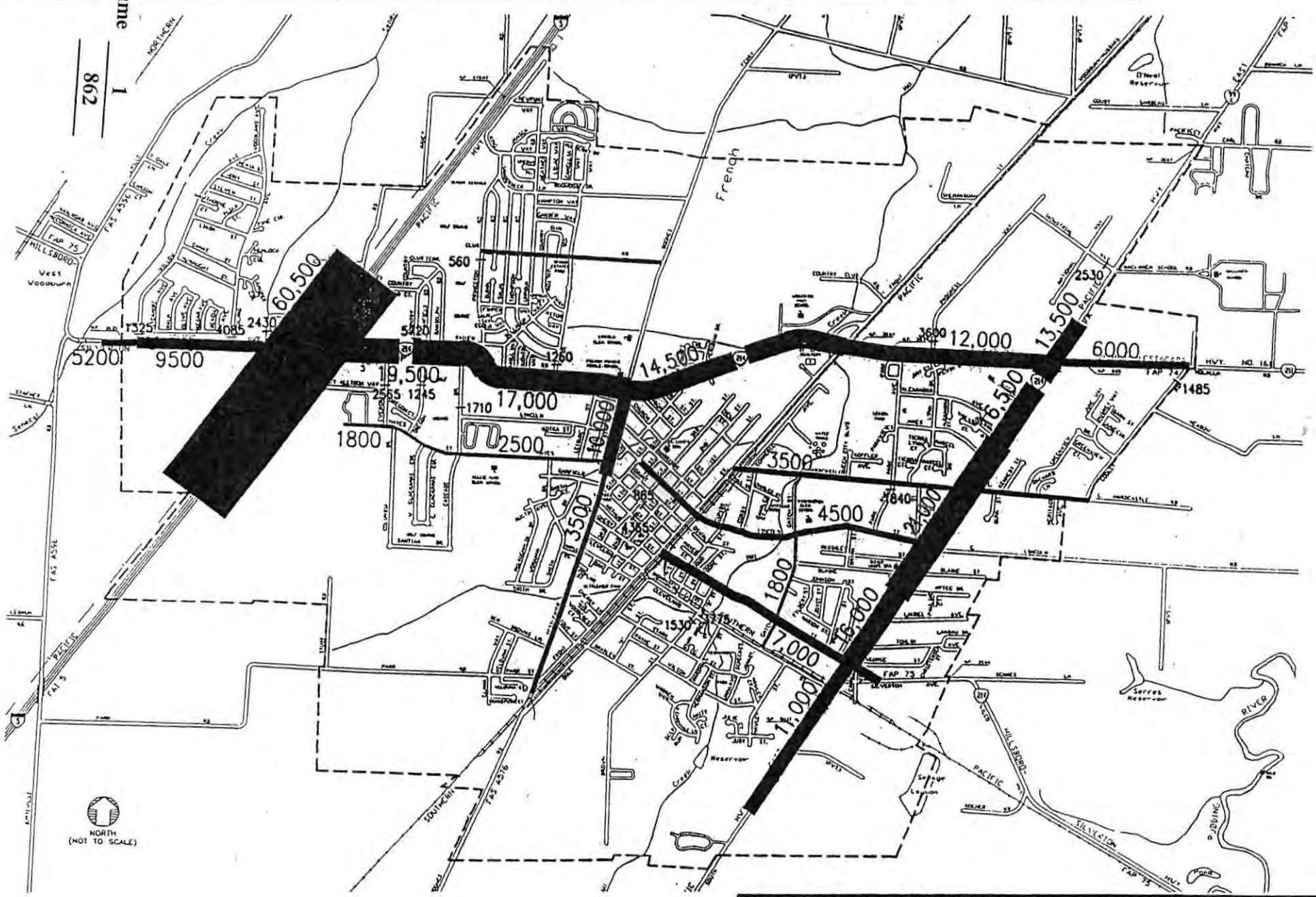
STOP SIGN

EXISTING INTERSECTION
TRAFFIC CONTROL/CHANNELIZATION

CITY OF WOODBURN
TRANSPORTATION SYSTEM PLAN
JUNE 1996

FIGURE
7

1373FCTL



RECENT WEEKDAY
24-HOUR TRAFFIC COUNTS (1991)

CITY OF WOODBURN
TRANSPORTATION SYSTEM PLAN
JUNE 1996

FIGURE 8

1373FADT

4.3.3 Intersection PM Peak Hour Traffic Volumes

Traffic counts were obtained from ODOT and from a private traffic survey firm during a typical weekday PM peak hour. The traffic count by ODOT was performed in August of 1994, and the traffic count by the private traffic survey firm was performed in January of 1995. This data is summarized in Figure 9. This data will be used in coordination with the intersection traffic control and lane channelization information to calculate traffic level of service.

4.3.4 Intersection Levels of Service (Weekday PM Peak Hour)

Traffic level of service (LOS) is measured on a scale of LOS A to LOS F. At intersections, LOS A means that drivers experience no delay or relatively low amounts of delay while traveling through the intersection; while LOS F means that drivers experience a great deal of delay while traveling through the intersection. Appendix A presents a detailed review of intersection traffic level of service concepts.

The LOS analysis performed for this study for the typical weekday PM peak hour revealed that traffic operations at the major intersections in Woodburn are generally acceptable (See Figure 10). All of the major study intersections operate at LOS C or better except for the Highway 214/I-5 interchange. At this interchange, the intersection of the northbound I-5 ramps with Highway 214 operates at LOS E for motorists turning left from the off-ramp onto westbound Highway 214, and operates at LOS D for motorists turning right from the off-ramp onto eastbound Highway 214. Also, during the PM peak hours, vehicle queues can form at the intersections of southbound I-5 ramps/Highway 214, northbound I-5 ramps/Highway 214, Highway 214/Evergreen Road, Highway 214/Country Club Road, and Highway 214/Oregon Way.

4.3.5 High Accident Locations

Figure 11 summarizes the incidence of accidents at the major Woodburn intersections over the following three years: 1992, 1993, and 1994. As shown on this map, and as expected, the most accidents occur at the highest volume intersections:

- Highway 214/99E/211;
- Highway 99E/Young Street; and
- Interstate 5/Highway 214.

Other high accident locations include the intersections of Highway 214/Settlemier Avenue, Front Street/Highway 214, and Young Street/Front Street. Again, these are also locations with relatively high traffic volumes.

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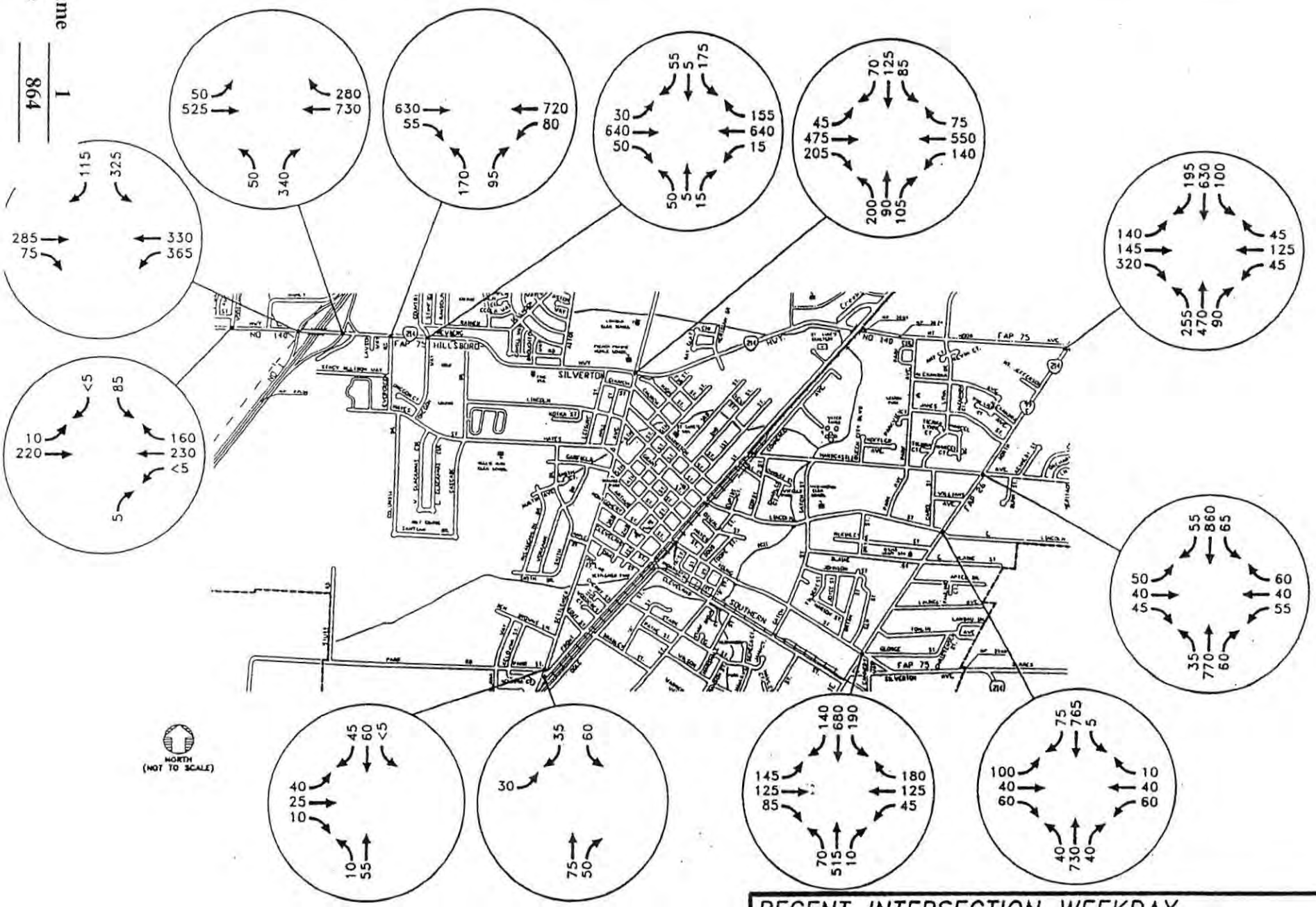

RECENT INTERSECTION WEEKDAY
PM PEAK HOUR VOLUMES (1994-95)

CITY OF WOODBURN
TRANSPORTATION SYSTEM PLAN

JUNE 1996

FIGURE
9

1373FVOL

RECENT INTERSECTION LEVELS OF SERVICE–WEEKDAY PM PEAK HOUR (1994–95)

CITY OF WOODBURN
TRANSPORTATION SYSTEM PLAN
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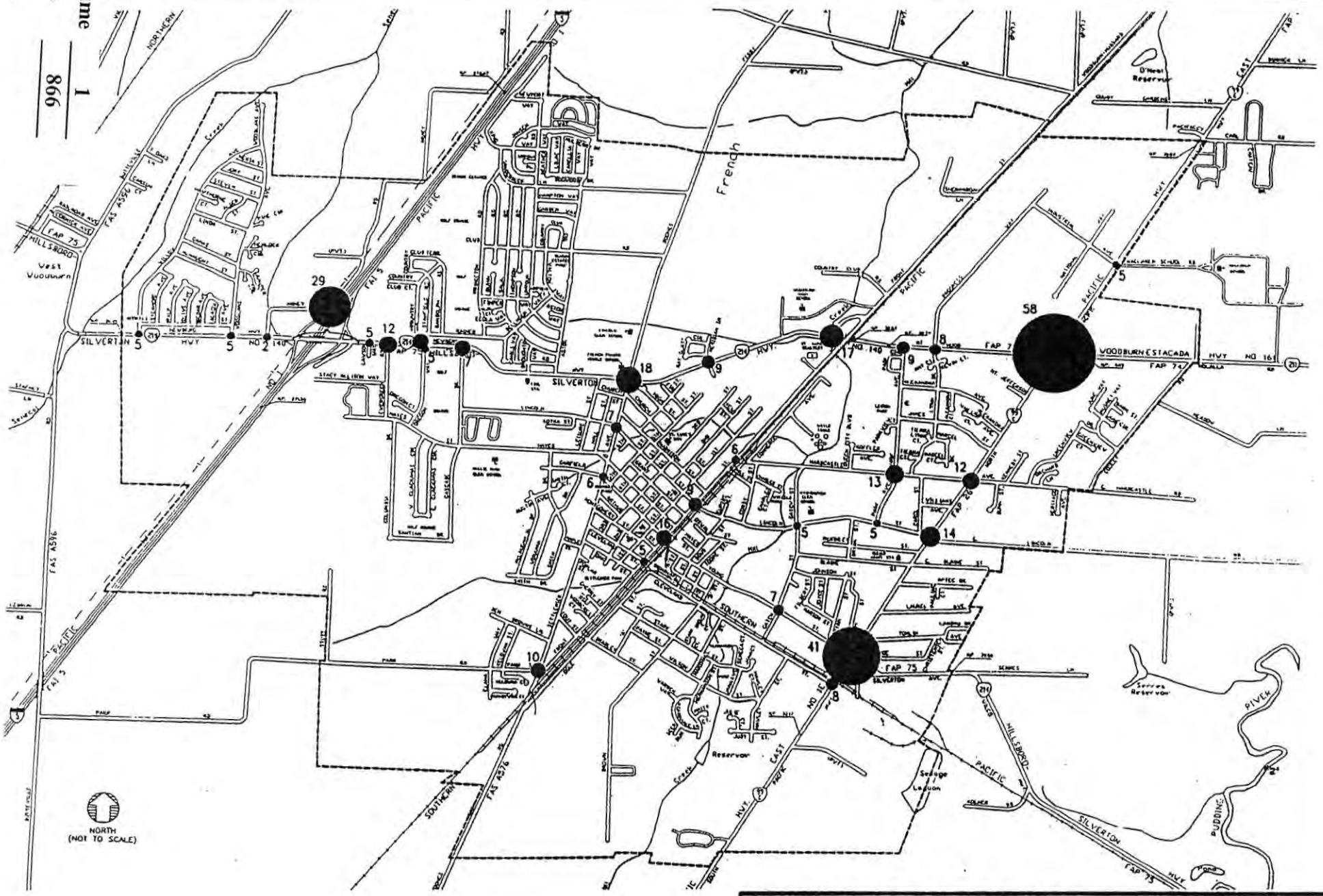
FIGURE 10

1373FLOS



HIGH ACCIDENT LOCATIONS
(1992–1994)

CITY OF WOODBURN
TRANSPORTATION SYSTEM PLAN

JUNE 1996

FIGURE 11

1373FACC

Section 5

Roadway System Alternatives Analysis

5.0 Roadway System Alternatives Analysis

5.1 Methodology

In this phase of the transportation system plan development, future travel demand (year 2015) was tested on four different roadway systems. The analysis period was the weekday PM peak hour. To forecast future travel patterns, a year 2015 weekday PM peak hour travel demand model was developed and future traffic volumes were assigned to each of the roadway system alternatives (described below). Following the assignment, an operations analysis was performed to evaluate the effectiveness of each of the alternative roadway improvement systems. This analysis is summarized below.

5.2 Travel Demand Model Development

As part of the roadway systems analysis, a year 2015 EMME/2 travel demand model was developed (see Appendix B for a description of the travel demand model). This model reflects future population and employment projections as provided by the City of Woodburn Community Development Department and was used to forecast future weekday PM peak hour traffic volumes. By the year 2015, it is anticipated that the population of Woodburn will grow from the current 15,000 to about 30,000. It is also projected that employment will grow from the current 5,000 employees to 14,000 employees. The population and employment projections include some development areas outside the existing Woodburn Urban Growth Boundary (UGB) with the expectation that the UGB will need to be expanded before year 2015 to handle anticipated growth.

5.3 Roadway System Evaluation Criteria

Several, evaluation criteria for the roadway system alternatives analysis were established:

Traffic Operations

- **Average Daily Traffic (ADT),** which is the forecast 24 hour traffic volume at a specific location on a given road. The model forecasts ADT for all roads on the network. With ADT, it is possible to compare 24 hour traffic volumes on the same road under different transportation alternatives.
- **Daily Vehicle Miles Traveled (VMT),** which is the forecast total number of vehicle miles traveled on the road system over a 24 hour period. For example, if the model forecasts that over a 24 hour period, 1,000 vehicles will travel on the road system and there are 20 miles of roads, the VMT is equal to 20,000 (1000 multiplied by 20). The lower the VMT the better, as this implies people do not have to travel as far for different trip purposes and thus less fuel is consumed and less pollution is created.
- **Daily Vehicle Hours of Travel (VHT),** which is the forecast number of hours that people spend traveling from their trip origin to their destination and back. For example, if, over a 24 hour period, there are 1,000 vehicles traveling on the road system and the model shows that it would take on average 30 hours for one vehicle to travel from its origin to its destination and back, the forecast VHT is equal to 30,000 (1,000 multiplied

by 30). VHT is a measure of system wide road congestion. The higher the VHT, the more congestion there is, as it takes vehicles longer to travel from their origins to their destinations.

- **Lane Miles Over Capacity,** which is the forecast number of lane miles operating over capacity. For example, if there is one four lane segment of road operating over capacity and that section of road is two miles long, the forecast lane miles over capacity is eight (two lane miles over capacity multiplied by four lanes). The fewer lane miles over capacity the less road congestion exists on the network.

In addition, on a microscopic level, the intersection traffic operations were evaluated based on a forecast weekday PM peak traffic level of service analysis.

Land Use Impact

The general land use impact of each alternative reflects the impact of roadway improvements to guide the location of and serve added development in the Woodburn area in the future. Critical to this analysis was the ability to develop roadway improvements within the existing Woodburn Urban Growth Boundary. This impact assessment also addressed right-of-way acquisition needs.

Environmental Impact

The general environmental impact of each roadway system alternative involves an assessment of how roadway improvements might impact environmentally sensitive areas, such as wetland and habitat areas.

Cost

The general cost of constructing the major roadway system improvements in each alternative was identified. The cost estimates are very conceptual and are based on assumed costs per mile for different types of facility improvements. Cost estimates are in 1995 dollars.

5.4 Description of Roadway System Alternatives

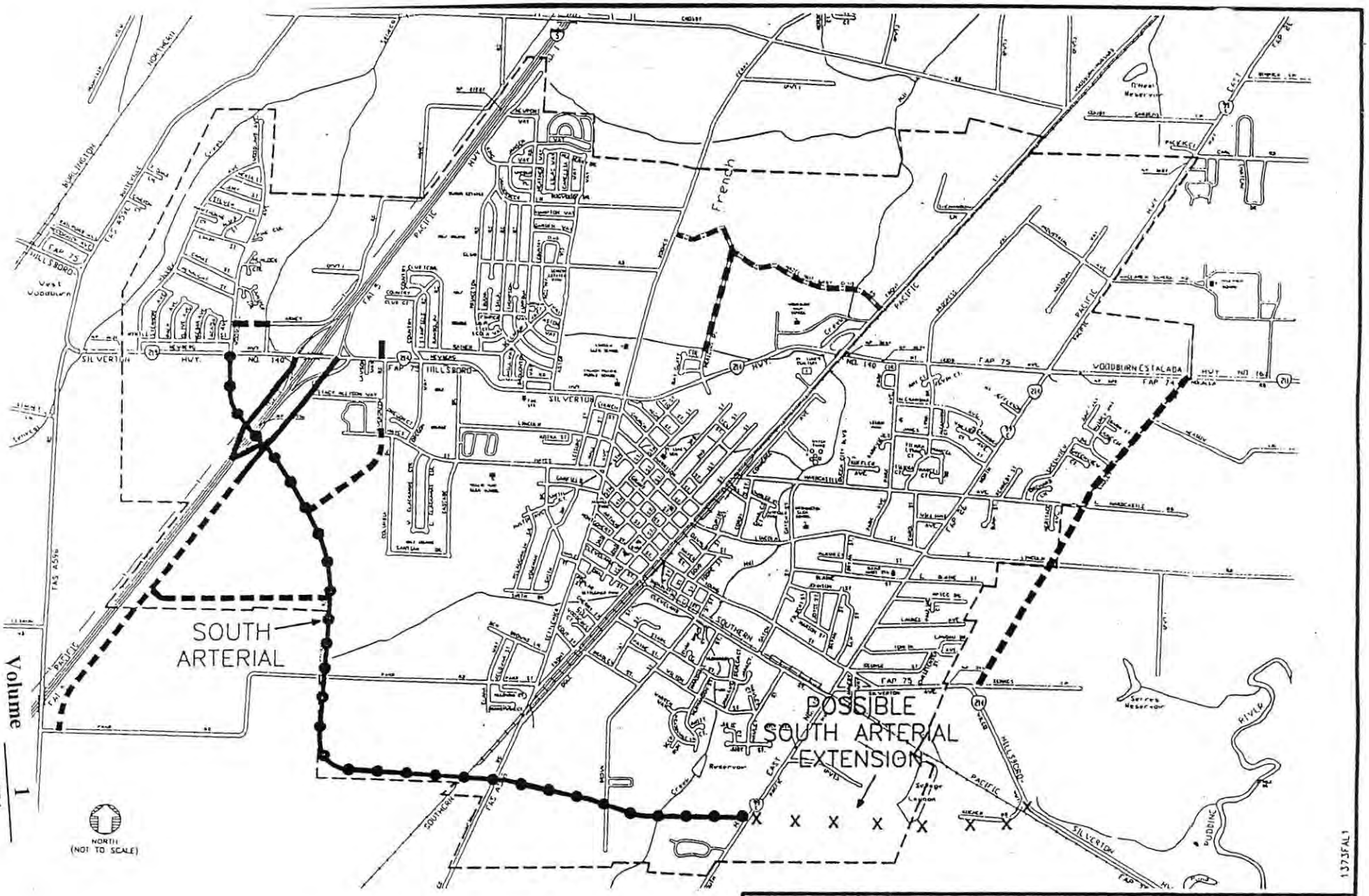
Traffic operations under four roadway alternatives were evaluated. These alternatives are:

No-Build Alternative:

In this alternative future traffic volumes are assigned to the existing roadway system. No substantial roadway improvements would be made.

Alternative #1: I-5 Split Diamond Interchange with South Arterial

The major features of this alternative, shown in Figure 12, include converting the existing I-5/Highway 214 interchange to a split diamond interchange, extending Woodland Road south to connect with the south end of the new split diamond interchange, constructing a South Arterial to connect with the new interchange, and developing a frontage road on the east side of Interstate 5.



LEGEND (NEW ROADWAYS)

- PRINCIPAL ARTERIAL/FREEWAY
- MINOR ARTERIAL
- COLLECTOR

ROAD ALT. #1:
I-5 SPLIT DIAMOND INTERCHANGE
WITH SOUTH ARTERIAL

CITY OF WOODBURN
TRANSPORTATION SYSTEM PLAN
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FIGURE 12

Also shown in Figure 12 is an alternative to extend the South Arterial east of Highway 99E to connect with Highway 214.

Alternative #2: Second I-5 Interchange with South Arterial

In this alternative, shown in Figure 13, a new Interstate 5 interchange would be constructed south of the existing Urban Growth Boundary just north of the existing Butteville Road over-crossing, with a South Arterial developed as an extension of Butteville Road to the east to connect with Highway 99E. Also in this alternative Woodland Road would be extended south to Butteville Road to connect with the interchange, with a frontage road constructed on the east side of I-5 between Highway 214 and the South Arterial.

Associated with this alternative is a possible extension of the South Arterial east of Highway 99E to connect with Highway 214.

Alternative #3: Improve Existing I-5 Interchange and Widen Highway 214/ South Arterial with I-5 Overpass

This alternative is shown in Figure 14 and does not include a new I-5 interchange. Instead in this alternative, the existing interchange would be improved by widening Highway 214 over Interstate 5, from Woodland Drive on the west to Highway 99E. In addition, a South Arterial would be constructed within the Urban Growth Boundary extending from Woodland Drive on the west to Highway 99E on the east, with an overpass of I-5.

Similar to Alternatives #1 and #2, shown in Figures 12 and 13, the South Arterial could be extended east from Highway 99E to connect with Highway 214.

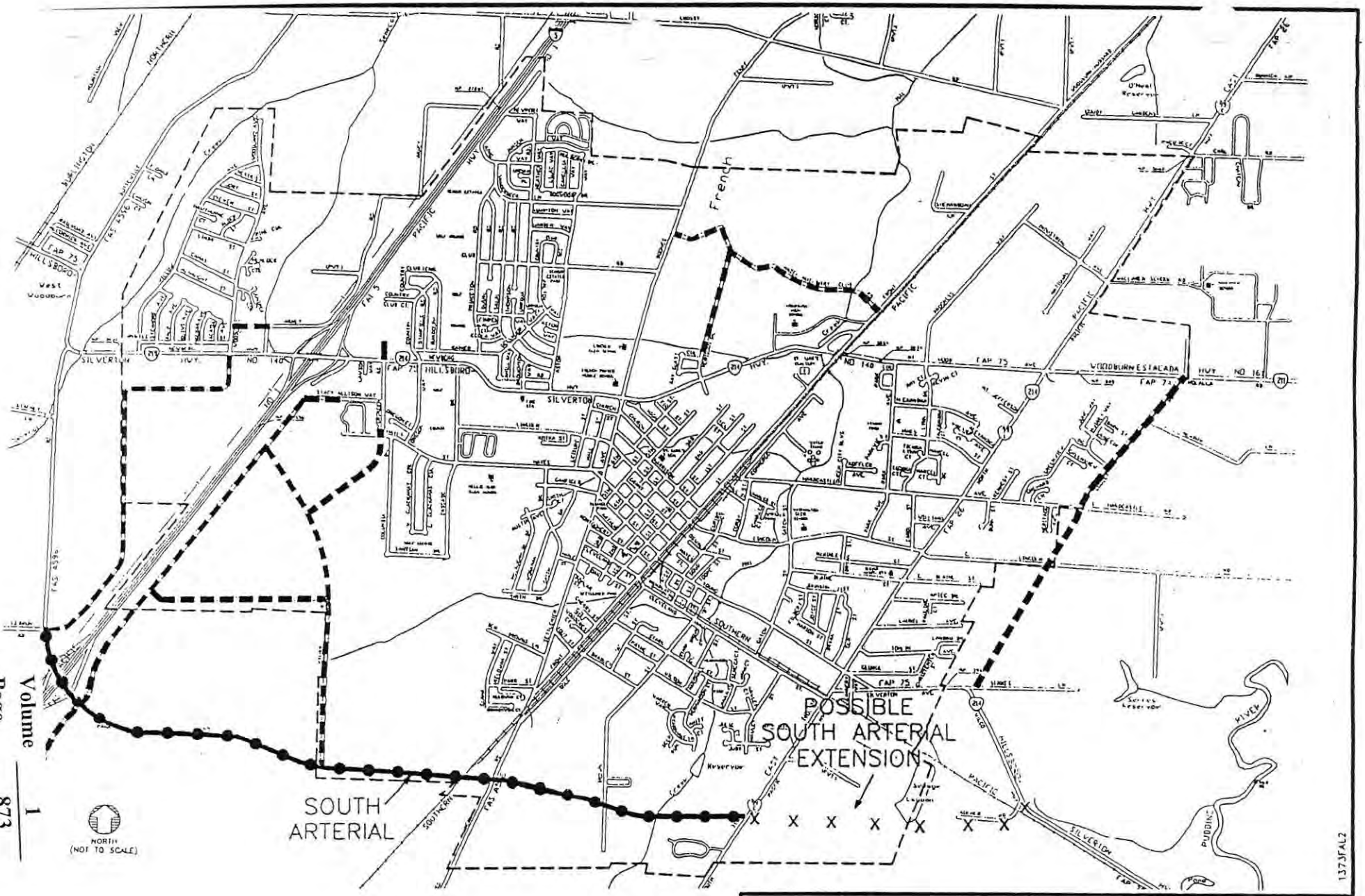
This alternative was analyzed for two improvement options to the existing I-5 Highway 214 interchange:

1. Improvements to the existing diamond configuration and,
2. Conversion to partial cloverleaf configuration.

5.5 Evaluation of Alternatives

5.5.1 Transportation System Performance

Following the development of three alternatives, and the incorporation of these alternatives (separately) into the travel demand model, the alternatives were evaluated using the previously described performance measures. Table 2 compares forecast average daily traffic (ADT) volumes at specific locations for the different roadway system alternatives (Appendix C presents year 2015 traffic flow maps for each alternative). As shown in the table, if no improvements were made to the road system, 2015 traffic volumes on Highway 214 east of the northbound Interstate 5 ramps would increase to 39,000 vehicles per day. This volume would decrease under each of the alternative road systems. Similarly at Highway 214 east of Settlemier Avenue, West Hayes Street west of Settlemier Avenue, and Young Street west of Highway 99E, the 2015 traffic volumes under any of the Build Alternatives would be less than the No-Build Alternative. This is not true on Highway 99E south of Lincoln Street. At this



LEGEND (NEW ROADWAYS)

PRINCIPAL ARTERIAL/FREEWAY

MINOR ARTERIAL

COLLECTOR

ROAD ALT. #2
SECOND I-5 INTERCHANGE/SOUTH ARTERIAL

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FIGURE 13

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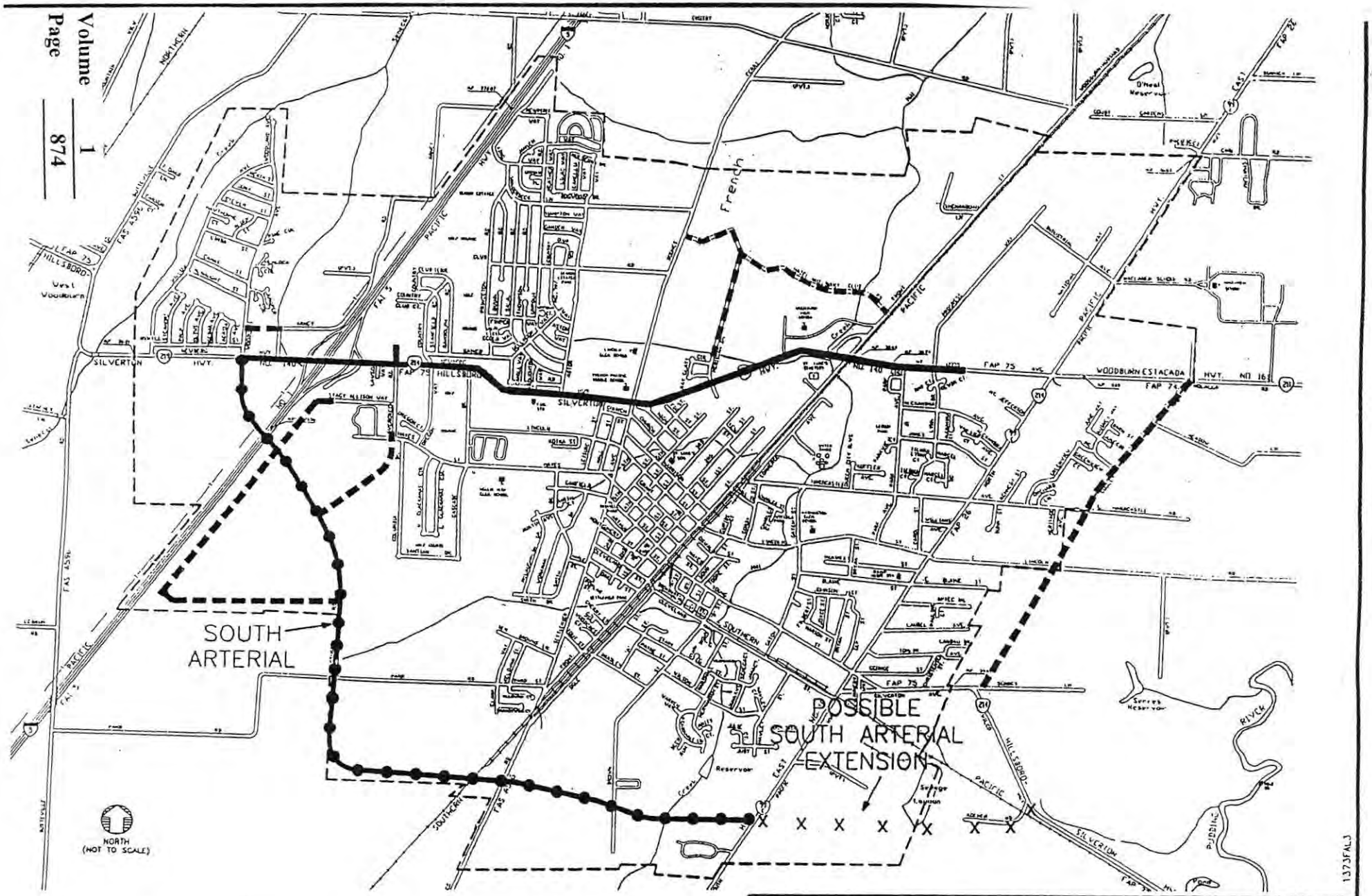

LEGEND (NEW ROADWAYS)
- PRINCIPAL ARTERIAL/FREEWAY
- MINOR ARTERIAL
- COLLECTOR

ROAD ALT. #3: IMPROVE EXISTING I-5 INTERCHANGE & HWY 219-214/SOUTH ARTERIAL WITH I-5 OVERPASS

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FIGURE 14

Table 2

Year 2015 Average Weekday Traffic for Roadway System Alternatives

Alternative	Hwy. 214 East of NB I-5 Ramp	Hwy. 214 East of Settlemier	West Hayes West of Settlemier	Hwy. 99E South of Lincoln	Young Street West of Hwy. 99E
2015 No-Build	39,000	26,000	12,000	23,000	10,000
2015 Alternative # 1*	30,000	20,000	5,000	18,000	6,000
2015 Alternative #2*	27,000	17,000	5,000	22,000	7,000
2015 Alternative #3*	34,000	20,0000	6,000	26,000	9,000

* South Bypass or Parr Road extension east to Highway 99E.

location, under Alternative #1 forecast daily traffic volumes decrease relative to the No-Build Alternative; under Alternative #2 the forecast traffic volumes remain essentially the same relative to the No-Build Alternative; and under Alternative #3 the forecast traffic volumes increase relative to the No-Build Alternative.

In Table 3, lane miles over capacity (weekday p.m. peak hour), weekday vehicle hours of travel (VHT), and weekday vehicle miles of travel (VMT) for each of the three study alternatives are compared. These measures are interrelated and are thus best evaluated together rather than individually. As shown in this table, daily VMT, VHT, and the number of lane miles over capacity would increase dramatically between now and year 2015. VMT would increase approximately 80 percent, and VHT would increase approximately 185 percent. This dramatic increase in VHT relates directly to the increase of lane miles over capacity. Under existing conditions, there are no lane miles operating over capacity; however in the future, if no road improvements were made, there would be 1.1 lanes miles of road over capacity. The over-capacity roads would be:

Table 3

Summary of Traffic Operations Performance Measures

Scenario	Lane Miles Over Capacity (Weekday P.M. Peak Hour)	Weekday Vehicle Hrs. of Travel (VHT)	Weekday Vehicle Miles of Travel (VMT)
Existing	0	8,955	406,255
No-Build	1.1	25,625	730,260
Alternative #1A*	0.26	24,035	717,120
Alternative #1B**	0.26	23,995	718,055
Alternative #2A*	0.52	23,730	715,360
Alternative #2B**	0.52	23,745	714,815
Alternative #3A*	0.50	24,665	707,840
Alternative #3B**	0.70	24,680	707,300

* South Arterial east to Highway 99E.

** South Arterial east to Highway 214.

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- Highway 214 between the northbound Interstate 5 ramps, and Evergreen Road,
- Evergreen Road between Highway 214 and its terminus, and
- Settlemier Avenue just south of West Hayes Street.

Figures 15 through 18 show which roadway segments are under, near, or over capacity for the no-build condition and roadway system alternatives #1, #2, and #3.

Compared to the No-Build Alternative, Alternative #1 (I-5 Split Diamond Interchange with South Arterial) would have the lowest number of lane miles over capacity of all three alternatives. In this alternative only 0.26 lane miles of Woodburn roads would be operating over capacity. This would be Woodland Road just north of Highway 219, which is likely a result of restricting the Arney Road access to Highway 219 to right-in, right-out only movements. Also, in this alternative Highway 214 would be operating near capacity for almost its entire length between Interstate 5 and Settlemier Avenue. Alternative #2 has the greatest effect of reducing congestion on Highway 214; however, it does have more lane miles over capacity than either Alternative #1 or #3. Congestion would occur on Parr Road in the vicinity of Interstate 5.

On a daily basis, VHT is the lowest in Alternative #2. VMT is most significantly reduced, relative to the No-Build alternative, by implementing Alternative #3 (South Arterial; Interstate 5 over crossing). However, both of these alternatives introduce more traffic congestion than Alternative #1 (I-5 Split Diamond Interchange).

5.5.2 Major Intersection Improvement Needs

An initial year 2015 level of service (LOS) analysis was performed at five major intersections assuming no improvements to the City of the Woodburn street system in the future. The critical intersections are southbound I-5/Highway 214, northbound I-5/Highway 214, Highway 214/Settlemier Avenue, Highway 214/99E and Highway 99E/Young Street. A new signal was assumed at the northbound I-5 ramp intersection. As shown in Table 4, under the No-Build

Table 4

Summary of Level of Service Analysis

Intersection	Weekday P.M. Peak Hour Level of Service				
	2015 No-Build	2015 No-Build Mitigated	Alt. #1* (I-5 Split Diamond Interchange)	Alt. #2* (2nd I-5 Interchange)	Alt. #3* (Impr. Existing Interchange/ Hwy. 214)
SB I-5/Hwy 214	F	E	D	C	D
NB I-5/Hwy 214	F	C	C	A	B
Hwy 214/Settlemier Avenue	F	E	D	D	D
Hwy 214/Hwy 99E	F	E	E	E	E
Hwy 99E/Young Street	F	D	C	D	C

* With South Arterial extension to Highway 99E

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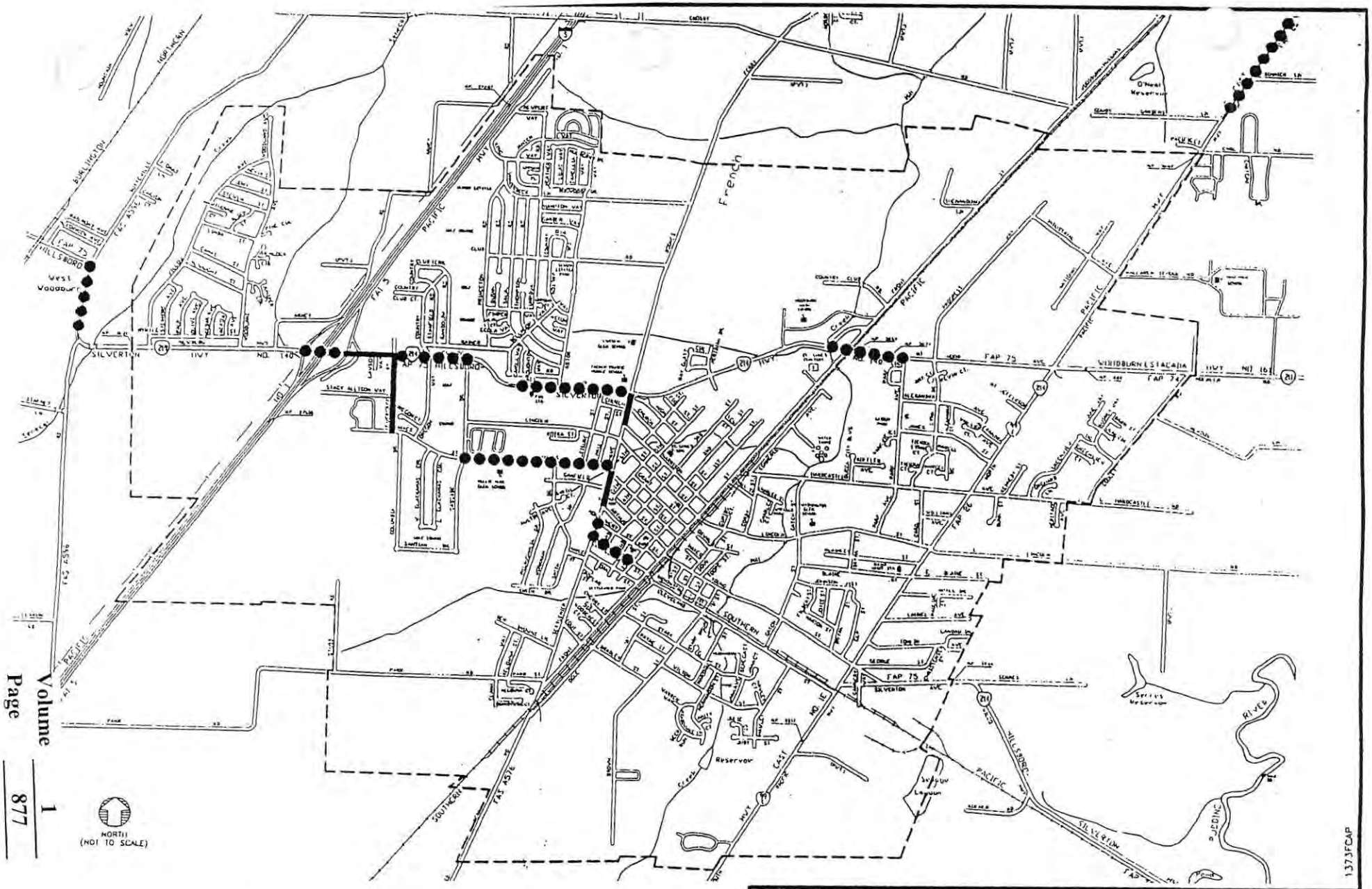

2015 WEEKDAY PM PEAK HOUR
LEVEL OF CONGESTION–
NO–BUILD ALTERNATIVE

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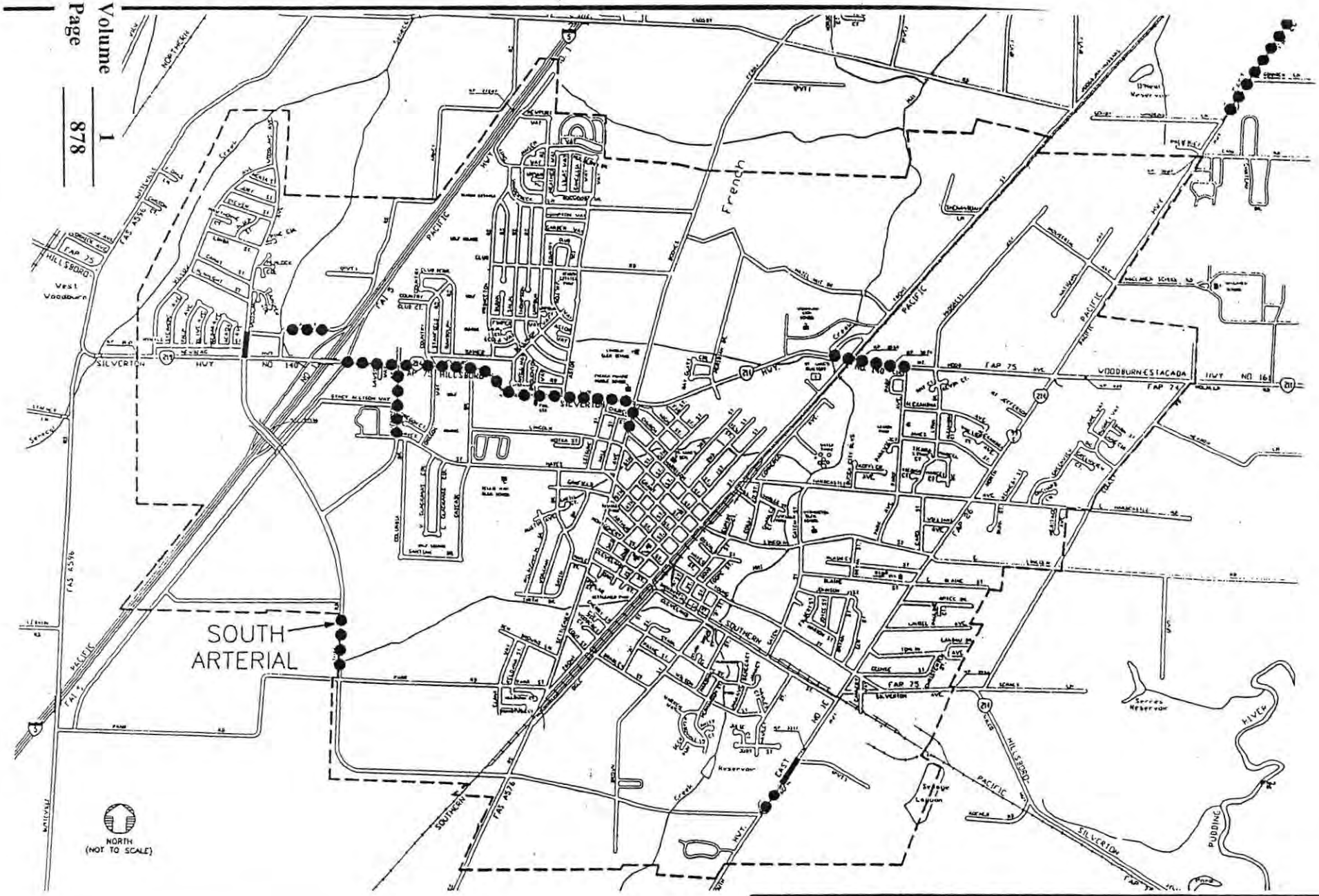
FIGURE
15



LEGEND

UNDER CAPACITY

APPROACHING CAPACITY

OR OVER CAPACITY

2015 WEEKDAY PM PEAK HOUR
LEVEL OF CONGESTION– ROAD ALT.#1–
I–5 SPLIT DIAMOND INTERCHANGE

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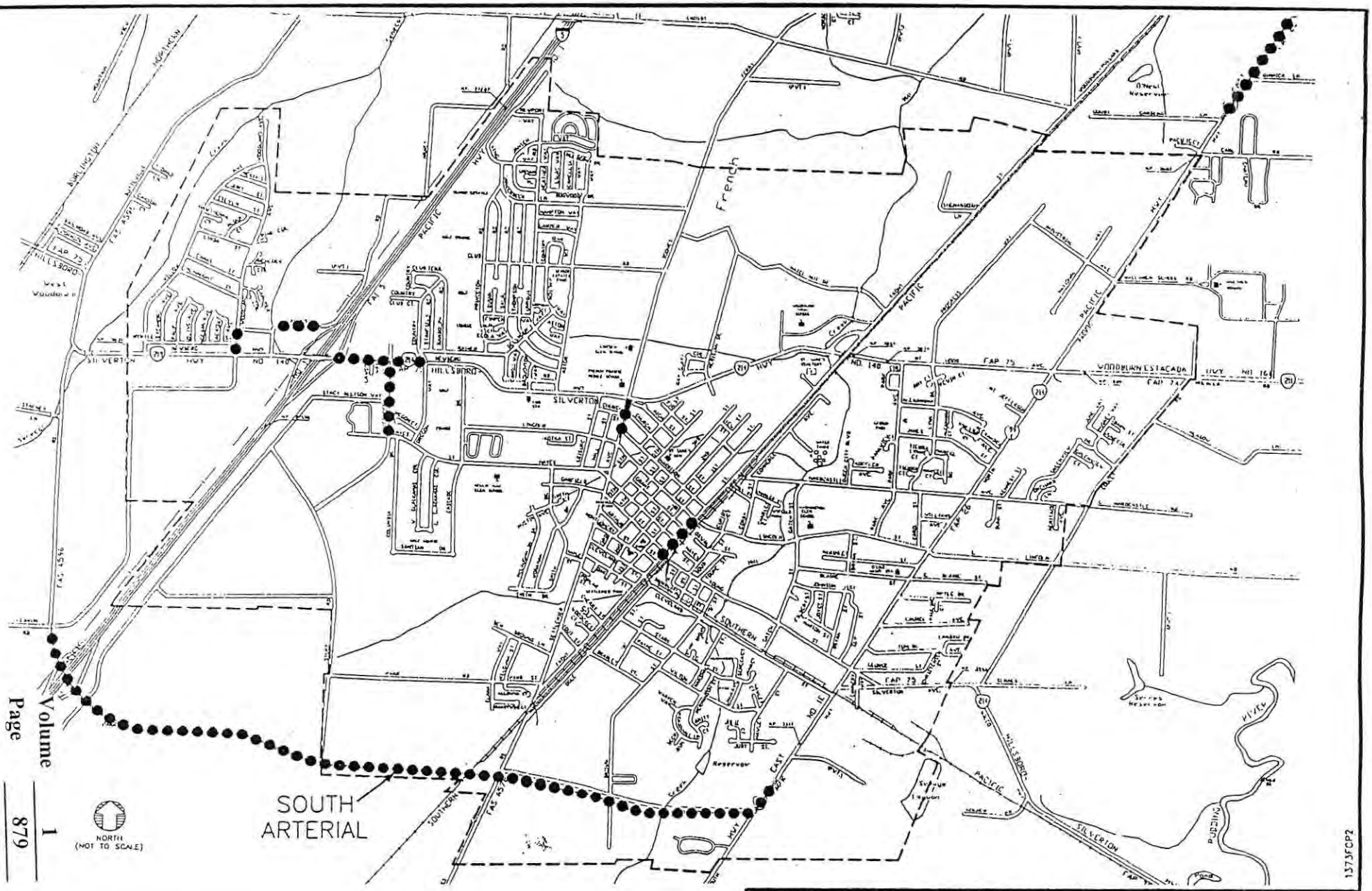
FIGURE 16



LEGEND

- UNDER CAPACITY
- APPROACHING CAPACITY
- AT OR OVER CAPACITY

2015 WEEKDAY PM PEAK HOUR
LEVEL OF CONGESTION– ROAD ALT.#2–
SECOND I–5 INTERCHANGE

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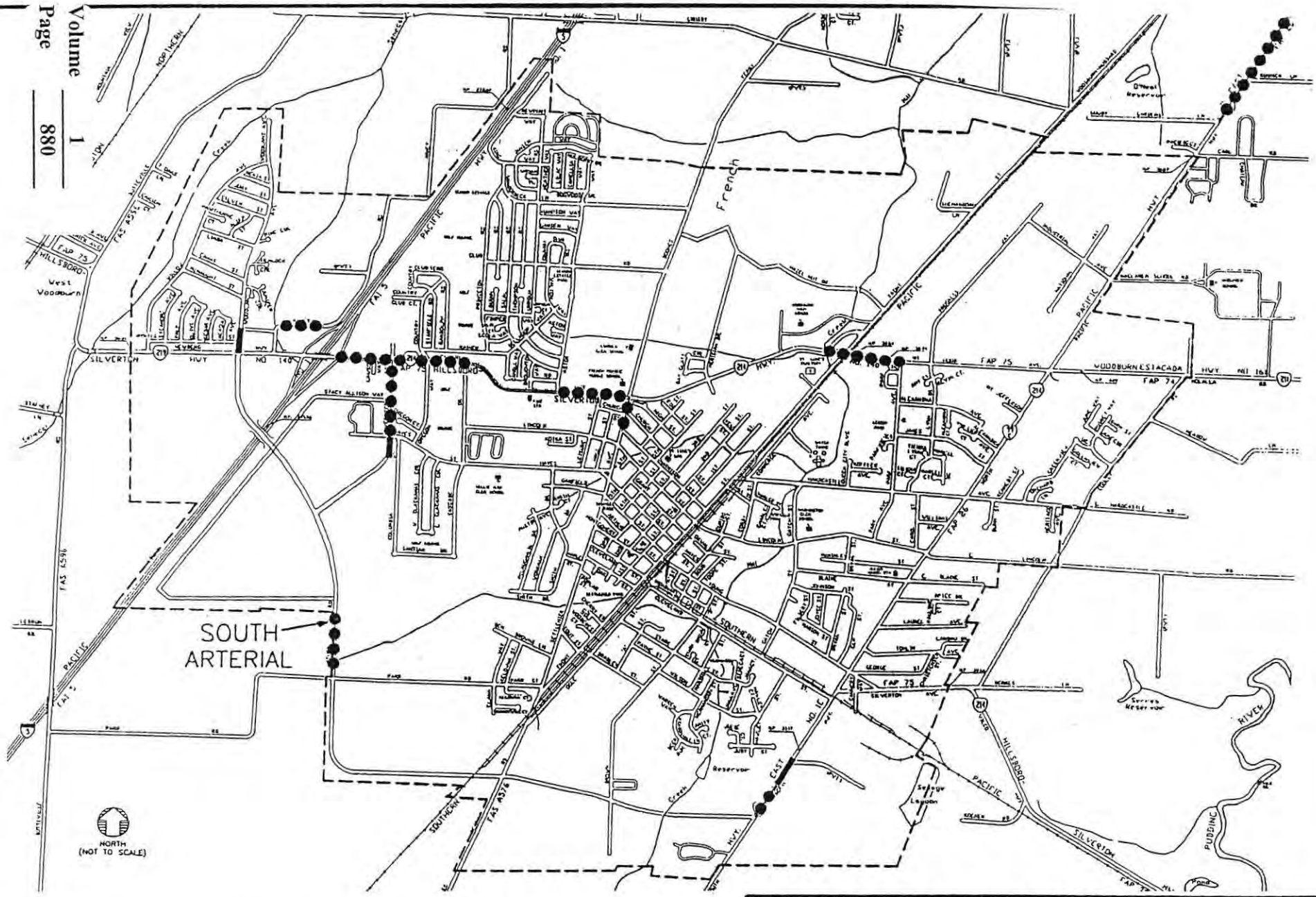
FIGURE 17



LEGEND

- UNDER CAPACITY
- APPROACHING CAPACITY
- OVER CAPACITY

2015 WEEKDAY PM PEAK HOUR
LEVEL OF CONGESTION– ROAD ALT.#3–
IMPROVE EXISTING I–5 INTERCHANGE

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FIGURE 18

alternative, future PM peak hour traffic level of service degrades to LOS F at all five intersections. LOS E is the typical minimum acceptable level of service at signalised intersections in an urban area such as Woodburn ("A" representing the best and "F" the worst level of service). Thus, all five identified intersections have an unacceptable level of service.

Given these unacceptable levels of service, a mitigation analysis was performed on the 2015 PM peak hour No-Build scenario in order to determine what intersection improvements at the five critical intersections would be necessary to restore acceptable operating conditions. The necessary improvements are as follows:

- **Southbound I-5/Highway 214:** Add a second left turn and right turn lane on the southbound I-5 ramp; re-stripe the eastbound intersection approach to include a through lane and a right turn lane; add a second left turn lane to the westbound approach.
- **Northbound I-5/Highway 214:** Signalize; add a second right turn lane on the northbound I-5 ramp; add a second left turn lane to the eastbound intersection approach; add a second through lane to the westbound approach.
- **Highway 214/Settlemier Avenue:** Optimize the signal timing to minimize delay; add a second left turn lane at the northbound approach; and restripe the southbound approach to the intersection to include one left turn, one right turn, and one through lane.
- **Highway 214/Highway 99E:** Add a second left turn lane to the eastbound intersection approach; restripe the westbound intersection approach to include one left turn lane, one through lane and one right turn lane; add a second left turn lane to the northbound approach to the intersection.
- **Highway 99E/Young Street:** Construct an exclusive right turn lane for vehicles to turn from westbound Young Street to northbound Highway 99E.

With these improvements all of the critical intersections will operate at LOS E or better in the future PM peak hour.

Next a PM peak hour traffic level of service analysis was performed on each of the identified road system alternatives. The intersection improvements identified above were included in this analysis. Under Alternative #1, the above mitigations will provide for acceptable operating conditions at all of the study intersections except the intersection of southbound Interstate 5/Highway 214. At this intersection it is necessary to add a second right turn lane to the southbound approach to the intersection.

As shown in Table 4, in Alternative #2 and #3 the above mitigation measures developed for the no-build alternative level of service analysis are sufficient to provide acceptable intersection operations in the future.

5.5.3 Operations Analysis of I-5 Interchange Alternatives

An additional analysis of year 2015 traffic conditions related to the various I-5 access alternatives under consideration in the Woodburn Transportation System Plan was undertaken. Appendix D identifies the traffic volumes and lane configurations assumed in the analysis to improve the I-5/Woodburn interchange operation.

Freeway Operation

The purpose of this evaluation was to develop a comparison of the expected freeway operation/level of service between the three interchange alternatives considered in the roadway system alternatives analysis. The three alternatives include:

- Existing diamond interchange;
- Split diamond interchange; and
- Two interchanges, with new interchange in the vicinity of the existing Butteville Road overcrossing.

A partial cloverleaf configuration at the existing Highway 214 interchange was also analyzed primarily to determine the benefits which might be achieved in the operation of Highway 214 through the signalized intersections. The results of this analysis along with the three alternatives are shown in Table 5.

Table 5

Southbound I-5 Freeway Operation - 2015 Weekday PM Peak Hour

Condition/Criteria	Interchange Configuration					
	Diamond	Split Diamond	Two Interchanges		Partial Cloverleaf	
			Butteville Road	Highway 214	First Ramp	Second Ramp
Diverge						
Ramp Volume (vph)	960	960	400	590	960	-
Freeway Volume (Lane 1,2)[1](vph)[2]	3,578	3,578	3,379	3,490	3,578	-
Level of Service	D	D	C	D	D	-
Density[3]	29	29	27	28	29	-
Speed (mph)	56	56	56	56	56	-
Merge						
Ramp Volume (vph)	1,220	1,220	790	460	840	380
Freeway Volume (Lane 1,2)[1](vph)[2]	3,968	3,968	3,769	3,352	3,534	3,579
Level of Service	D	D	D	C	D	D
Density[3]	32	32	30	27	29	29
Speed (mph)	54	54	55	56	55	56

1 lanes 1 and 2 are two right-most lanes on freeway

2 vph - vehicles per hour

3 density - vehicles per mile per lane

2015 PM peak hour traffic volumes were used for this analysis, as this period was considered the highest overall traffic period and most appropriate for design purposes. The PM peak hour volumes were derived from the daily EMME/2 model traffic projections by applying existing peak hour and intersection turning movement percentages. Southbound I-5 operations were analyzed as this represents the peak direction of traffic on I-5 during the PM peak hour.

The 1994 Highway Capacity Manual (HCM) procedures for freeway merge/diverge operation was applied for this analysis. This particular operational analysis procedure on I-5 represents a merge/diverge condition on the freeway rather than a weave since no added lanes between interchanges is proposed by any of the alternate interchange configurations.

As shown in Table 5, although there is a measurable change in calculated freeway/ramp junction level of service, there is no measurable change in the lane density or the operating speed of the freeway. This alone should not be used as criteria to select one alternative design over another.

Interchange Operation

The 2015 PM peak traffic volume/turn movement was assigned to the interchange configurations under consideration. A level of service analysis was performed for each ramp terminal with the cross street using the SIGCAP procedure, with a determination of the lane requirements at each intersection location to provide a balanced roadway section with adjacent intersections.

The results of this analysis is shown in Table 6. This provides a comparison of the volume to capacity ratio and level of service calculated for each alternative.

Table 6

I-5 Interchange Ramp Terminal Operation - 2015 Weekday PM Peak Hour

Interchange Configuration	Intersection	LOS	V/C
Diamond	Northbound Off/On-Ramp	E-F	98%
	Southbound Off/On-Ramp	E	96%
Split Diamond	North Off-Ramp (Hwy 214)	C-D	75%
	South Off-Ramp (South Arterial)	D-E	87%
	North On-Ramp (Hwy 214)	D	82%
	South On-Ramp (South Arterial)	D-E	88%
Two Interchanges	**Parr Road/Butteville Road**		
	Northbound Off/On-Ramp	D	84%
	Southbound Off/On-Ramp	D	81%
	Highway 214		
	Northbound Off/On-Ramp	C-D	75%
	Southbound Off/On-Ramp	D	77%
Partial Cloverleaf	Northbound Off/On-Ramp	E	97%
	Southbound Off/On-Ramp	E	94%

The analysis revealed that the best intersection level of service would be provided with the two interchange configuration, due to the dispersal of traffic. With this configuration, a three-lane section on the South Arterial at the interchange would be adequate.

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The mitigation required at the south ramp terminals of the split diamond interchange is to provide a five-lane roadway section in the vicinity of the interchange. The extremely poor level of service forecast for the diamond interchange includes separate right turn lanes on Highway 214 as well as double left turn lanes from Highway 214 to the on-ramps. This would represent the maximum possible "build" for a diamond interchange design. Even with this configuration, it is very likely that the level of service will be significantly worse since it would be difficult to achieve the lane balance assumed in the analysis because of the influence of the upstream and downstream signalized intersections.

Possible variation to the diamond interchange design is to construct of a partial cloverleaf at Highway 214. A very important consideration in any interchange design is the potential lane balance which will be accommodated by the design. Lane balance is determined by evaluating the design to ensure traffic is distributed equally in all lanes as assumed by the intersection analysis program or to make the appropriate adjustment in the calculation to reflect the influence of the upstream and downstream traffic signal and roadway.

Analysis of the year 2015 PM peak hour westbound traffic volumes on Highway 214 reveals that 1900 vehicles per hour will approach I-5 in the westbound direction. This traffic would pass through the Evergreen Road traffic signal which is located 800 feet east of the northbound I-5 ramp terminal. In order to accommodate the 1900 vehicles per hour through the Evergreen traffic signal, the two westbound lanes must equally share the total westbound traffic.

This is equivalent to 950 vehicles per hour per lane in order to access either I-5 northbound or I-5 southbound. Since the location is no greater than 800 feet east of I-5, all the traffic to northbound I-5 and a majority of the traffic must be in the right lane. Even with a right turn lane, traffic will not be able to move out of the through lane until the platoon approaches the traffic signal and if this traffic is provided signal progression, which is required to minimize the queue on this approach, only 470 vehicles of the 840 vehicles desiring to enter I-5 southbound can be in the right lane at the northbound ramp terminal. This requires a total of nearly 400 vehicles to shift two lanes immediately after passing through the northbound ramp terminal signal and prior to entering the loop ramp to I-5 southbound.

Similar conditions will be created for eastbound traffic approaching the southbound ramp terminal traffic signal. The total volume of 1450 vehicles per hour would leave the Woodland/Highway 214 traffic signal. With the southbound on-ramp followed by a northbound on-ramp to I-5, 930 of the 1450 vehicles per hour must be in the right lane to avoid lane shifts on the I-5 structure.

When these conditions of poor lane balance occur downstream of a signalized intersection, the assumed lane balance as provided with the intersection level of service calculation will not be achieved. If lane balance is achieved as assumed in the calculation, congestion will occur downstream of the traffic signal, at the diverge point resulting in delay and safety problems extending back to the adjacent intersection. This lane balance may be alleviated somewhat by increasing the distance between the adjacent traffic signal and the downstream diverge point.

Conclusions

Based on the freeway and ramp intersection operations analysis conducted for the different I-5 interchange alternatives, the following conclusions can be drawn:

1. All of the alternatives would have similar operating characteristics on I-5, ranging from level of service C to D.

2. The proximity of a second interchange at Butteville Road would not have an adverse impact on Highway 214 interchange ramp operations, and vice versa.

3. The existing diamond interchange would experience very poor level of service E to F at the ramp terminal intersections, even with added through and left turn lanes at the northbound and southbound ramp intersections. The overall cross-section of Highway 214 would be a 6-7 lane roadway.

4. The split diamond interchange would improve level of service at the Highway 214 ramp intersection from C to D, with only five lanes required on the I-5 overpass structure. At the south interchange, five lanes on the South Arterial would be required to provide level of service D or better.

5. The best intersection ramp terminal level of service (C-D) is achieved with two interchanges, due to the dispersal of traffic with this configuration. However, Highway 214 would still have to widen to a five-lane section on the I-5 overpass. The South Arterial could operate as a three-lane facility at the south interchange.

6. The partial cloverleaf interchange would operate very similarly to the diamond interchange or the split diamond interchange for freeway merge/diverge conditions. Analysis of the ramp intersection level of service as shown in Table 6, reveals the overall operation very similar to the diamond interchange. This is due to the extremely poor lane balance which would result at the interchange traffic signals because of the lane balance required at the adjacent signalized intersections at Woodland Road and Evergreen Road to achieve level of service D.

5.5.4 Land Use/Environmental/Cost Impacts

Table 7 summarizes the general land use, environmental and cost impacts of the highway system alternatives. A discussion of each impact area follows.

Table 7

Summary of Land Use/Environmental/Cost Impacts

Impact Category	Roadway System Alternative			
	No-Build	Alternative #1 I-5 Split Diamond Interchange	Alternative #2 2nd I-5 Interchange	Alternative #3 Improve Existing Interchange
Land Use				
Serves Overall Development in Area	–	+	–	O
Promotes Development Inside Current UGB	O	+	–	+
Right-of-Way Requirements	+	–	–	O
Environmental				
Wetlands/Environmentally Sensitive Areas	+	+	+	+
Air Quality	–	+	+	O
Cost	+	–	–	–

+ Positive impact

O Minimal impact or not applicable

- Negative impact

Land Use

The no-build and three roadway system improvement alternatives would have different impacts on future land development patterns within the Woodburn area. The no-build alternative would severely limit added development potential in the City, as most of the current undeveloped area within the existing Urban Growth Boundary is in the southwest portion of the City, which has virtually no street system currently developed. The absence of improved I-5 access and a South Arterial facility under this alternative would limit the ability to develop this area in the future, without having significant traffic infiltration impacts through the Senior Estates area south of Highway 214.

The development of an I-5 split diamond interchange with a northerly South Arterial alignment would improve access to the undeveloped southwest portion of the City, but the South Arterial facility east of I-5 would bisect the proposed Woodburn Crossing mixed-use development area and potentially have a negative impact on the master plan for this development. The development of a second I-5 interchange in the vicinity of Butteville Road, with a southerly South Arterial alignment, would provide the best overall access to the southwest portion of the City, but could stimulate development pressures outside the existing Urban Growth Boundary, as the new interchange and a portion of the South Arterial would be located outside the UGB. With a northerly South Arterial option, the road could be developed entirely within the existing urban growth boundary.

Finally, the alternative to improve the existing I-5/Highway 214 interchange without a direct connection to the South Arterial (which would have a separate overpass of I-5 under this alternative) would provide only limited improved access to the southwest portion of the City. This alternative (assuming a partial cloverleaf ramp improvement were developed) would also require the taking of some existing businesses in the northwest and southeast quadrants of the interchange. The other interchange build alternatives would be developed in existing undeveloped areas and not have as great an impact on existing businesses. Also the widening of Highway 219/214 under this alternative would probably require significant right-of-way from existing businesses along this roadway.

Under any of the build alternatives, other collector and local street extensions and widening would result in some right-of-way acquisition thus impacting adjacent development to some extent.

Environmental

Under any of the build highway system alternatives, the potential alignments for new interchange and street construction appear to result in limited impact on environmentally-sensitive areas in the Woodburn area. The South Arterial could be aligned south of the small reservoir on the south side of the City, and be potentially located around the south side of the sewage lagoon east of Highway 99E if eventually connected to Highway 214.

Cost

The difference in cost of the different highway system alternatives is primarily associated with the different I-5 interchange configurations. The I-5 split diamond interchange configuration is estimated to cost as much as $15 million, while a second I-5 interchange at Butteville Road could cost as much as $20 million (pending the final location of the I-5 over-crossing and the extent of frontage road development between the existing Highway 214 interchange to the north and the new interchange). Improvements to the existing interchange (assuming conversion to a partial cloverleaf configuration with a new bridge) could be up to $10 million, and possibly more pending the level of I-5 frontage road development south of Highway 214.

The South Arterial improvement cost (from Highway 219 to Highway 99E) would probably range from $5-10 million for the split diamond/new interchange alternatives, being of lesser cost with the second I-5 interchange at Butteville Road due to the shorter length of new roadway to be constructed.

The cost of other street improvements in each highway system alternative would be similar, with the possible exception of street improvements in the current undeveloped southwest portion of the City, whose configuration will be dependent on a final master plan for that area as well as the eventual I-5 interchange/South Arterial improvement configuration. Improvements to arterial and collector streets could total up to $40 million.

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Section 6

Transit Systems Alternatives Analysis

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6.0 Transit System Alternatives Analysis

6.1 Transit Demand

In fiscal year 1994 (July 1, 1993 through June 30, 1994), the Woodburn fixed-route bus service had an annual ridership of 28,998 passengers. The paratransit system only had an annual ridership of 3,159 passengers. The combined 32,157 annual ridership on the two systems related to 2.11 transit trips per resident in 1994. Over the past ten years, annual ridership has stayed about the same on the transit system.

Figure 19 shows the 1994 population distribution in Woodburn, for nine zones within the city. For each zone, the population associated with more transit-dependent persons - 0-17 and over 60 years of age - is identified. The map reveals a high concentration of elderly in the Senior Estates area just east of I-5. In central and east Woodburn, there are more youngsters and younger and middle age adults.

By year 2015, population in Woodburn is projected to increase by 100 percent, to 30,000 persons. Assuming transit ridership would grow in proportion to the population increase, and assuming increased transit service were provided to handle the added population, an annual ridership of about 65,000 would be expected combined on the city fixed-route and paratransit services.

With the increasing number of people moving to Woodburn and commuting to either the Portland metro area or Salem, there appears to be some potential demand for shuttle bus service between Woodburn and these two areas. There currently is no transit service serving this demand.

6.2 Major Activity Centers

There are several trip attractions in Woodburn which should be served by public transit service. Figure 20 identifies these attractions, which comprise retail centers, employment concentrations, parks, golf courses, schools, retail centers, downtown Woodburn, and city offices. The existing fixed-route bus route serves most of these attractions, with the notable exception of not serving the employment area along Progress Way north of Highway 214 and west of Highway 99E.

6.3 Intracity Fixed-Route Bus Alternatives

Four fixed route bus alternatives were identified to the existing condition, which is a single, one-way loop route. The existing route and alternatives are illustrated in Figures 21 through 24, and are compared in Table 8. A description of each alternative follows:

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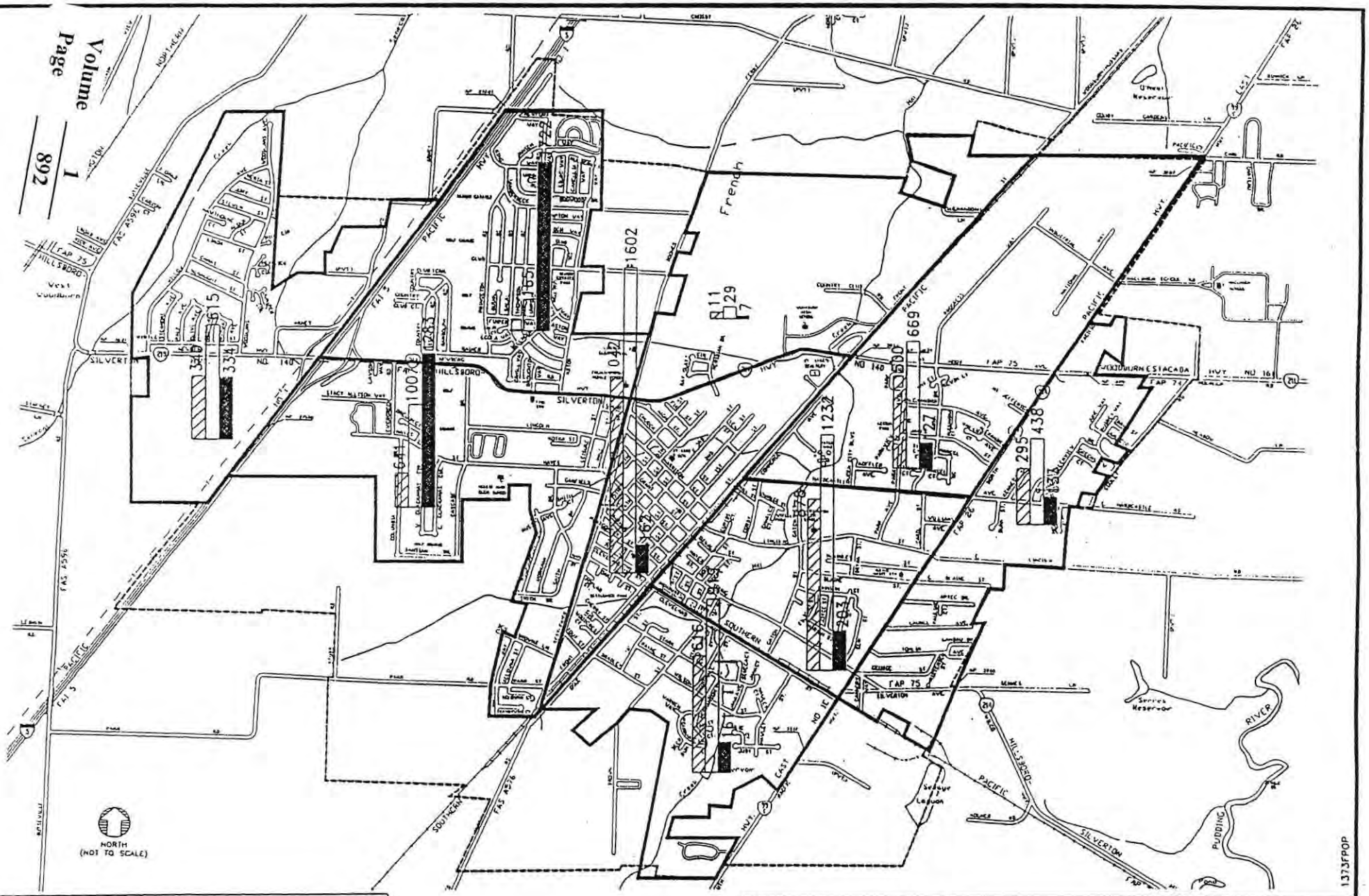
Alternative #1: Increase Service Frequency on Existing Route

With this alternative (see Figure 21), the existing 15.3 mile one-way loop route would be maintained, with service extended to a 12-hour period from 7 a.m. to 7 p.m., with buses operating every 30 minutes. An expansion of the hours of operation of the fixed route service should be considered, to include the a.m. and p.m. peak commuting times. To achieve the



LEGEND

- 1994 POPULATION: 0-17 YEARS OLD
- 1994 POPULATION: 18-60 YEARS OLD
- 1994 POPULATION: 60+ YEARS OLD
- ZONE BOUNDARY FOR POPULATION TABULATION

1994 POPULATION DISTRIBUTION

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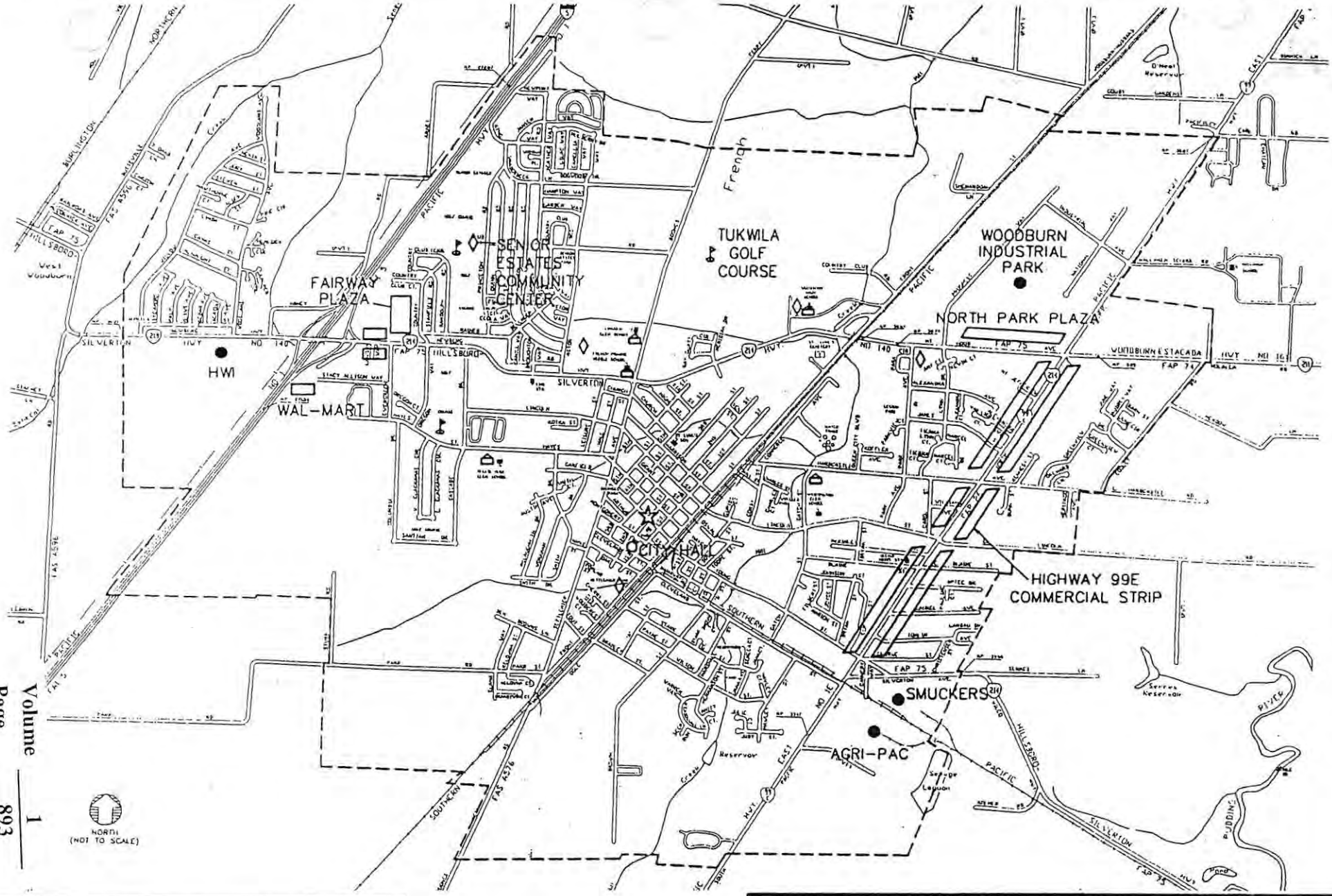
FIGURE 19



LEGEND

- ◊ = RECREATION
- = SCHOOL
- ▭ = RETAIL
- = GOLF COURSE
- ☆ = DOWNTOWN
- ● = EMPLOYMENT NODES
- ○ = GOVERNMENT CENTER

MAJOR ACTIVITY CENTERS

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FIGURE 20

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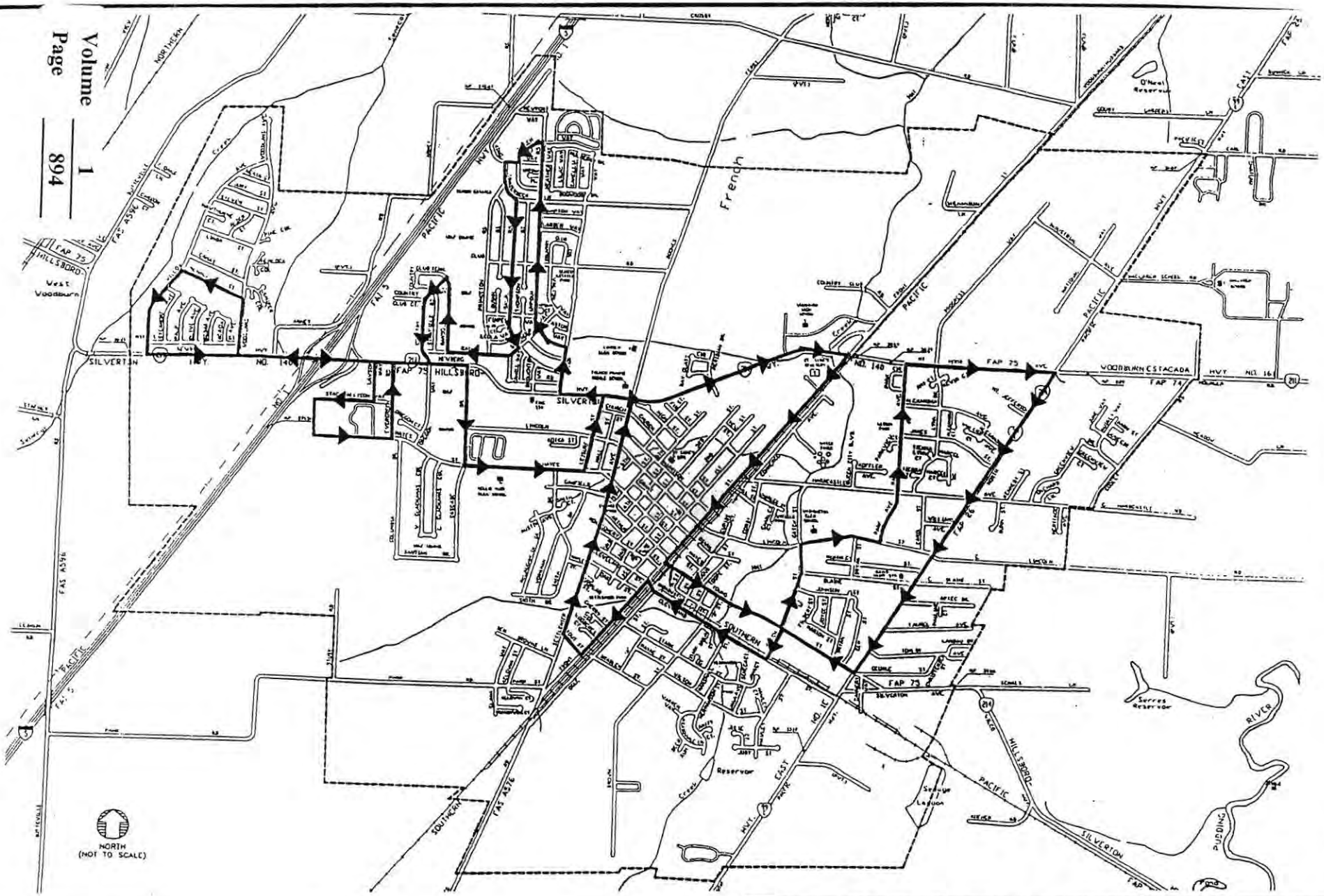

LEGEND

→ BUS ROUTE AND DIRECTION OF TRAVEL

FIXED-ROUTE BUS ALT. #1:
INCREASE SERVICE FREQUENCY
ON EXISTING ROUTE (ONE-WAY)

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FIGURE 21

Table 8

Comparison of Intracity Fixed- Route Bus Alternatives[1]

Fixed-Route Bus Alternative	Service Frequency (min.)	Route Miles (one-way)	Avg. Bus Travel Speed	# of Buses Required	Vehicle Miles/Year	Vehicle Hours/ Year	Added Vehicle Capital Cost	Vehicle Operating Cost/ Year[2]	Estimated Annual Ridership
1. Increase Service Frequency on Existing Route	30 (7a.m.-7p.m.)	15.3	13 mph	2	80,784	6,120	$110,000	$145,900	40,000
2. Convert Single Route to 2-way Operation	60 (7a.m.-7p.m.)	14.8	13 mph	2	78,336	6,120	$110,000	$141,500	50,000
3. Create 2 Routes (East/West) with 1-way operation	30 (7a.m.-7p.m.)	E - 9.15 W - 6.1	13 mph	3	80,478	6,120	$220,000	$145,400	60,000
4. Create 2 Routes (East/West) with 2-way Operation	30 (7a.m.-7p.m.)	E - 9.15 W - 6.1	13 mph	6	160,956	12,240	$550,000	$290,800	75,000
Existing	60 (9a.m.-4p.m.)	15.3	13 mph	1	40,392	3,060	$0	$73,000	28,000

1 Assumes bus operation only on weekdays for 51 weeks per year (thus accounting for no service on holidays).

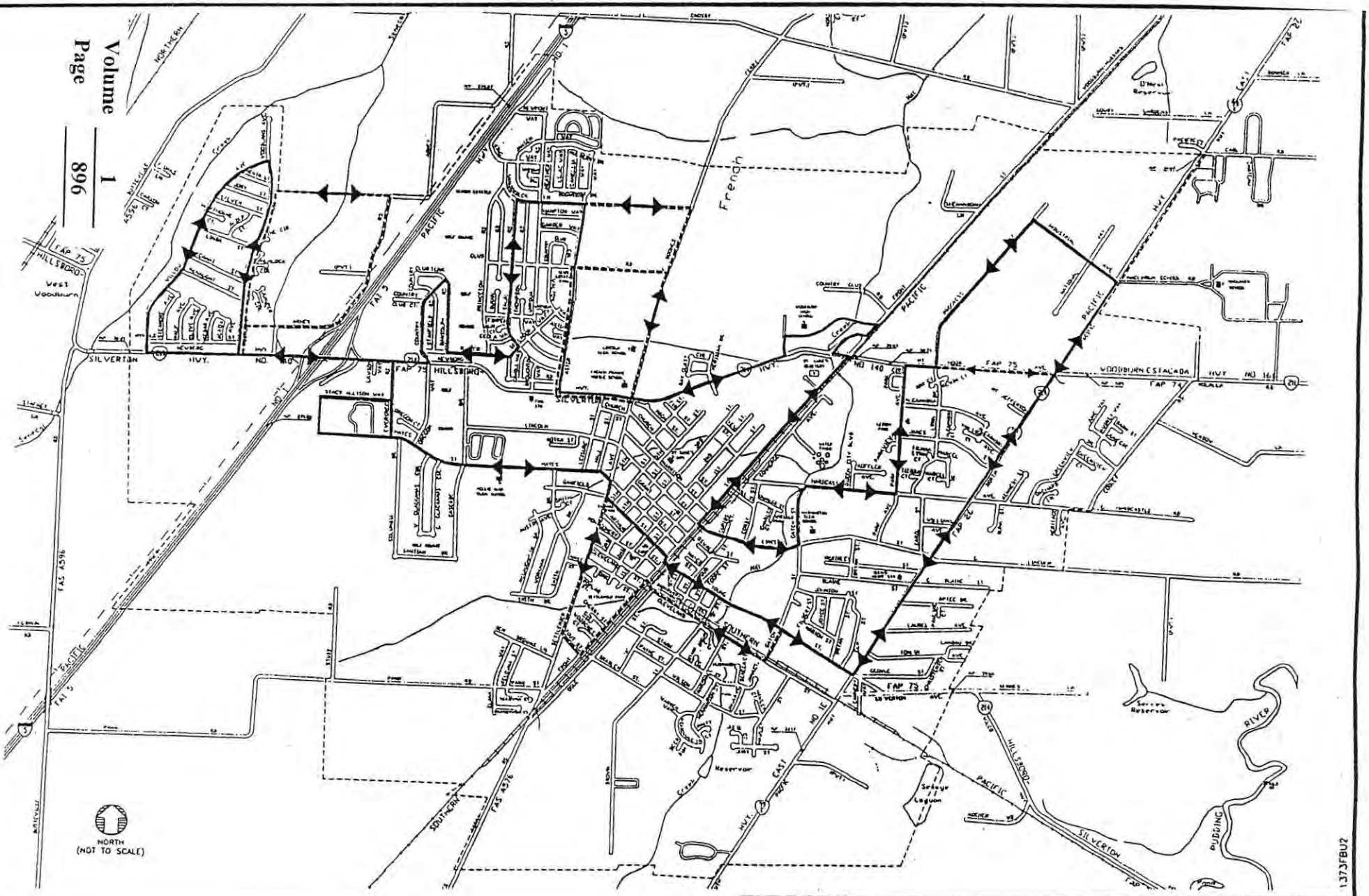
2 Based on 1993-94 Woodburn Transit System operating cost of $1.80/vehicle mile.

increased bus service, a second bus would need to be added to the fleet. Transit system operating costs are estimated to increase to $145,000 from the existing $73,000, assuming the operating costs would increase in proportion to the increase in vehicle miles and hours of travel (a somewhat conservative assumption as this also assumes a doubling of administrative costs). Also an added bus would be required, at a cost of about $110,000 (if the bus were similar to the existing 23-seat bus being operated).

A major problem with the one-way loop operation is that the bus service does not efficiently serve travel oriented in the opposite direction of the bus operation, particularly for short trips.

Alternative #2: Convert Single Route to Two-Way Operation

Overall passenger accessibility along the bus route could be improved by preserving the loop route, but modifying the service to have buses operating in both directions at the same time (see Figure 22). This service alternative was evaluated preserving the existing 60 minute service frequency, in each direction of travel. Service would be expanded to 7 AM to 7 PM on weekdays. This service concept would require two buses, with operating costs similar to increasing service frequency on the existing one-way loop route. Overall transit operating costs



LEGEND

→→ BUS ROUTE AND DIRECTION OF TRAVEL

- - - OPTIONAL BUS ROUTING

FIXED-ROUTE BUS ALT. #2: SINGLE ROUTE WITH TWO-WAY OPERATION

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FIGURE 22

($143,000) would be similar to increasing service frequency on the existing one-way loop route, with an added bus required ($110,000).

The basic route evaluated for this alternative is similar to the existing route, with the exception of focusing the service on one street as opposed to two closely parallel streets in certain areas (e.g. service east of downtown is focused on Young Street, while along the existing route buses operate on both Young Avenue and Cleveland Street, one block to the south). A reconfiguration of the bus route through Senior Estates north of Highway 214 is also proposed with this alternative, to operate along Astor Way instead of Umpqua Road, and not operate north of Vanderbeck Lane. This provides better spacing of the service in that particular area. The route would also extend north of Highway 214 west of Highway 99E to serve the industrial area along Progress Way, thus linking residential areas of the community with this employment area.

There are also several route options within this alternative. Northwest of the I-5 interchange, the bus service initially might be oriented to the residential development along Woodland Drive. In the future, if commercial development occurs along Arney Road, the bus route might make a loop through that area using Arney Road and Woodland Drive. Also as the Tukwila and other residential subdivisions develop along Boones Ferry Road, it could be desirable to extend the bus route operating through north Senior Estates east of Astor Way to access Boones Ferry Road.

Alternative #3: Create Two Routes (East/West) with One-Way Operation

This alternative would preserve the basic one-way bus route that operates today, but splitting the route into two routes: 1) a 9.15 mile west side route operating on the west side of the Southern Pacific Railroad tracks, and a 6.1 mile east side route operating on the east side of the railroad tracks (see Figure 23). This service concept was evaluated with an increased service frequency of 30 minutes, again between 7 a.m. and 7 p.m. on weekdays. The bus service on each route would be scheduled so that buses would have coordinated arrivals in downtown Woodburn, at a so-called transit center, to facilitate transfers between routes. The major disadvantage of this alternative is that cross-city transit commuting would require transferring to another route in downtown Woodburn. The advantages of this alternative are two-fold:

1. It improves service frequency with a minimum increase in buses (only two buses are required - one for each route), with reverse commute trips along a particular route (trips in the opposite direction of the service would be accommodated more conveniently due to the more frequent bus service and shorter bus route); and

2. The downtown transit center concept for bus transferring purposes could serve as a stimulus for downtown redevelopment, particularly if tied into an intercity bus and/or rail station.

With this alternative, both transit operating and capital costs would be higher than the other two alternatives previously described, as a total of three buses would be required (two on the west route, and one on the east route). Operating costs are estimated to be about $175,000 per year, while an added two buses would cost $220,000.

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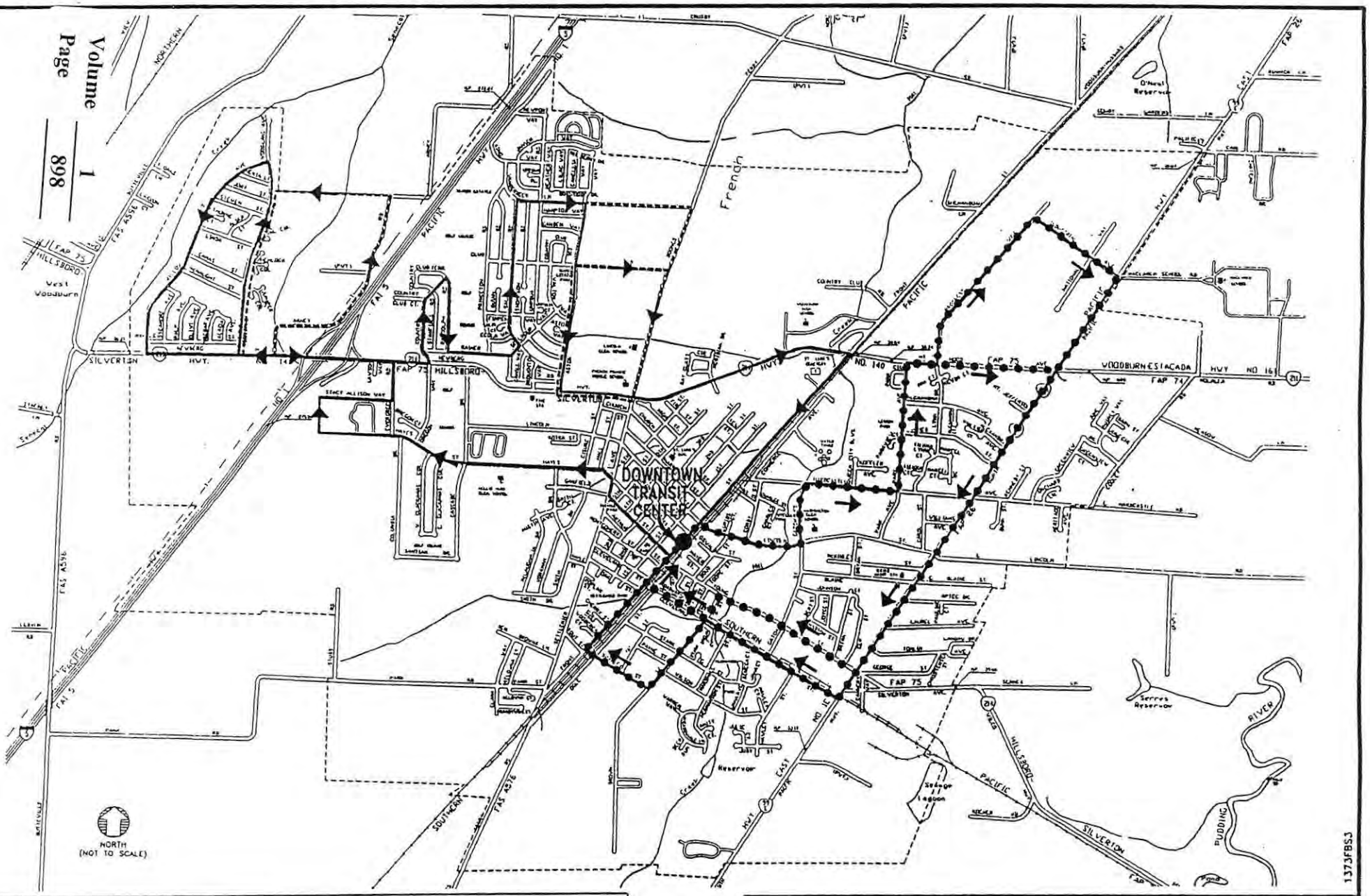

LEGEND

- EAST SIDE BUS ROUTE AND DIRECTION OF TRAVEL
- WEST SIDE BUS ROUTE AND DIRECTION OF TRAVEL
- OPTIONAL EAST SIDE BUS ROUTING
- OPTIONAL WEST SIDE BUS ROUTING

FIXED-ROUTE BUS ALT. #3: TWO ROUTES WITH ONE-WAY OPERATION

CITY OF WOODBURN
DRAFT TRANSPORTATION SYSTEM PLAN
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FIGURE 23

Alternative #4: Create Two Routes (East/West) with Two-Way Operation

Finally, this fixed-route alternative would operate two-way bus service on both west and east side routes, at a 30 minute service frequency (7 a.m. to 7 p.m. on weekdays) (see Figure 24). This is the most intensive service option, with operating costs increasing to about $290,000, and with six buses being required (four buses on the west route, and two on the east route). The five added buses over the existing single bus operation would cost $550,000. This service concept could operate with a 60 minute service frequency, which would reduce operating costs by about 50 percent (similar to alternatives #2 and #3).

6.4 Intracity Paratransit Service

If fixed-route bus service is improved, the City would have the option of reducing the provision of special paratransit service. However, the paratransit system, with door-to-door service, provides the only transit service option for many elderly and handicapped persons in the community, and thus some level of paratransit service will be desirable. In addition, the Americans with Disabilities Act (ADA) requires providers of fixed-route transit to provide complementary paratransit service.

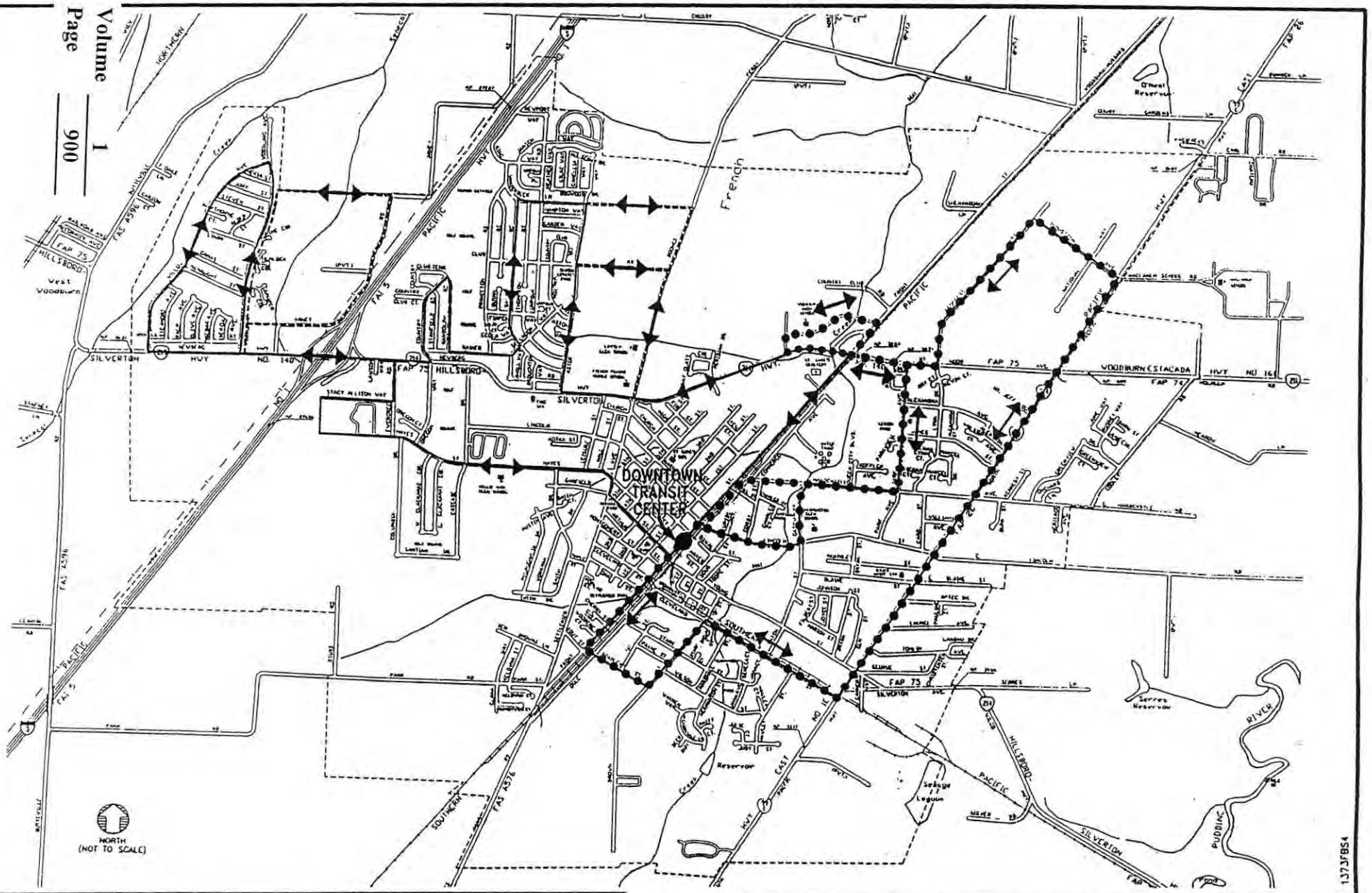
6.5 Intercity Transit Service

As mentioned previously, the Woodburn Transit System currently does not provide shuttle bus service between Woodburn and the Portland metro area or Salem. For direct service to the Portland Metropolitan area the City of Woodburn could either:

- provide service to Tualatin Park-n-Ride - transfer to Tri-Met bus routes to access downtown Portland and rest of system; or
- provide direct service to downtown Portland.

To Salem, the City of Woodburn could provide direct service to downtown Salem and the state office building area.

Table 9 identifies the estimated operating and capital costs for these three options. For all options, a weekday-only service was assumed to a particular destination, with two bus runs during the AM and PM peak hours, and a midday bus run. The same vehicle operating costs per mile ($1.80) were assumed for the intercity shuttle service as for the intracity bus service within Woodburn. The intercity bus service concepts in Table 9, for comparison purposes, were assumed to originate in the vicinity of the I-5/Highway 214 interchange, at a new park-n-ride facility. The service could be expanded further east into central Woodburn to either circulate through the city or terminate at a park-n-ride more central to the city. With a park-n-ride some distance away from the interchange, there would be less potential for this facility to be used by Salem to Portland commuters, thus increasing the available spaces for Woodburn commuters. If a park-n-ride were located in the vicinity of the interchange, it would be desirable to keep the park-n-ride east of I-5 so that commuter traffic accessing the park-n-ride would not have to travel through the I-5/Highway 214 interchange. Given current land availability, this would most likely result in the park-n-ride being developed in the southeast quadrant of the interchange, where there is still substantial undeveloped property. Further east in central Woodburn, there are more limited potential sites.



LEGEND

- EAST SIDE BUS ROUTE AND DIRECTION OF TRAVEL
- WEST SIDE BUS ROUTE AND DIRECTION OF TRAVEL
- OPTIONAL EAST SIDE BUS ROUTING
- OPTIONAL WEST SIDE BUS ROUTING

FIXED-ROUTE BUS ALT. #4: TWO ROUTES WITH TWO-WAY OPERATION

CITY OF WOODBURN
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FIGURE 21

Table 9

Comparison of Intercity Shuttle Bus Alternatives[1]

Intercity Shuttle Bus Alternative	Service Frequency (weekday)	Route Miles (one-way)[2]	# of Buses Required	Vehicle Miles/ Year	Vehicle Hours/ Year	Added Vehicle Capital Cost[5]	Vehicle Operating Cost/ Year[6]	Estimated Annual Ridership
1. Service to Tualatin Park-n-Ride	2 Trips (AM & PM) 1 trip (Mid-day)	17.5	1	44,625	1,230[3]	$150,000	$80,300	35,000
2. Service to Downtown Portland	2 Trips (AM & PM) 1 trip (Mid-day)	31	2	79,050	2,675[4]	$300,000	$142,300	50,000
3. Service to Central Salem	2 Trips (AM & PM) 1 trip (Mid-day)	16.5	1	42,075	1,230[3]	$150,000	$75,700	40,000

1 Assumes bus operation only on weekdays, for 51 weeks/year (thus accounting for no service on holidays).

2 Assumes Woodburn Park-n-Ride within 0.5 miles of I-5 interchange.

3 Assumes one hour round-trip travel time to Tualatin Park-n-Ride or central Salem during peak hours (50 minutes during mid-day).

4 Assumes 2.25 hours round-trip travel time to downtown Portland during peak hours (90 minutes during mid-day).

5 Assumes purchase of 45 passenger buses.

6 Based on 1993-94 Woodburn Transit System operating cost of $1.80/vehicle mile.

Assuming that larger 45-passenger buses are used for the intercity shuttle, and under the maximum service scenario of two peak hour and one midday bus runs, and service to both Portland and Salem, the largest number of spaces anticipated for a park-n-ride would be 225. If a park-n-ride were developed, consideration should be given to provide more spaces than the anticipated transit demand to accommodate carpooling to Portland and/or Salem. Thus a maximum-sized park-n-ride of closer to 300 spaces could be applicable. If intercity bus service is provided to only one destination, and/or the level of service is curtailed, then the park-n-ride can be proportionally smaller. To accommodate 300 parking spaces, about a three acre site would be required. Such a facility could cost up to $1 million.

Recently as part of the preparation of the Marion County Transportation System Plan, the concept of extending the Salem area bus service (CHERRIOTS) to serve Woodburn has been discussed. This service extension could replace or supplement any City of Woodburn - provided fixed route bus service to Salem. To the north, it is highly unlikely that Tri-Met (the public transit provider in the Portland metro area) would ever extend their service to Woodburn as they are attempting to limit their service to the Portland area.

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Section 7

Alternate Pedestrian/Bicycle TDM Strategies

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7.0 Alternate Pedestrian/Bicycle/TDM Strategies

7.1 Introduction

In addition to a set of highway and transit system alternatives for the City of Woodburn, there are also alternate strategies for pedestrian and bicycle facility development and Transportation Demand Management (TDM) measures which should be evaluated toward developing a truly multi-modal Woodburn Transportation System Plan. This section identifies and evaluates facility and policy alternatives with respect to these important transportation system components.

7.2 Pedestrian Strategies

As discussed in Section 4, there is a fairly discontinuous system of sidewalks along existing streets within Woodburn, as well as an absence of off-street pathways. Many sections of arterial and collector streets do not have sidewalks at all or only on one side. There are several locations where pedestrian connections between adjoining neighborhoods or subdivisions have not been developed or are circuitous.

There appear to be two alternate strategies to added pedestrian facility development within Woodburn. These strategies differ by the degree of on-street vs. off-street development of sidewalks and pathways.

Alternative #1: Focus on Added Sidewalk Development

The first alternative would focus pedestrian facility development on adding sidewalks to all sections of existing arterial and collector streets, preferably with sidewalks on both sides of these streets. All new streets, including local streets, would also include sidewalks on both sides. There would be no or very little off-street pathway development.

The major disadvantage of this alternative is that this strategy would be fairly expensive, and probably not the most cost-effective means of improving pedestrian access to major activity centers in the City or to connect adjoining neighborhoods and subdivisions.

Alternative #2: Balanced Program of Sidewalks on Major Streets and Off-Street Pathways

The second alternative would be to develop a balance between added on-street sidewalk development and the development of an off-street pathways system. In particular, a trail system would be developed along the Mill and Goose Creek corridors in the City, totaling over seven miles of trails. In addition, off-street pathways connecting adjoining neighborhoods and subdivisions would be developed where local street connections are not possible. This would be supplemented by more limited sidewalk development, focusing on having sidewalks on at least one side of all arterial and collector streets, but not necessarily on both sides of these streets.

The Mill and Goose Creek corridors through Woodburn provide an opportunity to integrate pedestrian facilities into open space areas, thus enhancing access to these areas by the public

as well as providing more direct pedestrian connections to many activity centers in the City. The Mill and Goose Creek corridors are located next to or in close proximity to all of the schools in Woodburn, thus providing an opportunity to have school children use of safer off-street trails. to access the schools. The on-street sidewalk system could be focused to some extent, at least through signage, on directing pedestrians to the trailheads for the off-street pathway system.

7.3 Bicycle Facilities

Also as discussed in Section 4, the absence of dedicated bicycle facilities in Woodburn is even more critical than the discontinuity of the current sidewalk system. The alternatives for improved bicycle facilities mirrors the alternatives for pedestrian facility development.

Alternative #1: Focus on On-Street Bike Lane Development

This alternative would focus on developing designated bicycle routes and adding bicycle lanes on existing arterial and collector streets, with no or very limited off-street bicycle trail development. Preferably dedicated shoulder bicycle lanes on all arterial and collector streets would be developed, with portions of this system designated as bicycle routes serving different portions of the City and/or major activity centers.

While this alternative would make strides in separating bicycles from motor vehicles on the Woodburn street system, bicycle safety would not be as high as if some off-street bike trails were developed to supplement on-street bicycle lanes. And along many existing streets bicycle lanes can only be developed by widening these streets and possibly requiring added right-of-way.

Alternative #2: Balanced Program of On-Street Bike Lanes and Off-Street Bike Trails

As for the pedestrian facilities, the Mill and Goose Creek corridors provide an opportunity for developing bicycle trails along these corridors, thus separating bicycle from motor vehicle traffic. This alternative would supplement the development of on-street bicycle routes and lanes by developing pathways for bicycles in these creek corridors. Options would exist on the degree of separation between bicycles and pedestrians in these corridors. Either a single pathway in each corridor could be developed to accommodate both bicycles and pedestrians (typically 8-10 feet wide), vs. developing separate pedestrian and bicycle trails (most likely on different sides of the creek corridors due to their limited width). With off-street bicycle trails in the creek corridors, school access by students would be tremendously enhanced.

7.4 Transportation Demand Management

The concept of Transportation Demand Management, or TDM, denotes the implementation of programs and policies to attract people to use modes of travel other than the single occupant auto for their travel, at least to their workplace. This strategy is an integral component of the Oregon State Transportation Planning Rule. Many TDM strategies are instituted or are suppported by employers. There is very limited application of TDM strategies by existing employers or businesses in Woodburn.

There are several potential TDM strategies that could have greater application in Woodburn. The characteristics of each strategy are discussed below (refer to Table 10 for a summary).

7.4.1 Transit Fare Subsidies

With the current transit system in Woodburn, and the potential for transit system expansion in the future, there will be an opportunity for employers to encourage their employees to ride transit to/from work by helping subsidize bus passes. Woodburn Transit today has no subsidy program for regular riders. Many jurisdictions or transit agencies operating bus services have instituted a partial subsidy program, with employees either receiving discounted bus passes or being reimbursed by their employer for actual bus fares.

7.4.2 Carpool Matching Programs

Likewise employers can sponsor carpool matching programs where a service is provided to match employees who live close to one another and on the same shift such that they can carpool together to and from work. In some cases, employers might actually purchase company vans which can be issued to certain employees who become designated vanpool drivers.

7.4.3 Carpool Parking Programs

An employer can also designate certain close up parking spaces to their building for recognized carpools or vanpools. The City of Woodburn could carry this a step further by instituting an ordinance that would reduce parking requirements for new developments if a certain number of parking spaces were reserved for carpools/vanpools. This parking limitation concept typically would encompass an overall employer ridesharing program including carpool matching programs and transit subsidies.

7.4.4 Flexible Work Hours

As most of the traffic congestion in an urban area occurs during commuter peak hours, employer provisions for flexible work hours will allow spreading of the peak hour during a weekday thus reducing congestion for any given peak period.

7.4.5 Telecommuting

Finally with the development of computers and communication software, including the Internet, it is becoming increasingly attractive for employers and businesses to allow their employees opportunities for telecommuting on their jobs, or to conduct other business. This in general reduces the number of vehicle trips on the street system.

7.4.6 Pedestrian/Transit-Oriented Development

The development of more pedestrian and transit-oriented developments, through added and direct sidewalk connections, bus stop provisions, and proper building orientation, can attract more local trip making to these developments via non-auto modes, thus serving as TDM strategies to an extent.

Table 10

Comparison of Transportation Demand Management Strategies

TDM Strategy	Development Applicability	Site Design Consideration	Impact Category			
			Employer Policy	Developer/ Employer Parking Reduction Incentives	Cost	Potential Impact on Trip Reduction
Transit Fare Subsidies	C, S, O, I	NO	YES	YES	Could be substantial pending employer interest & level of subsidy	Limited until bus service is expanded
Carpool Matching Program	C, S, O, I	NO	YES	YES	Minimal	Fairly high if combined with preferential carpool parking
Carpool/Bicycle Parking Program	C, S, O, I	YES	NO	YES	Minimal if tied to parking reduction strategy	Fairly high if combined with carpool matching program
Flexible Work Hours	C, S, O, I	NO	YES	YES	Minimal	Impact on reducing peak hour congestion
Telecommuting	S, O	NO	YES	MAYBE	Minimal	Limited currently
Direct Pedestrian/Bus Stop Connections from Adjacent Development	C, S, O, I	YES	NO	YES	Minimal with proper site planning	Fairly high if tied to other TDM measures

C - Commercial, S - Services, O - Office, I - Industrial

Section 8

Highway 219/214 & 99E Access Management Analysis

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8.0 Highway 219/214 & 99E Access Management Analysis

8.1 Introduction

The Oregon Highway Plan (OHP), adopted in June 1991, contains policies and strategies that will guide the Oregon State Highway Division's (OSHD) operating and fiscal activities during the 1991-2010 period. With implementation of the plan, the State will be able to maintain quality highways and bridges that are safe and cost-effective and provide efficient access throughout the state. One component of the Plan is the Access Management Policy, which is a long range policy intended to develop guidelines for the control of access (driveways, public roads and traffic signals) onto State facilities. This section of the TSP describes the Access Management Analysis performed on Highway 219/214 between the west city limits and Settlemier Avenue, and on Highway 99E between Lincoln Street, and the south terminus of the previous ODOT road improvement project on Highway 99E. The access management analysis includes a description of:

- the State Highway Access Management Guidelines;
- the methodology used to evaluate existing access conditions; and
- an analysis of existing access to Highway 219/214 and Highway 99E.

8.2 State Highway Access Management Plan Guidelines

The Access Management Policy guidelines of the OHP address the fact that, as communities have developed adjacent to state highways, the highways are no longer serving simply regional or interstate travel. In addition, some state facilities are serving a relatively high volume of local trips traveling to and from local services. The higher intensity of driveways, intersections, and traffic signals providing access to these local services has affected the safety, efficiency, and capacity of state highways.

The Access Management Policy guidelines categorize all state highways, including Highway 99E and Highway 219/214, based on a functional hierarchy in terms of Level of Importance (LOI). The four LOI categories are:

- **Interstate**: Providing connections between major cities, regions of the state, and other states.
- **Statewide Highways**: Providing access to larger urban areas, recreational areas, and the interstate system.
- **Regional Highways**: Providing connections to areas within regions of the state, between small urban areas and larger urban areas, as well as connections to higher level facilities.
- **District Highways**: Serving local traffic and land access, and having a relatively low level of significance from a statewide perspective.

The Access Management Policy guidelines also define for each LOI category guidelines for the management of access and traffic control. Table 11 summarizes the Access Management Policy.

Table 11

Oregon State Highway Access Management Classification System

Cat.	Access Treatment	Level of Importance	Urban/ Rural	Intersection				Signal Spacing	Median Control
				Public Road		Private Drive			
				Type	Spacing	Type	Spacing		
1	Full Control (Freeway)	Interstate/ Statewide	U R	Interchange Interchange	2-3 Mi. 3-8 Mi.	None None	NA NA	None None	Full Full
2	Full Control (Expressway)	Statewide	U R	At grade/intch At grade/intch	1/2-2 Mi. 1-5 Mi.	None None	NA NA	1/2-2 Mi. None	Full Full
3	Limited Control (Expressway)	Statewide	U R	At grade/intch At grade/intch	1/2-1 Mi. 1-3 Mi.	Rt Turns Rt Turns	800' 1200'	1/2 Mi. None	Partial Partial
4	Limited Control	Statewide/ Regional	U R	At grade/intch At grade/intch	1/4 Mi. 1 Mi.	Lt/Rt Turns Lt/Rt Turns	500' 1200'	1/2 Mi. None	Partial/None Partial/None
5	Partial Control	Regional/ District	U R	At grade At grade	1/4 Mi. 1/2 Mi.	Lt/Rt Turns Lt/Rt Turns	300' 500'	1/4 Mi. 1/2 Mi.	None None
6	Partial Control	District	U R	At grade At grade	500' 1/4 Mi.	Lt/Rt Turns Lt/Rt Turns	150' 300'	1/4 Mi. 1/2 Mi.	None None

Source: Oregon Highway Plan, ODOT, adopted 1991, pg. B-6

For each of these categories, the Access Management Policy guidelines also differentiate between Urban and Rural roadways. For both Urban and Rural roadways in each category, the guidelines specify the minimum spacing of public roads, private driveway spacing, and traffic signals along state highways. The Access Management Policy also addresses the type of intersections which can be built (at grade or interchange), and whether or not a median can be installed on the state highway. The state expects that these guidelines will be implemented as part of land development/redevelopment projects or roadway improvements projects on state highway corridors. Also, the Access Management Policy guidelines will be implemented to ensure that every property has access to a public road facility.

As an outcome of discussions between ODOT, and the city of Woodburn staff, it was determined that Highway 219/214 between the west city limits and Settlemier Avenue, and Highway 99E between Lincoln Street and the south city limits do "provide for efficient and safe medium to high-volume traffic movements on intercity, intracity, and inter-community routes. There is a reasonable balance between direct access and mobility needs within this category"[1]. Thus, these facilities should be categorized as Category 5 Urban. According to Table 10, on Category 5 Urban roadways:

- public roads should be spaced at least 1/4 mile apart;
- private driveways should be spaced at least 300 feet apart (which converts to approximately 18 driveways/mile on one side of the roadway); and

1 ODOT, Oregon Highway Plan, 1991, p. B-4

- traffic signals should be spaced at least 1/4 mile apart.

8.3 ODOT/City of Woodburn Access Management Agreement

In 1991, ODOT and the City of Woodburn entered into an agreement to implement an access management policy for state highway facilities within the City of Woodburn. The policy focused on three sections of roadway:

- Highway 99E from the north City limits to Lincoln Avenue;
- Highway 214 from the Front Street overpass to Highway 99E; and
- Highway 211 from Highway 99E to Cooley Street.

The purpose of the policy was to identify standards and procedures to preserve the capacity of state highway facilities in Woodburn, to carry traffic efficiently at a high level of service under safe conditions. The policy addressed criteria for public street intersection and driveway spacing and access restrictions, shared access between properties, and access design standards. The policy also included specific access management objectives for different undeveloped or partially developed properties along these roadways, including appropriate number of access points for these parcels, and the location of these access points.

This access management policy and objectives provided guidance on the structure of the access management analysis conducted as part of the Woodburn TSP preparation for the other sections of Highways 219/214 and 99E through the City.

8.4 Methodology

The first step in the access management analysis for the Highway 219/214 and 99E segments addressed in the Woodburn TSP was a review of traffic operations characteristics (level of service, accidents, intersection) and roadway intersection improvements identified in Woodburn Transportation System Plan (TSP) Technical Memorandums #2 (Existing Conditions) and #4 (Alternatives/Needs Analysis). This information was considered in conjunction with the access and signal spacing data in order to learn if there are any locations with unacceptable levels of service, or a high incidence of accidents which may be caused by the improper location of existing driveways, public roads, or traffic signals.

Next, in order to assess compliance with the guidelines of the potential access management classifications for Highways 219/214 and 99E, an existing conditions analysis was performed. In this analysis, a field survey of the study roadways was undertaken (April 1995) which identified the location of:

- existing driveways;
- existing public roads; and
- existing traffic signals.

Table 12 summarizes existing driveway spacing on these two roadways. The spacing of these facilities was evaluated and compared to the guidelines developed in the Oregon Highway Plan, as well as the previously developed traffic operations and safety data. The intent of this analysis

Table 12

Driveway Spacing Summary

Roadway Section	Length Ft. (Mile)	Number of Driveways				Driveways per Mile			
		Existing		With Consolidation		Existing		With Consolidation	
		N/E	S/W	N/E	S/W	N/E	S/W	N/E	S/W
Highway 214									
I-5 / Evergreen Rd.	900 (0.17)	6	5	4	5	35	29	23	29
Evergreen Rd./ Oregon Way	650 (0.12)	3	6	1	5[1]	25	50	8	42
Oregon Way / Broughton Way	800[1] (0.34)	1	5	1	5	3	15	3	15
Broughton Way/ Settlemier Ave.	1,700 (0.32)	2	9	2	9[1]	6	28	6	28
Highway 99E									
Lincoln St. / Aztec Dr.	1,000 (0.19)	10	7	6[1]	4	53	37	31	21
Aztec Dr./ Laurel Ave.	1,100 (0.21)	4	3	2	3	19	14	10	14
Laurel Ave./ Highway 214	900 (0.17)	10	11	5[1]	6[1]	59	65	29	35
Highway 214 / South City Limits	1,000 (0.19)	8	7	4	5	42	37	21	26

1 Additional consolidation should occur with redevelopment of the area.

was to develop a plan for managing access to Highway 214 and Highway 99E such that regional and local trips can be made in a safe and efficient manner.

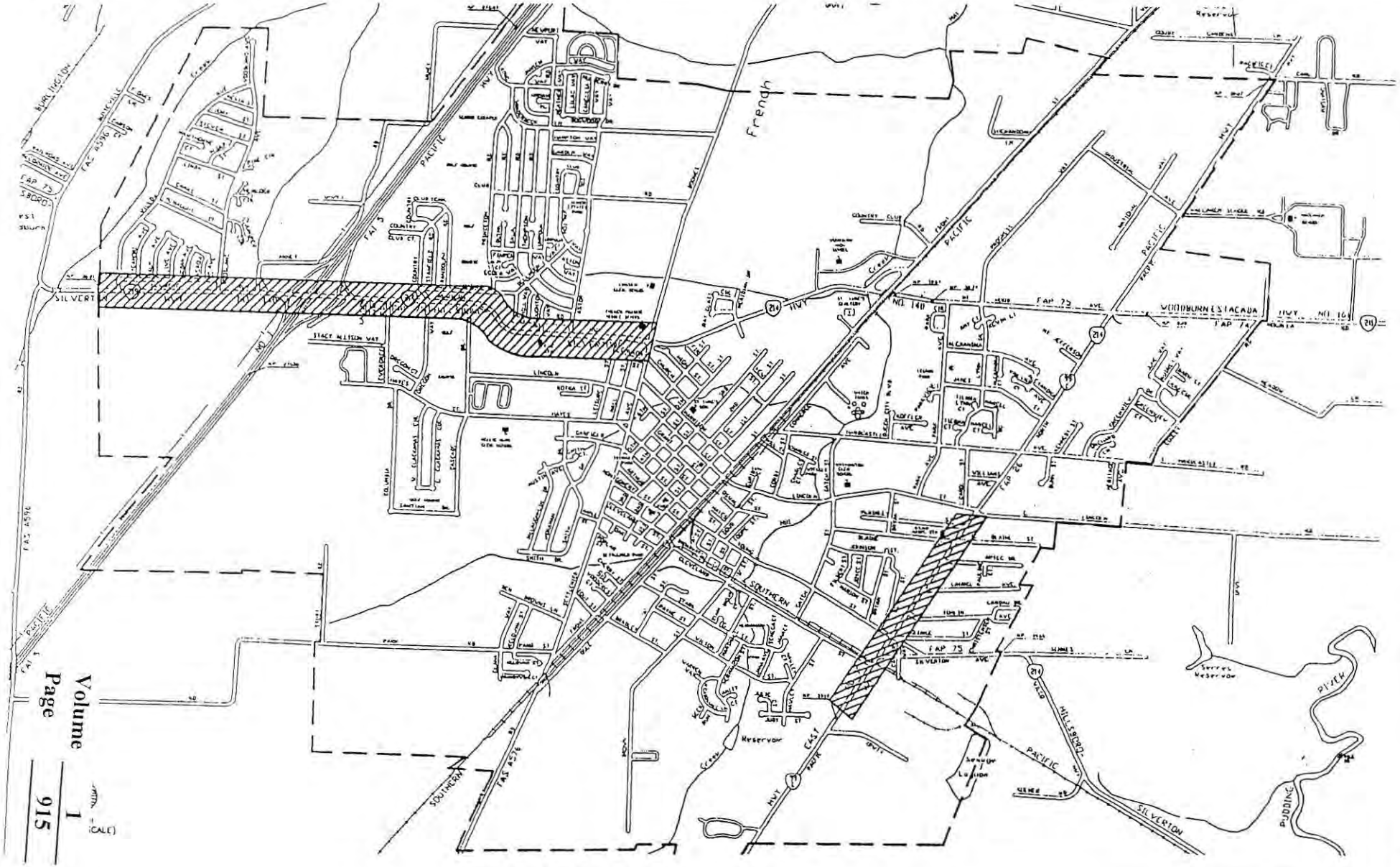
8.5 Highway 219/214 Analysis

Access to Highway 219/214 was studied between the west city limits, and Settlemier Avenue. This section of roadway is approximately 1.6 miles long. Highway 219 extends west of I-5, and Highway 214 east of I-5. In this section, Highway 219/214 is primarily a three-lane roadway consisting of one travel lane in each direction and a median lane for left turns. The Highway 219/214 study segments are summarized in Figure 25. Currently there are traffic signals at:

- the southbound I-5 ramp terminal;
- Evergreen Road;
- Oregon Way; and
- Settlemier Avenue/Boones Ferry Road.

For the purposes of the analysis, this segment of Highway 219/214 was divided into six subsections:

- Segment 1: West city limits to Woodland Road;
- Segment 2: Woodland Road to Interstate 5;
- Segment 3: Interstate 5 to Evergreen Road;
- Segment 4: Evergreen Road to Oregon Way;



LEGEND

- LIMITS OF ACCESS MANAGEMENT ANALYSIS
- URBAN GROWTH BOUNDARY

ACCESS MANAGEMENT STUDY CORRIDORS

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FIGURE 25

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- Segment 5: Oregon Way to Broughton Way; and
- Segment 6: Broughton Way to Settlemier Avenue/Boones Ferry Road.

A review of ODOT accident data (1990 - 1993) showed that, on average, eight accidents per year occurred on Highway 214 between the west City limits and I-5 (0.58 miles). This equates to an accident rate of 6.0 accidents per million vehicle miles. The statewide average for similar types of roads is 3.6 accidents per million vehicle miles.

Between I-5 and Boones Ferry Road (1.06 miles), the accident rate for the same period is 3.3 accidents per million vehicle miles. This rate is lower because of the longer roadway section, and the low incidence of accidents east of Evergreen Road. The majority of the accidents occurred in the westerly one-half mile of this segment.

8.5.1 Segment 1: West City Limits to Woodland Road

Traffic Operations

Under existing conditions, the intersection of Woodland Road/Highway 219 operates at LOS B. Over the three year study period summarized in Technical Memorandum #2 (1992, 1993, and 1994), there were five accidents at both the intersections of Woodland and Willow Avenues.

In the future (2015) it is expected that approximately 15,000 vehicles per day will travel on this segment of Highway 219. Currently, approximately 9,500 vehicles per day travel on this section of roadway.

Because of this higher volume of traffic, and in order to maintain acceptable traffic operations, the City plans to extend Arney Road to Woodland Road, with the existing Arney Road intersection converted to a right-in right-out intersection, and a traffic signal installed at Woodland Road.

Access Issues and Potential Modifications

Existing access conditions in this road segment are shown in Figure 26. The segment of Highway 219 between the west city limits and Woodland Road is approximately 1/4 mile in length and is in the early stages of light industrial/warehouse development on the south side of Highway 219, west of Woodland Road. Access to this 100+ acre industrial site is via Woodland Road south of Highway 214. As this area develops, an internal road system should be created to provide access to the parcels and circulation to Woodland Road. It appears that a curb cut could be provided at the westerly end of this property, approximately 1,200 feet west of Woodland Road to provide a secondary access into and out of the industrial/warehousing area. The area to the north of Highway 219, between Woodland Road and the west city limits, is primarily residential. As this area redevelops, care must be taken to control access along this portion of Highway 219 and provide the major access to this area via Woodland Road.

Since in this section of roadway, with the exception of the public roadways, there is no direct private access onto Highway 219, the driveway density is equal to zero. Thus, related to driveway density, there are no issues in developing compliance with the Oregon Highway Plan Access Management Policy.