"Making a difference through excellence of service"

CITY OF WARRENTON

WASTEWATER FACILITIES PLAN
Final Report

November 2002

This Final Wastewater Facilities Plan is being financed with a loan from the State Revolving Fund.

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WASTEWATER FACILITIES PLAN

FINAL REPORT

Prepared for:

CITY OF WARRENTON
P.O. Box 250
Warrenton, OR 97146

NOVEMBER 2002

REGISTERED PROFESSIONAL
ENGINEER
9443
OREGON
SEPTEMBER 23, 1977
RONALD G. LARSON
RENEWAL DATE: DECEMBER 31, 2002

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SECTION 1
SUMMARY AND RECOMMENDATIONS

1.1 PLANNING AREA

The planning area for this wastewater facilities plan consists of the City of Warrenton, its current Urban Growth Boundary (UGB), Fort Stevens State Park (currently inside City limits), and the Miles Crossing Sanitary Sewer District (currently outside the UGB). Additional information on and analysis of the planning area is presented in *Section 3* of this report.

1.2 EXISTING WASTEWATER SYSTEM

The first portion of Warrenton's existing sewage collection system, the sewer treatment plant, was built in 1968 to serve the then existing City. Prior to 1968, the majority of the City of Warrenton used septic tanks and drain fields for sewage disposal, or occupants of the City discharged raw sewage directly into adjacent waterways. The initial system was expanded in 1975 to serve east Warrenton and the Port of Astoria facilities, including the Clatsop County airport east of Highway 101. The Town of Hammond and Fort Stevens State Park were included in the City of Warrenton sewer system in 1981. As the City of Warrenton has expanded to meet the growing needs of the community, the sewage collection system has also expanded.

The flat terrain of the City of Warrenton service area has required the location of several pump stations within the collection system. The original City system consisted of three (3) pump stations and currently the City has 26 pump stations.

The existing sewage treatment system consists of a two (2) cell stabilization lagoon, currently operated in series, followed by disinfection by chlorination. The treatment plant currently has no pretreatment. A description of the entire treatment facility, including the history of the outfall ditch construction, can be found in Section 4 of this report. Background information supporting the justification of an extended mixing zone for the existing outfall can be found in Section 4.2.9.

1.3 POPULATION PROJECTIONS AND LAND USE PLANNING CONSIDERATIONS

The planning period is through the year 2023 (20-year period). The projected design population (year 2023) of 6,585 reflects a 3.2% straight-line annual growth rate of 113 persons per year and a current (2001) population of 4,210.

The City of Warrenton has much potential for growth due to its large area (greater than 11 square miles). With this in mind, the City of Warrenton Planning Department has developed a sound basic concept detailing community development that will encourage appropriate and balanced growth.

1.4 WASTEWATER FLOW AND LOAD PROJECTIONS

The wastewater flow and load projections are based on the existing and projected populations of the service area, as well as historical wastewater flows and loads measured at the treatment plant. The existing (2001) and 20-year (2023) wastewater flows and loads are shown in *Table 1-1.* These flows and loads include City growth, Clatsop County Corrections Transitional Facility (inside the UGB) and the Miles Crossing Sanitary Sewer District (outside the UGB). The analysis used to project the wastewater flows and loads is presented in *Section 5* of this report. Several definitions of the detailed terms used in wastewater systems are presented in *Section 5* of this report. Some of those terms are used in the following table. See *Section 5* for additional definitions.

TABLE 1.1 - EXISTING AND FUTURE WASTEWATER FLOWS AND LOADS

PARAMETER	EXISTING (2001)	FUTURE (2023)
Wastewater Flow (mgd)		
Annual Average	0.7	1.1
Maximum Month Wet Weather Avg	1.1	1.6
Maximum Day, Wet Weather	1.5	2.3
Hydraulic, Peak Instantaneous Flow	3.4	4.7
Wastewater Load (lb/day)		
BOD_5 (Bio-chemical Oxygen Demand)		
Annual Avg	1,000	1,720
Maximum Month Avg, Summer	1,500	2,500
TSS (Total Suspended Solids)		
Annual Avg	1,300	2,000
Maximum Month Avg	1,900	2,900
Ammonia, Max Month Avg	150	250

In brief summary, the wastewater lagoons are currently seriously overloaded. The BOD_5 and TSS influent loadings to the lagoons are now at about two times the recommended loading level for facultative lagoons as recommended by DEQ. Additional analysis and information on the wastewater flows and loads are presented in *Section 5* of this report. Additionally, the first lagoon (Cell #1) is not operating at peak efficiency due to the accumulation of large quantities of bio-solids (sludge) in Cell #1. Additional analysis and information on the biosolids (sludge) management issues are presented in *Section 8* of this report.

The DEQ estimates that the lagoons are only operating at 50% of design efficiency because of the build-up of biosolids. Therefore, the combination of the overloaded sewer influent and the build-up of biosolids results in an effective overloading of approximately four times the design capacity of the sewer lagoons. Additional analysis and information on the wastewater treatment system is presented in *Section 7* of this report.

1.5 COLLECTION SYSTEM

The core infrastructure of this system has exceeded its design life. It continues to perform well, but needs additional upgrades to meet new flows from additional areas of development.

Development in outer areas, away from the core system, and current core system conditions now necessitate upgrades to the gravity system, the pressure system, and the pump station system. These upgrades are described *in Section 6.3* of this Facilities Plan.

In the case of the pump station system, several of the Warrenton Pump stations are now obsolete, making in very difficult if not impossible to continue obtaining parts to keep them operating. It is now cost effective to renovate or replace those stations.

Inflow and infiltration (I/I) at the East Warrenton Industrial Park (currently inside City limits) continues to be a problem for the collection system, particularly the five (5) pump stations on the East Warrenton Interceptor. This additional loading is creating wear/maintenance/electrical costs to soar.

Additional areas of development that would add to the collection system are not part of this discussion, but all costs should be borne by the parties that are seeking to develop, such as the areas north of Harbor and east of Highway 101. This also would include outside sources seeking to benefit from the City of Warrenton's wastewater treatment.

1.6 INFLOW AND INFILTRATION (I/I) STUDIES

An I&I study was conducted as part of the Sewerage System Facilities Planning Report prepared by Westech Engineering Inc. in April of 1983. The report is included as an independent report in Appendix H of the Westech Report. The recommended program was never thoroughly implemented according to the proposed schedule. *Section 6.3* of this report recommends that the initial recommended program be implemented prior to conducting any additional studies. Once the program has been implemented, an evaluation of the program and remaining I/I problems in the system should be conducted. Such a study would make recommendations as to whether additional I&I removal effort would be beneficial and cost effective for the City.

1.7 MUTUAL AGREEMENT AND ORDER (MAO) AND COMPLIANCE ISSUES

The Department of Environmental Quality (DEQ) issued on March 29, 1999 and on February 9, 2001 a Notice of Noncompliance for NPDES Permit violations. DEQ believes that the City is having difficulty meeting the NPDES Permit limits because the facility remains overloaded by influent BOD and TSS. The DEQ further believes that due to the overloading, the facility will likely continue to violate discharge limits. Due to these issues and the fact that the DEQ and the City wish to limit any past and future violations, they entered into a Mutual Agreement and Order (MAO). [Source: Mutual

Agreement and Order, State of Oregon Environmental Quality Commission and City of Warrenton. See *Appendix H* for a copy of the MAO.]

The City of Warrenton signed the Mutual Agreement and Order (No. WG/M-NWR-01-281) with the Department of Environmental Quality (DEQ) on December 5, 2001. The DEQ signed the MAO on December 24, 2001. The City has had numerous discharge limit violations at their treatment plant and has been issued Notice of Noncompliance by DEQ on March 29, 1999 and February 9, 2001. The DEQ believes that the City is having difficulty meeting permit limits due to influent BOD and TSS overloading.

The interim wastewater treatment plant discharge limits from the MAO are shown on *Table 2*, from paragraph 8 B of MAO. Those same interim limits are shown in *Section 7.2.2* of this report.

<u>Interim Engineering Study</u>
The MAO required compliance schedule is included later in this report (See *Appendix H*). The MAO will allow additional connections to be made to the treatment facility, subject to the effluent limits shown on Table 2 of the MAO and provided that a plan to maintain interim discharge limits is approved by DEQ [paragraph 8 A (7) of MAO].

The Interim Capacity Increase Technical Memorandum (See *Appendix A*) is intended for submittal to DEQ for approval to satisfy these requirements for adding additional connections to the City of Warrenton wastewater treatment facility.

The recommended interim treatment upgrade presented in *Section 7* of this report is intended to provide the added treatment capacity for treatment of waste loads that are over the capacity of the existing lagoons (with the biosolids removed). Additional interim treatment will also provide extra capacity for proposed connections prior to completion of the secondary treatment facility upgrade, with provision for continued treatment with decreased size of the South Lagoon (Cell #1) during construction. The MAO provides interim effluent limits and a schedule for improvements to the City system needed to come into compliance with NPDES Permit requirements. The existing NPDES Permit (No. 100874) expired on March 31, 1997, but has remained in effect since the City has made timely application for renewal.

1.8 WASTEWATER TREATMENT ANALYSIS

The existing sewage treatment system consists of a two (2) cell stabilization lagoon, currently operated in series, followed by disinfection by chlorination. The existing wastewater treatment system is currently overloaded and has experienced permit violations that are expected to increase in frequency if improvements are not made. Some interim improvements have been made to the lagoon system. In March of 2000, construction of the Sewer Lagoon Improvements Project was completed. The improvement consisted of the following:

- Relocate 12" diameter force main into treatment plant

- Construct a new influent Parshall flume (flows frequently exceeded the capacity of the old flume)
- Install influent flume flow monitoring equipment
- Install an influent flume composite sampler
- Transfer pipe modifications with floating inlet to transfer pipe (to transfer flow from Cell #1 to Cell #2)
- Install a floating baffle in Cell #2 to redirect flow throughout all of Cell #2 preventing "short-circuiting" in this cell.

Following the above described improvements, the wastewater treatment system improved the overall quality of the effluent (See *Figure 7.5* in *Section 7.3*). Those improvements were not designed to completely address all of the sewer system overloads. Additional improvements to the wastewater treatment system are still needed both for the high level of influent loading and to allow for future growth.

The recommendations for possible approaches to upgrading the wastewater treatment system include the following:

1) Expand the existing lagoons onto the City-owned land to the West of the existing lagoons.
2) Modify and expand the existing lagoon system, incorporating an aerated lagoon and a constructed free water surface wetlands.
3) Modify the existing lagoons to construct a Sequencing Batch Reactor (SBR) with sludge holding lagoons and Ultraviolet (UV) disinfection.

Additional analysis and evaluation of the existing wastewater treatment system along with a detailed discussion of each of the above alternatives for upgrading the treatment system is presented in *Section 7* of this report. The final alternative selected by the City for further evaluation was the Sequencing Batch Reactor (SBR) system with sludge holding lagoons and ultraviolet (UV) disinfection. Details of the recommended system, including construction and annual operating costs, can be found in *Section 7* and *Appendix C.*

Since the preparation of the Draft Plan, a Mixing Zone Study (*Appendix B*) has been completed determining that an extended outfall to the Columbia River would be required to meet water quality standards. The estimated construction cost of the recommended treatment alternative including the proposed outfall pipe is as follows:

Sequencing Batch Reactor (SBR)	$5,736,000.00
Core Conveyance System Improvements	$1,123,000.00
Outfall to Columbia River	$1,130,000.00
Total Cost	**$7,989,000.00**

1.9 INTERIM IMPROVEMENTS

The MAO between the City and DEQ states that the City may submit for DEQ approval an Interim Engineering Study for proposed interim improvements to the existing lagoons needed to provide capacity to allow additional waste loads during the term of the MAO. The City has chosen to exercise this option and has therefore had a report prepared (*Appendix A*) that proposed interim improvements that would accommodate City growth in the interim period along with projected waste loads from the Miles Crossing Sewer District and Fort Clatsop National Park. The details of the interim improvements can be found in *Section 7* and *Appendix A*.

1.10 BIOSOLIDS MANAGEMENT

The Warrenton treatment facility has been accumulating solids since its original construction in 1969. Biosolids have accumulated to unacceptable levels contributing to overloading problems primarily due to the resulting reduction of the water column in the lagoons.

A Biosolids Management Plan, dated January 2002, and a Biosolids Site Authorization Submittal, dated February 2002, has been prepared by Lee Engineering, Inc. for the City of Warrenton. Both have been submitted to DEQ for review and approval. Both reports are included in *Appendix J* of this report. The purpose of the Biosolids Management Plan is to outline how the biosolids will be removed, transported and land applied in accordance with OAR 340-050-0031 and Federal 503 regulations. The submittal includes a management agreement between the City of Warrenton and the owner of the application site property and details regarding management of the sites.

The Biosolids Management Plan outlines two (2) methods of biosolids removal. They are as follows:

1) complete removal of all biosolids and land application this year (2002)
2) construct a dike that divides Cell #1 into two (2) smaller cells; pump to the new storage cell to the east and remove sludge over a longer period of time

The revised schedule of the biosolids removal meets the requirements of the recently signed Memorandum of Agreement and Order. The biosolids removal was originally scheduled for the summer of 2002. This schedule is contingent on the City receiving DEQ approval of the Biosolids Management Plan and the Biosolids Site Authorization.

The total estimated cost for biosolids removal, transportation, and land application is $480,000.00. The removal of biosolids must occur by September of 2003 to accommodate the proposed interim capacity improvements.

1.11 FINANCING

The City conducted a One-stop meeting on December 11, 2001 at which time the project was discussed along with available funding options. The meeting was attended by the City of Warrenton staff, United States Department of Agriculture Rural Development (USDA RD) staff, Oregon Economic and Community Development Department (OECDD) staff, DEQ staff and a representative from the Governor's Community Solutions Office. At this meeting, three sources of funds were identified as follows:

1) USDA RD has funding available for the project. Due to the high cost of the project, it is anticipated that USDA RD would participate with other funding agencies.
2) The DEQ may have funds for this project from the Clean Water State Revolving Loan Fund (SRF).
3) The Oregon Economic and Community Development Department (OECDD) has funds available for this project. The City qualifies for the Water/Wastewater Financing Program.

The OECDD, DEQ and state agencies will work with the City in pursuing funding for the project once overall scope and cost of the wastewater system improvements are determined. See *Section 9*, for further detail regarding financing for the project.

Since the time the Draft Wastewater Facilities Plan was submitted, the City has also decided to consider the submission of a General Obligation Bond to the Warrenton Voters in 2003 to pay for construction of the treatment plant.

1.12 WASTEWATER RATES

The City of Warrenton recently received recommendations for a new rate methodology for both their water and wastewater systems. To prepare the rate methodology study, the City and their consultant have used approximate cost estimates for system improvements developed to date. The City approved the new rates on March 20, 2002.

The City currently charges only a nominal connection fee without any System Development Charge (SDC) for new connections. The City should actively pursue and take all necessary steps to calculate and implement an appropriate SDC for the proposed sanitary sewer system improvements. An SDC for new sewer improvements will be required in order for new connections to pay their "fair share" of the needed improvements to the sanitary sewer system.

1.13 RECOMMENDED IMPROVEMENTS AND GENERAL SCHEDULE

The City of Warrenton is undertaking an aggressive schedule for implementing the planned wastewater improvements. A general schedule and estimated construction costs of the proposed improvements is shown in the following tables.

RECOMMENDED COLLECTION SYSTEM UPGRADES

Completion Date	Improvement	Estimated Cost
September 2003	Inflow/Infiltration Reduction Work at Airport	Cost Not Available
September 2005	Core Downtown Pump Station Improvements*	$1,123,000.00
By 2007	Main Avenue Sewer	$290,000.00
By 2008	Dolphin Road Sewer	$310,000.00
By 2015	Inflow/Infiltration reduction work throughout City	$675,000.00
By 2015	Conveyance System Upgrades throughout City	$3,800,000.00

* These improvements need to take place at the time of the treatment plant improvements.

RECOMMENDED TREATMENT SYSTEM UPGRADES

Date	Improvement	Estimated Cost
September 2003	Biosolids Removal	$480,000.00
May 2004	Wastewater Treatment Plant	$5,736,000.00
September 2004	Outfall Construction	$1,130,000.00

A detailed break down of the implementation program and finance plan can be found in *Section 9.6*. The tables above summarize the improvements while Section 9.6 identifies milestones for the submittals to DEQ, report preparation, permitting, construction and ultimately full operation of the proposed treatment plant.

This tabular summary concludes the summary and recommendations. Section 2, which is the Introduction, will provide background information on which the report was based. Section 3, Study Area Characteristics, will describe in detail additional area characteristics such as physical, environmental and demographic.

SECTION 2
INTRODUCTION

2.1 PURPOSE

The purpose of this wastewater facilities plan is to provide the City of Warrenton with a comprehensive wastewater utility planning document through the year 2022, and to identify additional work needed to bring the City's wastewater treatment facility into compliance with current and probable changes in the City's National Pollutant Discharge Elimination System (NPDES) permit requirements.

2.2 BACKGROUND

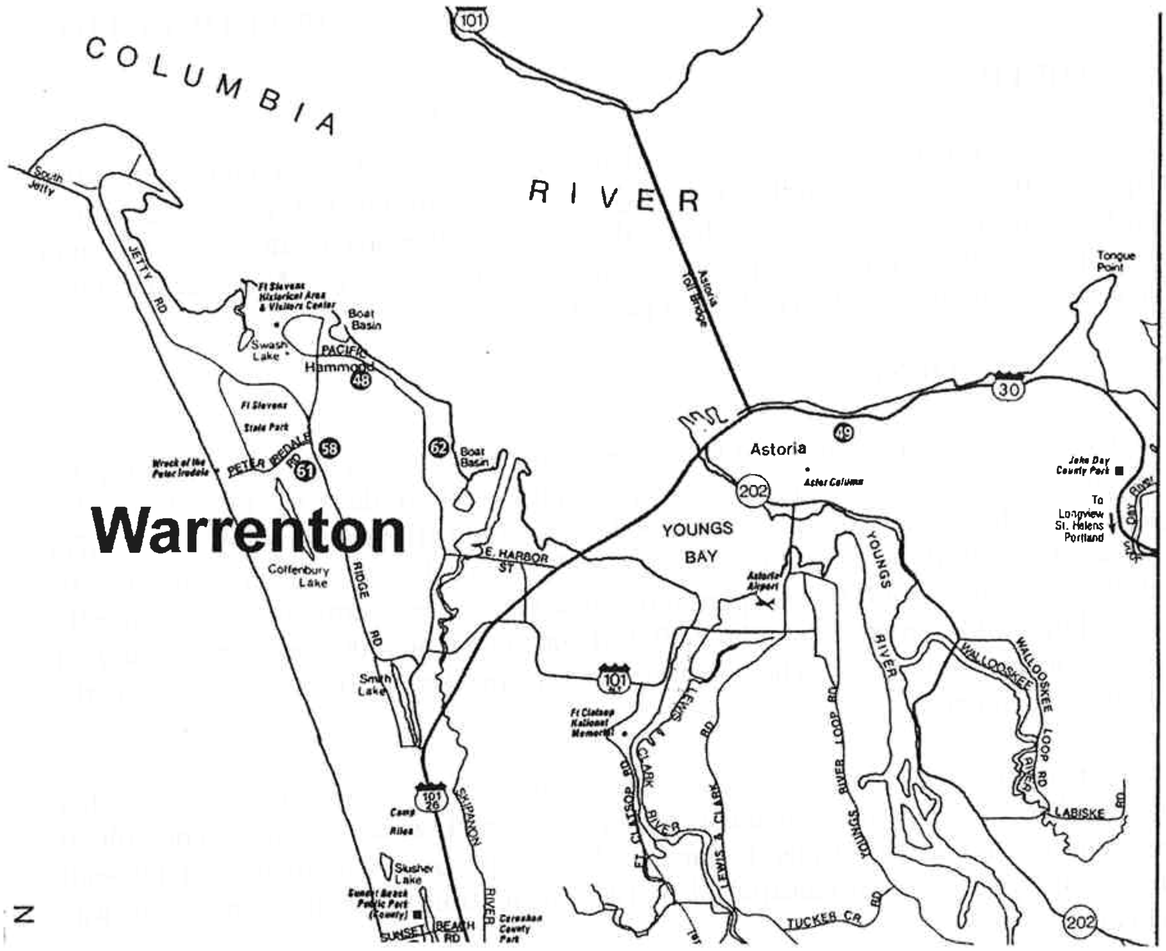
The City of Warrenton is located in the extreme northwest corner of Clatsop County, on the northern Oregon coast; approximately 2 miles west of the City of Astoria, See *Figure 2.1,* below. The City's current (2002) population is approximately 4,100. Also inside the City limits is Fort Stevens State Park, which has an annual attendance of about 902,000 visitors. Of these, approximately 200,000 are overnight campers, mostly present during the summer months. (Based on 2000 and 2001 average from Fort Stevens State Park's records). The total number of campsites at Fort Stevens State Parks is about 600 campsites.

Warrenton owns and operates a wastewater system that was originally constructed during the late 1960's. The system consists of a gravity flow collection system with 26 pump stations and a wastewater treatment plant. The treatment plant is a two-cell stabilization lagoon system constructed in 1969 at the time the collection system was initially completed.

The existing lagoon treatment system has failed to meet discharge permit limits for the mass load and concentration of bio-chemical oxygen demand (BOD) and total suspended solids (TSS) leading to permit violations. On March 29, 1999 and February 9, 2001, the Department of Environmental Quality issued a Notice of Noncompliance to the City for permit violations. The DEQ and the City recognize that because of the overloadings, it is likely that permit violations will continue unless necessary improvements to the City's facilities are made.

The City of Warrenton signed a Mutual Agreement and Order (MAO) with DEQ on December 5, 2001. The MAO outlines interim effluent discharge limits, a schedule for improvements to the City's facilities and penalties for non-compliance. A copy of the MAO can be found in *Appendix H*. The details of the MAO are described in *Section 7.3.2.*

FIGURE 2.1



2.3 SCOPE

The scope of work for this wastewater facilities plan includes the following key elements:

- Population, EDU, and land use considerations for current (2002) and future (design year 2022) conditions.

- Summary, inventory, and description of the existing wastewater collection, treatment, and disposal system.

- Evaluation and determination of current and projected future hydraulic and organic loadings.

- Review I/I removal efforts and address remaining needs including potential collection system and pump station improvements.

- Describe current effluent disposal and develop alternatives for improvements consistent with projected wastewater characteristics, treatment plant capacity, and regulatory requirements.

- Complete a Mixing Zone Study and Temperature Management Plan (TMP) for the existing outfall and an outfall extended to the Columbia River Channel.

- Describe and evaluate the existing wastewater treatment facility. Develop improvement and/or replacement alternatives to address current and projected future needs.

- Describe financing options.

- Prepare a development plan outlining selected improvement options, costs, implementation plan, and potential financing options.

2.4 AUTHORIZATION

In October of 1999 the City of Warrenton contracted with HLB & Associates, Inc. (HLB) to prepare a wastewater facilities plan in order to meet current and projected future planning needs and regulatory requirements for the City's wastewater system. In the interim, HLB has been asked to prepare a Lagoon Aeration Pre-Design Report, an Interim Capacity Technical Memorandum and a Technical Memorandum for sewer lagoon upgrade alternatives.

After submittal of the draft facilities plan in March 2002, the City of Warrenton authorized additional work on the following portions of this final report:

1. Update to the Request for Interim Capacity Increase Technical Memorandum, (*Appendix A*).
2. The Mixing Zone Study and Temperature Management Plant (TMP) to evaluate the current and proposed outfall from the plant, (*Appendix B*).
3. The detailed Wastewater Treatment Facility Upgrade & Expansion Plan that addresses added flows from outside the City's sewer service areas including the Miles Crossing Sanitary Sewer District and Fort Clatsop National Park, (*Appendix C*).

This wastewater facilities plan is being financed with a State Revolving Fund Loan from the Oregon Department of Environmental Quality (DEQ).

2.5 BASIS FOR OPINIONS OF PROBABLE COST

2.5.1 General

Opinions of probable cost presented in this study include four components, each of which is discussed separately in this section. It must be recognized that opinions of probable cost are preliminary and based on the level of planning presented in this study. As specific improvements proceed forward it may be necessary to update the costs as more complete information becomes available.

2.5.2 Construction Cost

Options of probable costs in this plan are based on preliminary layouts of the proposed improvements, actual construction bidding results for similar work, published cost guides, and the author's construction cost experience on the north Oregon coast.

2.5.3 Contingencies

In recognizing that the opinions of probable cost are based on preliminary design, allowances must be made for variations in final quantities, bidding market conditions, adverse construction conditions, unanticipated specialized investigations, and other difficulties that cannot be foreseen at this time. A contingency factor of 10% of the construction cost has typically been added for new facilities or minor upgrades/maintenance and existing facilities. Significant renovation or upgrades of existing structures, such as those proposed in this report, may utilize a higher contingency allowance of 20%.

2.5.4 Engineering and Construction Management Costs

Engineering, construction observation, and construction management costs have been assumed at 25% of the construction cost. This includes costs for the engineering company to conduct preliminary surveys, perform detailed design analyses, prepare construction drawings and technical specifications, advertise for construction bids, conduct construction stakeout surveys, provide partial construction observation during construction, administer construction related activities such as change orders, and to prepare record drawings showing the project as-built.

2.5.5 Opinion of Probable Cost Summary

Opinions of probable costs presented in this study include a combined allowance of 45% for contingencies and engineering. An allowance of 20% was used for construction contingency and an allowance of 25% was used for engineering and permitting costs.

2.6 PREVIOUS STUDIES AND OTHER DOCUMENTS

The following studies were reviewed and/or used in the completion of this report:

H.R. ESVELT ENGINEERING and HLB & ASSOCIATES, INC., *Request for Interim Capacity Increase Technical Memorandum*, draft version, January 4, 2002, final version, August 20, 2002 (*See Appendix A, for final version*)

H.R. ESVELT ENGINEERING and HLB & ASSOCIATES, INC., *Wastewater Treatment Facility Upgrade & Expansion Plan*, September 2002 (*See Appendix C*)

COSMOPOLITAN ENGINEERING GROUP, *Mixing Zone Study*, September 2002 (*See Appendix B*)

HLB & ASSOCIATES, INC., *Performance Evaluation Standards Manual for the Sewage Lagoons Improvement Project,* September 2000

HLB & ASSOCIATES, INC., *Lagoon Aeration Pre-Design Report for City of Warrenton*, January 2000

CH2M Hill, *Technical Memorandum for Sewer Lagoon Improvement Study, City of Warrenton*, June 23, 1995

WESTECH ENGINEERING, INC., *City of Warrenton Sanitary Sewerage System Facilities Planning Report*, April 1983

BST, INC, *Miles Crossing/Jeffers Gardens Wastewater Facilities Plan*, November 1997

JB RANKIN ENGINEERING, INC., *State of Oregon Military Department Feasibility Study of the Camp Rilea Wastewater Treatment System*, April 2000

SECTION 3
STUDY AREA CHARACTERISTICS

3.1 PLANNING AREA

The planning of sewerage facilities requires as a background a basic knowledge of the physical, environmental and demographic characteristics of the service area. The purpose of this section is to review these factors and to present basic data required for this study.

3.1.1 Existing Service Area

The City's service area is quite large relative to its population. This service area is defined by the location of the existing sewer mains (*See Map, Figure 4.1 in Appendix N*). The service area includes all of the land within the City Limits of the City of Warrenton boundary, which now includes the Town of Hammond. The Town of Hammond was incorporated into the City of Warrenton in 1991. The existing sewage collection system currently includes two areas inside of the City Limits that contribute substantial flows to the sewer collection and treatment system. Those inside City Limits service areas include the Fort Stevens State Park and the Port of Astoria Airport. Finally there are some unincorporated areas that are within the Urban Growth Boundary (UGB) near the southeast corner of the County Industrial Park that is served by sanitary sewer service.

Additionally, the City provides water services to the Fort Stevens State Park, Camp Rilea, Shoreline Estates, Sunset Beach and the City of Gearhart. For use in emergencies, the City's water system is interconnected with the City of Seaside water system. Warrenton supplies a substantially greater water service area than the current sanitary sewer service area. There are currently no indications of any potential expansion of sewer service into the areas noted above (Camp Rilea, Shoreline Estates, Sunset Beach and the City of Gearhart) that are now only provided with water service.

3.1.2 Proposed Ultimate Service Area

3.1.2.1 Urban Growth Boundary

The City of Warrenton interacts with several different communities and agencies providing water and sewer services. Because of these multiple planning interchanges, the ultimate planning needs of the City are being influenced by outside organizations. The outside organizations growth must be considered during the planning process. Warrenton, unlike most cities on the North Oregon Coast, is now greater than eleven (11) square miles. This makes Warrenton one of the largest cities by area (on a square mile basis) in the State of Oregon.

The projected sanitary sewer service area, according to the City of Warrenton Planning Department, is to remain within the Urban Growth Boundary (*See Map, Figure 4.1 in Appendix N*) of the City of Warrenton. Within the City's Urban

Growth Boundary, there is an extensive amount of undeveloped land. This land, together with some adjacent unincorporated land, will all have the potential to impact the utility services that the City provides.

Considering the large area of undeveloped land available, the City Planning Department has developed a sound basic concept detailing community development that will encourage appropriate and balanced growth. The aforementioned balanced growth inherently requires the consideration of efficient methods of expanding the City's public facilities and services. The City of Warrenton is committed to the concept that over the next twenty years the land needed for urban development will be made available in phases.

3.1.2.2 Fort Stevens State Park

The draft Master Plan for the Fort Stevens State Park was reviewed as a part of this facilities plan. Fort Stevens State Park has expanded its facilities according to the Park's draft Master Plan. A renovation program is currently underway.

Renovation has included the addition of two (2) new lanes to the existing sewer dump station with sewage dumping and cleanup connections. The toilet and shower facilities have been up-graded. The plans included the redesign of the roadways leading into the campground loops due to safety and access issues. The South campsites (currently 277 sites including yurts and group sites) are in the design phase at this time. There may be a few of the sites eliminated due to the redesign of the roadways and undesirable campsites, however, the number of these eliminated sites is unknown at this time. The renovation of the north campsites (currently 256 sites) is completed. There were a total of 615 campsites before the renovation. The park preliminarily estimated that they would lose 40 +/- campsites due to design changes, but should total around 600 campsites when the renovation project is completed.

The Fort Stevens State Park contributes a significant amount to both the sewer flows and the loadings that arrive at the sewage treatment plant. The sewer flows can be quite large on major weekends during the summer and early fall months. The chemicals that are typically used in an RV holding tank are very difficult to treat. New construction that might result in an increase in sewage flows and sewer effluent loadings from the State Park should be carefully monitored in the future.

3.1.2.3 Clatsop County Industrial Park

The County Industrial Park is also likely to expand its need for additional sewer service over the next twenty years; this will influence the City's planning needs. The City and County consider this industrial area, which is inside the Urban Growth Boundary, as committed for urban development. The basis of this commitment is not only the strong potential for economic impact on the region;

but the area's proximity to existing Airport and Port facilities, together with the accessibility to public services and major roads. A newer sewer pump station near the Oregon Youth Authority serves the Industrial Park with a flow capacity of 950 gallons per minute. This pump station was installed with a large capacity for future growth.

3.1.2.4 Camp Rilea

Camp Rilea occupies approximately 2,000 acres of land south of the City of Warrenton and west of Highway 101. The base is used as a training facility for units of the Oregon Army National Guard, the Air National Guard, United States active duty forces, and civilians (such as the Oregon State Police).

The population of the camp consists of 100 – 110 full-time staff, 100 – 800 inactive duty soldiers two (2) weekends per month, and up to 1,400 soldiers per day during summer annual training. Thus, population can vary from less than 100 to almost 1,500. The estimate yearly average usage is 450 persons per day.

The Oregon Military Department authorized a feasibility study of the Camp Rilea wastewater treatment system. The study, State of Oregon Military Department Feasibility Study of the Camp Rilea Wastewater Treatment System, was conducted by J.B. Rankin Engineering, Inc. dated April 2000. That study examined nine (9) options for dealing with the wastewater treatment system. Six (6) of the options involved some form of connection to the City of Warrenton whether that connection would be either full-time or only during the peak winter flows. Three (3) of the options involved either plant upgrades or the installation of a "package treatment plant".

The study concluded that construction costs only for the various connections to the City of Warrenton would range from \$312,500.00 to \$783,000.00. The construction costs for the plant upgrade/improvement options varied from \$332,000.00 to \$515,000.00. The study concluded that options #7 through #9 (the plant upgrade/improvement options) are feasible and the initial construction costs are slightly less than the first six (6) options (the connection to the City of Warrenton). The annual operating costs would likely be significantly less than connecting to the City of Warrenton. Since April 2000, the Oregon Military Department has been pursuing the plant upgrade options and there have been no further discussions with the City of Warrenton regarding any future connection to the City's sewer system. Therefore, Camp Rilea was not considered further as a future outside growth area for the City of Warrenton's sewer system.

3.1.2.5 Miles Crossing Sewer District

The Miles Crossing Sewer District is a newly created district that was formed to provide sanitary sewer service to the Miles Crossing/Jeffers Gardens community.

This community is located between Astoria and Warrenton, and between the Young's River and the Lewis and Clark River, and along Alternate (Old) Coast Highway 101. The community is composed of single-family and residential rental properties and a variety of commercial uses that serve residents of the area. The population of this area was approximately 650 people in 1997. Growth rate was estimated at 2.2% in the Wastewater Facilities Plan prepared by BST, Inc. and dated November 1997. Those population figures equate to a population of approximately 1,000 in the year 2017. Wastewater flows for the Miles Crossing Sewer District were extrapolated to the year 2023 based upon the wastewater flows presented in the Miles Crossing/Jeffers Gardens Wastewater Facilities Plan dated November 1997.

Wastewater Flows Projected for Miles Crossing

	1997	2001	2013	2017	2023
Avg Annual	.052	.057	.074	.080	.10
MWWMF	.068	.074	.096	.105	.13
MWWDF	.095	.104	.135	.147	.16
Hydraulic, PIF	.109	.119	.154	.168	.25

*Values for the years 1997-2017 are taken from Wastewater Facilities Plan for Miles Crossing/Jeffers Garden. Values for 2023 are extrapolated from the previous data.

Based upon recent discussions over the last six months between the Miles Crossing Sewer District and the City of Warrenton it is reasonable to assume that the Miles Crossing sewer District will connect to the City's sewer system. The City's wastewater facility upgrade and expansion plan as well as the Interim Capacity Increase Technical Memorandum have both been prepared assuming that Miles Crossing will connect to the City's sewer system.

It should be noted that the Miles Crossing Sewer District is also currently considering two (2) other options. Those options include a possible connection to the City of Astoria sewer system and a possible new sewer treatment plant to serve the Miles Crossing Sewer District.

3.1.2.6 Fort Clatsop

Fort Clatsop is a National Park located outside the UGB of Warrenton. It is also located between the Miles Crossing Sewer District and the City of Warrenton. If the Miles Crossing Sewer District decides to connect to the City's sewer system, then it would be a natural result to connect the Fort Clatsop site to the sewage force main from the Miles Crossing area. The Miles Crossing sewage force main would pass directly in front of the Fort Clatsop site.

The Fort Clatsop National Park is estimated to be a very small seasonal use that is equivalent to five (5) ERU's. There are no camping facilities at this day-use only park. This park consists mainly of an interpretive center in addition to the historic recreation of the original fort.

3.2 PHYSICAL ENVIRONMENT

3.2.1 Geography/Topography

The City of Warrenton is situated south of the Columbia River, west of Youngs Bay and east of the Pacific Ocean. Along the westerly side of the City, including Fort Stevens State Park (within the City Limits) and west to the Pacific Ocean, is a series of sandy dune formations that generally parallel the ocean shore line and are separated by low-lying interdune areas. A considerable amount of the area has been stabilized by implementing a barrier dune (foredune), and planting Ammophila arenaria (European beach grass), Cytisus scoparius (scotch broom) and by other means. These sand dunes can be as high as 40 feet National Geodetic Vertical Datum (NGVD).

The remaining area easterly to Youngs Bay is generally flat with some areas near the Columbia River and Youngs Bay being tidally influenced. This area is roughly between 2 feet and 25 feet, NGVD.

The City encourages development techniques that maintain the natural topography. Some of these techniques may include controlled grading and excavation, providing appropriate drainage solutions; reducing slope related problems and limiting the changes to the natural features of proposed development.

There are generally no constraints to development within the Warrenton Urban Growth Boundary (UGB) imposed solely by geography or topography.

3.2.2 Soils and Geology

The northwest coastline of Oregon is underlain with bedrock that is of low permeability, being a fine grain sandstone and shale (Astoria formation). These rocks can be seen along the edge of the Coast Range east of the Clatsop Plains area. This bedrock underlies the Clatsop Plains sand dune deposits along the shoreline at depths of over 100 feet. The overlying coastal dunal formation contains loose and unconsolidated fine to medium sand. The sand likely contains interbedded layers of ocean deposits and wind-blown sand material. At or near the surface the sand is wind-deposited material. Some discontinuous silty layers exist within the sand deposits of the dunal formations.

There are generally no constraints to development within the Warrenton UGB imposed solely by the geology of the area.

3.2.3 Climate

The North Oregon Coast has a climate of mild winters and cool summers, largely due to the moderating influence of the Pacific Ocean. The monthly average temperatures in this region vary from 40° Fahrenheit in January to 60° Fahrenheit in July. Westerly winds from the Pacific Ocean predominate over the coastal plain that includes Warrenton. Winter winds are from the Southwest and summer winds are from the Northwest.

The daily, monthly and annual rainfall data from the Astoria airport was used for the rainfall data for the City of Warrenton. See *Table 3.1* below, summarizes the monthly rainfall data for Warrenton for the years 1953 through 2001. The annual average rainfall is approximately 70 inches of rain per year. Approximately 68 percent of the total yearly precipitation is distributed through the months of November through March. The calculations for the 80th percentile of rainfall in January and the 90th percentile of rainfall in May are presented in *Table 3.2* below.

Table 3.1 - Monthly Rainfall Data for Astoria Airport, Warrenton, 1953 through 2001

Statistical summaries appear in Table 3.2

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
1953	n/a	5.32	6.43	2.82	4.11	2.9	0.65	3.12	3.9	4.3	12.82	12.21	n/a
1954	18.94	9.56	4.17	4.7	1.66	5.48	1.77	2.24	2.01	4.44	10.1	10.22	75.29
1955	5.96	6.33	8.64	8.04	1.62	2.73	3.42	0.1	3.57	12.24	14.64	16.57	83.86
1956	17.09	9.32	13.47	1.33	1.43	4.64	0.18	2.15	3.76	11.37	2.57	9.02	76.33
1957	4.76	6.9	9.73	3.94	2.82	3.31	1.63	1.34	0.82	5.43	7.68	11.97	60.33
1958	9.61	10.96	4.62	7.03	1.03	2.8	0.09	0.52	1.94	7.33	14.14	12.17	72.24
1959	13.24	8.04	7.88	4.4	3.45	3.77	0.91	0.92	5.56	6.48	11.4	8.36	74.41
1960	10.09	8.47	7.4	5.92	6.6	1.87	0.01	1.84	1.69	7.33	13.91	6.12	71.25
1961	9.03	21.89	10.69	5.47	2.9	1.1	0.5	1.3	1.45	7.32	8.34	10.4	80.39
1962	6.53	5.61	5.18	7.44	2.88	1.87	0.34	2.49	3.5	7.4	14.21	6.78	64.23
1963	4.76	6.44	6.13	5.76	1.91	1.8	1.52	1.2	2.2	9.58	13.16	9.12	63.58
1964	18.5	4.06	7.41	3.59	2.27	2.7	2.59	2.21	2.73	2.61	11.15	13.67	73.49
1965	16.59	6.77	0.93	5.47	2.74	0.75	0.46	1.95	0.51	3.97	11.82	11.78	63.74
1966	8.61	5.53	8.79	2.9	2.18	2.13	0.54	1.01	2.18	5.83	10	14.07	63.77
1967	14.95	6.07	8.38	5.52	1.37	1.14	0.22	0.19	3.07	11.06	5.94	9.04	66.95
1968	9.57	9.57	10.42	4.22	3.91	4.81	1.23	5.22	4.6	8.03	11.96	13.85	87.39
1969	12.02	5.67	3.16	3.84	3.92	3.63	0.56	0.62	6.55	5.28	5.77	11.69	62.71
1970	14.46	5.29	4.28	7.74	1.92	1.19	0.31	0.08	3.65	5.8	9.86	15.93	70.51
1971	16.69	6.67	9.96	4.09	2.3	2.97	1.55	1.14	4.65	6.34	9.08	13.83	79.27
1972	10.62	8.58	10.04	6.82	1.22	0.92	2.01	0.37	4.72	1.96	6.9	13.28	67.44
1973	5.72	2.6	5.71	2.38	3.16	4.26	0.07	0.46	4.19	5.92	14.93	15.75	65.15
1974	12.47	8.38	10.73	4.88	4.37	2.33	4.2	0.29	0.67	1.85	8.95	13.84	72.96
1975	15.21	8.03	5.66	3.9	2.41	1.99	0.22	2.82	0.04	12.56	12.28	15.66	80.78
1976	11.67	7.86	7.17	3.55	2.2	1.27	2.46	2.55	1.58	2.96	1.45	4.2	48.92
1977	3.2	5.22	9.74	1.65	6	1.36	0.44	3.85	5.44	4.38	12.37	14.34	67.99
1978	8.66	5.43	4.4	6.35	4.75	3.07	0.9	2.61	6.93	1.01	8.43	4.99	57.53
1979	3.83	11.76	4.52	4.38	4.19	1.82	0.92	0.81	4.35	8.46	7.87	13.18	66.09
1980	7.21	9.6	6.31	4.85	1.45	1.57	0.64	1.24	2.51	2.79	12.02	12.44	62.63
1981	2.63	8.69	5.8	7.3	2.97	5.47	1.06	0.62	2.77	8.67	10.66	11.8	68.44
1982	13.98	10.87	7.19	6.52	0.37	1.22	0.75	0.63	3.72	8.31	9.62	12.14	75.32
1983	13.52	8.66	8.84	4.26	3.59	4.53	4.39	1.14	1.83	1.87	16.75	9.44	78.82
1984	6.6	8.34	5.9	5.02	5.34	3.9	0.05	0.52	3.16	8.1	15.19	6.51	68.63
1985	0.69	4.09	7	2.95	1.9	3.09	0.78	1.11	3.23	8.11	5.96	2.67	41.58
1986	11.19	8.93	6.11	3.58	3.3	0.94	1.69	0.14	3.62	5.45	11.42	7.34	63.71
1987	10.38	5.08	8.52	3.02	3.97	0.65	1.1	0.16	0.95	0.52	4.33	8.85	47.53
1988	6.57	3.6	7.86	3.99	4.09	3.5	0.96	0.88	1.23	2.14	13.06	7.32	55.2
1989	8.2	6.61	10.09	2.27	3.01	2.58	1.64	0.84	0.5	5.3	6.73	7.4	55.17
1990	16.09	11.83	5.15	4.44	4	3.47	0.54	1.57	0.67	8.44	11.28	5.11	72.59
1991	6.76	8.57	5.65	9.47	2.68	1.86	0.33	2.31	0.07	2.44	10.53	6.6	57.27
1992	9.34	5.69	1.19	7.49	0.52	0.55	0.24	0.77	2.66	4.1	10.11	5.99	48.65
1993	6.27	1.35	n/a	n/a	n/a	3.7	1.81	0.57	0.12	2.25	6.68	9.63	n/a
1994	6.83	11.34	6.48	4.31	2.52	2.27	0.81	1.49	2.84	9.52	12.56	14.84	75.81
1995	10.59	5.94	8.76	5.8	2.14	2.63	0.64	1.95	2.19	7.13	17.47	11.26	76.5
1996	9.07	14.52	4.7	10.07	3.96	1.38	1.92	0.71	3.34	11.14	11.51	20.38	92.7
1997	12.74	3.95	15.31	6.62	3.61	4.53	1.35	2.9	7.27	11.56	7.65	7.99	85.48
1998	16.2	10.52	10.23	2.49	3.75	1.67	0.33	0.25	0.66	6.86	19.6	16.59	89.15
1999	13.87	18.26	9.53	2.59	5.61	3.43	0.78	0.31	0.27	3.64	15.88	12.86	87.03
2000	11.67	5.05	5.46	3.71	4.14	4.16	0.261	0.61	2.18	4.62	3.77	5.81	51.44101
2001	4.80	3.48	5.21	5.63	3.14	2.84	0.85	3.69	0.95	4.25	14.21	11.83	60.88
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
Average													
Avg. Rainfall, in.	10.17	7.78	7.23	4.89	3.03	2.62	1.07	1.38	2.71	6.09	10.59	10.75	68.75
Minimum, in/mo.	0.69	1.35	0.93	1.33	0.37	0.55	0.01	0.08	0.04	0.52	1.45	2.67	41.58
Maximum, in/mo.	18.94	21.89	15.31	10.07	6.60	5.48	4.39	5.22	7.27	12.56	19.60	20.38	92.70
80%	14.27	9.58	9.74	6.58	4.05	3.73	1.66	2.27	4.02	8.45	14.00	13.84	78.36
90%	16.32	11.42	10.29	7.46	4.48	4.53	2.10	2.84	4.86	11.08	14.98	15.68	84.51

Table 3.2 - Statistical Summary of Monthly Rainfall Data for Astoria Airport, Warrenton, 1953 through 2001

January

Year	Rainfall	Rank	Percent
1954	18.94	1	100.0%
1964	18.5	2	97.8%
1956	17.09	3	95.7%
1971	16.69	4	93.6%
1965	16.59	5	91.4%
1998	16.2	6	89.3%
1990	16.09	7	87.2%
1975	15.21	8	85.1%
1967	14.95	9	82.9%
1970	14.46	10	80.8%
1982	13.98	11	78.7%
1999	13.87	12	76.5%
1983	13.52	13	74.4%
1959	13.24	14	72.3%
1997	12.74	15	70.2%
1974	12.47	16	68.0%
1969	12.02	17	65.9%
2000	11.67	18	61.7%
1976	11.67	19	61.7%
1986	11.19	20	59.5%
1972	10.62	21	57.4%
1995	10.59	22	55.3%
1987	10.38	23	53.1%
1960	10.09	24	51.0%
1958	9.61	25	48.9%
1968	9.57	26	46.8%
1992	9.34	27	44.6%
1996	9.07	28	42.5%
1961	9.03	29	40.4%
1978	8.66	30	38.2%
1966	8.61	31	36.1%
1989	8.2	32	34.0%
1980	7.21	33	31.9%
1994	6.83	34	29.7%
1991	6.76	35	27.6%
1984	6.6	36	25.5%
1988	6.57	37	23.4%
1962	6.53	38	21.2%
1993	6.27	39	19.1%
1955	5.96	40	17.0%
1973	5.72	41	14.8%
2001	4.8	41	12.7%
1957	4.76	43	8.5%
1963	4.76	44	8.5%
1979	3.83	45	6.3%
1977	3.2	46	4.2%
1981	2.63	47	2.1%
1985	0.69	48	0.0%

For January 14.27 inches = 80.0%
with 48 16.32 inches = 90.0%
years of records

May

Year	Rainfall	Rank	Percent
1960	6.6	1	100.0%
1977	6	2	97.9%
1999	5.61	3	95.7%
1984	5.34	4	93.6%
1978	4.75	5	91.5%
1974	4.37	6	89.4%
1979	4.19	7	87.2%
2000	4.14	8	85.1%
1953	4.11	9	83.0%
1988	4.09	10	80.9%
1990	4	11	78.7%
1987	3.97	12	76.6%
1996	3.96	13	74.5%
1969	3.92	14	72.3%
1968	3.91	15	70.2%
1998	3.75	16	68.1%
1997	3.61	17	66.0%
1983	3.59	18	63.8%
1959	3.45	19	61.7%
1986	3.3	20	59.6%
1973	3.16	21	57.4%
1999	3.14	22	55.3%
1989	3.01	23	53.2%
1981	2.97	24	51.1%
1961	2.9	25	48.9%
1962	2.88	26	46.8%
1957	2.82	27	44.7%
1965	2.74	28	42.6%
1991	2.68	29	40.4%
1994	2.52	30	38.3%
1975	2.41	31	36.2%
1971	2.3	32	34.0%
1964	2.27	33	31.9%
1976	2.2	34	29.8%
1966	2.18	35	27.7%
1995	2.14	36	25.5%
1970	1.92	37	23.4%
1963	1.91	38	21.3%
1985	1.9	39	19.1%
1954	1.66	40	17.0%
1955	1.62	41	14.9%
1980	1.45	42	12.8%
1956	1.43	43	10.6%
1967	1.37	44	8.5%
1972	1.22	45	6.4%
1958	1.03	46	4.3%
1992	0.52	47	2.1%
1982	0.37	48	0.0%

For May 4.05 inches = 80.0%
with 48 4.48 inches = 90.0%
years of records

3.2.4 Water Resources

The Skipanon River flows through Warrenton from South to North and discharges into the Columbia River. The Skipanon River generally has dikes on each side to prevent flooding of the Warrenton downtown area and provides general area drainage for downtown Warrenton. The Skipanon River is the outlet for Cullaby Lake located several miles to the South. Two additional creeks provide general drainage to the northwesterly part of Warrenton, being Alder Creek and Tansy Creek. Tansy Creek is a tributary of Alder Creek and both drain into Alder Cove, which is a part of the Columbia River Basin.

Local use of waters and wetlands include pleasure boating, commercial fishing as well as sport fishing, hunting, and boat moorage. Physical modifications has involved the construction of pile dikes, moorage facilities, flood control structures, bridges, causeways and erosion control facilities. Some areas, such as many of the local lakes and the Skipanon River experience extensive human use, while other areas including Alder Cove receive minimal human usage.

The City of Warrenton is keenly aware of the unique environmental, economic and social value of the Columbia River waters, together with the surrounding wetlands and shorelands. The City has participated in a bi-state voluntary planning organization known as the Columbia River Estuary Task Force (CREST). The Warrenton area is part of the Youngs Bay – Astoria estuary planning area. This estuary planning effort has identified land use designations within the estuary. Some of the most notable designations are "natural" and "conservation" areas, and "aquatic" and "water dependent Industrial development." According to CREST the Columbia River, in general, has suffered damage largely due to human activities over the last 100 years. The Lower Columbia Bi-State Water Quality Program has collected a substantial amount of data on the lower river and has identified four types of water quality problems and they are as follows:

- **Toxics:** toxics have been found in sediment and fish tissue. Levels of PCB's, DDE and dioxin are high enough that they may be linked to reproductive failure in bald eagles, mink and river otter. They may also pose a threat to human health.

- **Water Quality:** Point source and non-point source pollution have affected the water quality itself. PH, temperature and dissolved oxygen are altered and in turn alter the ecological balance.

- **Habitat:** Activities of the last 100 years have significantly altered the estuary and resulted in habitat loss or modification. Dams, dikes, maintenance dredging, and land use, agricultural and forest practices all contribute to this alteration.

- **Species:** Anadromous fish runs have declined significantly in recent years. Several species are now listed as endangered or threatened.

The Department of Environmental Quality has acknowledged the aforementioned land use designations and established suitable water quality standards to preserve such uses. The water quality standards are set forth by Oregon Administration Rules (OAR) in 340-41-205.

In general, the Columbia River water quality is good and is being used as a drinking water source by several communities. According to Oregon and Washington water withdrawal permits, over 95 percent of water withdrawals for human consumption are from wells. In Oregon, the City of Rainier uses the Columbia River water as a seasonal water supply and the City of St. Helens uses river water as the primary water source year round. Youngs Bay water quality in general ranges from very good to relatively poor during low flow periods in portions of the Skipanon River. Salinities and water movements vary with the volume of fresh water flow, tides and other factors. Sedimentation has occurred in Youngs Bay and erosion has taken place in the vicinity of Tansy Point as well as other areas. Water quality within the Warrenton area has consistently met OAR standards and no significant problems have been reported with the exception of effluent from the Pacific Seafood Surimi Plant that discharges into the Skipanon River during the summer processing season.

3.2.5 Environmental Factors

Natural features in Warrenton and nearby areas are important to the City's future. These natural features provide a variety of development opportunities and constraints for the City. A viable potential for commercial, residential and recreational expansion exists reflecting the City's industrial growth prospects, and the scenic and recreational attractions in the area.

The City's Comprehensive Plan has adopted policies that are designed to preserve the area's numerous natural resources, while allowing for the development necessities of the present and future.

A considerable portion of Warrenton was once part of a large forested tidal swamp that is now protected by flood control dikes. A substantial amount of the former forested tidal marsh is now developed for commercial, residential and industrial uses. Alder Cove and Youngs Bay are very important ecological locations with very substantial wetland areas that include a significant amount of tidal marsh. Alder Cove and Youngs Bay, including the surrounding and adjacent wetland areas, can be characterized by high biological productivity with extensive use by waterfowl, salmon among other fish and bottom-dwelling crustaceans, such as crab and shellfish.

Consideration of the general elevation, wetlands, soils and groundwater conditions impose significant limitations on the potential use of on-site sewage disposal systems. These limitations will likely require the extension of the public sewers to many of the proposed land development sites. Development within areas having highly compressible soils, such as Brallier, Bergsvik or in some cases the Coquille variant and Coquille-

Clatsop complex may require special construction techniques. These techniques may include the use of corrosion resistant materials, piling, trench foundation stabilization and surcharging the site.

Construction within jurisdictional wetlands requires an Oregon Division of State Lands and US Army Corps of Engineers Joint Fill Permit. Mitigation of wetland losses through restoration, creation or enhancement is required. If on-site mitigation is not possible, the option of off-site mitigation becomes available to protect wetlands within the same watershed and in some cases protecting other watershed resources. Also the option of "Payment to Provide Protection", also known as "Payment in Lieu", may be sought in certain limited cases. The presence of large amounts of wetlands within the Warrenton service area will substantially reduce the development area due to wetlands requirements.

3.2.6 Flora and Fauna

The wetlands, shorelines and tidal marshes of the area provide a variety of wildlife habitats and plant communities. According to the Warrenton Wetland Conservation Plan Inventory, the adjacent land to the northwest of the sewage lagoons is rated medium low as a functional wildlife habitat. The following is a short list of some of the wildlife and plant species that are known to inhabit the wetlands, shorelines and tidal marshes in the area.

Mammals

Beaver	Elk
Raccoon	Pine Squirrel (Chickaree)
Otter	Chipmunk (Townsends)
Muskrats	Moles
Rabbit	Shrews
Weasels	Black Bear
Meadow Mice	Coyote
Ground Squirrel	Black-Tailed Deer

Birds

Cooper Hawk	Quail
Sparrow Hawk	Snowy Plover
Marsh Hawk	Killdeer
Bittern	Green Heron
Meadow Larks	Horned Owl
Sparrow	Snowy Owl
Pheasant	Screech Owl
Great Blue Heron	Red-Tailed Hawk
Oregon Junco	

Waterfowl

Mallard Wood Duck Canadian Goose American Widgeon European Widgeon Green-Winged Teal	Blue-Winged Teal Gadwall Scoters Shoveler Common Merganser

Trees

Cottonwood (Populus balsamifera) Western Hemlock (Tsuga heterophylla) Indian Plum (Oemleria cerasiformis) Pacific Crabapple (Malus fusca)	Red Alder (Alnus rubra) Sitka Spruce (Picea sitchensis) Cascara (Rhamnus purshiana) Shore Pine (Pinus contorta)

Shrubs

Pacific Nine Bark (Physocarpus capitatus) Dougals, spiraea (Spiraea douglasii) Willow (Salix spp.) Blackberry (Rubus spp.) Huckleberry (Vaccinium spp.)	Salal (Gaultheria shallon) Salmonberry (Rubus spectabillis) Scotch Broom (Cytisus scoparius) Red Elderberry (Sambucus racemosa) Twin Berry (Lonciera involucrate)

Herbs

Rushes (Juncus spp.) Sedges (Carex spp.) Dunegrass (Elymus mollis) Lady Fern (Athyrium filix-femina) Sword Fern (Polystichum munitum) Water Parsley (Oenanthe sarmentosa) Reed Canary Grass (Phalaris arundinacea)	Pacific Silverweed (Potentilla pacifica) Common Eel-Grass (Zostera marina) Ditch-Grass (Ruppia maritima) Skunk Cabbage (Lysichiton americanum) Horsetail (Equisetum arvense) European Beachgrass (Ammophila arenaria)

The United States Department of Interior, Fish and Wildlife Service (USFWS) maintains and periodically updates a list of endangered and threatened species. Listed species and their respective habitats receive particular consideration as designated by the federal Endangered Species Act of 1973, as amended. An endangered species is defined as any species or subspecies, which is in danger of extinction throughout all or a significant portion of its range. A threatened species is any species or subspecies, which is likely to become an endangered species within the foreseeable future throughout all or a significant portion of its range.

The Columbian White-tailed deer (Odocoileus virginianus leucurus) and the Oregon Silverspot Butterfly (Speyeria zerene Hippolyta) are listed as an endangered species. The Northern Bald Eagle (Haliaeetus leucoceghalus) and the Western Snowy Plover (charadrius alexandrinus nivosus) are listed as a threatened wildlife species. Although these wildlife species may occur in the study area, the Warrenton sewage lagoons and the adjacent area have not been identified as habitat for these wildlife species.

3.2.7 Public Health Hazards

The significant health hazards in Warrenton are related to raw wastewater overflows that can occur at surcharged and overloaded sewer pump stations. The most significant are the pump station at 3rd and Main Court and the discharge manhole from the East Harbor and Ensign pump station. The pump station at 3rd and Main Court is located in the downtown, urban core area of Warrenton. During heavy rainfall, this pump station can overflow into a local drainage ditch that then drains into Alder Creek drainage. The discharge manhole from the East Harbor and Ensign pump station is located at SE First Street and SE Anchor in downtown Warrenton and can overflow into a local drainage ditch that then drains into the Skipanon River.

Heavy flows in the collection system occur during periods of intense rainfall. Pump stations (PS) and manholes (MH's) can then surcharge, but generally do not overflow. Sometimes heavy storm events are coupled with the loss of power on the Pacific Power system. It has been the experience of the City sewer crews that both conditions (intense rainfall <u>and</u> loss of power) must both exist simultaneously in order to lead to an actual sewer overflow. Known overflows have occurred last in November 1998. Sewer crews are generally prepared to operate sewer pump trucks when heavy rainfall occurs in order to avoid raw sewage overflows.

The Warrenton Sewer Treatment Lagoons effluent discharge flows into a manmade drainage ditch on the West side of the lagoons that then drains into Alder Cove. The outfall is protected from tidal influences of Alder Cove by a tide gate. Any violation of the waste permit discharge limitations for the sewer lagoons would result in a potential public health hazard at Alder Cove; although those permit violations would likely not be detectable once mixed in the Columbia River due to the high volume of flow in the Columbia River.

3.3 SOCIOECONOMIC ENVIRONMENT

3.3.1 General Economic Conditions and Trends

The sewage flows that are collected by the wastewater system within the study area are dependent upon the overall, long-term population growth, economic growth of the area and transient seasonal populations that result from the seasonal nature of the area's recreational facilities. The rate of growth of these statistics will affect the timing of the proposed improvements to the wastewater system.

The economy of the area is subject to occasional downturns due to the limited diversity in the economy. For this reason, growth projections need to be tempered by the realities of the local, statewide and national economy. Some economists consider the recreation and tourism industries in this area to be funded by the discretionary spending habits of our statewide population. Such spending can fluctuate. Since recreation and tourism are such large factors in the area, these segments of the economy can be subject to sudden downturns.

3.3.2 Population

The general population of Warrenton is composed of full-time residential, retirement residential and transient seasonal residential components. The tourism, recreational, logging and fishing industries all supported by residents of the Warrenton area. The transient population increase due to tourism and recreation is significant during the summer season. The fishing season covers the summer and early fall season. A good fishing season will significantly increase the transient populations, particularly on weekends.

Industrial growth in the Warrenton area is deemed to be small, yet can be sporadic in nature. The industrial/heavy commercial uses include a lumber mill and fish food processing plant in the Northwest portion of Warrenton together with the Port of Astoria Airport and the relatively new Clatsop County Industrial Park in southeast Warrenton. There is no significant growth in the commercial/heavy commercial sector of Warrenton.

In recent years of the 1990's, Warrenton developed as a regional shopping center with the construction of two (2) large regional shopping stores (Fred Meyer and Costco). There is additional land available for additional large regional shopping centers similar to the type that were developed in Warrenton in the 1990's.

3.3.3 Population Growth Projections

Population data for the years 1970 through 2000 are shown on *Table 3.3* below. These data were obtained either directly from the City Planner or from the Portland State University (PSU) Internet site for population data. Data from regular census years are US Census data while intermediate data are estimates provided by PSU. The population projection shown on the enclosed *Figure 3.2* below shows both a straight-line growth rate and a compound growth rate.

We compared the straight-line growth of 113 persons per year with a compound growth rate of 3.2% by calculating the average annual rate of change from the estimated population data for 1990 to 1998. Population analysis and projection data is presented in *Table 3.3* below and in *Figure 3.2* below.

The variations in the actual population data are recognizable in the economic downturns that have historically affected the area in the 1960's and in the 1980's. Please refer to *Figure 3.2* below. The average growth rate for the decades of 1970, 1980 and 1990 have been calculated and are also shown in *Table 3.3*, below.

The population of the Warrenton area was analyzed for the time period of 1970 through 1999 by both year and by decades. In general, this analysis showed that the population is dependent upon economic conditions on the North Oregon Coast. The economic downturn of the 1980's is reflected in the population growth of that decade as shown below. There was a decline in the population in the years 1981 and 1982 and the overall population did not regain the 1980 level until 1985. Conversely, the economic growth seen across the nation in the 1990's is reflected in the population growth in Warrenton in that same decade. The recent downturn in the economy has resulted in a decrease to the population for the year 2000.

Figure 3.2 Projected Warrenton Population Growth

Residential Population 1950 - 2022

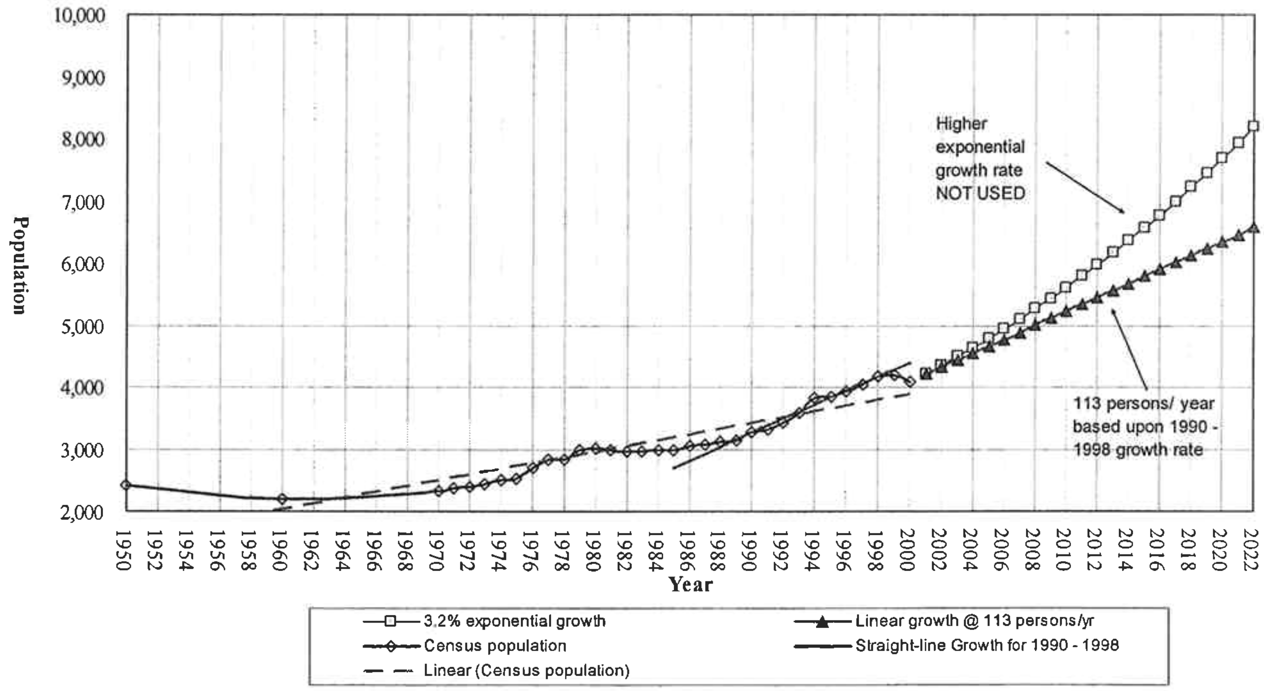


Table 3.3 - Population Analysis for Warrenton

Summary: Compound growth rate 3.2% per year
Average growth per year, for 1990 - 1998 is **113** persons per year
Projections based on =>> **113** persons per year

Year	**Actual Warrenton plus Hammond**	Hammond, population prior to annexation	Actual Warrenton Population values	Combined Population at calc'd Growth for this decade	Percent Growth Rate per Year	Combined Population Increase / Decrease
1950	2415	520	1895			
1960	2195	480	1715			-220
1970	2325	500	1825	Calculated		130
1971	2370	520	1850	2391	1.90%	45
1972	2395	520	1875	2458	1.04%	25
1973	2430	540	1890	2528	1.44%	35
1974	2510	560	1950	2600	3.19%	80
1975	2535	530	2005	2673	0.99%	25
1976	2695	545	2150	2749	5.94%	160
1977	2845	545	2300	2827	5.27%	150
1978	2845	515	2330	2907	0.00%	0
1979	2990	515	2475	2989	4.85%	145
1980	3009	516	2493	3004	0.63%	19
1981	2995	505	2490	3019	-0.47%	-14
1982	2970	510	2460	3035	-0.84%	-25
1983	2970	500	2470	3050	0.00%	0
1984	2980	520	2460	3066	0.34%	10
1985	3000	525	2475	3081	0.67%	20
1986	3060	550	2510	3097	1.96%	60
1987	3085	560	2525	3113	0.81%	25
1988	3120	585	2535	3129	1.12%	35
1989	3145	600	2545	3145	0.79%	25
1990	3270	589	3270	3245	3.82%	125
1991	3325	610	3325	3349	1.65%	55
1992	3420		3420	3456	2.78%	95
1993	3575		3575	3567	4.34%	155
1994	3820		3820	3681	6.41%	245
1995	3845		3845	3799	0.65%	25
1996	3940		3940	3921	3.05%	95
1997	4040		4040	4046	4.83%	100
1998	4175		4175	4176	3.23%	135
1999	4205		4205	4309	0.71%	30
2000	4096		4096	4447	-2.66%	-109

Annual Per Capita Growth for 1990 through 1998

Calculated for this Decade	32%	Compound Growth Rate/Year for 1990 through 1998
113 Average/ year, linear value		

Notes: Ignore the year of 1999 due to low numbers, not similar to 1990 - 1998.
Ignore the year of 2000 due to negative growth, not similar to 1990 - 1998.
However, start with actual population value for year 2000 and project at the higher growth rate calcualted from most recent decade, 1990 - 1998.

The growth rate for this study for the next 20 years is based upon the annual growth of 113 persons per year for the years 1990-1998. The growth for the years 1999 and 2000 were disregarded for the average growth rate, however, the population values for those years were used as a starting point for the population projections. See *Table 3.4* below.

TABLE 3.4 - ACTUAL WARRENTON AREA POPULATION GROWTH

Decade	Starting Population	Annual Growth, persons/year	Annual Percentage Growth
1970 – 1979	2325	68	2.83%
1980 – 1989	3010	26	0.51%
1990 – 1998	3270	113	3.2%

New growth in the RV parks in Warrenton is expected to occur. Interviews with the park operators indicated the following levels of services provided by the major RV park operators in Warrenton. The information on current RV park levels of accommodations is presented in the following *Table 3.4A*:

Table 3.4A	**RV Park Accommodations Provided in 2001 (approximate)**			
RV Park Name	**Sewered RV Sites**	**Non-sewered Cabins & Campsites**	**Restrooms**	**RV Dump Stations**
Ft. Stevens State Park	564	0	15	2
KOA Kampground	145	50	4	1
Hammond Marina RV Park	50	0	2 (w/ laundry)	0
Kampers West	139	25	2	1
Total in 2001	**898**	**75**	**23**	**4**

Based upon our interviews with the park operators and their general indications regarding anticipated growth for each park, the following information was projected for long-term growth patterns, although no timelines are associated with this projection. The information on projected RV park levels of accommodations is presented in the following *Table 3.4B*:

Table 3.4B	Projected Total RV Park Accommodations (approximate)			
RV Park Name	**Sewered RV Sites**	**Non-sewered Cabins & Campsites**	**Restrooms**	**RV Dump Stations**
Ft. Stevens State Park (approx. 20% growth, based upon Master Plan)	676	0	18	2
KOA Kampground (approx. 100% growth, based upon available area)	290	100	8	1
Hammond Marina RV Park (approx. 5% growth)	53	0	2 (w/ laundry)	0
Kampers West (approx. 5% growth)	146	26	2	1
Total in future % increase from 2001	**1165** +30%	**126** +68%	**30** +30%	**4** +0%

3.4 LAND USE REGULATIONS

Land use within the Warrenton area is described in the City's Comprehensive Plan and the Warrenton Zoning Ordinance. The Zoning includes Residential zones [varying from High Density Residential (R-H) to Rural Development (RD) to], Commercial and Industrial zones [such as General Commercial (C-1), Marine Commercial (C-2), Recreational Commercial (RC), General Industrial (I-1) and Water-Dependent Industrial Shorelands (I-2)]. Finally there are Natural and Aquatic zones [Aquatic Development (A-1), Aquatic Conservation (A-2), Aquatic Natural (A-3) and Coastal Lake and Freshwater Wetlands (A-5)]. *Figure 3.3 (See Appendix N)* shows the approximate layout of the zone boundaries within the Warrenton Urban Growth Boundary (UGB). The following *Tables 3.5 and 3.6* provide a summary of the zoning for non-residential and residential land uses.

TABLE 3.5 -NON-RESIDENTIAL LAND USES WITHIN UGB

ZONING CLASSIFICATION	AREA (Acres)	AREA (Sq. Miles)	AREA available for develop-ment, %	AREA available for develop-ment, acres
Aquatic Development (A-1)	268	0.4	0%	0
Aquatic Conservation (A-2)	100	0.2	0%	0
Aquatic Natural (A-3)	216	0.3	0%	0
Coastal Lakes & Freshwater Wetlands (A-5)	1,450	2.3	12%	167
SUBTOTAL OF AQUATIC AREAS	2,034	3.2		167
General Commercial (C-1)	374	0.6	42%	156
Marine Commercial (C-2)	36	0.1	21%	8
Tourist Commercial (C-3)	62	0.1	9%	6
East bank Skipanon Mediated Development Zone (EB)	355	0.6	0%	0
General Industrial (I-1)	1,364	2.1	41%	564
Water Dependent Development (I-2)	277	0.4	61%	168
Water Dependent Industrial (I-3)	9	0.0	31%	3
Light Industrial (LI)	5	0.0	18%	1
Marine Industrial (MI)	20	0.0	18%	4
Recreational – Commercial (RC)	43	0.1	36%	15
SUBTOTAL OF COMMERCIAL/INDUSTRIAL AREAS	2,545	4.0		925
Recreational – Open Space (RO)	106	0.2	8%	8
Shorelands Conservation (SC)	2	0.0	8%	0
Open Space & Institutional (OSI)	0	0.0	0%	0
SUBTOTAL OF OPEN SPACE AREAS	108	0.2		8
Total Non-Residential Lands	**4,687**	**7.4**		**1,100**

TABLE 3.6 - RESIDENTIAL LAND USES WITHIN UGB

ZONING CLASSIFICATION	AREA (Acres)	AREA (Sq. Miles)	AREA available for develop-ment, %	AREA available for develop-ment, acres
High Density Residential (RH)	177	0.3	62%	109
Medium Density Residential (RM)	615	1.0	59%	364
Intermediate Density Residential (R-10)	658	1.0	64%	423
Intermediate Density, Growth Management (R10 GM)	977	1.5	62%	608
Low Density Residential (R-40)	444	0.7	40%	179
Rural Development (RD)	1,472	2.3	48%	707
Total Residential Lands	**4,342**	**6.8**		**2,391**

Note: Areas are calculated from City of Warrenton 1992 Zoning Map.

The above tables estimate the total amount of land that is available for development within each zoning classification. These estimates attempt to take into account those lands already developed and those lands not generally available for development due to wetlands. Those areas are not available for development. Therefore, the columns in the above tables titled "AREA available for development" include only an estimate of those areas that are not generally wetlands and are not currently developed.

The amount of development that will occur within the 20-year planning period of this report is subject to development pressures on the Warrenton area. No attempt has been made in this report to quantify specifically where development will occur within the planning period.

3.5 EXISTING PUBLIC FACILITIES

3.5.1 Sanitary Sewer System

The City's existing sanitary sewer system and wastewater facilities are described in detail in *Sections 4 and 7* of this report.

3.5.2 Storm Sewer System

The City's storm drainage system consists of a mixture of non-continuous underground pipe system, open ditches and natural drainage channels. Almost all drainage is eventually drained to Tansy Creek, Alder Creek or the Skipanon River, all of which drain into the Columbia River.

3.5.3 Water System

The City of Warrenton owns and operates a domestic water system and also provides water to other water distributors (Camp Rilea and the City of Gearhart). The service area for the water system is much larger than the Warrenton Urban Growth Boundary. The Lewis and Clark River and three of its tributaries are located Southeast of the City of Warrenton and supply the City's water. This gravity-fed system has four intakes (small dams) that are located in the Coast Range east of Seaside. Water treatment now consists of a new water treatment plant which is currently on-line.

Currently the City has a new water filtration plant under construction. Completion is expected in October of 2002. The City has also explored the options of using additional raw water storage within the watershed as well as the addition of a supplemental groundwater supply in the Clatsop Plains area. Because of water quality, treatment costs and potential environmental impacts surrounding the possible use of ground water supply, the City prefers the option of adding raw watershed storage. The Clatsop Plains Aquifer (generally south and west of Warrenton) is susceptible to contamination from commercial activities, septic drain field leachate and storm water runoff.

3.5.4 Street System

Streets within the City limits include arterials, collectors and local access streets. Old Highway 101 and Spur 104 serve as main arterial streets in the South part of Warrenton. Highway 101, which runs through the South part of Warrenton, bypasses the downtown core and a majority of the residential areas of Warrenton. The Oregon Department of Transportation (ODOT) maintains these two (2) streets and Highway 101. South Main and Harbor Drive are additional arterial streets that provide access to and from Warrenton.

3.5.5 Other Public Facilities

Other public facilities include a City Hall (including police station and fire station), post office, public schools, community center, several public parks and two boat basins, both of which serve the recreational and commercial/industrial boating needs of the public. The Fort Stevens State Park is located within the City Limits.

This concludes Section 3, Study Area Characteristics. Section 4 will now describe the existing collection system.

SECTION 4
EXISTING WASTEWATER FACILITIES

4.1 WASTEWATER COLLECTION SYSTEM

4.1.1 General

The first portion of Warrenton's existing sewage collection system was constructed in 1969. During the post-World War II years, the City began to experience considerable growth. During the late 1960s, the City came to realize that some type of regional sanitary sewer collection and treatment system would be needed. In 1969 the initial portion of the sewer collection system and the sewer treatment plant was built to serve the then existing City. Prior to 1969, the majority of the City of Warrenton used septic tanks and drain fields for sewage disposal, or occupants of the City discharged raw sewage directly into adjacent waterways. The initial system was expanded in 1975 to serve east Warrenton, and the Port of Astoria facilities including the Clatsop County Airport East of Highway 101. The flat terrain of the City Warrenton service area has required the location of several pump stations within the collection system. As the City of Warrenton has expanded to meet the growing needs of the community, the sewage collection system has also expanded.

4.1.2 Gravity Collection System

The Town of Hammond and Fort Stevens State Park were included in the City of Warrenton sewer system in 1981. The majority of the sewer construction to serve the Town of Hammond and Fort Stevens State Park almost exclusively included PVC sewer lines, gravity sewers and force mains. Portions of the original collection system in Fort Stevens State Park used either asbestos-cement or cast iron pipe. The age of those original collection facilities dates back to the late 1950s and early 1960s. The original City of Warrenton sewer collection system consists almost entirely of concrete gravity sewers (with rubber O-ring joints) and asbestos cement force mains.

As originally constructed in 1969, the collection system included three (3) original pump stations and 7.6 miles of gravity collection system sewer mains. In 1981, the collection system included 18 sewer pump stations and over 16 miles of gravity collection system sewer mains. In 2002, the collection system now includes 26 sewer pump stations. One additional pump station is ready to soon go on line. The total number of miles of gravity sewer collection main is not available from the City of Warrenton.

Information regarding the existing wastewater collection system was obtained from collection system maps, interviews with City staff and site visits. The existing collection system is shown in schematic form on *Figure 4.1 in Appendix N.*

4.1.3 Pump Stations and Force Mains

The pump station locations and force main layout is shown in *Figure 4.1–4.3 in Appendix N.* Each of the 26 pump stations were individually inspected, and tested according to DEQ guidelines. The pump station's forcemains were commented on in the reports. Forcemain discharge manholes were probed with a 4 foot long ½ inch hot-rolled steel rod to determine water damage. Each pump station report contains four (4) components: 1) Report with recommendation for improvement, 2) DEQ flow test spreadsheet, 3) Pump curve, 4) Photos. The complete pump station reports are presented in *Appendix L.* The collection system hydraulic profile is shown in *Figure 4.5 in Appendix N.* In 2001, there were approximately 1,810 active sewer service connections. The residential population in 2001 was 4210 residents.

4.2 WASTEWATER TREATMENT FACILITY

4.2.1 General

Two (2) wastewater treatment lagoons were constructed in 1969 at the time the collection system was initially completed. The sewage treatment system consists of influent pumps, a Parshall flume, a two-cell stabilization lagoon, currently operated in series, followed by disinfection by gas chlorination, the chlorine contact chamber and an outfall weir. A schematic flow diagram showing the flow pattern through the existing treatment facility is shown in *Figure 7.1* below. A hydraulic flow profile showing the flow elevations through the existing treatment facility is shown in *Figure 4.5 in Appendix N*, or see full size plan in the pocket in the back cover of this report.

Facultative lagoons provide both aerobic and anaerobic treatment; the surface layer provides aerobic stabilization while the lower layer provides anaerobic stabilization. Oxygen is supplied to the service layer by wind action and by algae growth. The deeper areas of the lagoons remain anaerobic and allow for the anaerobic decomposition of solids as settled sludge.

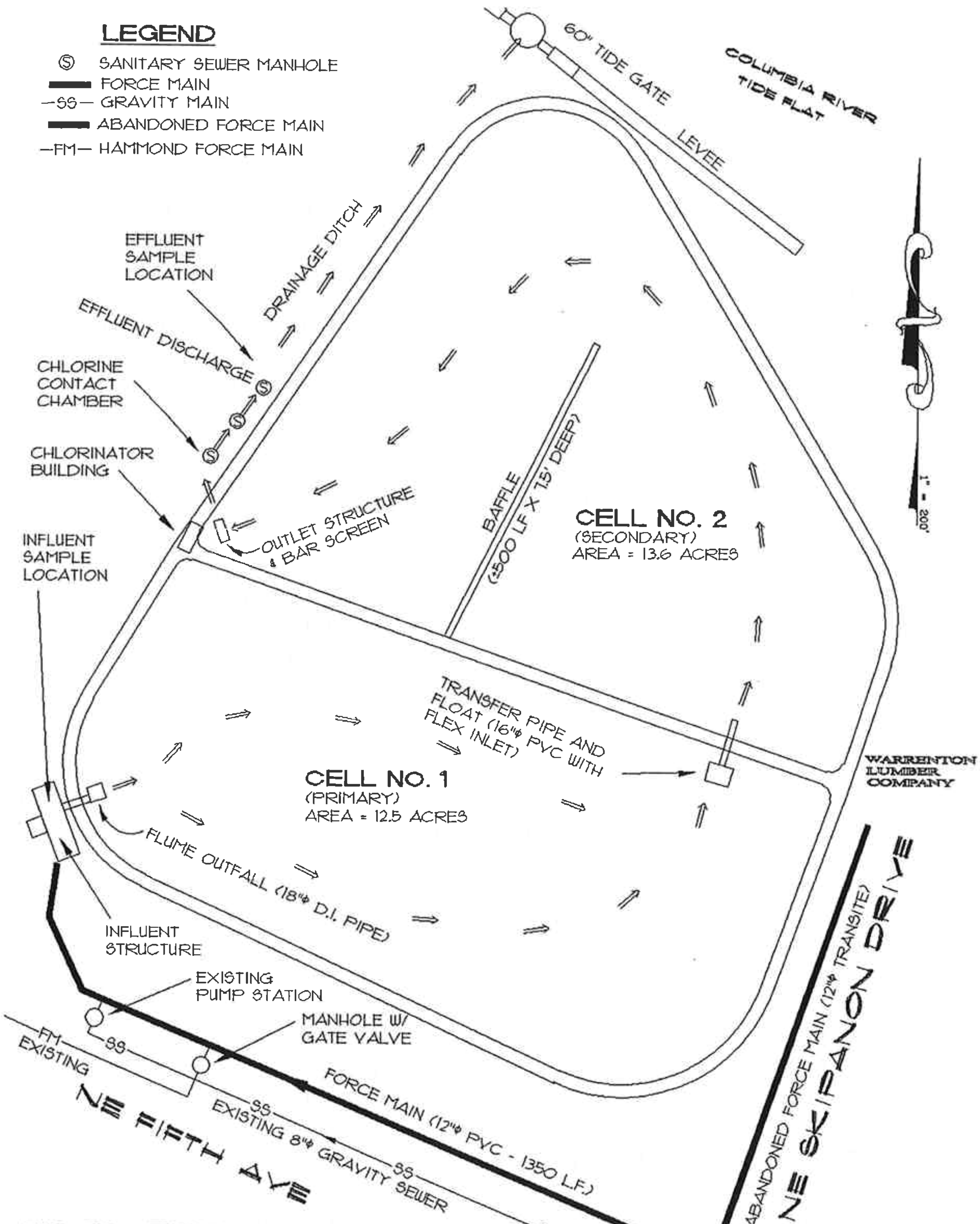


FIG. 7.1 - EXISTING LAGOON SYSTEM
SCALE: 1" = 200'

DRAWING: 3487-1300-06-04.DWG

APRIL 16, 2001
HLB & ASSOCIATES, INC./
H. R. ESVELT ENGINEERING

4.2.2 Influent Pumps and Parshall Flume

All sanitary sewage collected in the City of Warrenton is eventually pumped to the treatment facility by means of one of four (4) sewer pump stations. All of the pumped sewage eventually arrives at the sewage treatment facility in one 12-inch diameter force main. The sewage influent flows through the 12-inch Parshall flume that was recently constructed and placed in operation in March of 2000. Prior to this time, the 6-inch Parshall flume was used to measure all influent flow. The wastewater treatment plant operator has reported that there were several instances during periods of high flow when the old 6-inch Parshall flume would reach capacity and overflow onto the floor surrounding the Parshall flume. Due to the overflow conditions at the old Parshall flume, the treatment plant records do not accurately reflect some of the highest peak flows arriving at the plant.

As a part of the plant upgrade of March 2000, a fiberglass equipment shelter now houses the equipment listed below:

Sewage flows are sampled and held in a refrigerated composite sampler. The sampling equipment is ISCO, 3710 R. At the upstream portion of the Parshall flume, a sampling hose collects the samples based on a flow proportional system. This hose runs inside the shelter. The samples are then refrigerated until removed by City personnel and sent to the lab for analysis.

At about the center of the Parshall flume, a transducer assembly reads the depth of flow in the flume, and is hard wired to an Open Channel Monitor, (OCM-3 by Milltronics) which is inside the shelter and calibrated to this flume. This assembly has a totalizer which can account for total flow per day, month, and year since installation.

For the actual day to day recordings, inside the equipment shelter and hard wired to the OCM-3, is a Portlow MRC 5000 circular chart recorder. This allows individual time and day analysis of flows not possible with the Milltronics OCM-3.

For further information concerning the new equipment installed, please see the PERFORMANCE EVALUATION MANUAL for the SEWAGE LAGOON IMPROVEMENT PROJECT, dated September 2000, by HLB & Associates, Inc..

4.2.3 Primary Cell Influent Pipe

With the old 6-inch Parshall flume, raw sewage flowed into Cell #1 by means of the 15-inch diameter sewer pipe to a distribution box located in the east half of Cell #1. With the construction of the new 12-inch Parshall flume, raw sewage now flows into southwest corner of Cell #1 through an 18-inch diameter ductile iron influent pipe. Effluent now flows from the influent pipe at the southwest corner of Cell #1 across the full-length of Cell #1 to the transfer pipe in the northeast corner of Cell #1. With this change (in operation in March 2000), the entire area of Cell #1 is available for sewage treatment as intended. There are no dead spaces or short-circuiting in Cell #1 created by mislocated piping. Prior to this change, the effective area of Cell #1 was extremely limited due to a short circuit of raw sewage flow from the lagoon influent pipe to the transfer pipe located between Cell #1 and Cell #2. Additionally, the transfer pipe between Cell #1 and Cell #2 was recently upgraded (March 2000) to include the submerged effluent pipe suspended on a floating platform. This change allows effluent flowing from Cell #1 to Cell #2 to be drawn from the anaerobic layer below the surface of the lagoon.

4.2.4 Lagoons

There are currently two facultative lagoons with the total surface area of 26.1 acres; there being 12.5 acres in the primary lagoon or Cell #1, and 13.6 acres in the secondary lagoon or Cell #2. Initially, the lagoon system was operated with two (2) cells in parallel. They are now operated in series as the flows travels from Cell #1 and into Cell #2. The lagoons are operated at varying depths of between 3.5 and 7.0 feet. The City of Warrenton has never removed built up sludge since the lagoons were built in 1969. During a recent drawn down of Cell #1 for the construction of the new Parshall flume, a buildup of sludge in the form of an island was noted surrounding the original raw sewage discharge point. Sludge buildup has accumulated to depth of approximately three feet or more in Cell #1 around this original point of discharge and approximately 1-1/2 feet in other areas of Cell #1. A map showing the measurements of the sludge build-up in Cell #1 is shown in *Appendix J, Biosolids Management Plan – Appendix A, Lab Analysis, Figure A-1.* The City of Warrenton staff recognizes and accepts the requirement to remove all sludge buildup from the lagoons in the immediate future.

Cell #2 includes the floating baffle that runs north to south with the intended purpose of more fully utilizing the treatment capacity of this cell. The baffle causes the flow into Cell #2 from the transfer pipe to flow northerly around the north end of the baffle, thereby utilizing the north end of Cell #2. This baffle was installed in March 2000.

The dikes surrounding both of the lagoon cells appear to be in good condition with no evidence of erosion due to wind waves along any of the dikes surfaces. The wastewater treatment plant staff has effectively controlled vegetation in the shallow areas of the lagoon along the dike slopes.

4.2.5 Bar Screen Outlet Control Box

The bar screen outlet control box is constructed of concrete. It is 10.0 feet long by 4.0 feet wide and 4.0 feet deep. The bar screen is set into the concrete box at a 60 degree angle. It is 4'-8" high and 3'-4" wide. The vertical bar screen spacing is 1-3/4" clear opening dimension.

Behind the bar screen outlet control box there are four (4) each ¼" thick aluminum plates with differing orifices to control the lagoon elevations and effluent outflow to the chlorine contact chamber (CCC). The plant operator varies the orifice plates manually on an as-needed basis to meet the flow needs of the plant and to maintain lagoon levels. The following table reflects plate orifice size to corresponding flow, CCC maximum flow and effluent flow meter range.

Orifice Size	Flow to CCC
Plate #1 – Blank*	0.00 m.g.d.
Plate #2 – 3.5" dia.	0.30 m.g.d.
Plate #3 – 4.5" dia.	0.50 m.g.d.
Plate #4 – 6.0" dia.	0.90 m.g.d.
CCC Max Flow Rate	1.20 m.g.d.
Effluent flow meter max	1.75 m.g.d.

* Blank, no hole, shuts off effluent flow to the chlorine contact chamber.

4.2.6 Chlorine Injection

The chlorine gas injection system is located near the southwest corner of the Cell #2, on the cell levee in a small brick building. It is ventilated by a roof ventilator and ventilation grills in the access doors.

The gas chlorination injection equipment is manually adjusted for flow control. The plant operator reads the level of chlorine each day at the effluent weir box, and manually adjusts the manifold injection system accordingly.

The gas chlorinator is connected to the chlorine contact chamber (CCC) by a piping system. Inside the building, the chlorine gas cylinders are attached to a Wallace and Tiernan Series scale, with an individual dial indicator for the weight of each gas cylinder. It is observed through daily maintenance. Once the cylinder empties, the chlorinator is switched over to the fresh cylinder.

The chlorination shelter building and equipment appear to be in reasonably good condition and well maintained. There is no evidence of advanced corrosion in any of the equipment in this building.

4.2.7 Chlorine Contact Chamber

The existing chlorine contact chamber (CCC) was reconstructed in 1990 and consists of 160 linear feet of 60-inch diameter concrete pipe and has a volume of 23,500 gallons. The design of the chlorine contact chamber improvements completed in 1990 allows for one additional chlorine contact chamber of the same size. There is room at the site today for such an improvement. The CCC was sized in 1990 for a maximum flow rate of 1.2 million gallons per day (mgd). The chlorine detention time was calculated to be 75 minutes for the then average flow rate of 0.45 mgd. The City of Warrenton staff has noted that the chlorine contact chamber has not been cleaned since constructed in 1990. The City has obtained a Vactor Truck and now has the capability to clean and maintain the chlorine contact chamber.

4.2.8 Effluent Outfall Weir

The flow from the bar screen outlet control box is controlled primarily by the orifice plates described above. Once through the plates, the effluent flows through the chlorine contact chamber (CCC). Upon leaving the CCC, effluent flows through a ten-foot length of 12"ϕ pipe to the outfall weir box. The outfall weir consists of a weir box constructed of concrete, with an equipment shelter placed above the box. Box dimensions are 4'-8"(W) by 7'-0"(L) by 6'0"(H).

The equipment shelter houses a Stevens 61R tape recorder. It also contains an interior light, and a small heater assembly to control temperature and humidity. The recorder records instantaneous flows and totalized flows for 7-10 days typically before the paper recording tape has to be changed.

The outfall weir box is a split design. In the first chamber, a stilling well contains the calibrated float that inputs effluent flows to the Stevens recorder. In the second chamber, an aluminum weir plate, with a 45 degree "vee", is installed. The instrument calibration is specific to that weir. Should the effluent outfall weir box or weir plate assembly be modified, the Stevens recorder should be modified and recalibrated to reflect those changes.

4.2.9 Outfall Ditch and Tide Gate

Overview of Outfall Structure(s)

The effluent screen box is located inside lagoon Cell #2 at the southwest corner. It is connected through the dike wall to the chlorine contact chamber by a 12" diameter PVC pipe. After flowing through the chlorine contact chamber, the effluent flows through an above ground fiberglass flow recording station structure and a 45 degree v-notch weir. A 12" diameter PVC pipe is connected to the outlet of this effluent weir, which then runs under the service road, and is open at the outfall end, above the normal water level in the outfall ditch. This 12" diameter pipe has a slope of 0.03 ft/lineal foot from the recording station box to the open end, and the effluent then drops approximately 1.0-1.5' into the constructed outfall conveyance ditch.

From the 12" diameter lagoon outfall pipe, this constructed ditch then runs north for approximately 775 feet. The ditch location is parallel to and on the west side of the west levee of Cell #2. It then flows through a 60" diameter tide gate. See photos *Figure 4.8 through 4.15*. Flow through the channel of this outfall ditch appears to be choked with vegetation.

OUTFLOW DITCH PHOTOS

FIGURE 4.8

At Effluent Weir, Looking North
(Man Made Ditch)

FIGURE 4.9

Fig. 4.10
continues
to East

At North End of Cell #2, Looking Northwest
at Outfall of Tide Gate and Columbia River

FIGURE 4.10

Fig. 4.11 continues to East

At North End of Cell #2, Looking North
at Alder Cove Tide Flat and Columbia River

FIGURE 4.11

Fig. 4.12 continues to East

At North End of Cell #2, Looking Northeast
at Alder Cove Tide Flat and Columbia River

FIGURE 4.12

At North End of Cell #2, Looking East
at Alder Cove Tide Flat and Columbia River

FIGURE 4.13

Manmade Ditch Near Tide Gate
Looking South, Upstream.

FIGURE 4.14

Tide Gate Under Water at High Tide
at Alder Cove

FIGURE 4.15

Alder Cove Tide Flat at High Tide
Looking West to East across Alder Cove

History of the Conveyance Channel

Historical photos and an original construction plan set form the basis for the conclusion that this ditch has been cut in or created by spoils placement or a combination of methods, and that the outfall ditch is therefore man-made. *Appendix D* contains a drawing, photos and cross-sections, as they appear in the following text.

The available historical photo evidence is from a 1953, reverse aerial photo image from the Clatsop County Archives, labeled *Aerial Photo #1*. This photo indicates that, prior to the construction of the Warrenton Wastewater Lagoons; the general wetland area that is now the lagoons was drained by an existing natural waterway which crosses what is now the lagoon area.

Other available records that were used for this report were the "AS-BUILT" drawings from *Carl Greene & Associates*, dated 10-30-69. Sheet 18 of 22 shows clearly the same existing natural waterway as shown on the 1953 photo labeled Aerial *Photo #1*. This same natural waterway is also shown on the 1958, *Aerial Photo #2* and the 1966 *Aerial Photo #3*, reverse image photos.

This pre-existing natural waterway (prior to lagoon construction in 1968) started near 5th Avenue, and then meandered in an "S" shape across the location of Cell #2 to a drainage ditch against the railroad dike (parallel with the Columbia River), approximately 900 feet east of the tide gate. From the north end of this waterway, the flow pattern then turned to the west, and ran parallel with the dike, until it drained out to the Columbia River through the 60" diameter tide gate.

Before lagoon construction, the wetlands of this general area were drained by this pre-existing natural waterway described above. In 1968, during the construction of the waster water lagoons, the storm water flow in this natural ditch was re-routed into a newly constructed man-made drainage ditch on the west side of the wastewater lagoons.

As the lagoons were constructed, per the 1968 plans by Carl Greene & Associates the outflow from Cell #2 was changed to follow this new man-made channel along the west side of the lagoons. The new man-made or constructed drainage ditch flows in a straight line north to the tide gate. Sheet 18 of the 1968 plans denotes the spoils haul areas for both Cell #1 and Cell #2, and includes a plan note not to block the waterways.

The 1966 *Aerial Photo #3* shows the beginning of trenching in the lagoon area, as shown by the new straight line of the constructed ditch, while also showing the then still existing meandering channel.

Since the lagoons are obviously constructed, it must be assumed that the spoils were deposited in the areas identified by the plans. This spoils deposition on the west side of Cell #2 would also contribute to the creation of this man-made ditch.

A 1970 and 1987, reverse image *Aerial Photos #4 and #5*, show a distinct, linear, straight line cut to the existing tide gate. This cut is visible along the northwest edge of Cell #2; however, it is somewhat faint in the photographs.

Cross sections of the outfall ditch at 50 foot intervals were taken in June-July 2002 by Karl Foeste, PLS #849. These cross sections start at a location 100 foot south of the existing 12" diameter PVC effluent outfall pipe, and continue north, up the channel, through the tide gate, and out into the Alder Cove tidal flat approximately 3400 feet.

This ditch is a linear, well defined, reasonably regular channel as the table below shows:

*Cross Section**	*Description*	*Depth**	*Side Slope L**	*Side Slope R**
1+00	Effluent outfall	-2.2+/-	1:1	2:1
3+00	Along ditch	-2.3+/-	1:1	1:1
6+00	Along ditch	-0.8+/-	0.5:1	1:1
8+00	South of tide gate	-2.2+/-	1:1	2:1

*From survey information provided by Karl Foeste, PLS #849, June and July, 2002.

Cross sections for station 1+00, 3+00, 6+00 and 8+00 are typical of the ditch bottom conditions. These cross sections show an approximate 1:1 to 2:1 side slope cut on the sides of the ditch. The depth at cross section station 6+00 is affected by sediment or mud that has settled on the bottom. Graphical representations of these four (4) sections are included in *Appendix D, CS-Figures 1-4*.

History of Outfall Ditch Construction and Conclusion

The timing of the initial construction/operation of the ditch as an effluent outfall therefore has been documented to be during 1966-1968. The actual creation of the ditch could have been prior to 1966 however, as a method to allow drainage from the lagoon construction area. It may have also been created by the placement of spoils, (shown on Sheet 18 of the lagoon plans) such that they would direct water toward the existing 60" tide gate. This tide gate was originally a wooden tide box likely constructed in 1918 when the original dike was constructed along the Columbia River to protect the City of Warrenton from flooding. However, in 1937, the Corps of Engineers proposed replacing the tide box in this location with a 60" wood-stave barrel and metal tide gate. This proposed work was apparently completed since the existing type of tide gate is a barrel-stave wooden body, metal tide gate. Therefore, the "tide gate" was originally installed at this location in approximately 1918. Eventually, its operation and maintenance were turned over to the City of Warrenton.

This 60" diameter tide gate connects the outfall ditch to Alder Cove, through another levee. The tide gate is fully exposed at low tide, and any surcharging behind it (upstream) opens the tide gate to Alder Cove. Some slight leakage or backflow was observed. This gate appears to be regularly maintained and lubricated. However, due to its age, extensive maintenance or replacement will need to be considered in the future.

Bulleted items in DEQ letter of December 13, 2001

Item #1

From the history noted above, the construction of the tide box/tide gate preceded the channel construction. The outfall ditch design information is included in the *Appendix D* as a partial copy of Sheet 18, by *Carl Green and Associates*, dated October 30, 1969. The dike system was originally in place in 1918, updated by the Army Corps of Engineers in the late 1930's along with the tide box/gate replacement.

Item #2

The conveyance channel is located on property owned by the City of Warrenton.

Item #3

The majority of the current channel flow can be characterized as wastewater outfall especially during dry months. However, during storm events, it is probable that the significant storm runoff from surrounding areas west of the lagoon system are drained by this ditch. No hydrologic studies are available to quantify the amount of storm runoff through the ditch. Based on historical drainage and photos, it can be concluded that the constructed ditch was designed to redirect storm water runoff around the new lagoons and to convey the effluent to the 60" diameter tide gate.

Item #4

The drainage for this area, before construction of the lagoons, meandered in the "S" shape described above and shown on Sheet 18 by Carl Green and Associates, *Appendix D.* Since construction of the lagoon system, the ditch has been cut straight, northerly, and at a constant depth to the tide gate. Also, as mentioned above, see the 1970 *Aerial Photos #4 and #5*, reverse image, which shows a distinct, linear cut north to the existing tide gate. This cut is visible along the northwest edge of Cell #2. Typical cross sections are provided in the *Appendix D.*

Item #5

The cross sections after the tide gate do not change appreciably in depth. After the tide gate, the effluent enters a "tide flat" and as such, the effluent dispersal is a result of tidal action. Therefore, the depth of the channel after approximately 1000 feet north of the tide gate is at -1.0' or less. The shape of the channel is significant here also. It fans out, and although cross sections were taken approximately 1000 feet further, no appreciable change is noted in the cross sections. They are basically a flat line. At these distances out into the channel, it is apparent that as the tides move the sand in the tidal flat, they also move the channel.

Item #6 & 7

The Warrenton Wastewater Lagoon outfall channel is a distinct linear cut as shown in the photographs provided. No fish populations have been observed in this outfall. Since this ditch is primarily used for wastewater conveyance, no fish populations were anticipated.

HLB & Associates has contacted the Columbia River Estuary Study Taskforce, (CREST) to ascertain our initial observations. CREST reported that Alder Creek, the next stream to the west, meanders naturally and does contain fish populations. Since it is a natural stream, they would expect to see a variety of fish and visibly different water quality. This is apparent from CREST's observations. Please refer to the CREST letter in *Appendix D* for further details regarding the biological characteristics of both the ditch and Alder Creek.

4.2.10 Alder Cove Tide Flat and Columbia River Outfall

Once through the tide gate, the effluent flows out across Alder Cove. This is a small tidally influenced inlet or estuary on the Columbia River. At low tides Alder Cove has a channel, approximately 1000-1200 feet long that is visible from the tide gate out to the Columbia River; refer to photos *Figure 4.9 through 4.12*, in Section 4.2.9.

This concludes the description of the existing system. Section 5 contains wastewater characteristics specific to Warrenton, along with historical data.

SECTION 5
WASTEWATER CHARACTERISTICS

5.1 INTRODUCTION

The existing wastewater treatment facility for the City of Warrenton has, in recent years, violated the National Pollutant Discharge Elimination System (NPDES) permit for biochemical oxygen demand (BOD_5) the total suspended solids (TSS). From August 1999 through October 2000, there were a total of 21 violations of the NPDES discharge limits. In addition to not meeting the requirements of the NPDES permit as noted above, the population of the area is expected to grow and, as the population increases, demands on the treatment system will also increase. It is also anticipated that more stringent environmental regulations will require the treatment facilities to be upgraded. In order to identify future expansion needs and costs, wastewater flows and loads have been projected for the 20-year planning period. The City can thereby plan for future expansion to meet demands while avoiding system deficiencies and regulatory violations.

5.1.1 Definitions

The following terms are frequently used throughout this report and aid in understanding the key wastewater terms generally used in wastewater system reports:

Wastewater: The total fluid flow in a sewer system. Wastewater may include sanitary sewage, industrial waste, and infiltration and inflow (I/I).

Infiltration/Inflow (I/I): Groundwater and stormwater in the sewer system.

Biochemical Oxygen Demand (BOD): This parameter is a measure of wastewater strength in terms of the quantity of oxygen required for biological oxidation and the organic matter contained in the wastewater. The BOD loading imposes on a treatment plant influences both the type and degree of treatment which must be provided to produce the required effluent quality. All references to BOD in this report are to 5-day BOD at 20 degrees Celsius.

Total Suspended Solids (TSS): This parameter is a measure of the quantity of suspended material contained in the wastewater. The quantity of SS removed during treatment influences the sizing of sludge handling and disposal processes, as well as the effectiveness of the disinfections.

Sewerage System: The entirety of the City's sewage collection treatment and disposal system. It encompasses the service laterals, collector and interceptor sewers, manholes, lift stations, treatment and disposal facilities.

The following terms are used to define seasonal differences in wastewater flow characteristics:

Dry-Weather Period: Generally defined as the period when precipitation is limited and stream flows are low. This period is specifically defined by Oregon Administrative Rules (OAR) 340-41-21 as May 1 through October 31.

Wet-Weather Period: Generally defined as the period when precipitation is greatest and stream flows are highest. This period is specifically defined by Oregon Administrative Rules (OAR) 340-41-215 as November 1 through April 30.

The following terms are used to characterize wastewater flows:

Average Daily Flow (ADF): Total wastewater flow for one year, divided by the number of days in that year.

Average Dry-Weather Daily Flow (ADWDF): Total wastewater flow for the dry-weather period, divided by the number of days in the dry-weather period.

Maximum Dry-Weather Weekly Flow (MDWWF): Total flow for the week with the highest wastewater flow during the dry-weather (summer) period.

Maximum Dry-Weather Monthly Flow (MDWMF): Total wastewater flow for the month with the highest wastewater flow during the dry-weather period, divided by the number of days in that month.

Peak Dry-Weather Daily Flow (PDWDF): Total flow for the day with the highest wastewater flow during the dry-weather period.

Average Wet-Weather Daily Flow (AWDF): Total wastewater flow for the wet-weather period, divided by the number of days in the wet-weather period.

Maximum Wet-Weather Weekly Flow (MWWWF): Identical to MDWWWF except that the wet-weather (winter period) is used.

Maximum Wet-Weather Monthly Flow (MWWMF): Total wastewater flow for the month with the highest wastewater flow during the wet-weather period, divided by the number of days in that month.

Peak Wet-Weather Daily Flow (PWWDF): Total flow for the day with the highest wastewater flow during the wet-weather period.

The following terms are used to describe additional flow characteristics developed through statistical methods:

10-Year Maximum Dry-Weather Monthly Flow ($MDWMF_{10}$): The anticipated wastewater flow for the month of May, when precipitation equals the amount reported to have a 10% probability of reoccurrence.

5-Year Maximum Wet-Weather Monthly Flow ($MWWMF_5$): The anticipated wastewater flow for the month of January, when the precipitation total equals the amount reported to have a 20% probability of reoccurrence.

5-Year Peak Daily Average Flow ($PDAF_5$): The anticipated maximum daily wastewater flow that results from a 24-hour precipitation total having a 20% probability of reoccurrence, typically occurring during the wet-weather period.

5-Year Peak Instantaneous Flow (PIF_5) or Peak Wet-Weather Hourly Flow (PWWHF): The anticipated peak hourly wastewater flow associated with the $PDAF_5$, typically occurring during the wet-weather period.

The following terms are included for clarification:

Current: Generally refers to 2001 conditions.

Design: With regard to flows, "design" refers to anticipated flows that would occur under conditions corresponding to the flow characteristics defined above. "Design" takes into account a full analysis of the flows and generally ignores current system limitations such as inadequate pump stations, and collection system capacities. As a result, "current design" flows may vary considerable from the record of flow currently observed at the WWTP. Future design flows include allowances for community growth and, possibly, other changes in system characteristics.

5.1.2 Variation of Wastewater Flows and Loads

Variations on flows and loads occur annually at the wastewater treatment plant due to:

a) the seasonal effects of weather, and
b) the magnitude of tourist traffic in the community.

Influence on sewage flows from infiltration and inflow (I/I) are greatest during the months of November through May. The greatest percentage of the tourist traffic impact comes from Fort Stevens State Park, the KOA Campground and the transient nature of the sport fishing population. Tourist traffic is highest during the months from June through October.

One important variation on the flow characteristics is related to the fact that all of the sewage flow that comes to the waster water treatment facility is by means of pumped sewage. Several of the pumping stations can be overloaded and surcharged during periods of high flows. The maximum flow of individual pumps stations is therefore limited by the capacity of the pump rather than the actual sewage flow arriving at the pump station. This situation tends to limit the peak flows at the sewer treatment plant from the actual peak sewage flows in the sewer collection system if the pumps were to be correctly sized. Therefore, the actual peak sewer flows recorded at the treatment plant in the DMR's do not accurately reflect actual peak sewer flows in the sewer collection system. Once the sewer pump stations are appropriately upgraded with larger pumps, the peak flow at the treatment plant will increase (not as a result of population growth or I/I). Such growth is accounted for in this study by conservatively calculating the peak flow projections.

5.1.3 Effect of Population Growth on Flow Data

Plant flow data from 1992 through 2001 was reviewed for this report. During the period of 1990 through 1998, the population of Warrenton grew at an annual rate of approximately 113 people (3.2% per year, not compounded). Because of the high percentage of inflow and infiltration in the wastewater, the annual transient population fluctuation during this period does not appreciably affect the flow data. Additionally, it should be noted that the population records are available only for the permanent population, and do not include the transient population mentioned above.

In order to accurately account for the increased sewage flows resulting from transient populations and excessive sewer inflow and infiltration, a population equivalent was used in this report. A current population equivalent was calculated based upon the annual average BOD_5 loading of 1,000 pounds per day divided by a standard contribution of 0.18 lb of BOD/PE/day. This results in a current population equivalent (PE) of 5,600.

5.1.4 Effect of Inflow and Infiltration (I/I) Reduction Projects on Flow Data

Little has been done in the way of I/I reduction work since the publication of the Sanitary Sewerage System Facilities Planning Report prepared by Westech

Engineering, Inc. in April of 1983. In general, the recommended I/I reduction program outlined in the Westech Report has not been implemented with the exception of the replacement of the old clay tile sewer pipe in the Airport Basin System. Even with this replacement work, the Airport Basin system is seriously overloaded due to high volumes of I/I. It is critical that the City undertake some substantial efforts to reduce the I/I in the wastewater collection system, particularly in areas such as the Airport Basin System.

Lagoon influent flows are still quite responsive to variations in rainfall. Flow data at the treatment plant shows an almost instantaneous response to heavy rainfall indicating high inflow conditions. Further I/I studies resulting in actual I/I construction projects are recommended to identify additional improvements needed beyond those outlined in the Westech Report.

The Warrenton sewer service area is located at a very low elevation relative to sea level. Many of the sewer mains are located in saturated ground water areas year round, not just in the winter months. Infiltration in these areas has a significant impact upon the sewer flows at the treatment plant.

5.2 WASTEWATER FLOWS AND ANALYSIS

5.2.1 Observed Data

Treatment plant records for the years 1998 through 2001 were analyzed to estimate wastewater flows. The one-year period of January to December 2001 were also analyzed. Due to the abnormally low amount of rainfall during the winter of 2000 and 2001, the other years of 1998 through 2000 (years of normal rainfall) were used to check the current year data. Reported daily plant influent flow and daily rainfall totals for November 1999 – December 2000 and Year 2000 are shown in *Figure 5.1 and Figure 5.2,* below. Average monthly flow and monthly rainfall totals are shown in *Figure 5.3* through *Figure 5.6,* below for the years 1998 through 2001.

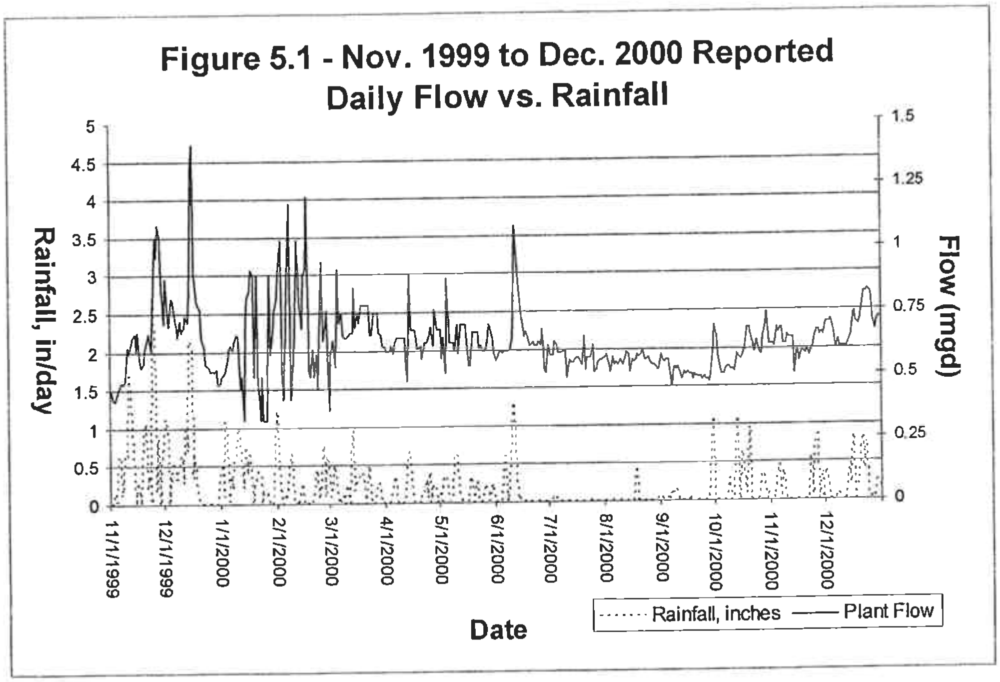


Figure 5.1 - Nov. 1999 to Dec. 2000 Reported
Daily Flow vs. Rainfall
Rainfall, in/day
Flow (mgd)
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
1.5
1.25
1
0.75
0.5
0.25
0
11/1/1999
12/1/1999
1/1/2000
2/1/2000
3/1/2000
4/1/2000
5/1/2000
6/1/2000
7/1/2000
8/1/2000
9/1/2000
10/1/2000
11/1/2000
12/1/2000
Date
Rainfall, inches
Plant Flow

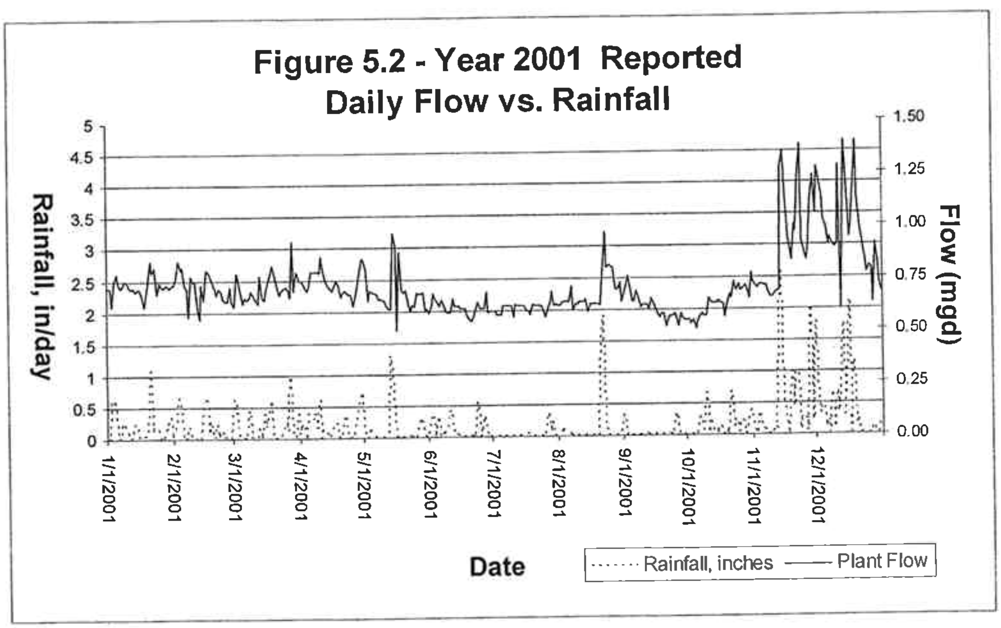


Figure 5.2 - Year 2001 Reported
Daily Flow vs. Rainfall
Rainfall, in/day
Flow (mgd)
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
1.50
1.25
1.00
0.75
0.50
0.25
0.00
1/1/2001
2/1/2001
3/1/2001
4/1/2001
5/1/2001
6/1/2001
7/1/2001
8/1/2001
9/1/2001
10/1/2001
11/1/2001
12/1/2001
Date
Rainfall, inches
Plant Flow

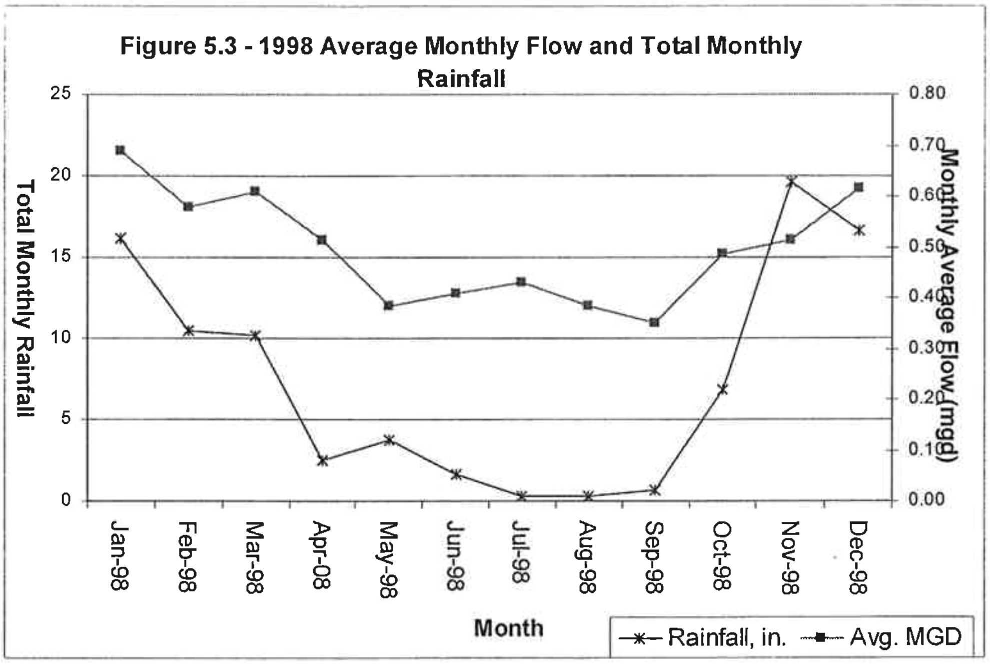


Figure 5.3 - 1998 Average Monthly Flow and Total Monthly Rainfall
Total Monthly Rainfall
Monthly Average Flow (mgd)
25
20
15
10
5
0
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
Jan-98
Feb-98
Mar-98
Apr-08
May-98
Jun-98
Jul-98
Aug-98
Sep-98
Oct-98
Nov-98
Dec-98
Month
Rainfall, in.
Avg. MGD

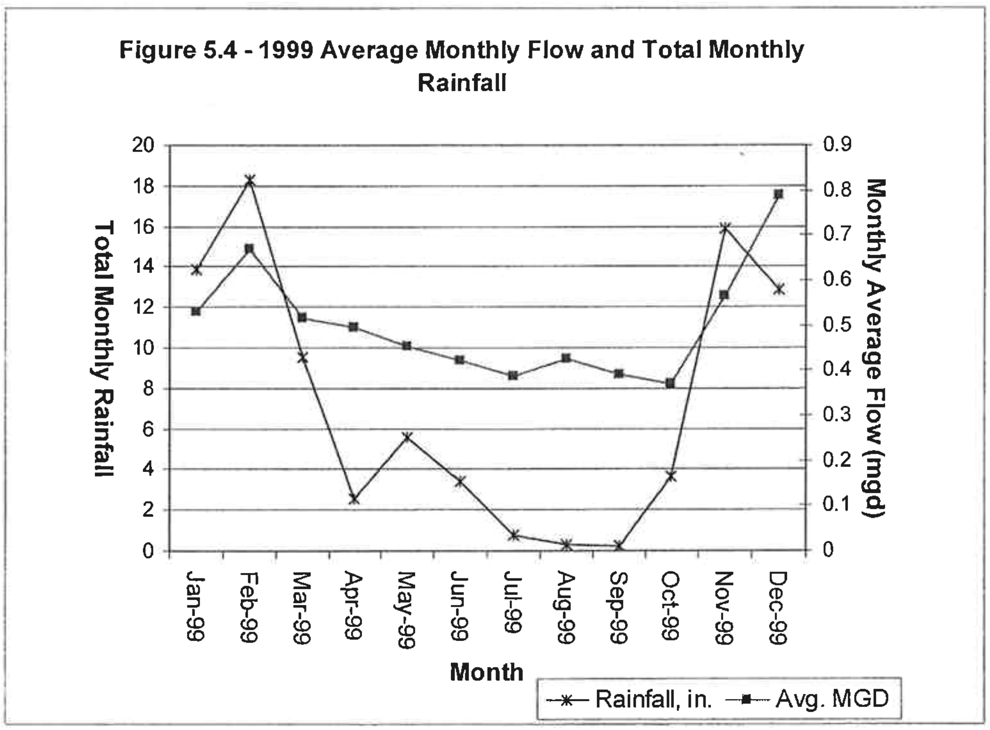


Figure 5.4 - 1999 Average Monthly Flow and Total Monthly Rainfall
Total Monthly Rainfall
Monthly Average Flow (mgd)
20
18
16
14
12
10
8
6
4
2
0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Jan-99
Feb-99
Mar-99
Apr-99
May-99
Jun-99
Jul-99
Aug-99
Sep-99
Oct-99
Nov-99
Dec-99
Month
Rainfall, in.
Avg. MGD

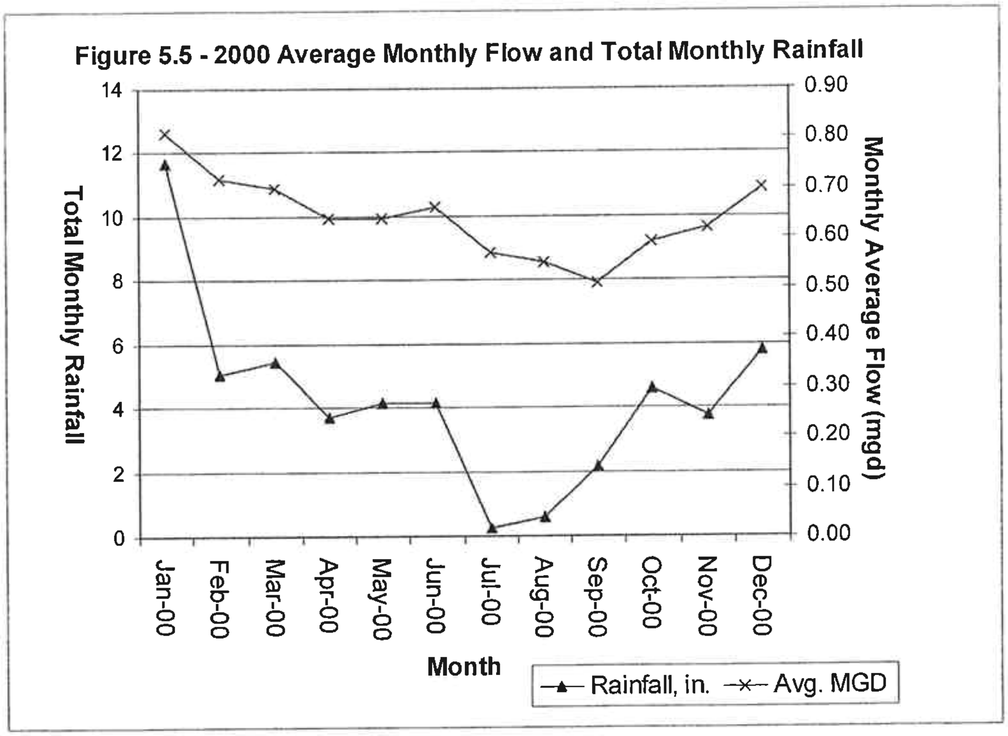


Figure 5.5 - 2000 Average Monthly Flow and Total Monthly Rainfall
Total Monthly Rainfall
Monthly Average Flow (mgd)
0
2
4
6
8
10
12
14
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
Jan-00
Feb-00
Mar-00
Apr-00
May-00
Jun-00
Jul-00
Aug-00
Sep-00
Oct-00
Nov-00
Dec-00
Month
Rainfall, in.
Avg. MGD

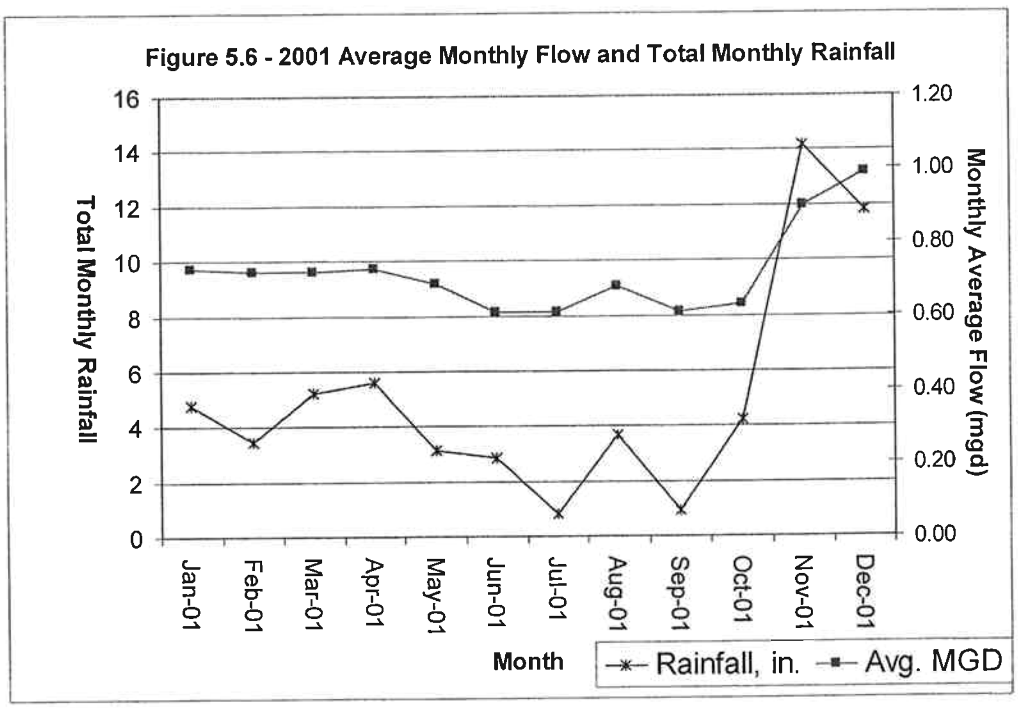


Figure 5.6 - 2001 Average Monthly Flow and Total Monthly Rainfall
Total Monthly Rainfall
Monthly Average Flow (mgd)
0
2
4
6
8
10
12
14
16
0.00
0.20
0.40
0.60
0.80
1.00
1.20
Jan-01
Feb-01
Mar-01
Apr-01
May-01
Jun-01
Jul-01
Aug-01
Sep-01
Oct-01
Nov-01
Dec-01
Month
Rainfall, in.
Avg. MGD

Table 5.1 below summarizes influent flow data for the years 1998 through 2001:

TABLE 5.1 - Historical Influent Flow Data At Treatment Plant

Year	ADWDF (mgd)	AWWDF (mgd)	MWWMF (mgd)	Annual Rainfall, inches
1998	.41	.59	.69	89.15
1999	.41	.59	.79	87.03
2000	.59	.70	.81	51.44
2001	.64	.80	.99	49.05

Table 5.2 below summarizes wastewater flows for year 2001:

TABLE 5.2 - Summary Of Wastewater Flows For 2001

Flow Characteristics	Influent Flow (mgd)	Date
Average Daily Flow	.718 mgd	
Dry-Weather (May 1 – Oct. 31)		
Average Daily Flow (ADWDF)	0.64 mgd	May 2001 – Oct. 2001
Maximum Monthly Flow (MDWMF)	0.69 mgd	May 2001
Maximum Weekly Flow (MDWWF)	0.81 mgd	Aug. 22 – Aug. 28, 2001
Peak Daily Flow (PDWDF)	0.97 mgd	Aug. 23, 2001
Wet-Weather (Nov. 1 – April 30)		
Average Daily Flow (AWWDF)	0.80 mgd	Jan. – April, Nov. – Dec. 2001
Maximum Monthly Flow (MWWMF)	0.99 mgd	Dec. 2001
Maximum Weekly Flow (MWWWF)	1.18 mgd	Dec. 13 – Dec. 19, 2001
Peak Daily Flow (PWWDF)	1.40 mgd	Dec. 19, 2001

5.2.2 Corrections to DMR's

During the data research phase of work on this report, a recurring error was found on all recent DMR's previously prepared by the City staff. That error has since been corrected and new DMR's have been prepared by the City staff and resubmitted to DEQ. The incorrect data was as a result of a new effluent weir that was installed as a part of the outlet construction project in approximately 1991. The v-notch weir was incorrectly calibrated and the measurements from the new weir were incorrectly reported at two times that actual flows. That incorrect reporting of two times the actual flow data continued through December 1999. Starting in January 2000, when the new influent flume was installed and put in service, the DMR's began to record the true flow data. All DMR data analysis and summary tables shown herein refer to the corrected plant flow data.

5.2.3 Pump Run Times

As noted previously, all of the sewage flow that comes to the wastewater treatment facility is by means of pumped sewage. Several of the pumping stations can be overloaded and surcharged during periods of high flows. The maximum flow of individual pumps stations is therefore limited by the capacity of the pumps rather than the actual sewage flow arriving at the pump station. Four pump stations ultimately pump sewage flow into the treatment plant through a common force main. Each pump station has a duplex pumping system. Currently, the maximum peak flow with all eight pumps on (four pump stations, each with two pumps on) is 2,200 gpm. This situation tends to limit the peak flows at the sewer treatment plant from the actual peak sewage flows in the sewer collection system if the pumps were to be correctly sized. Therefore, the actual peak sewer flows recorded at the treatment plant in the DMR's do not accurately reflect actual peak sewer flows in the sewer collection system. Once the sewer pump stations are appropriately upgraded with larger pumps, the peak flow at the treatment plant will increase (not as a result of population growth or I/I). Such growth is accounted for in this study by conservatively calculating the peak flow projections.

5.2.4 Statistical Analysis

Warrenton is located on the north Oregon Coast, where winter precipitation typically impacts wastewater flows. Monthly average wastewater flows for 1998, 1999, 2000 and 2001 were plotted against the month's cumulative rainfall totals (*Figure 5.1* through *Figure 5.4*). As this and the preceding graph show high wastewater flows are associated with high precipitation totals, and therefore DEQ's *Guidelines for Making Flow Projections for Sewage Treatment in Western Oregon (1996)* was used to estimate peak flows related to rainfall events.

The guidelines describe a method for calculating peak sewage flows with specific recurrence intervals using observed flow and rainfall statistics. A regression model was used to compute the correlation between flow and precipitation for the study period. The computations presented in this section are based on our interpretation of the guidelines.

Four statistical flow parameters are required for a complete evaluation of the City's wastewater flow characteristics. These are: maximum dry-weather monthly flow having a 10-year reoccurrence ($MDWMF_{10}$); maximum wet-weather monthly flow having a 5-year reoccurrence ($MWWMF_5$); peak daily flow associated with a 5-year storm ($PDAF_5$); and peak instantaneous flow attained during 5-year PDAF event (PIF_5).

<u>$MDWMF_{10}$ and $MWWMF_5$</u>. Maximum dry-weather monthly flow is typically the monthly average flow during the dry-weather period, during the rainiest month with high groundwater. According to the guidelines, this "almost invariably" occurs in May. The 10-year MDWMF is the anticipated monthly flow corresponding to precipitation totals during May having 10% probability of recurrence in any given year. MWWMF represents the anticipated maximum monthly flow occurring during the winter period of high groundwater. According

to the guidelines, high groundwater is usually not attained until January in areas west of the Cascade Range, and therefore the MWWMF occurs in January. The 5-year MWWMF corresponds to the January rainfall having a 20% probability of recurrence.

Statistical rainfall data were obtained from U.S. Weather Bureau for the years 1953 to 2001, for the Astoria, Oregon station. The Astoria Airport is the weather recording station and that location is actually with the UGB of Warrenton. The rainfall amount for a 90% probability not to exceed during the month of May is calculated as 4.48 inches. Rainfall having an 80% probability no to be exceeded during January is calculated as 14.27 inches.

To determine $MDWMF_{10}$ and $MWWMF_5$, a graph of selected monthly average wastewater flow versus the month's rainfall was created (*Figure 5.7, below*). Data was initially selected from December through March, when groundwater is high and extreme flows are most likely. The month of December 2001 (rainfall = 11.83 inches) was significant and was included because it followed another month of high rainfall (November 2001 rainfall = 14.21 inches). That combination of two months in a row of heavy rainfall combined for large plant flows. The rainfall for those two months slightly exceeded the monthly average of 10.59 and 10.75 inches of rain for November and December, respectively. Most notably, there was a heavy storm in December that dumped 6.24 inches of rain in one week in Warrenton, December 13th through 19th, 2001.

One dry month from 2000 and one from 2001 were also selected because of the very limited rainfall in the winter of 2000-2001. The five data points are listed in *Table 5.3,* below and in *Figure 5.7*, below.. A linear regression was performed on the data points, with an R^2 correlation coefficient of 0.998 computed. The linear equation, shown on the graph, was then used to calculate the $MDWMF_{10}$ and $MWWMF_5$.

TABLE 5.3 - $MDWMF_{10}$ and $MWWMF_5$ Data: Monthly Rainfall Totals and Average Daily Flows for 2001 Wet-Weather/High-Groundwater Months

Month	Average Daily Flow (mgd)	Reported Monthly Total Rainfall (in.)	Normal Monthly Rainfall (in.)
June 2001	0.61	2.84	2.62
June 2000	0.66	4.16	2.62
March 2001	0.72	5.21	7.23
April 2001	0.73	5.63	4.89
December 2001	0.99	11.83	10.75

The $MDWMF_{10}$ was computed as 0.68 mgd. This flow rate was met in August 2001 and slightly exceeded in May 2001. The average flow in August was 0.68

mgd with a monthly rainfall of 3.69 inches (normal for August 1.38 inches). This abnormally heavy rainfall was due to a heavy storm of 3.27 total inches of rain in two days.

The $MWWMF_5$ was calculated as 1.09 mgd for the 5-year January rainfall of 14.27 inches. This flow was not exceeded in December 2001 during the heavy storm described above, although the monthly rainfall was only 11.83 inches. We consider the calculated $MWWMF_5$ of 1.09 mgd to be a reasonable estimate of the 5-year maximum monthly wet-weather flow.

5-Year Peak Daily Average Flow ($PDAF_5$). $PDAF_5$ is the flow that will result from a 5-year storm during a period of high groundwater. Since plant records reflect 24-hour periods, the 5-year 24-hour storm event was used to estimate $PDAF_5$. Selected daily wastewater flow was plotted against the corresponding 24-hour rainfall total for seven selected wet-weather storms see *Figure 5.8,* below. Computation flow and rainfall data are provided in *Table 5.4,* below.

TABLE 5.4 - $PDAF_5$ Calculations: 24-Hour Precipitation Totals and Measured Daily Flows for Selected Storms

Date of Storm Event	24-Hour Rainfall, in.	Plant Flow, mgd
Data Used for PDAF5 Calculation		
11/25/1999	1.52	1.05
12/01/1999	1.12	0.89
11/23/2000	0.57	0.59
1/21/2001	1.09	0.84
11/19/2001	0.99	0.83
12/13/2001	1.77	1.14
Data NOT used for $PDAF_5$ calc., but shown in *Figure* 5.8		
11/14/2001	2.62	1.27
11/28/2001	2.04	1.12
11/30/2001	1.79	1.05

The $PDAF_5$ during wet-weather was estimated at 2.14 mgd with a calculated R^2 correlation coefficient of 0.998. The 5-year 24-hour storm event was estimated at 3.9 inches of rainfall from isopluvial maps in the *NOAA Atlas 2, Precipitation-Frequency Atlas of the Western United States, Vol. X, 1973*. A linear extrapolation was used to estimate the 3.9 inch storm event.

5-Year Peak Instantaneous Flow (PIF_5). PIF_5 is the peak hourly flow associated with the $PDAF_5$ event. For this study, the PIF_5 was estimated through extrapolation, using a straight line extrapolation on logarithmic probability chart. It is assumed for this estimate that the PIF_5 occurs during the $PDAF_5$ event occurs in the year that features the $MWWMF_5$. The ADF, as observed from plant records, is also used. The data, with associated probability of occurrence, is shown in *Table 5.5,* below.

TABLE 5.5 - PIF_5 Calculation: Flows and Corresponding Probability of Excellence

Flow Characteristic	Flow (mgd)	Probability of Exceedance
ADF	0.69 mgd	0.50 (1 in 2)
$MWWMF_5$	1.09 mgd	0.083 (1 in 12)
$PDAF_5$	2.14 mgd	0.00274 (1 in 365)
PIF_5	3.90 mgd	0.0001142 (1 in 8,760)

For PIF_5 Calculations see *Figure 5.9,* below.

Figure 5.7 - Selected Monthly Flows vs. Monthly Rainfall

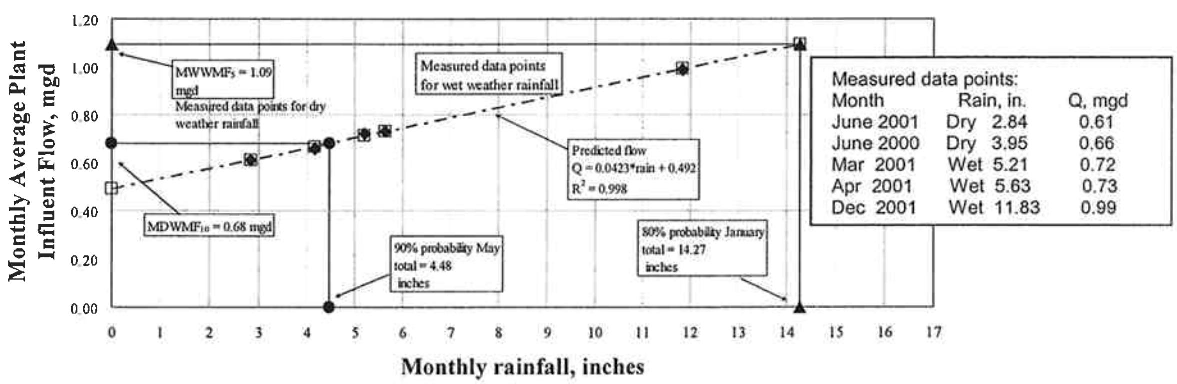


Figure 5.8 - Selected Daily Flows Vs. Daily Rainfall, for $PDAF_5$

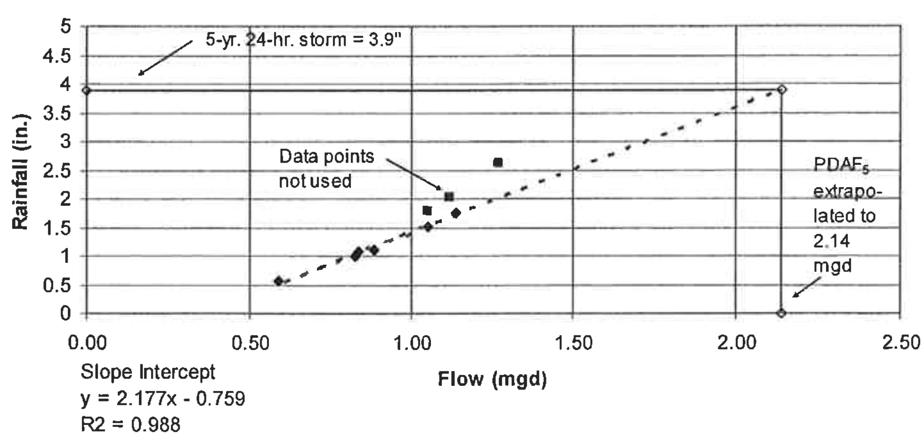
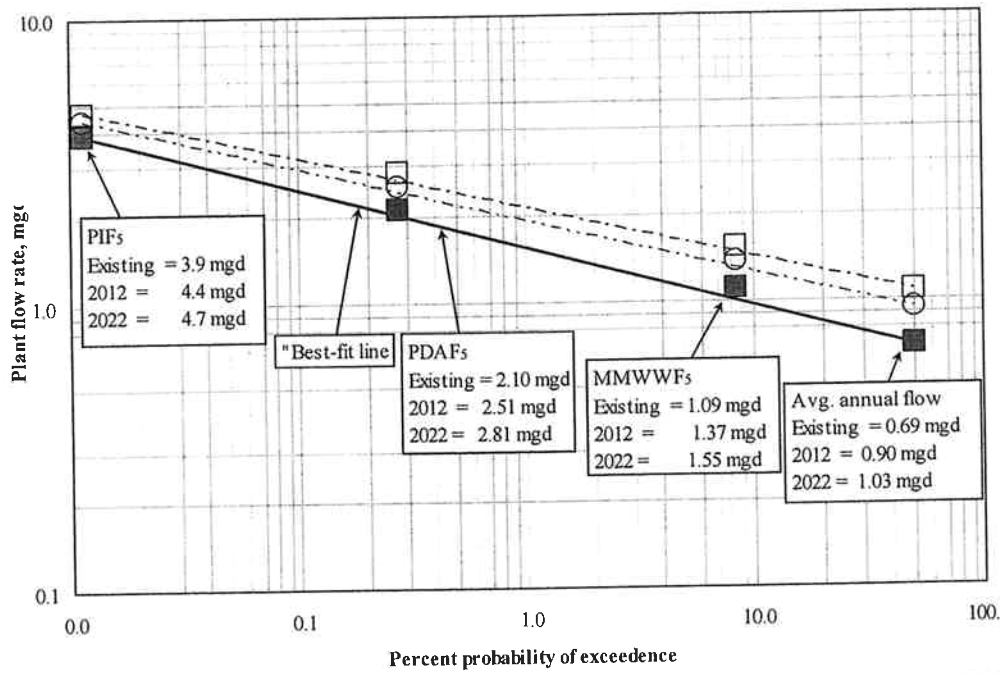


Figure 5.9 - Projections of Statistical Plant Flows for PIF_5



5.2.5 Current (2001) Design Flows

For the year 2001 the current design flows are summarized in *Table 5.6*, below:

TABLE 5.6 - Summary of Current (2001) Design Flows

Flow Characteristic	Flow (mgd)	Population	Flow (gpcd)	EDU's	Flow (gpd/EDU)	Population Equivalents (PE)	Flow (gpd/PE)
Dry-Weather Flows							
ADWDF	0.64	4210	152	2105	303	5600	114
$MDWMF_{10}$	0.68	4210	162	2105	323	5600	122
$MDWWF^{(1)}$	0.82	4210	196	2105	391	5600	147
PDWDF	0.97	4210	230	2105	460	5600	173
Wet-Weather Flows							
AWWDF	0.80	4210	190	2105	379	5600	143
$MWWMF_5$	1.09	4210	260	2105	520	5600	195
$MWWWF^{(1)}$	1.62	4210	384	2105	768	5600	289
$PDAF_5$	2.14	4210	508	2105	1017	5600	382
PIF_5	3.9	4210	926	2105	1853	5600	696
Annual Flows							
Average Daily Flow	0.72	4210	171	2105	341	5600	128

5.2.6 Projected Design Flows

The sanitary component of the wastewater flow is expected to increase proportionally with the increase in population. The per capita flows used to project future flows were determined using historical monitoring records. The per capita flows and resulting flow projections for 20-year planning are shown in *Table 5-7.*

To project design flows for 2022, it is first necessary to estimate additional flows due to general area growth (development within the study area other than major commercial or industrial), additional I/I associated with growth, and to allow for any known large scale commercial or industrial developments. These additional flow components are discussed in the following subsections.

General Area Growth. Population and EDU information are discussed in *Section 3*. An average annual growth rate (AAGR) of 113 persons per year is projected for combined residential, general commercial/industrial development and transient recreational populations. In addition to that growth rate there is a projected increase of 1000 population equivalents due to new service from outside of the planning area. The projected year 2022 population equivalent is 9500 population equivalents (an increase of 3900 persons over the 2001 population equivalent of 5600, an increase of 70%).

In 2000 the population in Warrenton was 4,096 persons with a total number of Equivalent Dwelling Units of 2050 EDU's. Based upon the calculated average of 2.0 persons per dwelling unit, the projected year 2022 EDU's are 3705 (an increase of 1600 over the 2001 EDU total of 2105).

Inflow/Infiltration. Anticipated growth in Warrenton is most likely to occur in new development areas rather than as infill and redevelopment. Much of the areas where sewer mains currently exist and are not developed are limited by the presence of wetlands and other similar development constraints. Therefore, new sewer construction is likely to consist of new PVC mains and laterals. A common allowance for new construction is 200 gallons per inch of pipe diameter per mile per day (gpd/in.-mi.)(Ten State Standards, 1990). Assuming 100 lineal feet of 8" main or service lateral per EDU yields an additional AWW flow in the year 2022 of 48,500 gpd (approximately 45 gpm) due to new infiltration. Since there is little experience in I/I work within the City, it is reasonable to assume that future remedial work will offset future (theoretical) increases in I/I outlined above, and therefore, no separate allowance for increased I/I has been included in the projected design flows.

I/I tends to become more of a problem over time. The City needs to be aware that I/I removal is an ongoing concern and that substantial future I/I removal work will be necessary *just to maintain the present level of I/I*[1]. Additional I/I removal will decrease hydraulic peaks and the potential for overflows. In addition, further I/I removal efforts prolong the useable life of the plant with regards to hydraulic capacity.

Commercial/Industrial Development. Major commercial/industrial development that would generate significant wastewater flows are not anticipated at this time. Future development with significant flows may be accommodated by further I/I removal. In general, for any major facility or addition of major flows, the City will need to evaluate both the flows and the organic loadings proposed. The continued growth of RV traffic in existing and new RV Parks can cause a problem with high loadings. Some form of controlled discharge and pretreatment may be advisable in these cases.

Projected 2022 Design Flows. Projected design flows for the study design year 2022 are summarized in *Table 5.7*. The general area growth allowance of 1000 PE was added to each of the population base of each of the design flow characterizations. In the case of the PDWDF, a 50% increase in the mgd flow was used to account for holiday and major summer weekend peaking.

TABLE 5.7 - Summary of Future (year 2022) Design Flows

Flow Characteristic	Flow (mgd)	Population	Flow (gpcd)	EDU's	Flow (gpd/EDU)	Population Equivalents (PE)	Flow (gpd/PE)
Dry-Weather Flows							
ADWDF	1.0	7405	131	3705	262	9500	102
$MDWMF_{10}$	1.0	7405	138	3705	276	9500	108
$MDWWF^{(1)}$	1.2	7405	168	3705	336	9500	131
PDWDF	1.5	7405	199	3705	397	9500	155
Wet-Weather Flows							
AWWDF	1.2	7405	158	3705	316	9500	123
$MWWMF_5$	1.5	7405	209	3705	417	9500	163
$MWWWF^{(1)}$	2.2	7405	297	3705	594	9500	232
$PDAF_5$	2.9	7405	386	3705	772	9500	301
PIF_5	4.7	7405	633	3705	1265	9500	493
Annual Flows							
Average Daily Flow	1.1	7405	145	3705	289	9500	113

1 The City should strive to achieve higher levels of I/I removal.

5.3 WASTEWATER LOADINGS

5.3.1 Load Classification

For the purpose of monitoring wastewater loads and identifying future design loads, the following classifications will be used.

Average Load – Average daily wastewater load.

Maximum Load – Maximum week wastewater load.

5.3.2 Historical Load Data

Suspended solids are a measure of particulate and insoluble matter transported in the wastewater, the quantity of which is determined by filtering a sample of wastewater and weighing the material retained on a filter. Suspended solids loadings are dependent on factors such as community makeup and general standards of living. For example, studies in the United States have noted average increases of approximately 60% in suspended solids following the installation of kitchen sink garbage disposers.

Oxygen demanding substances consist of soluble and insoluble organic matter that, as a result of bacterial decomposition, causes the removal of dissolved oxygen from the wastewater. The quantity of oxygen demanding substances present in the wastewater is expressed as the BOD_5 of the wastewater.

Since January 2000, when a composite sampler (ISCO 3710R) began service, influent samples are collected as a flow-proportional composite of samples, taken throughout the day, from the influent Parshall flume. Prior to January 2000, the samples were taken as grab samples from the influent Parshall flume.

Currently, sampling and testing are conducted by plant staff once per week for biochemical oxygen demand (BOD_5) and total suspended solids (TSS) influent concentrations. In previous years sampling was accomplished on a monthly basis up until April 1999. Sampling increased to once per week from April 1999 through September 1999. From October 1999 through September 2001 sampling occurred on a three-times per week schedule, typically on Tuesday, Wednesday and Thursday. Since October 2001, sampling has decreased to only once per week. Sampling is typically conducted on the same day each week, Thursday.

Influent BOD_5 and TSS loadings are summarized for 2001 through 1998 in *Table 5.8*. Annual average BOD_5 influent loads for 2001 were 1230 pounds/day for dry-weather periods and 1379 pounds/day for wet-weather periods. Maximum monthly BOD_5 loads for dry-weather and wet-weather periods in 2001 were 1518 ppd and 1826 ppd, respectively. For TSS, the annual average loading was 1997 ppd for dry-weather periods and 2069 ppd for wet-weather periods for 2001. Maximum monthly average TSS loads were 2506 ppd and 2569 ppd for dry-weather periods and wet-weather periods for 2001, respectively.

The loading concentrations, biochemical oxygen removal (BOD_5) and total suspended solids (TSS), at the influent flume are measured weekly. These concentrations have been converted to loading rates and are shown for the last four years in *Table 5.8* and *Table 5.9 and Table 5.10,* below.

TABLE 5.8 - Historical Influent Load Data At The Treatment Plant

Parameter	BOD_5		Total Suspended Solids	
	Average (lb/day)	Max. Monthly Avg. (lb/day)	Average (lb/day)	Max. Monthly Avg. (lb/day)
2001 Wet-weather	1379	1826	2069	2569
2001 Dry-weather	1230	1518	1997	2506
2000 Wet-weather	877	1071	1015	1453
2000 Dry-weather	956	1133	1189	1906
1999 Wet-weather	476	615	720	1012
1999 Dry-weather	534	623	835	988
1998 Wet-weather	543	822	692	916
1998 Dry-weather	553	688	941	1284

TABLE 5.9 - Measured Influent BOD_5 Loading Values for 2001

Flow Characteristic	BOD_5 ppd	Population /EDUs	BOD_5 (ppcd)	BOD_5 (ppd/EDU)
Dry-Weather				
Average Load	1230	4210/2105	0.29	0.58
Monthly Maximum Load	1518	4210/2105	0.36	0.72
Daily Maximum Load (May)	2103	4210/2105	0.50	1.00
Wet-Weather				
Average Load	1379	4210/2105	0.33	0.66
Monthly Maximum Load	1826	4210/2105	0.43	0.87
Daily Maximum Load (Dec.)	3153	4210/2105	0.75	1.50
Annual				
Average Load	1305	4210/2105	0.31	0.62

TABLE 5.10 - Measured Influent TSS Loading Values for 2001

Flow Characteristic	TSS ppd	Population /EDUs	TSS (ppcd)	TSS (ppd/EDU)
Dry-Weather				
Average Load	1997	4210/2105	0.47	0.95
Monthly Maximum Load	2506	4210/2105	0.60	1.19
Daily Maximum Load (May)	8023	4210/2105	1.91	3.81
Wet-Weather				
Average Load	2069	4210/2105	0.49	0.98
Monthly Maximum Load	2569	4210/2105	0.61	1.22
Daily Maximum Load (April)	6182	4210/2105	1.47	2.94
Annual				
Average Load	2000	4210/2105	0.47	0.95

5.3.3 Load Projections

The total wastewater loads are expected to increase roughly proportional to the increase in population. The per capita loads used to project future loads were based upon standard values of 0.18 lb of BOD/PE/day and checked against historical plant monitoring records. The per capita loads and resulting load projections for the 20-year planning period are shown in *Table 5.11,* below.

TABLE 5.11 - Wastewater Load Projections

Year	Population	BOD		TSS, Suspended Solids	
	Pop. Equiv. (PE)	Annual Average (lb/day)	Max. Month Average (lb/day)	Annual Average (lb/day)	Max. Month Average (lb/day)
2001	5,600	1,000	1,500	1,300	1,900
2005	6,200	1,120	1,670	1,410	2,100
2022	9,500	1,720	2,500	2,000	2,900

Wastewater loads (BOD_5 and TSS) entering the treatment plant are dependent on population, commercial/industrial customers and transient users. Therefore, it can be assumed that future loadings will increase with area growth. As discussed in *Section 3*, the service area growth is projected to increase at an average annual growth rate (AAGR) of 113 persons per year (PE increase of 132 PE per year), with an allowance for an additional 1,000 PE's for outside growth. The projection is for a 70% overall growth in the population over the next 20 years. Influent BOD_5 and TSS, as expressed in pounds per day, are also projected to grow a total of 70% (for BOD_5) and 53% (for TSS) increase in the 20-year planning period.

This concludes Section 5, Wastewater Characteristics. Having the background of Sections 4 and 5, we are now able to evaluate the collection system, which is the title of Section 6.

SECTION 6
COLLECTION SYSTEM EVALUATION

6.1 HYDRAULIC CAPACITY ANALYSIS

All flows to the City of Warrenton's Wastewater Treatment Lagoons must go through these four (4) critical stations; the 3rd / Main Court Pump Station (Original #1), the S.W. Alder Pump Station (Original #2), the Lagoon Pump Station (Original #3), and the N.W. Warrenton Drive Pump Station (Hammond "A").

The hydraulic capacity depends on the force main size (12"ϕ), and the pumping capabilities of these four (4) pump stations. Therefore, on February 2, 2000, HLB submitted to the City, a draft plan to test the stations and quantify the flows, under differing conditions.

This testing was directed by HLB and accomplished through a concerted effort by the City of Warrenton staff equipped with radios at each station. On April 4, 2000, measurements were observed and recorded from the new OCM-III Milltronics transducer and flow measuring equipment at the new 12" Parshall flume.

With all four (4) original pump stations on line, and with both pumps operating, the flow was measured at ***2,200 gallons per minute.*** Therefore, the amount of flow in the force main to the flume becomes the hydraulic capacity.

This testing resulted in two (2) additional conclusions.

Conclusion #1

It is apparent from this testing that the 12"ϕ force main discharging into the flume at the Lagoon Cell #1 will only be adequate for the short term, about 5-10 years. Since the hydraulic capacity is finite or limited, additional pumping from other areas that feed this force main, or larger pumps for upgrades of existing equipment will dictate an additional force main or replacement of this main with a larger main. By hydraulic analysis, the maximum capacity of this force main under the conditions it operates is limited to 2,400-2,500 gallons per minute. See *Section 6.3.4* below.

Conclusion #2

When we examined the 3rd / Main Court Pump Station (Original #1), it seemed to run excessively. Further examination revealed that it is overloaded from a capacity standpoint. The following *Table 6.1* highlights this.

TABLE 6.1 - CONTRIBUTORY FLOWS TO 3RD/MAIN COURT STATION (ORIGINAL PUMP STATION #1)

Date: 4/11/2001

Note:
1. These pump stations are direct contributors to ORIG #1, through gravity main system.
2. Need further hour meter data analysis from all stations/scheduled w/AJ.
3. The contributory flows are original design flows, and not hour meter data analysis.
4. The ORIG #1 pump station is observed to be in operation continuously during site visits.
5. All stations are dual pump design.

PUMP STATION	HP/PUMP	CONTRIBUTORY FLOW
Skipanon Station	10	1200 GPM
S.E. Anchor / 101 Station	5	200 GPM
S.W. 9th St. Station	5	150 GPM
E. Warrenton #1 Station	5	275 GPM
	***Possible design total seen by ORIG #1:**	**1825 GPM**

*Please note that this does not include I/I for the approximately 2 miles of gravity into ORIG #1 station.

From the foregoing, it becomes apparent that with the two (2) downtown stations (original #1 and #2) exceeding their service life, a new downtown station is needed.

6.2 INFILTRATION AND INFLOW (I/I)

A thorough evaluation of the current Infiltration/Inflow (I/I) problems within the City of Warrenton is beyond the scope of this Facilities Plan. This situation bears further investigation as to whether it is more cost effective to repair the collection and conveyance system or whether it is more cost effective to treat the additional and future I/I flows at the wastewater treatment plant.

During the preparation of this Facilities plan, there were indications of significant I/I problems. See the following Depth of Flows spreadsheet, *Table 6.2*.

TABLE 6.2 - DEPTH OF FLOWS IN SELECTED MANHOLES

Date: 11/4/1999 Time: 4-5x am
Crew: JGF/City

Weather Notes:

1. On the afternoon of the 3, area had a windy rain, however, it stopped at approx. 6:30 pm.

2. During the flow checking, the moon and stars were out, having not rained for approx 9-1/2 hours.

3. At the last MH near 7th and Fleet in Hammond, rain showers started.

4. NOAH Weather for the 3rd: avg of Feb 1-Feb 3 = 1.03 inches

TIME	MH LOCATION	FLOW VEL?	PIPE DIA.	FLOW DEPTH
4:10	E. Harbor / S.E. Heron	slow, effluent	10"	8.0"
4:20	2nd / Marlin Ave.	slow, effluent	10"	3.0"
4:40	Airport, Coast Guard, grass	fast, clear	8"	2.5"
5:00	3rd / Main Court	very fast, clear	12"	3.0"
5:05	S.W. Alder	fast, clear	10"?	4.75"
5:20	7th & Fleet, Hammond	very fast, clear	12"?	3.25"

OBSERVATIONS:

1. Rank of highest flows to lowest flows in influent manholes near stations:

   3rd / Main Court
   7th / Fleet
   S.W. Alder
   Airport
   E. Harbor / S.E. Heron
   2nd / Marlin Ave.

2. While it did not appear to be affected by high amounts of rainfall, it may have been tidally influenced. Tide at this time: approx: 1.77 ft. Source: Mobilgeographics

3. MH at the Airport was approx 300 -400' from the lift station. This MH had several additional pipes in it, some capped.

The results from this evaluation highlight areas to focus on for I/I reduction. This spreadsheet, generally characterizes the amounts of flow in selected manholes near pump stations on key routes of flow to the wastewater lagoons.

Further, two (2) additional studies of the I/I are included here as an indicator of the necessary repairs to the collection/conveyance system. An earlier study was done by Westech Engineering Inc. dated April, 1983. A more current, informal study was done by HLB & Associates, on February 2000. Both are helpful in determining where to direct maintenance/repair efforts.

The first additional study (by Westech), contained recommendations to reduce I/I which were never completely implemented per the proposed schedule in 1983. Therefore, a thorough evaluation of I/I should take place as a future study by the following means:

- Fully implement the Westech I/I recommendations
- Smoke test areas of high I/I, such as Hammond and the Airport
- Analysis of pump run time vs. basin flows/loadings to focus repair efforts

The second additional study (an informal study by HLB & Associates) contains an analysis of pump run time vs basin flows using values from the DEQ Table "Quantities of Sewage Flows."

A door-to-door interview with each of the owners/managers of all of the buildings in the Clatsop Airport Industrial area supplied the data necessary to complete *Table 6.3* below.

TABLE 6.3 - WASTEWATER FACILITIES PLAN I/I STUDY

Purpose: **Date: 2-17-00** **By: HLB/jgf**

To quantify the amount of I/I that the Airport pump station conveying to the treatment lagoons for analysis.
This is a complete inventory on 2-17-00 of all of the businesses/industrial area at the Airport.
Column "EST. BOD LOADINGS" and the fact that the Airport basin is approx. 55 Acres, is additional inf.

Basis:

Personal interviews with each of the building owners/residents at the Clatsop Airport. Reference used: Wastewater Engineering, by Metcalf & Eddy, page 28, 29, and "Quantities of Sewage Flows" Table 2, by DEQ. This table is found below.

Flowrates:

per person	Office	16	M/E
(gallons/day)	Factories	35	DEQ
	Industrial	16	M/E
	Boarding Scho	100	M/E
	Apartment	70	M/E

STRUCTURES/BLDG	CLASSIFICATI	#PEOPLE	EST. BOD LOADING (lbs)	EST. FLOWS (g.p.d)
W. Station/Pilots Loun	Office	8	1.60	128
Lektro Manufacturing	Factories	40	8.00	1400
Ast. Regional Airport B	Industrial	10	2.00	160
Twiss Air Service	Apartment	2	0.40	140
UPS Shipping center	Office	4	0.80	64
Pacific View Cabinets	Industrial	6	1.20	96
Ag Bag Manufacturing	Factories	10	2.00	350
US Coast Guard	Boarding Scho	170	34.00	17000
Over Bay House	Office	4	0.80	64
Precision Heating	Industrial	6	1.20	96
		260	52.00	19498
			say	20000
P #1 Capacity, gal/min. 190			actual flow for 14 days	280000

P #2 Capacity, gal/min. 210
Pump avg Capacity = 200
Pump run times from maintenance records

Date	PUMP RUN TIMES (HOURS) Pump #1	Pump #2
2/18/2000	11327.0	21794.8
2/4/2000	11266.0	21733.9
	61.0 hrs	60.9 hrs

61	hr avg	
60	min/hr	
3660	minutes of pumping time	
	TOTAL PUMPED IN PERIOD =	732000 gal
	actual flow for 14 days	-280000 gal
	I/I is runtime gal - act.	452000 gal
	Percentage of I/I @ AIRPT	61.75%

Gallons of actual flow per day being pumped by this station =	20000 gal
Additional gallons of I/I per day being pumped by this station =	32286 gal

This quantity of I/I is pumped by the 4 stations in the EWI, and the Original Station #1 @ 3rd/Main Court.

TABLE 2
OAR 340-71-220

QUANTITIES OF SEWAGE FLOWS

Type of Establishment		Column 1 Gallons Per Day	Column 2 Minimum Gallons Per Establishment Per Day
Airports		5 (per passenger)	150
Bathhouses and swimming pools		10 (per person)	300
Camps: (4 Persons per Campsite, where Applicable)	Campground with central comfort stations	35 (per person)	700
	With flush toilets, no showers	25 (per person)	500
	Construction camps — semi-permanent	50 (per person)	1000
	Day camps — no meals served	15 (per person)	300
	Resort camps (night and day) with limited plumbing	50 (per person)	1000
	Luxury camps	100 (per person)	2000
Churches		5 (per seat)	150
Country clubs		100 (per resident member)	2000
Country clubs		25 (per non-resident member present)	—
Dwellings:	Boarding houses	150 (per bedroom)	600
	Boarding houses – additional for non-residental boarders	10 (per person)	—
	Rooming houses	80 (per person)	500
	Condominiums, Multiple family dwellings — including apartments	300 (per unit)	900
	Single family dwellings	300 (not exceeding 2 bedrooms)	450*
	Single family dwellings — with more than 2 bedrooms	75 (for third & each succeeding bedroom)	450
Factories (exclusive of industrial wastes — with shower facilities)		35 (per per per shift)	300
Factories (exclusive of industrial wastes — without shower facilities)		15 (per person per shift)	150
Hospitals		250 (per bed space)	2500
Hotels with private baths		120 (per room)	600
Hotels without private baths		100 (per room)	500
Institutions other than hospitals		125 (per bed space)	1250
Laundries — self-service		500 (per machine)	2500
Mobile home parks		250 (per space)	750
Motels — with bath, toilet, and kitchen wastes		100 (per bedroom)	500
Motels — without kitchens		80 (per bedroom)	400
Picnic Parks — toilet wastes only		5 (per picnicker)	150
Picnic Parks — with bathhouses, showers, and flush toilets		10 (per picnicker)	300
Restaurants		40 (per seat)	800
Restaurants — single-service		2 (per customer)	300
Restaurants — with bars and/or lounges		50 (per seat)	1000
Schools:	Boarding	100 (per person)	3000
	Day — without gyms, cafeterias, or showers	15 (per person)	450
	Day — with gyms, cafeterias and showers	25 (per person)	750
	Day — with cafeteria, but without gyms or showers	20 (per person)	600
Service Stations		10 (per vehicle served)	500
Swimming pools and bathhouses		10 (per person)	300
Theaters:	Movie	5 (per seat)	300
	Drive-In	20 (per car space)	1000
Travel trailer parks — without individual water and sewer hookups		50 (per space)	300
Travel trailer parks — with individual water and sewer hookups		100 (per space)	500
Workers:	Construction — as semi-permanent camps	50 (per person)	1000
	Day — at schools and offices	15 (per shift)	150

* Except as otherwise provided in these rules.

As is highlighted in *Table 6.3,* the winter flows are comprised of 62% I/I. This is significant, as this pump station is at the east end of the five (5) pump stations that make up the East Warrenton Interceptor, which discharges into the downtown gravity system. All five (5) pump stations have to handle this flow and in addition, the last station in the East Warrenton Interceptor, the E. Harbor / S.E. Ensign Station (EWI-P/S #2) discharges into a gravity system that flows into an already overloaded station at 3rd / Main Court (Original #1). Altogether, the Airport I/I impacts six (6) stations.

Another significant factor is that the four (4) stations along the East Warrenton Interceptor are also placed in collection basins and not just used for conveyance. This would tend to increase I/I along this chain of stations.

As stated at the beginning of this section, a cost analysis must be made. However, it seems that the additional electrical costs for pumping, wear on the pumps, and the additional maintenance are incentives to investigate the areas where improvements are possible. It may be easier to upgrade the conveyance system and deal with I/I through treatment at the WWTP.

To more accurately assess the I/I issue at the airport, please note that this is a wintertime flow.

6.3 RECOMMENDED IMPROVEMENTS

6.3.1 Pump Stations

All pump stations were tested using the current DEQ Flow rate guidelines. A spreadsheet was developed per those guidelines, then measurements taken at each station.

Those measurements were then input into the specific spreadsheet for that station. This spreadsheet then became the basis for analysis of each pump station. Each station report also has at the end of it, a "RECOMMENDATIONS" section.

Those recommendations also form the basis for the recommended upgrades. See *Table 6.4* below.

6.3.2 Downtown Pump Station

A new station here would eliminate two (2) existing stations, one at 3rd / Main Court (Original #1), and one at S.W. Alder (Original #2). These stations have exceeded their useful design life. Upgrades would not be as cost effective as replacing these two (2) with a single downtown station located along N.W. Warrenton Drive.

The costs for this pump station are included in the treatment plant upgrade costs, and found in the Appendices, *Section C.*

6.3.3 East Warrenton Interceptor

This is a critical system to the collection and transportation of sewage to the Warrenton Wastewater Treatment Lagoons.

Evaluation of the improvements necessary to the five (5) pump stations that are part of this system are critical. This evaluation will allow the City of Warrenton to make a clear distinction between costs to upgrade their current collection system vs additional costs to add outside sources.

Any upgrades beyond those necessary to the existing system for outside sources (outside of the urban growth boundary) are therefore outside of the scope of this facilities plan.

During the winter of 2001-2002, of the five (5) pump stations, two (2) were found to be under capacity with two pumps running full time under a peak storm event. This is an indicator of inflow problems when the peak pump time coincides with the storm peak.

Therefore, all five (5) pump stations would require upgrades regardless of additional flows from outside sources. See *Table 6.4,* below.

6.3.4 Collection System – Force Mains and Gravity

Because the East Warrenton Interceptor system of five (5) pump stations is operating beyond design capacity, and contributing to over capacity in other stations, an additional force main should be connected to the E. Harbor / Ensign force main. This connection should be made at the west side of the existing river crossing and the additional force main would be routed north west to the existing force main. We anticipate the estimate size to be 6-8"ϕ. Costs are found in the downtown core conveyance system. Currently, after making the river crossing, the effluent flows by gravity into the 3rd / Main Court Station. It is discussed as a river crossing force main and the cost is included in the downtown core conveyance upgrades.

However, the downtown core conveyance also includes a gravity-fed pump station which eliminates two (2) existing stations, 3rd / Main Court and S.W. Alder. This new pump station could be temporarily connected to the existing force main. However, as a product of future growth, this pump station should have a new, separate, dedicated 12"ϕ force main which would then run parallel to the existing force main. Pump station costs are included in *Appendix C.*

This new, separate, dedicated 12"ϕ force main would take the pressure off of the existing force main. It would allow other developable areas to be sewered and pump into the existing force main without unnecessarily large pumps or pumping costs. In short, it would allow for additional hydraulic capacity by force main into the wastewater treatment plant. The costs for this new, dedicated force main are included in *Table 6.4.*

A situation has developed within the collection system whereby multiple pump stations are connected to a single force main. This situation occurs in the Hammond/Fort Stevens area and the 2nd / Marlin Avenue Pump Station (EWI P/S #3) on the east Warrenton influent. This is damaging to the collection system infrastructure.

Through preliminary hydraulic modeling, we have observed results greater than the accepted standard velocities of 2 ft/sec – 10 ft/sec. With the sandy basins that these stations operate in, and the additional loadings that the City is currently seeing at the Wastewater Lagoons, these pump station force mains should be separated out into additional gravity and pressure mains.

The estimated costs for all collection system improvements are presented below in *Table 6.4.* Please note that this table includes costs for both pump station upgrades, maintenance, force mains, and gravity mains.

TABLE 6.4 – City of Warrenton Operational Upgrades*	
	Facilities Plan/2002
PUMP STATIONS	**$1,545,000.00**
Does not include new downtown station	
COLLECTION SYSTEM	
Ft. Stevens gravity sewer	$710,000.00
River crossing from, elsewhere in FACILITY PLAN	
2nd / Marlin Avenue force main	$195,000.00
S.E. Marlin/101 force main	$200,000.00
Ensign discharge to new downtown station	$230,000.00
Shilo-101 force main	$195,000.00
Parallel 12" diameter force main	$700,000.00
OPERATIONAL UPGRADES	**$3,075,000.00**
(as of 3-1-02)	**$3,800,000.00**
*Notes:	
1. This is a preliminary cost analysis based on needed improvements. 2. City forces should inventory manholes to repair grade rings etc., cost not included here. 3. Other improvements north of Harbor or east of 101 would add additional cost to this preliminary estimate, these additional areas are not part of the operational upgrades necessary at this time.	

6.3.5 Infiltration and Inflow (I/I) Removal

The City needs to enforce I/I at the Airport and Hammond area. I/I problems at the Airport have not been thoroughly addressed.

An optional program is as follows:

Priority #1: Control I/I flows in Airport Basin

1. Complete smoke test repairs.
2. Measure wet weather flows at pump station and at key manholes. Locate lines contributing significant I/I.
3. TV inspect identified lines during wet weather. Inspect manholes for leakage.
4. Repair deficiencies found in Step #3.
5. Repeat Steps #2-4 until peak flows from the Airport Basin = 20,000 g.p.d. or less.

Priority #2: Control I/I flows in the remaining east Warrenton Basin

1. Measure wet weather flows at pump stations and at key manholes. Locate lines contributing significant I/I.
2. TV inspect identified lines during wet weather. Inspect manholes for leakage.
3. Repair deficiencies found in Step #2.
4. Repeat Steps #1-3 until peak flows from the entire east Warrenton area are reduced to 100,000 g.p.d. or less.

Priority #3: Control I/I flows in remaining basins

1. Monitor STP influent flows and pump station operations to determine the amount of I/I remaining in the system.
2. If I/I flows are still significant, take Steps #2-4 as outlined in Priority #2 above. Specific needs and time schedules depend on the completion time for Priority #2, as well as the remaining problem's magnitude.

The City will need to evaluate I/I removal work that has been completed to date to determine at which stage to proceed with the program. The priorities stated have not changed and have been reconfirmed by correlation of the observed response time of pump stations to the peak in storm events.

This concludes the collection system evaluation. Now, the Facilities Plan Report shifts to treatment of waste flows in Section 7, Wastewater Treatment Evaluation.

SECTION 7
WASTEWATER TREATMENT EVALUATION

7.1 EXISTING FACILITIES

The existing sewage treatment system, *Figure 7.1,* consists of a two (2) cell stabilization lagoon, currently operated in series, followed by disinfection by chlorination. In March of 2000, construction of the Sewer Lagoon Improvements Project was completed. The project consisted of the following:

- Relocate 12" diameter force main into treatment plant
- Construct a new influent (Parshall) flume (flows frequently exceeded capacity of existing flume)
- Install influent flume flow monitoring equipment
- Install an influent flume composite sampler
- Transfer pipe modifications with floating inlet to transfer pipe
- Install a floating baffle in Cell #2 to redirect flow through cell preventing "short-circuiting"

These improvements to the existing lagoon system were evaluated and documented in a Performance Evaluation Standards (PES) Manual prepared for the project by HLB & Associates, Inc. (HLB), dated September 2000.

Other improvements proposed for the existing lagoon system consist of aeration and biosolids removal. Aeration was addressed in a pre-design report prepared at the request of the City by HLB in January of 2000. It was determined at that time that aeration improvements could not take place until biosolids (sludge) were removed from the lagoon restoring the water column. It was later determined by H. R. Esvelt Engineering that the previously recommended addition of aeration was no longer acceptable due primarily to its inability to accommodate future growth.

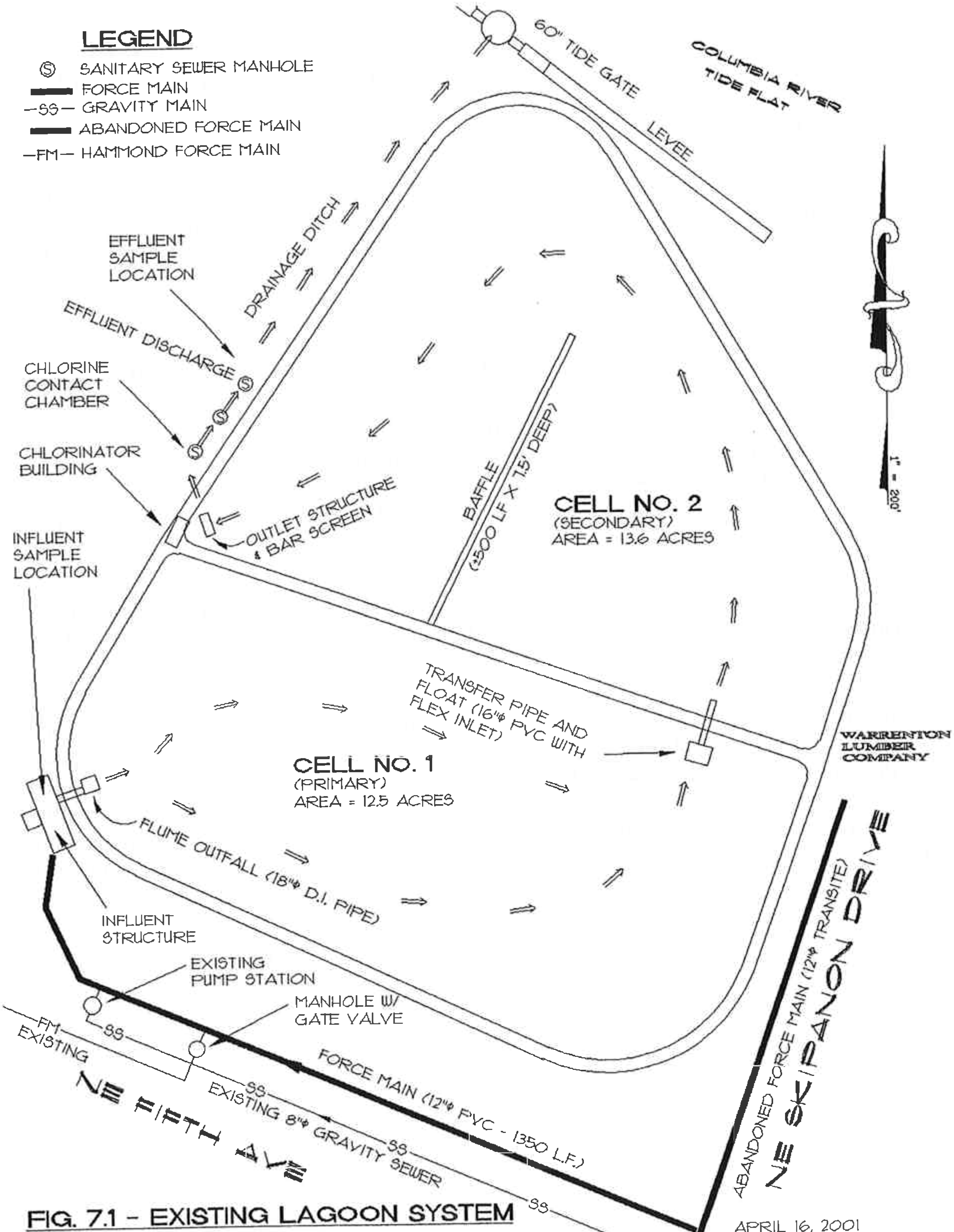


FIG. 7.1 - EXISTING LAGOON SYSTEM

SCALE: 1" = 200'

DRAWING: 3487-1300-06-04.DWG

7.2 PERFORMANCE EVALUATION

7.2.1 NPDES Permit Conditions

Warrenton's National Pollutant Discharge Elimination System (NPDES) Permit (No. 100874) was originally issued on April 10, 1992. A copy of the permit can be found in *Appendix G* of this report. The permit authorized the City to construct, install, modify or operate wastewater treatment control and disposal facilities and discharge adequately treated wastewater into the Columbia River.

The permit expired on March 31, 1997, but has remained in effect since the City has applied for renewal. Following are the waste discharge limits not allowed to be exceeded by the permit.

OUTFALL NUMBER 001 (Lagoon Discharge)
Applies all year

Average Effluent Concentrations			**Effluent Loadings**		
			Monthly	**Weekly**	**Daily**
Parameter	**Monthly**	**Weekly**	**Average**	**Average**	**Maximum**
	mg/L	**mg/L**	**lbs/day**	**lbs/day**	**lbs**
BOD	30	45	112	169	225
TSS	50	80	188	300	375
FC/100ml	200	400			

Other Parameters	
PH	Shall be within the range 6-9
BOD removal efficiency	Shall not be less than 85% monthly average
TSS removal efficiency	Shall not be less than 65% monthly average
Average Dry Weather Flow	0.45 MGD

The permit also outlines minimum monitoring and reporting requirements (*Schedule B*). Currently there is an ISCO 3710R refrigerated sampler that collects a flow proportional influent sample from the Parshall flume. Effluent samples are collected from the effluent weir box located after the chlorine contact chamber. The City Staff then collects all samples and then sends off for testing. Test results are logged into a daily monitoring report (DMR) and submitted to the DEQ on a monthly basis or more frequently if required.

Schedule C of the permit outlines requirements for a sludge management plan an infiltration and inflow reduction program, plans, specifications and a construction schedule for corrections of short-circuiting within the lagoon cells and a requirement that the permittee submit a notice of compliance or noncompliance within the established schedule.

7.2.2 Mutual Agreement and Order (MAO)

The new discharge limits do provide a margin of safety against violations, but no buffer for increased loadings.

It is feasible for the City to install capacity improvements as temporary or interim measures, enabling the City to accept additional waste loadings. DEQ offered penitential interim improvements that could enable the lagoons to treat additional sewer connections. These potential improvements along with other options were examined by HR Esvelt in the Interim Capacity Increase Technical Memorandum that can be found in *Appendix A*.

Interim Improvements

The City of Warrenton has requested that the Oregon Department of Environmental Quality (DEQ) approve an increase in the flows and loadings to the existing wastewater treatment facility on an interim basis. The recommended improvements to the City's lagoon treatment facilities recommended in this report are intended to provide the additional interim treatment capacity needed for treatment of added waste loads from Miles Crossing, Fort Clatsop and City growth, while meeting the interim effluent requirements as agreed to in the Mutual Agreement and Order (MAO), dated December 24, 2001.

The following table summarizes the list of improvements that will be required to complete the interim capacity upgrades. The improvements include pump station upgrades required to convey Miles Crossing effluent from the point of connection to the treatment plant.

Mechanical plant improvements	$555,000.00
Pump station improvements	
-Airport (E. Warrenton Interceptor Area)	$960,000.00
Marlin Avenue force main replacement *	$200,000.00
Biosolids removal**	$480,000.00
Total	$2,195,000.00

*The actual cost of pump station upgrades is dependent upon the amount of I&I removal at the airport, and tributary areas.
**Biosolids must be removed by September 2003.

It is critical that the inflow and infiltration (I&I) at the airport be reduced prior to implementation of the proposed interim improvements. The design of the pump station improvements at the airport will be based on design flows that will be effected by the percentage of I&I removal.

It is assumed at this time that the City of Warrenton will be receiving sewer flows from the Miles Crossing Sewer District. If this assumption changes, the cost will be less since infrastructure improvements would not be required at the time of the interim improvements.

The MAO defines interim waste discharge limits that are to be in effect until full operation of the facility has been achieved. The limits are as follows.

Interim Limits for the City of Warrenton Wastewater Treatment Facility					
All Year Round					
Outfall Number 001 (Lagoon Discharge)					
	Avg. Effluent Conc.		**AVERAGE**		
Parameters	**Monthly**	**Weekly**	**Monthly**	**Weekly**	**Daily**
	mg/L	**mg/L**	**lb/day***	**lb/day***	**lb/day***
BOD	75	100	469	704	938
TSS	75	120	469	704	938

The following table summarizes the wastewater improvements and corresponding scheduled outlined in the Memorandum of Agreement and Order.

MAO Schedule of Events:

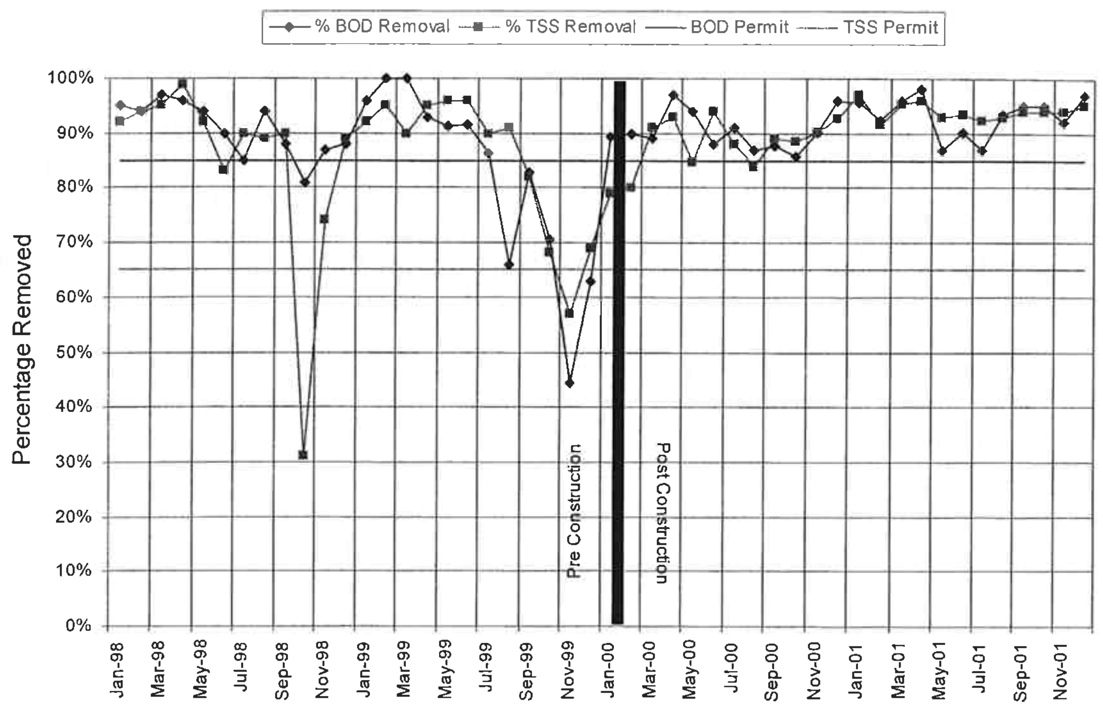
Date	Occurrence
April 10, 1992	DEQ issued a National Pollutant Discharge Elimination System Waste Discharge Permit No. 100874 (Permit) to the City of Warrenton.
March 21, 1997	The Permit expired. The Permit has remained in effect and is in effect on this date as the City has made timely application for renewal.
March 29, 1999 & February 9, 2001	DEQ issued a Notice of Noncompliance to the City for Permit violations.
March 29, 1999	Notice of Noncompliance addressed a lapse in reporting, high Fecal Coliform and BOD & TSS percent removal violations.
April 1, 2002	(or within 4 months) of having been provided by DEQ with proposed effluent limits and other proposed conditions to be included in renewed Permit, whichever is later, submit for DEQ review and comment a draft Wastewater Facilities Plan.
Within six (6) months of the signing of this MAO	City shall conduct a One-Stop Meeting

Prior to December 31, 2002	Once funding sources have been identified the City shall submit all appropriate funding applications
January 1, 2002	City anticipates its new user rate methodology will be effective
After May 30, 2002	City will allow for Public Input and Public Review process with respect to recommendations in the Draft Facilities Plan. City may prepare and submit for DEQ approval, any Interim engineering Study for proposed interim improvements to existing lagoons and timelines needed to provide capacity to allow additional waste loads during the term of the MAO
By September 30, 2002	Submit for DEQ approval a final plan
Within three (3) months of approval of the Plan	City shall submit a biosolids management plan that is consistent with approved Plan
By April 1, 2003	City to submit for DEQ approval a Predesign Report, conforming to DEQ requirements for such documents, consistent with the approved Plan
Within 243 days of DEQ approval of Predesign Report, but no later than December 31, 2003	City to submit for DEQ approval, Plans and Specifications (P&S) consistent with approved Predesign Report
Within 151 days of DEQ approval of the Plans & Specifications	Award contract(s) for construction of approved P&S
Within fifteen (15) months of the awarding of contracts	Complete construction of approved P&S
Within three (3) months of complete construction	Achieve full operation of new facilities so as to meet consistently all Permit limits, Minimum Design Criteria and applicable water quality standards

7.3 EFFLUENT QUALITY, DISCHARGE LIMITS & REMOVAL RATES

A chart of discharge limits and removal rates of the existing treatment plant both prior to and following the 2000 Sewer Improvements showed an increase in removal rates See *Figure 7.5,* below. Discharge limit violations continued to occur even after the improvements due to increasing BOD and TSS influent loadings. There were six (6) discharge limit violations of the NPDES permit requirements in the year 2000, primarily during dry weather conditions during the months of June, 2000 through October, 2000. The City of Warrenton received a second Notice of Non-Compliance on February 9, 2001.

Figure 7.5 - BOD - TSS Monthly Average Removal Efficiencies



7.4 DESIGN CRITERIA

Development of wastewater treatment system improvements are based primarily on projected (year 2023) flow and organic loadings developed in *Section 5*, existing system deficiencies, and compliance with state and federal law.

The intent of the treatment system improvement alternatives is to provide adequate handling and treatment of year 2023 design flows and loadings so as to comply with the more stringent requirements of either the current NPDES Permit or the North Coast – Lower Columbia Basin water quality standards. The proposed treatment system and the proposed core pump station upgrades will accommodate the projected peak hourly flow of 4.7 mgd so as to eliminate overflow of raw sewage under severe storm event conditions.

The most recently developed design (year 2023) criteria are from the *Request for Interim Capacity Increase, Technical Memorandum* (*Appendix A)* and include population equivalents from the Clatsop County Correctional Facility and the Miles Crossing Sanitary Sewer District. The design criteria is summarized in *Table 7.1*, below.

TABLE 7.1 - DESIGN CRITERIA DEVELOPMENT

Design Data	Current	Combined at WTP startup	2023 New Areas	w/ New Areas Design Year[6] 2023
POPULATION EQUIVALENTS	5,600[1]	6,200[5]	1,000	9,500[2]
FLOWS, million gallons/day, mgd				
Annual average	0.70	0.76	0.10	1.1
Maximum Month WW Avg	1.1	1.05	0.13	1.6[4]
Maximum day	1.5	1.6	0.16	2.3[4]
Hydraulic, PIF	3.4[3]	3.5	0.25	4.7
LOADING, pounds per day				
BOD, annual average	1,000	1,120	180	1,720[5]
max mon avg, summer	1,500	1,670	240	2,500
TSS, annual average	1,300	1,410	180	2,000
max mon avg	1,900	2,100	240	2,900
Ammonia, max mon avg	150	170	23	250

NOTES:

1. Current population equivalents (PE) based upon annual average BOD_5 loading of 1,000 pounds per day divided by contribution of 0.18 lb BOD/PE/day
2. 8,500 population equivalents calculated from: projected average growth for Warrenton (from HLB & Assoc.) over the next 22 years, times 5,600 PE, plus 1000 PE from new service areas.
3. Peak flow estimated from influent flow measurement circular charts with factor added for existing undersized pump stations.
4. Increase in flows are not proportional to population growth since new sewers will be much tighter with much lower infiltration and inflow than existing.
5. Includes City growth, Clatsop County Corrections Transitional Facility and Miles Crossing Sanitary Sewer District to be connected prior to City's WWTP startup.
6. Includes future City service area growth and new service areas (Miles Crossing Sanitary Sewer District and Fort Clatsop).

Note: Table by H. R. Esvelt Engineering

7.5 SITE CONSIDERATIONS

7.5.1 Alternate Locations

Because of topography and the layout of the exiting wastewater collection system, there are no alternate WWTP sites that merit further consideration. The existing WWTP site is linked to the collection system via the influent pump station and its 12" force main. Additional City land does exist to the west of the existing site, however, development of this land as additional lagoon area would not meet the required needs and additional land area would not be needed if an SBR option was chosen. This area to the west of the existing site is approximately 42 acres and is designated as wetlands.

7.5.2 Existing Site Characteristics

The City of Warrenton owns Tax Lots 2403 and 2501, located in Section 15, Township 8 North, Range 10 West. The land areas are 51.02 acres and 7.41 acres respectively and the properties are zoned General Industrial (I1). The total land area owned by the City is 58.43 acres.

The current Lagoon system, (cells, dikes, roads, fences, etc.) occupies approximately 38 acres of Tax Lot 2403. It is somewhat misleading to say that Cell #1 occupies 12.5 acres and Cell #2 occupies 13.6 acres. This would total only 26 acres. This is only the area of free water surface, and does not account for the rest of the system, including the dikes, roads and chlorine contact chamber.

If the gross area of the current system is subtracted from the land owned by the City of Warrenton, the resultant land for expansion of the system is only 23 acres. Out of those 23 acres, not all could be used for free water surface. Some of the 23 acres would be used for dikes, roads, fences, etc. This situation severely restricts use of the wetlands to the west. We would estimate that only 14 acres or less would be available for lagoon expansion.

Any additional land, beyond the approximately 23 acres available, would have to be purchased, have a wetlands determination, a wetlands delineation, and a final wetlands concurrence by the Department of State Lands, (DSL) and US Army Corps. of Engineers, (USCoE).

The existing site is adjacent to the tide flat of Alder Cove and is separated from Alder Cove and the Columbia River by a dike. Treated effluent enters Alder Cove through a 60" diameter tide gate located approximately 400 feet northwest of the northwest corner of Cell #2. Effluent then flows across the Alder Cove tide flat and enters the Columbia River.

7.6 IMPROVEMENT ALTERNATIVES

Currently the existing lagoon system is overloaded as described in *Section 5* of this report and loading will only increase over the next 20 year planning period. In April of 2001, HLB and H. R. Esvelt Engineering completed a Technical Memorandum for Sewer Lagoon Upgrade Alternatives. During the development of this report, three (3) alternatives were examined.

1) Alternative No.1 investigated the possibility of expanding the existing treatment system lagoons into the City-owned property to the west of the existing lagoons. This alternative was deemed not feasible after discussion of the wetlands considerations with representatives of the US Army Corps of Engineers (USCoE).
2) Alternative No.2 was a reconstruction of the lagoon system to an aerated lagoon, with settling ponds and wetlands.
3) Alternative No.3 consisted of a sequencing batch reactor (SBR) system.

Based on the information available at that time, and conversations with DEQ, H. R. Esvelt Engineering recommended Alternative No.2, an aerated lagoon. Since that time, additional outside sources requesting acceptance of their effluent by the City of Warrenton and further evaluation of potential growth have made it clear that Alternative No.3, an SBR system, is the most beneficial option for a new treatment system.

The Department of Environmental Quality (DEQ) issued a Notice of Noncompliance for NPDES Permit violations on March 29, 1999 and on February 9, 2001. DEQ believes that the City is having difficulty meeting the NPDES Permit limits because the facility remains overloaded by influent BOD and TSS. The DEQ further believes that due to the overloading, the facility will likely continue to violate discharge limits. Due to these issues and the fact that the DEQ and the City wish to limit any past and future violations, they entered into a Mutual Agreement and Order (MAO). [Source: Mutual Agreement and Order, State of Oregon Environmental Quality Commission and City of Warrenton. See *Appendix H* for a copy of the MAO.]

Do to the possible expansion of the sewer service areas and the MAO, the April 2001 Draft Technical memorandum is no longer valid. A revised technical memorandum has been completed for a secondary wastewater treatment plant using the batch extended aeration activated sludge process (SBR) (See Appendix C).

7.6.1 Alternative No.1 – Expand Existing Lagoons to the West

A preliminary discussion with the Portland office of the US Army Corps of Engineers (USCoE) indicates the following information relative to any use of the wetlands areas west of the existing lagoons [Personal conversation, Jeff Harrington, PE, HLB & Associates, Inc. with Teena Monical, USCoE]. This discussion determined that if an application to construct a wastewater lagoon in the 40 acres of wetland directly west of the existing lagoon were submitted, the Corps would push very strongly for an alternative site. The City of Warrenton would need to demonstrate that this is the only practical alternative site. Cost would be a factor, but they would strongly encourage an alternative site, even if the alternative site had a higher cost. If the wetlands area were to be developed, there would be three mitigation options available. The mitigation options consist of:

1) restoration of an existing wetlands (at a 1:1 ratio),
2) enhancement of an existing wetlands (at a 3:1 ratio) and
3) creation of new wetlands (at a 1.5:1 ratio).

Other factors considered by the Corps in evaluating the project would be the expected life of the alternative site and potential growth within the City's Urban Growth Boundary.

The Portland office of the USCoE is very familiar with the wetlands in the Warrenton area and is aware that Warrenton has great growth potential. The Corps believes (as does this consultant team) that the City of Warrenton should be planning for growth and a treatment plant that can accommodate the level of growth that Warrenton may experience in the future. Our discussion also touched upon the Warrenton Wetland Conservation Plan and the Corps staff pointed out that the purpose of the Plan is to allow development (even in areas of wetland that have been identified as less valuable, such as isolated wetlands) while complying with the 404 requirements.

Based upon the foregoing discussion with the USCoE staff, the alternative of expanding the lagoons was not further considered as a viable alternative, due to the following reasons:

1) The proposed expansion area is a regulated wetlands that regulations will not allow to be destroyed. The estimated costs for acquisition of alternate mitigation sites and construction of wetlands mitigation on those sites are judged to be cost-prohibitive.

2) The existing 26.1 acres of sewer treatment ponds are currently ≈200% over capacity, therefore, even if lagoons could meet future effluent requirements, there is no room for current loading let alone future loading.

3) Lagoon wastewater treatment alone cannot meet future increased discharge requirements.

For the above reasons, this Alternative No.1 was judged to not be a viable alternative and Alternative No.2 was evaluated.

7.6.2 Alternative No.2 - Modify Existing Lagoon System (Aerated Lagoon)

Alternative No.2 consists of adding influent fine screening to the Influent Structure, followed by flow measurement in the existing 12" throat Parshall flume and discharge to the aerated lagoon, followed by a settling pond with some aeration and a constructed free surface wetland. An in-channel fine screen, at the influent structure of the plant (preferably in the existing channel), is necessary to remove rubber and plastic products out of the wastewater to keep the lagoons presentable, to prevent binding up and/or clogging of the aerators and pumps, to keep them out of the receiving stream and out of the sludge which will eventually be land applied onto farmland. The screenings are washed and compacted so they are acceptable for disposal as solid waste.

An aerated lagoon with 8 days detention time is required for treatment of the projected loadings to the plant at winter temperatures. Using the projected 2023 maximum monthly average flows (1.6 million gallons per day, mgd) the required detention volume will be 20 million gallons. The basin will be 12 feet deep with 2 feet of freeboard with mixing to provide a complete mixed basin for full utilization of the required detention volume. Aspirating aerators are recommended to provide adjustable-direction mixing to keep the basin contents moving while meeting the oxygen demand. A concrete effluent structure with a fixed weir will maintain the desired water level. The aerated lagoon is proposed to be constructed in the west half of Cell #1.

The east half of Cell #1 would become a settling pond. The cell is currently lined with a bentonite liner and as long as the liner is not disturbed, it will be acceptable for future use. Monitoring wells may be required by DEQ for monitoring groundwater at the property line to ensure the lagoons are not leaking. (Reference: personal communication with David Mann, P.E., DEQ, April 6, 2001). The recommendation is to move the floating baffle installed in early 2000 in Cell #2 to Cell #1, at the time that Cell #2 is taken out of service for wetlands construction.

Aeration would be provided in the first part of the settling pond to keep the contents partially mixed and aerobic. This settling pond would provide for settling and storage of sludge and would require removal (biosolids removal, since the sludge would be stabilized, after one year, to meet requirements of 40 CFR, Part 503 for Class B biosolids for land application). Annual removal of the sludge, each summer, would be the preferred method of sludge disposal, if the City buys sludge removal and application equipment. This will keep the annual task of sludge removal from overwhelming the City staff (which may occur if the sludge removal is done every few years). A second alternative is for biosolids removal every few years by contract.

The settling pond effluent is proposed to flow through a flow control structure. The structure will include level control for the settling pond, bypass lines for maintenance of the settling pond or wetland and flow splitting to the two wetlands treatment cells.

The constructed free water surface wetlands will provide effluent polishing with additional carbonaceous waste, BOD removal by bacterial communities resident in the wetlands and plant uptake, removal of algae by the effluent passing through the plants, and settling of solids, as well as an added benefit of nutrient removal. A properly designed and constructed wetland in a coastal climate can provide a high level of treatment year around. The wetlands would be designed with two (2) flow paths, for parallel operation, approximately 6.0 acres in each. There is not adequate elevation head available for flexibility to provide series operation (changing elevation head would require raising existing grades, including bottoms of ponds, which would be cost prohibitive). The design would include alternating open water, deeper sections and planted shallower sections.

Chlorine disinfection is proposed to treat effluent from the wetlands. Contact time will be provided in chlorine contact basins prior to discharge through the existing outfall to the drainage ditch. Each pipeline to the basin will go through a manhole ahead of the flash-mixing chamber. A submersible pump can be dropped into one of the manholes to pump the pipe dry for maintenance. The flash mixer will quickly mix the chlorine solution into the treated effluent. The contact basin will be in two flow paths so each side can be taken off line periodically to pump any accumulated solids back to the aerated lagoon. Dechlorination equipment is included (dechlorination feed equipment and flash mixing). Effluent flow measurement and flow-paced, refrigerated composite sampler are included.

Note that Ultraviolet (UV) disinfection is not recommended for Alternative No.2, since during certain times of the year effluent turbidity from a wetland inhibits transmission of the UV light.

Alternative No.2, Aerated Lagoon, Settling Pond and Wetland, can be phased by installation of the Aerated Lagoon and the Operations/Electrical Building, in an initial phase with Wetlands, Settling Pond and Chlorine Contact Basin improvements constructed in a future phase. The DEQ would like the City to implement the first phase as soon as possible and the phasing should help accommodate this request. A first phase would not include a standby generator and the operation of the remainder of Cell #1 and Cell #2 would be unchanged.

It is our opinion that some effluent violations will continue, even with the addition of the Aerated Lagoon, since the current discharge requirements cannot be achieved 12 months per year without additional treatment modifications following the aerated lagoon. The additional treatment is required to: provide for secondary treatment process-level BOD removal and remove algae to meet the TSS average concentrations. These limits will be met most of the year, but violations will continue with shallow, facultative lagoons following an aerated lagoon. Multiple-depth draw off points will not provide an effluent that meets discharge requirements year around, since the lagoons are shallow. Therefore, construction of the wetlands in Cell #2 should closely follow phase 1 completion.

7.6.3 Alternative No. 3 - Sequencing Batch Reactor (SBR) System

This alternative is developed as a full secondary treatment level process to meet more stringent biochemical oxygen demand (BOD) and total suspended solids (TSS) effluent concentration limits, with ammonia removal and maximum flexibility for future growth.

The Influent Structure modifications are proposed to be the same as for Alternative No.2. From the Influent Structure, the flow will enter a valve vault with motorized automatic valves. The valves divert the flow to the SBR aeration basin in service (the basin that is in the aeration/fill mode of the batch cycle).

Influent from the valve vault enters one of the three SBR aeration basins by gravity flow. Each aeration basin is proposed as an earthen-diked basin, lined with an HDPE liner and a concrete slab bottom. Each basin will include two high speed floating surface aerators, a series of level controls that can override the timed cycle control program and one floating effluent decanter. Following the decanter is a redundant motorized butterfly valve contained in a manhole for access. Effluent is discharge from each SBR aeration basin in a gravity line to the chlorine contact basin.

This alternative includes construction of the SBR aeration basins in the West end of Cell #1. The remainder of Cell #1 would be converted into two (2) sludge-holding/stabilization lagoons. Each year solids would be removed from one of the sludge holding lagoons that has not been in service (sludge has not been wasted to it) during the previous 6 months, to insure the biosolids meet the requirements of 40 CFR, Part 503 for Class A biosolids.

Ultraviolet (UV) disinfection is proposed for this alternative. UV is an acceptable alternative method of disinfection for the proposed activated sludge process. The UV disinfection process, as proposed, would include: New concrete channel with roof, hoist and lift for removing the UV modules from the channel for cleaning and other maintenance. The UV disinfection process would also include low pressure, low intensity UV equipment, effluent flow measurement and a refrigerated, flow-paced, composite sampler.

The SBR Extended Aeration Activated Sludge, secondary treatment plant could be phased by later adding the Sludge Holding Lagoons dividing dike (splitting the lagoon into two (2) basins) and installing UV disinfection, in future upgrades.

In summary, Alternative No.3, the SBR Extended Aeration Activated Sludge System was selected for the following reasons:

1) Alternative No.2 cannot meet possible future ammonia limits consistently.
2) Alternative No.2 may not meet tighter discharge limits forthcoming from DEQ consistently during certain times of the year.
3) Alternative No.2 would use all the land area for current service area over the next 20 years. This does not provide expansion capability for future growth or faster growth or service to outside service areas.
4) Alternative No.2 does not provide for a treatment process that can be improved in the future for possibly more stringent future limits.

7.6.4 No Improvement Alternative

The No Improvement Alternative is not applicable to this project since the City has agreed to the schedule outlined in the Memorandum of Agreement and Order between the City and DEQ. Without improvements to the treatment system, discharge violations would continue and the City's economic future would suffer from the imposed sanitary hookup moratorium.

7.6.5 Other Alternatives

Continuous flow alternatives were not considered to be cost effective for further evaluation. This is due to the relatively expensive capital cost requirements to construct concrete clarifiers and return/waste sludge pumping facilities on the soft soils found at this site. Such alternatives were judged to be prohibitively expensive and were not considered further.

7.6.6 Selected Alternative

For a detailed description of the SBR system see Chapter 3 of *Appendix C*.

7.7 CORE CONVEYANCE SYSTEM IMPROVEMENTS

The existing, original #1 pump station (capacity of 300 g.p.m.) is seriously overloaded and runs continuously during periods of heavy rainfall. This pump station serves a large area of the downtown in addition to receiving sewage flows from an existing upstream pump station (existing Skipanon Station) with a capacity of 1200 g.p.m. Also, the existing, original #2 pump station at SW Alder is failing.

Any upgrade or replacement of these pump stations should be matched to the proposed lagoon improvements.

To correctly balance the chosen wastewater treatment alternate with the infrastructure improvements, certain improvements should be undertaken, concurrent with construction of the lagoon treatment improvements. These improvements will also serve the projected growth for the 20 year planning period.

During construction of the chosen alternative to the wastewater lagoon, a separate contract should be let to accomplish core conveyance upgrades to the downtown system.

Specifically, these upgrades are:

1) addition of a new "Downtown" pump station, increasing discharge force main size from the new station to existing force main,
2) construction of two new gravity sewers,
3) demolishing two 33 year-old, and by now, sub-standard pump stations,
4) re-routing of a 6" diameter force main alignment from a pump station east of the river, to attach to existing 12" diameter force main.

Each improvement is discussed in detail below. Please see *Figure 7.6*, PROPOSED CORE INFRASTURCTURE IMPROVEMENTS, immediately following this discussion.

Item #1 - New "Downtown" Pump Station. The original #1 pump station, located at the intersection of SW 3rd and SW Main Court, is under-sized for the existing flows and is 33 years old. The planning period and useful life for pump stations is typically 20-25 years before upgrades. This station has served the community well, but now, due to higher flows and increased development, must be upgraded or replaced.

A new energy efficient "Downtown" pump station should be added near the intersection of SW Main Court and NW Warrenton Drive. This station would have variable frequency drive controlled pumping with telemetry controlled at the Public Works Building. By the addition of this larger station, the inflow to the treatment plant will be better controlled at the newly designed headworks for the chosen alternative. This addition will also accommodate the larger flows currently being seen in the downtown area, while being able to accommodate future flow increases from the peripheral areas under consideration for development. The new station will accommodate approximately 2200 g.p.m. flow at projected growth during planning period.

With the construction of this new station, the original #1 pump station will be demolished, and a new 18"ϕ force main will be run from the intersection of 3rd and SW Main Court to the new "Downtown" pump station.

Item #2 - New Gravity Sewers. Two new gravity sewers are proposed. With the addition of a new "Downtown" pump station, the original #2 pump station at SW Alder (capacity of only 300 g.p.m.) can now be eliminated. With this pump station removed, one new 12"ϕ gravity sewer main will be added, from this location (original #2 pump station) to the new "Downtown" pump station. Second, a new 18"ϕ gravity sewer main will be added that will run from the location of the original #1 pump station to the new "Downtown" pump station.

These recommended improvements will result in cost savings in four (4) areas:

1. Electrical energy cost savings resulting from the elimination of one station.
2. Equipment maintenance cost savings from the elimination of one station.
3. Operating personnel cost savings from the elimination of one station.
4. Operating and maintenance cost savings realized by the new pump station as compared with two, older high maintenance pump stations.

Also, as a result of the addition of the new pump station and gravity sewer mains, system complexity is reduced, further enhancing the reliability of the City's wastewater collection system.

Item #3 - Demolishing Two Sub-Standard Pump Stations. With the addition of a new "Downtown" pump station, the discharge force main should be sized at 18"ϕ (to reduce friction head losses) and run from the new station to connect with the existing 12"ϕ force main. The recommended alignment would be to use the current alignment of the existing 10"ϕ force main, under NW Warrenton Drive, and then east to connect with the existing12"ϕ force main. This new force main would be 275 feet long and would replace the existing 10"ϕ force main.

Item #4 - Re-Routing of Force Main Alignment. The existing 6"ϕ force main from pump station #1 of the East Warrenton Interceptor should be re-routed after it crosses the Skipanon River to the west. Currently the force main discharges into a manhole (near the bridge), then effluent flows by gravity approximately 1300 feet south to the Original #1 station at SW 3rd and SW Main Court.

This force main should be re-routed and extended from the manhole to go north, under the bridge, then west along NE Harbor St., until it ties into the existing 12"ϕ force main that discharges at the lagoon. The approximate length of this alignment modification is 1050 feet. Possible associated costs may be a bore under the bridge and pump station upgrades necessary to meet the higher head requirements. Additional investigation will be required during the design to verify exact locations and grades of the modified conveyance system components and pump volumes.

This modification will reduce the amount of sewage flowing through the new Downtown Pump Station. Re-routing would increase overall system efficiency and it would have an added benefit in quicker transfer of effluent to the wastewater treatment.

The estimated cost for the proposed core downtown infrastructure improvements is shown below.

CORE DOWNTOWN INFRASTRUCTURE IMPROVEMENTS	
ITEM 1 – New Downtown Pump Station	$425,000.00
ITEM 2 – Remove Pump Station #1, Add Gravity	$249,000.00
ITEM 3 – Remove Pump Station #2, Add Gravity	$ 63,000.00
ITEM 4 – New Downtown Pump Station Force Main (Temp)	$ 41,000.00
ITEM 5 – Re-Route 6" Diameter Force Main	$119,800.00
Subtotal	$898,000.00
Engineering, Surveying & Permits (25%)	$225,000.00
TOTAL ESTIMATED PROJECT COST	**$1,123,000.00**

For a detailed cost breakdown of the five (5) items above, see the following two (2) tables.

City of Warrenton Downtown Pump Station
Construction cost estimate for new cason pump station
Note: Estimate prepared by H. R. Esvelt Engineering

Process Component *Item Description*	*Estimated Quantity*	*Units*	*Unit Cost*	4/18/2001 *Amount with OH&P*
Downtown Pump Station				
14' dia cason wet well w/ hatch	1	ls	34,000	34,000
slurry seal, base, excavation	1	ls	7,400	7,400
pumps, 2 pumps, 65 HP, submersible	2	ea	21,000	42,000
pump installation	20	%	42,000	8,400
piping & valves	1	ls	11,000	11,000
valve vault	1	ls	9,000	9,000
building, CMU, generator & controls	360	sf	60	21,600
electrical, gen set, ATS, alarm dialer, control	1	ls	110,000	110,000
pump variable frequency drives	1	ls	46,000	46,000
dewatering	1	ls	2,000	2,000
Sitework, piping, drainage, access, restor.	1	ls	25,000	25,000
Contractor Overhead	12	%	316,400	37,000
			Subtotal	**354,000**
Subtotal construction from above			**Subtotal**	354,000
Contingency (20% of construction)	20	%	354,000	71,000
Total construction including contingency				**425,000**
Engineering, surveying & permits (25%)	25	%	425,000	107,000
TOTAL ESTIMATED PROJECT COST				**532,000**

City of Warrenton - Infrastructure Core Conveyance Improvements
Construction cost estimate

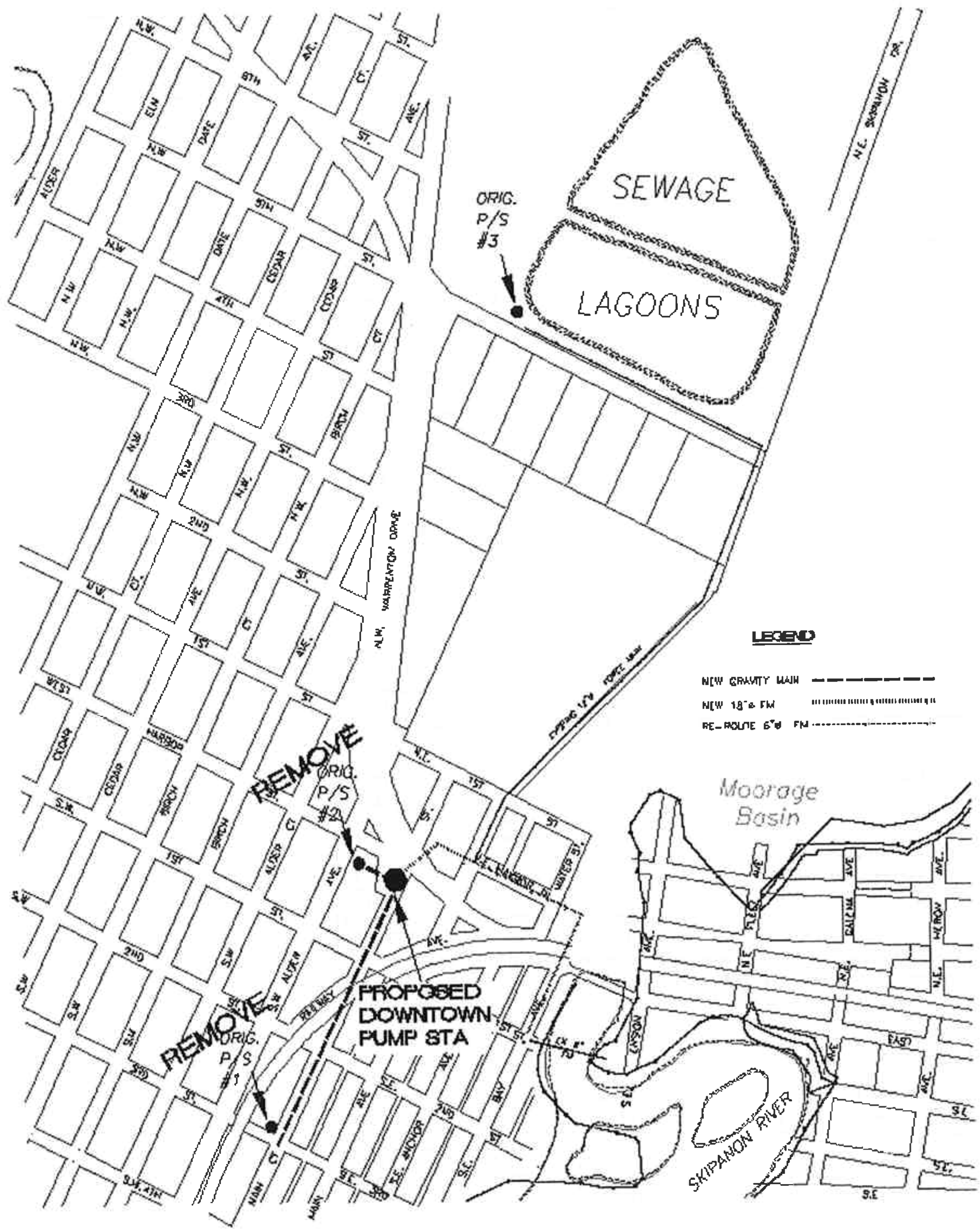
Date: 4/17/2001
By: JGF
Chk'd by: JAH

Improvement

Description	*Quantity*	*Units*	*Unit Cost*	*Total*	*Range*	
Item #1 - Remove Pump Station #1, add gravity to new pump station						
Assumes existing force main abandoned in place						
Assumes no bore necessary for old RR crossing					Values rounded up to nearest 1k	
Gravity sewer 18"	1350	FT	$85	$114,750		
Connections (existing mh, new ps)	2	EA	$700	$1,400		
Repave (2 places, 450x3, and 40x3)	19	TON	$70	$1,330		
3/4 rock (depth 10')	1500	CY	$25	$37,500		
Clean sand (depth 10')	1500	CY	$12	$18,000		
Reconnect 8 SS connections	8	EA	$175	$1,400		
Manhole	1	EA	$2,000	$2,000		
Demo Pump Sta. (est)	1	EA	$20,000	$20,000		
Mob/Traffic control	1	LS	$10,000	$10,000		
				$207,000	$207,000	
	const. contingency @ 20%			$41,400		
	subtotal for item			$249,000		$249,000
Item #2 - Remove Pump Station #2, add gravity to new pump station						
Gravity sewer 12"	250	FT	$75	$18,750		
Connections (existing mh, new ps)	2	EA	$700	$1,400		
Repave/Sidewalk	1	LS	$1,500	$1,500		
3/4 rock (depth 10')	278	CY	$25	$6,950		
Clean sand (depth 10')	278	CY	$12	$3,336		
Reconnect 8 SS connections	0	EA	$175	$0		
Manhole	0	EA	$2,000	$0		
Demo Pump Sta. (estimated)	1	EA	$20,000	$20,000		
Mob/Traffic control	0	LS	$10,000	$0		
				$52,000	$52,000	
	const. contingency @ 20%			10,400		
	subtotal for item			$63,000		$63,000
Item #3 - Remove and Replace 10" dia existing FM w/ 18" dia, new PS to NE Harbor and Main.						
Assumes 16" casing (under NW Warrenton Dr) is removed by open cut						
Force Main, new 18" dia.	275	FT	$75	$20,625		
Connections	2	EA	$700	$1,400		
Reducer, fittings	1	LS	$2,500	$2,500		
Repave/Sidewalk, both sides	1	LS	$4,500	$4,500		
3/4 rock (depth 4', width, 3')	122	CY	$25	$3,050		
Clean sand (depth 4', width 3')	122	CY	$12	$1,464		
				$34,000	$34,000	
	const. contingency @ 20%			6,800		
	subtotal for item			$41,000		$41,000
Item #4 - Re-route 6" force main from E. Warrenton Interceptor						
Assumes no bore under bridge, bore costs additional, see below						
Force Main, new 6" dia.	1070	FT	$30	$32,100		
Connections	2	EA	$700	$1,400		
Air Release Valve	1	EA	$700	$700		
Reducer/fittings/thrust blocks	1	LS	$1,500	$1,500		
Repave (500+/- feet)	13	TONS	$70	$910		
3/4 rock (depth 3.5', width, 2')	277	CY	$25	$6,925		
Clean sand (depth 3.5', width 2')	277	CY	$8	$2,216		
				$46,000	$46,000	
	const. contingency @ 20%			20,700		
	subtotal for item			$67,000		$67,000
Bore, at $250/ft	75	FT	$250	$18,750		
EWI #1 upgrade	1	EA	$25,000	$25,000		
				$44,000	$44,000	
	const. contingency @ 20%			$8,800		
	subtotal for item			$52,800		$52,800
	CONST. COST RANGE				**$383,000** to	**$472,800**
	ENG/SURV/PERMITS @ 25%				**$95,750**	**$118,200**
					$479,000	**$591,000**

FIGURE 7.6 - Proposed Core Infrastructure Improvements

SCALE: 1" = 600' = +/-



7.8 RECOMMENDED IMPROVEMENT ALTERNATIVE

The recommended improvement option for the City is a new SBR (Alternative #3) facility described earlier in *subsection 7.6.3*. Since the preparation of the Draft Plan, a Mixing Zone Study has identified the need for an extended outfall to the Columbia River. The cost of the piped outfall has been added to the recommended alternative cost. The estimated costs of the improvements are as follows:

Sequencing Batch Reactor (SBR)	$5,736,000.00
Core Conveyance System Improvements	$1,123,000.00
Outfall to Columbia River	$1,130,000.00
Total Cost	**$7,989,000.00**

The proposed alternative is the most economical system that can be built by the City and meet all State and Federal requirements, comply with water quality standards and allow for future expansion.

The estimated cost for the SBR system includes all construction at the treatment plant. The core conveyance system improvements include upgrades that will improve reliability, reduce operating and maintenance costs and provide a steady waste stream to the new treatment plant. The proposed outfall cost consists of an 18" diameter outfall pipe from the treatment plant to the Columbia River. The outfall pipe was found to be more economical than the costly upgrades that would be necessary at the treatment plant (described in more detail below) to discharge directly to the existing outfall ditch. For a detailed description of the recommended treatment plant option, see *Appendix C*. For additional information regarding the Mixing Zone Study and the determination for the need of an outfall pipe to the Columbia River, see *Appendix B*.

<u>Outfall Pipe vs. Treatment Plant Upgrades</u>

The additional cost to the recommended SBR plant to discharge directly into the existing outfall is estimated at $2,172,000.00. The estimated construction cost of the extended outfall pipe is estimated at $1,130,000.00. Based on these costs, the outfall pipe is the most economical outfall alternative. *Section 2.4 of Appendix C* includes a more detailed outfall alternative evaluation.

<u>Additional Treatment Plant Cost Without Mass Load Limit Increase</u>

The additional treatment cost to add effluent equalization and filtration to meet the current effluent waste load allocation is estimated at $2,071,000.00. The estimated additional annual O&M costs are estimated at $118,700.00. A detailed estimate is included in *Attachment 3 of Appendix C.*

This concludes the Wastewater Treatment Evaluation. A key factor in the implementation of any considered alternative is the removal of bio-solids from the lagoons. This is considered in the following Section 8, Biosolids Management.

SECTION 8
BIOSOLIDS (SLUDGE) MANAGEMENT

8.1 INTRODUCTION

A Biosolids Management Plan (BMP), dated January 2002, and a Biosolids Site Authorization Submittal (BSAS), dated February 2002, has been prepared by Lee Engineering, Inc. for the City of Warrenton. Both have been submitted to DEQ for review and approval. Both reports are included in *Appendix J* of this report. This section summarizes the plan in general terms for easy reference. *Appendix J* should be referred to for additional details of the plan.

The Warrenton treatment facility has been accumulating solids since 1969. Biosolids have accumulated to unacceptable levels contributing to overloading problems. The volume of the biosolids in Cell #1 was estimated to be 26,200 cubic yards (1,050 metric tons dry weight based on an average concentration of 5% total solids). The average depth of the sludge in Cell #1 was determined to be 1.6 feet. Sludge in the center of the cell is estimated to have accumulated to a depth of 3 feet. The total water column depth in both cells has been significantly reduced because of the accumulation of biosolids.

8.2 MUTUAL AGREEMENT AND ORDER (MAO) REQUIREMENTS

The MAO between the City of Warrenton and DEQ requires that the City submit a Biosolids Management Plan for DEQ approval within three months of approval of the final Facilities Plan. The final Facilities Plan is required to be submitted by September 30, 2002. The Biosolids Management Plan has been completed, and submitted to DEQ as of this date.

8.3 BIOSOLIDS MANAGEMENT PLAN

The purpose of the BMP is to outline how the biosolids will be removed, transported, and land applied in accordance with OAR 340-050-0031 and Federal 503 regulations.

The City is considering two possible methods of sludge removal. The first alternative would be complete removal and land application this year. The second alternative would consist of constructing a levy that divides Cell #1 into two smaller cells. All of the primary cell sludge could be pumped to the new storage cell to the east and allowed to settle. Excess water would be siphoned or pumped back into the primary cell. This alternative would allow sludge removal to be addressed over a longer period of time.

The existing transfer pipe would need to be relocated into the new westerly cell. The levy constructed would be consistent with proposed treatment plant upgrades. A diagram of this alternative *Figure 3-2* can be found in Chapter 3 of the Biosolids Management Plan, *(Appendix J).*

The sludge is proposed to be dredged from Cell #1 and pumped to a screening tank located on the shore of the lagoon. All particulate greater than ½ inch will be screened out and hauled to a landfill for disposal. From the screening tank, the sludge would be pumped through a grinder pump into temporary storage tanks until loaded into tankers for transport. Liquid biosolids would be transported in tanker trailers or trucks to the land application site for either direct surface application or transfer to a field applicator.

The total estimated cost for biosolids removal, transportation, and land application is $480,000.00

8.4 BIOSOLIDS SITE AUTHORIZATION SUBMITTAL

A key component in biosolids management is site authorization to dispose of sludge gathered under the BMP. This process is termed the Biosolids Site Authorization Submittal.

The City has applied for authorization to land apply biosolids to four (4) sites located four to six miles from the sewage treatment lagoon. The sites are privately owned pasture land. A general vicinity map, *Drawing 1* can be found in *Section 1* of the BSAS, *(Appendix J).* The drawing also delineates the proposed haul route from the sewage treatment lagoons to the application sites. The proposed application sites are described in detail in the BSAS.

The BSAS also includes a management agreement between the City of Warrenton and the Owner of the application site property and details regarding management of the sites.

8.5 BIOSOLIDS REMOVAL SCHEDULE

The City considers biosolids removal a high priority and plans to dredge during the summer of 2003. Originally the City had planned to dredge during the summer of 2002, but this did not occur. This schedule is contingent on the City receiving DEQ approval of the Biosolids Management Plan and the Biosolids Site Authorization.

The proposed interim improvements will require that biosolids be removed to gain the projected interim capacity required by the City and Miles Crossing. The schedule for the interim capacity improvements will also require removal of biosolids during the summer of 2003.

This concludes the Biosolids Management Evaluation. How are these improvements financed? Section 9 contains financing options and implementation programs.

SECTION 9
FINANCING OPTIONS AND IMPLEMENTATION PROGRAM

9.1 INTRODUCTION

The funding of needed wastewater improvements for the City of Warrenton may utilize one or more of the following sources:

- Sale of Bonds by Acquiring Federal or State Grants and/or Loans
- Special Assessments
- Local Improvement Districts
- Serial Levies
- Capital Improvements (Sinking) Funds
- Systems Development Charges

The most successful financing plans utilize state or federal grants and/or loans that best address the characteristics of needed improvements. It is difficult to finance improvements with grant funding alone. Some level of local funding or borrowing from available loan programs is usually necessary. Funding programs vary in terms of their economic impact on the community. Some funding programs are available to create and retain jobs or benefit areas of low to moderate income families, while other programs provide for specific types of infrastructure improvements, such as improvements to the wastewater treatment system.

A thorough consideration of applicable state and federal funding programs, in addition to a potential means of securing local funding, is needed to minimize the long-term cost of wastewater system improvements, while providing quality construction.

9.2 PUBLIC WORKS FINANCING PROGRAMS

Following is a general summary of public works grant and loan/bond programs, which have the potential to accommodate the City of Warrenton.

Each of the available grant and loan programs varies in terms of the extent and complexity of the application process. In all cases, it is extremely important to communicate the program needs to the funding agency at the earliest possible date. A close working relationship with the potential grantor or lending agency is critical for the success of the grant and/or loan process. A brief overview of potential public works financing programs follows.

9.2.1 Economic Development Administration

The emphasis of the Economic Development Administration *(EDA)* grant program is on projects, which create permanent jobs, especially in economically depressed areas. There is a higher chance of receiving the grant if the community can demonstrate that the existing system is at capacity; for example, if there is a moratorium on new connections. Grants require a local match, usually between the 40% to 50% range of the project cost, although local match can be as low as 20%.

9.2.2 Rural Development (RD)

The Water and Wastewater Disposal Grants and Loans program is under the administration of U.S. Department of Agriculture, Rural Development (RD), under the old guidelines of Farmers Home Administration (FMHA). The program is limited to rural communities, which have a population of less than 10,000 people; community population must not be likely to decline in the foreseeable future.

RD Grant Program

RD now utilizes "MEDIAN HOUSEHOLD INCOME" (MHI) rather than Median Family Income in their computations for determining eligibility for their program. This allows for single-person households to count as family-type households.

RD is currently basing its grant and loan determination on 1990 census data. Availability of grants from the RD is dependent on the (MHI); projects are competitive with one another on the basis of community need.

RD requires eligible communities to finance the project with loans up to the extent of the community's ability to pay; the grant is then available to cover the remainder. The actual formula to determine the maximum burden per household is quite complicated, and costs for commercial users are typically higher. RD determines the debt burden required in each case.

RD Loan Program

The City falls within the established criteria for loans. Items which determine a borrower's eligibility are listed below:

- Unable to obtain needed funds from other sources at reasonable rates and terms.
- Have legal capacity to borrow and repay loans, to pledge security for loans, and to operate and maintain the facilities or services.
- Be financially sound and able to manage the facility effectively.
- Have a financially sound facility based on taxes, assessments, revenues, fees, or other satisfactory sources of income to pay all facility costs, including costs that pertain to operation and maintenance. Furthermore, it must be shown that debts will be retired and financial reserves maintained.

Since the proposed improvements would involve a substantial commitment of RD loan funds, it may be possible, and is expected, that a combination of loan and grant sources be considered. This could allow for a commitment of some loan funds from RD and additional funds from another program.

9.2.3 Community Development Block Grant Program

The State of Oregon Economic Development Department administers the Community Development Block Grant (OCDBG) program. This program is funded by the U .S. Department of Housing and Urban Development. Funds allocated under the heading of this grant program are provided for projects designed specifically to improve the conditions of low and moderate income housing areas. Depending on the type of facility being funded, the maximum grant amount is either $150,000, $300,000 or $600,000.

All Community Facility projects must meet one of the three (3) National Objectives. The three (3) national objective are: principle benefit to low- and moderate-income persons, elimination of slums and blights (area or spot basis), and urgent need.

9.2.4 Special Public Works Fund (SPWF)

The State of Oregon Economic Development Department (OECDD) administers the Oregon Special Public Works Fund (SPWF) program. The purpose of the Special Public Works Fund is to create jobs, especially family-wage jobs, for Oregonians; loans and grants to construct public infrastructure to support industrial/manufacturing and eligible commercial economic development. “Eligible commercial” means commercial activity that is marketed nationally or internationally and attracts business from outside Oregon. The fund was created by the Oregon State Legislature in 1985. It is capitalized with lottery funds appropriated each biennium and with the sale of state revenue bonds.

The Special Public Works Fund is primarily a loan program. Grant funds are available based upon economic need of the municipality. The maximum loan term is 25-years; however, loans are generally made for 20-year terms.

The grant/loan amounts are determined by a financial analysis based on a demonstrated need and the applicant’s ability or inability to afford additional loans (debt capacity, repayment sources and other factors). Borrowers that are “credit worthy” may be funded through sale of state revenue bonds. Loans are generally repaid with Utility Revenues, Local Improvement Districts, and General Funds or Voter Approved bond Issues.

Projects must build public infrastructure to assist a business expanding, thus creating jobs, or build needed infrastructure capacity for future economic growth in the community. OECDD has separated the program into three categories:

1. Firm business commitment for permanent job creation
2. Capacity building, high probability of job creation or retention
3. Capacity building for severely affected communities

The three OECDD categories of the SPWF (Bond Funds) Program are discussed below:

Firm Business Commitment (Bond Funds)
Bond loans up to $1,000,000 (not to exceed $1,500,000) are available. Grants of up to $500,000 are available for projects, which have a firm commitment from a business(es) to create permanent jobs if the project is constructed. The grant is dependent on the number of jobs, which would potentially be created with maximum assistance of up to $10,000 per job. Of jobs created, 30% must be "family wage" jobs.

Capacity Building High Probability of Job Creation/Retention
This category of the SPWF program finances bond loans up to $10,000,000, and collateral loans up to $1,000,000 (not to exceed $1,000,000).

Capacity Building for Severely Affected Communities
SPWF has loans to $10,000,000 and grants up to $250,000 for severely affected communities. Communities are able to apply for grants of up to $250,000 from this fund even if they do not have a waiting business that needs the infrastructure. This will give communities who are seeking to attract business growth the chance to prepare in advance for these opportunities.

Warrenton would need to demonstrate that this project is necessary to create and/or retain jobs in the industrial sector. SPWF staff emphasize that the program is primarily a loan program and that applicants should not be overly optimistic about securing maximum grant dollars.

9.2.5 Water/Wastewater Financing Program

The purpose of the Water/Wastewater Financing Program is to provide financing for the construction of public infrastructure needed to ensure compliance with the Safe Drinking Water Act or the Clean Water Act. It is intended to assist local governments, which have been hard hit with state and federal mandates for public drinking water systems and wastewater systems.

The program was created by the Oregon State Legislature in 1993. It is capitalized with lottery funds appropriated each biennium and with the sale of state revenue bonds.

Public infrastructure required to ensure compliance with the Safe Drinking Water act or the Clean Water Act by creating or improving the following:

- Water source, treatment, storage and distribution
- Wastewater collection and capacity
- Storm system
- Purchase of rights-of-way and easements necessary for infrastructure
- Design and construction engineering

Ineligible activities include:

- General administrative costs
- Privately owned facilities and infrastructure
- Purchase of property not related to infrastructure

The Water/Wastewater Financing Program's guidelines, project administration, loan terms, and interest rates are similar to the Special Public Works Fund program. The maximum loan term is 25 years; however, loans are generally made for 20-year terms. The maximum direct loan amount is $500,000 when financed with lottery funds. The maximum bonded loan, when funded through the sale of State Revenue Bonds, is $10,000,000. Loans are generally repaid with Utility Revenues, General Funds or Voter Approved Bond Issues. Borrowers that are "credit worthy" may be funded through sale of state revenue bonds. The maximum grant is $500,000, including the cost of issuance and debt service reserve, in the case of a bonded loan. The grant/loan amounts are determined by a financial analysis based on a demonstrated need and the applicant's ability or inability to afford additional loans (debt capacity, repayment sources and other factors).

Technical Assistance grants and loans may finance preliminary planning, engineering studies, and economic investigations to determine project feasibility. The basis for eligibility is similar to construction projects, those needed to assist local governments in meeting the Safe Drinking Water Act and the Clean Water Act. Up to $10,000 in grant funds and $20,000 in additional loan, funds may be awarded to eligible applicants under 5,000 in population.

9.2.6 State Revolving Fund

The State Revolving Fund (SRF) loan program provides low-interest rate loans to public agencies for the planning, design and construction of water pollution control facilities, as well as for some publicly owned estuary management and non-point source control projects. This funding program is administered by DEQ. Priority is given to projects addressing documented water-quality problems and health hazards. Interest rates are typically below market rates.

9.3 LOCAL FUNDING SOURCES

A portion of the project may need to be financed with local funding sources. If the City does receive a low interest loan from state or federal agencies, the annual payment may be reduced. However, the method of repayment selected will be conditional upon agency approval. Typical local funding sources are listed below:

- General Obligation Bonds
- Revenue Bonds
- Improvement Bonds (Local Improvement District)
- Serial Levies
- Sinking Funds
- Ad Valorem Tax
- System User Fees Assessments
- System Development Charges (SDC's)

9.3.1 General Obligation Bonds (GO)

Financing of wastewater improvements by General Obligation (G. O.) Bonds is accomplished by the following procedures:

1. The Consulting Engineer prepares a detailed cost estimate to determine the total moneys required for construction.

2. An election is held.

3. When voter approval is granted (by a majority of the registered voters), bonds are offered for sale. The money for detailed planning and construction is obtained prior to preparation of final engineering plans and the start of project construction unless interim financing has been developed.

G .O .bonds are backed by the full credit of the issuer and authorize the issuer to levy ad Valorem taxes. The issuer can make the required payments on the bonds solely from the new tax levy or may instead use revenue from assessment, user charges, or some other source.

Oregon Revised Statutes limit the maximum term of G. O. bonds to 40 years for cities and 25 years for sanitary districts. Except in the event that RD purchases the bonds, the realistic term for which general obligation bonds would be issued is 15 to 20 years.

9.3.2 Revenue Bonds

A revenue bond is one that is payable solely from charges made for the services provided. Such bonds cannot be paid from tax levies or special assessments, and their only security is the borrower's promise to operate the wastewater system in a way that will provide sufficient net revenue to meet the obligations of the bond issue. Revenue bonds are most commonly retired with revenue from user fees.

Successful issuance of revenue bonds depends on bond market evaluation of the dependability of the revenue pledged. Normally there are no legal limitations on the amount of revenue bonds to be issued, but excessive bond issue amounts are generally unattractive to bond buyers because they represent high investment risk. In rating revenue bonds, buyers consider the economic justification for the project, reputation of the borrower, methods for billing and collection, rate structures, and the degree to which forecasts of net revenues are realistic. RD will fund revenue bonds in which user rates are committed for the repayment of the bonds.

9.3.3 Improvement Bonds (Local Improvement District)

Improvement bonds may be issued to assess certain portions of wastewater improvements directly against the parties being benefited. An equitable means of distributing the assessed cost must be utilized so that all property, whether developed or undeveloped, receives the assessment on an equal basis. Cities are limited to improvement bonds not exceeding 3% of true cash value. For a particular improvement, all property within the assessment area is assessed on an equal basis, regardless of whether it is developed or undeveloped.

Improvement bond financing requires that an improvement district be formed, the boundaries be established, and the benefited properties and property owners are determined. The engineer usually determines an approximate assessment based on a square-foot, a front-foot basis, or a combined basis. Property owners are then given an opportunity to remonstrate against the project. The assessment against the properties is usually not levied until the actual total cost of the project is determined. Since this determination is normally not possible until the project is completed, funds are not available from assessments for the purpose of making monthly payments to the contractor. Therefore, some method of internal financing must be arranged, or a pre-assessment program, based on the estimated total costs, must be adopted. It is common practice to issue warrants, which are paid when the project is completed, to cover debts.

The primary disadvantages to this source of revenue (improvement bonds) are described below:

1. The property to be assessed must have a true cash valuation at least equal to 50% of the total assessments to be levied. This may require a substantial cash payment by owners of undeveloped property.

2. An assessment district is very cumbersome and expensive when facilities for an entire community are contemplated.

3. The project is impacted by Measure 5 tax limitations because the improvement bonds are backed or guaranteed by the City's authority to raise revenue via taxation. If the City is in compaction, then a general election (same procedures as for a general obligation bond) is required. If the City's property taxes are not under compaction, then the City can proceed with a L.I.D. as in the past; however, the project cost will count against the $10.00 limitation for non-school taxes.

This program should not be considered for improvements to satisfy City needs in general, but could be a definite consideration for future expansions to annexations or property developments.

9.3.4 Serial Levies

Under Oregon Revised Statutes, if approved by the voters, the City can levy taxes for a fixed period of time to construct new facilities and maintain existing facilities. Generally, when a serial levy is presented to the voters, it is based upon a specific program and listing of planned improvements.

Since the time frame required for construction of the needed wastewater improvements is quite limited, it is doubtful that residents could afford a serial levy of sufficient size to provide for needed construction revenues.

9.3.5 Sinking Funds

Sinking funds can be established by budget for a particular capital improvement need. Budgeted amounts, from each annual budget, are carried in a sinking fund until sufficient revenue is available for the needed project. Funds can also be developed with revenue derived from system development charges or serial levies. Again, the City's wastewater system financial needs cannot be met with a sinking fund because of the limited time in which improvements must be completed.

9.3.6 Ad Valorem Tax

Many communities utilize an ad Valorem tax as the basis for repaying general obligation bonds for system expansions, and supplement them with additional wastewater use charges. This means of financing reaches all property to be ultimately benefited by the wastewater system, whether the property is presently developed or not. Construction costs are more equally distributed among all property owners and the program does not impose a penalty on existing residential or business development.

9.3.7 System User Fees

Monthly charges are made to all residences, businesses, etc., that are connected to the wastewater system. Wastewater use charges are established by resolution, and can be modified as needed to serve increased or decreased operating costs. Rates are established depending on the various classes of users and the metered demand through their connection. By establishment of proper use charges, the City could repay the local share of bond amortization without imposition of property taxes. This appears to be most favorable; however, a proposal to substantially increase monthly use charges might meet resistance from citizens with low or fixed incomes who would otherwise gain some financial advantage from repayment via taxation.

9.3.8 Assessments

In some cases, the beneficiary of a public works improvement can simply be assessed for the cost of the project. It is not uncommon for an industrial or commercial developer to provide up-front capital to pay for a community administered improvement, which serves the development.

9.3.9 System Development Charges

System Development Charges (SDC's) are charges assessed against new development to recover the costs incurred by local government who provide the capital facilities required to serve the new development. SDC's apply to new developments that generate revenue for the expansion or construction of facilities located outside the boundaries of new development When capital improvements increase usage, SDC's can be billed for water, wastewater, drainage and flood control, transportation, and parks or recreational facilities.

9.4 PROPOSED FINANCIAL PROGRAM

The City of Warrenton has already held a One-Stop Meeting and began the funding process for the proposed improvements. The meeting was held on December 11, 2000 and included representatives from the City of Warrenton, USDA Rural Development, DEQ, OECDD and the Governors Community Solutions Office. Discussions at the meeting were based on the following City background information:

- Population: 4,096 persons
- Low and Moderate Income: 38.44%
- Median Household Income: $24,667
- 1.75% of Median HH Income: $35.97
- Unemployment Rate: 4.7%

The following funds were identified as potential sources for the project:

USDA RD
USDA RD has funds available for this project. Due to the high cost of this project, it is anticipated USDA would "participate with other funding agencies. The interest rate for a community depends on the median income levels. (Based on 1990 census information, Warrenton's interest rate would be 4.75% over 40 years. The loan and grant determination is based on the user rates of the system (approx. $37.00 being a threshold).

State Revolving Fund (SRF)
Oregon Department of Environmental Quality may have funds for this project from the Clean Water State Revolving Fund. The City needs to get on DEQ's Intended Use Plan, which will be submitted to EPA early next year. Once a pre-application is submitted, projects are rated and ranked. Pre-applications are due late January/early February for the fiscal year 2003 funding. Funding determination is based on availability of funds and ranking of the project. This is a loan only program but the interest rate is 3.4% over 20 years. There are other fees (1.5% principal loan fee, 0.5% of annual principal) associated with this program but the effective interest rate (APR) is about 4.16%.

OECDD Funds
Oregon Economic and Community Development Department has funds available for this project. The City qualifies for the Water/Wastewater Financing Program. The grant/loan determination is based on the affordability of the user rates. The threshold applied is 1.75% of median household income for the community. Upon completion or shortly thereafter, the user rates for the wastewater system must be at least 1.75% of median household income. The interest rate for the loan is currently at 5.5% over 25 years. The interest rate changes quarterly.

Below is a table showing the different funding options evaluated during the meeting. Also included is the increase in rates required to service the debt. The monthly rate is the total of the monthly debt service and current City rates. This assumes the current operation and maintenance costs are covered in the current rates. Note that the EDU's is at 2050.

Warrenton Rate Scenarios

Option	Fund	Loan	Interest Rate	Term	Grant	Annual Payment	Monthly Debt Service	New Monthly Rate	EDUs	Current Rate
1	SPWF	$7,200,000	5.50%	25	$300,000	$514,390.54	$20.91	$35.91	2050	$15.00
2	RUS	$4,000,000	4.75%	40	$0	$225,186.98	$10.07	$34.77	2050	$15.00
	SPWF	$3,200,000	5.50%	25	$0	$238,557.93	$9.70			
3	DEQ	$7,900,000	4.16%	20	$0	$589,561.31	$23.97	$38.97	2050	$15.00

Notes: (1) Option 2 is the most desirable in that it requires no grant and results in a lower monthly sewer rate for customers. (2) Warrenton needs to clarify O&M with expansion and number of EDU's; (3) This scenario includes the 350 EDU's from Miles Crossing;

(4) Option 3 includes the 10% Debt Service reserve. (5) RUS includes 10% rate surcharge.

The City of Warrenton is also considering the submission of a general obligation bond to the Warrenton voters to pay for construction of the treatment plant in 2003.

9.5 RATE METHODOLOGY STUDY

The City of Warrenton recently received recommendations for a new rate methodology for both their water and wastewater systems. To prepare the rate methodology study, the City and their consultant have used approximate cost estimates for system improvements developed to date. The City approved the new 2001/2002 rates on March 20, 2002. Based on the rate methodology prepared by FCS Group, Inc. the proposed future sanitary sewer rate increases will be as follows:

Fiscal Year	Annual Rate Increase	Proposed Rate
2001/2002	100%	$26.00
2002/2003	0%	$26.00
2003/2004	42.31%	$37.00
2004/2005	36.49%	$50.50
2005/2006	0%	$50.50
2006/2007	0%	$50.50
2007/2008	0%	$50.50
2008/2009	0%	$50.50
2009/2010	0%	$50.50
2010/2011	0%	$50.50
2011/2012	0%	$50.50

The in-city single-family bill is currently $13.00 and would be considered very low when compared to other cities or the state average of approximately $38.00.

It is highly recommended that the City incorporate final sanitary sewer improvement cost estimates presented in this report for all improvements prior to finalization of their rate structure.

9.6 IMPLEMENTATION PROGRAM AND FINANCE PLAN

The City of Warrenton is undertaking an aggressive schedule for implementing the planned wastewater improvements. Subject to funding limitations and requirements, the proposed wastewater system is planned to be in full operation by January of 2006. The following table summarizes all milestones for both **MAO activities (shown in bold)** and recommended improvements. The milestones associated with the City's proposed interim improvements as listed separately at the end of the table.

Date	Activity	Estimated Cost	Method of Financing
Submitted March 8, 2002[1]	Submit Biosolids Management Plan	N/A	N/A
September 30, 2002[2]	**Submit Final Wastewater Facilities Plan to DEQ**	**N/A**	**N/A**
April 1, 2003[2]	**Submit Pre-design Report for WWTP to DEQ**	**N/A**	**N/A**
April 1, 2003 – July 2004	Permit Application Submittal for outfall (Biological Assessment, Federal, Local and Easement Documents from State Lands)	N/A	N/A
September 2003	Biosolids Removal	$480,000.00	CWSRF and/or RD
September 2003	Inflow/Infiltration Reduction Work at Airport[4]	Cost Not Available	N/A
December 31, 2003[2,3]	**Submit Plans and Specifications for WWTP to DEQ**	**N/A**	**N/A**
May 2004[2]	**Award contract for WWTP**	**$5,736,000.00**	**CWSRF and/or RD**
July 2004 – Sept. 2004	Outfall Construction	$1,130,000.00	CWSRF and/or RD
September 2005	Core Downtown Pump Station Improvements	$1,123,000.00	CWSRF and/or RD
September 2005[2]	**Complete construction of approved Plans and Specifications for WWTP**	**N/A**	**N/A**
January 2006[2]	**Achieve full operation of new facility**	**N/A**	**N/A**
By 2007	Main Avenue Sewer	$290,000.00[6]	Wastewater Capital Improvements Fund
By 2008	Dolphin Road Sewer	$310,000.00[6]	Wastewater Capital Improvements Fund
By 2015[5]	Inflow/Infiltration reduction work throughout City	$675,000.00[6]	Wastewater Capital Improvements Fund
By 2015[5,7]	Conveyance System Upgrades throughout City	$3,800,000.00	Wastewater Capital Improvements Fund

Interim Improvements (DEQ Suggested Schedule)			
September 2002	Concept Approval by DEQ	N/A	N/A
December 2002	Complete Plans and Specifications for Interim Improvements Project	N/A	N/A
December 2002 / January 2003	Following City Review/Approval, Submit Plans and Specifications to DEQ for Review	N/A	N/A
January / February 2003	DEQ Review, City Fix-up, Publish Final Plans and Specifications, and Bid Contract	N/A	N/A
February 2003	Award Contract	[8]	[8]
May 2003	Design Engineer Complete Draft O&M Manual for DEQ Review	N/A	N/A
June / July 2003	DEQ Review of Final O&M Manual	N/A	N/A
June / July 2003	Start Biosolids Removal From South Lagoon per Approved Management Plan	$480,000.00	CWSRF and/or RD
July / August 2003	DEQ Review of Facilities Start-up Report and Initial Operations	N/A	N/A
July 2003	Design Team Responsible for Inspection Services During Construction	N/A	N/A
August 2003	Resume Submittal of Sewer Extension Plans to DEQ	N/A	N/A
September 2003	Completion of biosolids removal in South Lagoon	N/A	N/A

NOTE: Due to occurrences outside of the control of the City of Warrenton (such as obtaining permits and receipt of funding) and the schedule constituting a "best estimation" of time line, the City maintains the right to revise the proposed schedule, either to accelerate or delay the implementation, based upon real time events and requirements.

[1] Plan submittal on March 8, 2002.

[2] From Mutual Agreement and Order (MAO) Schedule.

[3] Within 243 days of DEQ approval of Pre-design Report, but no later than December 31, 2003.

[4] This work should be implemented as soon as possible and will become a high priority item if Miles Crossing flows are to run through the Airport series of pump stations.

[5] This date indicates a general goal to be met by the City, understanding that a detailed program will need to be implemented by the City, prioritizing the improvements required to meet this goal.

[6] Estimate based on City of Warrenton Water & Sewer Rate Study dated April 2002.

[7] Pump station upgrades at the airport will need to take place prior to Miles Crossing sewer District flows running thought this portion of the City system. The Marlin Avenue force main will also need to be replaced.

[8] Total cost is estimated at $555,000.00 and does not include biosolids removal or required infrastructure improvements. Cost only includes work at plant.

This concludes the facilities plan general information. Specific information is contained in Section 10, which contains Appendices A-N.

SECTION 10
APPENDICES

APPENDIX A:	REQUEST FOR INTERIM CAPACITY INCREASE TECHNICAL MEMORANDUM
APPENDIX B:	MIXING ZONE STUDY
APPENDIX C:	WASTEWATER TREATMENT FACILITY UPGRADE & EXPANSION PLAN
APPENDIX D:	SUPPORTING INFORMATION FOR HISTORY OF OUTFALL DITCH & TIDE GATE
APPENDIX E:	WATER & SEWER RATE STUDY (February 6, 2002 & April 2002)
APPENDIX F:	MISCELLANEOUS COORESPONDENCE
APPENDIX G:	NPDES PERMITS
APPENDIX H:	MUTUAL AGREEMENT AND ORDER (MAO)
APPENDIX I:	OREGON ADMINISTRATIVE RULES (OREGON WATER QUALITY STANDARDS FOR THE NORTH COAST LOWER COLUMBIA BASIN)
APPENDIX J:	BIOSOLIDS MANAGEMENT PLAN AND BIOSOLIDS SITE AUTHORIZATION
APPENDIX K:	CITY OF WARRENTON USE ORDINANCE AND STATE PARKS AGREEMENT
APPENDIX L:	PUMP STATION REPORTS
APPENDIX M:	WASTEWATER TREATMENT PLANT MONITORING REPORT (DMR'S FOR 2000 & 2001)
APPENDIX N:	REPORT MAPS